Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Summer 2007

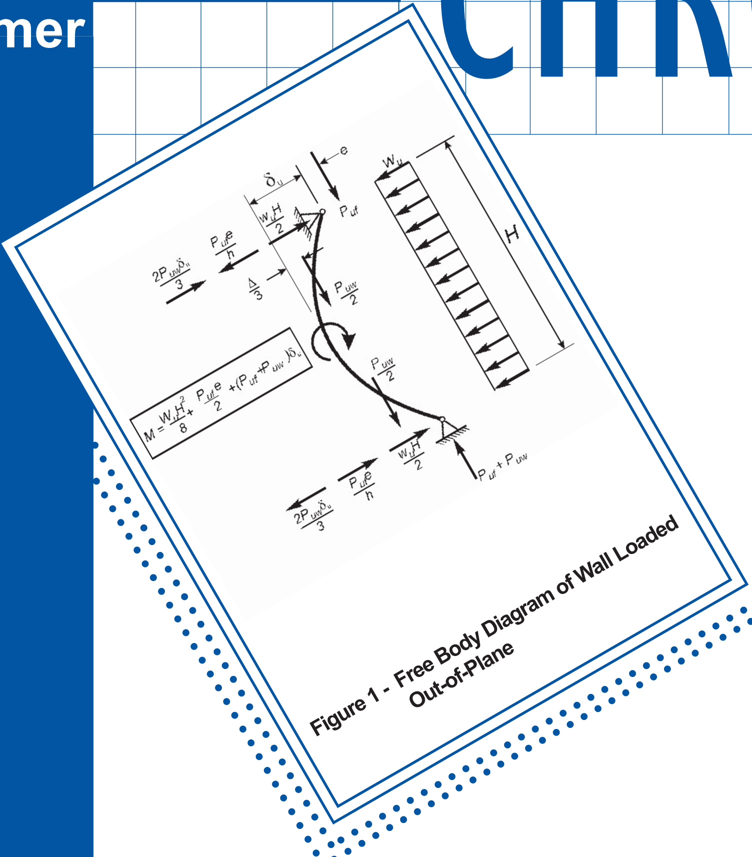

Figure 1 - Free Body Diagram of Wall Loaded Out-of-Plane

The Effects of Axial Load on the Strength Design of Slender Out-of-Plane Concrete Masonry Walls

Introduction

The Spring 2007 edition of *Masonry Chronicles* discussed the effects of axial load on out-of-plane slender walls designed using allowable stress design procedures. This edition will elaborate further on the subject by discussing the out-of-plane design of concrete masonry walls using strength design procedures.

Examples of out-of-plane wall designs with varying levels of axial load will be provided to illustrate the differences between the calculation methods found in the 1997 Uniform Building Code (UBC) [1] and the 2006 International Building Code (IBC) [2]. For masonry design, the IBC references the ACI 530-05/ASCE 5-05/TMS 402-05 [3], which is also referred to as the 2005 Masonry Standards Joint Committee Building Code (MSJC).

A major difference between typical allowable stress design and strength design procedures is the fact that secondary deformation (*P-Δ*) effects are included with strength design and ignored when allowable stress design is used. The examples will also illustrate the differences in designs obtained using the two methodologies by comparing results from strength design to those obtained from allowable stress design.

Out-of-Plane Analysis of Masonry Walls

Additional information regarding out-of-plane design loads can be found in the Winter 2003-04 issue of *Masonry Chronicles*, while in-plane considerations (allowable stress and strength design) were addressed in the Winter 2006-07 issue of *Masonry Chronicles*. Out-of-plane wall design utilizing allowable stress design procedures were covered in the Spring 2007 article. Past issues of *Masonry Chronicles* can be found on the Concrete Masonry Association of California and Nevada (CMACN) website (www.CMACN.org).

