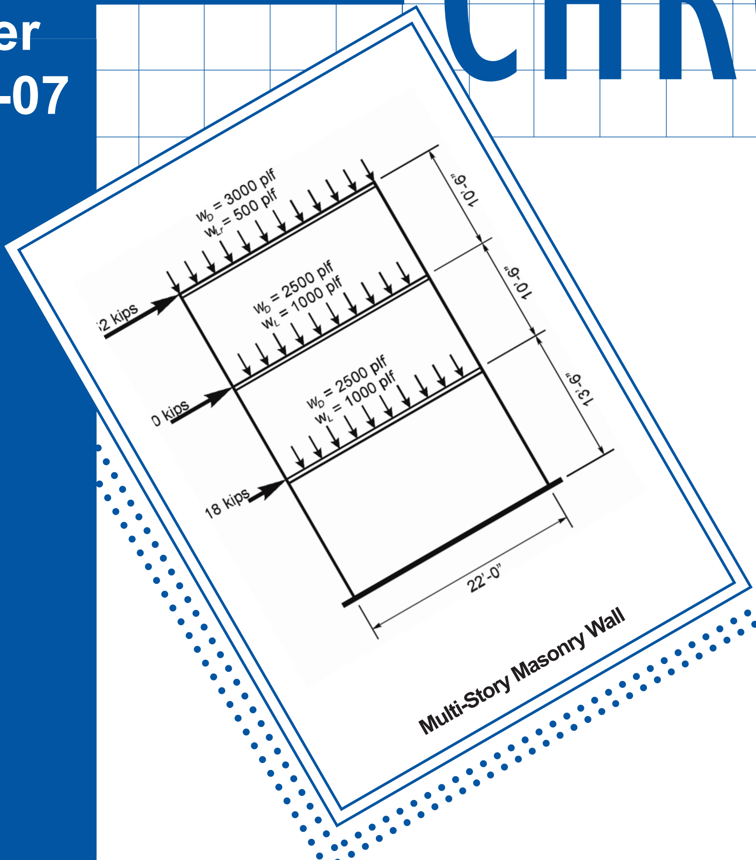
Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Winter 2006-07



Comparison of the 2006 IBC and 1997 UBC Provisions for the Design of Shear Walls Subjected to In-Plane Axial and Flexural Loads

Introduction

Even with the introduction of strength design procedures, concrete masonry is still often designed according to the elastic theories of allowable stress design. This design methodology incorporates several major assumptions: plane sections remain plane, stresses remain in the elastic range, the modulus of elasticity remains constant, and the tensile strength of masonry is neglected, among others. Ultimately, stresses calculated in the materials must not exceed code prescribed allowable stresses.

Columns, bearing walls, and shear walls are often subjected to flexural and axial loads at the same time. Compressive stresses may be induced by the gravity dead and live loads while bending stresses may result from lateral wind or seismic forces, from lateral soil pressures, or from the eccentric application of gravity loads. In the design of concrete masonry members subjected to flexural and axial loads, the unity equation has often been used. However, this technique has come under scrutiny, since it is not completely accurate and can sometimes lead to flawed designs.

Strength design procedures are based on the mechanics of the individual components (steel and concrete masonry) working together to achieve sufficient strength to endure specified loadings due to dead, live, and lateral loads. In strength design, strains in the materials are no longer limited to the elastic range. This results in designs which are able to utilize the ductility and strength, which can often be found in the inelastic range of material response, thereby allowing more effective and economical designs.

The 1997 Uniform Building Code (UBC) [Ref.1] allowed for the unity equation to be used in the design of reinforced masonry structures subjected to combined flexural and axial loading. In the 2006 International Building Code (IBC) [Ref. 2], this is no longer permitted for reinforced concrete masonry construction. Currently the 2006 IBC has been adopted or is in the process of being adopted in jurisdictions throughout the country. For masonry design, this code references the ACI 530-05/ASCE 5-05/TMS402-05 [Ref. 3], which is also referred to as the 2005 Masonry Standards Joint Committee Building Code (MSJC).

