Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Revised Spring 2007

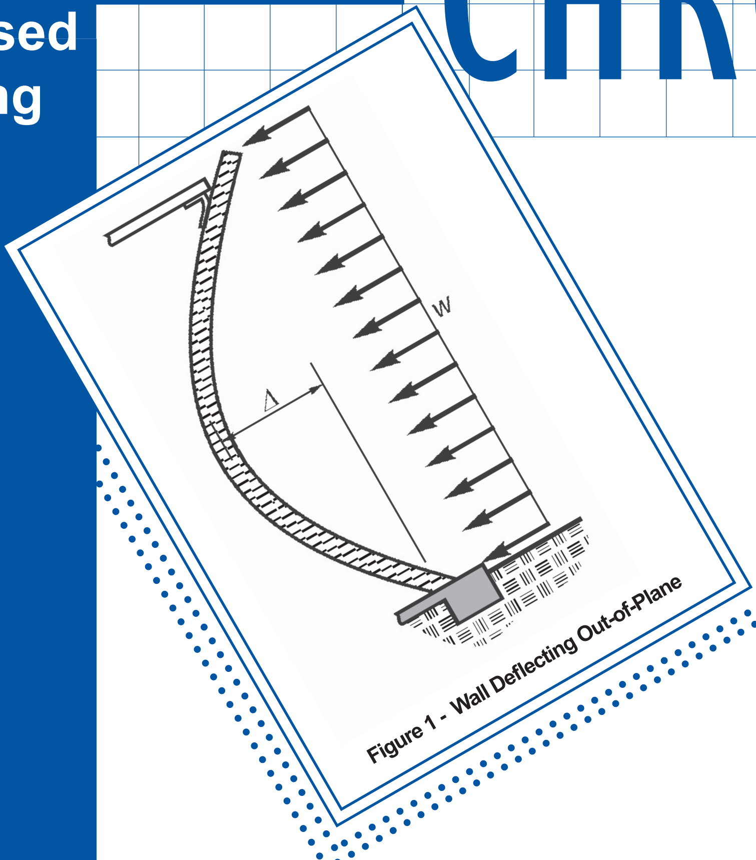

Figure 1 - Wall Deflecting Out-of-Plane

Comparison of the Effects of Axial Load on Out-of-Plane Slender Walls: Allowable Stress Design

Preface

Please note that this is a revised and updated version of the Spring 2007 article. Due to some typographical errors, the solutions to some of the examples were found to be in error. These errors and omissions include:

1. In Example 3, a roof load of 80 lb/ft was used to design the wall instead of the correct load of 3,000 lb/ft.

2. The moment on the walls due to the eccentricity of the roof loads was multiplied by an incorrect load factor (1/1.4 instead of .9).

3. In Examples 1 and 3, which were based on the IBC, the code-required check that the axial load component is less than the allowable axial load was omitted.

To rectify the situation, the Spring 2007 article is being reissued. We apologize for any inconvenience this may have caused.

Note that the *Masonry Chronicles* only highlights some key issues in the design of walls to resist out-of-plane loads, and does not represent a comprehensive description of all the steps required to design concrete masonry walls. For brevity, only one load condition is considered, and wall detailing requirements and in-plane design procedures are not described here (please see other issues of *Masonry Chronicles* for more information at www.cmacn.org).

Introduction

The design of masonry walls to resist out-of-plane loads is an important aspect in the design of masonry buildings. In most large buildings that use masonry walls as the lateral load resisting system, out-of-plane response is the critical phase of the design. Typical layouts of common masonry warehouse-type buildings do not contain an abundance of openings. As a result, these types of buildings can usually resist lateral demands imposed by wind or earthquake loads. However, the large story heights inherent in these structures may result in considerable out-of-plane demands.

