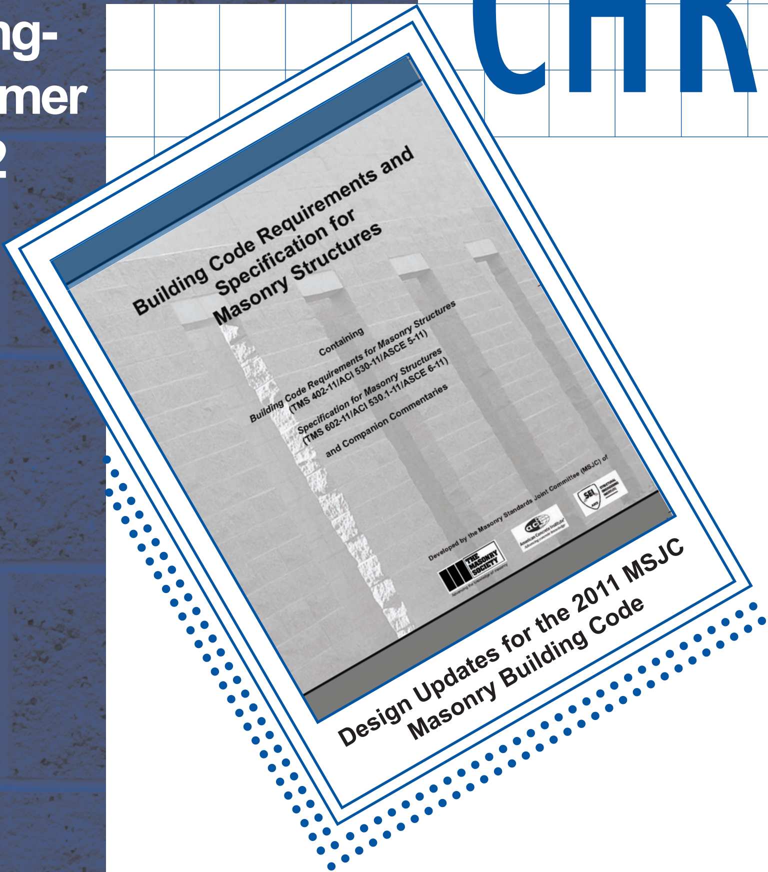
Engineering Notes For Design With Concrete Block Masonry

MASONRY CHRONICLES

Spring-Summer 2012



Allowable Stress Recalibration in the 2011 TMS 402 Building Code

Introduction

This edition of "Masonry Chronicles" will discuss major changes implemented in the 2011 Building Code Requirements for Masonry Structures [TMS 402-11] relating to Allowable Stress Design (ASD) provisions. Prior to this edition, allowable stresses were permitted to be increased by one-third when considering load combinations including wind or seismic forces. The origin and the reason for the one-third stress increase are unclear [Ellifritt 1977]. From a structural reliability standpoint, the one-third stress increase is a poor way to handle load combination effects [Ellingwood, 1980]. Due to these shortcomings of the one-third stress increase, other materials had eliminated this provision with masonry being the only structural material still permitting this provision. This provision was eliminated in the 2011 TMS Building Code.

Along with the elimination of the one-third stress increase, all of the allowable stresses in the 2011 Building Code were examined to determine if there was sufficient rationale for a change in the allowable value. A substantial amount of trial design work was done in support of the effort as well as a review of applicable research. This article provides a compilation of the changes and the supporting rational that was used for each change.

Summary of Changes to the 2011 TMS 402 Building Code

Anchor Bolts

No changes were made in anchor bolt allowable stresses. There was a major revision to the anchor bolt design requirements in Chapter 2 (Allowable Stress Design of Masonry) during the 2008 Code cycle. This revision significantly increased the anchor bolt allowable stresses and harmonized allowable stress design and strength design of anchor bolts. The one-third stress increase permitted in the 2008 Building Code should not be applied to anchor bolt design.

Allowable Bearing Stress

The allowable bearing stress in TMS 402-08, $0.25f'_m$, is harmonized with Chapter 3 Strength Design of Masonry (SD) values. If the nominal bearing strength, $0.60f'_mA_{br}$, is multiplied by the resistance factor for bearing of 0.60 and divided by an "average" load factor of 1.4, the resulting stress value in the equation is $0.26f'_m$, or approximately the allowable bearing stress. Note that in comparison to other codes, the design bearing strength, $\phi(0.60f'_m)A_{br} = 0.36f'_mA_{br}$, is low. The nominal bearing strength in concrete [ACI 318, 2008] is $0.85f'_c$ times the bearing area. The strength reduction factor is 0.65 resulting in a design strength of $0.55f'_c$ times the bearing area. The nominal bearing strength in the Canadian Masonry Code [CSA, 2004] is $0.85f'_mA$. The strength reduction factor is 0.55 resulting in a design strength of $0.47f'_mA$.

