Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

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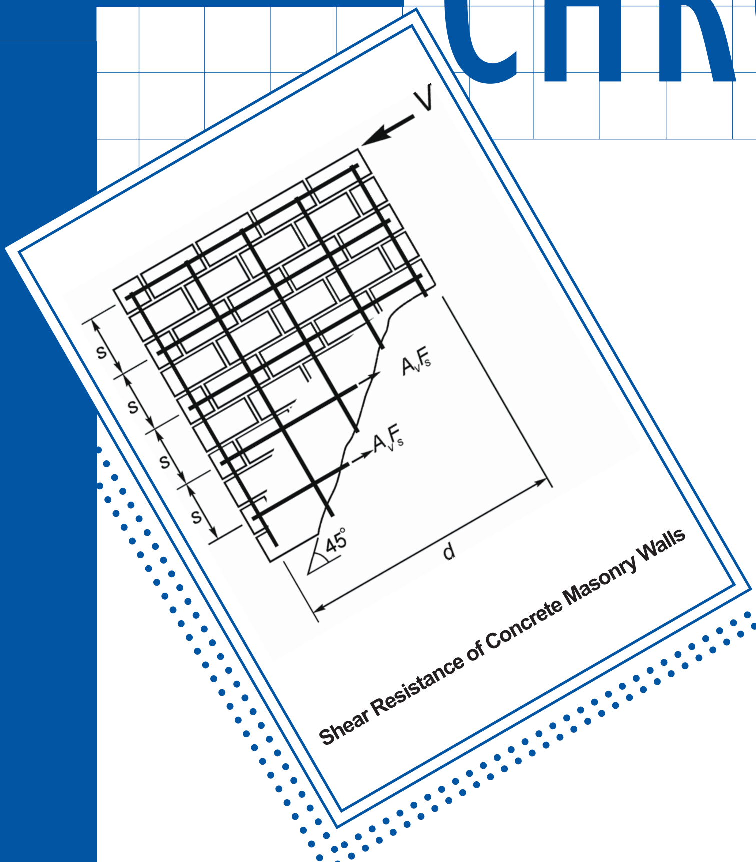

The Comparison of 2006 IBC and 1997 UBC Provisions for the Shear Resistance of Concrete Masonry Shear Walls

Introduction

The design of the shear resistance provided by shear walls remains an important aspect of shear wall design. Since shear failure usually occurs suddenly, without warning, deficiencies in the shear strength of structural members should be avoided. To preclude the possibility of a brittle shear failure, special attention in the form of ductile detailing, provisions dictating the amount and spacing of shear reinforcing, along with the use of various capacity reduction factors, are imposed by the governing building codes.

The Winter 2007 issue of *Masonry Chronicles* discussed the flexural and axial design of concrete masonry shear walls. This edition will discuss the design of concrete masonry shear walls to resist in-plane shear loads.

Examples of calculations illustrating the required amount of horizontal reinforcement in shear walls will be provided to illustrate the differences between the methodologies 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/TMS402-05 [3], which is also referred to as the 2005 Masonry Standards Joint Committee Building Code (MSJC).

For examples involving the shear resistance of concrete masonry shear walls we will revisit the examples initially utilized in the Winter 2007 issue of *Masonry Chronicles*. Results from this prior issue will aid in ensuring the compliance of our design with the governing building codes. Past issues of *Masonry Chronicles* are conveniently located on the Concrete Masonry Association of California and Nevada (CMACN) website (www.cmacn.org).

Nomenclature

The nomenclature used in this article is as follows:

A_e, A_n = Cross-sectional area of masonry considered
A_{mv} = Net area of masonry in direction of shear force considered
A_s, A_v = Cross-sectional area of shear steel reinforcement
b = Effective width
C_d = Nominal shear strength coefficient
d = Distance to rebar
d_v = Actual depth of masonry in direction considered
f = Flexural tension
f'_m = Specified compressive strength of masonry
F_s = Allowable shear stress in reinforcement
f_v = Computed shear stress
F_v = Allowable shear stress in masonry
f_y = Yield stress of reinforcement
h = Effective height of wall
j = Ratio between centroid of flexural compressive forces and centroid of tensile forces
l = Effective length of wall
M = Maximum moment on section
M_E = Moment due to earthquake loading
M_u = Factored moment