Nomenclature

The nomenclature used is as follows:

a = effective depth of compression block
A_g = effective area of masonry
A_s = area of steel
A_{se} = effective area of steel
b = effective width
c = distance to neutral axis
d = distance to rebar from face of block
E_m = masonry modulus of elasticity
e = eccentricity of P
f'_m = masonry block design strength
f_r = modulus of rupture
H = effective wall height
h = depth of concrete block
I_g = gross moment of inertia
I_{cr} = cracked moment of inertia
L = live load
M_E = moment demand at wall mid-height
M_n = nominal moment
M_u = moment capacity at wall mid-height
M_{cr} = cracking moment
P = axial load
P_{uf} = factored load from trib roof/floor areas
P_{uw} = factored weight of tributary wall area
P_u = combined loads, $P_{uf} + P_{uw}$

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Q_E = effect of horizontal seismic forces
S_n = section modulus of wall
w_u = factored out-of-plane uniform dist load
α = tensile reinforcement strain coefficient
β = stress block coefficient
δ_u = wall deflection due to factored loads
ε_{mu} = maximum usable strain in masonry
ε_y = yield strain in steel rebar
ϕ = capacity reduction factor
ρ = steel reinforcement ratio
ρ_b = steel reinforcement ratio producing balanced strain conditions
ρ_{max} = maximum steel reinforcement ratio

IBC Strength Design Procedures

Strength design procedures stipulated by the 1997 UBC and the 2006 IBC are similar. The major differences can be found in the capacity reduction factor, ϕ, the size of the compression block for the cross-section loaded in flexure, and the value for the modulus of rupture, f_r. The IBC, by reference to MSJC Equation 3-24, provides the following equation for the moment at the mid-height of a wall:

$$M_u = \frac{w_u H^2}{8} + P_{uf}\frac{e_u}{2} + (P_{uw} + P_{uf})\delta_u \qquad (1)$$

Equation 1 considers the effect of wall deflection on moment demand (*P-Δ* effects) and can be derived from Figure 1, which assumes pinned supports at the top and bottom of the wall. This secondary moment can be significant in large buildings with high walls and large axial loads.

When the moment demand, M_u, is less than the cracking moment, M_{cr}, the wall deflection, δ_u, is calculated from (MSJC Equation 3-30):

$$\delta_u = \frac{5M_u H^2}{48E_m I_g} \qquad (2)$$

and if $M_u > M_{cr}$ the wall deflection is calculated from (MSJC Equation 3-31):

$$\delta_u = \frac{5M_{cr} H^2}{48E_m I_g} + \frac{5(M_u - M_{cr})H^2}{48E_m I_{cr}} \qquad (3)$$

The cracked moment of the wall is given by:

$$M_{cr} = S_n f_r \qquad (4)$$

where f_r can be found in Table 3.1.8.2.1 of the MSJC. Since the moment depends on the deflection and the deflection depends on the moment, it is apparent that a solution can be reached by trial and error as shown in the *Seismic Design of Masonry using the 1997 UBC,* which is available through CMACN. Fortunately, this approach is not particularly cumbersome and converges within a few attempts. However, a closed-form solution can be reached by combining the deflection (where $M_u > M_{cr}$) and moment equations (Equations 1 and 3) to arrive at the following equation [4]:

$$\delta_u = \frac{\left(\frac{w_u H^2}{8} + \frac{P_{uf} e_u}{2}\right) - M_{cr}\left(1 - \frac{I_{cr}}{I_g}\right)}{\frac{48E_m I_{cr}}{5H^2} - P_u} \qquad (5)$$

Similarly, if $M_u < M_{cr}$ the equation for the deflection in this case can be obtained (using Equations 2 and 3):

$$\delta_u = \frac{\frac{w_u H^2}{8} + \frac{P_{uf} e_u}{2}}{\frac{48E_m I_g}{5H^2} - P_u} \qquad (6)$$

Example 1:
Design of a Slender Concrete Masonry Shear Wall (Out-of-Plane) Under Low Axial Loads Using 2006 IBC Strength Design

Determine the vertical steel required to resist out-of-plane forces for the wall shown in Figure 2. The fully grouted wall (78 psf) is constructed with 8-inch medium-weight concrete masonry units. A 3-foot tall parapet sits on top of the wall above the roof level. The specified masonry compressive strength is 1500 psi and Grade 60 steel is used as reinforcement. Out-of-plane seismic loading is 35 psf. An axial load of 80 lbs/ft is offset 7.3 inches from the wall centerline.