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Based on comparison with other codes, the nominal bearing strength for strength design is increased from $0.6f'_m$ to $0.8f'_m$, resulting in a design strength of $0.6(0.8f'_m)A_{br} = 0.48f'_mA_{br}$, which is still below most codes. A similar increase in allowable bearing stress, from $0.25f'_m$ to $0.33f'_m$, is incorporated into allowable stress design (ASD).

Axial Compression

No changes were made in the 2011 TMS Building Code for allowable axial stresses. There are two reasons for this.

For unreinforced masonry, allowable stress design is currently well harmonized with strength design. The allowable axial stress, F_a, is $0.25f'_m$ multiplied by a slenderness reduction factor. The nominal axial strength, P_n, is $0.8(0.8)f'_mA_n$ multiplied by the same slenderness reduction factor. The resistance factor for unreinforced masonry is 0.6, and using an "average" load factor (LF) of 1.4 results in $\phi 0.8(0.8)/LF = 0.6(0.8)(0.8)/1.4 = 0.27$, or about ¼. Thus, ASD and SD are fairly well harmonized for unreinforced masonry.

For the design of prestressed walls, allowable stresses are increased by 20% for the stress condition immediately after transfer. Any change in allowable axial stress would need to be coordinated with prestressed design. No change is being made at present, but the code committee continues to examine this issue.

Unreinforced Masonry

Unreinforced masonry design of members subjected to lateral loads is primarily governed by flexural tension, whether loaded in-plane or out-of-plane. Kim [2002] performed a reliability analysis of unreinforced walls under wind loading where 327 full-scale wall tests were examined. The reliability analysis showed that unreinforced masonry walls have a sufficient safety level even with the one-third stress increase. This provides a justification for increasing the allowable flexural tension values by 4/3 when eliminating the one-third stress increase. It is also noted that unreinforced masonry walls have performed well under wind loading, even when using the one-third stress increase in design. Finally, it is noted that most unreinforced masonry walls designed by current standards will be controlled by wind load as unreinforced masonry is only allowed for participating elements in Seismic Design Categories A and B.

The allowable flexural compression stress was left as $1/3f'_m$, where f'_m is the specified compressive strength of the masonry. It was felt that there was insufficient data at present to justify an increase. The allowable flexural compression stress rarely controls the design of unreinforced masonry, and thus the impact of not changing this value is minimal.

The allowable shear stress values for unreinforced masonry were not changed. It was felt that there was insufficient data at present to justify any increase. Almost all unreinforced masonry shear walls will be controlled by flexural tension and not shear. Thus, the impact of not changing this value is minimal.

Reinforced Masonry - Allowable Steel Stress

The allowable steel stress for Grade 60 reinforcement was increased from 24 ksi to 32 ksi for both tension and compression. No change was made to the allowable tensile stress for Grade 40 reinforcement or wire joint reinforcement. The use of Grade 40 reinforcement is rather limited, and the current allowable tensile stress, 20 ksi, is approximately the same percentage of the yield stress as being proposed for Grade 60 reinforcement. There is a modest increase to the allowable compressive stress for Grade 40 reinforcement from $0.4f_y$ = 16 ksi to 20 ksi, where f_y is the specified yield strength of the reinforcement. This provides consistency between tensile and compressive allowable stresses.

To justify the increase in the allowable stress for Grade 60 reinforcement, the committee examined a non-bearing wall under out-of-plane wind load, which it believed to be a critical case. Allowable stress design is compared to strength design. At the time of the analysis, the load factor for wind loads was 1.6. The strength reduction factor is 0.9. Using these factors with Grade 60 reinforcement results in an allowable stress of (60ksi) x (0.9) / 1.6 = 33.75 ksi. Strength design explicitly includes second-order effects while allowable stress design does not. Thus, allowable stress design should be more conservative than strength design, or there should be an explicit inclusion of second-order effects in allowable stress design. The committee adopted having conservative allowable stress design values, and not having an explicit second-order analysis in allowable stress design. There is also some additional conservatism with allowable stress design since the internal lever arm is usually smaller than in strength design.