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P_D = Axial dead load
P_L = Axial live load
P_{Lr} = Axial roof live load
S = Section Modulus
t = Actual thickness of masonry
V = Maximum shear on section
$V_{ductile}$ = Shear demand developed from corresponding flexural strength
V_E = Shear due to earthquake loading
V_m = Nominal shear strength of masonry
V_n = $V_m + V_s$
V_s = Nominal shear strength of shear reinforcement
V_u = Required shear strength
ρ_n = Ratio of distributed shear reinforcement on
plane perpendicular to plane of A_{mv}.

Example 1:
Shear Resistance of Concrete Masonry Bearing Wall using the 1997 UBC Allowable Stress Design Procedures

Determine the horizontal reinforcement and minimum vertical steel required at the first story for the wall with the earthquake loads shown in Figure 1. The fully grouted wall is constructed with 8-inch medium-weight concrete masonry units (78 psf) and is located in a building assigned to seismic zone 4. The specified masonry compressive strength is 1500 psi and grade 60 steel is used. Use the alternate load combinations of Section 1605.3.2 of the UBC (a one-third increase in allowable stresses are permitted).

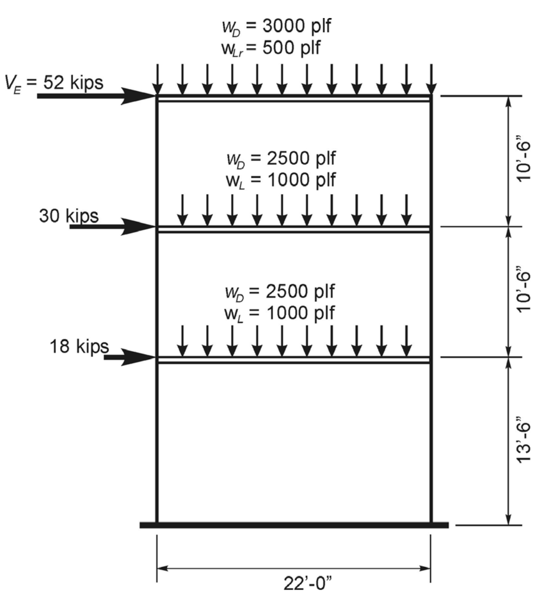

Figure 1 – Multi-Story Concrete Masonry Shear Wall

Solution:

The in-plane earthquake loads from the first floor are equal to:

$$V_E = 52 + 30 + 18 = 100 \text{ kips}$$
$$M_E = 52(34.5) + 30(24) + 18(13.5) = 2757 \text{ kip} - \text{ft}$$

Since the applied shear must be multiplied by 1.5 (UBC Section 2107.1.7), the earthquake loads for use with allowable stress design are as follows:

$$V = \frac{1.5V_E}{1.4} = \frac{1.5(100)}{1.4} = 107.1 \text{ kips}$$
$$M = \frac{M_E}{1.4} = \frac{2757}{1.4} = 1969 \text{ kip} - \text{ft}$$

The gravity loads at the first floor are equal to:

$$P_D = \frac{22(3000 + 2500 + 2500)}{1000} + \frac{78(22)(34.5)}{1000}$$
$$= 235 \text{ kips}$$
$$P_L = \frac{22(1000 + 1000)}{1000} = 44 \text{ kips}$$
$$P_{Lr} = \frac{22(500)}{1000} = 11 \text{ kips}$$

Assuming the cover to the reinforcing steel is 4 inches:

$$d = 12(22) - 4 = 260 \text{ in} = 21.67 \text{ ft}$$

The average shear stress in the wall is computed by Equation 7-37 of the 1997 UBC. From the Winter 2007 issue of *Masonry Chronicles, j* is equal to 0.924:

$$f_v = \frac{V}{bjd} = \frac{107.1(1000)}{(7.63)(0.924)(21.67 \text{x} 12)} = 58.4 \text{psi}$$
$$\frac{M}{Vd} = \frac{2757}{100(21.67)} = 1.27 > 1.0$$

