Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Fall 2003

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

Part II: In-Plane Loads on Masonry Walls

Introduction

The previous issues 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 provide examples for the design of masonry walls for in-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. Since the 2003 International Building Code (2003 IBC) references the MSJC Code, modifications to the MSJC Code in the 2003 IBC that effect design will be mentioned.

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

Design of Wall to Resist In-Plane Forces

In order to compare the design of CMU walls to resist in-plane loads, a typical 10" (nominal) CMU wall (f'_m=2500 psi)) from a 6-story residential building will be designed. The average wall height is 10-feet. The loads on the wall to be designed are given in Table 1.

Table 1: Loads acting on example wall for in-plane shear

	Dead (kips)	Live (kips)	V (kips)	M (kip-ft)
Roof	38	2	87	870
5th	76	5	180	2670
4th	114	9	253	5200
3rd	152	12	306	8260
2nd	190	15	339	11650
1st	228	17	354	15190

Comparison of Working Stress Design of In-Plane Loads Between the 1997 UBC and the MSJC Code

The factored loads on the wall for Working Stress Design are given as:

$$V = 253 \text{ kips}$$
$$M = 10850 \text{ kip - ft}$$

The design of in-plane loads between the 1997 UBC and the MSJC Code differs mainly in the inclusion, in Section 2107.1.7 of the 1997 UBC, of a 1.5 multiplier when calculating shear or diagonal tension stresses in Seismic Zones 3 and 4. This multiplier is not included in the MSCJ Code.
Thus, if we assume the distribution of reinforcement in the sample wall is as shown in Figure 1. Then,

$$d = 348 \text{ inches}$$
$$j = 0.875$$
$$b = 9.625 \text{ inches}$$

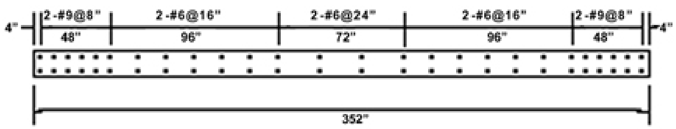

Figure 1: Reinforcement for Working Stress Design

The shear stress on the wall is given by:

$$f_v = \frac{V}{bjd} = \frac{1.5(253000)}{9.625(0.875)(348)} = 129 \text{ psi}$$

…Equation (7-37)

At the first story,

$$\frac{M}{Vd} = \frac{10850(12)}{253(348)} = 1.47 > 1.0$$

Thus, when shear reinforcement is provided per Section 2107.2.17 of the 1997 UBC:

$$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{2500} = 75 \text{ psi}$$

…Equation (7-22)

A one third increase is allowed in Working Stress Design by the 1997 UBC for short-term loads. Thus,

$$1.33F_v = 100 \text{ psi} < f_v = 129 \text{ psi}$$

The shear stress on the wall exceeds the allowable shear stress using working stress design per the 1997 UBC. To satisfy the shear requirement the wall thickness should be increased or masonry with a higher compressive strength should be used.

Since the MSJC Code does not use a similar multiplier in the calculation of shear stress in Working Stress Design, the shear stress on the same wall is given by:

$$f_v = \frac{V}{bd} = \frac{(253000)}{9.625(348)} = 68.5 \text{ psi}$$ …Equation (2-19)

At the first story,

$$\frac{M}{Vd} = \frac{10850(12)}{253(348)} = 1.47 > 1.0$$

Thus, when shear reinforcement is provided per Section 2.3.5.3 of the MSJC Code:

$$F_v = 1.5\sqrt{f'_m} = 1.5\sqrt{2500} = 75 \text{ psi} > f_v = 68.5 \text{ psi}$$

…Equation (2-25)

When the wall is designed according to the MSJC Code the wall has adequate shear strength to resist the loads. Note, however, that the 1.5 multiplier is required by Section 2106.5.1of the 2003 IBC.

Strength Design for In-Plane Loads Using the 1997 UBC

The factored loads on the wall for Strength Design are given as:

$P_{u1} = 337$ kips …Combination 1

$P_{u2} = 205$ kips …Combination 2

For both combinations:

$$V_u = 354 \text{ kips}$$
$$M_u = 15190 \text{ kip - ft}$$

The area of distributed steel required in the wall can be estimated by:

$$A_{s,req} \approx \frac{3.5M}{df_y} \approx \frac{3.5(15190 \text{ x}12)}{348(60)} = 30.6 \text{ in}^2$$

If we use only jamb steel, the required area of reinforcing steel can be estimated by (assuming $d - a/2 = 0.9d$):

$$A_{s,req} \approx \frac{M}{\phi f_y 0.9d} \approx \frac{(15190 \text{ x}12)}{0.8(60)0.9(348)} = 12.1 \text{ in}^2$$

Using a combination of distributed steel and jamb steel, we can try 8-#9 at each end ($A_s = 8.0 \text{ in}^2$) and 2-#5 @ 16" o.c. for the rest of the wall ($A_s = 9.92 \text{ in}^2$), as shown in Figure 2.

