Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Summer 2003

Examples To Illustrate the Differences Between the 1997 UBC and the 2002 MSJC Code

Part I: Out-of-Plane Loads on Masonry Walls

Introduction

The previous issue of "Masonry Chronicles" highlighted the differences between the masonry design provisions of the *1997 Uniform Building Code* (1997 UBC) and *Building Code Requirements for Masonry Structures: ACI 530-02/ASCE 5-02/TMS 402-02* by the Masonry Standards Joint Committee (MSJC Code). This issue will continue with that theme by providing examples for the design of masonry walls for out-of-plane loads. The examples will be done using both the method prescribed by the 1997 UBC and the method prescribed by the MSJC Code with major differences pointed out in the MSJC Example.

The examples used in this issue are modified from examples used in the *Seismic Design of Masonry Using the 1997 UBC*, by Ekwueme and Uzarski.

Working Stress Design

The Working Stress Design procedures for out-of-plane loading change very little between the 1997 UBC and the MSJC Code. Ultimately, these changes have little effect on design.

Strength Design

For this example a 29′-0″ high wall constructed of solid grouted 10″ (nominal) concrete masonry units (f'_m =1500 psi) will be designed for out-of-plane loading. The vertical reinforcing steel consists of two-layers of #5 @ 16″ o.c. The example wall is shown in Figures 1 and 2. The factored loads are given by:

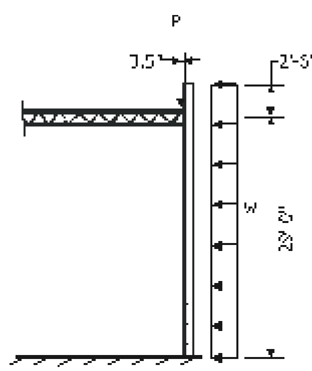

Figure 1: Example Wall

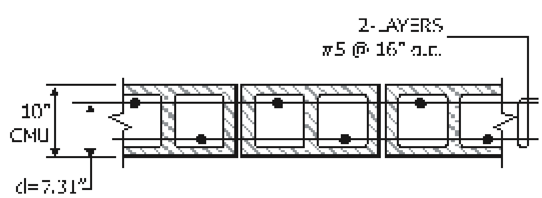

Figure 2: Cross Section of Example Wall

Strength Design for Out-of-Plane Loads Using the 1997 UBC

The eccentricity of the roof load, as shown in Figure 1, is given by:

$$e = t/2 + 3.5 = 9.625/2 + 3.5 = 8.3 \text{ inches}$$

For service loads,

$$\frac{P_w + P_f}{A_g} = \frac{(300 + 400) + 1666}{12(9.625)} \quad \text{…Equation (8-19)}$$
$$= 20.5 \text{ psi} = 0.01 f'_m < 0.04 f'_m$$

Therefore, the wall will be designed with the procedures in Section 2108.2.4.4. The moment at the mid-height of the wall is given by the following equation, which accounts for the secondary moments caused by wall displacement:

$$M_u = \frac{w_u h^2}{8} + P_{uf}\, e/2 + P_u \Delta_u \quad \text{…Equation (8-20)}$$

Where Δ_u is the deflection at mid-height due to factored loads. The deflection at mid-height depends on whether the wall moment has exceeded the cracking limit state, as specified in Section 2108.2.4.6 of the 1997 UBC. Thus, the cracking properties of the cross-section are calculated as follows.

The modulus of rupture of fully grouted hollow-unit masonry is given by:

$$f_r = 4\sqrt{f'_m} = 4\sqrt{1500} = 155 \text{ psi} \quad \text{…Equation (8-31)}$$

The cracking moment strength is equal to:

$$f_r = 4\sqrt{f'_m} = 4\sqrt{1500} = 155 \text{ psi} \quad \text{…Equation (8-30)}$$

The gross moment of inertia is equal to:

$$I_g = \frac{b(t)^3}{12} = \frac{12(9.625)^3}{12} = 891.7 \text{ in}^4/\text{ft}$$