Figure 1 illustrates how a wall would respond to out-of-plane loading. When this type of loading occurs, the walls are no longer part of the lateral load resisting system in the direction of the lateral load being considered. Instead, they act as structural elements or components in the structure that support the loads directly imposed on them. The wall deflecting out-of-plane must span between supports and transfer lateral loads to the floor or roof diaphragms, which in turn transfer the loads to the walls that form the lateral load resisting system.

Evaluation of the walls is complicated by the fact that the walls are slender relative to their height. Therefore, deflections induced by lateral

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loads may, in certain cases, be comparable to the width of the wall. As a result, secondary deformation effects (P-Δ effects) will need to be considered in order to accurately determine the wall demands. In allowable stress design, P-Δ effects are not considered. A future issue of *Masonry Chronicles* will deal with the design of slender walls using strength design. With this design methodology, the effect of displacements on wall demands is considered.

Examples of out-of-plane wall designs will be provided to illustrate the differences between the calculation methods permitted 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/TMS402-05 [3], which is also referred to as the 2005 Masonry Standards Joint Committee Building Code (MSJC).

The same example problem will be worked in two separate ways to demonstrate the effects of axial load, the use of the new code and its implications. Only out-of-plane issues will be dealt with in this issue. In-plane considerations were addressed in the winter 2007 issue of *Masonry Chronicles.*

Determination of Design Loads

Out-of-plane loads on masonry walls in buildings are usually induced by inertial earthquake forces or wind pressures. In basement walls, out-of-plane loads are also caused by lateral soil pressures, but this will not be specifically addressed here. Also, the design of free-standing fence walls will not be addressed. It should be noted that while the loads on retaining walls and fence walls are calculated in a slightly different matter, their designs follow the principles described herein. For additional information regarding out-of-plane design loads, please refer to the winter 2003-2004 issue of *Masonry Chronicles*, which can be found on the Concrete Masonry Association of California and Nevada (CMACN) website (www.CMACN.org).

The 1997 UBC and the 2006 IBC differ in the way out-of-plane loads are calculated. For load calculations please reference section 1632 of the 1997 UBC, or the 2006 IBC equivalent in section 1613. Table 1 shows the different forces determined according to the 1997 UBC and 2006 IBC requirements at four different city hall locations in California and Nevada.

1997 UBC Out-Of-Plane Loads

Los Angeles	San Francisco	Sacramento	Las Vegas
35 psf	35 psf	26 psf	19 psf

2006 IBC Out-Of-Plane Loads

Los Angeles	San Francisco	Sacramento	Las Vegas
35 psf	37 psf	17 psf	17 psf

Table 1 – Out-of-Plane Loads for the 1997 UBC and 2006 IBC

Loads shown in Table 1 are similar for Los Angeles, San Francisco, and Las Vegas, but differ for Sacramento. A discussion regarding the difference in the way the respective codes were developed is beyond the scope of this article. However, it is obvious that the implications of these loads on design outcomes may be significant. For each example problem discussed here, an out-of-plane load of 35 psf will be used.

Out-Of-Plane Analysis of Masonry Walls

It is a common assumption that masonry walls are restrained by pin supports at the floor and roof levels. This is a reasonable design approach, since the wall to floor connection does not usually possess sufficient stiffness or strength to transfer wall moments into the floor. Therefore, a rigid connection cannot be justified. In addition, since earthquake and wind response are dynamic phenomena, the assumption of pinned supports is consistent with the modal response of the walls subjected to earthquake and wind loads.

Masonry walls are typically analyzed differently for out-of-plane loads, depending on whether working stress or strength design procedures are used. This article will attempt to investigate the differences and consequences of allowable stress design conducted using the 1997 UBC and the 2006 IBC under varying levels of axial loading.

The 1997 UBC working stress provisions sanction the use of the unity equation (UBC 2107.2.7) for the design of masonry walls subjected to axial and flexural loads. However, this technique has come under scrutiny, since it is not completely accurate and can lead to flawed designs as discussed in the winter 2007 issue of *Masonry Chronicles.*

In the 2006 IBC, the use of the unity equation for the design of reinforced concrete elements is no longer permitted. Instead, stresses for each material are calculated independently. Doing so allows the designer to take into account the beneficial effects of axial load – which is neglected by the unity equation.