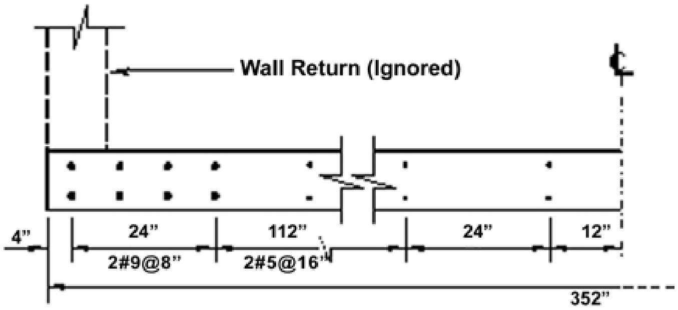

Figure 2: Vertical Reinforcing Steel in Example Wall

To obtain the moment capacity of the wall under various axial loads, an axial load-moment interaction diagram of the wall, with the selected reinforcement, has to be created. For simplicity, three control points will be located on the interaction diagram. This is a conservative approximation; a more accurate curve can be obtained by calculating more points on the interaction diagram.

With no eccentricity of axial load (Control Point 1) and masonry with a compressive strength of 2500 psi (Section 2108.2.5.4 of the 1997 UBC):

$$P_o = 0.85 f'_m (A_e - A_s) + A_s f_y \quad \text{...Equation (8-34)}$$

$$P_o = 0.85(2500)(352 x 9.625 - 25.92) + (25.92)(60) = 8700 \text{ kips}$$

The wall axial load is limited by Equation (8-35):

$$P_u = 0.8\phi P_o = 0.8(0.65)(8700) = 4524 \text{ kips}$$

For Control Point 2, which has no axial load, we iterate to obtain the neutral axis of the section so that the sum of vertical forces equals zero. A rectangular compressive stress block, with a maximum usable compressive strain of 0.003 is used for the masonry, as stipulated in Section 2108.2.1.2 of the 1997 UBC. Using a spread-sheet for calculating the in-plane moment in the wall with no axial load, the neutral axis is located 38.45 inches from the extreme compressive fiber and the moment on the cross-section is equal to 20708 kip-ft. A capacity reduction factor of 0.85 may be used for walls with symmetric reinforcement and no axial load.

At the balanced strain condition (Control Point 3), the strain in the extreme compressive fiber is equal to 0.003 and the reinforcing steel is just yielding as shown in Figure 3. The neutral axis location is given by:

$$\text{NA} = \frac{0.003}{0.003 + 0.00207}(348) = 205.9$$

Checking the equilibrium of the wall cross-section with the given neutral axis location, the balanced axial load is found to be 3739 kips and the balanced moment is 41091 kip-ft. Figure 3 shows the interaction diagram for the wall using the three control points obtained.

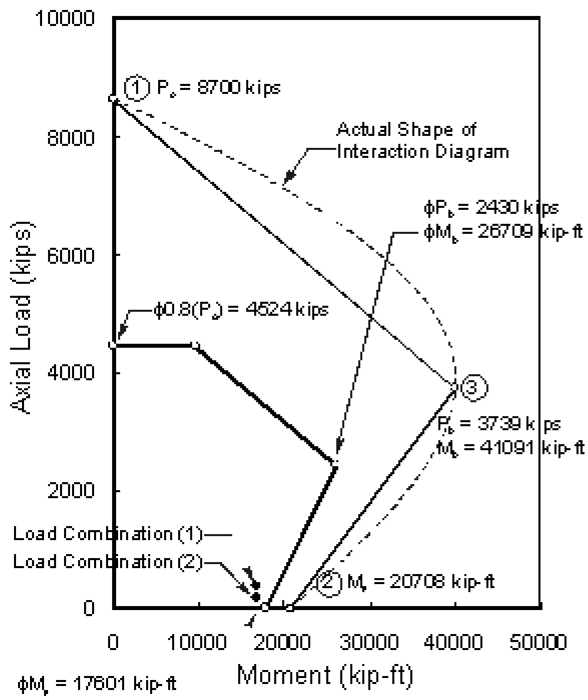

Figure 3: Interaction Diagram for Example Wall

When the wall failure mode is in flexure, Section 2108.2.5.2 of the 1997 UBC stipulates that the flexural strength of the wall should be at least 1.8 times the cracking moment strength. This is to prevent a sudden loss of strength in the section when the masonry cracks during loading. For fully grouted hollow unit masonry, the modulus of rupture is given by:

$$f_r = 4.0\sqrt{f'_m} = 4.0\sqrt{2500} = 200 \text{ psi}$$

…Equation (8-31)

The cracking moment strength is equal to:

$$M_{cr} = Sf_r = \frac{9.625(352)^2}{6}(0.2)\frac{1}{12} = 3313 \text{ kip - ft}$$

…Equation (8-30)

$$1.8 M_{cr} = 5963 \text{ kip - ft} < M_n = 20708 \text{ kip - ft} \quad \text{…OK}$$

In-Plane Shear 1st Story

Recall:

$$V_u = 354 \text{ kips}$$
$$M_u = 15190 \text{ kip - ft}$$

For a ductile flexural failure mode, the shear capacity must exceed the shear corresponding to development of the wall nominal flexural strength. For load combination

(1), which has the larger axial load and thus the larger nominal moment, the axial load is 337 kips. From the interaction diagram, the nominal moment strength is estimated to be:

$$\begin{aligned} M_n &= 20708 + \frac{(41091 - 20708)}{3739}(337) \\ &= 22545 \text{ kip - ft} \end{aligned}$$

The shear corresponding to this moment is equal to:

$$V_{ductile} = 337\left(\frac{22545}{15190}\right) = 500 \text{ kips}$$

The required shear reinforcement is obtained from Equation (8-39) of the 1997 UBC:

$$\rho_n = \frac{V_u}{A_{mv} f_y} = \frac{500}{(352)(9.625)60} = 0.0025$$

If we try 2 layers of #4 bars spaced at 16 inches (ρ_n = 0.0026) the nominal shear in the region at the base of the wall (where a plastic hinge may form) is given by:

$$\begin{aligned} V_n &= A_{mv}\rho_n = (352 \times 9.625)(0.0026)60 \\ &= 529 \text{ kips} \geq V_{ductile} = 500 \text{ kips} \end{aligned}$$

Thus, we do not expect shear failure to occur, and the wall should respond in a ductile flexural mode.

Since the nominal shear capacity exceeds the shear corresponding to development of the wall nominal flexural strength, two shear regions exist as stipulated in Section 2108.2.5.5 of the 1997 UBC. In the region at the base of the wall:

$$\begin{aligned} \phi V_n &= \phi A_{mv}\rho_n = 0.8(529) \\ &= 423 \text{ kips} > 337 \text{ kips} \end{aligned}$$

For the region above half the story height:

$$\frac{M}{Vd} = \frac{15190}{337(29)} = 1.55 \Rightarrow C_d = 1.2 \qquad \text{…Table (21-K)}$$

The shear strength is given in Section 2108.2.5.5 of the 1997 UBC as:

$$\begin{aligned} \varphi V_n &= \varphi(V_m + V_s) = \varphi\left(C_d A_{mv}\sqrt{f'_m} + A_{mv}\rho_n f_y\right) \\ &= 0.65\left\{\begin{aligned} &\frac{1.2}{1000}(352 \times 9.625)\sqrt{2500} \\ &+ (352 \times 9.625)(0.0026)60 \end{aligned}\right\} \\ &= 476 \text{ kips} > 337 \text{ kips} \qquad \text{…OK} \end{aligned}$$

<u>Boundary Members</u>

In Section 2108.2.5.6 of the 1997 UBC Boundary elements are specified for walls resisting in plane loads when the compressive strains in the wall, determined using factored forces and R_w equal to 1.5, exceed 0.0015. Also the maximum reinforcement ratio in the 1997 UBC is given as $0.5\rho_b$.

Note that in this section the 1997 UBC refers to the obsolete Rw factor that has been replaced by the R_w factor in the 1997 UBC. As discussed in *Design of Reinforced Masonry Structures* by Brandow, Hart and Virdee, published by the Concrete Masonry Association of California and Nevada, a comparison of the old Rw factor and the R factor (4.5 vs. 6 for masonry bearing walls) results in using an *R* of 1.1.

