STEEL-MASONRY INTERFACE STRENGTH AND BEHAVIOR

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ABSTRACT

Hybrid masonry consists of a combination of structural steel framing and reinforced concrete masonry unit shear panels to provide resistance to both gravity and lateral loads. A critical link in this system is the connection between the steel members and the masonry shear panels. Experiments were performed at the University of Hawaii at Manoa (UHM) to develop ductile fuse and rigid link plate connectors to transfer in-plane loads from the steel floor beams to the masonry shear panels.

During an earthquake, the ductile fuse connectors will yield prior to substantial damage to the masonry shear panels, thereby dissipating seismic energy in the replaceable links rather than damaging the steel frame or masonry shear panels. After the event, the fuse connectors can be replaced to restore the building to its original condition. In contrast, rigid link plate connectors are designed to remain elastic during seismic events. In-plane shear loads are transferred directly to the masonry shear panels which are designed to undergo inelastic cycling to dissipate energy and provide the necessary inelastic deformation without loss of lateral load capacity.

Integral to these fuse and link connectors are the thru-bolts that connect the plates to the masonry shear panel. Lack of design guidance for the capacity of these thru-bolts led to additional testing to characterize the capacity of thru-bolts in reinforced masonry walls. This paper presents results of the connector plate tests, with particular emphasis on the performance of the thru-bolted connection between the fuse or link plates and the masonry shear panel.

KEYWORDS: reinforced concrete masonry, earthquake, shear walls, structural steel frames, ductile fuses, thru-bolt connection

INTRODUCTION

Hybrid masonry is a new seismic structural system which involves structural steel frames with concrete masonry unit (CMU) walls [1]. The CMU walls serve as shear walls to replace traditional steel bracing. They have different functions depending on the type of hybrid masonry. The CMU walls are either used to transfer only lateral loads through the height of the structure or lateral and gravity loads. Type I Hybrid Masonry uses the CMU walls to resist only in-plane lateral loads. In Type II Hybrid Masonry the CMU walls are also used to support vertical loads. In Type III Hybrid Masonry the CMU walls act as the web of a composite shear wall with the steel columns as boundary elements.

In Type I Hybrid Masonry, the CMU shear panel is separated from the steel frame on three sides as shown in Figure 1. Vertical gaps are provided between the masonry and steel columns to prevent contact during lateral movement. A gap is also provided at the top of the wall and connector plates are used as the connection between the steel beam and CMU wall. These connector plates transfer lateral loads being applied by the steel frame to the CMU shear panel. They are friction bolted to side plates welded to the flanges of the floor beam (Figure 1).

The connector plates are either ductile fuse elements or non-ductile link plates. The ductile fuses are designed to undergo non-linear yielding behavior during design level seismic events, thereby protecting the CMU walls and steel frame from damage. The bolted fuses can then be removed and replaced after the earthquake to restore the building to its original condition. Type I Hybrid Masonry walls with non-ductile link plates rely on ductility in the masonry shear panels to absorb seismic energy and displacement. The link plates are designed with an over strength factor to avoid yielding during the design level earthquake.

In Type I Hybrid Masonry the transfer of in-plane shear from the fuse or link plates to the CMU panel utilizes bolts which pass through the CMU wall and vertical slotted holes in the connector plates on either side of the wall. These thru-bolts must not be the weak link in the Hybrid Masonry system since failure of the bolts in shear or masonry breakout would represent a non-ductile response that is not suitable for seismic design. This paper presents the results of eleven bolt push-out tests performed on six 203mm (8in) nominal thickness grouted CMU walls to evaluate the performance of the thru-bolts. Based on these tests a number of conclusions were drawn and recommendations are provided for design of thru-bolted connections.

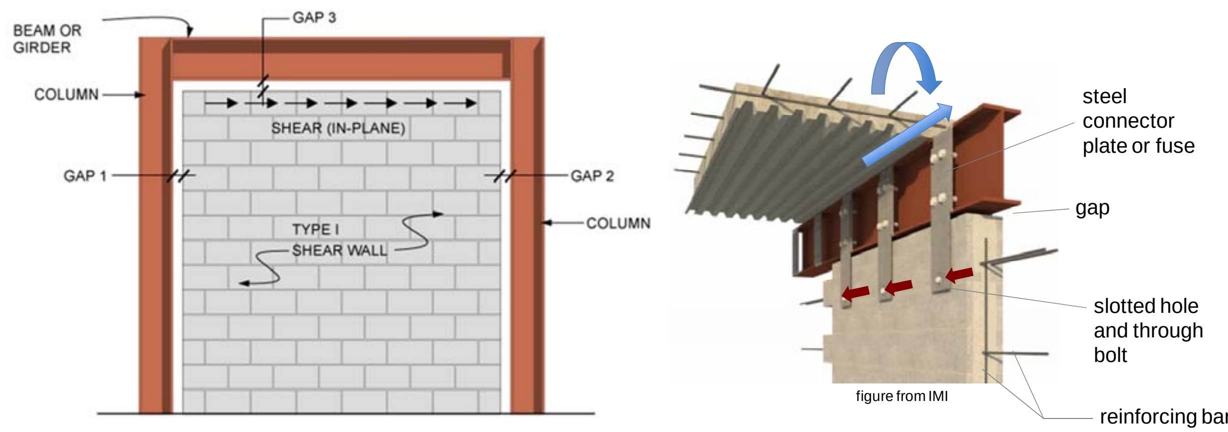

Figure 1: Schematic of Single Bay of Type I Hybrid Masonry (left) and Link Connector Plates between Steel Beam and CMU Shear Panel (right)

