BEHAVIOR OF COMPOSITE BRICK WALLS

By

M. L. PORTER
Professor, Civil Eng.,
Iowa State University
Ames, Iowa

A. M. WOLDE-TINSAE
Assoc. Professor, Civil Eng.,
Univ. of Maryland
College Park, Maryland

M. H. AHMED
Lecturer,
Assuif Univ.
Cairo, Egypt

ABSTRACT

The results of testing five brick-to-brick reinforced composite masonry panels subjected to gravity and in-plane shear loads are discussed herein. Each wall contained two wythes with a nominal 50.8 mm (2-in.) collar joint. This joint was grouted and reinforced with either welded wire fabric or vertical and horizontal bars. The loads (either vertical or horizontal) were applied as distributed loads along the top of the wall (which was free to move), with the base fixed. The wall panels were approximately 1.22 m (4 ft) wide, 1.83 m (6 ft) high and 22.9 cm (9 in.) thick. Strength characteristics of the tested walls are compared herein.

PREVIOUS RELATED WORK

In 1959, Schneider [1] evaluated the behavior of the three basic types of masonry units forming integral parts of a composite masonry wall. The tested units were clay brick, concrete block, and shel-brick. These walls were reinforced with vertical and horizontal bars. When loaded with in-plane loads at the upper end, the walls failed through diagonal tension. Schneider concluded that:

1. the reinforced grouted masonry was highly recommended for resisting lateral in-plane loading;

2. the ultimate shear index (i.e., the shear force divided by the gross area of the wall) was 985 kPa (143 psi) with a height-to-width ratio approximately equal to one; and

3. the type of mortar mix used had little effect on the overall shear resistance of brick walls.

Meli [2] (1974) tested 56 walls, of about 2 m × 2 m (6'-6"×6'-6"), built on stiff concrete beams. The walls were either brick or concrete block, vertically reinforced through holes in the units. The walls were tested as cantilevers fixed at the base and free at the top. The loads were applied in-plane with or without precompression applied before the horizontal load or in cycles of alternate loads. For walls with low vertical reinforcement ratios and low vertical precompression, failure was governed by flexure (horizontal crack at the bed joints). The behavior of these walls was similar to that of an underreinforced concrete beam. Precompression of this type of wall caused a small increase in horizontal strength, compared to a large increase in vertical stresses. Failure was governed by diagonal shear for walls with high vertical reinforcement ratios. The cracking loads increased because of precompression of approximately 40% of the total applied vertical load (providing that this vertical load did not exceed one-third of the ultimate vertical strength of the wall). The cracks gradually formed through the joints.

Meli concluded that in walls with interior vertical reinforcement, behavior was nearly elastoplastic with remarkable ductility; i.e., failure was governed by

bending. If failure was governed by diagonal cracking, ductility was small and behavior was brittle when high vertical loads were applied.

Hatzinikolas et al. [3] (1978) tested the effect of joint reinforcement on the vertical load capacity of masonry walls built of hollow concrete blocks and loaded vertically (only). They concluded that joint reinforcement produced stress concentration, reducing the ultimate load bearing capacity of the wall.

Williams and Geschwindner [4] (1982) studied shear transfer between a concrete masonry wythe and a brick wythe. The shear transferred through a 0.95 cm (3/8-in.) collar joint of three different mixes of mortar and grout. The results of their tests of assemblies with and without joint reinforcement proposed an equation for shear-bond strength in the collar joint:

$$v_{SB} = 38.9 + 0.0103\ f'_m \tag{1}$$

where:

v_{SB} = the shear-bond strength (psi with 1 psi = 6.89 kPa)

f'_m = the ultimate compressive strength of the collar joint (psi with 1 psi = 6.89 kPa).

Schrivener [5] (1982) tested six full-scale reinforced brick masonry walls of two wythes 10.2 cm (4 in.) each, having a cavity of about 3.6 cm (1.4 in.) filled with mortar and reinforced with vertical and horizontal bars. The walls were constructed on a reinforced concrete beam and topped with another beam. The walls were loaded in-plane at the top until failure occurred when joints near the base first cracked because of excessive tensile stresses caused by the bending moment, followed by a major diagonal crack accompanied by vertical splitting cracks. Schrivener concluded that the first shear cracks were consistent for all walls, with an average value for lateral loads of 328 kPa (47.6 psi), and that no effect was attributed to different aspect ratios. Maximum shear strengths were in the range of 342 to 546 kPa (49.6 to 79.3 psi).