Two 8 in. CMU walls were evaluated to examine the effect of the increase in allowable stress. The walls have an f'_m=1.5 ksi and Grade 60 reinforcement. The results of the analysis are shown in Table 1. With an allowable stress of 32 ksi, there is around an 8% conservatism over strength design. This is considered sufficient to account for second-order effects.

Table 1. Comparison of allowable stress and strength design for non-bearing wall

Reinforcement	M_n (k-ft/ft)	M_{allow} (k-ft/ft)	$0.9M_n/1.6$ (k-ft/ft)	$M_{allow}/[0.9M_n/1.6]$
#4 @ 48 in.	0.927	0.475	0.521	0.912
#5 @ 32 in.	2.075	1.071	1.167	0.918

M_n = nominal moment capacity; M_{allow} = allowable moment capacity

Reinforced Masonry - Allowable Combined Flexural and Axial Compressive Masonry Stress

The allowable compressive stress for combined flexural and axial compressive loads were increased from $0.33f'_m$ to $0.45f'_m$. The justification for this is as follows.

Under pure flexure, there is little increase in allowable moment with increase in reinforcement area when the masonry allowable compressive stress controls the design, Figure 1. An allowable masonry compressive stress can be chosen so that the masonry compressive stress controls when the reinforcement area reaches some ratio of the balanced reinforcement ratio (balanced based on strength design, the reinforcement yields just as the masonry reaches the maximum useable strain). This can be derived as follows for Grade 60 reinforcement and CMU masonry, although a similar derivation could be constructed for other conditions.

For strength design, the balanced reinforcement ratio is:

$$\rho_{b,strength} = \frac{0.8(0.8)f'_m}{f_y}\left(\frac{\varepsilon_m}{\varepsilon_m+\varepsilon_y}\right) = \frac{0.8(0.8)f'_m}{f_y}\left(\frac{0.0025}{0.0025+0.00207}\right) = 0.3502\frac{f'_m}{f_y} \tag{1}$$

where $\rho_{b,strength}$ is the balanced reinforcement ratio (based on strength design), ε_m is the maximum useable compressive strain, and ε_y is the yield strain. Now consider the balanced ratio for allowable stress design (ratio of reinforcement for which the masonry compressive stress reaches the allowable exactly when the steel tensile stress reaches the allowable), as shown in Equation 2.

$$\rho_{b,ASD} = \left[\frac{n}{n+\frac{F_s}{F_b}}\right]\left[\frac{1}{2\frac{F_s}{F_b}}\right] = \left[\frac{\frac{29000}{900f'_m}}{\frac{29000}{900f'_m}+\frac{x_S f_y}{x_M f'_m}}\right]\left[\frac{1}{2\frac{x_S f_y}{x_M f'_m}}\right] = \left[\frac{32.22}{32.22+\frac{x_S(60)}{x_M}}\right]\left[\frac{1}{2\frac{x_S f_y}{x_M f'_m}}\right] \tag{2}$$

where $F_s = x_s f_y$ is the allowable reinforcement tensile stress, $F_b = x_m f'_m$ is the allowable masonry compressive stress, $n = E_s/E_m$ is the ratio of the modulus of elasticity of the reinforcement to the modulus of elasticity of the masonry, and the modulus of elasticity of concrete masonry is obtained as $E_m = 900f'_m$, the relationship used in TMS 402. For an allowable reinforcement tensile stress of 32 ksi (x_s = 0.533) and an allowable masonry compressive stress of $0.45f'_m$ (x_m = 0.45), the masonry stress begins to control at $\rho_{b,ASD} = 0.375\rho_{b,strength}$. The masonry allowable compressive stress provides a practical limit on the amount of reinforcement, as again, once masonry controls, there is little increase in moment with increasing reinforcement. Masonry elements will still fail in flexural tension (a ductile failure) even when the masonry compressive stress controls the design.

The comparison of allowable stress design (ASD) and strength design (SD) is shown in Figure 1 for CMU elements and Grade 60 reinforcement. The nominal moment is multiplied by a strength reduction factor of 0.9 and divided by a load factor of 1.6 for comparison to the allowable moment. For low amounts of reinforcement, the reinforcement allowable stress will control the design. As pointed out in the previous section, ASD with the new allowable values will give slightly more conservative designs than SD. At higher reinforcement ratios, the masonry allowable stress will control the design. However, the failure of the member will still be ductile as the reinforcement ratio is below the balanced reinforcement ratio for strength design.

