HYBRID MASONRY CONNECTOR PLATE AND HEADED STUD SMALL-SCALE WALL TESTING

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ABSTRACT

Hybrid masonry is a new seismic structural system which involves structural steel frames with concrete masonry unit (CMU) walls. 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. Using the CMU walls as a gravity member, the steel beams in the frames can be reduced resulting in a lower construction cost. In Type III Hybrid Masonry the CMU walls act as the web of a composite shear wall with the steel columns as boundary elements for columns.

Drs. Ian Robertson and Gaur Johnson, along with graduate student researchers Seth Goodnight, Reef Ozaki-Train, and Steven Mitsuyuki have designed and tested prospective connector plates to develop connectors that can transfer lateral loads. The first type tested was the ductile fuse connector, which was designed to transfer many cycles of lateral load for Type I Hybrid Masonry. During cycling, the fuse connector plates dissipated energy caused by the simulated seismic activity. The fuse connector possessed the capability to dissipate large amounts of energy and allowed relatively large deflections without damaging the CMU wall. This ductility is an advantage because only the fuse connector plates must be replaced after a seismic event. The second type of connector that was tested was a link plate, which was designed as a uniform plate with no fuse section. These link plates were designed to transfer higher lateral loads than the fuse connector. Rather than allowing large displacements by yielding, this connector plate provided a rigid system that transferred the lateral loads to the CMU infill wall which must be designed for ductile response.

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 through bolts must not be the weak link in the Hybrid Masonry system since failure of the bolts in shear or masonry break-out could represent a non-ductile response that is not suitable for seismic design.

This report presents the results of eleven bolt push-out tests were performed on six 8" thick grouted CMU walls to evaluate the performance of the through bolts. Based on these tests a number of conclusions were drawn and recommendations are provided for design of through bolted connections.

Type II Hybrid Masonry utilizes headed studs to transfer shear from the steel floor beam to the CMU shear panel below. Type III Hybrid Masonry utilizes headed studs to transfer shear from both the steel floor beam and the columns to the CMU shear panel. Three fully grouted 8" thick CMU walls were tested to evaluate the performance of headed studs at different spacings in these connections. In all cases, the headed studs were ¾" diameter by 6" long, welded to the bottom of the floor beam bottom flange. They were embedded in a 9" high by 7-5/8" thick grout beam on top of the 8" CMU wall. The top course of the CMU wall was a bond beam with one #4 bar and the wall vertical reinforcement consisted of #4 bars at 24" on center. Conclusions and recommendations are presented based on the results of these tests.
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1 Introduction

1.1 Introduction

There are many responsibilities that Structural Engineers are given. These include the integrity of existing buildings, design of new buildings, as well as being economical with the designs. Designing new buildings involves deciding which structural system should be used as the main component to resist any lateral loads that are applied through everyday environments to natural hazards that may occur (e.g. hurricanes and earthquakes). Steel frames have been used in buildings for many years and have shown their efficiency as well as strength. Another structural system that has been used in many applications is concrete masonry unit (CMU) bearing walls and shear walls.

When steel frames are used in new structures, CMU walls are often used to provide heat insulation and sound proofing. The infill walls are not utilized in the design of the steel frames because they are typically used only to resist the out-of-plane loads that are applied to the CMU walls. The CMU walls are also more complex to build in these structures because the steel frame system may use braced frames for lateral load resistance. With braced frames there is a diagonal brace going through the bay of the frame which requires the mason to cut CMU blocks diagonally to fit around the braces. Even though CMU shear walls are very effective in withstanding in-plane shear loads, they have not been utilized for this purpose in steel framed buildings.

Hybrid Masonry was introduced by Structural Engineer David Biggs. Hybrid Masonry involves utilizing the CMU walls within the steel frame for in-plane loads that are applied to the structure. Utilizing the CMU infill wall, the steel frames can be reduced in the following categories: size, weight, and the diagonal brace can be removed. There are three categories in Hybrid Masonry, Type I, Type II, and Type III. In the Type I Hybrid Masonry wall, the CMU infill wall is separate from the steel frame on three sides as shown in Figure 1-1. 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 frame and CMU infill wall. These connector plates transfer lateral loads being applied by the steel frame to the CMU infill wall, (Figure 1-2).

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 II Hybrid Masonry walls there is no gap between the top of the CMU wall and the top beam in the steel frame as shown in Figure 1-3. The top course of the CMU wall is replaced with a grout beam connecting the top of the CMU wall to the bottom flange of the steel beam above. Headed studs welded to the bottom flange of the top steel beam provide the shear transfer from beam to CMU wall. The headed studs offer a rigid connection compared with the ductile steel fuses used in Type I Hybrid Masonry. A typical construction and layout of the headed stud connection is shown in Figure 1-5 [Biggs, 2010]. A few changes were made in the design of the headed stud connection.
for UHM’s research and will be discussed in Chapter 4. Non-linear seismic response is concentrated in the CMU wall panels. The gap between the sides of the CMU wall and the steel columns remains but can be smaller than for Type I.

![Figure 1-3: Type II Hybrid Masonry](image)

Figure 1-3: Type II Hybrid Masonry

Type III Hybrid Masonry is similar to Type II, although the gaps between the sides of the CMU wall and the steel columns are removed and headed studs are added to the columns as shown in Figure 1-5. This type of Hybrid Masonry is the most rigid of the three types and is expected to obtain the highest lateral load capacity of the three types. The sacrifice is that the CMU wall may need to be replaced with a new wall after a design level earthquake.

![Figure 1-4: Typical headed stud connection in Type II and Type III Hybrid Masonry walls.](image)
This research represents the fourth and final phase of Hybrid Masonry testing at the University of Hawaii at Manoa (UHM). The research program at UHM is part of an ongoing project led by Professor Daniel Abrams at the University of Illinois, Urbana-Champaign (UIUC). The project was 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. High seismic zones would involve a Seismic Design Category (SDC) of D and E. UHM was given the responsibility of developing the steel connector plates between the steel frame and the CMU infill walls for the Type I Hybrid Masonry. The research for this portion of the project was performed by Seth Goodnight, Reef Ozaki-Train, and Steven Mitsuyuki under the direction of Dr. Ian Robertson. Information on the development of steel fuse connections used for Type I Hybrid Masonry walls is provided in reports by Seth Goodnight, Reef Ozaki-Train, and Steven Mitsuyuki, [Goodnight et al, 2011; Ozaki-Train et al, 2011; Mitsuyuki et al, 2012]. This report will focus on the determination of the failure modes of the through bolt for the Type I Hybrid Masonry walls and the connection behavior of the headed studs for the Type II Hybrid Masonry walls. The results of this research will aid UIUC in their development of the large-scale Hybrid Masonry specimens to be tested at the NEES MUST-SIM laboratory at UIUC.

1.2 Objective

The scope of the research included in this report is limited to the investigation of the through bolt connection used in the Type I Hybrid Masonry connections and the behavior of the headed stud connection used in the Type II and III Hybrid Masonry walls.
2 Literature Review

2.1 Hybrid Masonry Lateral Load Resisting System

Hybrid Masonry is a new structural system that was introduced in 2006 [Biggs, 2006]. This system incorporates two well tested structural systems widely used in the industry, steel frames and ductile reinforced CMU shear walls. There are three different types of Hybrid Masonry systems. The types vary in the components of loads transferred from the steel frame to the masonry as well as the load paths as shown in Figure 2-1 [Biggs, 2011].

![Figure 2-1: Hybrid Masonry Types I, II, III, shown clockwise from top left.](image)

A Type I Hybrid Masonry wall is designed to allow only in-plane shear loads to transfer from the top beam to the CMU wall through either fuse connectors or link plates. To ensure no vertical loads are transferred to the CMU wall, a 1 inch gap is left at the top of the CMU wall. The fuse connectors and link plates have vertically slotted holes for the through bolt to ensure only lateral forces are transferred through the connector plates to the CMU wall. The Type I system with fuse connectors is unique from the other types of Hybrid Masonry because the designed failure mechanism is ductile yielding of the connector plates. This reduces the damage to the CMU wall.
After a seismic event the connector plates will be the only damaged component of the system and may need to be replaced. Minor fills may be required for the CMU walls, but no major structural damage should have occurred. This efficient and economical repair can be obtained through proper design of the fuse connectors in the Type I Hybrid Masonry walls.

A Type II Hybrid Masonry wall is constructed with a rigid connection between the top of the CMU wall and the top steel beam. The rigid connection is designed with headed studs welded to the bottom flange of the top beam and the top course of CMU is replaced with a grout beam to bind the top steel beam to the CMU wall. This rigid connection also allows vertical loads to be transferred from the steel frame to the CMU infill wall. A 1 inch gap is provided between the sides of the CMU infill wall and the steel columns. This is efficient because reinforced CMU works well in compression and will reduce the size and weight of the top steel beam. The sacrifice with this type compared to Type I is that the damage incurred in the CMU wall during a seismic event could be significant. The rigid connection between the steel frame and the CMU wall requires that the deformations experienced during an earthquake be resisted by ductility in the CMU wall.

Both structural steel frames and reinforced CMU shear walls are addressed in the International Building Code [IBC, 2009]. The IBC references both the American Institute of Steel Construction Manual (AISC) for structural steel systems and the Specifications for Masonry Structures (ACI 530-08) [MSJC 2008]. There have been buildings designed with Hybrid Masonry in the United States, however, only in low seismic zones (SDC A, B, and C) [IMI, 2009]. The systems were designed with the criteria set forth in the IBC for reinforced CMU shear walls. The Hybrid Masonry systems are predicted to follow the design parameters set forth for reinforced CMU shear walls, although there are three different classifications. Reinforced CMU shear walls are designated as ordinary, intermediate, and special reinforced masonry shear walls. Table 2-1 shows the criteria assigned for the response modification factor (R), system over-strength factor ($\Omega_0$), deflection amplification factor ($C_d$), and height limitations based on the structure’s SDC. The use of Hybrid Masonry in high seismic zones is currently under investigation.

