VERIFICATION OF FUSE CONNECTOR PERFORMANCE
FOR HYBRID MASONRY SEISMIC STRUCTURAL SYSTEMS

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Steven Mitsuyuki

Thesis Committee:

Ian Robertson, Chairperson
David Ma
H. Ronald Riggs
ABSTRACT

In steel construction, there are various seismic structural systems that have been successfully utilized in the past including eccentric and concentric braced frames and moment frame systems. When these systems are used, the building envelope between structural members is filled with architectural components such as masonry infill walls, which are only expected to carry out of plane loads, such as wind, on each individual wall panel. Hybrid masonry is a relatively new type of structural system which incorporates the masonry infill walls within steel frames into the structural system to resist lateral loads imposed on the structure. This research represents the third phase of testing at the University of Hawaii at Manoa (UHM) and is part of an ongoing project funded by the National Science Foundation (NSF) and Network for Earthquake Engineering Simulation Research (NEESR) to investigate the applicability of hybrid masonry as an adequate structural system.

In phases I and II of hybrid masonry connector plate development research at UHM, a wide variety of connector designs were explored to determine the positive and negative aspects to different fuse and link connector plate designs and connection methods. Fuse connector tests demonstrated the viability of tapered fuses to dissipate large amounts of energy during cyclic loading. These tests also showed that various practical connector orientations and methods can be applied for hybrid masonry connectors, particularly the straight bolted side plate connection. In addition, during phase I of testing, a series of bolt push out tests were performed to determine the shear strength of post-installed through-bolts in masonry.

Observations from the first two phases of testing were used as a guide for selecting 4-inch and 6-inch tapered fuse designs for testing on partially grouted and fully grouted masonry wall specimens. Multiple pairs of both tapered fuse connectors were slip-critically bolted to side plates, which were welded to a steel beam above the masonry wall specimens. These tests were performed to verify that the behavior of multiple pairs of connector plates in series is similar to the behavior of the individual pairs of connector plates. In addition, the tests were performed to observe the interaction between the steel connector plates and masonry wall specimens.

Based on numerous observations and results in this phase of testing, the design of Type I hybrid masonry connections was improved. The fuse connector tests showed that the strength and behavior of multiple pairs of fuse connectors can be approximated by designing a single connector plate and multiplying the single plate’s capacity by the number of connector plates used. These fuse connectors displayed the capability of dissipating large amounts of seismic energy before failure. Based on the results of these connector subassembly tests, recommended design procedures for connector plates were modified. Both masonry walls failed by means of shear friction at the location of the joint between the masonry and concrete slab. The results from UHM’s testing will guide the development of full-scale two-story hybrid masonry test frames which will be tested at the University of Illinois Urbana-Champaign.
ACKNOWLEDGEMENTS

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1 Introduction

1.1 Introduction

Engineers are tasked with the responsibility of ensuring that the system they are designing performs successfully while remaining an economically feasible design. Specifically in building construction, the structural system chosen by the structural engineer to resist the expected loads to which the structure will be exposed has a large influence on the economic efficiency of the entire project. In steel construction, there are various structural systems that have been successfully utilized in the past including eccentric and concentric braced frames and moment frame systems. When these types of systems are applied, voids in the building envelope between structural members are enclosed with architectural components such as masonry infill walls, which are only expected to carry out of plane loads such as wind on each individual wall panel.

Hybrid masonry is a relatively new type of structural system which incorporates the masonry infill walls within steel frames into the structural system to resist lateral loads imposed on the structure (e.g. wind or seismic loads). These masonry infill walls would be designed and constructed as a reinforced masonry shear wall transferring lateral loads through the structure and into the structure's foundation. Hybrid masonry is already being used as a structural system in lower seismic zones. There are three different types of hybrid masonry systems referred to as Type I, Type II, and Type III as depicted in Figure 2-1.

