Engineering Notes for Design
with Concrete Block Masonry

MASONRY CHRONICLES

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Illustration of the Destructive Nature of Coring

Coring of Concrete Masonry Walls: Is it Necessary?

Introduction

The 1933 Long Beach earthquake showed that unreinforced double-wythe masonry brick walls did not perform well. Consequently, California regulators imposed a requirement that double-wythe brick masonry be reinforced and grouted and that the newly constructed masonry be destructively tested by drilling a core specimen horizontally through the wall and that the bond between the clay masonry unit and grout be tested for shear capacity. The bond criteria for grout to masonry unit was arbitrarily set at 100 psi. In 1983, the bond criteria was changed to $2.5\sqrt{f'_m}$ psi, a value nearly equal to 100 psi.

Over the past 75 years, the requirement has morphed into application to single-wythe hollow unit masonry walls, which was never the intent of the provision and ignores the benefit of webs and tapers in Concrete Masonry Units. Additionally, there is discussion at the national level on whether or not destructively coring and testing the masonry cores is a worthwhile effort. The following analysis is based on current code provisions and puts the discussion into a rational perspective.

When a reinforced masonry wall is subjected to out-of-plane loads, the tension is carried by the reinforcement, and the compression by the masonry, Figure 1. In this context, the masonry is a combination of masonry units, mortar, and grout. There are also shear stresses in the wall. The shear stresses are both perpendicular to the face of the wall, as well as parallel to the face of the wall. The shear stresses parallel to the face of the wall are similar to those that develop between the structural steel and the concrete in a composite steel/concrete slab beam. The stresses in the cross-section are shown in Figure 2.

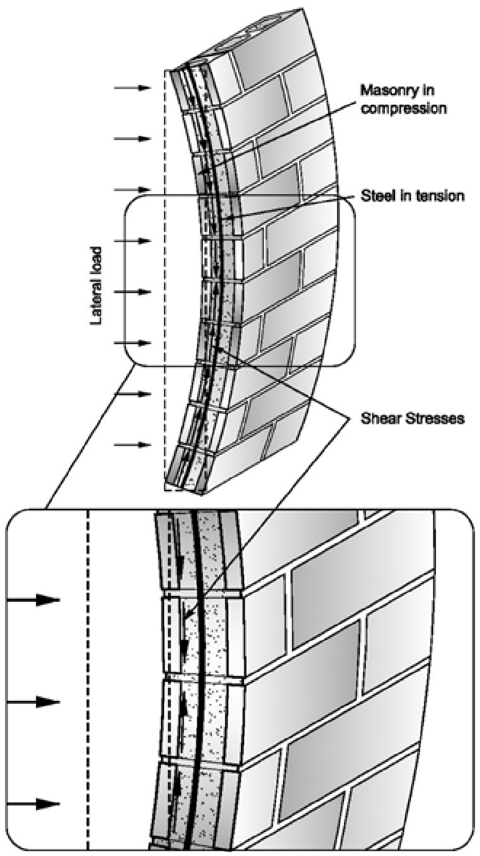

Figure 1. Masonry wall subjected to out-of-plane load

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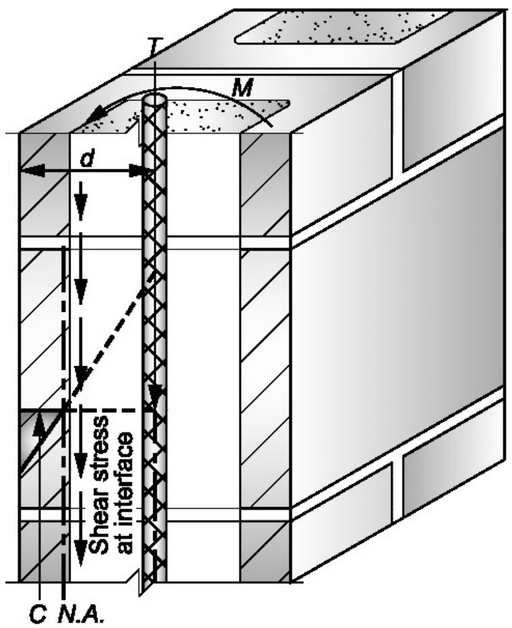

Figure 2. Stresses in a reinforced masonry wall

The TMS 402 Code, Building Code Requirements for Masonry Structures, requires the wall to be designed to carry the shear forces perpendicular to the face of wall (2013 TMS 402 Section 8.3.5 for Allowable Stress Design, and Section 9.3.5.3 for Strength Design). There are no requirements in TMS 402 with regard to the shear stresses parallel to the face of the wall. However, the California Division of State Architect and the California Office of Statewide Health Planning and Development have requirements for core testing of masonry walls. The minimum average unit shear interface requirement between the grout and face shell has been arbitrarily set at $2.5\sqrt{f'_m}$ psi. This requirement is presumably to verify that there is sufficient bond between the grout and the masonry unit to carry the shear stresses. The coring, shown in Figure 3, demonstrates the destructive nature of the testing. The question is whether this coring is necessary, and whether TMS 402 should even consider a similar requirement.