<table>
<thead>
<tr>
<th>Wall Classification</th>
<th>$R$</th>
<th>$\Omega_0$</th>
<th>$C_d$</th>
<th>Structural System Limitations and Building Height (ft.) Limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ordinary reinforced masonry shear walls</td>
<td>2</td>
<td>2.5</td>
<td>2</td>
<td>B: NL, 160; C: NP, D: NP, E: NP, F: NP</td>
</tr>
<tr>
<td>Intermediate reinforced masonry shear walls</td>
<td>4</td>
<td>2.5</td>
<td>4</td>
<td>B: NL, C: NL, D: NP, E: NP, F: NP</td>
</tr>
<tr>
<td>Special reinforced masonry shear walls</td>
<td>5.5</td>
<td>2.5</td>
<td>4</td>
<td>B: NL, C: NL, D: 160, E: 160, F: 100</td>
</tr>
</tbody>
</table>

* NL = Not Limited; 160 = Building Height Limit (ft.); NP = Not Permitted
2.2 UHM Research

2.2.1 Bolt Push Out Tests

The ACI 530-08 provides design guidelines for shear strength of cast-in-place anchor bolts (J- and L- bolts) into grouted masonry [MSJC, 2008]. However, code requirements do not exist for through-bolts in masonry. In Phase I of hybrid masonry connector plate development research, Goodnight performed a series of bolt push out tests to determine the shear strength of post-installed through-bolts for masonry [Goodnight et al, 2011]. Four masonry specimens were constructed with different design features (i.e. fully-grouted or partially grouted, having either a half-block or full-block in the outer-center cell). Each specimen was loaded in displacement control, with the load being transferred into the wall by means of a through-bolt inserted in the second cell from the wall edge, as shown in Figure 2-2. Most of the specimens were loaded twice, first in one direction until failure, then rotated 180-degrees and loaded again.

![Figure 2-2: Bolt push out test schematic](1in = 25.4mm)

A majority of the masonry specimens experienced shear breakout failure at or above 20 kips (89 kN). In addition, these tests proved that ¾ “ diameter A307 threaded rod fails at or before the breakout failure of the masonry, showing that bolts or threaded rods stronger than ¾ “ diameter A307 should be used for future testing. For full-scale testing, these tests indicated that the strength for a pair of fuse connector plates should remain below the breakout failure load of 20 kips (89 kN) in order for the fuse connectors to remain the ductile elements in the system. Also, the edge distance to the bolt proved to be vital in the breakout strength of the masonry. In order to protect the future full-scale wall specimens from break-out failure, it was decided that the UIUC wall tests would be performed with through-bolts no closer than the third cell from the edge of the walls rather than the second cell from the edge [Goodnight et al, 2011].
2.2.2 Connector Plates

2.2.2.1 Connector Plate Development Research - Phase I

In addition to the masonry bolt push out testing during Phase I of the connector plate development research, Goodnight also performed numerous cyclic displacement tests on steel connector plates. Goodnight tested various connector plate shapes and two different methods for connecting the plate to the bottom of the beam flange above [Goodnight et al, 2011]. Figure 2-3 shows the various shapes of connectors tested during his research. All of the connector plate specimens were 4 inches (102 mm) wide with varying thicknesses of ¼-inch to ½-inch (6.4 mm to 13 mm). A majority of the connectors tested were straight in profile and fillet welded to a plate meant to represent the bottom flange of the beam above. However, a second series of tests investigated the connectors in a bent plate orientation, which were intended to hang off the top of the bottom flange of the beam above (similar to the orientation shown in Figure 2-3).

Goodnight’s testing showed that the link connector plate and tapered fuse connector plate (Fuse Type T in Figure 2-3) were the most successful connector plate shapes. The link connector plate achieved a relatively large load of 10 kips (45 kN) at a displacement of 1.25 inches (32 mm), rupturing the welded connection to the bottom flange. The tapered fuses also provided promising results, showing that each fuse was capable of resisting 3 kips (13 kN) of lateral load and dissipating large amounts of energy while performing numerous cycles at large displacements of 3 inches (76 mm) [Goodnight et al, 2010]. These results showed the viability of link connector plates and tapered fuse connector plates for future tests. In addition, the results showed that the nominal width of the tapered fuses should be increased if higher loads are to be achieved. The bent plate testing proved unsuccessful due to premature failure of the flange weld, caused by prying action [Goodnight et al, 2011]. The bent connector plate orientation was not sufficient for the purposes of transferring large cyclic loads. Another important result of Goodnight’s testing was that connector plates thinner than ½-inch (13 mm) experienced excessive and undesirable out-of-plane buckling [Goodnight et al, 2011].

![Figure 2-3: Diagrams of various connector types tested in inverted orientation](1in = 25.4mm)
During this phase of testing, all of the specimens were either straight or bent connector plates welded to a base plate. However, this connector profile led to undesired weld rupture at the base plate. Consequently, for Phase II of testing, the connector profile was changed to a side plate orientation rather than the straight or bent profile with a bolted connection to mitigate the weld rupture failure. This involved testing the connector plates in the actual orientation rather than previous tests in the inverted orientation. Also, for these tests, side plates were welded to the steel beam above and the connector plates were either slip critically bolted with A490 bolts or welded to these side plates, while the bottom frame of the test setup had bolts for the slotted hole connections at the bottom of the connector plate [Ozaki-Train et al, 2011].

2.2.2.2 Connector Plate Development Research - Phase II (Ozaki-Train)

In Phase II of the connector plate development research, Reef Ozaki-Train performed tests on various link and fuse connector plates. All of the connector plates were ½-inch (13 mm) thick and tested in pairs to resemble the actual orientation of connector plates on each face of the masonry wall. Test results for a pair of 6-inch (152 mm) wide bolted link connector plates, as shown in Figure 2-4, showed that a pair of these link connector plates could achieve a maximum of approximately 35 kips (160 kN) of load before rupture at the net cross-sectional area at the center of the bottom pair of bolts. Similarly, three tests on 6-inch wide tapered fuse connector plates were performed, each with a different connection to the beam above. As shown in Figure 2-4 and Figure 2-5, a straight plate bolted connection, straight plate welded connection, and bent plate welded connection were each tested in pairs to determine if the different connection types would cause premature failure of the connectors. All three fuse connectors performed as expected, achieving a maximum of approximately 18 kips (80 kN) per pair of fuse connectors with numerous cycles at 3-inch (76 mm) displacement. All three of the connection types tested proved to be viable connecting options for this fuse type [Ozaki-Train et al, 2011].

Prior to this research, only tests on 4-inch wide connector plates were performed. Ozaki-Train’s testing showed that both the 6-inch wide link and tapered fuse connector plates are viable design options for future testing when more load transfer capacity is required than with the 4-inch wide connector plates. This testing also showed that more than one option is available for connecting the plates to the beam above (i.e. bolting to a side plate, welding to a side plate, or in a bent-plate profile welded to the web and bottom flange of the beam). However, both the welded connectors and connectors in the bent-plate orientation are more difficult to replace than the bolted connectors. Since the fuse connectors are meant to be replaceable, a bolted connection to the side plate is the most practical connection for the fuse connectors.
Figure 2-4: 6” wide straight link connector plate on the left and 6” wide tapered fuse connector plate on the right [Ozaki-Train et al, 2011] (Dimensions shown in mm) (1in. = 25.4mm)

Figure 2-5: 6” wide welded straight fuse connector plate on the left and 6” wide welded bent fuse connector plate on the right [Ozaki-Train et al, 2011] (Dimensions shown in mm) (1in. = 25.4 mm)
2.2.2.3 Connector Plate Development Research – Phase III (Mitsuyuki)

In Phase III of the connector plate development research, Steven Mitsuyuki performed tests on 8" thick CMU walls using various configurations of link and fuse connector plates. All the plates were ½" thick and tested with different numbers of plates per wall specimen. There were two wall specimens tested in Mitsuyuki’s research. The first wall specimen was partially grouted with #5 Grade 60 vertical reinforcing spaced at 24” on centers and one #5 Grade 60 horizontal bar in the bond beam located in the second course from the top of the CMU wall. The plates were bolted to the side plates on the top beam and to the wall specimen with a ¾ “ diameter Grade 8 through bolt. Three experiments were run on the first wall specimen. With one pair of 4” fuse connectors, the maximum capacity held by the specimen was 6.4 kips. Two pairs of 4” fuse connectors held a maximum capacity of 12.8 kips. Finally, at three pairs of 4” fuse connectors, the maximum capacity of the specimen was 19.2 kips. The main finding with these experiments was that the capacity of the fuse connectors had a linear relationship. The capacity could easily be determined by multiplying the capacity of one 4” fuse connector by the total number of fuses used on the wall specimen.

On the second wall specimen the 6” fuse plates were tested. There was only one test run with this wall specimen because the two pairs of fuses connecting the top beam to the CMU infill wall caused the wall specimen to fail. The maximum capacity of the fuses were 42.2 kips being double that of the pair of 6” fuse plates tested by Goodnight.
Material Properties

3.1 Compression Tests

A series of compression tests were performed to determine the strength of the CMU used in the wall specimens. Tests were performed on mortar cubes (Figure 3-1), four and six inch diameter grout cylinders (Figure 3-2), and CMU single cell prisms (Figure 3-3). Each of the cylinder and prism specimens were capped with sulfur capping compound per ASTM C1552-09a. The test procedures complied with ASTM C1314-11a to obtain the ultimate compressive strength of the CMU prisms and the ASTM C39 was followed to obtain the ultimate compressive strength of the concrete cylinders. The testing machine used for the compression test was a Riehle Four Post Frame which has a capacity of 300,000 pounds.

The compression test average results for each testing set is shown in Table 3-1. The individual testing values are given in the Appendix B.

Table 3-1: Average compressive strength of grout, mortar, and CMU prisms.