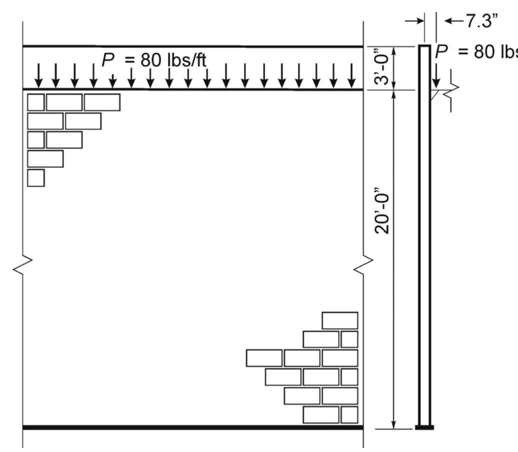


Figure 2 – Masonry Wall Under Low Axial Loads: Front and Side Views

Solution:

The axial load at mid-height where the maximum bending moment occurs is:

$$P = 80 + 78(10) + 78(3) = 1{,}094 \text{ lb/ft}$$

For brevity, one basic load combination, (0.9D+1.0E+1.6H), found in Section 1605.2.1 of the IBC will be used. For a complete design all the applicable basic load combinations contained in Section 1605.2.1 of the IBC should be considered. From MSJC Section 3.3.5.4:

$$\frac{P_u}{A_g} = \frac{0.9(1094)}{12(7.63)} = 10.76 \text{ psi} \le 0.05f'_m \therefore \text{O.K.}$$

We will solve for the deflection using equation (5) under the assumption that $M_u > M_{cr}$. First, we must obtain the cracked moment of inertia, I_{cr}. The equation, developed by the Structural Engineering Association of California (SEAOC), may be used:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3} \tag{7}$$

If we assume that #4 rebars spaced at 24 inches on center (A_s = 0.1 in^2/ft) are used, we arrive at the following:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.9(1.094)+0.10(60)}{60}$$
$$= 0.12 \text{ in}^2$$
$$a = \frac{P_u + A_s f_y}{0.8 f'_m b} = \frac{0.9(1.094)+0.10(60)}{0.8(1.5)(12)}$$
$$= 0.49 \text{ in} \qquad c = \frac{a}{0.8} = 0.61 \text{ in}$$
$$E_m = 900 f'_m = 1{,}350{,}000 \text{ psi}$$
$$n = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

Then the cracked moment of inertia can be calculated:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$
$$= 21.5(0.12)(3.81-0.61)^2 + \frac{12(0.61)^3}{3}$$
$$= 27.3 \text{ in}^4$$

Also, the gross moment of inertia is equal to:

$$I_g = \frac{bh^3}{12} = \frac{12(7.63)^3}{12} = 444 \text{ in}^4$$

Meanwhile, the cracking moment, M_{cr}, is calculated using the modulus of rupture taken from Table 3.1.8.2.1 of the MSJC (fully-grouted wall with type S mortar: tensile stresses perpendicular to bed joints):

$$M_{cr} = S_n f_r = \frac{12(7.63)^2}{12(6)} 163 = 1{,}582 \text{ lb-ft}$$

And using Equation (5), the deflection at wall mid-height is:

$$\delta_u = \frac{\left(\frac{35(20)^2 12}{8} + \frac{0.9(80)(7.3)}{2}\right) - 1{,}582(12)\left(1 - \frac{27.3}{444}\right)}{\frac{48(1{,}350{,}000)(27.3)}{5(240)^2} - 0.9(1094)}$$
$$= 0.67 \text{ in}$$

The moment demand including *P-Δ* effects can now be obtained:

$$M_u = \frac{w_u H^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u$$
$$= \frac{35(20)^2 12}{8} + 0.9(80)\frac{7.3}{2} + 0.9(1094)(0.67)$$
$$= 1{,}826 \text{ lb-ft/ft} \geq M_{cr}$$

The moment capacity can be found by MSJC Equation 3-27:

$$M_n = (A_s f_y + P_u)\left(d - \frac{a}{2}\right)$$
$$= (0.1(60) + 0.9(1.094))\left(3.81 - \frac{0.49}{2}\right) = 2{,}075 \text{ lb-ft/ft}$$

After the application of the appropriate capacity reduction factor, ϕ = 0.90, the out–of-plane flexural capacity of the wall remains larger than the demand.

$$\phi M_n = 0.9(2{,}075) = 1{,}868 \text{ lb-ft/ft}$$
$$1{,}868 \geq 1{,}826 \text{ lb-ft/ft} \therefore \text{O.K.}$$

The use of #4 rebars at 24 inches on center is sufficient to resist the calculated flexural out-of-plane demands.