The allowable shear considering the masonry alone is (UBC Section 2107.2.8):

$$1.33F_v = 1.33\sqrt{f'_m} = 1.33\sqrt{1500} = 51.5 \text{ psi}$$
$$= 1.33(35) = 46.6 \text{ psi} \leftarrow \text{governs}$$

Since the shear stress in the wall (58.4 psi) is greater than that allowed (46.6 psi), shear reinforcement is required. From UBC Equation 7-22, the shear stress in the wall must not exceed:

$$1.33F_v = 1.33(1.5)\sqrt{f'_m} = 77.2 \text{ psi} \leftarrow \text{governs}$$
$$< 1.33(75) = 99.8 \text{ psi}$$

The shear demand (58.4 psi) is less than 77.2 psi; a thicker wall does not need to be used. The horizontal reinforcing is now designed to resist all of the shear forces. Assuming #5 bars, the required spacing is obtained from UBC Equation 7-38:

$$s = \frac{A_v F_s d}{V} = \frac{0.31(32000)(260)}{107.1(1000)} = 24.1 \text{ in}$$

Therefore, we can use #5 bars at 24 inches on center to fit within the block module. From the Winter 2007 article of *Masonry Chronicles*, the vertical reinforcement was #5 bars at 16 inches on center. Since the building has been assigned to seismic zone 4, the wall must comply with detailing provisions dictated in Section 2106.1.12.4 of the UBC. This means that the reinforcement placement must satisfy the following requirements:

$$A_{s\,\min} \geq 0.2 \text{ in}^2 \therefore \text{OK}$$

$$s < 48 \text{ in} \therefore \text{OK}$$

$$s < \frac{l}{2} = \frac{22(12)}{2} = 132 \text{ in} \therefore \text{OK}$$

$$s < \frac{h}{2} = \frac{13.5(12)}{2} = 81 \text{ in} \therefore \text{OK}$$

Section 2106.1.12.4 of the 1997 UBC also states that the minimum area in each direction needs to be greater than 0.0007 times the cross-sectional area of the wall and the sum of the areas must exceed 0.002 times the cross-sectional area of the wall.

$$\left(\frac{A_s}{st}\right)_{hor} = \frac{0.31}{24(7.63)} = .0017 \geq .0007 \therefore \text{OK}$$

$$\left(\frac{A_s}{st}\right)_{ver} = \frac{0.31}{16(7.63)} = .0025 \geq .0007 \therefore \text{OK}$$

$$0.0017 + 0.0025 = 0.0042 \geq 0.002 \therefore \text{OK}$$

Consequently, the use of #5 bars at 24 inches on center for horizontal reinforcing, and #5 bars at 16 inches on center for vertical reinforcing, satisfies the spacing requirements.

Example 2:
Shear Resistance of Concrete Masonry Bearing Wall using the 2006 IBC Allowable Stress Design Procedures

Determine the horizontal reinforcement and minimum vertical steel requirements for the 8-inch thick fully grouted wall composed of medium-weight concrete masonry units (78 psf) as shown in Figure 1. The wall is located in a building assigned to seismic design category D. Masonry compressive strength is 1500 psi and the steel is Grade 60.

Solution:
The loads are identical to those defined in example 1.

Note that as required by the 2006 IBC, the shear force is multiplied by a factor of 1.5 (IBC 2106.5.1), which is identical to the 1997 UBC requirement. The IBC, by reference to the MSJC, refers to the use of different equations for determining the allowable shear on a wall, depending on whether or not a wall is subjected to flexural tension. Check flexural tension occurs using the load combination with the smallest axial load (.9D+E/1.4):

$$\begin{aligned} f &= \frac{P}{A_n} - \frac{M_E}{S_n} \\ &= \frac{0.9(235)(1000)}{7.63(22\text{x}12)} - \frac{(1969\text{x}12)(1000)6}{7.63(22\text{x}12)^2} \\ &= -162 \text{psi} \end{aligned}$$

