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Shear Capacity of Pre-insulated Reinforced Concrete Masonry Walls

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ABSTRACT

In the United States, pre-insulated concrete masonry units (CMU) for single-leaf (single-wythe) have become more popular in the past decade due to their thermal performance. However, thermal considerations, coupled with broader objectives to increase operational energy efficiency, have driven the development of an array of various integrally insulated concrete masonry units that can be used with either partially grouted walls or fully (solid) grouted walls. These units were made possible by changes to ASTM C90, Standard Specification for Loadbearing Concrete Masonry Units, which now allows reduced web areas connecting the face shells. For the assemblies evaluated in this paper, the web height for the partially-grouted units is reduced only at the insulation inserts, while the full width of the webs is reduced for the fully-grouted units. In either case, however, the reduced webs increase the possibility of web shear failures.

This paper will present shear testing results from research that demonstrates such reduced-web units, when reinforced and partially- or fully-grouted, exhibit performance like the design models stipulated in TMS 402, *Building Code Requirements for Masonry Structures*, which were originally developed for uninsulated, full web height CMU. In addition, web shear test results will be presented and compared to ASTM and TMS design criteria.

KEYWORDS

Integrally-insulated, Concrete Masonry, Shear Strength, Web Shear, Partially Grouted, Insulation Inserts.

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INTRODUCTION

In 1971, CBIS/Korfil® first introduced insulation inserts in the United States which are now distributed by the Concrete Products Group [1] for use in the hollow cells of standard two-cell, three-web concrete masonry units (CMU). The inserts are manufactured from expanded polystyrene (EPS) insulation and are factory-installed in the CMU. They improve the thermal performance of the masonry, particularly single-leaf (single-wythe) walls. Some other benefits include reduced sound transmission, elimination of site-installed insulation, ease of handling, and the ability to place non-structural components (conduit, pipes, ducts, and other utilities) in grouted and ungrouted cells without interrupting the insulation.

Initially, the inserts were U-shaped (see Fig. 1) with a notched web. They were sized to fit in standard 2-cell CMU (see Fig. 2) and intended to remain in place regardless of whether the cell containing the insert was grouted or not grouted. To highlight the insert profile within an assembly, Figure 2 shows the insert as would be seen when viewed from the bottom of a unit. When viewed from the top, the insert would not be visible along the length of the web due to the presence of the insert notch that was molded into the insert to facilitate handing the units with the center web. Structurally, the inserts acted as a bond breaker; the only contact interface between the grout and the CMU occurred at one web of a grouted cell and the insert notch. The CMU is laid in either a running bond or a stack bond pattern. Due to their geometry, the inserts are only used in the vertical cells and not placed in horizontal bond beams.