The Unity Equation

Historically, the unity equation has been used to design concrete masonry members subjected to combined axial and flexural stresses. In lieu of procedures that strictly adhere to principles of mechanics, the 1997 Uniform Building Code (UBC) permitted the use of the following equation:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1 \qquad \text{Equation 1}$$

where f_a is the computed compressive stress due to axial loads only, and f_b is the computed compressive stress due to flexural loads alone. F_a and F_b are the allowable compressive stresses for axial and flexural loads, respectively.

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When using Equation 1, the axial and flexural stresses are calculated independently. This approach is technically incorrect for reinforced concrete masonry. since superposition is not an accurate procedure for analyzing cracked members subjected to combined stresses. This is due to the fact that the location of the neutral axis varies with the amount of cracking, and thus the stress distribution and cross-section properties are different for various values of axial load. The MSJC code requirements only provide the unity equation for unreinforced masonry, and the MSJC commentary specifically states that the unity equation is not applicable to reinforced masonry. However, in some instances the unity equation may provide a quick and conservative solution that is very useful for developing preliminary designs. Designers using the unity equation should verify that the unity equation does provide a conservative solution.

Since the use of the unity equation is no longer allowed in the 2006 IBC, new tools for allowable stress design procedures need to be developed. Figures 1 and 2 provide a comparison of the unity equation and interaction diagrams obtained for singly reinforced cross-sections with no compression steel. The interaction diagrams were developed using principles of mechanics as required by the 2006 IBC. The nomenclature for the interaction diagrams is as follows:

- P = Factored Axial load
- M = Moment
- b = width under consideration
- d = distance to steel reinforcement from edge
- h = depth in direction of loading
- $n = E_s/E_m$
- F_s = Allowable Tensile Stress in Steel Reinforcing
- F_{a+b} = Allowable Compressive Stress in Masonry due to combine axial and flexural loads

The upcoming 2006 Design of Reinforced Masonry Structures (DORMS), published by CMACN, will contain similar diagrams for use in the design of cross-sections with a wide range of reinforcement ratios and axial loads.

As can be seen from the interaction diagrams, the unity equation is generally conservative at low reinforcement ratios. However, the unity equation may slightly over-estimate the allowable moment on highly reinforced cross sections.

Examples will be provided to illustrate the difference between the calculation methods for combined axial and flexural loading. The same problem will be worked four separate ways to demonstrate the effects of the new code and its implications. 1997 UBC prescriptions for allowable stress and strength design, as well as the 2006 IBC provisions for allowable stress and strength design, will be utilized. Only in-plane design will be considered in these examples. Out-of-plane issues will be dealt with in a future issue of *Masonry Chronicles.*

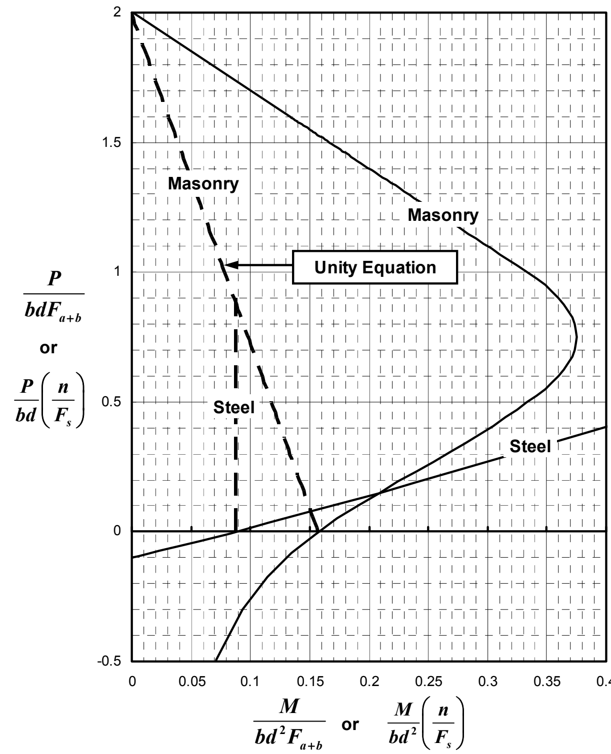

Figure 1 – Comparison of Interaction Diagram with Unity Equation for np = 0.1 and h/d = 2.0