Flexural tension exists in the wall. The average shear stress in the wall is equal to:

$$f_v = \frac{V}{bd} = \frac{107.1(1000)}{(7.63)(260)} = 54 \text{ psi}$$

$$\frac{M}{Vd} = \frac{2757}{100(21.67)} = 1.27 > 1.0$$

The allowable shear considering the masonry alone is (MSJC Equation 2-2):

$$\begin{aligned} 1.33F_v &= 1.33\sqrt{f'_m} = 1.33\sqrt{1500} = 51.5 \text{ psi} \\ &= 1.33(35) = 46.6 \text{ psi} \leftarrow \text{governs} \end{aligned}$$

Since the shear stress in the wall (54 psi) is greater than that allowed (46.6 psi), shear reinforcement is required. From MSJC Equation 2-25, the shear stress in the wall must not exceed:

$$\begin{aligned} 1.33F_v &= 1.33(1.5)\sqrt{f'_m} = 77.2 \text{ psi} \leftarrow \text{governs} \\ &< 1.33(75) = 99.8 \text{ psi} \end{aligned}$$

The shear demand (54 psi) is less than 77.2 psi; a thicker wall does not need to be used. The horizontal reinforcing is now designed to bear all of the shear forces. Assuming #5 bars, the required spacing is obtained from MSJC Equation 2-26:

$$s = \frac{A_v F_s d}{V} = \frac{0.31(32000)(260)}{107.1(1000)} = 24.1 \text{ in}$$

Therefore, we can use #5 bars at 24 inches on center to fit within the block module. From the Winter 2007 article of *Masonry Chronicles*, the vertical reinforcement was #4 at 32 inches on center. Since the building has been assigned to seismic design category D, the wall must comply with the special provisions dictated in Sections 1.14.2 and 1.14.6 of the MSJC. This means that the reinforcement placement must satisfy the following detailing requirements which differ slightly from those found in the 1997 UBC:

$A_{s\min} \geq 0.2$ in^2 $\therefore$ OK

$s < 48$ in $\therefore$ OK

$s < \frac{l}{3} = \frac{22(12)}{3} = 88$ in $\therefore$ OK

$s < \frac{h}{3} = \frac{13.5(12)}{3} = 54$ in $\therefore$ OK

Similar to the 1997 UBC, the minimum area in each direction needs to be greater than 0.0007 times the cross-sectional area of the wall and the sum of the areas must exceed 0.002 times the cross-sectional area of the wall.

$$\left(\frac{A_s}{st}\right)_{hor} = \frac{0.31}{24(7.63)} = .0017 \geq .0007 \therefore \text{OK}$$

$$\left(\frac{A_s}{st}\right)_{ver} = \frac{0.2}{32(7.63)} = .0008 \geq .0007 \therefore \text{OK}$$

$$0.0017 + 0.0008 = 0.0025 \geq 0.002 \therefore \text{OK}$$

Unlike the 1997 UBC, the IBC requires the minimum cross-sectional area of the vertical reinforcement needs to be at least one-third of the required shear reinforcing (MSJC Section 2.3.5.3.2):

$$\left(\frac{A_s}{st}\right)_{ver} \geq \frac{1}{3}\left(\frac{A_s}{st}\right)_{hor} = \frac{.0017}{3} = .00057 \therefore \text{OK}$$

Consequently, the use of #5 bars at 24 inches on center for horizontal reinforcing and #4 bars at 32 inches on center for vertical reinforcing is satisfactory.

Example 3:
Shear Resistance of Concrete Masonry Bearing Wall using the 1997 UBC Strength Design Procedures

The loads and geometry remain the same as shown in example 1.

Solution:

From example 1, the shear and moment demand on the wall is 100 kips and 2,757 kip-ft, respectively.

The calculation of shear strength of a wall using the 1997 UBC provisions varies depending on whether the wall is designed to yield in flexure or if a brittle failure is expected to occur. When the shear strength exceeds the shear that develops from the corresponding flexural strength, the wall is expected to yield in flexure, and only the shear steel can be expected to resist the shear forces in the plastic hinge zone. Outside the plastic hinge zone, contributions from both the steel and masonry can be calculated to resist the shear forces. Assuming that the wall will be designed to yield and respond in a ductile manner, the shear strength that develops from the corresponding nominal flexural strength (3,876 k-ft, as shown in the *Winter 2007 Masonry Chronicles* article) is as follows:

$$V_{ductile} = (100)\frac{3876}{2757} = 140.6 \text{ kips}$$

From Table 21-J of the 1997 UBC, the maximum nominal shear strength is:

$$V_{n\max} = 4A_e\sqrt{f'_m}$$
$$= 4(7.63)(22\text{x}12)\sqrt{1500}/1000$$
$$= 311.9 \text{ kips}$$