Examples will be provided to illustrate how axial loads affect the final design of slender masonry walls. The same problem will be worked in separate ways to determine the effects of the code. 1997 UBC prescriptions for allowable stress design (unity equation), as well as the 2006 IBC provisions for allowable stress design, will be utilized.

Nomenclature

The nomenclature used in this paper is as follows:

b = Effective width
d = Distance to rebar
e = Eccentricity of roof load
E_s = Steel modulus of elasticity
E_m = Masonry modulus of elasticity
f_b = Maximum calculated stress in masonry
f_s = Maximum calculated stress in steel
F_b = Maximum allowable stress in masonry
F_s = Maximum allowable stress in steel
F_{a+b} = Maximum compressive stress from combined axial and flexural loads
h = Effective height of wall
jd = Distance between the centroid of flexural compressive force and the centroid of the tensile force
kd = Effective depth of compression area
M = Total moment at wall mid-height
M_E = Moment due to out-of-plane load
n = Ratio of the modulus of elasticity of steel and masonry
P_D = Axial load at mid-height of wall
P_{uf} = Factored axial load
r = Radius of gyration
T = Actual thickness of masonry
ρ = Reinforcement ratio

Example 1:

Design of a Slender Concrete Masonry Wall (Out-of-Plane) Under Low Axial Loads Using 2006 IBC Allowable Stress Design

Determine the vertical steel to resist out-of-plane forces for the wall with the dead load shown in Figure 2. The fully grouted wall (78 psf) is constructed with 8-inch medium-weight concrete masonry units. A 3-foot 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. Out-of-plane-loading is 35 psf. An axial load of 80 lb/ft is offset 7.3 inches from the wall centerline. Use the alternative load combinations in the 2006 IBC (A one-third increase in allowable stresses is permitted).

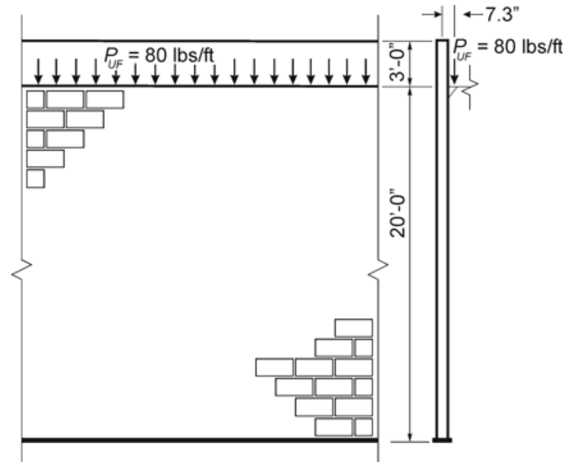

Figure 2 – Masonry Wall Under Low Axial Loads: Front and Side Views

Solution:

For brevity, only one load combination (.9D+E/1.4) will be used. For a complete design, all the combinations contained in section 1605.3.2 of the IBC should be evaluated. The maximum out-of-plane bending moment (per unit foot width of wall) at mid-height is equal to:

$$M_E = \frac{wh^2}{8} = \frac{35(20)^2}{8} = 1{,}750 \text{ lb} - \text{ft/ft}$$

$$M_D = \frac{Pe}{2} = (80)\left(\frac{7.3}{2(12)}\right) = 24.3 \text{ lb} - \text{ft/ft}$$

$$\sum M = .9M_D + M_E / 1.4 = .9(24.3) + (1750/1.4)$$
$$= 1272 \text{ lb-ft/ft}$$

and the axial load (per unit foot width of wall) at mid-height where the maximum bending moment occurs is:

$$P_D = 80 + 78(10) + 78(3) = 1094 \text{ lb/ft}$$

Using the one-third increase in allowable stress design as permitted by the IBC:

$$F_{a+b} = \frac{1}{3} f'_m \times 1.33 = 667 \text{ psi}$$

$$F_s = 24{,}000 \times 1.33 = 32{,}000 \text{psi}$$

The applied compressive stress at mid-height is:

$$f_a = \frac{0.9(1094)}{12(7.63)} = 10.75 \text{ psi}$$

The radius of gyration is calculated using the minimum wall thickness. Therefore:

$$r = \frac{t}{\sqrt{12}} = \frac{7.63}{\sqrt{12}} = 2.2 \text{ in}$$

Where the allowable stress due to axial load, F_a, is calculated in accordance with MSJC Section 2.2.3.1:

$$F_a = .25(1500)\left(\frac{70(2.2)}{20(12)}\right)^2 = 154.4 \text{ psi}$$

$$f_a \le F_a \therefore \text{ O.K.}$$

If we try #4 bars at 16 inches on center:

$$n = \frac{29{,}000{,}000}{1{,}500(900)} = 21.5$$

$$\rho = \frac{A_s}{bd} = \frac{0.2(12/16)}{12(3.81)} = 0.0033$$

$$n\rho = 0.071$$

$$\frac{P}{bd} = \frac{0.9(1094)}{12(3.81)} = 21.54\text{psi}$$

$$\frac{h}{d} = \frac{7.625}{3.81} = 2.0$$

$$\frac{n}{F_s} = \frac{21.5}{32{,}000} = 0.00067$$

First, we calculate the allowable moment based on the masonry compressive stress. The neutral axis for this case is given below:

$$k = \sqrt{\left(n\rho - \frac{P}{bdF_{a+b}}\right)^2 + 2n\rho} - \left(n\rho - \frac{P}{bdF_{a+b}}\right)$$
$$= \sqrt{(.071-.032)^2 + 2(.071)} - (.071-.032)$$
$$= 0.34$$

The allowable moment can be calculated from:

$$\frac{M}{bd^2} = F_{a+b}\left[k\left(\frac{h}{4d} - \frac{k}{6}\right) - n\rho\left(1 - \frac{h}{2d}\right) + \frac{n\rho}{k}\left(1 - \frac{h}{2d}\right)\right]$$
$$= 667\left[.34\left(\frac{1}{2} - \frac{.34}{6}\right) - 0 + 0\right] = 100.5 \text{ psi}$$

If the cross-section is governed by the steel tensile stress:

$$k = \sqrt{\left(n\rho + \frac{nP}{F_s bd}\right)^2 + 2\left(n\rho + \frac{nP}{F_s bd}\right)} - \left(n\rho + \frac{nP}{F_s bd}\right)$$
$$= \sqrt{(.071+.0145)^2 + 2(.071+.0145)} - (.071+.0145)$$
$$= 0.337$$

The allowable moment can be calculated from:

$$\frac{M}{bd^2} = \frac{P}{bd}\left(\frac{h}{2d} - \frac{k}{3}\right) + n\rho\frac{F_s}{n}\left(1 - \frac{k}{3}\right)$$
$$= \frac{0.9(1094)}{12(3.81)}\left(1 - \frac{.337}{3}\right) + .0705\left(\frac{32{,}000}{21.5}\right)\left(1 - \frac{.337}{3}\right)$$
$$= 112.26\text{psi}$$

Therefore, the allowable moment for the given axial load is governed by the allowable masonry stress and is equal to:

$$\frac{M}{bd^2} = 100.5\text{psi}$$

$$M = \frac{100.5(12)(3.81)^2}{(12)} = 1459 \text{ lb-ft/ft}$$

The moment demand at mid-height is equal to:

$$M = 1272 \text{ lb-ft/ft}$$

$$1272 \text{ lb-ft/ft} < 1459 \text{ lb-ft/ft} \therefore \text{O.K.}$$

Therefore, #4 bars at 16 inches on center is an acceptable solution.

