TMS RESPONDS

Answers to questions regarding masonry design, construction, evaluation and repair
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Joint Reinforcement Types

18.1-1 Does it matter whether truss-type joint reinforcement or ladder-type joint reinforcement is used in masonry?

Response by Rochelle Jaffe, Rochelle C. Jaffe Consulting, P.C., and Dan Zechmeister, Masonry Institute of Michigan

Masonry Construction magazine published an article written by Mario Catani in 1995 titled *Selecting the Right Joint Reinforcement for the Job*. Mr. Catani stated; "Truss-type reinforcement is the best configuration for all single-wythe walls except those that are vertically reinforced. The diagonal cross wires contribute to the reinforcement's tensile capacity to resist shrinkage stresses, tension due to bending in the horizontal plane, and in-plane shear... In vertically reinforced single-wythe walls, ladder-type joint reinforcement is preferred. Though not as strong as the truss-type, it interferes less with the placement of vertical steel." His comments are still valid today. Figure 1 illustrates ladder-type and truss-type joint reinforcement.

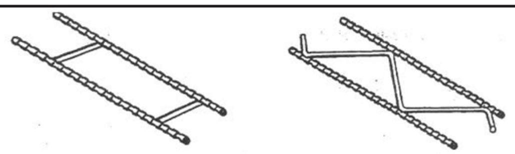


Figure 1 - Ladder Type and Truss Type Joint Reinforcement

Except in high seismic regions, where there is typically sufficient horizontal bar reinforcement to resist shrinkage stresses over long masonry lengths, designers use horizontal joint reinforcement in combination with control joints in concrete masonry to control shrinkage stresses. The longitudinal wires in either ladder-type or truss-type joint reinforcement are adequate to resist the shrinkage stresses that accumulate between control joints when the control joints are located in accordance with industry recommendations and the joint reinforcement is spaced no more than 16 in. on center in the bed joints. A great majority of walls are designed to span in the vertical direction to resist lateral loads. In the last 25 years or so, masonry designs in other than high seismic regions have changed from unreinforced (in the vertical direction) to containing vertical reinforcing bars to enhance masonry's strength and ductility. The presence of vertical reinforcing bars in combination with joint reinforcement can affect the mason's ability to erect quality masonry.

TMS 602-16 Specification for Masonry Structures states "Where vertical reinforcement is present in a masonry wall, location of the truss type joint reinforcement may conflict with the vertical reinforcement" in its Commentary to Article 2.4 C.

When ladder-type joint reinforcement with cross wires welded at 16 inches on center is placed so that the cross wires are located on top of the webs of the concrete block, none of the cross wires will pass over the cells of the blocks. As a result, there will be no interferences for placing the vertical bars, the grout, and/or insulation in the cells of the block. Note that some manufacturers of joint reinforcement fabricate it with cross-wires spaced at less than 16 inches, which is undesirable. With truss-type joint reinforcement, the diagonal cross wire passes over the cell of the block and could interfere with placing the vertical bars. To solve this problem, the mason contractor often cuts the diagonal wire. The diagonal cross wires also collect mortar droppings as the masonry units are laid, resulting in obstructions that could interfere with mechanical consolidation and re-consolidation of the grout. See Figure 2. The diagonal cross wires could also interfere with placement of cell insulation.

In this Issue, TMS Members respond to questions on:

Figure 2 - Mortar droppings on truss-type joint reinforcement diagonal cross-wires

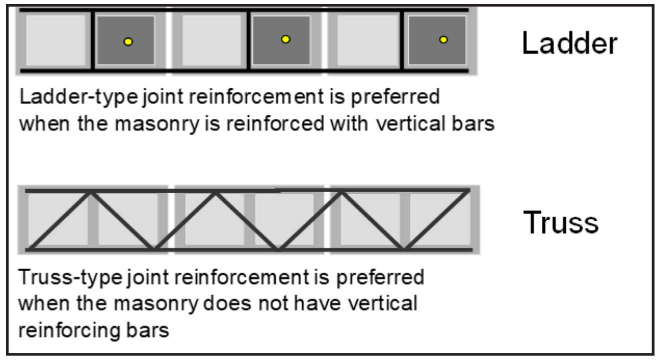


Figure 3 - Recommended use of horizontal joint reinforcement

For these reasons, ladder-type joint reinforcement is preferred over truss-type in masonry that includes vertical reinforcing bars. In concrete masonry that contains no vertical reinforcement, which is becoming rare today, truss-type joint reinforcement is preferred. These recommendations are summarized in Figure 3.