<table>
<thead>
<tr>
<th>Material</th>
<th>Average compressive strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout</td>
<td>5118</td>
</tr>
<tr>
<td>Mortar</td>
<td>3934</td>
</tr>
<tr>
<td>CMU Prism</td>
<td>2727</td>
</tr>
</tbody>
</table>

Figure 3-1: Typical compression failure of 2” mortar cubes.
3.2 Reinforcing Bar Direct Tension Tests

Direct tension tests were performed to obtain accurate values for the material properties of the ASTM grade 60 reinforcing steel used in the CMU walls for the connector tests, as shown in Figure 3-4. The grade 60 reinforcing bars have a required minimum yield strength of 60 ksi (414 MPa), but the actual strength of the material can often be higher. Similarly, the ultimate strength of grade 60 steel is required to be at least 90 ksi (621 MPa), but may be higher. Tension tests were performed on four
samples of the #4 and #5 (16 mm) bars each. The tension test results are summarized in Table 3-2. The stress-strain curves are shown in Appendix C.

![Figure 3-4: Steel Reinforcing Bar Tension Test Setup](image)

**Table 3-2: Grade 60 Steel Reinforcing Bar Tension Test Results**

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Rebar Size</th>
<th>Yielding Stress (ksi)</th>
<th>Ultimate Stress (ksi)</th>
<th>Young's Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>#4</td>
<td>73</td>
<td>116</td>
<td>29400</td>
</tr>
<tr>
<td>2</td>
<td>#4</td>
<td>74</td>
<td>118</td>
<td>29676</td>
</tr>
<tr>
<td>3</td>
<td>#4</td>
<td>74</td>
<td>118</td>
<td>31079</td>
</tr>
<tr>
<td>4</td>
<td>#4</td>
<td>73</td>
<td>116</td>
<td>31065</td>
</tr>
<tr>
<td>5</td>
<td>#5</td>
<td>108</td>
<td>119</td>
<td>30876</td>
</tr>
<tr>
<td>6</td>
<td>#5</td>
<td>104</td>
<td>120</td>
<td>32530</td>
</tr>
<tr>
<td>7</td>
<td>#5</td>
<td>105</td>
<td>120</td>
<td>34845</td>
</tr>
<tr>
<td>8</td>
<td>#5</td>
<td>102</td>
<td>120</td>
<td>28532</td>
</tr>
<tr>
<td><strong>AVERAGE</strong></td>
<td>-</td>
<td><strong>89</strong></td>
<td><strong>118</strong></td>
<td><strong>31000</strong></td>
</tr>
</tbody>
</table>

(1 ksi = 6.895 MPa)
4 Approach

4.1 Masonry Wall Specimen Testing

4.1.1 Experimental Setup (Type I and II)

The test frame used for the bolt push out and Type II Hybrid Masonry specimen tests is similar to the frame setup for the Type I Hybrid Masonry specimens. The full test setups for the through bolt push-out and the Type II Hybrid Masonry are shown in Figure 4-1 and Figure 4-2, respectively.

Figure 4-1: Test frame setup for the bolt push-out experiments.
Figure 4-2: Test frame setup for Type II Hybrid Masonry experiments.

The in-plane system consisted of a structural steel reaction-frame supporting a horizontally oriented 300,000 pound MTS hydraulic actuator. The wide-flanged (WF) beam above the CMU wall is maintained in a horizontal position by means of two pin-ended load rods. The following portions of the testing frame were tensioned to the laboratory strong floor with either 1” or 2” diameter threaded rod: The reaction-frame, load rods, and hybrid wall specimen. In the event that friction was not sufficient to prevent specimen movement, two 1” diameter reinforcing bars were welded on each side of the testing frame to two intermediate support points, and the Hybrid Masonry specimen. The purpose for these bars was to provide a tie between the testing frame and the specimen and restrain relative displacement between the components of the testing frame.

Lateral bracing was provided to restrain the reaction-frame and top WF steel beam from moving out-of-plane during loading. The reaction-frame had a structural steel HSS 4x4 strut bracing the top of the reaction-frame against a concrete wall. Four pin-ended unistrut members connected the top WF beam to the adjacent concrete wall to prevent out of plane displacement or twisting. The unistrut members were adjusted for every test to ensure the top WF beam and the load rods were vertical at the start of each test. Figure 4-3 and Figure 4-4 show the test setup.
4.1.2 Experimental Setup Strengthening

During testing of the first Type II Hybrid Masonry wall with headed studs at 8” on centers, portions of the reaction-frame began to yield and the 1” diameter clevis pin and bearing used to connect the MTS hydraulic actuator to the top WF beam failed during the final cycle of wall loading. After the experiment was completed the testing frame was strengthened as follows: The bolt holding the edge of the A-frame down was increased from a 1” diameter bolt to a 2” diameter bolt, stiffener plates, and structural steel HSS 4x4 tubes were welded to each side of the bottom channel, and the clevis pin was increased to \(1 \frac{3}{8}“\) diameter. The retrofits ensured that the testing frame did not fail from the loads applied during subsequent tests.

During some of the Type II Hybrid Masonry specimens and the bolt pushout tests the bolted clevis plate connected to the top WF beam started yielding under higher tension loads. During subsequent compression loads this non-linear deformation was restored, resulting in a period of low stiffness in the control load-displacement hysteretic loops. For the final Type II wall test the clevis plate was welded to the WF beam end plate to prevent this yielding behavior.

Figure 4-3: Full view of experimental test setup
4.2 Type I Hybrid Masonry Bolt Push-Out Test

4.2.1 Masonry Wall Specimens

A series of six wall specimens were constructed by masons from Ono Construction for the bolt pushout tests. All specimens were constructed with fully-grouted standard 8" x 8" x 16" CMU blocks and Grade 60 reinforcing steel without joint reinforcing. The dimensions of the specimens were 6'-8" long and 2'-8" high (e.g. five blocks long and four courses high). The second course from the top was a bond beam. These dimensions were used to simulate a portion of the CMU infill wall on the second story of a typical building structure. The CMU walls were built on a 6" thick reinforced concrete slab on a W24x76 steel beam which simulates a typical concrete floor system in a steel framed building. The steel beam was bolted to the concrete strong floor with three bolts. The two end bolts were 2" diameter threaded rods and one ¾ “diameter threaded rod in the center. At the threaded rod locations on the W24x76 beam, stiffener plates were welded to strengthen the beam web.

4.2.2 Experimental Test Specimens

Six wall specimens were built for the bolt push-out tests. The specifications of the wall specimens are shown in Table 4-1. In the specimen name, the first three letters indicate that the wall is a fully grouted wall, FGW. The next set of letters and numbers show the number and size of reinforcing bars used in the bond beam (BB). The number
at the end of the name shows the test number for that specimen. Four Grade 8 – 1” diameter bolts were used to connect the double link plates to the steel side plates welded on the top WF beam. Load indicating washers were used to ensure the bolts were properly torqued to obtain slip-critical connections.

Three different test procedures were used for the bolt pushout wall specimens. Wall specimen FGW-1#4BB-G-1 was the only specimen to be tested as a group. The through bolts were place 12” and 28’’ from the edge of the CMU wall as shown in Figure 4-5. Two test were performed on the remaining bolt pushout wall specimens. Test number 1 was done on a through bolt in the bond beam 12”, second cell, from the edge of the CMU wall and test number 2 was on a through bolt in the bond beam 20”, third cell, from the edge of the CMU wall. These tests are shown in Figure 4-6 and Figure 4-7.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vert. rein. bar size</th>
<th>Spacing of vert. rein. bars (in.)</th>
<th>Horiz. rein. bars</th>
<th>Horiz. rein. bar location</th>
<th>Push-out test type</th>
</tr>
</thead>
<tbody>
<tr>
<td>FGW-1#4BB</td>
<td>#4</td>
<td>24</td>
<td>1-#4</td>
<td>Second course from top</td>
<td>1) 12” from edge, 2) 20” from edge</td>
</tr>
<tr>
<td>FGW-1#4BB-G</td>
<td>#4</td>
<td>24</td>
<td>1-#4</td>
<td>Second course from top</td>
<td>1) 12” and 28” from edge (group)</td>
</tr>
<tr>
<td>FGW-1#4BB-TCH</td>
<td>#4</td>
<td>24</td>
<td>1-#4</td>
<td>Second course from top, top course hollow</td>
<td>1) 12” from edge, 2) 20” from edge</td>
</tr>
<tr>
<td>FGW-1#4TBB</td>
<td>#4</td>
<td>24</td>
<td>1-#4</td>
<td>Top course</td>
<td>1) 12” from edge, 2) 20” from edge</td>
</tr>
<tr>
<td>FGW-2#4BB</td>
<td>#4</td>
<td>24</td>
<td>2-#4</td>
<td>Second course from top</td>
<td>1) 12” from edge, 2) 20” from edge</td>
</tr>
<tr>
<td>FGW-3#4BB</td>
<td>#4</td>
<td>24</td>
<td>3-#4</td>
<td>Top three courses</td>
<td>1) 12” from edge, 2) 20” from edge</td>
</tr>
</tbody>
</table>

Link plates were used to transfer the horizontal load from the top steel beam to the CMU wall. The link plates were 6” x 30” x ½ “ thick and the bolt connecting the plates to the CMU wall was a 1” diameter x 12” long, high strength Grade 8 (minimum tensile strength of 65 ksi and minimum yield strength of 50 ksi).
Figure 4-5: Direction of loading for FGW-1#4BB-G-1.

Figure 4-6: Direction of loading for FGW-1#4TBB.
Figure 4-7: Direction of loading for FGW-1#4BB, FGW-1#4BB-TCH, FGW-2#4BB, and FGW-3#4BB.

Figure 4-8: Hybrid Masonry wall specimen FGW-1#4BB reinforcing steel layout.
4.2.2.1 Steel Beam Above Wall Specimens

A W18x40 structural steel beam was placed above the CMU wall to transfer the lateral forces from the hydraulic actuator. The steel beam was laterally braced to restrain lateral displacement and rotation during loading of the specimen. For Hybrid
Type I specimens there was a 1” gap between the top beam and the wall specimen to ensure no vertical forces were transferred to the CMU wall. The top beam was supported by two pin-end vertical load rods to keep the beam horizontal during loading. The two load rods had an axial capacity of 35 kips. The load rods also monitored the overturning forces due to the lateral load applied to the top beam. Negative readings indicated compression in the rods. A two-part clevis was used to connect the hydraulic actuator to the end of the top beam. The pin in the clevis was 1-¾” diameter Grade 8 steel.