A finite element study of the interfacial forces was performed by Anand and Young [6]. They studied the magnitude and distribution of the shear-bond stresses, particularly for cases where the load was applied to the interior wythe of block masonry of a composite wall. They found that the shear stresses were principally transferred in the upper portion of the wall near the point of load application.

COMPOSITE WALL TESTS

Specimens

Five composite brick-to-brick walls were built in three different stages. Walls of two wythes, with a nominal 5.1 cm (2 in.) reinforced collar joint, were tested. All walls were approximately 1.83 m (6 ft) high and 1.22 m (4 ft) wide, were built either on a steel T-section (WT6×32.5) or on a steel plate 3.18 cm (1.25 in.) thick. Straight coil loops were welded to the steel base and positioned to align with the holes of the masonry. The holes of the first two layers of the brick wythes were grouted to bond the coil loops to the masonry, after which the walls were cured in accordance with ASTM-E447-74 [7] and tested after at least 28 days. The walls were designated as W2, W4, W6, W8 and W10. Table 1 indicates the age of each wall and its amount of reinforcement. These brick-to-brick walls were part of an overall investigation that also included brick-to-block walls for a total of 11 specimens.

Table 1. Test age and reinforcement details for brick-to-brick walls.

Wall			Reinforcement	
Group	No.	Test Age (days)	Vertical	Horizontal
I	W2	80		WWF* 4×4×4×4
II	W4	66		WWF 4×4×4×4
II	W6	76		WWF 4×4×4×4
III	W8	37	1#3 & 2#4 bars†	5#3 bars
III	W10	38	1#3 & 2#4 bars	4#2 bars & truss joint‡

*Welded wire fabric consisted of No. 4 gage horizontal and vertical wires.
†The vertical reinforcement was welded to the steel base.
‡The truss joint was 3.2 mm (1/8 in.) thick, 14.3 cm (5 5/8 in.) wide and placed horizontally in the bed joint every 40.6 cm (16 in.) (6 brick layers). For all other walls, the reinforcement was placed in the collar joint.

Five composite prisms were built using materials similar to those of each wall. The prisms and walls were cured under the same conditions and tested on the same day. The average dimensions were 40.1 cm (15.8 in.) high and 39.9 cm (15.7 in.) wide for the first wall only; for the other walls, these dimensions were 40.1 cm (15.8 in.) high and 19.3 cm (7.6 in.) wide. Thickness and reinforcement were similar to the corresponding full-size wall. The prisms were loaded vertically in accordance with ASTM specifications [7] and failed as follows: Horizontal cracks along the bed joints started in both wythes at about two-thirds of the ultimate load. These cracks were followed by vertical separation between the masonry wythe and the collar joint. Vertical cracks crossing the masonry units of both wythes occurred next, followed by complete failure. Average dimensions of the walls and compressive strengths of composite prisms are given in Table 2.

Table 2. Average dimensions of the walls and compressive strengths of the composite prisms.

Wall Designation	Width (cm)	Thickness (cm)	Height (cm)	Thickness of Collar Joint (cm)	Area (cm^2)	f'_m (kPa)	c.o.v. %
W2	123.2	22.2	183.1	4.14	2732	20,808	21.0
W4	121.2	22.7	183.1	4.67	2752	19,912	13.6
W6	120.6	23.2	183.6	5.16	2796	16,894	8.8
W8	120.6	24.0	181.9	5.97	2894	14,386	6.2
W10	120.9	23.1	182.4	5.03	2790	14,717	13.1

Mortar and Grout

Type "M" mortar was used as specified in the UBC Code given in Ref. [7] for reinforced masonry. The mortar was mixed in accordance with ASTM-C270-73 specifications [7] and proportioned by volume at 1:0.25:3.5 (cement:lime:sand). The cement was Portland cement Type "1" (ASTM-C150-78a); the lime was hydrated lime Type "S" (ASTM-C207-76); the sand was in accordance with ASTM-C144-76 [7].

Grout mixed in accordance with ASTM-C476-71 was used and proportioned by volume as 1:3 (cement:fine aggregate). The cement was Portland cement Type "1" (ASTM-C150-78a); the fine aggregate was in accordance with ASTM-C404-76 [7].

Brick Units and Prisms

The brick units were made of 3-hole clay brick, having a nominal size of 5.72 cm × 9.21 cm × 19.37 cm (2 1/4×3 5/8×7 5/8 in.) with a net area greater than 75% of the gross. The average dimensions, physical properties and strength properties for the brick units are given in Table 3. These properties were found according to ASTM-C67-78 [7].