Joint Reinforcement - Heavy Duty vs. Standard

18.1-2 Is heavy duty joint reinforcement better than standard joint reinforcement?

Response by Rochelle Jaffe, Rochelle C. Jaffe Consulting, P.C.

Heavy duty joint reinforcement is better suited to meet certain structural requirements of TMS 402. In most cases, however, standard joint reinforcement has adequate structural capacity and its installation does not present the same problems as placing heavy duty joint reinforcement. In order to understand the issues involved, we will start by defining what "standard joint reinforcement" and "heavy-duty joint reinforcement" are.

Firstly, the term "joint reinforcement" is used to define a manufactured product that is intended to be placed in mortar bed (horizontal) joints in masonry. Joint reinforcement is an assembly of wires, consisting of two to four longitudinal wires (oriented parallel to the length of the masonry wall) that are connected by cross wires. The cross wires may be perpendicular to the longitudinal wires, in which case the joint reinforcement is identified as "ladder type", or may be diagonal to the longitudinal wires, which identifies the joint reinforcement as "truss type". Manufacture of the assembly is governed by ASTM A951 Standard Specification for Steel Wire for Masonry Joint Reinforcement. Figure 1 illustrates ladder type joint reinforcement placed on the units before the mortar is placed.

Secondly, the wires sizes used in joint reinforcement are consistent from manufacturer to manufacturer. The most widely used joint reinforcement is "standard joint reinforcement" and is made of 9 gage (W1.7 (MW 11)) longitudinal wires and cross wires. Heavy duty joint reinforcement has 3/16 in. (W2.8 (MW17)) longitudinal wires and 9 gage cross wires.

Thirdly, the maximum size of the wire is limited by TMS 402 Section 6.1.2.3 to one-half the mortar joint thickness to allow free flow of mortar around the joint reinforcement wires. This limitation is illustrated in Figure 2. Although not specifically stated in TMS 402, this limitation is interpreted to be based on the specified mortar joint thickness and not the as-installed mortar joint thickness. The as-installed thickness is permitted to vary from the specified thickness by a tolerance of plus or minus one-eighth inch, according to TMS 602 Article 3.3F.1.b. This means that a specified 3/8-inch joint can be as small as 1/4-inch and still be in compliance with TMS 602.

Fourthly, joint reinforcement can perform many structural functions:

1. Control vertical cracking in concrete masonry resulting from shrinkage volume changes;
2. Strengthen masonry to resist out-of-plane lateral (horizontal) loading;
3. Strengthen masonry to resist in-plane vertical (dead and live) loading (beams, for example);
4. And strengthen masonry to resist in-plane lateral loading (shear walls).

Standard joint reinforcement performs all of the structural functions. However, when the masonry is designed by the strength design method and joint reinforcement is needed to resist shear from in-plane lateral loading, then (and only then) the joint reinforcement must be "heavy duty" per TMS 402 Section 9.3.3.7. The strength design requirement resulted from testing that showed that the larger wire performs better under seismic conditions because it is less brittle than standard wire.

Because the longitudinal wires are larger, heavy duty joint reinforcement is stronger in tension than standard joint reinforcement. But there is only one specific set of design conditions that mandates the use of heavy duty joint reinforcement. When those conditions do not exist, standard joint reinforcement is preferred because heavy-duty joint reinforcement is difficult to place in a specified 3/8 in (10 mm) mortar joint. Some of the conditions that make placement of heavy duty joint reinforcement difficult include:

- The height of a masonry unit is permitted to vary from its specified height by the tolerances specified within the pertinent ASTM standard for the type of unit. The difference in height of adjacent masonry units is accommodated by varying the thickness of the mortar joint. Thus, the mortar joint is thinner over a masonry unit that varies in the "plus" (taller) direction. If the mortar joint is thinner than twice the wire thickness, mortar may not freely flow around the wire to encapsulate it.
- Joint reinforcement is not manufactured to be perfectly flat along its length; it may have some warp or curl. Handling and storage of the joint reinforcement increases its "out-of-flatness". Thicker mortar joints than specified are typically required to encapsulate heavy duty joint reinforcement wires in mortar.
- As discussed above, mortar joint thickness may be one-quarter inch and still meet the tolerance requirement of TMS 602 for bed joint thickness. However, a 3/16 inch wire cannot be fully encapsulated in mortar when the joint is only one-quarter inch thick.