Since the shear demand (140.6 kips) is less than 311.9 kips, a thicker wall does not need to be used. The plastic hinge zone is defined as the region from the base to a distance of L_w above the base, where L_w is the length of the wall. However, the plastic hinge zone need not exceed half the story height. In this region, the wall's shear capacity relies solely on the shear reinforcement used. Note that per Section 2108.1.4.3.2 of the 1997 UBC, the ϕ factor changes from 0.6 to 0.8 in this region. Through the use of Equation 8-38 in the 1997 UBC and assuming #4 bars at 16 inches on center:

$$V_s = A_{mv}\rho_n f_y$$

Where:

$$\rho_n = \frac{A_s}{bd} = \frac{0.20}{(16)(7.63)} = 0.0016$$

Therefore:

$$\phi V_n = 0.8(260)(7.63)(.0016)(60) = 152 \text{ kips}$$

$$\phi V_n = 152 \text{ kips} \geq 140.6 \text{ kips} \therefore \text{OK}$$

The spacing of bars in the first zone cannot exceed 3 times the thickness of the wall or 24 inches.

In the rest of the wall, the shear strength is calculated by summing the steel and masonry contributions. The nominal shear strength of the wall can be found through Equations 8-36, 8-37, and 8-38 in the 1997 UBC (Section 2108.2.5.5):

$$V_n = V_m + V_s$$

Where:

$$V_m = C_d A_{mv}\sqrt{f'_m}$$
$$V_s = A_{mv}\rho_n f_y$$

The nominal shear coefficient (C_d) is found in Table 21-K of the 1997 UBC to be equal to 1.2. Now, the strength contributed by the masonry alone is equal to:

$$V_m = 1.2(22\text{x}12)(7.63)\sqrt{1500} = 93.6 \text{ kips}$$
$$\phi V_m = 0.6(93.6) = 56.2 \text{ kips}$$

Since the shear demand is greater than the nominal strength provided by the masonry alone, horizontal shear reinforcing is required. A capacity reduction factor of 0.6 is used in accordance with UBC Section 2108.1.4.3.2. At a minimum, the horizontal reinforcing needs to be designed to account for any of the shear force that exceeds the shear capacity of the concrete masonry. The required steel contribution will be:

$$V_{s\ req} = \frac{V_u - \phi V_m}{\phi}$$
$$V_{s\ req} = \frac{140.6 - 56.2}{0.6} = 140.7 \text{ kips}$$
$$\rho_n = \frac{V_{s\ req}}{A_{mv} f_y} = \frac{140.7}{(7.63)(260)(60)} = .0012$$

If we use #4 bars at 16 inches on center, we have the following:

$$\rho_n = \frac{A_s}{bd} = \frac{0.2}{16(7.63)} = .0016 \geq .0012 \therefore \text{OK}$$

From the Winter 2007 issue of *Masonry Chronicles*, #4 bars at 16 inches on center is sufficient to resist the in-plane flexural demands. Since the building has been assigned to seismic zone 4, the wall must comply with the special provisions dictated in Section 2106.1.12.4 of the UBC. The spacing for the second shear region is used, since that spacing governs the detailing requirements. This means that the reinforcement placement must satisfy the following requirements:

$$A_{s\min} \geq 0.2 \text{ in}^2 \therefore \text{OK}$$
$$s < 48 \text{ in} \therefore \text{OK}$$
$$s < \frac{l}{2} = \frac{8(12)}{2} = 48 \text{ in} \therefore \text{OK}$$
$$s < \frac{h}{2} = \frac{13.5(12)}{2} = 81 \text{ in} \therefore \text{OK}$$

The minimum area in each direction needs to be greater than 0.0007 times the cross-sectional area of the wall and the sum of the areas must exceed 0.002 times the cross-sectional area of the wall.