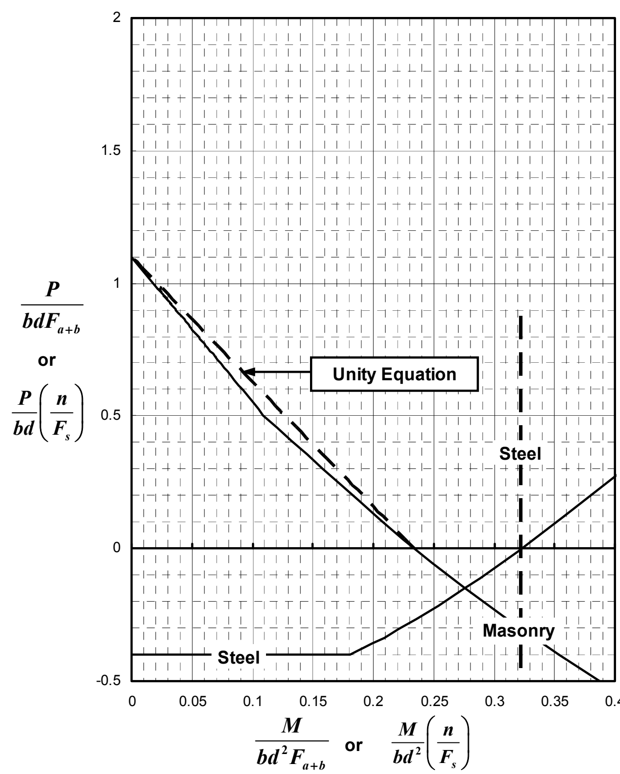

Figure 2 - Comparison of Interaction Diagram with Unity Equation for $n\rho$ = 0.4 and h/d = 1.1

Example 1 – Design of a Multistory Concrete Masonry Shear Wall (In-Plane) Using IBC Allowable Stress Design

Determine the horizontal and vertical steel required at the first story for the wall with the earthquake loads shown in Figure 3. The wall is constructed with 8-inch medium-weight concrete masonry units. The specified masonry compressive strength is 1500 psi and Grade 60 steel is used as reinforcement. S_{DS} = 2.2g. Use the alternative load combinations in the IBC (A one-third increase in allowable stresses is permitted).

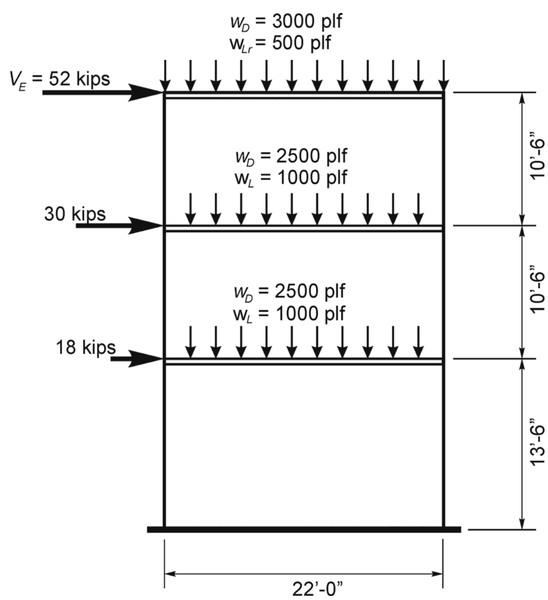

Figure 3 - Multi-Story Masonry Wall

Solution:

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

$$V_E = 52 + 30 + 18 = 100 \text{ kips}$$

$$M_E = 52(34.5) + 30(24) + 18(10.5) = 2703 \text{ kip-ft}$$

Since the applied shear must be multiplied by 1.5 (IBC 2106.5.1), the earthquake loads for use with allowable stress design are as follows:

$$V = \frac{1.5V_E}{1.4} = \frac{1.5(100)}{1.4} = 107.1 \text{ kips}$$

$$M = \frac{M_E}{1.4} = \frac{2703}{1.4} = 1931 \text{ kip-ft}$$

The gravity loads at the first floor are:

$$P_D = \frac{22(3000 + 2500 + 2500)}{1000} + \frac{78(22)(34.5)}{1000}$$
$$= 235 \text{ kips}$$

$$P_L = \frac{22(1000 + 1000)}{1000} = 44 \text{ kips}$$

$$P_{Lr} = \frac{22(500)}{1000} = 11 \text{ kips}$$

Assuming uncracked properties, check for flexural tension for the load combination 0.9*D* + *E*/1.4:

$$\frac{P}{A_n} - \frac{M}{S_n} = \frac{0.9(235)1000}{(7.63)(22 \times 12)} - \frac{(1931 \times 12)(1000)6}{(7.63)(21.67 \times 12)^2}$$
$$= -165 \text{ psi}$$

Flexural tension exists, so the shear stress in the wall is given by:

$$f_v = \frac{V}{bd} = \frac{(107.1)(1000)}{(7.63)(21.67 \times 12)} = 54 \text{ psi}$$

$$\frac{M}{Vd} = \frac{2703}{100(21.67)} = 1.25 > 1.0$$

Therefore, the allowable shear considering the masonry alone is (MSJC 2.3.5.2.2):

$$F_v = 1.33\left(1.0\sqrt{f'_m}\right) = 1.33\sqrt{1500} = 51.5 \text{ psi}$$
$$= 1.33(35) = 46.6 \text{ psi} \leftarrow \text{governs}$$

The allowable shear stress using masonry alone is less than the demand, therefore shear reinforcement is required. Check to see if the wall needs to be made thicker (MSJC 2.3.5.2.3):

$$F_v = (1.5\sqrt{f'm})1.33 = 77.2 \leftarrow \text{governs} > f_v \text{ ...\underline{OK}}$$
$$= 75(1.33) = 99.8 \text{ psi}$$

Assuming #5 bars, the required spacing is as follows (MSJC 2.3.5.3):

$$s = \frac{A_v F_s d}{V} = \frac{0.3(32000)(21.67 \times 12)}{107.1 \times 1000} = 24.1 \text{ in}$$

We can use #5 bars spaced at 24 inches on center. Then (MSJC Section 1.14):

$$\left(\frac{A_s}{st}\right)_{hor} = \frac{0.31}{24(7.63)} = 0.0017 > 0.0007 \text{ ...\underline{OK}}$$

The horizontal reinforcement must be at least one-third the vertical reinforcement (MSJC Section 2.3). We can try #4 bars at 32 inches on center:

$$A_s = 0.20 \text{ in}^2 \geq A_{s,\min} = 0.2 \text{ in}^2 \text{ ...\underline{OK}}$$

$$s = 32 \text{ in} < 48 \text{ in ...\underline{OK}}$$

$$s << \frac{h}{3}; \frac{L}{3} \text{ ...\underline{OK}}$$

$$\left(\frac{A_s}{st}\right)_{ver} = \frac{0.20}{32(7.63)} = 0.0008 > 0.0007 \text{ ...\underline{OK}}$$

$$\left(\frac{A_s}{st}\right)_{hor} + \left(\frac{A_s}{st}\right)_{ver} = 0.0017 + 0.0008 = 0.0025$$
$$> 0.002 \text{...\underline{OK}}$$

$$\left(\frac{A_s}{st}\right)_{ver} > \frac{1}{3}\left(\frac{A_s}{st}\right)_{hor} = \frac{0.0017}{3} = 0.00056 \text{ ...\underline{OK}}$$

Next, we will check if the vertical steel is adequate to resist the in-plane flexural loads by considering all the steel distributed within the cross-section. This is achieved by an iteration process that involves the following steps:

1. Assume the location of the neutral axis by guessing at the depth of the compression block, *kd.*

2. Assuming that allowable moment is determined by the masonry allowable compressive stress, the strain in the extreme compression fiber is equal to:

   $$\varepsilon_{m,\max} = \frac{F_{a+b}}{E_m}$$

   The total compression force in the masonry is equal to:

   $$C_m = \frac{1}{2} F_{a+b} bkd$$

   and its reaction is located at a distance *kd*/3 from the extreme compression fiber.