Neglecting the effect of the compression steel, the depth of the compressive stress block is:

$$a = \frac{P_u + A_s f_y}{0.85 f'_m b} = \frac{1769 + 0.31\, 12/16\, (60000)}{0.85(1500)12}$$
$$= 1.03 \text{ inches}$$

…Equation (8-25)

The depth of the neutral axis is given by:

$$c = \frac{a}{0.85} = 1.21 \text{ inches}$$

The cracked moment of inertia is calculated using the formula recommended by the Structural Engineers Association of Southern California (SEAOSC):

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

The effective area of reinforcing steel, which includes the influence of the axial compression force on the wall, is given by:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{1769 + 0.31\, 12/16\, (60000)}{60000}$$
$$= 0.26 \text{ in}^2/\text{ft}$$

…Equation (8-24)

Thus,

$$I_{cr} = 25.8(0.26)(7.31 - 1.21)^2 + \frac{12(1.21)^3}{3}$$
$$= 256.7 \text{ in}^4/\text{ft}$$

The ultimate moment in the wall depends on the deflection of the wall. Since the deflection can not be initially known, an iterative procedure is used to determine both the ultimate moment and the deflection. For the first iteration, an assumption of $\Delta_{u1} = 0$ for the mid-height of the wall is made. Thus, the ultimate moment at the mid-height of the wall is equal to:

$$M_{u1} = \frac{w_u h^2}{8} + P_{uf}\, e/2 + P_u \Delta_u \quad \text{…Equation (8-20)}$$

$$M_{u1} = \frac{48(29 \times 12)^2}{8} + 270\, 8.3/2 + 1769(0)$$
$$= 5139.4 \text{ lb-ft/ft}$$

Since $M_{u1} > M_{cr}$ the wall deflection is calculated using:

$$\Delta_{u1} = \frac{5M_{cr}h^2}{48E_m I_g} + \frac{5(M_u - M_{cr})h^2}{48E_m I_{cr}} \quad \text{…Equation (8-29)}$$
$$\Delta_{u1} = \frac{5(2393 \times 12)(29 \times 12)^2}{48(1125000)891.7}$$
$$+ \frac{5[(5139.4 - 2393)x12](29 \times 12)^2}{48(1125000)256.7}$$
$$= 1.80 \text{ inches}$$

Using the calculated ultimate moment and displacement values from the previous iteration, the displacements quickly converge:

Iteration 2:

$$M_{u2} = 5404.8 \text{ lb-ft/ft}$$
$$\Delta_{u2} = 1.94 \text{ inches}$$

Iteration 3:

$$M_{u3} = 5425.3\ \text{lb-ft/ft}$$
$$\Delta_{u3} = 1.95\ \text{inches}$$

Since $\Delta_{u2} = 1.94\ \text{inches} \approx \Delta_{u3} = 1.95\ \text{inches}$ the process has converged and $M_u = 5425.3\ \text{lb-ft/ft}$. The design must now be checked against the moment capacity

$$\phi M_n \geq M_u \qquad \text{…Equation (8-22)}$$

Ignoring the contribution of the compression steel, the moment capacity of the wall, with the reinforcement at the face of the wall, is given by:

$$\phi M_n = \phi\left[A_{se} f_y \left(d - \tfrac{a}{2}\right) - P_u\left(d - \tfrac{t}{2}\right)\right]$$
$$\phi M_n = 0.80\left[\begin{array}{l}0.26(60000)\left(7.31 - \tfrac{1.03}{2}\right)\\ -1769\left(7.31 - \tfrac{9.625}{2}\right)\end{array}\right]$$
$$= 6772\ \text{lbs-ft/ft} > M_u = 5440\ \text{lbs-ft/ft}$$

Thus, the design is adequate for out-of-plane loads.
The equation for ϕM_n differs from Equation (8-23) in the 1997 UBC. This is because the code equation is only applicable to cross-sections with the reinforcement in the center of the wall.