$$\left(\frac{A_s}{st}\right)_{hor} = \frac{0.20}{16(7.63)} = .0016 \geq .0007 \therefore \text{OK}$$
$$\left(\frac{A_s}{st}\right)_{ver} = \frac{0.2}{16(7.63)} = .0016 \geq .0007 \therefore \text{OK}$$
$$0.0016 + 0.0016 = 0.0032 \geq 0.002 \therefore \text{OK}$$

Finally, the minimum cross-sectional area of the vertical reinforcement needs to be at least one-half of the required shear reinforcing, #4 @ 16" O.C. (UBC Section 2108.2.5.2):

$$\left(\frac{A_s}{st}\right)_{ver} = .0016 \geq \frac{1}{2}\left(\frac{A_s}{st}\right)_{hor} = .0008 \therefore \text{OK}$$

Therefore, the use of #4 bars at 16 inches on center for horizontal reinforcing in the plastic hinge region, #4 bars at 32 inches on center in the second shear region, and #4 bars at 16 inches on center for vertical reinforcing is permissible.

Example 4:
Shear Resistance of Concrete Masonry Bearing Walls using the 2006 IBC Strength Design

The loads and geometry remain the same as shown in example 2.

Solution:

From example 2, the shear and moment demand on the wall is 100 kips and 2,757 kip-ft, respectively.

According to Section 3.1.3 of the 2005 MSJC, the design shear strength is required to be at least 1.25 times the shear developed by the flexural strength. This is limited by an upper bound of 2.5 times the required shear strength. The nominal flexural strength of this wall was found to be 3,071 k-ft in the Winter 2007 article of *Masonry Chronicles*. As a result, the shear demand of a ductile wall will be the lesser of:

$$V_{ductile} = (1.25)\frac{3071}{2757} = 139 \text{ kips} \leftarrow \text{governs}$$
$$2.5V_E = 250 \text{ kips}$$

From Section 3.3.4.1.2 of the MSJC the maximum nominal shear strength of the wall is not to exceed:

$$V_n \leq 4A_n\sqrt{f'_m} = 4(7.63)(260)\sqrt{1500}$$
$$= 307 \text{ kips}$$

Since shear demand (139 kips) is less than 307 kips, a thicker wall does not need to be used. The plastic hinge zone defined by Section 2106.5.2 of the 2006 IBC is identical to provisions found in the 1997 UBC. In this zone, the shear strength is defined as (IBC Equation 21-1):

$$V_n = V_s = A_n \rho_n f_y$$

In the first shear region, the wall's shear capacity relies solely on the shear reinforcement used:

$$\rho_n = \frac{V_n}{\phi A_n f_y} = \frac{139}{0.8(7.63)(260)(60)} = .0015$$

If we use one #4 bar at 16 inches on center we have the following:

$$\rho_n = \frac{A_s}{bd} = \frac{0.2}{7.63(16)} = .0016 \geq .0015 \therefore \text{OK}$$

The shear strength of the rest of the wall can be determined by summing the individual shear resistance of the steel and masonry components. The nominal shear strength of the wall can be found through MSJC Equations 3-18, 3-21, and 3-22:

$$V_n = V_m + V_s$$
$$V_m = \left[4.0 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f'_m} + .25 P_u$$

$$V_s = 0.5\left(\frac{A_v}{s}\right) f_y d_v$$

where the term $\left(\frac{M_u}{V_u d_v}\right)$ need not be > 1

As a result, the shear strength of the masonry alone can be shown to be:

$$V_m = [4 - 1.75(1)](260)(7.63)\sqrt{1500} + .25(235)$$
$$= 232 \text{ kips}$$
$$\phi V_n = 0.8(232) = 185.6 \text{ kips}$$

Since the shear demand is less than the strength provided by the masonry alone, horizontal shear reinforcing is not required. Nevertheless, unreinforced masonry is not permitted in high seismic regions. As a result, shear reinforcing will be placed in accordance with the detailing requirements found in Sections 1.14.2 and 1.14.6 of the MSJC.