3. With the maximum compressive strain, ε_m and assumed neutral axis location, determine the strain in each reinforcing bar using similar triangles. The strain in each bar is given by:

   $$\varepsilon_{si} = \frac{\varepsilon_m (d_i - kd_i)}{kd}$$

   where d_i is the distance of the bar from the extreme compression fiber.

4. Calculate the stress f_s, and corresponding force, T, in each reinforcing bar from the calculated strains using the following equations:

   $$f_{si} = \varepsilon_{si} E_s \quad ; \quad T_{si} = f_{si} A_{si}$$

   Note that reinforcing subjected to compressive strains will have no force if lateral tie reinforcement is not provided.

5. Verify that the stress in all reinforcing bars is less than the allowable steel stress, F_s. If the stress in any reinforcing bar is greater than F_s, the allowable moment is governed by the allowable steel stress. The tensile strain in the extreme tension fiber is given by:

   $$\varepsilon_{s,\max} = \frac{F_s}{E_s}$$

   The strain in each bar and masonry compressive stress should be recalculated using the assumed neutral axis location and the following equations:

   $$\varepsilon_{si} = \frac{\varepsilon_{s,\max} (d_i - kd_i)}{(d - kd)}$$

   $$\varepsilon_m = \frac{F_s}{E_s} \frac{kd}{(d - kd)}$$

   $$F_{a+b} = \varepsilon_m E_m \quad ; \quad C_m = \frac{1}{2} f_{a+b} bkd$$

6. Sum all the compressive and tensile forces in the masonry and steel. If the sum of forces is equal to the applied axial load (within about 5%) the assumed neutral axis location is correct. If the sum of forces is less than the applied axial compression load, the depth of the neutral axis needs to be increased and steps 1 through 6 repeated. Similarly, if the sum of the compressive forces is larger than the applied loads, the depth of the neutral axis should be reduced.

7. When satisfactory convergence between the applied and calculated axial load is achieved, calculate moments about any point on the cross-section to determine the allowable moment.

The above iteration process is best performed using a computer program or spreadsheet to simplify the process. Figure 4 illustrates a converged solution obtained using a spreadsheet. Moments are taken about the center of the cross-section and the allowable moment is equal to:

$$M = \frac{24758}{12} = 2063 \text{ kip-ft} > \frac{M_E}{1.4} = 1931 \text{ kip-ft} \;..\underline{\text{OK}}$$

Therefore, #5 bars spaced horizontally at 24 inches on center and vertical reinforcement of #4 bars spaced at 32 inches on center is sufficient to resist the in-plane and out-of-plane loads demands, and satisfies the minimum detailing requirements.

<u>Material Properties</u>

Strength of Masonry, f'_m =	**1500**	psi
Yield Stress of Reinforcing Steel, f_y =	**60**	ksi
Modulus of Elasticity of Masonry, E_m =	1350	ksi
Modulus of Elasticity of Steel, E_s =	29000	ksi

<u>Section Properties</u>

Width, b =	**7.625**	in
Total Depth, h =	**264**	in
Allowable Masonry Stress =	**667**	psi
Assumed Neutral Axis Depth, kd =	**90.112**	in

	A (in^2)	*d* (in)	Lateral Ties?	*e* (in/in)	*Fs* (psi)	*C* (kips)	*M* (kip-in)
Masonry		30.04		0.00049		229.15	23364.75
Bar 1	0.20	4	N	0.00047	-	-	-
Bar 2	0.20	36	N	0.00030	-	-	-
Bar 3	0.20	68	N	0.00012	-	-	-
Bar 4	0.20	100	N	-0.00005	-1572	-0.31	-10.06
Bar 5	0.20	132	N	-0.00023	-6660	-1.33	-
Bar 6	0.20	164	N	-0.00041	-11748	-2.35	75.19
Bar 7	0.20	196	N	-0.00058	-16836	-3.37	215.51
Bar 8	0.20	228	N	-0.00076	-21925	-4.38	420.95
Bar 9	0.20	260	N	-0.00093	-27013	-5.40	691.52
						212.00	24757.86
							2063.2