For the reasons listed, masons typically have to make mortar bed joints that contain heavy duty joint reinforcement thicker than specified (unit height variation and joint reinforcement warpage). In an attempt to meet the tolerance requirements of TMS 602 Article 3.3 F.2, including variation from level and true to a line, contractors typically make the mortar joints that do not contain the heavy duty joint reinforcement thinner than specified. This not only adversely affects appearance, because the mortar joint thickness is inconsistent, but the attempt is usually unsuccessful in achieving the overall level tolerance requirements and meeting the specified elevations.

Consequently, the authors strongly recommend that standard 9 gage joint reinforcement be used rather than heavy duty joint reinforcement, even if it means adding joint reinforcement to more bed joints or supplementing with reinforcing bars in horizontal bond beams. Heavy duty joint reinforcement should only be used when the structural engineer determines that it is required to resist in-plane shear in strength design and multiple reinforced bond beams are not economical.

Figure 1 - Ladder type joint reinforcement laid dry on the masonry units

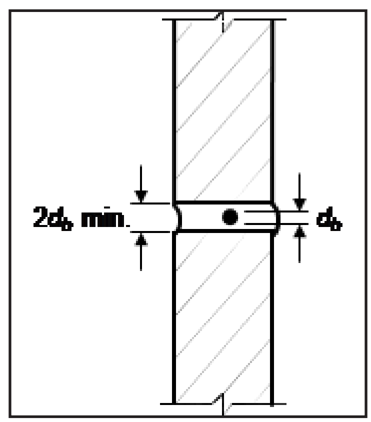


Figure 2 - TMS 402 requirements for maximum diameter of joint reinforcement

Allowable Shear Special Walls

18.1-3 When I design special reinforced masonry shear walls with Allowable Stress Design, there is a penalty on the allowable masonry shear strength. This penalty is not applied in Strength Design to the nominal masonry strength. This seems to cause shear walls designed using Allowable Stress Design to require a higher amounts of shear reinforcement than walls designed using Strength Design. Why is the penalty applied to the allowable masonry shear strength and not the nominal masonry strength?

Response by Richard Bennett, University of Tennessee
Reviewed by John Hochwalt, KPFF Consulting Engineers

The shear capacity provisions (TMS 402 Section 7.3.2.6.1), combined with the reduced allowable masonry shear stresses for special masonry walls, are intended to account for the limited ductility provided by masonry in shear. For strength design this goal is accomplished solely within the shear capacity provisions by requiring the design shear strength, ϕV_n, to exceed 1.25 times the shear associated with the development of a nominal moment strength, M_n, except the nominal shear strength, V_n, need not exceed 2.5 times the factored shear, V_u. The latter is equivalent to a doubling of the shear force, as for non-special walls the nominal strength has to exceed V_u/ϕ=1.25V_u. For ASD the same goal is accomplished by two code provisions - the shear capacity provision which amplifies the imposed shear stresses by a factor of 1.5, and by Equation 8-25 which reduces the allowable shear stress in the masonry by a factor of 2. The result of the difference in the approaches to addressing limited shear ductility is that the ASD provisions require more shear reinforcing than the SD provisions when the shear demands are low, and less reinforcing when the shear demands are high. This is explained in more detail below.

The reason for the different approaches in the ASD and SD provisions can be found in the history of the development of these provisions. There was a major change to the allowable shear stress between the 2008 and 2011 TMS 402 Codes. The previously allowed one-third stress increase for load combinations including seismic and wind loads was deleted, and the allowable stresses were recalibrated. This resulted in the allowable shear stress being approximately the nominal shear strength divided by a factor of safety of 2, and also divided by area to obtain a stress. For non-special shear walls, Allowable Stress Design (ASD) and Strength Design (SD) give reasonably the same designs, although there are slight differences. This is shown in Table 1.