In order to bolt the fuse or link plates to the top beam, three 8” wide by ¾” thick Grade A36 steel plates were welded to the edges of the beam top and bottom flanges with 7/16” fillet welds. Three plates were welded on each side of the beam to accommodate connector plates at different locations on the CMU wall. Each plate had six 1-1/16” diameter holes as shown in Figure 4-11. The plate surfaces were cleaned with a grinder for the slip critical connection with the link and fuse connector plates. The locations of the plates were determined by the required placement of the through bolts in the CMU wall.

![Figure 4-11: Steel side plates on top beam](image)

**4.2.3 Test Instrumentation**

Various instruments were used to monitor the specimen response during testing. All instruments were monitored at 2 Hz throughout each test. An MTS actuator capable of applying loads of ±300 kips (1335 kN) and displacements of ±15 inches (381 mm) provided the horizontal load applied to the top beam above the masonry wall specimen.
There were two different deflection instruments used in the experiments, namely linear variable displacement transducers (LVDT) and string potentiometers (SP).

For the bolt pushout tests the LVDTs were used to monitor relative displacement between the top beam and the edge of the CMU wall and between the CMU wall and the through bolt. An SP was used to monitor the displacement of the top steel beam relative to the middle of the CMU wall (Figure 4-13).

![Image of instrumentation layout](image)

**Figure 4-12: Instrumentation layout for bolt push-out tests.**

![Image of close up of sensors](image)

**Figure 4-13: Close up of SP and LVDT sensors for bolt push-out tests.**
4.2.4 Procedure

4.2.4.1 Through Bolt Pushout Testing

Prior to testing, the wall specimens were bolted to the strong floor with three threaded rods. The nuts were tightened with an impact wrench to improve the friction between the floor and bottom wide flanged beam supporting the wall specimen. Then the bottom steel beam was welded to the two 1” diameter bars running along the bottom of the testing frame. The top beam was lifted above the wall specimen and connected to the load rods with ¾” pins. With the top beam vertically supported, the link plates were bolted to the beam side plates to align with the wall specimen. The top beam was moved in-plane with the wall specimen until the load rods were vertical. The lateral bracing was then bolted to the top beam.

With the link plates connected to the top beam, the hole placement for the through bolt was marked on the wall specimen. An impact drill was used to drill the hole for the through bolt in the wall specimen. A ¼” diameter pilot hole was drilled followed by the full size 1” diameter hole drilled from both sides to avoid breakout of the face of the CMU block. The double 6” link plates were then bolted to the CMU infill wall with A490 load indicating squirter-type washers with 1” diameter Grade 8 bolts to develop a slip critical connection. An impact wrench was used to torque the bolts until at least seven of the eight protrusions on the load indicating washer had squirted out. The specification for the load indicating washers is included in the Appendix. After all link plates were bolted, the 1” diameter through bolt was installed to secure each pair of link plates to the CMU wall. The outer link plates were painted with a layer of whitewash to observe yielding in the link plates through flaking of the whitewash.

A laptop was used to collect data from a National Instruments (NI) Data Acquisitions system (DAQ). The NI-DAQ recorded the load rods, string potentiometers, and LVDT data as well as the actuator load and displacement. Two video cameras were set up to record the testing: one of the cameras recorded a “global” view of the entire wall specimen to observe the entire system, while the other camera recorded a “local” view, which was focused on the area near the through bolt.

Once the MTS control system for the actuator and the NI-DAQ were running, the actuator was attached to the end of the top WF steel beam. The specimen was loaded monotonically in displacement control under a constant rate of 0.3 inches/min. The displacement continued until the wall reached its maximum capacity and the lateral load reduced below 80% of the maximum capacity.

4.3 Type II and III Hybrid Masonry Headed Stud Connection Test

4.3.1 Masonry Wall Specimens

Three specimens were constructed by Ono Construction for the Type II and III headed stud connection experiments. The typical construction from the foundation up consisted of the following: W24x76 steel base beam with ½” thick steel collars at the edges, 6” reinforced concrete slab, three courses of standard 8” x 8” x 16” CMU blocks, reinforced grout beam, and headed studs welded to the bottom flange of the top beam. Similar to the through bolt push-out test, no joint reinforcement was used. The grout beam took the place of the top course of CMU that was removed to fit the headed studs.
into the wall. The three specimens varied in the number and spacing of headed studs and the L-shaped reinforcing bars connecting the grout beam to the CMU wall. For the headed studs spaced at 8” on center and 24” on center, the connecting L-bars were placed at the same 24” on center spacing as the vertical bars in the CMU wall. For the specimen with headed studs at 16” on center, L-bars were also added to each cell aligned with a headed stud. This was intended to improve the connection between the steel beam and the CMU wall. The layout for FGW-HS16-1 is shown in Figure 4-14.

Figure 4-14: Hybrid Masonry wall specimen FGW-HS16-1 headed stud and reinforcing steel layout.

4.3.2 Test Instrumentation

For the Type II Hybrid Masonry specimens there were six SP sensors (Figure 4-5). Four diagonal SP sensors were used to determine deformation within the CMU wall. Two SP sensors were used to monitor the relative vertical deformation between the top corners of the CMU walls and the steel beam below. Four LVDTs were also used on the specimen. One LVDT monitored the relative horizontal deformation between the CMU wall and the concrete slab, while another monitored movement between the concrete slab and the steel beam below. The other two LVDTs were used to monitor the relative displacement between the top of the CMU wall and the laboratory floor and the top steel beam to the top corner of the grout beam. Two load cells, capable of recording ±30 kips (133.5 kN), were located on vertical load rods. These load cells measured the vertical forces required to keep the beam horizontal above the wall specimen. The sum of these forces is equal and opposite of the vertical load in the CMU wall. No vertical load was applied to the CMU wall during testing, even though
Type II and III Hybrid Masonry systems are designed to transfer gravity load through the CMU wall.

A steel collar was welded to each end of the floor beam to restrain the end of the concrete lab supporting the CMU wall (Figure 4-16). Prior tests had shown that premature failure of the slab edge reduced the CMU wall capacity. In reality, the slab is continuous and is therefore restrained against edge breakout.

Figure 4-15: Instrumentation layout for Type II and III Hybrid Masonry tests.
4.3.3 Procedure

4.3.3.1 Headed Stud Hysteretic Testing

Prior to testing, the procedure is similar to the through bolt push-out testing. The difference is that the top beam was connected to the CMU infill wall with a grout beam and headed studs. The same lateral bracing and vertical load rods were used in this experiment. No vertical loads were applied to the top beam although the full-scale wall tests for Type II and III Hybrid Masonry walls to be held at UIUC will have vertical loads applied to the top beam.

The cyclic displacement routine consisted of sets of three cycles at increasing displacements (Table 4-2 and Figure 4-17). This displacement routine was the same as used for the Type I wall tests. Each cycle consisted of the actuator starting at zero and retracting to the set maximum negative displacement for that cycle followed by an extension of the actuator past neutral to the maximum positive displacement for that cycle, finally concluding in the retraction of the actuator to the zero-displacement position.

The displacements used in the cycles were the same as the levels used for the Type I wall specimens [Goodnight et al, 2011]. The increments of displacement are
shown in Table 4-2. The cycling frequencies were constant at 0.01 Hz until 1” displacement was reached. The frequency was then decreased to 0.005 Hz.

Table 4-2: Displacement and frequencies followed during headed stud wall specimens.

<table>
<thead>
<tr>
<th>Deformation Step Number</th>
<th>Actuator Input Displacement (in)</th>
<th>Frequency (Hz)</th>
<th># of Cycles</th>
<th>Total Cycles</th>
</tr>
</thead>
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<tr>
<td>1</td>
<td>0.01</td>
<td>0.01</td>
<td>3</td>
<td>3</td>
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<tr>
<td>4</td>
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<td>0.01</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
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<td>3</td>
<td>15</td>
</tr>
<tr>
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<td>0.25</td>
<td>0.01</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>7</td>
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<td>0.01</td>
<td>3</td>
<td>21</td>
</tr>
<tr>
<td>9</td>
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<td>1.75</td>
<td>0.005</td>
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<td>36</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>0.005</td>
<td>3</td>
<td>39</td>
</tr>
</tbody>
</table>

Figure 4-17: Cyclic loading routine for Type II Hybrid Masonry
4.4 Future Hybrid Masonry Testing

During the next phase of testing, UIUC will be conducting large-scale tests based on the results obtained at UHM. UIUC will use two-story wall specimens for the Type I, II, and III Hybrid Masonry walls as shown in Figure 4-18. The Type I specimens will be subjected to the same lateral displacement cycling as used for the UHM tests. No vertical load will be applied to the specimen.

For the Type II and III specimens a constant vertical load will be applied to the specimen while it is cycled through the same series of increasing lateral displacement.
5 Results

5.1 Through Bolt Push-out Results

5.1.1 Specimen Narratives and Load-Displacement Diagrams

The following sections describe the performance of each of the eleven bolt push-out tests. Monotonic load-displacement plots are provided for each specimen based on LVDT measurement of displacement of the through bolt relative to the CMU wall.

5.1.1.1 FGW-1#4BB-1

Specimen FGW-1#4BB-1 represents the control specimen for many of the bolt push-out tests. In FGW-1#4BB-1, the through bolt was positioned in the bond beam, second course from the top, and two cells from the edge of the CMU wall (Figure 5-1). The first cracks occurred at 20 kips load, and the maximum capacity of the bolt was 28.3 kips at a displacement of 0.6” relative to the undamaged interior portion of the CMU wall (Figure 5-1). The failure was a vertical tension crack extending from the through bolt along the mortar and through the middle of the CMU block above. The crack continued to the bottom corner of the concrete footing. During the test, the LVDT monitoring the displacement of the through bolt relative to the CMU wall de-bonded at a displacement of 2.4”. Figure 5-2 shows the load versus displacement diagram for the experiment.