BACKGROUND

This research was performed at the University of Hawaii at Manoa (UHM) as part of a larger project funded by the National Science Foundation (NSF) through the Network for Earthquake Engineering Simulation (NEES) to investigate the use of Hybrid Masonry in moderate to high seismic zones [2]. UHM was responsible for development of the connections between the masonry shear panels and the steel framing. These connectors were then used in two-story Hybrid Masonry frames tested in the MUST-SIM facility at the University of Illinois, Urbana Champaign (UIUC) [2].

CONNECTOR DEVELOPMENT

The steel connector plates between the steel frame and the CMU shear walls for the Type I Hybrid Masonry were developed through a series of test programs performed in the Structures Laboratory at UHM. Goodnight et al. [3] performed cyclic tests on individual plate elements to evaluate various energy dissipating fuse concepts. Ozaki-Train et al. [4] further investigated the more promising fuse concepts by testing pairs of fuses in the proposed configuration for Type I Hybrid Masonry connections. The slip-critical bolted connection to a side plate on the steel beam was developed to allow for easy replacement of the fuses after a seismic event. Figure 2 shows the final designs of the fuse and link plates developed at UHM.

Mitsuyuki and Robertson [5] evaluated the performance of these replaceable fuse and link plates as connectors on full-scale CMU wall panels. Figure 3 shows the hysteretic response of three different tests to demonstrate the cumulative effect of increasing the number of fuses. The hysteretic responses plotted in Figure 3 are for a wall test with 3 pairs of 100mm (4in) fuses, a wall test with 2 pairs of fuses magnified by 1.5, and a test of one individual fuse by Goodnight et al. [3] magnified by 6.0. The good agreement between the hysteretic responses indicates that the performance of multiple fuse connectors is a simple multiple of the response of a single fuse. It was noted that the fuses used in the wall test showed less ductility than those tested individually. This is attributed to the presence of the wall limiting the out-of-plane buckling of the fuses, which tends to increase the demand on the fuse. It was concluded that the fuse plates could achieve 51mm (2in) lateral displacement after multiple loading cycles. Similar results were obtained for the 152mm (6in) fuses shown in Figure 2, though the lateral load capacity was larger. Reports on these studies are available at www.cee.hawaii.edu/content/resreport.htm .