<u>Strength Design for Out-of-Plane Loads Using the MSJC Code</u>

The eccentricity of the roof load is given by:

$$e = 8.3\ \text{inches}$$

For service loads,

$$\frac{P_u}{A_g} = 20.5\ \text{psi} = 0.01 f'_m < 0.05 f'_m$$

Therefore, the wall will be designed with the procedures in Section 3.2.5.4. Note that the stress cut-off level for P-Δ effects differs from that used in the 1997 UBC.

The moment at the mid-height of the wall is given by the following equation, which accounts for the secondary moments caused by wall displacement:

$$M_u = \frac{w_u h^2}{8} + P_{uf}\,\tfrac{e_u}{2} + P_u \delta_u \qquad \text{…Equation (3-24)}$$

This equation is similar to Equation (8-20) from the 1997 UBC. The deflection at mid-height, δ_u, is due to factored loads. The deflection at mid-height depends on whether the wall moment has exceeded the cracking limit state, as specified in Section 3.2.5.6 of the MSJC Code. We must therefore calculate the cracking properties of the cross-section.

The modulus of rupture of fully grouted hollow-unit masonry, with the direction of the flexural tensile stress normal to the bed joints, is given by Table 3.1.7.2.1 as:

$$f_r = 145\ \text{psi}$$

This value differs from that given by Equation (8-31) from the 1997 UBC. Thus, the cracking moment strength is equal to:

$$M_{cr} = S_n f_r = \frac{12(9.625)^2}{6} 145 \qquad \text{…Equation (3-32)}$$
$$= 26866\ \text{lbs-in/ft} = 2239\ \text{lbs-ft/ft}$$

The gross moment of inertia is equal to:

$$I_g = \frac{b(t)^3}{12} = \frac{12(9.625)^3}{12} = 891.7\ \text{in}^4/\text{ft}$$

Neglecting the effect of the compression steel, the depth of the compressive stress block is:

$$a = \frac{P_u + A_s f_y}{0.80 f'_m b} = \frac{1769 + 0.31\,\tfrac{12}{16}(60000)}{0.80(1500)12} \qquad \text{…Equation (3-28)}$$
$$= 1.09\ \text{inches}$$

The depth of the neutral axis is given by:

$$c = \frac{a}{0.80} = 1.36\ \text{inches}$$

Note that the masonry compressive stress block provided by Section 3.2.2(g) in the MSJC code is smaller that that allowed in Section 2108.2.1.2 of the 1997 UBC.

The cracked moment of inertia is calculated using the formula recommended by the Structural Engineers Association of Southern California (SEAOSC):

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

The effective area of reinforcing steel, which includes the influence of the axial compression force on the wall, is given by:

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{1769 + 0.31\,\tfrac{12}{16}(60000)}{60000}$$
$$= 0.26\ \text{in}^2/\text{ft}$$

Thus,

$$I_{cr} = 25.8(0.26)(7.31 - 1.36)^2 + \frac{12(1.36)^3}{3}$$
$$= 247.5\ \text{in}^4/\text{ft}$$

The ultimate moment in a wall depends on the deflection of the wall. Since the deflection can not be initially known, an iterative procedure is used to determine both the ultimate moment and the deflection. For the first iteration, an assumption of $\delta_{u1} = 0$ for the mid-height of the wall is made. Thus, the ultimate moment at the mid-height of the wall is equal to:

$$M_{u1} = \frac{w_u h^2}{8} + P_{uf} \frac{e_u}{2} + P_u \delta_u \quad \text{…Equation (3-24)}$$

$$M_{u1} = \frac{48(29 \times 12)^2}{8} + 270 \frac{8.3}{2} + 1769(0)$$
$$= 5139.4 \text{ lb-ft/ft}$$

Since $M_{u1} > M_{cr}$ the wall deflection is calculated using:

$$\delta_{u1} = \frac{5 M_{cr} h^2}{48 E_m I_g} + \frac{5(M_u - M_{cr})h^2}{48 E_m I_{cr}} \quad \text{…Equation (3-31)}$$

$$\delta_{u1} = \frac{5(2239 \times 12)(29 \times 12)^2}{48(1350000)891.7} + \frac{5[(5139.4 - 2239)\times 12](29 \times 12)^2}{48(1350000)247.5}$$
$$= 1.6 \text{ inches}$$

Per Section 1.8.2.2.1 of the MSJC Code:

$E_m = 900 f'_m$ for concrete masonry

This results in an increase in stiffness of 20% over that provided by the 1997 UBC.