Figure 5-1: Failure cracks in FGW-1#4BB-1
5.1.1.2 FGW-1#4BB-2

In FGW-1#4BB-2, the through bolt was positioned in the bond beam, second course from the top, and three cells from the edge of the CMU wall (Figure 5-3). This test was performed on the same specimen as FGW-1#4BB-1, but on the opposite end of the wall. The first cracks occurred at 24 kips load, and the maximum capacity of the bolt was 38.3 kips at a displacement of 0.33" relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack extending from the through bolt along the double link plate to the top of the CMU wall. The crack continued to the bottom corner of the CMU wall (Figure 5-3). The initial portion of the load versus displacement diagram in Figure 5-4 shows the effect of yielding behavior in the testing frame discussed in the Approach section of this report. At approximately 0.32" displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a sudden drop in load.

![Figure 5-2: Load versus displacement diagram of FGW-1#4BB-1.](image)
5.1.1.3 FGW-1#4BB-G-1

This specimen was intended to evaluate the group effect of multiple through bolts. In FGW-1#4BB-G-1, there were two through bolts positioned in the bond beam, second course from the top. One was placed two cells from the edge of the CMU wall.
while the other was placed four cells from the edge (Figure 4-5). The two through bolts were tested together in this experiment. The first cracks formed at 29 kips load, and the maximum capacity of the bolts was 37.8 kips at a displacement of 0.88” relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack going through the CMU block from the bolt hole to the mortar and it continued through the CMU block above to the top of the CMU infill wall (Figure 5-5). The crack continued nearly vertical to the concrete footing and then went along the mortar bed between the bottom course and the concrete slab (Figure 5-5). The failure of the specimen at the maximum capacity of the wall occurred when the #4 bar in the bond beam failed due to tension rupture (Figure 5-6). This showed that the group of through bolts was able to mobilize the CMU wall and the bond was strong enough to make steel yielding and rupture the failure mechanism. This is optimal because steel is a ductile material and the failure strength can be predicted relatively accurately. Figure 5-7 shows the load versus displacement diagram for the experiment, indicating large ductility prior to failure of the #4 bar in the bond beam at 2.7” displacement.

Figure 5-5: Failure cracks in FGW-1#4BB-G-1.
5.1.1.4 FGW-1#4BB-TCH-1

This specimen was intended to evaluate the effects of an un-grouted top course on the CMU wall. Because the CMU wall will typically be constructed after the steel frame, it would be difficult to grout the top course of the CMU wall because of the beam flange above. In FGW-1#4BB-TCH-1, the through bolt was positioned in the bond beam, second course from the top, and two cells from the edge of the CMU wall (Figure 5-8). The top course of this wall specimen was left un-grouted. The first cracks formed at 14 kips load, and the maximum capacity of the bolt was 14.8 kips at a displacement of 0.23” relative to the undamaged interior portion of the CMU wall. The failure was a
vertical tension crack extending from the through bolt to the top of the CMU wall. The crack continued to the bottom corner of the concrete slab (Figure 5-8). At approximately 0.45" displacement, the bond between the CMU wall specimen and the horizontal reinforcing bars failed causing a sudden drop in load. Figure 5-9 shows the load versus displacement diagram for the experiment.
5.1.1.5 FGW-1#4BB-TCH-2

In FGW-1#4BB-TCH-2, the through bolt was positioned in the bond beam, second course from the top, and three cells from the edge of the CMU wall (Figure 5-10). The top course of this wall specimen was left un-grouted. This test was performed on the same specimen as FGW-1#4BB-TCH-1, but on the opposite end of the wall. The first cracks formed at 22 kips load, and the maximum capacity of the bolt was 31.0 kips at a displacement of 0.84" relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack extending from the top of the CMU wall down near the interior face of the double link plate. The crack continued nearly vertical to the bottom course and then diagonally to the bottom corner of the concrete slab (Figure 5-10). At approximately 0.83" displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a sudden drop in load. Figure 5-11 shows the load versus displacement diagram for the experiment.

![Failure cracks in FGW-1#4BB-TCH-2.](image)

Figure 5-10: Failure cracks in FGW-1#4BB-TCH-2.
5.1.1.6 FGW-1#4TBB-1

This specimen was intended to evaluate the effect of placing the through bolt in the top course of the CMU wall. In FGW-1#4TBB-1, the through bolt was positioned in the bond beam at the top course, and two cells from the edge of the CMU wall (Figure 5-12). The first cracks formed at 12 kips load, and the maximum capacity of the bolt was 12.9 kips at a displacement of 0.15” relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack extending from the through bolt along the exterior face of the double link plates to the top of the CMU wall. The crack continued to the bottom corner of the CMU wall (Figure 5-12). The load versus displacement plot for this specimen is shown in Figure 5-13. The load spike at approximately 0.15” displacement was due to an error in the load cell readings. This increase in capacity was a false reading and was ignored when determining the maximum load applied to the wall specimen.
5.1.1.7 FGW-1#4TBB-2

In FGW-1#4TBB-2, the through bolt was positioned in the bond beam at the top course, and three cells from the edge of the CMU wall (Figure 5-14). This test was performed on the same specimen as FGW-1#4TBB-1, but on the opposite end of the
The first cracks formed at 16 kips load, and the maximum capacity of the bolt was 19.9 kips at a displacement of 0.60" relative to the undamaged interior portion of the CMU wall. The failure was a tension crack extending from the through bolt along the interior face of the double link plates to the top of the CMU wall. The crack continued to the bottom corner of the CMU wall (Figure 5-14). The initial portion of the load versus displacement diagram in Figure 5-15 shows the effect of yielding behavior in the testing frame discussed in the Approach section of this report. At approximately 0.63” displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a sudden drop in load.

Figure 5-14: Failure cracks in FGW-1#4TBB-2.

Figure 5-15: Load versus displacement diagram of FGW-1#4-TBB-2.
5.1.1.8 FGW-2#4BB-1

This specimen was intended to evaluate the effect of increasing the horizontal reinforcing steel in the bond beam to 2#4 bars. In FGW-2#4BB-1, the through bolt was positioned in the bond beam, second course from the top, and two cells from the edge of the CMU wall (Figure 5-16). The first cracks formed at 30 kips load, and the maximum capacity of the bolt was 30.5 kips at a displacement of 3.06” relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack extending from the through bolt along both edges of the double link plates to the top of the CMU wall. The crack continued to the bottom corner of the concrete slab (Figure 5-16). Figure 5-17 shows the load versus displacement diagram for the experiment. At approximately 0.45” displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a drop in load.

![Failure cracks in FGW-2#4BB-1.](image-url)
5.1.1.9 FGW-2#4BB-2

In FGW-2#4BB-2, the through bolt was positioned in the bond beam, second course from the top, and three cells from the edge of the CMU wall. This test was performed on the same specimen as FGW-1#4TBB-1, but on the opposite end of the wall. The first cracks formed at 34 kips load, and the maximum capacity of the bolt was 37.2 kips at a displacement of 2.06” relative to the interior portion of the CMU wall. The final failure was a shear crack extending from the through bolt through the mortar stepping up to the top edge single cell block. The crack continued nearly vertical from the bolt hole to the concrete slab and then horizontally through the mortar between the CMU and concrete slab. This crack on the bottom eventually went through the concrete slab to the corner of the steel wide-flange beam below. Tension cracks also formed in the interior portion of the CMU wall (Figure 5-18). At approximately 0.24” displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a drop in load. Figure 5-19 shows the load versus displacement diagram for the experiment.
5.1.1.10 FGW-3#4BB-1

This specimen was intended to evaluate the effect of adding reinforced bond beams above and below the course supporting the through bolt. In FGW-3#4BB-1, the through bolt was positioned in the bond beam, second course from the top, and two cells from the edge of the CMU wall (Figure 5-20). The first cracks formed at 21 kips load, and the maximum capacity of the bolt was 26.3 kips at a displacement of 2.47" relative to the undamaged interior portion of the CMU wall. The failure was a vertical tension crack extending from the through bolt along interior edges of the double link plates to the top of the CMU wall. The crack continued to the bottom corner of the CMU wall.
infill wall (Figure 5-20). At approximately 0.26” displacement, the bond between the CMU wall specimen and the horizontal reinforcing bar failed causing a drop in load. Figure 5-21 shows the load against displacement diagram for the experiment.

![Figure 5-20: Failure cracks in FGW-3#4BB-1.](image)

![Figure 5-21: Load versus displacement diagram of FGW-3#4BB-1.](image)

5.1.11 FGW-3#4BB-2

In FGW-3#4BB-2, the through bolt was positioned in the bond beam, second course from the top, and three cells from the edge of the CMU wall (Figure 5-22). This test was performed on the same specimen as FGW-1#4TBB-1, but on the opposite end of the wall. The first cracks formed at 32 kips load, and the maximum capacity of the
A bolt was 49.3 kips at a displacement of 0.43” relative to the interior portion of the CMU wall. The final failure was tension cracks through the interior portion of the CMU wall. The cracks that formed in the CMU wall extended to the portion of the wall damaged by FGW-3#4BB-1 (Figure 5-22). At approximately 0.26” displacement, the side plate welded to the top wide-flanged steel beam failed causing termination of the experiment. After proper repairs were made to the side plate, the test resumed at zero displacement and followed the same displacement rate. The effect of this failure in the testing frame is evident in the load versus displacement plot shown Figure 5-23. A dashed line is added to indicate the anticipated load-displacement relationship if the side plate weld had not failed.

![Figure 5-22: Failure cracks in FGW-3#4BB-2.](image)

![Figure 5-23: Load versus displacement diagram of FGW-3#4BB-2.](image)
5.2 Headed Stud Specimen Results

5.2.1 Load-Displacement Diagrams and Specimen Narratives

An LVDT was mounted to an independent support system to record the relative displacement of the wall specimen to the strong floor. A minimum of two diagonal string potentiometers were mounted to the CMU wall from top corner to opposite bottom corner of the CMU wall on the same face. These string potentiometers recorded the deformation within the wall specimen during the cyclic loading. The LVDT and string potentiometer data were plotted against the applied load to create hysteresis plots. The following sections describe the hysteretic plot and narrative explaining significant events that occurred during testing of each specimen.