Table 3. Material properties for brick.

Width (cm)	Length (cm)	Height (cm)	Gross area (cm^2)	Net area (cm^2)	Net area (%)	Absorption (%)	Moisture (%)	Weight (N)	Compressive strength (kPa) Based on Gross area	Net area
9.0	19.0	5.8	171.4	131.9	77	2.6	16.6	16.5	85,987	112,031

One-wythe prisms were built of brick materials similar to those of the walls and cured under the same conditions, in accordance with ASTM-E447-74 [7]. The prisms were tested under compression only but with load either perpendicular to or parallel with the bed joint. Average dimensions of the prisms were 40.1 cm (15.8 in.) high, 19.3 cm (7.6 in.) wide and 8.9 cm (3.5 in.) thick. The average properties of all brick tests were:

For load perpendicular to bed joint: f'_m = 23,226 kPa (3371 psi); E = 2.05 × 10^7 kPa (4.23 × 10^6 psi); v = 0.21.

For load parallel to bed joint: f'_m = 17,370 kPa (2521 psi); E = 2.18 × 10^7 kPa (3.16 × 10^6 psi); v = 0.15

Reinforced Grout Prisms

One-wythe prisms of reinforced grout were built from materials similar to the walls and cured under the same conditions, in accordance to ASTM-E447-74 [7]. The average prism dimensions were 21.7 cm (8.55 in.) high, 11.2 cm (4.4 in.) wide, and 5.6 cm (2.2 in.) thick. The reinforcement was welded wire fabric (WWF 4×4×4×4). A total of eight prisms were tested, with the following results:

For loads perpendicular to main steel: f'_m = 17,845 kPa (2590 psi); c.o.v. = 11.6%;

For loads parallel to main steel: f'_m = 23,082 kPa (3350 psi); c.o.v. = 7.4%.

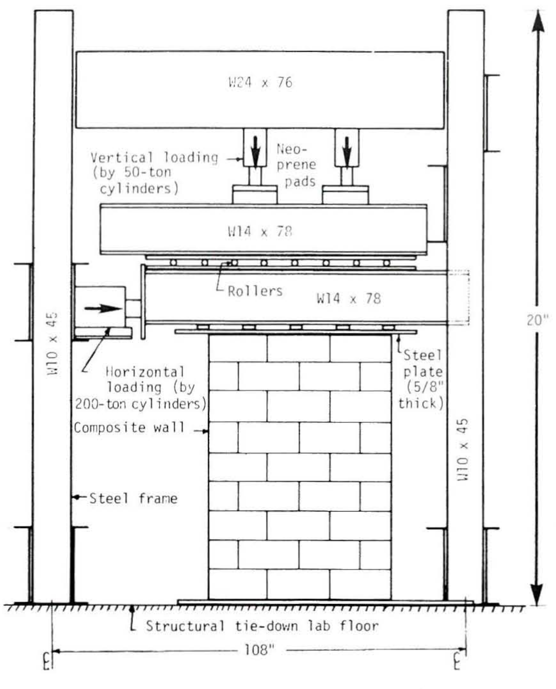
Test Procedure and Frame

The test procedure for all walls was the same:

1. Strain gages and load cell (to give the vertical load readings) were connected to a programmable data acquisition system.
2. Initial readings were taken after applying the vertical load in three cycles, from zero to 44.5 kN (10 kips) and back to zero.
3. Vertical load was applied in increments up to the intended precompression load, then kept approximately constant until the end of the test. The value of the intended vertical load was determined as different ratios (1, 0.9, 0.75 and 0.5) of the allowable load. After applying high values of horizontal load, the vertical load changed. Therefore, the vertical load was readjusted for every load point.
4. Next, the horizontal load was applied in increments until the wall failed, i.e., reached its ultimate load-carrying capacity.
5. At every load point, strains, deflections, and loads were recorded. Cracks were recorded, marked, and numbered with the same number as the load point.

The walls were oriented so that the horizontal load was always applied from east to west.