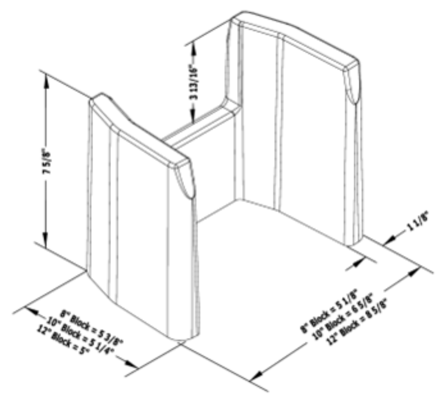

Figure 1: Expanded Polystyrene (EPS) Insulation Insert

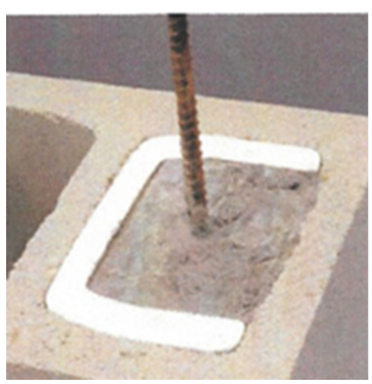

Figure 2: EPS Insert Inverted and Placed in Grouted Cell

The introduction of inserts was a major step in reducing thermal transmission loss through the assembly by limiting thermal bridging through the face shell-grout-face shell pathway. Depending on the density, thickness, and configuration of the concrete masonry units and the type of insulation insert used, these systems have the potential to increase the thermal resistivity of a concrete masonry assembly 2.3 to 7 times over comparable uninsulated masonry assemblies [10]. In the decades since their introduction, numerous improvements have been made to these systems, including refinements to the inserts and the development of modified concrete masonry units per ASTM C90 [2]. Today, several manufacturers have products that utilize insulation inserts. This paper addresses walls constructed using the CBIS/Korfil® inserts incorporated into specially designed HI-R® and HI-R-H™ concrete masonry units that were created to accept the insulation inserts. However, the resistance mechanisms present in this system are representative of a broad class of insulated masonry systems that feature polystyrene insulation inserts that are placed in direct contact with grout on some faces and concrete masonry on others. However, the extrapolation of these results to other proprietary integrally-insulated systems given the wide variations in unit and insert configurations should be limited to a notional model of friction resistance in cases where polystyrene inserts and grout reside in cavities of concrete masonry units. Additional testing would be required to determine the resistance of such units. The two proprietary units evaluated in this paper are shown in Figure 3. The two-web unit (Fig. 3a) is used for partially grouted masonry construction, whereas both units shown may be used for fully grouted construction.

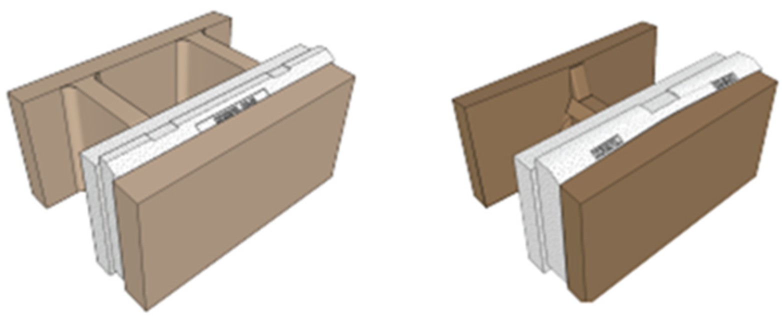

Figure 3: Integrally-Insulated CMU a) HI-R® (PG), b) HI-R-H™ (FG)

Results of a comprehensive physical testing program [3, 4], demonstrated masonry assemblies containing insulation inserts and constructed using HI-R® (designated PG – partially grouted here) and HI-R-H™ (designated FG – fully grouted here) units perform compositely and are like conventionally grouted and reinforced masonry construction subjected to similar loading conditions. In addition, reinforcement lap splice tests were provided and compared to the detailing and performance criteria of TMS 402, Building Code Requirements for Masonry Structures [5]. The conclusions of this investigation validated the use of the TMS 402 provisions when designing and detailing these specific integrally insulated concrete masonry assemblies. This paper provides further details from that testing program specifically related to the in-plane performance of the wall assemblies as well as the web-shear capacity of the reduced height webs.

In-Plane Shear Test Program and Observations

Test Panels

Three panels, three units wide were constructed and tested following the requirements of ASTM E519/E519M, *Standard Test Method for Diagonal Tension (Shear) in Masonry Assemblages* [6] Two panels were of identical configuration, constructed using the 25 cm (10 in.) PG units containing 64 mm (2.5 in.) of insulation and partially grouted. The remaining panel was constructed using the 30 cm (12 in.) FG units with 89 mm (3.5 in.) of insulation and solid grouted. The selection of these test panels was driven by the desire to a) confirm the strength and performance of these assemblies relative to the design provisions of TMS 402; and b) given that this test program was focused on validating the strength and performance of these systems, a robust text matrix consisting of multiple repeat and duplicate specimens was felt to be unwarranted. As the goal of this testing was to verify the masonry contribution to the shear strength of an assembly, none of the test specimens contained reinforcement.

The nominal length and height of each panel was 1.2 x 1.2 m (48 x 48 in.) constructed in a typical running bond pattern using face shell mortar bedding. As the PG panels were partially grouted, the cross-webs adjacent to the grouted cells were mortared to confine the grout. For all three panels, the insulation inserts were removed from the loaded corners before grouting to provide a solid bearing location for the loading shoes; otherwise, insulation inserts were maintained in their specified location across the interior of the panels. A schematic of the PG panels is illustrated in Figure 4. The FG panel was similar but fully grouted.