Figure 4 - Calculation of Allowable Moment using Distributed Steel (2006 IBC, Allowable Stress Design)

Example 2 – In-Plane Flexural Design Using the 1997 UBC Unity Equation and Considering Only Jamb Steel

Solution:

First we must determine the allowable compressive stress due to axial loads alone. Since the wall is supported laterally at each floor level, the effective height for axial loads is given by:

$$h = 13.5(12) = 162 \text{ in}$$

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

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

and:

$$\frac{h}{r} = \frac{162}{2.2} = 73.6 < 99$$

Therefore per UBC requirements (2107.2.5):

$$F_a = 0.25(1500)\left[1 - \left(\frac{73.6}{140}\right)^2\right] = 271 \,\text{psi}$$

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

$$f_a = \frac{10165}{7.63(12)} = 111 \text{ psi}$$

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:

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

$$= 500\left(1.33 - \frac{111}{271}\right) = 460 \quad \text{psi}$$

For the in-plane moments, assuming the center of gravity of the jamb steel is 8 inches from the wall end:

$$d = 22(12) - 8 = 256 \text{ in}$$

$$K = \frac{M}{bd^2} = \frac{(2703 \times 12)1000}{1.4(7.63)(256)^2} = 46.3 \text{ psi}$$

From 210.7.2.15 of the 1997 UBC:

$$\left(\frac{1}{n\rho j}\right) = \frac{F_s}{n}\left(\frac{1}{K}\right) = \frac{32{,}000}{21.5}\left(\frac{1}{46.3}\right) = 32.1$$

$$n\rho = 0.034;\; k = 0.228;\; j = 0.924 \leftarrow \text{governs}$$

$$\left(\frac{2}{jk}\right) = \frac{F'_b}{K} = \frac{460}{46.3} = 9.945$$

$$n\rho = 0.030;\quad k = 0.217;\quad j = 0.928$$

Therefore:

$$A_{s,req} = \frac{n\rho bd}{n} = \frac{.034(7.63)(256)}{21.5} = 3.09 in^2$$

We can use 4-#8 (A_s = 3.16 in^2) bars at each end of the wall to resist the in-plane flexural demands. #5 bars @ 12 inches on center will be required to resist the out-of-plane moments. The method in which this is calculated will be detailed in the next edition of *Masonry Chronicles*.

Example 3 – In-Plane Flexural Design using 2006 IBC Strength Design

Solution:

Earthquake and gravity loads are the same as those found in the previous examples.

Assuming #4 @ 32 inches on center, we will check if the vertical steel is adequate to resist the in-plane flexural loads. This is achieved by an iteration process very similar to the process described in Example 1. The difference in the iteration process is found in the calculation of the compression force in steps 2 and 5. Additionally, steel stress is not allowed to exceed 60 ksi.

Replace the compression equations with the following:

$$C = 0.8 f'_m\, ab \quad where \quad a = 0.8c$$

The iteration process is best performed using a computer program or spreadsheet to simplify the process. Figure 5 illustrates a converged solution obtained using a spreadsheet.

Material Properties

Masonry Strain =	0.0025	
Strength of Masonry, f'_m =	1500	psi
Yield Stress of Reinforcement, f_y =	60	ksi
b_c =	0.8	

Cross-Section Properties

Width =	7.625	in
Total Depth =	264	in
Wall Axial Load =	211.5	kips
Assumed Neutral Axis Depth	40.11	in