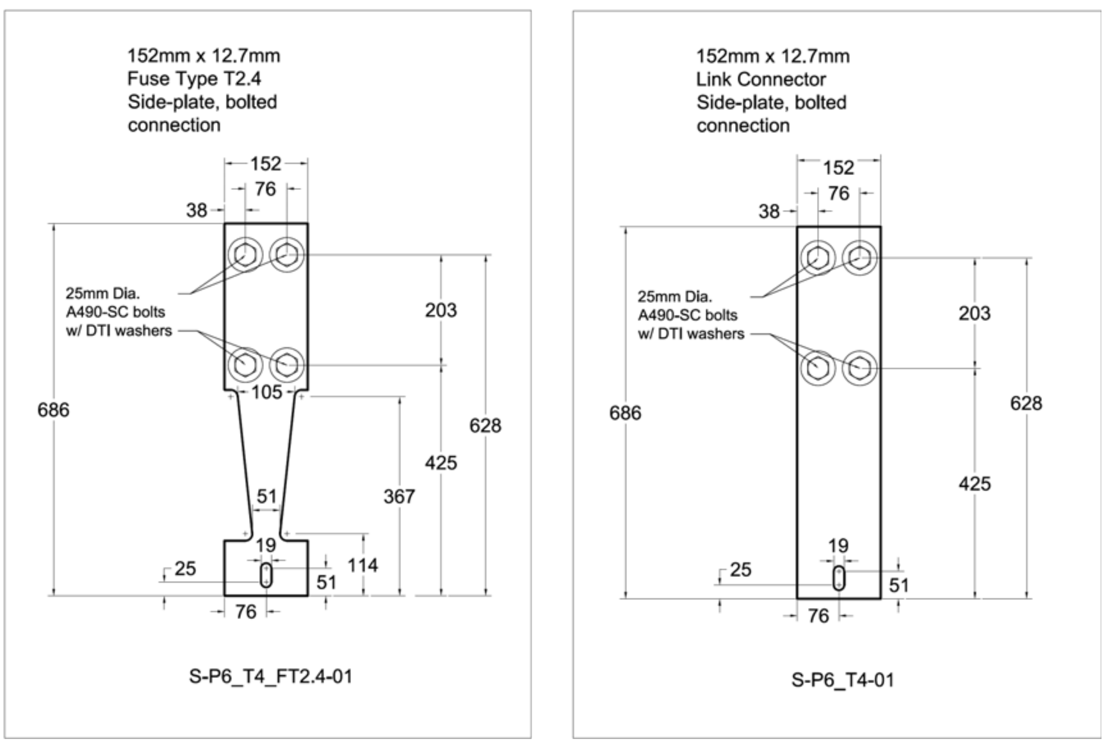

Figure 2: Tapered Fuse (left) and Link Plate (right) for Type I Hybrid Masonry

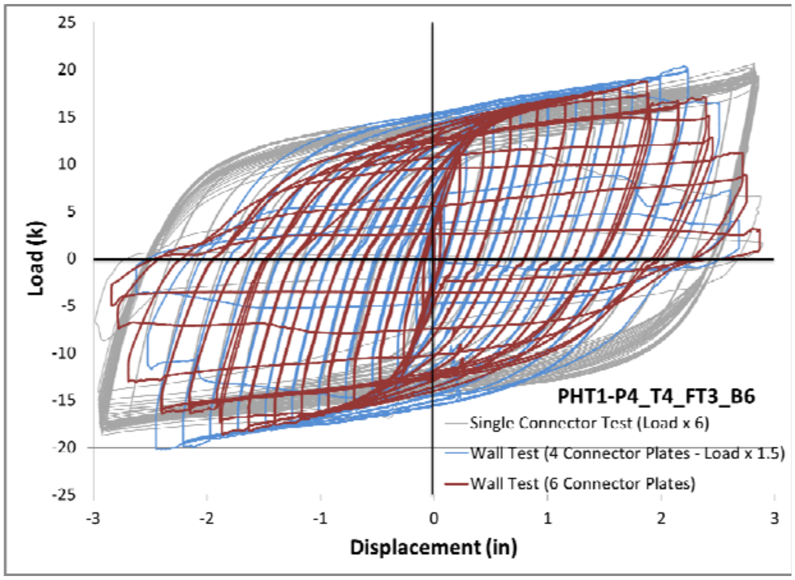

Figure 3: Hysteretic responses for 100mm tapered fuses (1k = 4.448kN, 1in = 25.4mm)

In order to ensure that the thru-bolt connection does not result in a premature breakout failure during cyclic testing of the fuses, a series of bolt push-out tests were performed [6]. This paper will focus on the determination of the failure modes of the thru-bolts for Type I Hybrid Masonry walls. The results of this research were used by UIUC in their development of the large-scale Type I Hybrid Masonry specimens tested at the NEES MUST-SIM laboratory at UIUC.

BOLT PUSH-OUT TESTS

In order to evaluate the breakout capacity of the thru-bolts connecting the fuse or link plate to the masonry shear panel, a series of eleven bolt push-out tests was performed. The tests utilize the same test setup as the prior wall tests of the fuse and link plates, but with only a single thru-bolt and double link plates as shown in Figure 4. Lateral displacement of the steel beam above the wall resulted in a horizontal force applied to the thru-bolt (Test 1) inducing a breakout failure at the right edge of the wall panel. A second test (Test 2) was performed on the thru-bolt at the left side of the wall so as to utilize each wall panel for two push-out tests. The bolt for Test 1 was located at the center of the second cell from the edge of the wall (ie. 305mm or 12in from the wall edge) while the bolt for Test 2 was located in the center of the third cell from the edge of the wall (ie. 508mm or 20in from the wall edge).

The thru-bolts were located below the horizontal reinforcing in a bond beam as shown in Figure 4. The fully grouted wall was reinforced with 13mm (0.5in) vertical bars at 610mm (24in) on center, and had no horizontal joint reinforcement. Average compressive strengths for the grout, mortar and grouted CMU prisms were determined by standard ASTM tests to be 35.3 MPa (5118 psi), 27.1 MPa (3934 psi) and 18.8 MPa (2727 psi), respectively.

Table 1 shows details of the 6 wall specimens used to perform the 11 thru-bolt push-out tests. The control specimen, FGW-1#4BB had one 13mm (0.5in) horizontal bar in the bond beam located at the second course from the top of the wall. An identical specimen was used for a group effect test (G), while another identical specimen was used to evaluate the effect of leaving the top course ungrouted (TCH) to simplify construction of the wall panels inside the steel frame. The remaining specimens evaluated the effect of locating the bond beam in the top course (TBB), increasing the horizontal steel in the bond beam (2#4BB), or making all three top courses bond beams with one 13mm (0.5in) bar in each course (3#4BB). Except for the group test, all specimens were tested twice; once with the thru-bolt located 305mm (12in) from the wall edge (Test 1) and once with the thru-bolt located 508mm (20in) from the wall edge (Test 2).