The design forces for the bearing wall were calculated with an *R* factor of 4.5. Since the check for boundary members must be performed with an *R* of 1.1, the factored loads can be multiplied by 4.5/1.1 = 4.09. Then, boundary members are required if the moment capacity of the wall at a maximum compressive strain of 0.0015 is less than $4.09M_u$.

To calculate the moment capacity at a maximum compressive strain of 0.0015, a linear masonry compressive stress-strain curve should be assumed, with a strain at peak stress of 0.002. Thus, a triangular masonry compressive stress block can be used to calculate the moment, and the stress at the extreme fiber is then given by:

$$f'_m = 2500\,\frac{0.0015}{0.002} = 1875 \text{ psi}$$

Figure 4 shows the stress and strain in the wall cross-section at a factored axial load of 165 kips.

The moment about the center of the cross-section is equal to:

$$\begin{aligned} M_{0.0015} &= \Sigma f_{si} A_{si}\left(\frac{L_w}{2} - d_i\right) \\ &+ \frac{0.75 f'_m (c) t_w}{2}\left(\frac{L_w}{2} - \frac{c}{3}\right) \\ &= 20943 \text{ kip - ft} < 4.09\text{M}_u \end{aligned}$$

where :

$$4.09\text{M}_u = 4.09(16709) = 68340 \text{ kip - ft}$$

Thus, boundary members are required. The minimum length of the boundary members is three times the wall thickness.

Strength Design for In-Plane Loads Using the MSJC Code

Recall that the factored loads for Strength design are given as:

$$P_{u1} = 337 \text{ kips} \qquad \text{…Combination 1}$$

$$P_{u2} = 205 \text{ kips} \qquad \text{…Combination 2}$$

For both combinations:

$$V_u = 354 \text{ kips}$$
$$M_u = 15190 \text{ kip - ft}$$

Similar to the 1997 UBC example, we will try a combination of distributed steel and jamb steel, 8-#9 at each end (A_s = 8.0 in^2) and 2-#5 @ 16" o.c. for the rest of the wall (A_s = 9.92 in^2).

The calculation for the control points on the interaction diagram using the MSJC Code is similar to the procedure used for the 1997 UBC. However, the MSJC Code nominal axial compressive strength calculation takes into account slenderness of the wall by the inclusion of slenderness dependent modification factors. Also, the stress block for the MSJC Code is equal to $(0.8)^2 f'_m cb$ and the maximum compressive strain in concrete masonry is 0.0025 per Section 3.2.2 of the MSJC Code. In addition, the strength reduction factor for reinforced walls subjected to combinations of flexure and axial load is taken as 0.9 per Section 3.1.4.1 of the MSJC Code.

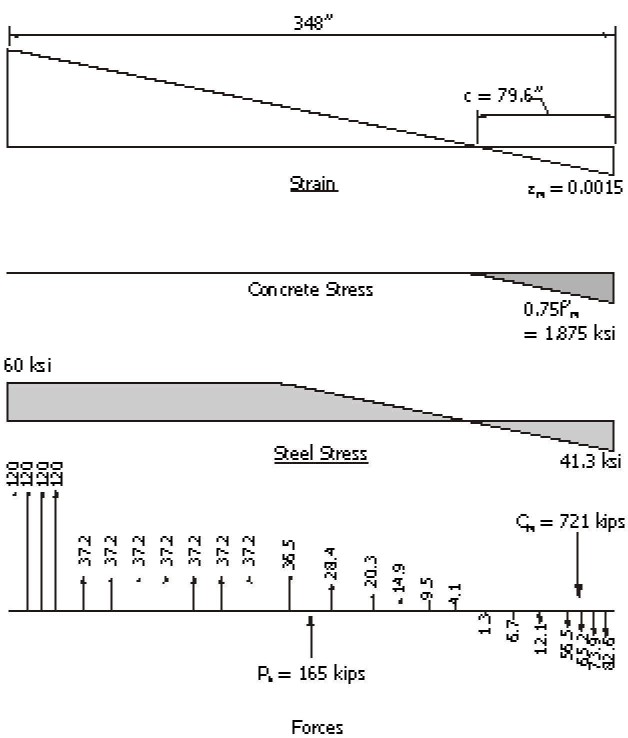

Figure 4: Equilibrium of Cross Section at Maximum Compressive Strain of 0.0015

Control Point 1 occurs where the wall experiences no eccentricity of axial load. The axial capacity of the wall using the MSJC Code is:

$$P_n = 0.80\left[0.80 f'_m (A_n - A_s) + f_y A_s\right]R \qquad \text{…Equation (3-16)}$$