Type I hybrid masonry refers to a steel frame and masonry infill wall in which a gap is maintained between the masonry infill wall and the two steel frame columns, as well as the steel beam above the wall. Steel connector plates are used to transfer only lateral loads into the masonry infill walls from the beam above. Type II hybrid masonry refers to a system similar to Type I hybrid masonry except that the gap between the top of the wall and beam above the wall is eliminated. The lack of gap above the wall results in the masonry wall resisting both vertical and lateral loads. The lateral load is transferred into the wall through headed studs that are welded to the bottom flange of the beam above and cast into the top portion of the wall. Type III hybrid masonry refers to a system similar to Type II hybrid masonry except that there are no gaps between the wall and the steel frame. A headed stud connection similar to the Type II hybrid masonry system is used to resist vertical shear loads between the steel columns and sides of the masonry infill walls.

This research represents the third phase of testing at the University of Hawaii at Manoa (UHM) and is part of an ongoing project funded by the National Science Foundation (NSF) and Network for Earthquake Engineering Simulation Research (NEESR) to investigate the applicability of hybrid masonry as an adequate structural system in higher seismic zones (e.g. structures with Seismic Design Categories [SDC] D and E). UHM was tasked with developing connectors between the steel frame and masonry infill wall for the different types of hybrid masonry. The research included in this portion of the project is an extension to the previous research performed by Seth Goodnight and Reef Ozaki-Train under the direction of Dr. Ian Robertson. The intent of the research for this report is to test energy dissipating tapered fuses installed on full-scale masonry wall specimens to investigate the interaction between the steel
connectors and the masonry, as well as further development of methods for Type I hybrid masonry design. The results found during this research will also be beneficial for the preparation of large-scale tests to be conducted by the University of Illinois at Urbana-Champaign (UIUC).

1.2 Objective

The scope of the research included in this report is limited to the investigation of Type I hybrid masonry connector plates. The objective of this report is to further investigate fuse connector plates designed in previous phases of the project by through-bolting them onto masonry wall specimens and determining their ability to transfer lateral loads successfully into the wall specimens.
2 Literature Review

2.1 Hybrid Masonry Lateral Load Resisting System

Using hybrid masonry as a structural system, which was first proposed in 2006 [Biggs, 2006], incorporates non-structural masonry infill walls into the structural load resisting system. Hybrid masonry can be subdivided into three different types based on how load is transferred through the structure, as shown in Figure 2-1 [Biggs, 2011].

A Type I hybrid masonry wall is designed with a gap between the two sides and top of the masonry infill wall and the steel frame which surrounds it, preventing the wall from experiencing vertical gravity loads. This results in the masonry wall only resisting lateral loads by means of steel connector plates between the wall and steel beam above the wall. These connector plates are designed with a vertically slotted hole for the connection to the masonry wall to prevent any vertical load transfer from the beam above. The benefit to using this type of system is that the connector plates could potentially be designed to accommodate all inelastic deformations so that the wall does not become damaged during an earthquake. Therefore, after a seismic event, the repair costs to replace the connector plates would be much lower relative to replacing the masonry infill walls.

Figure 2-1: Hybrid Masonry Types I, II, and III, shown clockwise from top left
A Type II hybrid masonry wall is designed with a gap between the two sides of the masonry infill wall and steel frame, but without a gap at the top of the wall. This results in both lateral and vertical force transfer through the masonry wall. A Type III hybrid masonry wall is designed without any gaps between the wall and the steel frame, also resulting in lateral and vertical forces to be resisted by the masonry walls. The forces caused by lateral loads, which need to be resisted by these walls would be transferred by means of headed studs welded to the steel frame, either on the bottom flange of the beam above the wall for Type II or on the bottom flange of the beam above the wall and the two flanges of the columns at each side facing the wall for Type III, and cast into grout or concrete at the boundary elements of the wall. The benefit of these two systems is the redundancy in the structural system to resist vertical and lateral loads imposed by a seismic event, possibly resulting in a prevention of progressive collapse throughout the structure.