5.2.1.1 FGW-HS24-1

In FGW-HS24-1, four 6” long x ¾” diameter headed studs were welded to the bottom flange of the top wide-flanged steel beam at 24 inches on center with the first stud located in the end cell of the CMU wall. During the 0.5” displacement cycles, vertical cracks formed in the grout beam at the locations of the edge studs (Figure 5-24). The displacement readings from the LVDT were 0.093” when the cracks occurred. These cracks affected the LVDT readings because the contact point for the sensor was on the loose section of grout beam. The portion of the graph on Figure 5-25 that strays to the left is the readings after the 0.5” deflection cycles. The wall specimen had a maximum capacity of 60.1 kips at a displacement of 0.240”. This displacement and load capacity occurred during the first cycle to 0.75” actuator displacement. Because of edge cell breakout, only three studs were effective at transferring shear from steel beam to grout beam in each loading direction. Therefore, the shear in each stud at maximum capacity was 60.1 kips divided into three studs, equating to approximately 20 kips per stud. The displacement recordings from the MTS hydraulic actuator were different from the recording on the LVDT because of the deformation in the reaction frame. The ultimate failure of the wall specimen occurred during the initial cycle to 1.00” actuator displacement as shown in Figure 5-26. Failure involved splitting portion of the grout beam along the line of the headed studs (Figure 5-27).
Figure 5-24: Cracks caused by headed stud located in end cell of CMU wall.

Figure 5-25: Hysteretic plot of load versus displacement of FGW-HS24-1.
Figure 5-26: Failure cracks in FGW-HS24-1.

Figure 5-27: Crack observed in grout beam after failure of FGW-HS24-1.

5.2.1.2 FGW-HS16-1

In FGW-HS16-1, five 6" long x ¾" diameter headed studs were welded to the bottom flange of the top wide-flanged steel beam at 16 inches on center with the first stud starting in the second cell from the edge of the CMU wall. At the right side of the CMU wall two headed studs had to be placed eight inches on center because of the uneven spacing for a total of five studs (Figure 5-28). The displacement recordings from the MTS hydraulic actuator were different from the recording on the LVDT because of deformation in the reaction frame. During the 1.25” actuator displacement cycles, diagonal cracks formed at the edges of the grout beam (Figure 5-29). These cracks
affected the LVDT readings because the contact point for the sensor was on the end of the CMU wall beyond the crack. The hysteretic plot is shown in Figure 5-31. The last three cycles to 1.50" actuator displacement show a lower stiffness and strength because of the cracks that formed at the edges of the CMU wall. The ultimate failure of the wall specimen occurred during the initial cycle to 1.5" displacement as shown in Figure 5-30. The wall specimen had a maximum capacity of 99.2 kips at a LVDT displacement of 0.395". During the 1.5" cycles, three of the four vertical reinforcing steel bars failed in a combination of tension and shear. The locations of the failures and the reinforcing steel are shown in Figure 5-32 and Figure 5-33 respectively.

Headed studs at 16" on center, with none in the end cells, was successful at transferring shear from the top beam to the CMU wall. All 5 studs were effective at transferring a shear of at least 20 kips per stud. No relative displacement was observed between the top steel beam and the grout beam, indicating that the maximum capacity of the headed stud connection had not been reached prior to failure of the CMU wall.

![Image: Headed stud layout for FGW-HS16-1.](image-url)
Figure 5-29: Initial diagonal cracking formed on FGW-HS16-1.

Figure 5-30: Crack distribution after failure of FGW-HS16-1.
Figure 5-31: Hysteretic plot of load versus displacement for FGW-HS16-1.

Figure 5-32: Location of vertical reinforcing steel failure.
5.2.1.3 FGW-HS8-1

In FGW-HS8-1, ten 6" long x ¾" diameter headed studs were welded to the bottom flange of the top wide-flanged steel beam spaced at eight inches on center with the first stud starting in the end cell of the CMU wall. During this experiment the testing frame failed before the wall specimen could reach its maximum capacity. The maximum load applied to the wall specimen was 95.2 kips and the corresponding horizontal deformation of the wall specimen was 0.071". The final crack pattern for FGW-HS8-1 is shown in Figure 5-34. The displacement in the plot shown in Figure 5-35 is the average of the horizontal component of the four diagonal string potentiometers. The maximum load occurred during the initial 1.25" actuator displacement cycle. The displacement recordings from the MTS hydraulic actuator were different from those recorded by the string potentiometers because of deformation in the reaction frame. The plot in Figure 5-35 is not a smooth line because the measurements were recorded by diagonal string potentiometers mounted to the CMU wall from top corner to opposite bottom corner of the CMU wall on the same face. The string going across the wall specimen was disturbed by falling debris leading to the fluctuations seen in the hysteretic plot.

Headed studs at 8" on center, was successful at transferring shear from the top beam to the CMU wall. Because of edge cell breakout, only nine studs were effective at transferring shear from the steel beam to grout beam in each loading direction. Therefore, the shear in each stud at maximum capacity was 95.2 kips divided into nine studs, equating to approximately 11 kips per stud.

This experiment was performed in three stages because of failure of components of the testing frame. At the 1.5" actuator displacement, the clevis pin connection between the top beam and the MTS hydraulic actuator failed. The experiment was stopped at this point because the CMU wall had experienced significant cracks and started to lift off the concrete slab at both ends.
Figure 5-34: Final crack pattern in FGW-HS8-1.

Figure 5-35: Hysteretic plot of load versus deflection of FGW-HS8-1.
6 Analysis

6.1 Comparison of Expected Results vs. Experimental Results

6.1.1 Masonry Wall Bolt Push-Out Specimens

According to the Masonry Standards Joint Committee (MSJC) in the Building Code Requirements for Masonry Structures (TMS 402-08 / ACI 530-08 / ASCE 5-08) there are four failure mechanisms for an anchor bolt embedded in a CMU wall. Masonry Breakout strength is given by:

\[ B_{vnb} = 4 * A_{pv} * \sqrt{f_m} \]  \hspace{1cm} \text{6-1}

where; \( A_{pv} \) is the projected shear area on masonry surface, in. \(^2\). Masonry Crushing strength is given by:

\[ B_{vnc} = 1050 * 4\sqrt{f_m} * A_b \]  \hspace{1cm} \text{6-2}

where; \( A_b \) is the cross-sectional area of an anchor bolt, in. \(^2\). Anchor Bolt Pryout strength is given by:

\[ B_{vpry} = 8 * A_{pt} \sqrt{f_m} \]  \hspace{1cm} \text{6-3}

where; \( A_{pt} \) is the projected tension area on masonry surface, in. \(^2\). Steel Yielding strength is given by:

\[ B_{vns} = 0.6 * A_b * f_y \]  \hspace{1cm} \text{6-4}

In this study through bolts were used so anchor bolt pryout was not considered as a possible failure mechanism. Failure loads for each through bolt test based on these expressions are given in Table 6-1. These mechanisms were compared with the experimental results to determine which represents the initial failure of the CMU wall specimens.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>( B_{vnb} ) (lbs)</th>
<th>( B_{vnc} ) (lbs)</th>
<th>( B_{vns} ) (lbs)</th>
<th>NOTES</th>
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</thead>
<tbody>
<tr>
<td>FGW-1#4BB-1</td>
<td>47735</td>
<td>14286</td>
<td>47124</td>
<td>Control specimen</td>
</tr>
<tr>
<td>FGW-1#4BB-2</td>
<td>65755</td>
<td>14286</td>
<td>47124</td>
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<tr>
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<td>77017</td>
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<tr>
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<td>37900</td>
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<td>47124</td>
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<tr>
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<td>47124</td>
<td>Top course hollow</td>
</tr>
<tr>
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<td>14286</td>
<td>47124</td>
<td>Through bolt in top course</td>
</tr>
<tr>
<td>FGW-1#4TBB-2</td>
<td>53610</td>
<td>14286</td>
<td>47124</td>
<td>Through bolt in top course</td>
</tr>
<tr>
<td>FGW-2#4BB-1</td>
<td>47735</td>
<td>14286</td>
<td>47124</td>
<td>2#4 reinforcing bars in BB</td>
</tr>
<tr>
<td>FGW-2#4BB-2</td>
<td>65755</td>
<td>14286</td>
<td>47124</td>
<td>2#4 reinforcing bars in BB</td>
</tr>
<tr>
<td>FGW-3#4BB-1</td>
<td>47735</td>
<td>14286</td>
<td>47124</td>
<td>Added bond beam above and below</td>
</tr>
<tr>
<td>FGW-3#4BB-2</td>
<td>65755</td>
<td>14286</td>
<td>47124</td>
<td>Added bond beam above and below</td>
</tr>
</tbody>
</table>
All failures observed in the test specimens represent Masonry Breakout failure mechanism. The Masonry Breakout mechanism is dependent on the shear/tension area of the CMU wall affected by the bolt. Based on the observed failure cracks, the typical failure path expected to occur for through bolts placed in the second course from the top and 12” from the edge of the CMU wall is shown in Figure 6-1 as a dashed line. The arrow represents the direction of the load applied to the CMU wall by the through bolt. Based on the observed failure cracks, the typical failure path for through bolts placed in the second course from the top and 20” from the edge is shown in Figure 6-2. The dashed line represents the expected failure path and the arrow shows the direction of loading.

Figure 6-1: Expected failure lines of typical through bolt 12” from edge of wall.
The nominal strengths of the masonry breakout shown in Table 6-1 were determined based on projected shear area on the masonry surface. All the theoretical values were greater than the experimental results. This indicates that the failure stress $f_m$ used in the masonry break out equation ($B_{nbl}=4*A_{pv}*f_m$; 6-1) overestimates the combined tension/shear failure stress (Figure 6-3).

Figure 6-2: Expected failure lines of typical through bolt 20” from edge of wall.

Figure 6-3: Plot of the proposed theoretical and experimental data against the plot of the MSJC theoretical and experimental data.
The difference between the theoretical and experimental results shows that a reduced stress should be used in calculating the breakout capacity of the through bolt. The experimental results for the cracking strength were divided by the projected shear area to obtain the average stress on the failure plane in the CMU walls Table 6-2. Based on these stresses, the coefficient that should be used in finding the tensile stress capacity is $1.58$ for through bolts in CMU walls as shown in Table 6-3. The proposed theoretical values computed using,

$$B_{vb} = 1.58 * A_{pv} * \sqrt{f_{m'}};$$

are compared with the experimental values in Figure 6-4.