From the winter 2007 issue of *Masonry Chronicles*, #4 bars at 32 inches on center is sufficient to resist the in-plane load demands. With this information, the minimum reinforcement requirements (MSJC Sections 1.14.2 and 1.14.6) can be checked as follows:

$$A_{s\min} \geq .2 \text{ in}^2 \therefore \text{ OK}$$
$$s < 48 \text{ in} \therefore \text{ OK}$$
$$s < \frac{l}{3} = \frac{8(12)}{3} = 32 \text{ in} \therefore \text{OK}$$
$$s < \frac{h}{3} = \frac{13.5(12)}{3} = 54 \text{ in} \therefore \text{OK}$$

The minimum area in each direction needs to be greater than 0.0007 times the cross-sectional area of the wall and the sum of the areas must exceed 0.002 times the cross-sectional area of the wall.

$$\left(\frac{A_s}{st}\right)_{hor} = \frac{0.20}{16(7.63)} = .0016 \geq .0007 \therefore \text{OK}$$
$$\left(\frac{A_s}{st}\right)_{ver} = \frac{0.20}{32(7.63)} = .0008 \geq .0007 \therefore \text{OK}$$
$$0.0016 + 0.0008 = 0.0024 \geq 0.002 \therefore \text{OK}$$

Note that if more vertical steel was required, the use of less steel reinforcement in the second shear region would have been possible.

Finally, the minimum cross-sectional area of the vertical reinforcement needs to be at least one-third of the required shear reinforcing (MSJC Section 2.3.5.3.2):

$$\left(\frac{A_s}{st}\right)_{ver} = .0008 \geq \frac{1}{3}\left(\frac{A_s}{st}\right)_{hor} = .0005 \therefore \text{OK}$$

Now, the use of #4 bars at 32 inches for vertical reinforcing, #4 bars at 16 inches on center for horizontal reinforcing in the plastic hinge region, and #4 bars at 16 inches in the second shear region is acceptable.

Conclusions

Provisions for the allowable stress design of walls to resist in-plane shear forces are similar in the 1997 UBC and the 2006 IBC. Looking at the results of examples 1 and 2 (also shown in Table 1), we see that identical results are achieved with both building codes.

Building Code and Design Methodology	Shear Steel Reinforcement
‘97 UBC - ASD	#5 @ 24” O.C.
‘06 IBC - ASD	#5 @ 24” O.C.
‘97 UBC - Strength	#4 @ 16” O.C.
‘06 IBC - Strength	#4 @ 16” O.C.

Table 1 – Horizontal Steel Reinforcement Requirements

Shear design, in accordance with strength design methodologies of the 1997 UBC and 2006 IBC, also yielded similar results (See Table 1). Although requirements regarding the use of the two shear regions are identical, there were variations in the design forces as well as the way in which the steel contribution is accounted for. Note that the in-plane flexural design of the UBC wall, #4 bars at 16 inches were required, whereas the use of #4 bars at 32 inches on center were sufficient for IBC strength design.

Shear capacity from the strength of the masonry alone was significantly higher in the IBC, which may result in less steel reinforcing away from the plastic hinge zone.

Provided that the flexural designs for the UBC and IBC were similar, it is likely that the IBC case would have less shear reinforcing in the 2^{nd} region. It is currently shown to be #4 at 16 inches on center in order to satisfy detailing requirements.

In allowable stress, as well as strength design, the spacing requirements are slightly different in the IBC. In the UBC, the amount of vertical reinforcing used needs to be at least half the area of the horizontal reinforcing. However, the IBC stipulates that the amount of vertical reinforcing shall be at least one third the area of the horizontal reinforcing. Additionally, tighter spacing of rebars is required by the IBC. According to the commentary of the 2005 MSJC, these requirements are imposed in order to improve ductile behavior under earthquake loading.

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

References

[1] International Conference of Building Officials (ICBO), 1997 Uniform Building Code, International Conference of Building Officials, Whittier, California, 1997.

[2] International Code Council (ICC), 2006 International Building Code, International Code Council, Inc., Falls Church, Virginia, 2006

[3] Masonry Standards Joint Committee (MSJC), Building Code Requirements for Masonry Structures, Masonry Standards Joint Committee, Boulder, Colorado, 2005

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