Where R is the slenderness dependent modification factor. Since, the slenderness ratio for the wall is:

$$h/r = \frac{60 \times 12}{2.78} = 43.2 < 99$$

The slenderness modification factor and nominal axial strength are equal to:

$$\left(1 - \left(\frac{h}{140\,r}\right)^2\right) = 1 - \left(\frac{43.2}{140}\right)^2 = 0.9$$

And,

$$\begin{aligned} P_n &= 0.80\begin{bmatrix} 0.80\,(2500)\times\left[(352 \times 10) - 25.92\right] \\ + (60)25.92 \end{bmatrix} \\ &= 0.80\left[8279\right]0.9 \\ &= 5993 \text{ kips} \end{aligned}$$

For Control Point 2, which has no axial load, we iterate to obtain the neutral axis of the section so that the sum of vertical forces equals zero. A rectangular compressive stress block, with a maximum usable compressive strain of 0.0025 is used. The neutral axis is located 43.1 inches from the extreme compressive fiber, and the moment on the cross-section is equal to 20575 kip-ft.

At the balanced strain condition (Control Point 3), the strain in the extreme compressive fiber is equal to 0.0025 and the reinforcing steel is just yielding. The neutral axis location is given by:

$$\text{NA} = \frac{0.0025}{0.0025 + 0.00207}(348) = 190.4 \text{ inches}$$

Checking the equilibrium of the wall cross-section with the given neutral axis location, the balanced axial load is found to be 3023 kips and the balanced moment is 38960 kip-ft. Figure 5 shows the interaction diagram for the wall using the three control points obtained. Notice that the ultimate moment acting on the wall is 86% of the reduced capacity per the 1997 UBC and 82% of the reduced capacity per the MSJC Code.

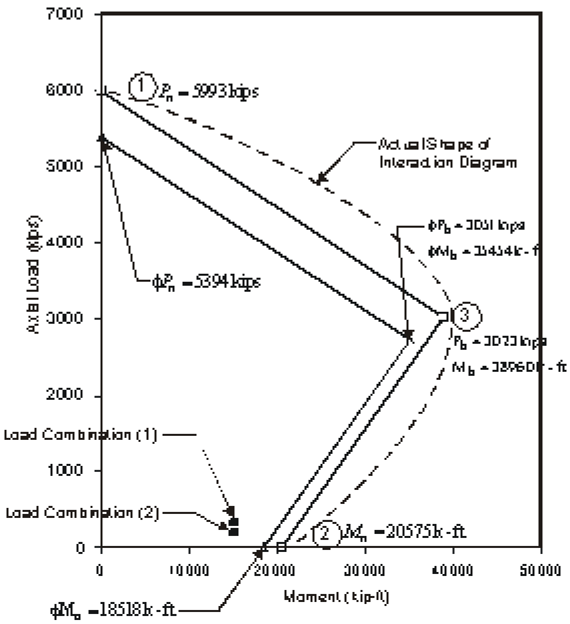

Figure 5: Interaction Diagram for Example Wall

When the wall failure mode is in flexure, Section 3.2.4.2.2.2 of the MSJC Code stipulates that the flexural strength of the wall should be at least 1.3 times the cracking moment strength. This is to prevent a sudden loss of strength in the section when the masonry cracks during loading. For masonry subjected to in-plane loads, the modulus of rupture, f_r, normal to the bed joints shall be taken as 250 psi per Section 3.1.7.2.2 of the MSJC Code.

Thus, the cracking moment strength is equal to:

$$M_{cr} = S_n f_r = \frac{10\,(352)^2}{6} 250 = 4302 \text{ kip - ft}$$

…Equation (3-32)

$$1.3 M_{cr} = 5593 \text{ kip - ft} < M_n = 20576 \text{ kip - ft} \quad \text{…OK}$$