The *International Building Code* [IBC, 2006] and *The Building Code Requirements and Specifications for Masonry Structures* (ACI 530-08) [MSJC 2008] currently list three different classifications of masonry shear walls for seismic design: ordinary, intermediate, and special reinforced masonry shear walls. As shown in Table 2-1, each classification is assigned a response modification factor (R), system over-strength factor ($\Omega_0$), deflection amplification factor ($C_d$), and height limitations based on the structure’s SDC. Hybrid masonry as a structural system has already been implemented in lower seismic zones (SDC A, B, and C) [IMI, 2009]. These structures are designed using the seismic forces and design values for reinforced masonry shear walls from the IBC for reinforced masonry shear walls. Use of hybrid masonry as a viable load resisting system option in higher SDCs is currently under investigation.

**Table 2-1: Seismic Design Values for Reinforced Masonry Shear Wall Building Frame Systems [ASCE, 2005]**

<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>NL 160 NP NP NP</td>
</tr>
<tr>
<td>Intermediate reinforced masonry shear walls</td>
<td>4</td>
<td>2.5</td>
<td>4</td>
<td>NL NL NP NP NP</td>
</tr>
<tr>
<td>Special reinforced masonry shear walls</td>
<td>5.5</td>
<td>2.5</td>
<td>4</td>
<td>NL NL 160 160 100</td>
</tr>
</tbody>
</table>

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

Prior to Seth Goodnight’s research performed at UHM [Goodnight et al, 2011], the design of the connector plates for hybrid masonry systems has focused solely on design for in-plane load transfer to the wall and to brace the frame for out-of-plane loads [Biggs, 2011]. Using this design approach, little to no additional ductility is provided to the structural system because the masonry wall is assumed to crack under large seismic loads [Goodnight et al, 2011]. The current detailing requirement of the connector plates for hybrid masonry from the International Masonry Institute (IMI) is shown in Figures 2-2 and 2-3. With this type of detailing, the connector plate can be idealized as a cantilever and computer analysis can be used to determine the total shear to each connector plate and multiplied by the distance from the bottom of the beam flange to the center of the connector plate’s slotted hole to determine the connector plate’s required flexural strength. Alternatively, a second option would be to detail the connector plate as a structural fuse to dissipate energy within the structural system caused by seismic loads and minimize damage to the masonry wall [Biggs, 2011], as discussed previously.

2.3 Structural Steel Fuses

Numerous investigations into structural fuses as part of structural systems have been performed in order to determine their behavior and how they can benefit systems exposed to high seismic loads. These fuses have two main purposes within the structural system. First, structural fuses are used to dissipate energy absorbed by the structure during an earthquake and ultimately increase a structure’s ductility. Second,
structural fuses are used to isolate damage and protect other building components from getting damaged. By isolating the damage caused by an earthquake, repair of the structure is much quicker and less costly.

One example of an investigation into structural fuse construction is the development of ductile fuses in HSS seismic tension bracing systems for low-rise buildings [Prion and Timler, 2010], as shown in Figure 2-4. Another example of sacrificial structural fuses providing “satisfactory seismic performance while facilitating post-earthquake repair” was demonstrated during testing of a composite multi-column pier [Bruneau et al, 2010], as shown in Figures 2-5 & 2-6. Additionally, previous testing on sacrificial seismic fuse devices for masonry infill walls confirmed their ability to successfully resist lateral loads while dissipating seismic energy and preventing damage to the masonry infill walls [Aliaari & Memari, 2007].