Example 2:

Design of a Slender Concrete Masonry Shear Wall (Out-of-Plane) Under Low Axial Loads Using 1997 UBC Unity Equation

Determine the steel required to resist out-of-plane loading for the wall in Example 1.

Solution:

Note that the use of the unity equation is not recommended by the MSJC code for use with reinforced masonry. However, its use is illustrated here to compare the traditional allowable stress design approach with the solution provided in the previous example.

The out-of-plane loads are the same as those found in the first example. Initially, we must determine the allowable compressive stress due to axial loads alone.

$$\frac{h}{r} = \frac{240}{2.2} = 109.1 > 99$$

Therefore per UBC requirements (2107.2.5):

$$F_a = 0.25(1500)\left(\frac{70(2.2)}{240}\right)^2 = 154 \text{ psi}$$

The applied compressive stress at mid-height of the first story is equal to (UBC 2107.1.6.1):

$$f_a = \frac{0.9(1094)}{12(7.63)} = 10.75 \text{ psi}$$

Using the unity equation in the form contained in 1997 UBC with the one-third increase in allowable stresses (UBC 2107.2.7):

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.33$$

which means that the allowable flexural compressive stress considering the presence of axial loads is given by:

$$f_b = F_b\left(1.33 - \frac{f_a}{F_a}\right)$$
$$= 500\left(1.33 - \frac{10.75}{154}\right) = 630.1 \text{ psi}$$

If we try #4 bars at 16 inches on center:

$$\rho = \frac{A_s}{bd} = \frac{0.2(12/16)}{12(3.81)} = 0.00328$$

$$n = \frac{29{,}000{,}000}{750(1{,}500)} = 25.8$$

$$n\rho = 0.0846$$

From 2107.2.15 of the 1997 UBC:

$$k = \sqrt{(n\rho^2) + 2n\rho} - n\rho$$
$$= \sqrt{(.0846)^2 + 2(.0846)} - .0846 = 0.335$$

$$j = 1 - \frac{k}{3} = 1 - \frac{.335}{3} = 0.89$$

Now, the compressive stress in the masonry, as well as the tensile stress in the longitudinal reinforcement due to flexural loads alone, can be found:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = \frac{1272(12)}{(12)(3.81)^2}\left(\frac{2}{.89(.335)}\right)$$
$$= 588 \text{ psi} \le 630 \text{ psi} \therefore \text{O.K.}$$

$$f_s = \frac{M}{A_s jd} = \frac{1272(12)}{(.15)(.89)(3.81)}$$
$$= 30{,}010 \text{ psi} \le 32{,}000 \text{ psi} \therefore \text{O.K.}$$

Both compressive and tensile stresses are below allowable values. Thus, our initial assumption of one #4 bar at 16 inches on center has been validated.

Example 3:

Design of a Slender Concrete Masonry Wall (Out-of-Plane) Under High Axial Loads Using 2006 IBC Allowable Stress Design.

Determine the vertical steel to resist out-of-plane forces for the wall with the dead load as shown in Figure 3. This example is identical to the problem statement in Example 1, except that the loading has been increased from 80 lbs/ft to 3000 lb/ft.

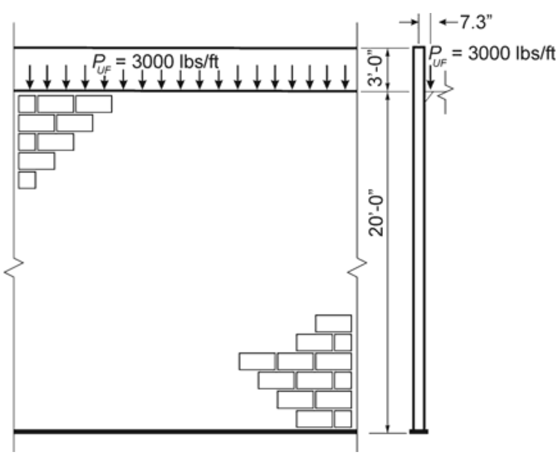

Figure 3 – Masonry Wall Under High Axial Loads: Front and Side Views

Solution:

The maximum out-of-plane bending moment (per unit foot width of wall) at mid-height is equal to (using the .9D + E/1.4 alternate load combination):

$$M_E = \frac{wh^2}{8} = \frac{35(20')^2}{8} = 1{,}750 \text{ lb-ft/ft}$$

$$M_D = \frac{Pe}{2} = (3000)\left(\frac{7.3}{2(12)}\right) = 912.5 \text{ lb-ft/ft}$$

$$\sum M = .9M_D + M_E/1.4 = .9(912.5) + (1750/1.4)$$
$$= 2071 \text{ lb-ft/ft}$$

and the axial load at mid-height (per unit foot width of wall) where the maximum bending moment occurs is:

$$P_D = 3000 + 78(10) + 78(3) = 4{,}014 \text{ lb/ft}$$

Using the one-third increase in allowable stress design as permitted by the IBC:

$$F_{a+b} = \frac{1}{3} f'_m \times 1.33 = 667 \text{psi}$$

$$F_s = 24{,}000 \times 1.33 = 32{,}000 \text{ psi}$$

The applied compressive stress at mid-height is:

$$f_a = \frac{0.9(4014)}{12(7.63)} = 39.5 \text{ psi}$$

Where the allowable stress, F_a, is calculated in accordance with MSJC Section 2.2.3.1:

$$F_a = .25(1500)\left(\frac{70(2.2)}{20(12)}\right)^2 = 154.4 \text{ psi}$$

$$f_a \le F_a \therefore \text{O.K.}$$

If we try #6 bars at 16 inches on center:

$$n = \frac{29{,}000{,}000}{1{,}500(900)} = 21.5$$

$$\rho = \frac{A_s}{bd} = \frac{0.44(12/16)}{12(3.81)} = 0.0072$$

$$n\rho = 0.155$$

$$\frac{P}{bd} = \frac{0.9(4014)}{12(3.81)} = 79.02 \text{ psi}$$

$$\frac{h}{d} = \frac{7.625}{3.81} = 2.0$$

$$\frac{n}{F_s} = \frac{21.5}{32{,}000} = .00067 \text{ psi}$$

First, we calculate the allowable moment based on the masonry compressive stress. The neutral axis for this case is given below:

$$k = \sqrt{\left(n\rho - \frac{P}{bdF_{a+b}}\right)^2 + 2n\rho} - \left(n\rho - \frac{P}{bdF_{a+b}}\right)$$
$$= \sqrt{(.155 - .118)^2 + 2(.155)} - (.155 - .118)$$
$$= 0.52$$

The allowable moment can be calculated from:

$$\frac{M}{bd^2} = F_{a+b}\left[k\left(\frac{h}{4d} - \frac{k}{6}\right) - n\rho\left(1 - \frac{h}{2d}\right) + \frac{n\rho}{k}\left(1 - \frac{h}{2d}\right)\right]$$
$$= 667\left[.52\left(\frac{1}{2} - \frac{.52}{6}\right) - 0 + 0\right]$$
$$= 143.4 \text{psi}$$

If the cross-section is governed by the steel tensile stress:

$$k = \sqrt{\left(n\rho + \frac{nP}{F_s bd}\right)^2 + 2\left(n\rho + \frac{nP}{F_s bd}\right)} - \left(n\rho + \frac{nP}{F_s bd}\right)$$
$$= \sqrt{(.155 + .053)^2 + 2(.155 + .053)} - (.155 + .053)$$
$$= 0.47$$