To ensure that members possess sufficient ductility to perform as expected during earthquakes, Section 3.3.3.5.1 of the MSJC requirements limits the amount of reinforcement that can be placed in a cross-section. For walls loaded out-of-plane, the area of flexural tensile reinforcement must not exceed the amount required to maintain equilibrium with strain in the extreme tensile reinforcement equal to 1.5 times the yield strain at the maximum compressive strain in the concrete masonry of 0.0025. Equilibrium calculations should be performed with the load combination D+0.75L+0.525 Q_E. For singly reinforced members the above requirement can be summarized by Equation (8):

$$\rho_{max} = \frac{A_{s\,max}}{bd} = \frac{0.64 f'_m \left(\frac{\varepsilon_{mu}}{\varepsilon_{mu} + \alpha\varepsilon_y}\right) - \frac{P}{bd}}{f_y} \tag{8}$$

Where α is the required minimum strain in the tensile reinforcement. For walls loaded out-of-plane, α is equal to 1.5. In this example:

$$\rho_{max} = \frac{A_{s\,max}}{bd} = \frac{0.64(1500)\left(\frac{0.0025}{0.0025+1.5(0.00207)}\right) - \frac{1094}{12(3.83)}}{60{,}000}$$
$$A_{s\,max} = 0.31 \text{ in}^2/\text{ft} \geq A_s = 0.1 \text{ in}^2/\text{ft} \therefore \text{O.K.}$$

Example 2:
Design of a Slender Concrete Masonry Shear Wall (Out-of-Plane) Under Low Axial Loads Using 1997 UBC Strength Design

Determine the vertical steel required to resist out-of-plane loading for the wall in Example 1 using the 1997 UBC strength design provisions.

Solution:

1997 UBC provisions for the strength design of masonry can be found in Section 2108. Generally, the assumptions, equations, and provisions, are similar to the strength design provisions in the 2005 MSJC. Hence, the solution can be obtained by making minor modifications to the solution outlined in Example 1. From section 2108.2.4.4 of the 1997 UBC:

$$\frac{P}{A_g} = \frac{1094}{12(7.63)} = 11.95 \text{ psi} \leq 0.04 f'_m \therefore \text{O.K.}$$

Similar to Example 1, we use the equation for the cracked moment of inertia developed by SEAOC. Preliminary calculations show that #4 rebars at 24 inches on center used in the previous example is not sufficient. Assuming the use of #4 rebars at 16 inches on center (A_s = 0.15 in^2/ft), the calculations are performed in a manner similar to that in Example 1:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.9(1.094) + 0.15(60)}{60}$$
$$= 0.166 \text{ in}^2$$
$$a = \frac{P_u + A_s f_y}{0.85 f'_m b} = \frac{0.9(1.094) + 0.15(60)}{0.85(1.5)(12)}$$
$$= 0.65 \text{ in} \qquad c = \frac{a}{0.85} = 0.77 \text{ in}$$
$$E_m = 750 f'_m = 1{,}125{,}000 \text{ psi}$$
$$n = \frac{29{,}000{,}000}{1{,}125{,}000} = 25.8$$

Then the cracked moment of inertia can be calculated:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$
$$= 25.8(0.166)(3.81 - 0.77)^2 + \frac{12(0.77)^3}{3}$$
$$= 41.4 \text{ in}^4$$

The gross moment of inertia is equal to:

$$I_g = \frac{bh^3}{12} = \frac{12(7.63)^3}{12} = 444 \text{ in}^4$$

The cracking moment, M_{cr}, is calculated using the modulus of rupture according to Equation 8-31 of the 1997 UBC.

$$M_{cr} = S_n f_r = \frac{12(7.63)^2}{12(6)} 4\sqrt{1500} = 1{,}503 \text{ lb-ft/ft}$$

And using Equation (5), the deflection at wall mid-height is:

$$\delta_u = \frac{\left(\frac{35(20)^2 12}{8} + \frac{0.9(80)(7.3)}{2}\right) - 1{,}503(12)\left(1 - \frac{41.4}{444}\right)}{\frac{48(1{,}125{,}000)(41.4)}{5(240)^2} - 0.9(1094)}$$
$$= 0.72 \text{ in}$$