A rigid steel frame with a horizontal maximum capacity of 1,780,000 N (400,000 lbs) and a vertical maximum capacity of 1,112,500 N (250,000 lbs) was designed at Iowa State University. The horizontal load was adjusted vertically to meet the height of the specimen. The frame was fixed to the structural lab floor, which is of the tie-down type, with a maximum capacity of 4,450,000 N (1,000,000 lbs). A horizontal load was applied using a hydraulic cylinder of 890,000 N (200,000 lbs). Vertical loads were applied at two points using hydraulic cylinders of 445,000 N (100,000 lbs) each. Figure 1 shows the frame and load set-up. The loads were applied to all wythes of the entire composite wall to allow for a study of the overall wall behavior.



1 inch = 0.0254
1 ton (Engl) = 8.9 kN

Fig. 1. Loading mechanism.

DISCUSSION OF WALL BEHAVIOR

The tested brick-to-brick walls showed bearing failure of the compressive corner at the bottom of the wall, followed by bond failure between the masonry and the collar joint. Table 4 summarizes these modes of failure and the maximum measured loads for all walls. The load-deflection curves are shown in Figs. 2 and 3. The initial straight-line portion in these curves occurred only for low loads. No reinforcement yielded in any wall.

Table 4. Maximum loads and modes of failure for the brick-to-brick walls.

Wall Designation	Intended Precompression Load (kN)	Measured* Precompression Load (kN)	Ultimate Lateral Load (kN)	First Crack Load (kN)		Separation Load (kN)		Mode of Failure	
				North	South	North	South	North	South
W2	712	876	401	214	214	--	392	Bearing failure	Bearing failure and diagonal crack started and bond failure
W4	801	892	420	178	178	401	401	Bearing failure and bond failure	Bearing failure and bond failure
W6	601	767	379	80	80	--	--	Bearing failure	Bearing failure and vertical and diagonal failure
W8	699	778	427	160	214	427	427	Bearing failure and bond failure	Bearing failure and bond failure
W10	699	773	401	107	187	401	401	Bearing failure and bond failure	Bearing failure and bond failure

*The precompression load increased during test and was readjusted at every load point. This value is the measured one at the ultimate lateral load point (at failure).

Figure 2 shows the load-deflection curves for W2, W8 and W10. These three walls were subjected to about the same intended precompression load (Nu). These walls differed only in type of reinforcement. They had, however, almost the same area of steel. Comparing the test results of these three walls indicates:

1. Wall W2 was stiffer at very low loads, but W10 was stiffer than the other two walls at higher lateral loads.

2. The joint reinforcement in W10 reduced the ultimate lateral load, but not significantly (about 6%). This agrees with similar conclusions for a load bearing (only) single wythe wall [3].

3. The modes of failure in the three walls were similar (bearing failure and separation).

4. The use of mesh reinforcements or truss joint reinforcements each provided the walls with greater ductility than vertical and horizontal bars and allowed the wall to deflect more before failure.

5. The vertical separation occurred only at the ultimate lateral load. This may indicate that the bond failure was due to the large deflection rather than the high lateral loads.

6. The first crack in wall W2 occurred at a higher lateral load than either of wall W8 or W10.

Figure 3 shows the load-deflection curves for walls W4 and W6. These were identical, with different intended precompression loads. This figure indicates similar deflections at low lateral loads and at higher loads. Increasing the precompression load had a smaller effect on the ultimate lateral load (e.g., a 33% increase in the precompression load increased the ultimate lateral force by only 10.7%).

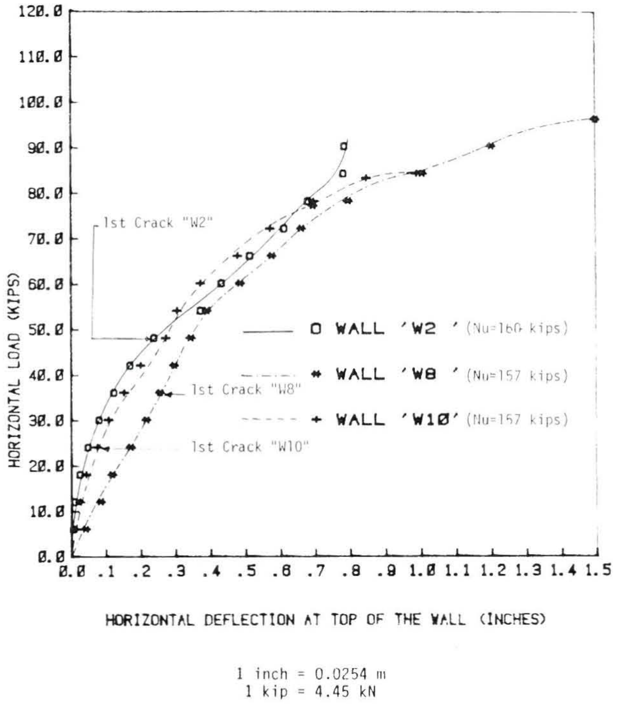


Fig. 2. Load-deflection curves for walls W2, W8, and W10.