	A (in^2)	Lateral Ties?	d (in)	e_s (in/in)	f_s (ksi)	F_s (kips)	M (kip-in)
Masonry			16.043	0.0025		293.6	34043.2
Bar 1	0.2	n	4	0.0023	0.0	0.0	0.0
Bar 2	0.2	n	36	0.0003	0.0	0.0	0.0
Bar 3	0.2	n	68	-0.0017	-50.4	-10.1	-645.4
Bar 4	0.2	n	100	-0.0037	-60.0	-12.0	-384.0
Bar 5	0.2	n	132	-0.0057	-60.0	-12.0	0.0
Bar 6	0.2	n	164	-0.0077	-60.0	-12.0	384.0
Bar 7	0.2	n	196	-0.0097	-60.0	-12.0	768.0
Bar 8	0.2	n	228	-0.0117	-60.0	-12.0	1152.0
Bar 9	0.2	n	260	-0.0137	-60.0	-12.0	1536.0
	1.8					211.50	36854
						Converged	
							3071.15

Figure 5 - Calculation of Allowable Moment using Distributed Steel (2006 IBC, Strength Design)

After convergence has been achieved, moments are taken about the center of the cross-section, and the allowable moment is equal to (Φ = 0.9):

$$M_n = 36854 \;/ 12 = 3071 \text{ k'}$$

$$M_u = \phi M_n = 0.9(3071) = 2764\text{k'} > 2703\text{k'} \;\; \therefore \underline{\text{OK}}$$

Therefore, #4 bars at 32 inches on center is sufficient to resist the in-plane loads demands.

Example 4 – In-Plane Flexural Design using 1997 UBC Strength Design Procedures

Solution:

Earthquake and gravity loads are the same as those found in the first example.

Assuming #4 @ 16 inches on center, we will check if the vertical steel is adequate to resist the in-plane flexural loads. This is achieved by an iteration process very similar to the process described in Example 1. The only difference in the iteration process is found in the calculation of the compression force in steps 2 and 5. Additionally, steel stress is not allowed to exceed 60 ksi. Replace the compression equations with the following:

$$C = 0.85 f'_m ba$$
$$where \;\; a = 0.85d\{\varepsilon_{mu} / [\varepsilon_{mu} + (f_y / E_s)]\}$$

The strength reduction factor is calculated in accordance with 2108.1.4.3.1 from the 1997 UBC. For this instance it has been calculated to be ϕ= 0.79. The iteration process is best performed using a computer program or spreadsheet to simplify the process. Figure 6 illustrates a converged solution obtained using a spreadsheet. After convergence has been achieved, moments are taken about the center of the cross-section and the allowable moment is equal to:

$$M_n = 46513/12 = 3876k'$$

$$M_u = \phi M_n = 0.79(3876) = 3062k' > 2703k' \;\; \therefore \underline{\text{OK}}$$

Therefore, #4 bars spaced at 16 inches on center is sufficient to resist the in-plane loads demands.

Since the design of this wall is governed by flexure, the UBC stipulates that the flexural strength of the wall should be at least 1.8 times greater than the cracking moment strength.

$$f_r = 4.0\sqrt{f'_m} = 4.0(\sqrt{1500}) = 154.9\,psi$$
$$M_{cr} = Sf_r = \frac{7.625(264)^2}{6}(.155)\frac{1}{12} = 1144k - ft$$
$$1.8M_{cr} = 2059k - ft < M_n = 3876k - ft \;\; \therefore O.K.$$

Material Properties

Masonry Strain =	0.0030	
Strength of Masonry, f'_m =	1500	psi
Yield Stress of Reinforcement, f_y =	60	ksi
b_c =	0.85	

Cross-Section Properties

Width =	7.625	in
Total Depth =	264	in
Wall Axial Load =	211.5	kips
Assumed Neutral Axis Depth	44.49	in