Figure 3. Illustration of the Destructive Nature of Coring
In many cases, as in photo 1 of 3, the first attempt hits reinforcement causing further damage.

To answer the question on the necessity of coring, a variety of wall configurations were analyzed. All walls were considered to be fully grouted and simply supported. The analysis procedure was as follows:

1. Select a wall height, block size, reinforcement bar size, reinforcement bar spacing, axial load, and a specified compressive strength, f'_m. Type S Portland cement-lime mortar was assumed for all walls. Wall weights were determined based on 125 pcf units, although this assumption has a negligible effect on the results. The axial load was assumed to act concentric with the wall. Any eccentricity to the axial load would reduce the out-of-plane load the wall could carry.
2. The wall was analyzed using the "slender wall procedure", Sections 9.3.5.4.2 of the 2013 TMS 402 Code, to determine the maximum out-of-plane load the wall could carry. In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.
3. Based on the maximum out-of-plane load, the maximum shear force was calculated. From the maximum shear force, the shear stress at the interface between the grout and face shell was calculated. If the wall is treated as a traditional composite section, and the equivalent rectangular stress block is in the face shell, the shear force at the grout/face shell interface will be based on the yield force of the steel. If part of the equivalent rectangular stress block were in the grouted core, the shear stress at the interface would be reduced. The shear stress can be obtained as the shear force divided by the shear area over half the wall height.

In some cases, loads were unrealistically high, being several hundred psf, but the load was still used.

A sample calculation is shown below.

Given: 20 ft high 8 inch CMU fully grouted wall; concentric dead load of 0.2 k/ft; #5 Grade 60 bars at 16 inches; f'_m=2000 psi.

Required: Determine maximum out-of-plane load using 2013 TMS 402 Section 9.3.5.4.2. Calculate shear stress at grout/face shell interface.

Solution: Based on a spreadsheet calculation, the maximum out-of-plane load is 40.7 psf. Check this value. Use load combination 0.9D+E. The spreadsheet checks all load combinations and for higher axial loads, 1.2D+E will often control.

Use a wall weight of 81 psf (ASCE 7, 125 pcf units)
Based on an out-of-plane load of 72.6 psf, determine S_{DS}.

$0.4S_{DS}(\text{weight}_{wall})$ = 72.6 psf (ASCE 7, Section 12.11.1) $\quad S_{DS} = 2.24$

$P_u = P_{uw} + P_{uf} = (0.9\text{-}0.2S_{DS})[(81\text{psf})(20/2)\text{ft} + 200\text{lb/ft}] = 456\text{ lb/ft}$

For fully grouted wall, A_n = 91.5 in^2/ft; I_n = 443.3 in^4/ft (from NCMA TEK 14-1B)
Find M_{cr}. Modulus of rupture, f_r = 163 psi.

$$M_{cr} = \left(f_r + \frac{P}{A_n}\right)\left(\frac{I_n}{t_{sp}/2}\right) = \left(163\text{ lb/in.}^2 + \frac{456\text{ lb/ft}}{91.5\text{ in.}^2\text{/ft}}\right)\left(\frac{443.3\text{ in}^4\text{/ft}}{7.63/2\text{ in.}}\right) = 19{,}520\text{ lb} - \text{in./ft}$$

Find I_{cr} (2013 TMS 402 Equations 9-34 and 9-35).

A_s = 0.31in^2/16in.(12in./ft) = 0.232in^2/ft

$n = E_s/E_m$ = 29000000psi/(900×2000psi) = 16.1

$$c = \frac{A_s f_y + P_u}{0.64 f_m' b} = \frac{0.232\ \text{in}^2/\text{ft} \times 60{,}000\ \text{psi} + 456\ \text{lb/ft}}{0.64 \times 2{,}000\text{psi} \times 12\text{in./ft}} = 0.936 in.$$

$$I_{cr} = n\left(A_s + \frac{P_u}{f_y}\frac{t_{sp}}{2d}\right)(d-c)^2 + \frac{bc^3}{3}$$

$$= 16.1\left(0.232\ \tfrac{\text{in.}^2}{\text{ft}} + \frac{456\ \frac{\text{lb}}{\text{ft}}}{60000\ \frac{\text{lb}}{\text{in}^2}}\right)\left(\frac{7.63\ \text{in.}}{2} - 0.936\ \text{in.}\right)^2 + \frac{12\ \frac{\text{in.}}{\text{ft}} \times 0.936^3\ \text{in.}^3}{3} = 35.2\ \text{in.}^4/\text{ft}$$