Table 1 shows the three components that contribute to the shear strength: the masonry, the axial load, and the shear reinforcement. For seismic loads, the allowable stress level shear is 0.7 times the strength level shear, per the ASCE-7 ASD load combinations or the IBC basic ASD load combinations. For wind loads, the allowable stress level shear is 0.6 times the strength level shear, per the ASCE-7 ASD load combinations or the IBC basic ASD load combinations.

For the masonry shear, the allowable shear is 0.625 times the design shear, where the design shear is the strength-reduction factor of 0.8 times the nominal shear strength. Since the allowable stress design load is only 0.7 of the strength design load, ASD results in a 0.625/0.7=0.893 of the masonry strength as SD. However, for wind, ASD results in 0.625/0.6=1.042 of the masonry strength of SD.

The comparison of the shear from the axial load is a little more complicated, since the axial load includes a vertical seismic component. For S_{DS}=0, the ratio of ASD to SD is 0.6/0.63=0.952. For the maximum value of S_{DS} for a non-special wall of 0.5, the ratio of ASD to SD is 0.946.

The comparison for the steel strength is based on Grade 60 steel, which has an allowable stress of 32 ksi. The factor of safety is thus slightly less than 2.

From Table 1, it is seen that for non-special walls, ASD is a bit more conservative than SD for seismic loads, while it is a bit less conservative than SD for wind loads. This is simply a function of the load combinations.

	Design Shear	**Allowable Shear**	**Allowable Shear / Design Shear**	**ASD/SD Seismic**	**ASD/SD Wind**
Masonry	$0.8V_{nm}$	$0.5V_{nm}$	0.625	0.893	1.042
Axial	$0.25(0.9-0.2S_{DS})D$	$0.25(0.6-0.14S_{DS})D$	$\frac{0.6-0.14S_{DS}}{0.9-0.2S_{DS}}$	$\frac{0.6-0.14S_{DS}}{0.63-0.14S_{DS}}$ {0.946 to 0.952}	1.111
Steel	$0.8V_{ns}$	$\frac{32}{60}V_{ns}$	0.667	0.952	1.111

Table 1 - Comparison of ASD and SD shear design for non-special reinforced masonry shear walls

As mentioned, TMS 402 has shear capacity design requirements for special reinforced masonry shear walls as part of the seismic design provisions in order to minimize the chance of a brittle shear failure. When the allowable shear strength was changed in 2011, various options were considered to harmonize ASD and SD shear capacity design. One option considered was to change the 1.5 factor in ASD to 2.0. However, given the 1.5 factor had a long history of use in the Uniform Building Code, another option was chosen. The option chose was to decrease the allowable shear strength of the masonry by a factor of 2. Further information is in Bennett and Huston (2011).

Table 2 shows the comparison between ASD and SD for special shear walls. For SD, the shear capacity provisions are based on the shear being doubled. Two comparisons are made for the shear from the axial load. One uses the standard factor of 0.6 for dead load. The other uses the exception allowed in ASCE-7 that 0.6D can be replaced with 0.9D in ASD for special reinforced masonry shear walls. The alternative ASD load combinations in the IBC allow an ASD factor of 0.9D for all shear walls, not just special shear walls. However, the factor on seismic loads in the alternative ASD load combinations is 1/1.4=0.714, or 2% greater.

The reduction in the allowable masonry shear for special reinforced shear walls results in a low ratio of allowable shear to design shear for the masonry component. The ratio of ASD to SD is 0.312(2)/(0.7*1.5)=0.595.

For the axial load, the ratio depends on the value of S_{DS}. For S_{DS}=0.5, the ratio of ASD to SD is 1.262 using a dead load factor of 0.6, and 1.976 using a dead load factor of 0.9. In the limit, as S_{DS} increases, the ratio of ASD to SD is 1.333 independent of the dead load factor. Thus, ASD gives a higher strength than SD. Likewise, for the shear due to steel, ASD gives a higher strength than SD.