	A (in^2)	Lateral Ties?	d (in)	e_s (in/in)	f_s (ksi)	F_s (kips)	M (kip-in)
Masonry			18.908	0.0030		367.6	41576.6
Bar 1	0.2	n	4	0.0027	0.0	0.0	0.0
Bar 2	0.2	n	20	0.0017	0.0	0.0	0.0
Bar 3	0.2	n	36	0.0006	0.0	0.0	0.0
Bar 4	0.2	n	52	-0.0005	-14.7	-2.9	-235.0
Bar 5	0.2	n	68	-0.0016	-46.0	-9.2	-588.5
Bar 6	0.2	n	84	-0.0027	-60.0	-12.0	-576.0
Bar 7	0.2	n	100	-0.0037	-60.0	-12.0	-384.0
Bar 8	0.2	n	116	-0.0048	-60.0	-12.0	-192.0
Bar 9	0.2	n	132	-0.0059	-60.0	-12.0	0.0
Bar 10	0.2	n	148	-0.0070	-60.0	-12.0	192.0
Bar 11	0.2	n	164	-0.0081	-60.0	-12.0	384.0
Bar 12	0.2	n	180	-0.0091	-60.0	-12.0	576.0
Bar 13	0.2	n	196	-0.0102	-60.0	-12.0	768.0
Bar 14	0.2	n	212	-0.0113	-60.0	-12.0	960.0
Bar 15	0.2	n	228	-0.0124	-60.0	-12.0	1152.0
Bar 16	0.2	n	244	-0.0135	-60.0	-12.0	1344.0
Bar 17	0.2	n	260	-0.0145	-60.0	-12.0	1536.0
	3.4					211.50	46513
						Converged	
							3876.086

Figure 6 - Calculation of Allowable Moment using Distributed Steel (1997 UBC, Strength Design)

Conclusions

Figure 7 shows the solutions obtained with different methods and codes. Note that, for brevity, not all steps required for the complete design of the walls were included in this paper. The walls were checked for out-of-plane forces, as well as the maximum reinforcement ratio requirements of both the 2006 IBC and the 1997 UBC. In addition, the shear requirements for both codes were satisfied. 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. As a result, the unity equation requires significantly more reinforcement to resist the in plane flexural demands. In the 2006 IBC this method of accounting for combined axial and flexural loads is no longer permitted for reinforced concrete masonry.

The 2006 IBC provisions result in designs that require significantly less vertical reinforcement. It is able to accomplish this by more accurately representing the mechanics of reinforced concrete masonry under externally applied loadings. Without the use of the unity equation, the results in allowable stress design was very similar to the strength design case – as seen in examples 1 and 3 (2006 IBC ASD and Strength Design).

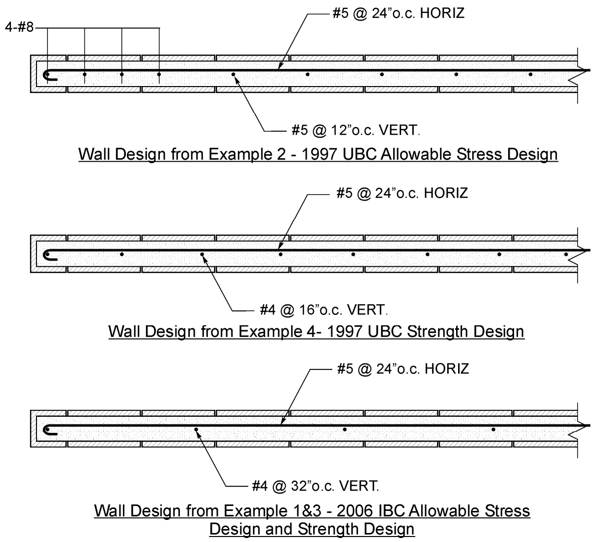

Figure 7 – Calculation of Required Vertical Reinforcement

When utilizing strength design in the 1997 UBC we obtain less conservative solutions than designs using the unity equation. Our results were in line with this expectation as example 4 (UBC Strength Design) yielded a slightly less conservative design when compared to example 2 (Unity Equation). The capacity reduction factor, Φ, prescribed for 97 UBC strength design for in-plane wall loads is 0.79. However, in the IBC, this factor is 0.9, which accounts for the large discrepancy between examples 3 and 4.

The wall design utilizing the design provisions in the 2006 IBC results in less conservative, yet structurally sound and acceptable results than those found in the 1997 UBC requirements. In these examples it has been shown that the new codes require less vertical steel to be used – allowing for cost savings in material and labor.

This edition of Masonry Chronicles contains excerpts from the 2006 edition of Design of Reinforced Masonry Structures, to be released shortly by the Concrete Masonry Association of California and Nevada (CMACN).

References

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About the Author

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