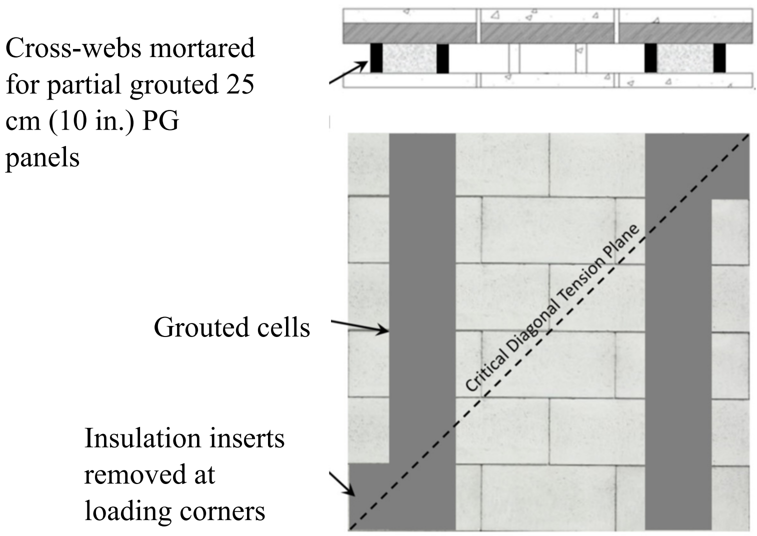

Figure 4: ASTM E519/C519M Diagonal Tension PG Panel Schematic

Each panel was fitted with a loading shoe at opposite corners and positioned within the loading frame such that the centroid of the applied load was located at the geometric centroid (not the mass centroid) of the panel (Fig. 5). This was done to impart load into the masonry on both sides of the insulation inserts in roughly equally proportional magnitudes. Elastic shortening along the compression axis and elastic elongation along the tension axis of the panels were measured using both analog and digital displacement gauges. This dual measurement combination provided higher resolution at lower strains as well as backup measurements in the event one or more gauges was dislodged because of cracking during testing.

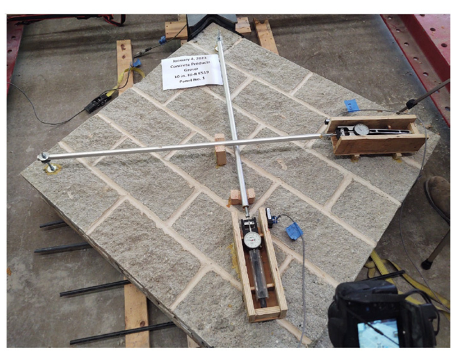

Figure 5: Diagonal Tension Panels Following Grouting and Before Testing

Material Properties

Throughout testing, material properties were determined for the units per ASTM C140 [7] (see Table 1), Type S mortar per ASTM C780 [8], and coarse grout per ASTM C1019 [9] (see Table 2). Given the calendar time required to complete all tests, these material properties were checked periodically as testing progressed to determine if the additional curing time changed the material's compressive strength. The concrete masonry units were produced nearly 12 months before specimen construction. As such, their compressive strength remained constant throughout testing. Likewise, the mortar compressive strength varied little from the beginning to the end of testing, with an average compressive strength of 11.6 MPa (1,690 lb./in.^{2}) and a range of 0.6 MPa (90 lb./in.^{2}). The grout compressive strength did vary throughout testing, however, as reflected in Table 2. This variation in grout compressive strength was subsequently considered during the analysis of the specimens' performance.

In Table 1, the 25 cm (10 in.) units had one split face resulting in one face shell being slightly thicker than the other. While the presence of the split surface does not have any impact on the results of these tests, the two face shell thicknesses are reported separately here as they are considered separately in subsequent analyses. The difference in the face shell thicknesses of the 30 cm (12 in.) units is negligible. The compressive strength of the 25 cm (10 in.) PG units was higher than preferred for this investigation; however, these units are representative of products available from a producer near the research facility.