Thus, whether ASD or SD gives a more efficient design for special reinforced masonry shear walls depends on the level of shear. For lower shear forces, where the strength is primarily from the strength of the masonry, SD is more efficient. For large shear forces, or for high axial forces, ASD is more efficient than strength design. Perhaps TMS 402 should be changed to increase the allowable stress level shear by a factor of 2 instead of 1.5, and to not decrease the allowable masonry shear stress. This would give more consistent designs between ASD and SD.

Reference

Bennett, R.M., and Huston, E.T. (2011). “Allowable Stress Shear Design Provisions: Trial Designs.” 11th North American Masonry Conference, Paper 34.

	Design Shear	Allowable Shear	Allowable Shear / Design Shear	ASD/SD Seismic
Masonry	$0.8V_{nm}$	$0.25V_{nm}$	0.312	0.595
Axial	$0.25(0.9-0.2S_{DS})D$	$0.25(0.6-0.14S_{DS})D$	$\frac{0.6-0.14S_{DS}}{0.9-0.2S_{DS}}$	$\frac{0.6-0.14S_{DS}}{0.472-0.105S_{DS}}$ {1.262 to 1.333}
Axial, with 0.9D in ASD	$0.25(0.9-0.2S_{DS})D$	$0.25(0.9-0.14S_{DS})D$	$\frac{0.9-0.14S_{DS}}{0.9-0.2S_{DS}}$	$\frac{0.9-0.14S_{DS}}{0.472-0.105S_{DS}}$ {1.976 to 1.333}
Steel	$0.8V_{ns}$	$\frac{32}{60}V_{ns}$	0.667	1.270

Table 2 - Comparison of ASD and SD shear design for special reinforced masonry shear walls

Allowable Shear Special Walls

18.1-4 In Equation (8-26) in TMS 402-16, the allowable shear stress resisted by the masonry includes a contribution based on the axial force on the wall divided by the net area. In Equation (8-27), the allowable shear stress resisted by the steel reinforcement is calculated based on the net shear area. These equations are referencing different wall areas when determining the allowable stress in the masonry and steel reinforcement, respectively. Why is that?

Also, how do you determine the net shear area of a partially grouted CMU wall? There is no reference in the code that I can find that indicates how to determine the net shear area for a partially grouted CMU wall.

Response by Richard Bennett, University of Tennessee
Reviewed by John Hochwalt, KPFF Consulting Engineers

The two equations that are being referred to are given below for convenience.

$$F_{vm} = \frac{1}{2}\left[\left(4.0 - 1.75\left(\frac{M}{Vd_v}\right)\right)\sqrt{f'_m}\right] + 0.25\frac{P}{A_n} \quad \text{(Equation 8-26)}$$

$$F_{vs} = 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right) \quad \text{(Equation 8-27)}$$

The reason that A_{nv} is used in Equation 8-27 is to account for flanged shear walls. For a flanged shear wall, the shear area is only the wall web, as shown in Figure CC-2.2-1 from the TMS 402 commentary. This is similar to concrete design, in which a web thickness, b_w, is used to determine shear strength. Any horizontal shear reinforcement perpendicular to the direction of the applied load would be ineffective in resisting shear. Another way of looking at it is that the shear stress is determined by dividing the shear force by A_{nv}, $f_v = V/A_{nv}$, TMS 402 Equation (8-21). Thus, to get an allowable shear stress, the shear resistance would be divided by the same area.