Using the calculated displacement value from the previous iteration the displacements quickly converge:

Iteration 2:

$$M_{u2} = 5374.6 \text{ lb-ft/ft}$$
$$\delta_{u2} = 1.7 \text{ inches}$$

Iteration 3:

$$M_{u3} = 5390.3 \text{ lb-ft/ft}$$
$$\delta_{u3} = 1.71 \text{ inches}$$

Since $\delta_{u2} = 1.7 \text{ inches} \approx \delta_{u3} = 1.71 \text{ inches}$ the process has converged and $M_u = 5390.3 \text{ lb-ft/ft}$.

The design must now be checked against the moment capacity

$$\phi M_n \geq M_u \quad \text{…Equation (3-26)}$$

Per Section 3.1.4.1 the strength reduction factor, ϕ, for combinations of flexure and axial load in reinforced masonry shall be taken as 0.9. This represents an increase in capacity over the 1997 UBC where ϕ=0.8 for walls with unfactored axial load of 0.04f'_m or less.
Ignoring the contribution of the compression steel, the moment capacity of the wall with the reinforcement at the face of the wall is given by:

$$\phi M_n = \phi\left[A_{se} f_y \left(d - \frac{a}{2}\right) - P_u\left(d - \frac{t}{2}\right)\right]$$

$$\phi M_n = 0.90\left[\begin{matrix} 0.26(60000)\left(7.31 - \frac{1.09}{2}\right) \\ -1769\left(7.31 - \frac{9.625}{2}\right)\end{matrix}\right]$$
$$= 7583.7 \text{ lbs-ft/ft} > M_u = 5381.1 \text{ lbs-ft/ft}$$

Thus, the design is adequate for out-of-plane loads.

The equation for differs from Equation ϕM_n (3-27) in the MSJC Code because the code equation is only applicable to cross-sections with the reinforcement in the center of the wall.

Due to the higher stiffness, the smaller compression block, and the larger strength reduction factor allowed by the MSJC Code the wall design for the MSJC Code allows the designer to use less reinforcement than that allowed by the 1997 UBC.

The following example redesigns the wall using 2-layers of #6 @ 32" o.c. (A_s = 0.17 in^2)

Neglecting the effect of the compression steel, the depth of the compressive stress block is now:

$$a = \frac{P_u + A_s f_y}{0.80 f'_m b} = \frac{1769 + 0.44\frac{12}{32}(60000)}{0.80(1500)12}$$
$$= 0.81 \text{ inches}$$

Thus, the depth of the neutral axis is given by:

$$c = \frac{a}{0.80} = 1.02 \text{ inches}$$

Since,

$$A_{se} = \frac{P_u + A_s f_y}{f_y} = \frac{1769 + 0.44\frac{12}{32}(60000)}{60000}$$
$$= 0.20 \text{ in}^2/\text{ft}$$

Thus,

$$I_{cr} = nA_{se}(d-c)^2 + \frac{bc^3}{3}$$
$$= 25.8(0.20)(7.31-1.02)^2 + \frac{12(1.02)^3}{3}$$
$$= 204 \text{ in}^4/\text{ft}$$

For the first iteration, an assumption of $\delta_{u1} = 0$ for the mid-height of the wall is made. Thus, the ultimate moment at the mid-height of the wall is equal to:

$$M_{u1} = \frac{\varpi_u h^2}{8} + P_{uf}\,{}^{e_u}\!/_2 + P_u\delta_u$$

$$M_{u1} = \frac{48(29 \times 12)^2}{8} + 270\,{}^{8.3}\!/_2 + 1769(0)$$
$$= 5139.4 \text{ lb-ft/ft}$$