Table 1: Properties of Concrete Masonry Units

Measured Property	25 cm (10 in.) PG Units	30 cm (12 in.) FG Units
Compressive Strength (f'_u) MPa (lb./in.^{2})	48.5 (7,030)	23.2 (3,370)
Absorption, kg/m^3 (lb./ft^3)	95 (5.9)	146 (9.1)
Density, kg/m^3 (lb./ft^3)	2,308 (144.1)	2,154 (134.5)
Face Shell Thicknesses, mm (in.)	45.2/50.8 (1.78/2.00)	40.9 (1.61)
Normalized Web Area, mm^2 (in.^{2})	9,161 (14.2)	7,613 (11.8)

Table 2: Properties of Coarse Grout

Sample	Cure Time	Average Compressive Load, kN (lb.)	Average Compressive Strength, MPa (lb./in.2)
1	25 Days	314.7 (70,760)	45.1 (6,540)
2	28 Days	325.1 (73,090)	47.4 (6,870)
3	45 Days	349.8 (78,630)	51.6 (7,490)

Net Shear Area of Panels

The critical diagonal tension plane shown in Figure 4 was used in determining the net shear area (A_{NV}) for each set of panels, which in turn was used in comparing the tested shear stresses to the design shear stresses of TMS 402. For the partially grouted 25 cm (10 in.) PG panels, the net shear area is the sum of the areas of the two face shells, the area of the webs, and the area of grout coincident with the assumed failure plane (discounting any areas where an open cell or insulation insert bisects this surface). The calculated net shear area is 2,816 cm^2 (436.5 in.^{2}) for the 25 cm (10 in.) panels. Similarly, the net shear area of the fully grouted (FG) panels is the length of the diagonal between the loaded corners multiplied by the panel thickness minus the thickness of the insulation, resulting in a net shear area of 3,532 cm^2 (547.6 in.^{2})

Diagonal Tension Test Results

Each of the three diagonal tension test panels was loaded as required by ASTM E519/E519M [6] and deformations parallel to the principal compression and tension axes were measured. The corresponding gauge length over which the compressive and tensile deformations were measured averaged 109 cm (43 in.) for all panels. A peak failure load was achieved for both partially grouted panels. However, a peak load was not reached with the solid grouted (FG) specimen as the panel's strength exceeded the capacity of the loading equipment. These results are summarized in Table 3. Figure 6 shows the failure of one of the PG panels.

Figure 6: PG Shear Failure

In addition to the prototypical stair-step crack that developed between the two loading corners, some webs of each PG panel fractured as the panels failed. It is difficult to assess the sequence of the internal cracking as these unreinforced specimens would exhibit little to no external distress immediately before failure, which would suddenly cascade across the specimens. From a video of the tests, a diagonal crack was seen forming along the mortar joints in the moments before peak load was reached with little to no cracking elsewhere in the panels. Once this crack propagated across the full diagonal length of the specimens, the two separate wedges of the panel would abruptly rotate out and concurrently release the built-up strain in the panel. While there is no definite cause/effect observable, given that neither of the PG panels exhibited load loss before impending failure it is likely some, if not all, of the web failures occurred after the formation of the diagonal crack.

The calculated nominal values for diagonal shear per TMS 402, Section 9.2.6 are $V_{\text{TMS402}} = 0.386$ A_{nv} ($56A_{\text{NV}}$) for partially grouted sections such as the PG panels and $V_{\text{TMS402}} = 0.620$ A_{nv} ($90A_{\text{NV}}$) for the fully grouted (FG) panel.

Table 3: Summary of Diagonal Tension Testing

Panel	A_{NV} cm^2 (in.^{2})	Peak Load kN (lb)	$V_{\text{TMS 402}}$ kN (lb)	Peak Load /$V_{\text{TMS 402}}$
1 (PG)	2,816 (436.5)	169.3 (38,060)	108.7 (24,444)	1.56
2 (PG)	2,816 (436.5)	175.5 (39,460)	108.7 (24,444)	1.61
3 (FG)	3,532 (547.6)	418.1 [A] (94,000)	219.2 (49,284)	1.91

[A] Corresponds to the peak load applied. Specimen failure did not occur.

Considering the test results and observations of the diagonal tension testing undertaken as part of this project, the application of the TMS 402 shear strength design provisions to the PG and FG systems is both supported and conservative. Further, the presence of a layer of insulation should not be considered a differentiator between partial and solid grouted construction in the context of in-plane shear design under the provisions of TMS 402. Instead, the presence of a continuous grouted core within the assembly would be the sole trigger for the application of the partial or solid grouted design provisions of TMS 402.