Use solution to simultaneous equations of 2013 TMS 402 Equations 9-27 and 9-29 to find M_u. Since the axial load is concentric, $e_u = 0$.

$$M_u = \frac{\dfrac{w_u h^2}{8} + P_{uf}\dfrac{e_u}{2} + \dfrac{5M_{cr}P_u h^2}{48E_m}\left(\dfrac{1}{I_n} - \dfrac{1}{I_{cr}}\right)}{1 - \dfrac{5P_u h^2}{48E_m I_{cr}}}$$

$$= \frac{\dfrac{72.6\text{psf}\,(20\text{ft})^2}{8}\ \tfrac{12\text{in}}{\text{ft}} + \dfrac{5\left(19{,}520\ \frac{\text{lb-in}}{\text{ft}}\right)\left(456\ \frac{\text{lb}}{\text{ft}}\right)(240\ \text{in})^2}{48(1{,}800{,}000\ \text{psi})}\left(\dfrac{1}{443.3\ \frac{\text{in}^4}{\text{ft}}} - \dfrac{1}{35.2\ \frac{\text{in}^4}{\text{ft}}}\right)}{1 - \dfrac{5\left(456\ \frac{\text{lb}}{\text{ft}}\right)(240\ \text{in})^2}{48(1{,}800{,}000\ \text{psi})\left(35.2\ \frac{\text{in}^4}{\text{ft}}\right)}} = 44600\ \frac{\text{lb}-\text{in}}{\text{ft}}$$

Compare to capacity, 2013 TMS Commentary 9.3.5.2.

$$a = \frac{A_s f_y + P_u/\phi}{0.80 f_m' b} = \frac{0.232\ \frac{\text{in.}^2}{\text{ft}}(60000\ \text{psi}) + 456\ \frac{\text{lb}}{\text{ft}}/0.9}{0.80(2000\ \text{psi})\left(12\ \frac{\text{in}}{\text{ft}}.\right)} = 0.751\text{in.}$$

$$M_n = \left(P_u/\phi + A_s f_y\right)\left(d - \frac{a}{2}\right)$$

$$= \left(456\ \tfrac{\text{lb}}{\text{ft}}/0.9 + 0.232\ \tfrac{\text{in.}^2}{\text{ft}}(60000\ \text{psi})\right)\left(3.81\text{in.} - \frac{0.751\text{in.}}{2}\right) = 49500\ \frac{\text{lb}-\text{in}}{\text{ft}}$$

ϕM_n = 0.9(49500 lb-in/ft) = 44600 lb-in/ft = M_u = 44600 lb-in/ft

This checks, and the maximum out-of-plane load the wall can carry is 72.6 psf.
Based on an out-of-plane load of 72.6 psf, the factored shear force is 72.6 psf(10ft) = 726 lb/ft.

Determine the shear stress.

$$f_v = \frac{A_s f_y}{b(h/2)} = \frac{0.232\ \frac{\text{in}^3}{\text{ft}}(60000\text{psi})}{12\ \frac{\text{in}}{\text{ft}}\left(\frac{240\text{in.}}{2}\right)} = 9.6\text{psi}$$

The first set of results examines an 8 inch CMU wall with #5@16 inches. The wall height and the wall axial load were varied. The maximum axial load was 5 kip/ft. This is a high axial load for most masonry structures, and there would typically only be a load this high in a multi-story bearing wall building. Above an axial load of 5 kip/ft, the equivalent rectangular stress block would no longer be in the face shell. If part of the equivalent rectangular stress block were in the grouted core, the shear stress at the interface would be reduced. Note that the shear stress is constant for a given height since the shear stress is just a function of the yield force in the reinforcement.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
20	7.625	0.2	72.6	5	16	2000	726	9.6
20	7.625	1	72.5	5	16	2000	725	9.6
20	7.625	5	62.4	5	16	2000	624	9.6
16	7.625	0.2	113.4	5	16	2000	907	12.0
16	7.625	1	114.0	5	16	2000	912	12.0
16	7.625	5	116.5	5	16	2000	932	12.0
12	7.625	0.2	198.2	5	16	2000	1189	16.0
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	5	186.1	5	16	2000	1117	16.0
8	7.625	0.2	425.4	5	16	2000	1702	24.0
8	7.625	1	392.7	5	16	2000	1571	24.0
8	7.625	5	298.6	5	16	2000	1194	24.0

In looking at these results, analysis shows that the shear stress increases as wall height decreases. The highest shear stress is 24 psi, which is for an 8 ft tall wall. An 8 ft wall is very short, and most masonry walls are at least 10 ft high. This shear stress was also for an out-of-plane load of at least 299 psf, an unrealistically high out-of-plane load.