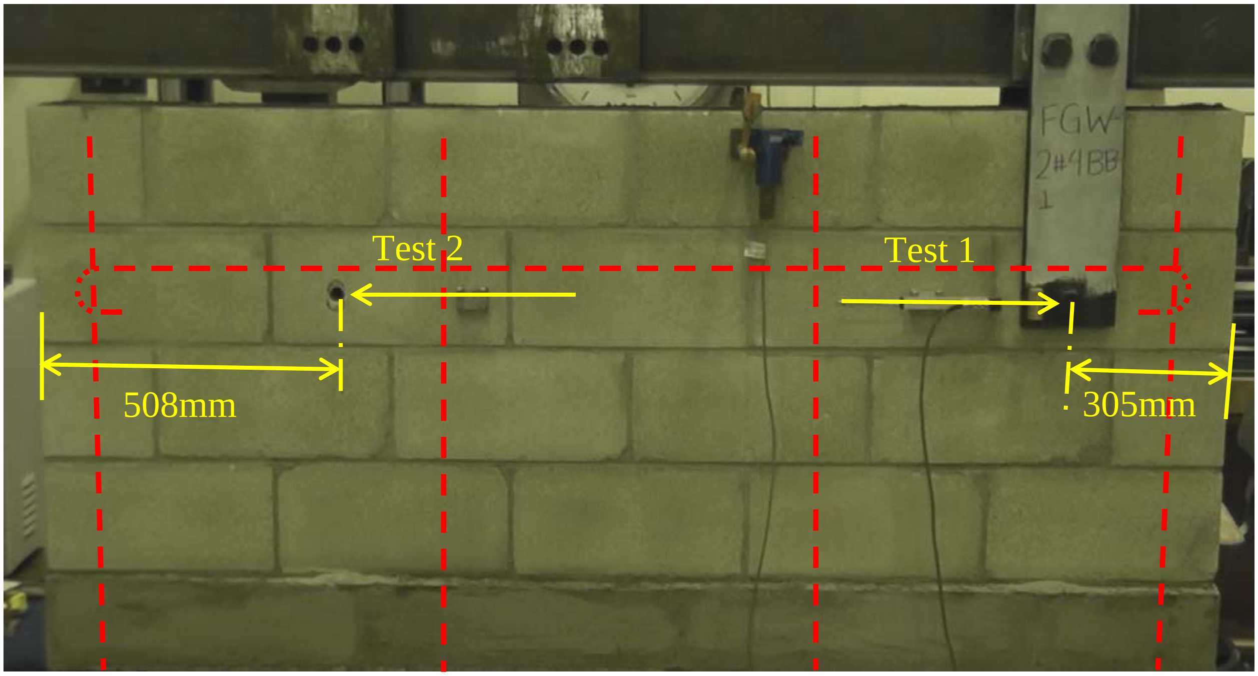

Figure 4: Test configuration for Bolt Push-out Test 1 and Test 2 on typical wall panel

Table 1: Bolt pushout wall specimens.

Specimen	Vert. reinft. bar size	Spacing of vert. reinf. bars	Horizontal reinf. bars	Horizontal reinf. bar location	Push-out test type
FGW-1#4BB	13 mm (#4)	610 mm (24 in)	1 – 13 mm (1 - #4)	Second course from top	1) 305 mm (12") from edge, 2) 508 mm (20") from edge
FGW-1#4BB-G	13 mm (#4)	610 mm (24 in)	1 – 13 mm (1 - #4)	Second course from top	1) 305 mm (12") and 711 mm (28") from edge (group)
FGW-1#4BB-TCH	13 mm (#4)	610 mm (24 in)	1 – 13 mm (1 - #4)	Second course from top, top course hollow	1) 305 mm (12") from edge, 2) 508 mm (20") from edge
FGW-1#4TBB	13 mm (#4)	610 mm (24 in)	1 – 13 mm (1 - #4)	Top course	1) 305 mm (12") from edge, 2) 508 mm (20") from edge
FGW-2#4BB	13 mm (#4)	610 mm (24 in)	2 – 13 mm (2 - #4)	Second course from top	1) 305 mm (12") from edge, 2) 508 mm (20") from edge
FGW-3#4BB	13 mm (#4)	610 mm (24 in)	2 – 13 mm (2 - #4)	Top three courses	1) 305 mm (12") from edge, 2) 508 mm (20") from edge

PUSH-OUT TEST RESULTS

Figure 5 shows typical specimens after failure of the thru-bolt connection by CMU breakout. The load-displacement responses for these connections are shown in Figure 6. The increased edge distance for Test 2 results in an increase in breakout strength, as expected. The group effect reduces the strength below the sum of the strengths of the individual bolt capacities.

Figure 5: Specimen FGW-1#4BB Test 1 (left), Test 2 (center) and FGW-1#4BB-G Group test (right) after failure.