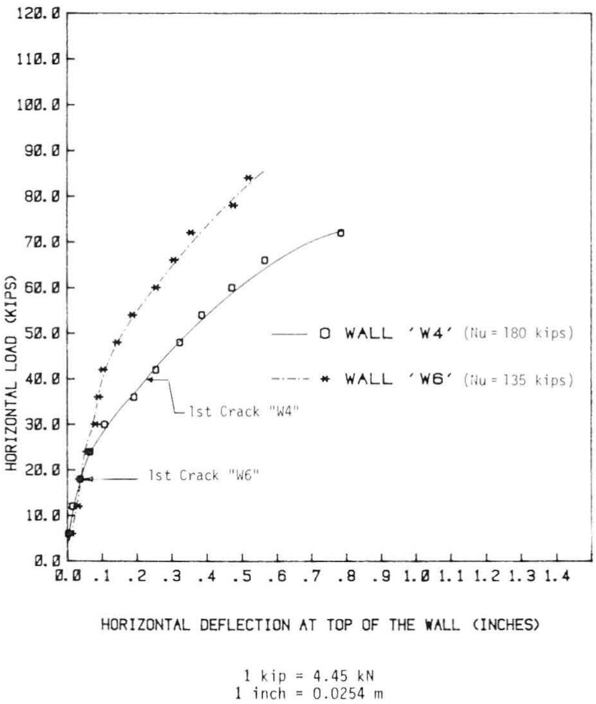


Fig. 3. Load-deflection curves for walls W4 and W6.

Table 5 shows the ultimate bond stresses that caused the separation and compares them to the values obtained from the Equation (1) given in [4]. The average safety factor was 3.25, which is reasonable for masonry. The table indicates that the actual bond stresses were more than 689 kPa (100 psi), as suggested in Ref. 8. Equation (1) appears to give reasonable results to determine the allowable bond stresses for composite masonry walls (based on these tests). However, the loading methods of obtaining the shear-bond strength by Equation (1) versus the loading of these tests are different.

The allowable shear strength, v, as given by the ACI Code [9] for walls with height-to-width ratio of more than one is:

$$v = 1.5 \sqrt{f'_m} > 75 \text{ psi} \quad \text{(with 1 psi = 6.89 kPa)} \qquad (2)$$

Table 5. The bond stresses using the proposed Equation (1) [4] and the test data.

Wall	W2	W4	W6	W8	W10
f'_m (kPa)	20,808	19,912	16,894	14,386	14,717
v_{SB} (kPa)	482	473	442	416	420
Ultimate measured bond stresses (kPa)	1,432	1,454	--*	1,475	1,434
Factor of safety	2.97	3.07		3.54	3.42

*The separation failure did not take place in this wall.

A comparison between ultimate and allowable shear stresses is shown in Fig. 4. These ultimate values are given in Table 6.

The average value of the safety factor was 2.9 (From Table 6), meaning that the allowable value given in the codes (Equation 2) is applicable for composite masonry

The relationship between ultimate shear strength and precompression stress can be written in the general form:

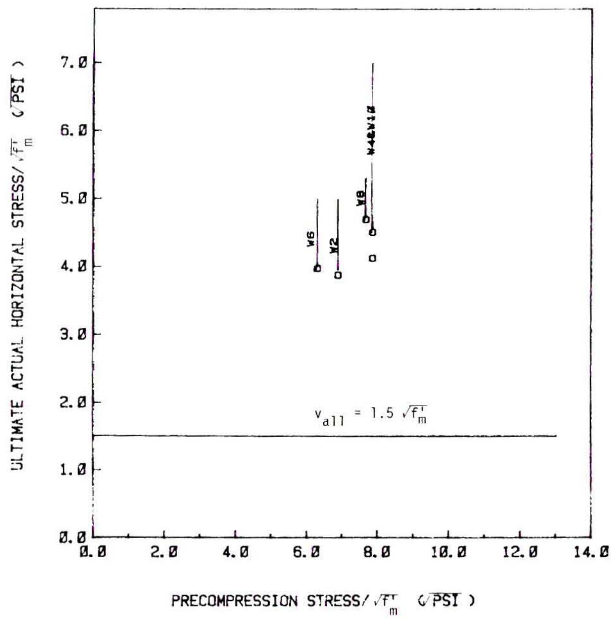


Fig. 4. Comparison between the allowable shear strength and the experimental results of brick-to-brick walls.