Since $M_{u1} > M_{cr}$ the wall deflection is calculated using:

$$\delta_{u1} = \frac{5M_{cr}h^2}{48E_mI_g} + \frac{5(M_u - M_{cr})h^2}{48E_mI_{cr}}$$
$$= \frac{5(2239 \times 12)(29 \times 12)^2}{48(1350000)891.7}$$
$$+ \frac{5[(5139.4-2239)\times 12](29 \times 12)^2}{48(1350000)204}$$
$$= 1.88 \text{ inches}$$

Iteration 2:

$$M_{u2} = 5416.3 \text{ lb-ft/ft}$$
$$\delta_{u2} = 2.03 \text{ inches}$$

Iteration 3:

$$M_{u3} = 5438.8 \text{ lb-ft/ft}$$
$$\delta_{u3} = 2.04 \text{ inches}$$

Since $\delta_{u2} = 2.03 \text{ inches} \approx \delta_{u3} = 2.04 \text{ inches}$ the process has converged and $M_u = 5438.8 \text{ lb-ft/ft}$.

Design check:

$$\phi M_n \geq M_u$$

Again ignoring the contribution of the compression steel:

$$\phi M_n = \phi\left[A_{se}f_y\left(d - {}^{a}\!/_2\right) - P_u\left(d - {}^{t}\!/_2\right)\right]$$

$$\phi M_n = 0.90\begin{bmatrix}0.26(60000)\left(7.31 - {}^{0.81}\!/_2\right) \\ -1769\left(7.31 - {}^{9.625}\!/_2\right)\end{bmatrix}$$
$$= 5731.2 \text{ lbs-ft/ft} > M_u = 5438.8 \text{ lbs-ft/ft}$$

Thus, for this example, optimizing the out-of-plane wall design using the MSJC Code allows the designer to use about 20% less steel than when the wall is designed using the procedures from the 1997 UBC.

CONCLUSIONS

There is little difference between the design methodologies in the 1997 UBC and the MSJC Code. However, the MSJC Code allows for a stiffer masonry section and a larger strength reduction factor than that allowed by the 1997 UBC. These factors combine such that a wall designed using the MSJC Code requires less flexural steel than one designed using the 1997 UBC.

ERRATA FOR "*SUMMER 2003*" ISSUE:

In the calculation of effective area, A_{se}, for the MSJC Code designs, the modular ratio of elasticity, *n*, should be:

$n = E_s/E_m = 29000/900 \times 1.5 = 21.5$ not $n = 25.8$

This results in an increase in the deflection and the ultimate moment but does not affect the over-all design of the section.

The next issue of *"Masonry Chronicles"* will provide examples to highlight the differences in in-plane design between the 1997 UBC and the MSJC Code.

This issue of "Masonry Chronicles" was written by Melissa Kubischta of Hart-Weidlinger.

Please contact the Association Office, with any comments or suggestions for future issues.

Additional *"Masonry Chronicles"* can be seen on the CMACN web site at http://www.cmacn.org.

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CMACN IS PROUD TO ANNOUNCE OUR NEW EXECUTIVE DIRECTOR, PAUL D. BAMBAUER

Paul D. Bambauer, Executive Director, Concrete Masonry Association of California and Nevada (CMACN)

The Concrete Masonry Association of California and Nevada announces the appointment of Paul Bambauer as the new Executive Director of the Association. Paul joins the CMACN with 20 years in the cement and concrete products industry working in financial accounting, administration, sales and marketing. Most recently, Paul has been consulting in the strategic market development areas for various concrete wall systems, and prior to consulting, was Western Region Vice President of Sales for Southdown Cement, covering the California, Arizona and Nevada markets. As Southdown's representative on various market development boards and committees within the National Concrete Masonry Association, Portland Cement Association and local product promotion groups, Paul held leadership positions in most areas of product promotion and has a broad range of experience. He earned a Bachelor of Science in Business Administration from the University of Arizona in Tucson in 1977.

CMACN believes Paul's promotional and administrative skills will provide new opportunity and direction for CMACN marketing activities.

Paul may be contacted at Paul@cmacn.org or (714) 504-4497.

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