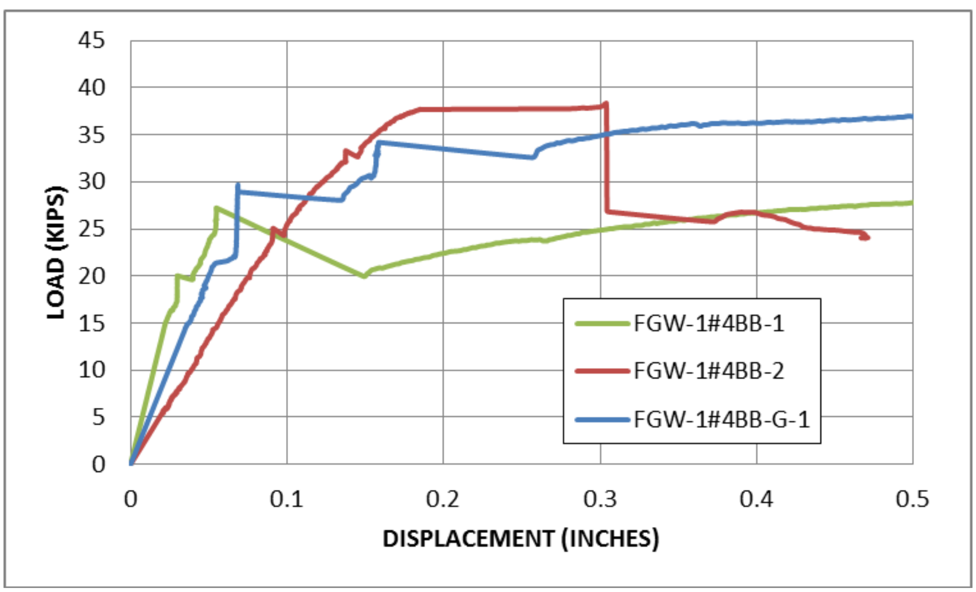

Figure 6: Comparison of group and control specimens (1kip = 4.448kN; 1in = 25.4mm).

Figure 7 shows a comparison between the control specimen, FGW-1#4BB-2, and the equivalent test without grouting the top course of the wall, FGW-1#4BB-TCH-2. There is a significant drop in cracking and ultimate capacity of the thru-bolt breakout when the top course is not grouted.

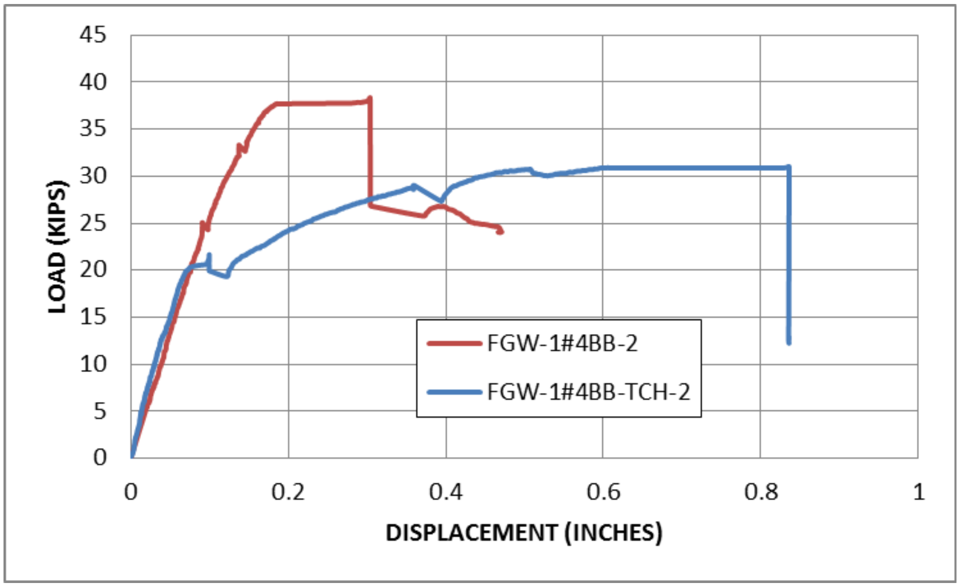

Figure 7: Comparison of specimen with ungrouted top course and 508 mm (20 in) edge distance and control (1kip = 4.448kN; 1in = 25.4mm).

Figure 8 shows a comparison between the control specimen, FGW-1#4BB-2, and the specimen with bond beam in the top course of the wall, FGW-1#44TBB-2. Moving the bond beam through which the bolts are installed from the second from the top, to the top course of the wall resulted in a significant reduction in push-out capacity.

Figure 9 shows a comparison between the control specimen, FGW-1#4BB-2, and a similar specimen with two 13mm (0.5in) bars in the bond beam, FGW-2#4BB-2. The increased bond beam reinforcement does not increase either the cracking or ultimate capacity for bolt push-out.