Table 6. Ultimate lateral stresses and the precompression stresses for brick-to-brick walls.

Wall	W2	W4	W6	W8	W10
Intended precompression stresses (kPa)	2,604	2,907	2,146	2,412	2,501
Ultimate lateral stresses (kPa)	1,465	1,525	1,356	1,475	1,434
Measured precompression stresses (kPa)	3,204	3,237	2,739	2,685	2,768
Allowable lateral stresses (kPa) (Equation 2)	517	517	512	472	477
Safety factor	2.83	2.95	2.65	3.13	3

$$v_{ult.} = v_{SB} + \mu\sigma_C \tag{3}$$

where:

$v_{ult.}$ = the ultimate shear stress
v_{SB} = the ultimate shear-bond strength
μ = the coefficient of friction
σ_c = the precompression stress

The values of v_{SB} and μ for a single brick wythe are given by the research investigations indicated in Table 7. The proposed constants for composite walls are also shown. Therefore, Equation (3) can be written in the form:

$$v_{ult.} = 141 + 0.19\,\sigma \quad \text{(in psi with 1 psi = 6.89 kPa)} \tag{4}$$

The comparison shows that the proposed equation is valid for all tested composite walls.

Table 7. Comparison between the constants of Equation 4, as given by previous research, for one wythe and proposed constants for composite walls.

	Constants		W2		W4		W6		W8		W10	
Ref.	v_{SB}^*	μ	$v_{ult.}^{c\,*}$	$v_{ult.}^{m\,*}$	$v_{ult.}^{c\,*}$	$v_{ult.}^{m\,*}$	$v_{ult.}^{c\,*}$	$v_{ult.}^{m\,*}$	$v_{ult.}^{c\,*}$	$v_{ult.}^{m\,*}$	$v_{ult.}^{c\,*}$	$v_{ult.}^{m\,*}$
2	1516	1.1	4630	1465	4113	1525	3852	1356	4547	1475	3514	1434
12	103	0.167	579	1465	496	1525	462	1356	565	1475	407	1434
Proposed	971	0.19	1466	1465	1524	1525	1379	1356	1430	1475	1447	1434

*Stresses given in kPa (1 kPa = 0.145 psi).

CONCLUSIONS AND RECOMMENDATIONS

The following conclusions can be made for brick-to-brick composite walls based upon this investigation:

1. The failure modes for brick-to-brick walls were mainly bearing failure in both wythes in addition to separation failure involving either one or both wythes and the collar joint.

2. The precompression load had only a small effect on the ultimate shear stress. However, a wider range of precompression loads should be considered in the future.

3. Williams' and Geschwindner's proposed Equation (1) for the bond stresses [4] as applied to these composite walls provided a safety factor of 3.25.

4. Joint reinforcement reduced the ultimate shear load, but not significantly ($\cong$ 6%). This reduction agrees with similar conclusions [3] for load bearing single wythe walls. However, additional tests are needed to substantiate this conclusion.

5. Although the minimum amount of steel required by the ACI Code was used, the steel did not yield. Therefore, further studies should be done using less steel.

6. Ultimate shear stresses ranged from 1,356 kPa to 1525 kPa (196.8 psi to 221.3 psi).

7. The load-deflection curve can be approximated as a trilinear relationship, as proposed by Meli [2]. More tests should be conducted considering different parameters to find the constants that define this curve.

8. Ultimate shear strength can be predicted using Equation (4). The allowable shear stress can then be calculated using a safety factor of 3 as follows: $v_{all} = 47 + 0.0625\ \sigma_c$ (in psi with 1 psi = 6.89 kPa).

9. The allowable value (Equation 2) of shear stresses as given in the ACI Code [9] was applicable for these composite walls.

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Notation

E = modulus of elasticity

f'_m = ultimate compressive strength of masonry element

μ = coefficient of friction

ν = Poisson's ratio

N_u = ultimate compression wall load

σ_c, σ = precompression stress

v, v_{all} = allowable shear stress

v_{SB} = shear-bond strength

$v_{ult.}$ = ultimate shear stress

WWF = designation for welded wire fabric