The moment demand including P-Δ effects can now be obtained:

$$M_u = \frac{w_u H^2}{8} + P_{uf} \frac{e_u}{2} + P_u \delta_u$$
$$= \frac{35(20)^2 12}{8} + 0.9(80)\frac{7.3}{2} + 0.9(1094)(0.72)$$
$$= 1{,}831 \text{ lb-ft/ft} \geq M_{cr}$$

The moment capacity can be found by (Section 2108.2.4.4 of the UBC):

$$M_n = A_{se} f_y \left(d - \frac{a}{2}\right)$$
$$= 0.166(60)\left(3.81 - \frac{0.65}{2}\right) = 2{,}893 \text{ lb-ft/ft}$$

After the application of the appropriate capacity reduction factor, ϕ = 0.80, the flexural capacity of the wall remains larger than the demand.

$$\phi M_n = 0.8(2{,}893) = 2{,}314 \text{ lb-ft/ft}$$
$$2{,}314 \geq 1{,}831 \text{ lb-ft/ft} \therefore \text{O.K.}$$

As a result, the use of #4 rebars at 16 inches on center is adequate to resist the given out-of-plane loads.

According to 2108.2.4.2 of the 1997 UBC, the reinforcement ratio is not to exceed 0.5ρ_b. The axial load according to the UBC Section 2108.2.3.3 should be 1.0 *D*+1.0*L*+1.0*E*. This requirement can be given as Equation (9).

$$\rho_b = \frac{0.72 f'_m \frac{\varepsilon_{mu}}{\varepsilon_y + \varepsilon_{mu}} - \frac{P}{bd}}{f_y}$$
$$= \frac{0.72(1500)\frac{0.003}{0.00207+0.003} - \frac{1094}{12(3.81)}}{60000} = 0.010 \qquad (9)$$
$$A_{s\,max} = 0.5(0.010)(12)(3.81) = 0.23 \text{ in}^2\text{/ft}$$
$$A_s = 0.15 \text{ in}^2\text{/ft} \leq 0.23 \text{ in}^2\text{/ft} \therefore \text{O.K.}$$

Example 3:
Design of a Slender Concrete Masonry Shear Wall (Out-of-Plane) Under High Axial Loads Using 2006 IBC Strength Design

Determine the vertical steel required to resist out-of-plane forces for the wall shown in Figure 3. This example is identical to Example 1 except the loading has increased from 80 lbs/ft to 3000 lb/ft.

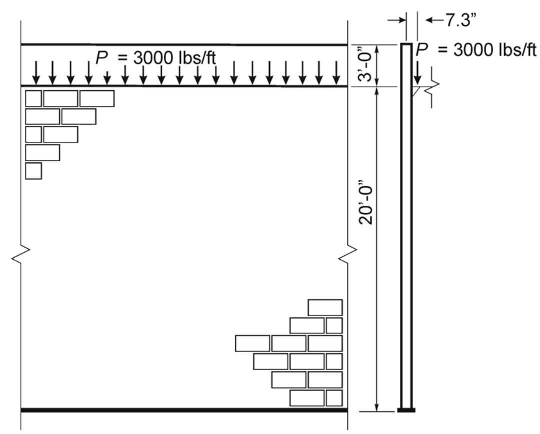


Figure 3 – Masonry Wall Under High Axial Loads: Front and Side Views

Solution:

The axial load at mid-height where the maximum bending moment occurs is:

$$P = 3{,}000 + 78(10) + 78(3) = 4{,}014 \text{ lb/ft}$$

From Section 3.3.5.4 of the MSJC:

$$\frac{P_u}{A_g} = \frac{.9(4014)}{12(7.63)} = 39.5 \text{ psi} \leq 0.05 f'_m \therefore \text{O.K.}$$

Similar to Example 1, we use the equation for the cracked moment of inertia developed by SEAOC. Assuming the use of #5 rebars spaced at 16 inches on center (A_s = 0.23 in^2/ft), the calculations are performed in a manner analogous to the process in example 1:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.9(4.014) + 0.23(60)}{60}$$
$$= 0.29 \text{ in}^2$$
$$a = \frac{P_u + A_s f_y}{0.8 f'_m b} = \frac{0.9(4.014) + 0.23(60)}{0.8(1.5)(12)}$$
$$= 1.21 \text{ in} \qquad c = \frac{a}{0.8} = 1.51 \text{ in}$$
$$E_m = 900 f'_m = 1{,}350{,}000 \text{ psi}$$
$$n = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