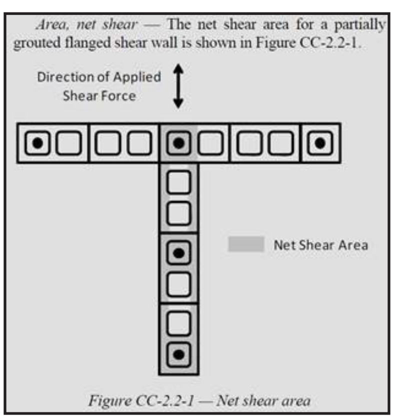
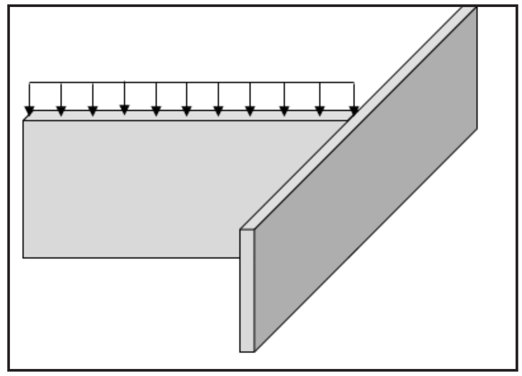
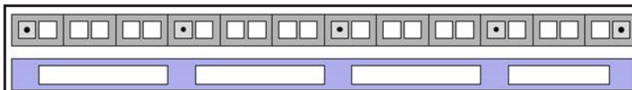


Figure CC-2.2-1 — Net shear area

Equation 8-26 is a little trickier. The assumption is that the axial force is acting over the entire wall, so to determine the axial stress the axial force is divided by the entire net area, or the area of both the flange and the web. However, there are exceptions. For the example wall shown below, dividing the axial force by just the area of the web of the wall would be appropriate. Our best advice is to just use good engineering judgement of how the load is carried.



Now, what is the net shear of a partially grouted shear wall. The net shear area should be the area of the mortared face shells and the grouted cells. For the following 16 ft long shear wall, the net shear area would be:



Net Shear Area: $A_{nv} = 2(1.25\text{in.})(192\text{in.}) + 5(8\text{in.})(7.625\text{in.} - 2.5\text{in.}) = 685\text{in.}^2$

The basis for this interpretation is the work that was done when the partially grouted shear wall factor of 0.75 was introduced into the code. Based on the table below, for fully grouted walls, the experimental shear strength averages 1.16 times what is predicted as the nominal shear strength. For partially grouted walls, the experimental strength is 0.90 times what is predicted as the nominal strength. The Code Committee took the ratio, got 0.776, and rounded to 0.75 for the partially grouted shear wall factor due to slightly higher variability for partially grouted walls. Another option, which some argued for, was to just use the face shells of a partially grouted shear wall. Per below, though, this is very conservative, and also has a very high variability.

Method	V_{exp}/V_n	
	Mean	St. Dev.
Partially Grouted Walls (Minaie et al, 2010; 60 tests)		
2008 Provisions	0.90	0.26
Multiply shear strengths by A_n/A_g	1.53	0.43
Using just face shells	1.77	0.78
Fully Grouted Walls (Davis et al, 2010; 56 tests)		
2008 Provisions	1.16	0.17

0.90/1.16 = 0.776; rounded to 0.75

Flexural Tension

18.1-5 TMS 402-11 removed provision 2.1.2.3 from TMS 402-08 that allowed a one-third increase for allowable stresses and loads when considering wind and seismic and just multiplied the allowable tensile flexural stresses from TMS 402-08 Table 2.2.3.2 by four-thirds to arrive at the allowable flexural tensile stresses found in TMS 402-11 Table 2.2.3.2. TMS 402-13 and TMS 402-16 moved the allowable flexural tensile stresses to Table 8.2.4.2 and retained all of the allowable flexural tensile stress values that were increased by four-thirds in TMS 402-11 Table 2.2.3.2 except for the values for Normal to Bed Joints Hollow Units Fully Grouted, which reverted back to the TMS 402-08 values that had not been increased by four-thirds (i.e., 65 psi, 63 psi, 61 psi, 58 psi). However, the commentary does not address this change in TMS 402-13 or 16, and in fact still includes an example (p. C-113 in TMS 402-13; p. C-111 in TMS 402-16) that uses an aforementioned value increased by four-thirds from TMS 402-11 (i.e., 86 psi rather than 65 psi is used in the example). I have checked the available errata for these editions, but do not see this addressed.

Are the allowable flexural tensile stresses for Normal to Bed Joints Hollow Units Fully Grouted in TMS 402-13 and 16 Table 8.2.4.2 (i.e., 65 psi, 63 psi, 61 psi, 58 psi) correct and the associated commentary incorrect?