WEB-SHEAR TEST PROGRAM AND OBSERVATIONS

Test Specimens and Testing Protocols

In the development of the research variables associated with this investigation, it was anticipated that the strength and performance of the PG and FG systems could potentially be controlled by web failure mechanisms, particularly for assemblies subjected to out-of-plane loading conditions. This, coupled with the web shear checks of TMS 402 when units incorporating reduced-sized webs are used, prompted additional testing to explore the strength of the webs of the FG units subjected to a direct shearing force. These web shear tests used a modified version of ASTM C482, Standard Test Method for Bond Strength of Ceramic Tile to Portland Cement Paste [10]. For these tests, the web/face shell interface at the exterior (insulation) side of the unit was saw-cut from a full-size unit, restrained within the shear jig (Fig 7a) and loaded in direct shear (Fig. 7b) until failure occurred (Fig. 7c). It should be emphasized that this direct shear loading protocol does not generate shear stresses directly analogous to the flexural-induced (transverse) shear stresses in the webs of the panels loaded out-of-plane. Nevertheless, given the difficulty in conducting material shear testing without introducing other confining effects that influence the test results, this setup is considered reasonable for benchmarking purposes.

Web Shear Test Results

Table 4 summarizes the test results conducted on the individual face shell/web interface specimens. Most of the specimens exhibited a dual failure mechanism where a crack would develop at the interface of the web and face shell that did not correspond to the maximum load applied, an example of which is shown in Figure 7b. Initially, this crack did not fully propagate through the interface but instead turned into the face shell where this failure plane encountered the confining compression stresses at the base of the specimen resulting in the compound failure plane illustrated in Figure 7c. For this discussion, failure is deemed to have occurred once the first crack appeared, which also coincided with a momentary drop in the applied load.

a) FG Test Specimen **b) Onset of Web-Shear Failure** **c) Web Shear Failure**

Figure 7: Web Shear Testing

Table 4: Web Shear Test Results of 30 cm (12 in.) FG Units

Specimen	Average Web Width, mm (in.)	Average Web Height, mm (in.)	Load at First Crack, kN (lb)	Maximum Applied Load, kN (lb)	Direct Shear Stress[A], MPa (lb/in.^{2})
1	61.6 (2.426)	113.2 (4.458)	28.9 (6,500)	29.9 (6,730)	4.1 (601)
2	61.7 (2.428)	113.6 (4.474)	30.2 (6,800)	31.8 (7,160)	4.3 (626)
3	60.9 (2.398)	114.9 (4.522)	29.8 (6,700)	36.8 (8,270)	4.3 (618)
4	61.7 (2.429)	115.7 (4.555)	27.2 (6,120)	27.2 (6,120)	3.8 (553)
5	61.5 (2.423)	113.6 (4.475)	27.8 (6,260)	33.6 (7,550)	4.0 (577)
Average	61.5 (2.422)	114.2 (4.497)	28.8 (6,480)	31.9 (7,170)	4.1 (595)

[A]Shear stress calculated as the load at first crack divided by the average web/face shell interface area.

Transverse Shear Strength

For full assembly composite action, the webs, grout, or some combination of these components must be able to transfer stresses between the face shells of the units. As one design shear check, TMS 402 Section 9.2.6.2 limits the nominal web shear strength to:

$$(1)\quad V_N = V_{TMS\ web\ shear} = \frac{0.316\sqrt{f'_m}(I_N)(b_{web})}{Q} \qquad \left(\frac{3.8\sqrt{f'_m}(I_N)(b_{web})}{Q}\right)$$

As previously reported [4], the average failing shear load for the FG test panels subjected to out-of-plane (transverse) loading was 58.7 kN (13,190 lb). Due to the differences in how the direct shear stresses per Table 4 and the transverse shear stresses of the out-of-plane loaded panels are generated, directly comparing these shearing stresses should be approached with caution. Further, the critical shear plane for the transversely loaded out-of-plane panels is taken at the inside face of the insulation (e.g., the web/grout interface, not the web/face shell interface as was done for these direct shear specimens) as web/grout interface location would couple the largest transverse shearing forces with the smallest area of masonry available to resist these forces. This comparison does, however, provide assurances that the provisions of TMS 402 can be safely applied to the PG and FG systems.

The material and section properties of the FG panels loaded out-of-plane are summarized in Table 5 [4]. Of note, the equivalent web thickness reported in Table 5 is not simply the measured web thickness as the webs of the units of this investigation units do not extend over the full height of the units. In checking the shear in the webs, an equivalent web thickness must be used by proportionally reducing the web thickness based on the actual web height. This is reflected in the calculation of the equivalent web thickness per course shown in Table 5 by dividing the total web area per course by the nominal course height (20 cm or 8 in.).