The second set of results is for varying f'_m with the height and axial load held constant at 12 ft and 1 k/ft, respectively. The primary effect of f'_m is on the out-of-plane load. The shear stress remains constant as it is just a function of the yield strength of the reinforcement, and the height of the wall.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	1	199.2	5	16	2500	1195	16.0
12	7.625	1	189.9	5	16	1500	1139	16.0

The third set of results is for the bar spacing increasing from 16 inches up to 32 inches with the height and axial load held constant at 12 ft and 1 k/ft, respectively.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	195.8	5	16	2000	1175	16.0
12	7.625	1	139.1	5	24	2000	835	10.7
12	7.625	1	110.3	5	32	2000	662	8.0

The fourth set of results is for varying eccentricity, e, of the axial load at the top of the wall. The bar spacing is 16 inches and the height and axial load are held constant at 12 ft and 1 k/ft, respectively. Increasing eccentricity decreases the shear force.

Height (ft)	t (inch)	Axial (k/ft)	e (inch)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	7.625	1	0	195.8	5	16	2000	1175	16.0
12	7.625	1	3	190.1	5	16	2000	1141	16.0
12	7.625	1	12	172.9	5	16	2000	1037	16.0

The fifth set of results is for a 12 inch CMU wall with the bars offset (*d*=9.5 inches). This would increase the flexural strength and the out-of-plane load on the wall would increase. Again, a 12 ft high wall with 1 kip/ft axial load was used, and the reinforcement spacing was varied. The shear stress again is 16 psi for a 16 inch spacing of the reinforcement, but in this case a 489 psf (3.4 psi) out-of-plane load is required to develop the shear stress of 16 psi. There is no realistic scenario for that level of loading.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
12	11.625	1	263.5	5	32	2000	1581	8.0
12	11.625	1	339	5	24	2000	2034	10.7
12	11.625	1	488.9	5	16	2000	2933	16.0

The final set of results is for a 32 ft high wall. Due to the height of the wall, #6 vertical reinforcement at 16 inches is used in order to carry the out-of-plane load. The shear stress is only 8.6 psi.

Height (ft)	t (inch)	Axial (k/ft)	w_u (psf)	Bar Size (#)	Bar Spacing (inch)	f'm (psi)	Shear (lb)	Shear stress (psi)
32	7.625	1	25.7	6	16	2000	411.2	8.6
32	7.625	1	29.8	6	16	3000	476.8	8.6

To summarize, the analyses made several conservative assumptions, resulting in a very conservative analysis. To review, the conservative assumptions were:

1. The axial load is considered to act concentrically, resulting in the largest shear force for a given moment capacity.
2. The wall is loaded to the maximum out-of-plane that it can carry. Typically, due to discrete reinforcement sizes and spacings, and prescriptive reinforcement requirements, walls are not loaded to the maximum out-of-plane capacity.
3. Any interlocking due to offset webs, block taper, etc. was neglected. The shear surface was considered to be planar.

Even with a very conservative analysis, the maximum shear stress was only 24 psi. The 24 psi was for an 8 ft high wall with unrealistically high out-of-plane loads. Under typical load conditions, the shear stress was 16 psi or less. This shear stress is much less than the 100 psi that was the initial arbitrary California requirement, and also much less than $2.5\sqrt{f'_m}$, which would be about 97 psi for f'_m=1500 psi and 112 psi for f'_m =2000 psi.

Based on the above results, two conclusions can be drawn.

1. No core testing is required. The shear stresses are very low. Additionally, the above analysis does not consider the benefit of the homogeneous concrete masonry unit which has a continuous connection between the cross web and face shell
2. TMS 402 is justified in not requiring designers to check the shear stress at the grout/face shell interface. That will not control the design.

This issue of "Masonry Chronicles" was written by:

Richard Bennett, PhD, PE
Professor, Civil and Environmental Engineering
The University of Tennessee, Knoxville
Chair, 2016 TMS 402/602 Code Committee

About the Author:

RICHARD M. BENNETT, PhD, PE, is a professor of Civil and Environmental Engineering, and the Director of Engineering Fundamentals, at the University of Tennessee, Knoxville. He received his PhD from the University of Illinois, Urbana-Champaign. He served as Chairman of the Flexural and Axial Loads Subcommittee of the TMS 402/602 Code Committee from 2004 to 2010. From 2010 to 2013, he was the Vice-Chair of the Main committee and is currently Chair of the 2016 TMS 402/602 Committee.

Concrete Masonry Association
of California and Nevada
6060 Sunrise Vista Drive, Suite 1990
Citrus Heights, CA 95610
(916) 722-1700
info@cmacn.org
www.cmacn.org

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