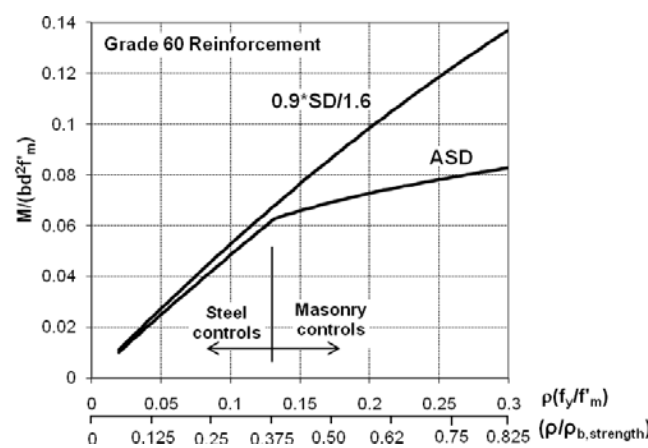

Figure 1. Comparison of ASD and SD flexural design for CMU elements

Historically, the Uniform Building Code [UBC 1997] limited the reinforcement to one-half of $\rho_{b,strength}$. One consideration in developing the allowable masonry stress was to determine an allowable masonry stress for which the masonry stress would start controlling at a reinforcement ratio of $0.5\rho_{b,strength}$. This value would be $0.534f'_m$ for CMU masonry and $0.528f'_m$ for clay masonry. Based on this, a masonry allowable stress of $0.5f'_m$ was considered, but was ultimately not adopted. Historically, allowable stress design of concrete used $0.45f'_c$ as the limiting compressive stress value, and it was decided to use the 0.45 value for masonry.

The same trends observed for members in pure flexure also were observed when considering combined flexural and axial loads using the new allowable stress values. When the reinforcement allowable stress controlled the design, ASD and SD gave similar results, with ASD being slightly conservative. When the allowable masonry stress controlled the design, ASD was much more conservative than SD. This is seen in the interaction diagram in Figure 2, which is for a 2-ft wide, 8-ft high wall segment made of 8-in. CMU (f'_m = 1500 psi). The reinforcement is one #5 bar in each end cell. Figure 2 compares the allowable axial force and moment using ASD to the allowable using SD, where the allowable using SD is obtained by multiplying the nominal strength by the strength reduction factor, 0.9, and dividing by a load factor of 1.6. As with pure flexure, ASD designs are quite conservative with respect to SD when the masonry allowable stress controls the design. Even when the masonry stress controls the design, the failure mode will often be yielding of the reinforcement.

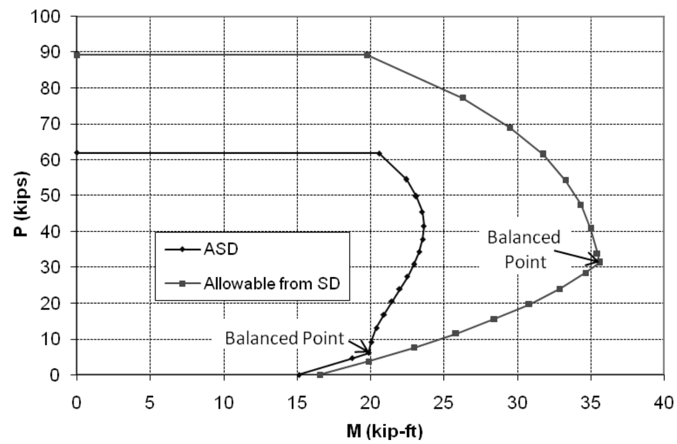

Figure 2. Comparison of ASD and SD interaction diagrams

Reinforced Masonry - Allowable Shear Stresses

Historically with US building codes, allowable stress design has not added the shear resistance from the masonry and the reinforcement. Rather, either the masonry had to have sufficient capacity to carry the entire shear force or the reinforcement had to have sufficient capacity to carry the entire shear force. This is different from strength design, where the shear capacity from the masonry and the reinforcement are permitted to be added together. The justification given in the commentary of TMS 402-08 for not allowing the shear resistance from the masonry and the reinforcement to be added together in allowable stress design is a 1974 reference [Priestley 1974]. In more recent work [Paulay 1992] it has been proposed that the masonry and reinforcement shear strength can be added together.