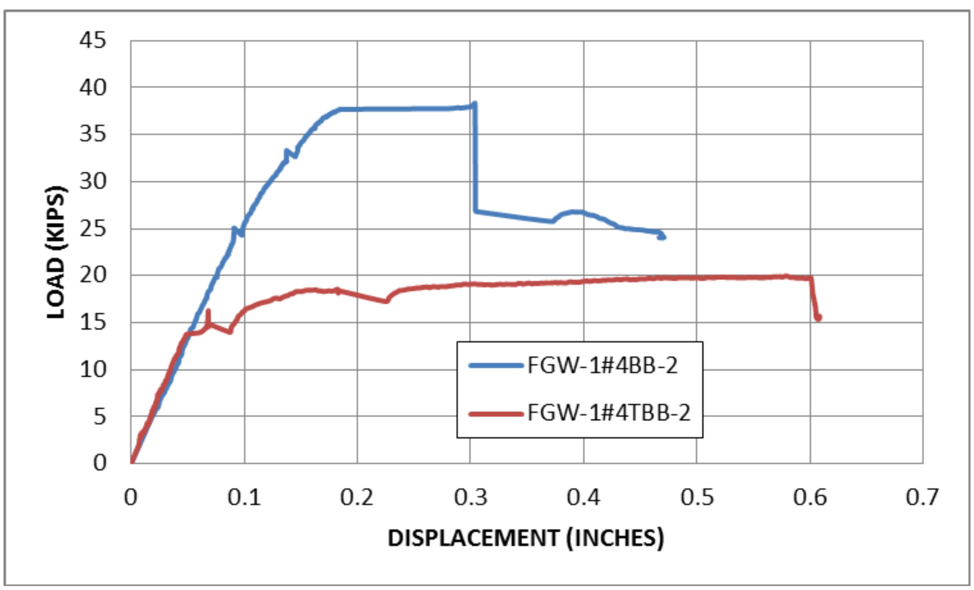

Figure 8: Comparison of specimen with through bolt in top course and control placed 508 mm (20 in) from the edge of the CMU wall (1kip = 4.448kN; 1in = 25.4mm).

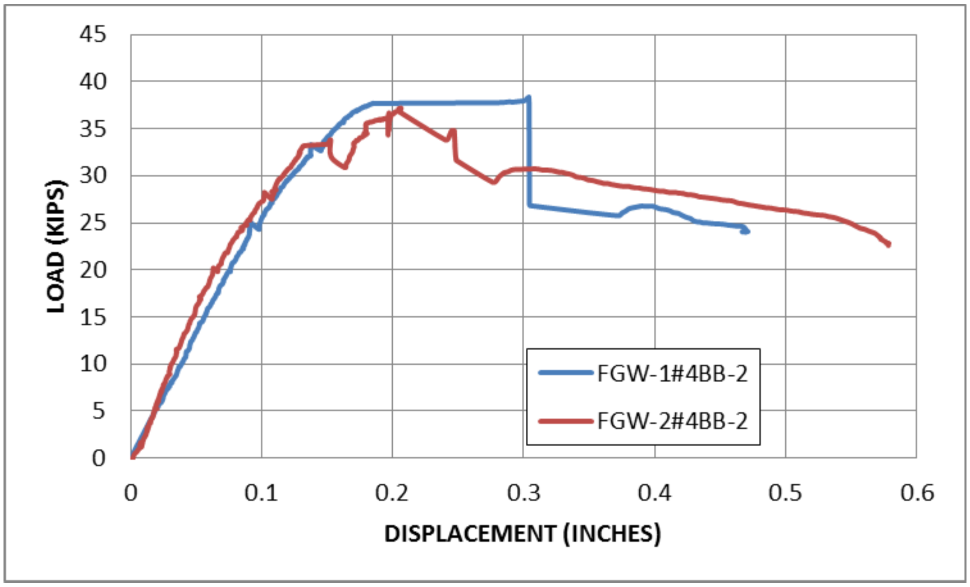

Figure 9: Comparison of specimen with 2-#4 bars in the bond beam and control with thru-bolt placed 508 mm (20 in) from the edge of the CMU wall (1kip = 4.448kN; 1in = 25.4mm).

PROPOSED BREAKOUT STRENGTH

The MSJC code [7] does not provide design capacities for thru-bolts in masonry walls. However, bolts embedded in masonry walls are considered, and two identical bolts embedded on either side of a CMU wall could be assumed to provide shear strength comparable with a thru-bolt of the same diameter. MSJC considers four failure modes for bolts embedded in masonry and subjected to shear. The pryout failure mechanism is not possible for a thru-bolt, and masonry crushing was not observed in these tests. The high-strength bolts were selected to preclude a bolt shear failure. Hence the remaining failure mechanism is masonry breakout.

Although the ultimate capacity of the breakout tests exceeded the cracking strength, the deformation required to achieve the ultimate capacity often exceeded 13mm (0.5in) which is excessive if the thru-bolt is part of a Hybrid Masonry system required to resist cyclic lateral loads. It was therefore decided that the cracking strength would be taken as the nominal capacity of the thru-bolt breakout failure.

Based on all of the thru-bolt push-out tests performed in this study, a typical breakout failure plane is shown in Figure 10. The breakout capacity of the thru-bolt is based on the tensile failure of a vertical plane from the thru-bolt to the top of the wall, and a 45 degree inclined plane from the thru-bolt to the edge of the wall below the bolt.

The MSJC breakout strength for an embedded bolt in shear is given by:

$$B_{vnb} = 0.33\sqrt{f'_m}A_{pv} \quad \text{(MPa and mm)} \qquad (1)$$

$$B_{vnb} = 4\sqrt{f'_m}A_{pv} \quad \text{(psi and in.)} \qquad (1)$$

where A_{pv} is the area of the failure surface and f'_m is the CMU compressive strength. Eqn. 1 significantly overestimates the breakout cracking strength for the thru-bolts in this study as shown in Figure 11, where the diagonal line represents perfect agreement with the experimental results. Based on a least squares best fit with the test data, a more appropriate average stress on the failure surface assumed in Figure 10 would be $0.13\sqrt{f'_m}$ in MPa ($1.58\sqrt{f'_m}$ in psi). It is therefore proposed that the cracking strength for a thru-bolt breakout mechanism can be estimated using:

$$B_{vnb} = 0.13\sqrt{f'_m}A_{pv} \quad \text{(MPa and mm)} \qquad (2)$$

$$B_{vnb} = 1.58\sqrt{f'_m}A_{pv} \quad \text{(psi and in.)} \qquad (2)$$

The MSJC and proposed predicted strengths are compared with the experimental cracking strengths for all specimens in this study in Table 2 and Figure 11. The area of the failure plane is computed according to Figure 10 using a masonry width of 194 mm (7.625 in) and f'_m of 18.8 MPa (2727 psi). The predictions for a typical specimen are shown in Figure 12.