The MSJC and proposed failure capacities are plotted against the individual push-out tests in Figure 6-5 through Figure 6-15 to show the difference between the two results.

Table 6-2: Data for calculation of the theoretical constant used to determine the tensile stress capacity.

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<tr>
<th>SPECIMEN</th>
<th>$A_{pv}$ (in²)</th>
<th>$f_{m'}$</th>
<th>$P_{cr}$ (lbs)</th>
<th>$f_{m}$ (psi)</th>
<th>$\sqrt{f_{m'}}$</th>
<th>$P_u$ (kips)</th>
<th>$P_u/P_{cr}$</th>
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*BOLD - Approximate value
Table 6-3: Data for the theoretical against the experimental values using a tensile coefficient of 1.58.

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Figure 6-4: Plot of the proposed theoretical and experimental data against a 45 degree line to show the accuracy of the predicted CMU crack capacities.
Figure 6-5: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4BB-1.

Figure 6-6: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4BB-2.
Figure 6-7: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4BB-G-1.

Figure 6-8: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4BB-TCH-1.
Figure 6-9: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4BB-TCH-2.

Figure 6-10: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4TBB-1.
Figure 6-11: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-1#4TBB-2.

Figure 6-12: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-2#4BB-1.
Figure 6-13: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-2#4BB-2.

Figure 6-14: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-3#4BB-1.
Figure 6-15: Predicted MSJC theoretical and proposed capacity and experimental results for FGW-3#4BB-2.

6.2 Specimen Performance Comparison

6.2.1.1 FGW-1#4BB-2 – Control Specimen – 20” Edge Distance

Figure 6-15 shows the plots of load against displacement for FGW-1#4BB-1 and 2. The plot was trimmed at 0.5” displacement and the initial portion of both plots had to be corrected because of initial settling of the testing frame. The increase in edge distance for test 2 results in an increase in push-out capacity as suspected.

Figure 6-16: Comparison of 12” and 20” edge distance in FGW-1#4BB.
6.2.1.2 FGW-1#4BB-G-1 – Group Push-out

Figure 6-17 shows the load versus displacement plots for FGW-1#4BB and FGW-1#4BB-G. The initial cracking of the group specimen was similar to that for the through bolt at 20" edge distance, and the ultimate strength was virtually the same as FGW-1#4BB-2. This shows that the group of through bolts did not increase the capacity, although it does allow the specimen to hold the ultimate capacity through a larger displacement.

![Figure 6-17: Comparison of group and control specimens.](image)

6.2.1.3 FGW-1#4BB-TCH-1

In FGW-1#4BB-TCH-1, the initial stiffness of the specimen was lower than FGW-1#4BB-1(Figure 6-18). Without grout in the top course there is less confinement for the through bolt. Grouting the top course almost doubled the capacity of the through bolt. The displacement of the through bolt relative to the CMU wall was also sacrificed if the top course is un-grouted.
6.2.1.4 FGW-1#4BB-TCH-2

In FGW-1-#4BB-TCH-2, the initial behavior of the specimen was similar to FGW-1#4BB-2 (Figure 6-19). The cracking capacity was 20 kips at a displacement of roughly 0.075" compared with 37 kips at 0.18" displacement. A beneficial characteristic of this specimen is that it gains strength as the displacement increases until roughly 0.8 inches, then the wall failed when the bond between the CMU wall and reinforcing failed. The ultimate capacity of the specimen was roughly 20% lower than FGW-1#4BB-2.
In a comparison between the two FGW-1#4BB-TCH specimens, it was concluded that a minimum of three cell edge distance is required to ensure reliable behavior is obtained from the system (Figure 6-20).

![Graph showing load versus displacement](image)

**Figure 6-20: Comparison of 12” and 20” edge distance in FGW-1#4BB-TCH.**

6.2.1.5 FGW-1#4TBB-1

Figure 6-21 and Figure 6-23 compare load versus displacement plots for specimens with the through bolt and bond beam in the top course and the second to the top course with the top course un-grouted. The result shows that having an un-grouted top course only offers a slight increase of cracking and maximum capacity.

In Figure 6-23 and Figure 6-24 the results from FGW-1#4TBB-1 and 2 are compared against the control specimen, FGW-1#4BB-1 and 2, respectively. The through bolts lost approximately 50% of their capacity when the bond beam was the top course instead of the second course from the top. This was expected because the through bolt is less confined and the failure plane decreases when the through bolt is placed in the top course of the CMU wall.
Figure 6-21: Comparison of specimen with through bolt in top course and through bolt in second to the top with the top course un-grouted placed 12” from the edge of the CMU wall.

Figure 6-22: Comparison of specimen with through bolt in top course and through bolt in second to the top with the top course un-grouted placed 20” from the edge of the CMU wall.
Figure 6-23: Comparison of specimen with through bolt in top course and control.

Figure 6-24: Comparison of specimen with through bolt in top course and control placed 20” from the edge of the CMU wall.

In Figure 6-25 the 12” and 20” edge distance specimens for FGW-1#4TBB are shown. The added edge distance increases the capacity of the through bolt by roughly 50% while the specimen also gains ductility.
6.2.1.6 FGW-2#4BB-1

FGW-2#4BB-1 is plotted against the control specimen, FGW-1#4BB-1, in Figure 6-26. The increase in maximum capacity was approximately 7% from the control specimen. Although the increase in strength is not significant, the added #4 bar in the bond beam provided more reliable performance after initial cracking.

Doubling the reinforcing in the bond beam did not significantly increase the ultimate strength for the through bolt connection.

Figure 6-25: Comparison of 12” and 20” edge distance in FGW-1#4TBB.

Figure 6-26: Comparison of specimen with 2-#4 bars in the bond beam and control placed 12” from the edge of the CMU wall.
6.2.1.7 FGW-2#4BB-2

In FGW-2#4BB-2, the maximum capacity of the through bolt was similar to FGW-1#4BB-2, although the initial cracking occurred at a lower load (Figure 6-27). From Figure 6-26 and Figure 6-27, it can be concluded that adding a #4 bar in the bond beam does not significantly increase the capacity of the through bolt connection.

In Figure 6-28 the results of the 12” and 20” edge distance for FGW-2#4BB are shown together. The two experiments had approximately the same initial stiffness, although the 20” edge distance had an increase in both cracking and maximum capacity.

![Figure 6-27: Comparison of specimen with 2-#4 bars in the bond beam and control placed 20” from the edge of the CMU wall.](image1)

![Figure 6-28: Comparison of 12” and 20” edge distance in FGW-2#4BB.](image2)
6.2.1.8 FGW-3#4BB-1

FGW-3#4BB-1 is compared to the control specimen FGW-1#4BB-1 in Figure 6-29. The maximum capacity did not increase with the added #4 bars in the course above and below the through bolt. The displacement applied to the specimen before failure was only approximately 0.375” for FGW-3#4BB-1, while FGW-1#4BB-1 went to approximately 2.4” displacement before failure.

![Graph](image)

Figure 6-29: Comparison of specimen with bond beams located above and below through bolt course and control placed 12” from the edge of the CMU wall.

6.2.1.9 FGW-3#4BB-2

Figure 6-30 shows the comparison between the 12” and 20” edge distance specimens for FGW-3#4BB. The maximum capacity increased approximately 87% from the 12” to the 20” edge distance. Both specimens behaved similarly during initial loading. The reason for the vertical drop in FGW-3#4BB-2 is the side plate weld failing and the restart of the test after proper repairs were made.

In Figure 6-31 FGW-3#4BB-2 was compared to the control specimen, FGW-1#4BB-2. The added reinforcing increased the maximum capacity by approximately 28%. The failure of the CMU wall specimen showed that the added reinforcing in FGW-3#4BB-2 mobilized a greater portion of the CMU infill wall. This is apparent from the failure cracks throughout the whole wall specimen shown in Figure 5-22.
Figure 6-30: Comparison of 12” and 20” edge distance in FGW-3#4BB.

Figure 6-31: Comparison of specimen with bond beams located above and below through bolt course and control placed 20” from the edge of the CMU wall.

6.2.2 Type II and III Hybrid Masonry Specimens

According to the American Institute of Steel Construction Manual (AISC) in the Specification for Structural Steel Buildings the failure mechanisms for headed studs casted into concrete are:

Nominal strength of stud shear connector is given by:

$$ V' = \sum Q_n $$
where; \( Q_n = 0.5 \cdot A_{sc} \sqrt{f_c' \cdot E_c} \leq R_g R_p A_{sc} F_u \),

\( A_{sc} \) is the projected shear area on masonry surface, in.\(^2\),
\( f_c' \) is the specified compressive strength of concrete, ksi,
\( E_c \) is the modulus of elasticity of concrete in compression, ksi,
\( R_g \) is 1.0 for any number of studs welded in a row directly to the steel shape,
\( R_p \) is 1.0 for studs welded directly to the steel shape and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures,
\( F_u \) is the specified minimum tensile strength of a stud shear connector, ksi.

Concrete Crushing strength is given by:

\[ V' = 0.85f_c' A_c \]  

where; \( A_c \) is the area of concrete slab within effective width, in.\(^2\).

Tensile Yielding of the Steel Connector’s strength is given by:

\[ V' = F_y A_s \]  

where; \( A_s \) is the area of steel cross-section, in.\(^2\),
\( F_y \) is the specified yield strength of steel, ksi.

In this study through bolts were used so anchor bolt pry-out was not considered as a failure mechanism. These mechanisms were compared with the experimental results to determine which represents the initial failure of the CMU wall specimens.

**Table 6-4: Calculated and experimental results for Type II Hybrid Masonry tests.**

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<th>( Q_{nf}' ) (kips/stud)</th>
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6.2.2.1 Hysteretic Plots

Figure 6-32 shows the hysteretic plots of each Type II Hybrid Masonry wall specimen tested.
Figure 6-32: Comparison of three Type II Hybrid Masonry specimens.
7 Summary / Conclusions / Recommendations

A study was performed to evaluate the strength of through bolts used in Type I Hybrid Masonry, and the shear capacity of headed stud connections used in Type II and III Hybrid Masonry.