Or, are the allowable flexural tensile stresses for Normal to Bed Joints Hollow Units Fully Grouted in TMS 402-11 Table 2.2.3.2 (i.e., 86 psi, 84 psi, 81 psi, 77 psi) the correct values and TMS 402-13 and 16 Table 8.2.4.2 incorrect?

Response by Richard Bennett, University of Tennessee

To fully answer this question, we need to go back to 1999, when the allowable flexural tension stresses for fully grouted normal to the bed joint were 68, 58, 41, and 29 psi, for Type M or S PCL mortar, Type N PCL mortar, Type S masonry cement mortar, and Type N masonry cement mortar, respectively. These were changed in the 2002 code to 65, 63, 61, and 58 psi. This was based on work by Brown and Melander (1999). The new allowable flexural tension stresses of 65, 63, 61, and 58 psi were approximately 1/2.5 of the experimental modulus of rupture values obtained by Brown and Melander (1999).

\When the one-third stress increase was removed in 2011, TMS 402 did increase the allowable flexural tension values, as you noted. The increase was based on 327 tests on unreinforced walls (Kim and Bennett, 2002). Further information is given in the commentary to Section 8.2.4.2 of TMS 402-16. All of these tests were on ungrouted walls. The committee made a mistake, and should not have increased the allowable flexural tension values for fully grouted walls. In fact, in hindsight, the new allowable flexural tension stresses in 2002 should have been ¾ of what they were to account for the one-third stress increase.

In 2013, the TMS 402 committee increased the modulus of rupture values to match the increase in the allowable flexural tension stresses. However, the committee realized the mistake they had made in increasing the allowable flexural tensile stress for fully grouted walls with the stress normal to the bed joint. Those were changed back to the allowable flexural tension value in the 2008 TMS 402 code.

The values on the TMS 402 code are correct, and are unlikely to change. The required change to the commentary to match the code change was missed by the committee. The 2022 TMS 402 Code Committee has balloted a change to correct the commentary.

Reference

Brown, R. and Melander, J. (1999). "Flexural Bond Strength of Unreinforced Grouted Masonry Using PCL and MC Mortars," Eighth North American Masonry Conference, Austin, TX, The Masonry Society.

Kim, Y.S. and Bennett, R.M. (2002). "Flexural Tension in Unreinforced Masonry: Evaluation of Current Specifications." TMS Journal, The Masonry Society, 20(1), 23-30.

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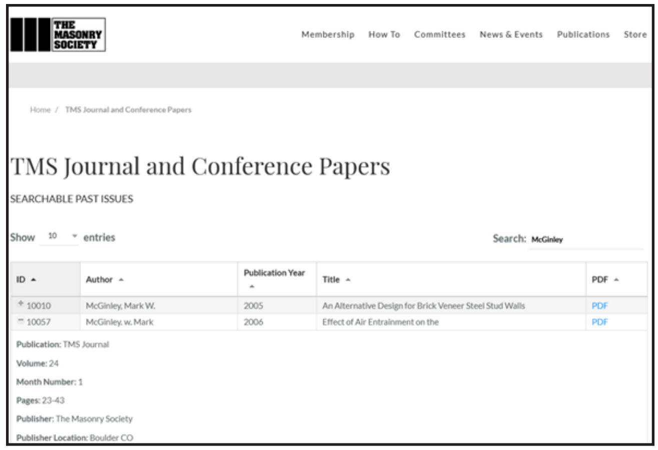

Figure 1 - Image taken from TMS's Online Searchable Database of TMS Journal and Conference Papers for a search of the Author McGinley, and where the second entry was selected by clicking the "+" that was to the left of the paper ID listing (in this view it shows as a "-" sign, and clicking it conflates the view.

I understand there is errata for TMS 402-13. Where can I access it?

We do our best to make sure information in our publications and standards is accurate and correct. However, errors and omissions unfortunately sometimes happen. Accordingly, TMS posts errata for its publications at masonrysociety.org/errata. Currently, errata are posted for the 2018, 2013, 2011, and 2008 editions of TMS 402/602, as well as the 4th Edition of Masonry Structures: Behavior and Design. If you identify a possible issue in something The Masonry Society publishes, please let us know. We will confirm the error, and if needed, publish a new errata.

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