The cracked moment of inertia can be calculated as:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$
$$= 21.5(0.29)(3.81-1.51)^2 + \frac{12(1.51)^3}{3}$$
$$= 46.75 \text{ in}^4$$

While the gross moment of inertia is:

$$I_g = \frac{bh^3}{12} = \frac{12(7.63)^3}{12} = 444 \text{ in}^4$$

From Example 1:

$$M_{cr} = S_n f_r = \frac{12(7.63)^2}{12(6)} 163 = 1{,}582 \text{ lb - ft}$$

And using Equation (5), the deflection at wall mid-height is:

$$\delta_u = \frac{\left(\frac{35(20)^2 12}{8} + \frac{0.9(3000)(7.3)}{2}\right) - 1{,}582(12)\left(1 - \frac{46.75}{444}\right)}{\frac{48(1{,}350{,}000)(46.75)}{5(240)^2} - 0.9(4014)}$$
$$= 2.01 \text{ in}$$

The moment demand including *P-Δ* effects can now be obtained:

$$M_u = \frac{w_u H^2}{8} + P_{uf}\frac{e_u}{2} + P_u \delta_u$$
$$= \frac{35(20)^2 12}{8} + 0.9(3000)\frac{7.3}{2} + 0.9(4014)(2.01)$$
$$= 3{,}176 \text{ lb} - \text{ft/ft} \geq M_{cr}$$

The moment capacity can be found by MSJC Equation 3-27:

$$M_n = (A_s f_y + P_u)\left(d - \frac{a}{2}\right)$$
$$= (0.23(60) + .9(4.014))\left(3.81 - \frac{1.21}{2}\right) = 4{,}651 \text{ lb} - \text{ft/ft}$$

After the application of the appropriate capacity reduction factor, ϕ = 0.90, the capacity of the wall remains larger than the demand.

$$\phi M_n = 0.9(4{,}651) = 4{,}186 \text{ lb} - \text{ft/ft}$$
$$4{,}186 \geq 3{,}176 \text{ lb} - \text{ft/ft} \therefore \text{O.K.}$$

As a result, the use of #5 rebars at 16 inches on center is adequate to resist the given out-of-plane loads.

The reinforcing ratio is now checked in the same manner as illustrated in Example 1 using Equation (8):

$$\rho_{max} = \frac{A_{s\,max}}{bd} = \frac{0.64(1500)\left(\frac{0.0025}{0.0025 + 1.5(0.00207)}\right) - \frac{4014}{12(3.81)}}{60{,}000}$$
$$A_{s\,max} = 0.26 \text{ in}^2\text{/ft} \geq A_s = 0.23 \text{ in}^2\text{/ft} \therefore \text{O.K.}$$

Example 4:
Design of a Slender Concrete Masonry Shear Wall (Out-of-Plane) Under High Axial Loads Using UBC Strength Design

Determine the steel required to resist out-of-plane loading for the wall in Example 3 using 1997 UBC strength design procedures.

Solution:

The solution can be obtained by making minor modifications to the solution outlined in Example 3. From section 2108.2.4.4 of the 1997 UBC:

$$\frac{P}{A_g} = \frac{4014}{12(7.63)} = 43.8 \text{ psi} \leq 0.04 f'_m \therefore \text{O.K.}$$

This time we will try #4 bars at 8 inches (A_s = 0.3 in^2) on center. Preliminary calculations show that #4 rebars at 16 inches on center as well as #5 rebars at 16 inches on center is not sufficient. Similarly, the other parts of Example 3 are modified as follows:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{0.9(4.014) + 0.30(60)}{60}$$
$$= 0.36 \text{ in}^2$$
$$a = \frac{P_u + A_s f_y}{0.85 f'_m b} = \frac{0.9(4.014) + 0.30(60)}{0.85(1.5)(12)}$$
$$= 1.41 \text{ in} \qquad c = \frac{a}{0.85} = 1.66 \text{ in}$$
$$E_m = 750 f'_m = 1{,}125{,}000 \text{ psi}$$
$$n = \frac{29{,}000{,}000}{1{,}125{,}000} = 25.8$$

Then the cracked moment of inertia can be calculated as:

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$
$$= 25.8(0.36)(3.81-1.66)^2 + \frac{12(1.66)^3}{3}$$
$$= 61.23 \text{ in}^4$$