7.1 Type I Hybrid Masonry Through Bolt Connections
Type I Hybrid Masonry utilizes either ductile fuse plates or elastic link plates to transfer shear between floor beams and the CMU shear panels making up the Hybrid Masonry system. 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 through bolts must not be the weak link in the Hybrid Masonry system since failure of the bolts in shear or masonry break-out could represent a non-ductile response that is not suitable for seismic design.

Eleven bolt push-out tests were performed on six 8" thick grouted CMU walls to evaluate the performance of the through bolts. Based on these tests, the following conclusions were drawn:

- 1 inch diameter high-strength (A490 or Grade 8) through bolts were adequate to avoid bolt failure in all tests performed in this study.
- Failure of all push-out tests resulted from combined shear and tension failure of the CMU wall causing a break-out failure.
- Because the bolt is a through bolt and not a single sided bolt as assumed by the MSJC design manual, bolt pry-out and bearing failures do not control the failure mechanism.
- The through bolt should be located in a bond beam in the second course from the top of the CMU wall. This will ensure a greater capacity and a ductile failure.
  - Having a course above the through bolt reduces the rotation of the CMU block holding the through bolt. When the through bolt is located in the top course, the rotation of the blocks reduced the capacity of the connection.
  - Additional testing is recommended for through bolts placed in the second course from the top with the top course un-grouted.
- During construction of the through bolt connection, the bolts should be located below the horizontal bar in the bond beam. The bolt hole should be the same diameter as the through bolt. It should be created by means of a small pilot hole which is increased incrementally by drilling from both sides of the wall up to the diameter of the bolt. This will reduce the potential for face break-out of the CMU blocks during drilling.
- Through bolts should have a minimum edge distance of 12” from the end of the CMU wall. This implies placing the end bolt through the center of the second cell from the end of the wall. No bolts should be placed in the end cell.
- Through bolts with an edge distance of 20” from the end of the CMU wall, ie. located in the third cell from the end of the wall, had significantly greater strength and ductility than those with an edge distance of 12”.
• The reinforcing steel used in the bond beam does not offer an increase in initial cracking capacity, although it increases the ductility of the connection after cracking.
• When a bond beam was added both above and below the through bolt course the maximum capacity and ductility of the wall increased.
• A CMU wall specimen with two through bolts placed in the same bond beam at 16” on center demonstrated that the group strength is less than the sum of the individual bolt capacities. Failure of this specimen occurred when the horizontal bar in the bond beam failed in tension, indicating that greater ultimate capacity may have been achieved with more reinforcement in the bond beam. It is proposed that bolt spacing be increased to avoid a group effect, and that additional testing be performed to evaluate this effect in more detail.

7.2 Type II and III Hybrid Masonry Headed Stud Connection
Type II Hybrid Masonry utilizes headed studs to transfer shear from the steel floor beam to the CMU shear panel below. Type III Hybrid Masonry utilizes headed studs to transfer shear from both the steel floor beam and the columns to the CMU shear panel. Three fully grouted 8” thick CMU walls were tested to evaluate the performance of headed studs at different spacings in these connections. In all cases, the headed studs were ¾” diameter by 6” long, welded to the bottom of the floor beam bottom flange. They were embedded in a 9” high by 7-5/8” thick grout beam on top of the 8” CMU wall. The top course of the CMU wall was a bond beam with one #4 bar and the wall vertical reinforcement consisted of #4 bars at 24” on center. Based on these tests the following conclusions were drawn:
• Headed studs at 8” and 16” on center were able to transfer sufficient shear from the top steel beam to the CMU wall to ensure failure of the CMU wall prior to significant damage to the headed stud connection. For the 16” spacing, no studs were located in the end cells of the CMU wall.
• Headed studs at 24” on center failed due to end stud break-out and splitting of the grout beam along the line of headed studs before significant yielding or shear failure of the CMU wall.
• The shear capacity of headed studs provided by the AISC Steel Design Manual overestimated the capacity observed in these tests.
• Headed stud should be placed with a minimum of 12” from the end of the CMU to reduce the potential for early break-out failure of the grout beam. The studs shall be placed in the center of the grout beam to provide maximum edge distance to each side of the grout beam.
• L-shaped reinforcing bars should be placed at each vertical reinforcing bar in the CMU wall and at each headed stud location to improve the connection between the grout beam and the CMU wall. The L-shaped reinforcing bars should be the same size as the vertical reinforcing bars. A horizontal tie bar shall be placed in the grout beam alongside the headed studs and horizontal leg of the L-shaped bars.
• In Type II and III headed stud connections should follow a minimum overstrength factor of 1.5. This overstrength factor will ensure the failure occurs in the CMU wall rather than the grout beam connecting the steel beam to the CMU wall.
8 References


ACI (2008): ACI 2008: American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318-08). American Concrete Institute, MI.

APPENDICES
Fastenal Product Standard: FNL.DTLA490.SQRT.P

Direct Tension Indicator, Type A490, Squirt®, Plain Finish
The information below lists the required dimensional, chemical and physical characteristics of the fasteners in this purchase order. If the order received does not meet these requirements, it may result in a supplier corrective action request, which could jeopardize your status as an approved vendor. Unless otherwise specified, all referenced consensus standards must be adhered to in their entirety.

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Specification Requirements:
- Dimensions: ASTM F959, Type 490.
- Material and Mechanical Properties: ASTM F959, Type 490.
  The orange epoxy located in the filled recesses, the amount of epoxy released is approximate, and is comparable to the amount released when a skidmore measures the proper clamp load under controlled conditions when tested per ASTM F959.
  For more information, visit engineer@fastenal.com
- Product Marking: Manufacturing ID, Lot number and “490”.
- Finish: Light protective oil.
- Material Test Reports: The MTR must have documented lot traceability, including full chemical and mechanical figures, to the specification(s) above.
# B1: Compression Test Results for Grout

## Grout Cylinder Specimens

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<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>1</td>
<td>17</td>
<td>47000</td>
<td>3740</td>
<td></td>
</tr>
<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>2</td>
<td>17</td>
<td>53750</td>
<td>4277</td>
<td></td>
</tr>
<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>3</td>
<td>17</td>
<td>50750</td>
<td>4039</td>
<td></td>
</tr>
<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>1</td>
<td>25</td>
<td>78500</td>
<td>6247</td>
<td></td>
</tr>
<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>2</td>
<td>25</td>
<td>66000</td>
<td>5252</td>
<td></td>
</tr>
<tr>
<td>Grout 4&quot; diam. cyl.</td>
<td>3</td>
<td>25</td>
<td>68500</td>
<td>5451</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>1</td>
<td>18</td>
<td>140500</td>
<td>4969</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>2</td>
<td>18</td>
<td>136500</td>
<td>4828</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>3</td>
<td>18</td>
<td>130000</td>
<td>4598</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>1</td>
<td>17</td>
<td>126500</td>
<td>4474</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>2</td>
<td>17</td>
<td>128500</td>
<td>4545</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>3</td>
<td>17</td>
<td>145500</td>
<td>5146</td>
<td></td>
</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
<td>1</td>
<td>25</td>
<td>177500</td>
<td>6278</td>
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</tr>
<tr>
<td>Grout 6&quot; diam. cyl.</td>
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<td>25</td>
<td>184000</td>
<td>6508</td>
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</tr>
<tr>
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<td>25</td>
<td>181500</td>
<td>6419</td>
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</tr>
</tbody>
</table>

**Average (psi): 5118**
# B2: Compression Test Results for Mortar and CMU Prisms

## 2” Cube Mortar Specimens

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen</th>
<th>Specimen #</th>
<th>Age (days)</th>
<th>Compression load (lbs)</th>
<th>Compressive stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>1</td>
<td>17</td>
<td>11200</td>
<td>2800</td>
</tr>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>2</td>
<td>17</td>
<td>12425</td>
<td>3106</td>
</tr>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>3</td>
<td>17</td>
<td>11200</td>
<td>2800</td>
</tr>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>1</td>
<td>25</td>
<td>21550</td>
<td>5388</td>
</tr>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>2</td>
<td>25</td>
<td>19200</td>
<td>4800</td>
</tr>
<tr>
<td>Mortar</td>
<td>2” cube</td>
<td>3</td>
<td>25</td>
<td>18850</td>
<td>4713</td>
</tr>
</tbody>
</table>

*Average (psi): 3934*

## 8” CMU Prisms

<table>
<thead>
<tr>
<th>Material</th>
<th>Specimen</th>
<th>Specimen #</th>
<th>Age (days)</th>
<th>Compression load (lbs)</th>
<th>Compressive stress (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>1</td>
<td>18</td>
<td>125000</td>
<td>2349</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>2</td>
<td>21</td>
<td>179000</td>
<td>3079</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>1</td>
<td>17</td>
<td>128500</td>
<td>2210</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>2</td>
<td>17</td>
<td>173500</td>
<td>2984</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>3</td>
<td>17</td>
<td>175000</td>
<td>3010</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>1</td>
<td>25</td>
<td>151000</td>
<td>2597</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>2</td>
<td>25</td>
<td>167500</td>
<td>2881</td>
</tr>
<tr>
<td>CMU prism</td>
<td>8” blocks</td>
<td>3</td>
<td>25</td>
<td>157500</td>
<td>2709</td>
</tr>
</tbody>
</table>

*Average (psi): 2727*
# C1: Tension Test Stress-Strain Curves

## #4 Reinforcing Bar Specimens

Bar Diameter (in): 0.5  
Cross-Sectional Area (in.$^2$): 0.2
#4 Reinforcing Bar Specimens

Bar Diameter (in): 0.5
Cross-Sectional Area (in.²): 0.2
C3: Tension Test Stress-Strain Curves

#5 Reinforcing Bar Specimens

**#5 Bar Test-1**

---

**#5 Bar Test-2**

---

Bar Diameter (in): 0.625
Cross-Sectional Area (in.²): 0.31
#5 Reinforcing Bar Specimens

**#5 Bar Test-3**

**#5 Bar Test-4**

Bar Diameter (in): 0.625
Cross-Sectional Area (in.²): 0.31