Table 5: Out-of-Plane Panel Properties for Web Shear of 30 cm (12 in.) FG Units

Property		30 cm (12 in.)
Unit Compressive Strength (f'_u),	MPa (lb/in.^{2})	23.2 (3,370)
System Compressive Strength (f'_m)	MPa (lb/in.^{2})	20.3 (2,950)
Moment of Inertia, I_N, per Course	mm^4 (in.^{4})	1,923 x 10^6 (4,622)
First Moment of Area, Q , per Course	mm^3 (in.^{3})	7.6 x 10^6 (463.3)
Measured Web Thickness,	mm (in.)	58.9 (2.32)
Measured Web Height,	mm (in.)	114.3 (4.50)
Total Web Area per Course,	mm^2 (in.^{2})	20,206 (31.32)
Equivalent Web Thickness per Course (b_{web}),	mm (in.)	99.6 (3.92)

Rearranging Eq. (1), the peak transverse shear stresses based on the average failing shear load of the out-of-plane loaded specimens can be calculated as follows:

$$(1)\ f_{MAX} = \frac{(V_{MAX})(Q)}{(I_N)(b_{web})} = \frac{(58{,}700)(7.6\text{x}10^6)}{(1{,}923\text{x}10^6)(99.6)} = 2.32\text{ MPa} \quad \left(337\,\frac{\text{lb}}{\text{in}^2}\right)$$

Again, while not directly comparable, this average peak transverse shear stress aligns reasonably well with that of Table 4. For direct comparison to the nominal shear strength stipulated by TMS 402 and applying the properties given in Table 5, Eq. (1) becomes:

$$(2)\ V_N = V_{TMS-f'_m} = \frac{0.316\sqrt{20.3}(1{,}923\text{x}10^6)(99.6)}{7.6\text{x}10^6} = 35.9\text{ kN }(8{,}070\text{ lb})$$

Comparing this to the average tested transverse shear strength of the test panels (58.7 kN (13,190 lb)) the ratio of the tested-to-nominal shear strength is 1.64, indicating that the TMS 402 shear checks are reasonably conservative for application to the units testing in this study. Given that the transverse shear strength is actually governed by the webs, a reasonable argument would be to use the compressive strength of the units, not the assembly strength, in determining the nominal shear strength. Replacing the compressive strength of the masonry with the compressive strength of the unit in the above expressions yields:

$$(3)\ V_N = V_{TMS-f'_u} = \frac{0.316\sqrt{23.2}(1{,}923\text{x}10^6)(99.6)}{7.6\text{x}10^6} = 38.4\text{ kN }(8{,}620\text{ lb})$$

Applying the above metric for the nominal assembly shear strength, the ratio of the tested-to-nominal shear strength would be 1.53, slightly less conservative than would be indicated by applying the assembly compressive strength, but still reasonably conservative.

CONCLUSIONS AND SUMMARY

This project investigated multiple key strength and performance attributes of masonry constructed using reduced web concrete masonry units through the lens of verifying and benchmarking these properties against those defined by TMS 402. The testing of the PG and FG systems for in-plane shear and web shear validates the use of the TMS 402 criteria with these systems.

One unique aspect of the performance of the specimens evaluated in this project was the observation of web shear failures, particularly with the out-of-plane flexure specimens. While web shear (or transverse shear) failure is possible with many structural systems, reducing the area of the webs does increase the possibility of this limit state. While web shear should be checked for cases defined by TMS 402, it is not expected to control the design often.

The results of this investigation identified two possible clarifications warranting additional discussion by TMS 402. First, the transverse web shear checks are technically only triggered for unreinforced masonry assemblies. Given that this failure mechanism is possible for reinforced and unreinforced assemblies, consideration should be given to expanding this design check. Second, the nominal transverse shear strength defined by TMS 402 is governed by the assembly compressive strength (f'_m). Given that the grout is not engaged when the webs of these assemblies fail in shear, consideration should be given to replacing the nominal shear strength calculated by Eq. (1) with the smaller of the assembly strength or unit strength. While this appears to be inconsistent, it does provide a degree of conservatism since f'_m is always less than f'_u. This could be explored with further testing.

ACKNOWLEDGMENTS

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