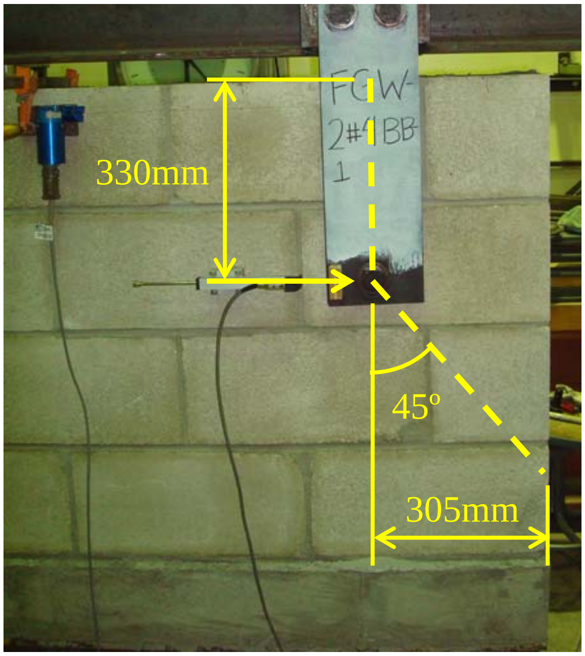

Figure 10: Proposed failure plane for strength prediction

Table 2: Breakout strengths based on MSJC (Eqn. 1) and proposed method (Eqn. 2)

Specimen	A_{pv} in mm^2 (in^2)	B_{vnb} from Eqn. 1	B_{vnb} from Eqn. 2	P_{cr} Experimental
FGW-1#4BB-1	147,700 (229)	211 kN (47.8 kips)	83.1 kN (18.9 kips)	89 kN (20.0 kips)
FGW-1#4BB-2	203,400 (315)	291 kN (65.8 kips)	114 kN (26.0 kips)	107 kN (24.0 kips)
FGW-1#4BB-G-1	238,100 (369)	340 kN (77.1 kips)	134 kN (30.5 kips)	127 kN (28.5 kips)
FGW-1#4BB-TCH-1	116,800 (181)	167 kN (37.8 kips)	65.8 kN (14.9 kips)	62 kN (14.0 kips)
FGW-1#4BB-TCH-2	165,800 (257)	237 kN (53.7 kips)	93.4 kN (21.2 kips)	97 kN (21.7 kips)
FGW-1#4TBB-1	109,700 (170)	157 kN (35.5 kips)	61.8 kN (14.0 kips)	57 kN (12.9 kips)
FGW-1#4TBB-2	165,800 (257)	237 kN (53.7 kips)	93.4 kN (21.2 kips)	62 kN (14.0 kips)
FGW-2#4BB-1	147,700 (229)	211 kN (47.8 kips)	83.1 kN (18.9 kips)	93 kN (21.0 kips)
FGW-2#4BB-2	203,400 (315)	291 kN (65.8 kips)	114 kN (26.0 kips)	125 kN (28.0 kips)
FGW-3#4BB-1	147,700 (229)	211 kN (47.8 kips)	83.1 kN (18.9 kips)	71 kN (16.0 kips)
FGW-3#4BB-2	203,400 (315)	291 kN (65.8 kips)	114 kN (26.0 kips)	138 kN (31.0 kips)

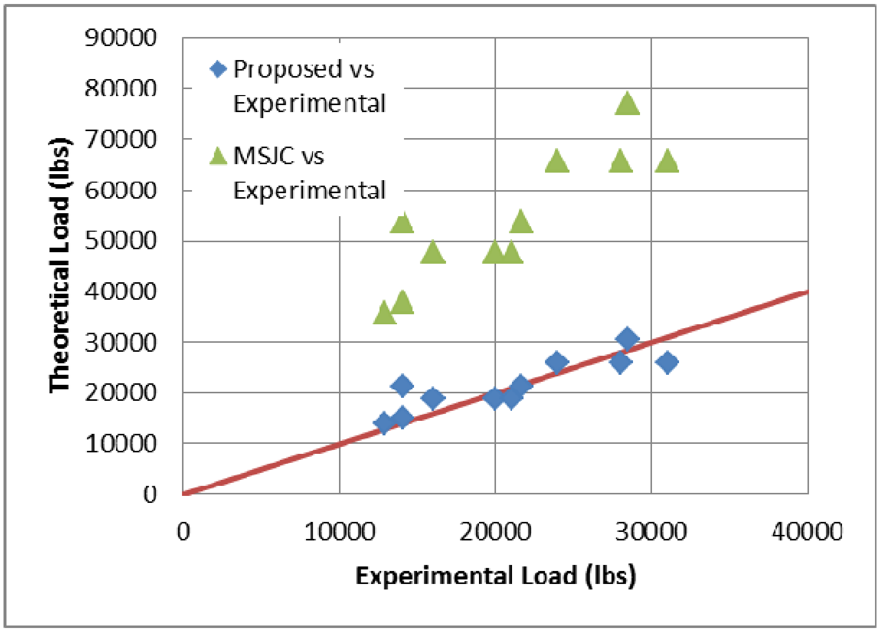

Figure 11: Predicted strength compared with experimental results (1lb = 4.448N)