A recent study [Davis 2010] compared eight different methods for predicting the in-plane shear capacity of masonry walls with the results from fifty-six tests of masonry walls failing in in-plane shear. The test data encompassed both concrete masonry walls and clay masonry walls, all of which were fully grouted. Of the eight different methods examined, the design provisions of TMS 402 Chapter 3 (strength design) were found to be the best predictor of shear strength. The average ratio of the test capacity to the calculated capacity was 1.16 with a coefficient of variation of 0.15. The TMS 402-08 allowable stress shear design equations were found to be both very conservative and to have a high amount of scatter. The average ratio of the test capacity to the calculated capacity using just the masonry shear strength was 8.51 with a coefficient of variation of 0.25. The average ratio of the test capacity to the calculated capacity using just the reinforcement shear strength was 9.62 with a coefficient of variation of 0.48.

Based on the results of Davis [2010], the shear strength equations of Chapter 3 (strength design) were adopted for allowable stress design with the following modifications.

- All strength capacities are divided by a factor of 2. This factor was obtained as a load factor of 1.6 divided by a resistance factor of 0.8.
- Service loads are used instead of factored loads. For example, the masonry shear strength includes a term for the contribution of axial load to the shear strength. The axial load is expressed in terms of service load instead of factored load.
- The equations are written in terms of stress instead of force to be consistent with the rest of Chapter 2.

To summarize, the allowable shear stress, F_v, is obtained as the sum of the allowable shear stress resisted by the masonry, F_{vm}, and the allowable shear stress resisted by the shear reinforcement, F_{vs}.

$$F_v = F_{vm} + F_{vs} \qquad \textbf{(3)}$$

The allowable shear stress resisted by the masonry, F_{vm}, is obtained as:

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n} \qquad \textbf{(4)}$$

where M is the applied moment, V is the applied shear, d is the distance from the extreme compression face to the centroid of the reinforcement, and P is the applied axial force. The allowable shear stress resisted by the shear reinforcement, F_{vs}, is obtained as:

$$F_{vs} = 0.5\left(\frac{A_v F_s d}{A_n s}\right) \qquad \textbf{(5)}$$

where A_v is the area of the shear reinforcement and s is the spacing of the shear reinforcement. The contribution to the allowable shear stress provided by shear reinforcement, Equation 5, represents half the theoretical contribution. In other words, the allowable shear stress is determined as the full masonry contribution plus one-half the contribution from the shear reinforcement. Other coefficients for the contribution of the shear reinforcement were evaluated (0.6, 0.8, and 1.0), but the best fit to the experimental data was obtained using the 0.5 factor [Davis 2010].

A significant number of trial designs were conducted to evaluate the new allowable stress shear design provisions. A summary of a few of the trial designs is given in Table 2. The TMS 402-11 allowable stress design provisions require essentially the same amount of shear reinforcement as the strength design provisions, and in general less shear reinforcement than the TMS 402-08 allowable stress design provisions. Further details on the trial designs are available in Huston [2011].

Special Reinforced Shear Walls

TMS 402-08 has shear capacity design requirements for special reinforced masonry shear walls as part of the seismic design provisions. For strength design, the design shear strength, φV_n, must exceed the shear corresponding to the development of 1.25 times the nominal flexural strength, M_n, of the wall, except that the nominal shear strength, V_n, need not exceed 2.5 times required shear strength, V_u. For allowable stress design, the design load is required to be increased by a factor of 1.5. Trial designs for special shear walls using these shear capacity design requirements and the allowable shear stresses given in Equations 3-5 showed that allowable stress design would require much less shear reinforcement than strength design. The committee did not feel this was appropriate.

Various options to address this situation were examined. The option chosen was to use a reduced value for the allowable masonry shear stress to account for the degradation of masonry shear strength that occurs in plastic hinging regions [Anderson 1992]. Davis [2010] recommended a reduction factor of 1.0 (no reduction) for wall ductility ratios of 2.0 or less, and decreasing linearly to zero as the ductility ratio increases from 2.0 to 4.0. The committee chose a constant value of 0.5 for design convenience. The resulting allowable shear stress due to masonry for special shear walls is:

$$F_{vm} = \frac{1}{4}\left[\left(4.0 - 1.75\left(\frac{M}{Vd}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n} \qquad \textbf{(6)}$$

Again, numerous trial designs were performed with a summary in Table 2, and further details are given in Huston [2011].