The gross moment of inertia is equal to:

$$I_g = \frac{bh^3}{12} = \frac{12(7.63)^3}{12} = 444 \text{ in}^4$$

From Example 2:

$$M_{cr} = S_n f_r = \frac{12(7.63)^2}{12(6)} 4\sqrt{1500} = 1{,}503 \text{ lb - ft/ft}$$

And using Equation (5), the deflection at wall mid-height is:

$$\delta_u = \frac{\left(\frac{35(20)^2 12}{8} + \frac{0.9(3000)(7.3)}{2}\right) - 1{,}503(12)\left(1 - \frac{61.23}{444}\right)}{\frac{48(1{,}125{,}000)(61.23)}{5(240)^2} - 0.9(4014)}$$
$$= 1.95 \text{ in}$$

The moment demand, including P-Δ effects, can now be obtained:

$$M_u = \frac{w_u H^2}{8} + P_{uf}\frac{e_u}{2} + P_u\delta_u$$
$$= \frac{35(20)^2 12}{8} + 0.9(3000)\frac{7.3}{2} + 0.9(4014)(1.95)$$
$$= 3{,}158 \text{ lb} - \text{ft/ft} \geq M_{cr}$$

The moment capacity can be found by (Section 2108.2.4.4 of the UBC):

$$M_n = A_{se} f_y \left(d - \frac{a}{2} \right)$$
$$= 0.36(60)\left(3.81 - \frac{1.41}{2} \right) = 5{,}589 \text{ lb} - \text{ft/ft}$$

$$M_{cr} = S_n f_r = \frac{12(7.63)^2}{12(6)} 4\sqrt{1500} = 1{,}503 \text{ lb} - \text{ft/ft}$$
$$M_n > M_{cr} \therefore \text{O.K.}$$

After the application of the appropriate capacity reduction factor, ϕ = 0.80, the capacity of the wall remains larger than the demand.

$$\phi M_n = 0.8(5{,}589) = 4{,}471 \text{ lb} - \text{ft/ft}$$
$$4{,}471 \geq 3{,}158 \text{ lb} - \text{ft/ft} \therefore \text{O.K.}$$

As a result, the use of #4 bars at 8 inches on center is more than adequate to resist the calculated wall demands. The difference between this design and that of the 2006 IBC can be attributed to the smaller capacity reduction factor used.

The reinforcing ratio is now checked in the same manner as previously illustrated in Example 2:

$$\rho_b = \frac{0.72 f'_c \dfrac{\varepsilon_{mu}}{\varepsilon_y + \varepsilon_{mu}} - \dfrac{P}{bd}}{f_y}$$

$$= \frac{0.72(1500)\dfrac{0.003}{0.00207 + 0.003} - \dfrac{4014}{12(3.81)}}{60000} = 0.009$$

$$A_{s\,\max} = 0.5(0.009)(12)(3.81) = 0.21 \text{ in}^2\text{/ft}$$

$$A_s = 0.30 \text{ in}^2\text{/ft} \therefore \text{N.G.}$$

This wall contains more steel reinforcement than is allowed by the 1997 UBC. Therefore, this design is not an acceptable solution. One possible solution would be to increase the wall thickness.

Conclusions

From the four example problems it can be seen that under different loading conditions, varying amounts of reinforcing steel are required to meet the code requirements for out-of-plane loading. Note, that for brevity, not all steps required for the complete design of the wall were included here. Furthermore, these walls were not designed for in-plane loading conditions. Depending on the magnitude of the in-plane lateral load imposed, that design condition may govern. As a result, additional steel may be required to satisfy requirement published in the 1997 UBC and the 2006 IBC. Figure 4 shows the solution obtained by all four examples.