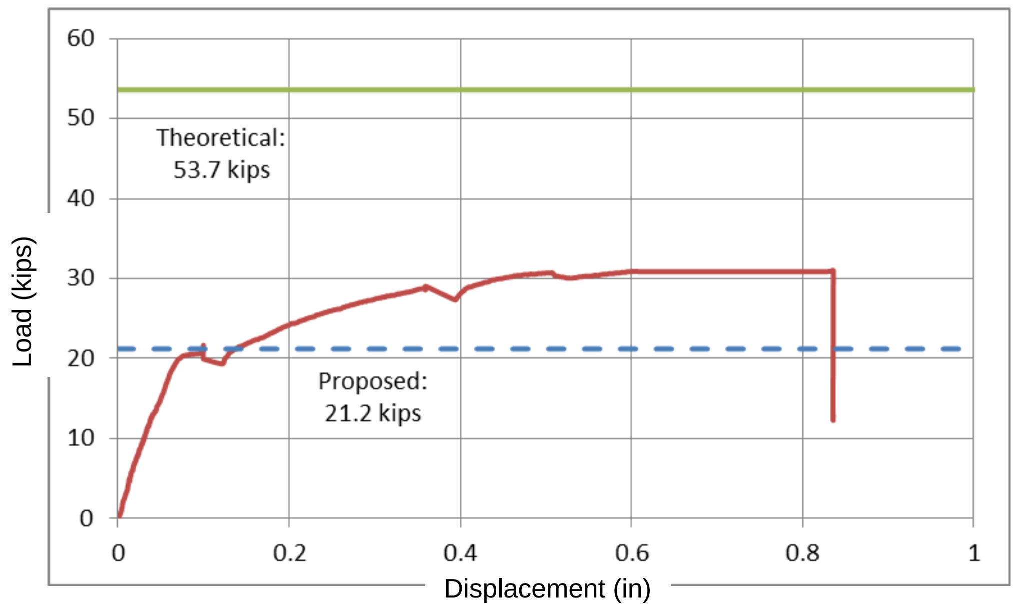

Figure 12: Load-displacement response for FGW-1#4BB-TCH-2 with MSJC theoretical and proposed capacity (1kip = 4.448kN; 1in = 25.4mm).

SUMMARY AND CONCLUSIONS

Hybrid Masonry is a new structural seismic system that utilizes CMU wall panels built in-plane with the steel framing as the lateral force resisting components of the structure. For Type I Hybrid Masonry, the top of the CMU wall is connected to the steel beam above by means of replaceable ductile fuse plates or non-ductile link plates. A critical component of Type I Hybrid Masonry is the thru-bolt that connects the fuse or link plate to the masonry shear panel. This paper summarizes results of a series of push-out tests on thru-bolts located in a bond beam either 2 or 3 cells from the edge of a 203mm (8in) solid grouted CMU wall panel. The thru-bolts were located below the horizontal bar(s) in the bond beam in a hole matching the size of the thru-bolt.

Based on the eleven push-out tests performed in this study, with f'_m of 18.8 MPa (2727 psi), the following conclusions were drawn regarding thru-bolts used for Type I Hybrid Masonry connectors:

- Thru-bolts for use in Type I Hybrid Masonry connectors should have a minimum edge distance of 305mm (12in) from the end of the CMU wall panel. This implies placing the bolt through the center of the second cell from the end of the wall. No bolts should be placed in the end cell.
- Thru-bolts with an edge distance of 508mm (20in) from the end of the CMU wall, i.e. located in the third cell from the end of the wall, provided increases of 40% and 60% in cracking and ultimate strengths, respectively, compared with thru-bolts located 305m from the end of the CMU wall.
- The MSJC code does not currently address thru-bolt breakout capacity, but using the expression for two anchor bolts co-located on opposite sides of the wall significantly overestimates the bolt push-out capacity.
- An empirical expression is proposed for predicting the thru-bolt breakout cracking strength based on a vertical crack from the bolt location to the top of the wall and a 45 degree crack below the bolt location to the edge of the wall. Based on the experimental results from this study, the average cracking stress on this failure plane is $0.13\sqrt{f'_m}$ in MPa ($1.58\sqrt{f'_m}$ in psi).
- Increasing the reinforcing steel in the bond beam does not increase the cracking capacity, but may increase ductility and ultimate strength of the connection.
- One test demonstrated that the group effect can significantly reduce the total capacity of two thru-bolts located at 406mm (16in) on center. Additional testing is required to evaluate this effect in more detail.

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