<u>In-Plane Shear 1st Story</u>

Recall,

$$V_u = 354 \text{ kips}$$
$$M_u = 15190 \text{ kip - ft}$$

Per Section 3.1.3 of the MSJC Code, for a ductile flexural failure mode, the shear design capacity must exceed 1.25 times the shear corresponding to development of the wall nominal flexural strength. For load combination (1), which has the larger axial load and thus the larger nominal moment, the axial load is 337 kips. From the interaction diagram, the nominal moment strength is estimated to be:

$$M_n = 20575 + \frac{(38960 - 20575)}{3023}(337)$$
$$= 22625 \text{ kip - ft}$$

The shear corresponding to this moment is equal to:

$$V_{ductile} = 337\left(\frac{22625}{15190}\right) = 502 \text{ kips}$$

The nominal shear strength, as computed in accordance with Section 3.2.4.1.2 of the MSJC Code, includes both masonry and steel shear strength for the entire wall.

$$V_n = V_m + V_s \qquad \text{…Equation (3-18)}$$

Where *Vn* shall not exceed the following:

$$\text{Where } M/Vd_v \le 0.25$$
$$V_n \le 6 A_n \sqrt{f'_m} \qquad \text{…Equation (3-19)}$$

$$\text{Where } M/Vd_v \ge 1.00$$
$$V_n \le 4 A_n \sqrt{f'_m} \qquad \text{…Equation (3-20)}$$

The maximum value of V_n for M/Vd_v between 0.25 and 1.00 may be interpolated.

Since,

$$M/Vd_v = \frac{(15190)12}{(354)352} = 1.46$$

$$V_{n,max} \le 4(358\,x10)\sqrt{2500} = 716 \text{ kips}$$

Per Section 3.2.4.1.2.1 of the MSJC code, the nominal masonry shear strength equals:

$$V_m = \left[4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right] A_n \sqrt{f'_m} + 0.25 P$$

…Equation (3-21)

$$V_m = [4.0 - 1.75\,(1.0)]3580\,\sqrt{2500} + 0.25(371)$$
$$= 496 \text{ kips}$$

Try 2-layers of #4 bars spaced at 16 inches. Per Section 3.2.4.1.2.2 of the MSJC Code the nominal shear strength provided by the reinforcement equals:

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v \qquad \text{…Equation (3-22)}$$

$$V_s = 0.5\left(\frac{0.2\text{x}2}{16}\right)(60)(352) = 264 \text{ kips}$$

Given the strength reduction factor for shear is 0.80 per Section 3.1.4.3 of the MSJC Code:

$$\phi V_n = \phi\left(V_m + V_s\right) = 0.80\left(496 + 264\right)$$
$$= 608 \text{ kips} < 1.25 V_{ductile} = 665 \text{ kips} \quad \text{...OK}$$

Except that the nominal shear strength, V_n, need not exceed 2.5 times the required shear strength, V_u.

$$V_n = 960 \text{ kips} < 2.5 V_u = 885 \text{ kips} \quad \text{...OK}$$

Thus, the wall should respond in a ductile flexural mode. Note that Section 2106.5.2 of the 2003 IBC specifies the use of steel shear strength only in the base of the wall for Seismic Design Category D when the ductile flexural mode governs design.

Maximum Reinforcement Percentages

Per section 3.2.3.5 of the MSJC Code, masonry strain in shear walls is limited to that which can be developed when strain is limited to 5 times yield in the extreme tension reinforcement. The calculation of the equilibrium includes unfactored gravity axial loads. In addition, the stress in the tension reinforcement is assumed to be $1.25f_y$ and the strength of the masonry compression zone is 80% f'_m times 80% of the area of the compression zone.

Using the limitations in Section 3.2.3.5, the maximum masonry compressive strain can be calculated using a trial and error procedure.

Thus,

$$\varepsilon_m = 0.002$$

Since this value does not exceed the allowable masonry strain of 0.0025 specified in Section 3.2.2 of the MSJC Code, the reinforcement is adequate.

Conclusion

Working Stress Design for in-plane loads using the MSJC Code allows for larger shear capacities due to the exclusion of the 1.5 multiplier for shear loads that is included in the 1997 UBC. Strength Design for in-plane loads includes several significant differences between design using the 1997 UBC and design using the MSJC Code. These differences include the consideration of slenderness in axial strength, the smaller stress block, smaller maximum usable strain, and larger strength reduction factors for axial load and flexure. In addition, the shear strength calculation using the MSJC Code for the entire wall includes the contributions from both masonry and steel. In the 1997 UBC, when the ductile flexural mode governs design the shear strength at the base of the wall includes only the contribution of the steel. Finally, the MSJC Code does not allow for the inclusion of boundary elements to limit strain like the 1997 UBC. Instead the maximum reinforcement percentage is limited to that which can develop a strain of 5 times yield in the extreme tension reinforcement and a maximum masonry strain of 0.0025.

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

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.

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