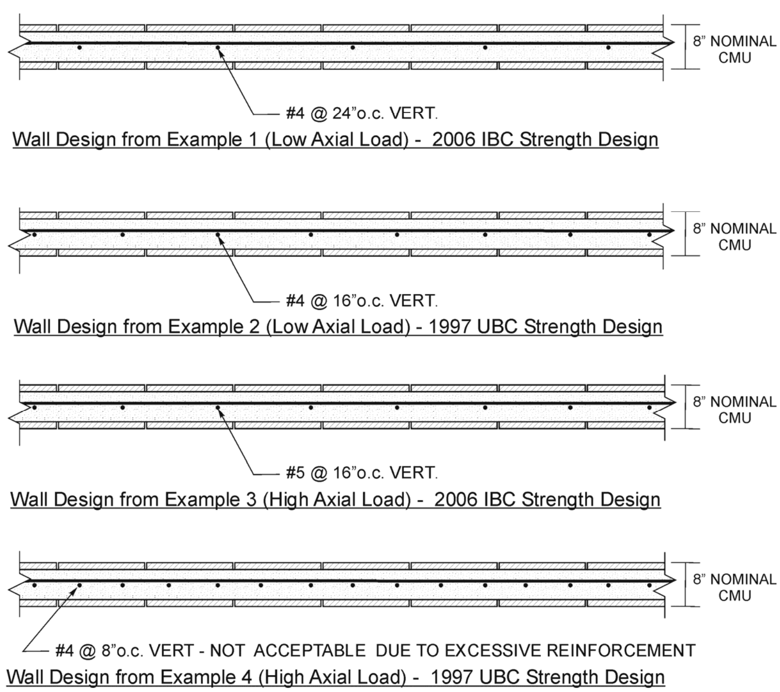
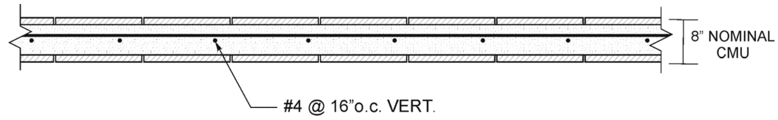
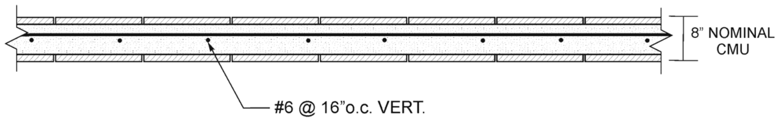


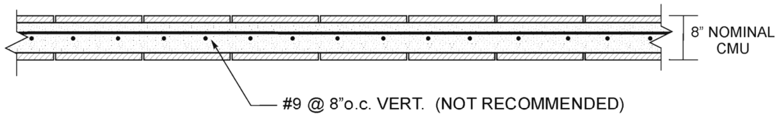
Figure 4 – Required Vertical Reinforcement for Examples 1-4 (Strength Design)



Wall Design from Example 1&2 (Low Axial Load) - 1997 UBC Allowable Stress Design (Unity Equation) and 2006 IBC Allowable Stress Design



Wall Design from Example 3 (High Axial Load)- 2006 IBC Allowable Stress Design



Wall Design from Example 4 (High Axial Load) - 1997 UBC Allowable Stress Design (Unity Equation)

Figure 5 – Required Vertical Reinforcement for Allowable Stress Design (from Spring Masonry Chronicles)

From Figure 4 it is apparent that for both low and high axial loads, the 2006 IBC requires the use of less reinforcing steel. This can be primarily attributed to the use of more conservative ϕ factors in the 1997 UBC.

The Spring 2007 article of *Masonry Chronicles* provided solutions to the examples discussed here using allowable stress design. Figure 5 depicts the solutions from allowable stress designs in accordance with the 1997 UBC and 2006 IBC.

For low axial loads, allowable stress design results in the same design as the 1997 UBC strength design approach (which is slightly more conservative than the 2006 IBC strength design results. The unity equation permitted by the 1997 UBC does not take into account the beneficial effect of axial load on flexural capacity, but considers the axial and flexural loads independently. Consequently, under high axial loads, the unity equation provides the most conservative results - as shown in example 4 of Figure 5 (see the Spring 2007 issue for more details).

The results for strength design shown in Figure 4 are more accurate from a technical standpoint since secondary moment (P-Δ) effects were accounted for. The allowable stress designs in Figure 5 do not account for the effect of displacement on the wall demand, since it assumes that the wall remains elastic and deflections are small.

For more information and detail regarding the comprehensive design of slender walls please see the 2006 edition of *Design of Reinforced Masonry Structures,* available fall 2007. This publication is published by and made available through the Concrete Masonry Association of California and Nevada (CMACN).

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This edition of "Masonry Chronicles" was written by Henry Huang and Chukwuma Ekwueme, PhD., of Weidlinger Associates, Inc., Marina Del Rey, California.

Concrete Masonry Association of California and Nevada
6060 Sunrise Vista Drive, Suite 1990
Citrus Heights, CA 95610
(916) 722-1700
Info@cmacn.org
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