Table 2. Results of shear wall trial designs

Wall #	f'_m (psi)	t (in)	L (in)	h (ft)	M/(V*d_v)	Spacing of #5 Grade 60 shear reinforcement (in)					
						Non-special shear wall			Special shear wall		
						08 ASD	08 SD	11 ASD	08 ASD	08 SD	11 ASD
2	1500	7.625	232	20	1.03	NR	NR	NR	16	16	16
12	3000	5.625	96	8	1.08	24	NR	NR	16	40	32
12B	3000	5.625	96	8	1.08	24	NR	NR	16	40	32
9	3000	7.625	176	10	0.68	16	NR	NR	8	16	16
9B	3000	7.625	176	10	0.68	16	NR	NR	8	16	16
20	1500	7.625	504	12	0.29	NR	NR	NR	40	40	24

NR = not required; 08 ASD = shear reinforcement required by TMS 402-08 allowable stress design provisions; 08 SD = shear reinforcement required by TMS 402-08 strength design provisions (unchanged in TMS 402-11), and 11 ASD = shear reinforcement required by TMS 402-11 allowable stress design provisions.

Shear Wall Example

A 10 foot high by 16 foot long, 8-inch fully grouted CMU shear wall is constructed using Grade 60 reinforcement. The specified compressive strength, f'_m, is 1500 psi. The vertical reinforcement is 2-#5s at each end and #5s @ 32 inch on center. There is a superimposed dead load of 1kip/ft.

This wall will be analyzed under in-plane loads using the load combination of 0.9D + 0.7E (ASCE 7 permits this load combination for special reinforced masonry shear walls). For illustrative purposes and simplicity, vertical earthquake forces will not be considered, although they would need to be considered in an actual design. Based on flexure (overturning), the maximum in-plane load, E, would be 90.5 kips. Since this is a special reinforced masonry shear wall, the load would have to be increased by 1.5 for shear design, and the alternate equation for shear capacity would have to be used (Equation [6] in this paper; Equation (2-29) in the Code). The shear stress under this load is 64.9 psi. The maximum allowable shear stress is 96.8 psi, so the wall is OK by this criterion. The calculated required shear reinforcement is #5s @ 19.6 inches, or #5s @ 16 inches would be used.

By comparison, the 2008 ASD provisions would have resulted in the same capacity based on flexure if the one-third stress increase had been used. The applied shear stress of 64.9 psi is just less than the maximum allowable shear stress of 65.3 psi. The calculated required shear reinforcement using the 2008 ASD provisions is #5s @ 20.3 inches, or #5s @ 16 inches would be used. We see that there is little difference in the required shear reinforcement between the 2008 and 2011 ASD provisions, but the 2011 provisions permit a higher maximum allowable shear stress.

If the vertical reinforcement were #5s @ 24 inch on center instead of #5s @ 32 inch on center, the maximum in-plane load, E, would be 105.8 kips. The shear stress under 1.5 times the load would be 72.2 psi, which is still less than the maximum allowable shear stress permitted under the 2011 provisions. The calculated shear reinforcement is #5s @ 16.0 inches, so the shear reinforcement would not change; #5s @ 16 inches would be used. Under the 2008 provisions, the maximum allowable shear stress is limited to 65.3 psi, or the maximum in-plane load, E, would be 95.6 kips. The wall would be limited by the shear provisions in the 2008 Code.

For comparison, the original wall (#5s @ 32 inches) will be analyzed using the 2011 strength design provisions. The maximum in-plane earthquake load, E, would be125.9 kips. The 39% greater load is partly due to ASD provisions being slightly more conservative than strength design provisions, and partly due to distributed bars not counting as much in ASD as in strength design. In strength design, most tension bars will have yielded, irrespective of their location. In ASD, where a linear stress distribution across the cross-section is assumed, bars that are not at the end of the wall will both have a lower stress and a smaller lever arm for computing flexural capacity.

Summary

The major changes to the allowable stresses in TMS 402-11 can be summarized as follows:

1. The allowable flexural tensile stresses for clay and concrete masonry were increased based on historical performance and the results of a reliability analysis reported in the literature.

2. Allowable stresses for axial compression for either unreinforced or reinforced masonry were not changed.

3. The allowable reinforcement stress and allowable masonry compressive stresses due to flexure or flexure in combination with axial load were increased based on a comparison with strength design procedures.

4. Allowable shear stresses for reinforced masonry elements were changed to be similar to strength design based on a recent comparison of predicted strengths using a variety of code methods to experimental strength. It is now permitted to add the shear strength of the masonry and the shear strength of the reinforcement to determine the allowable shear strength.

5. Anchor bolt stresses were not changed due to a recent major revision of the allowable anchor bolt stresses in the 2008 code.

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