The allowable moment can be calculated from:

$$\frac{M}{bd^2} = \frac{P}{bd}\left(\frac{h}{2d} - \frac{k}{3}\right) + n\rho\frac{F_s}{n}\left(1 - \frac{k}{3}\right)$$
$$= \frac{0.9(4014)}{12(3.81)}\left(1 - \frac{.47}{3}\right) + .155\left(\frac{32{,}000}{21.5}\right)\left(1 - \frac{.47}{3}\right)$$
$$= 261 \text{ psi}$$

Therefore, the allowable moment for the given axial load is governed by the allowable masonry stress and is equal to:

$$\frac{M}{bd^2} = 143.4 \text{ psi}$$

$$M = \frac{143.4(12)(3.81)^2}{(12)}$$

$$= 2082 \text{ lb-ft/ft} \geq 2071 \text{ lb-ft/ft} \therefore \text{O.K.}$$

Therefore, #6 bars at 16 inches on center is an acceptable solution.

Example 4:

Design of a Slender Concrete Masonry Wall (Out-of-Plane) Under High Axial Loads Using 1997 UBC Unity Equation.

Determine the steel required to resist out-of-plane loading for the wall in Example 3.

Solution:

The out-of-plane loads are the same as those found in the previous example. Initially, we must determine the allowable compressive stress due to axial loads alone. Since the wall is supported laterally at the roof, the effective height for axial loads is given by:

$$\frac{h}{r} = \frac{240}{2.2} = 109.1 > 99$$

Therefore per UBC requirements (2107.2.5):

$$F_a = 0.25(1500)\left(\frac{70(2.2)}{240}\right)^2 = 154 \text{ psi}$$

The applied compressive stress at mid-height of the first story is equal to (UBC 2107.1.6.1):

$$f_a = \frac{0.9(4014)}{12(7.63)} = 39.5 \text{ psi}$$

Using the unity equation in the form contained in 1997 UBC with the one-third increase in allowable stresses (UBC 2107.2.7):

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.33$$

which means that the allowable flexural compressive stress considering the presence of axial loads is given by:

$$f_b = F_b\left(1.33 - \frac{f_a}{F_a}\right)$$

$$= 500\left(1.33 - \frac{39.5}{154}\right) = 537 \text{ psi}$$

Similar to Example 2, try #6 bars at 16 inches on center:

$$n = \frac{29{,}000{,}000}{1{,}500(900)} = 21.5$$

$$\rho = \frac{A_s}{bd} = \frac{0.44(12/16)}{12(3.81)} = 0.0072$$

$$n\rho = 0.155$$

From 2107.2.15 of the 1997 UBC:

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

$$= \sqrt{(.155)^2 + 2(.155)} - .155 = 0.42$$

$$j = 1 - \frac{k}{3} = 1 - \frac{.42}{3} = 0.86$$

Now, the compressive stress in the masonry, as well as the tensile stress in the longitudinal reinforcement, can be found:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = \frac{2071(12)}{(12)(3.81)^2}\left(\frac{2}{(.86)(.42)}\right)$$

$$= 789 \text{ psi} \geq 537 \text{ psi} \therefore \text{N.G.}$$

$$f_s = \frac{M}{A_s jd} = \frac{2071(12)}{(.33)(.86)(3.81)}$$

$$= 22{,}984 \text{ psi} \leq 32{,}000 \text{ psi} \therefore \text{O.K.}$$

Compressive stress in the masonry exceeds allowable values. Thus, our initial assumption of one #6 bar at 16 inches on center is inadequate. More steel is required to reduce the stresses in the masonry. Now we will try #9 bars at 8 inches on center:

$$\rho = \frac{A_s}{bd} = \frac{1.0(12/8)}{12(3.81)} = 0.033$$

$$n = \frac{29{,}000{,}000}{750(1{,}500)} = 25.8$$

$$n\rho = 0.85$$

From 2107.2.15 of the 1997 UBC:

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

$$= \sqrt{(.85)^2 + 2(.85)} - .85 = 0.706$$

$$j = 1 - \frac{k}{3} = 1 - \frac{.706}{3} = 0.765$$

Now, the compressive stress in the masonry, as well as the tensile stress in the longitudinal reinforcement, can be found:

$$f_b = \frac{M}{bd^2}\left(\frac{2}{jk}\right) = \frac{2071(12)}{(12)(3.81)^2}\left(\frac{2}{.765(.706)}\right)$$

$$= 528 \text{ psi} \leq 537 \text{ psi} \therefore \text{O.K.}$$

$$f_s = \frac{M}{A_s jd} = \frac{2071(12)}{(1.5)(.765)(3.81)}$$

$$= 5{,}684 \text{ psi} \leq 32{,}000 \text{ psi} \therefore \text{O.K.}$$

Both compressive and tensile stresses are below allowable values. Thus, our assumption of one #9 bar at 8 inches on center has been validated. Obviously, the use of #9 bars at such a close spacing is not practical in real world situations, and is only presented here for illustration and comparison purposes. In lieu of using #9 bars at 8 inches on center, a thicker wall should be used. This would reduce the stresses on the masonry, and allow for the use of less reinforcing steel. With the current design, the masonry stresses govern the design; the steel is not being fully utilized, and this results in an inefficient design.

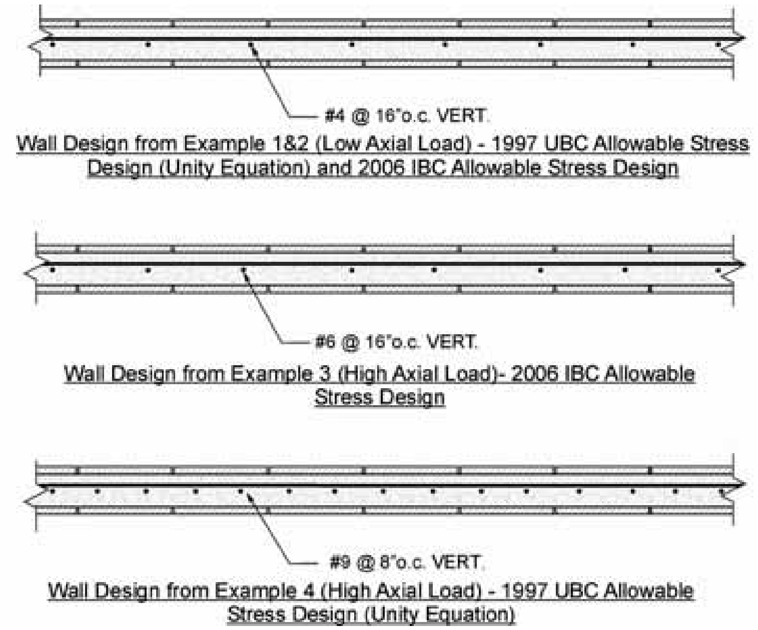

Figure 4 – Amount of Vertical Steel Required to Resist Out-of-Plane Loads in the 1997 UBC and 2006 IBC

Conclusions

From the four example problems it can be seen that under different loading conditions, varying amounts of vertical 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. 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.

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 more conservative results. However, under low axial loads, the unity equation provides the same solution as the 2006 IBC (see Figure 4).

The 2006 IBC provisions result in designs that require significantly less vertical reinforcement under high axial loads. It is able to accomplish this by more accurately representing the mechanics of reinforced concrete masonry under externally applied loads. When design is conducted using the 1997 UBC unity equation, the amount of steel required is directly proportional to the axial load. As illustrated in Figure 4, more vertical reinforcement was required when the axial load was increased. However, this is due to the fact that a large eccentric moment was generated by the offset at the top of the wall. If the load was concentric, less steel may have been required when using the IBC, since concentric axial load contributes to the flexural resistance provided by the wall. Another consideration is that secondary moment effects are neglected in allowable stress design. These effects can be significant, especially under large axial loads. The summer 2007 issue will explore this topic in more depth, and how they are dealt with through strength design methodologies.

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

References

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