Another example of an investigation into structural fuse construction is the research performed on rocking steel frames with replaceable steel butterfly fuse elements [Deierlein et al, 2011]. In these experiments, a controlled rocking system was constructed with a stiff steel braced frame which was not secured to the foundation, allowing the frame to rock. In between two of these frames, replaceable structural fuses, depicted in Figure 2-7, were placed to absorb seismic energy as the frames rocked back and forth during testing, as shown in Figure 2-8. The test results showed that “the controlled rocking system exhibited excellent self-centering properties, and effectively concentrated the energy dissipation and structural damage in the replaceable fuse elements [Eatherton et al, 2008].
2.4 UHM Research

2.4.1 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 are not present 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 & 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 2-9. Most of the specimens were loaded twice, first in one direction until failure, then rotated 180-degrees and loaded again.
A majority of the masonry specimens experienced shear breakout failure at or above 20 kips (89 kN). In addition, these tests proved that A307 threaded rod fails at or before the breakout failure of the masonry, showing that bolts or threaded rods stronger than A307 strength should be used for future testing. Therefore, for full-scale testing the 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 full-scale wall specimens for future testing, it was decided that the wall tests would be performed with through-bolts in the third cell from the edge of the walls rather than the second cell from the edge [Goodnight et al, 2011].

2.4.2 Connector Plates

2.4.2.1 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 2-10 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 $\frac{1}{4}$-inch to $\frac{1}{2}$-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 2-10) 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 they were capable of resisting 3 kips (13 kN) of 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 \( \frac{1}{2} \)-inch (13 mm) experienced excessive and undesirable out-of-plane buckling [Goodnight et al, 2011].

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 welded 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.

2.4.2.2 Connector Plate Development Research - Phase II

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 tested were \( \frac{1}{2} \)-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 2-11, 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 Figures 2-12 through 2-14, a straight plate profile bolted connection, straight plate profile welded connection, and bent plate profile 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.
3 Material Properties

3.1 Compression Tests

Masonry test specimens for compression testing were prepared at the same time that the wall specimens were being constructed. Compression tests were performed in order to obtain accurate values for the material properties of the ungrouted and grouted masonry used for the wall specimens. Masonry has a required minimum compressive strength of 1.5 ksi (10.3 MPa), but the actual strength of the material can often be higher. One ungrouted masonry specimen and two grouted masonry specimens were capped with sulfur capping material per ASTM C1552-09a. The masonry specimens were tested in compression per ASTM C1314-11a to obtain the ultimate compressive strength of the masonry wall specimens, as shown in Figure 3-1.

The compression test results are shown in Table 3-1. However, one of the grouted masonry specimens experienced a one-sided failure, causing the strength value to be significantly lower than expected. Therefore, this value was excluded from the test results to avoid underestimating the compressive strength of the masonry wall specimens. All of the specimens failed as a result of face shell separation, as shown in Figure 3-2.

<table>
<thead>
<tr>
<th>Specimen Description</th>
<th>Cross-sectional Area (sq. in.)</th>
<th>Compressive Force (k)</th>
<th>Compressive Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ungrouted</td>
<td>35.29</td>
<td>54.25</td>
<td>1.54</td>
</tr>
<tr>
<td>Grouted</td>
<td>58.14</td>
<td>103</td>
<td>1.77</td>
</tr>
</tbody>
</table>

Figure 3-1: Masonry Compression Test Setup

Figure 3-2: Masonry Face Shell Separation Failure

Table 3-1: Masonry Compression Test Results
3.2 Tension Tests

3.2.1 A36 Steel Bar for Connector Plates [Ozaki-Train et al, 2011]

Direct tension tests were performed in order to obtain accurate values for the material properties of the particular stock of ASTM grade A36 steel used for the connector tests [Ozaki-Train et al, 2011], as shown in Figure 3-3. A36 steel has a required minimum yield strength of 36 ksi (248 MPa), but the actual strength of the material can often be significantly higher. Likewise the ultimate strength of A36 steel is required to be at least 58 ksi (400 MPa), but may be higher. To clearly define the connector steel's stress-strain behavior, tension tests were performed according to ASTM E8 [ASTM, 2008]. The tension test results are summarized in Table 3-2.

![Figure 3-3: A36 Steel Bar Tension Test Setup [Ozaki-Train et al, 2011]](image)

Table 3-2: A36 Steel Bar Tension Test Results

<table>
<thead>
<tr>
<th>Specimen Stock</th>
<th>Yield Stress (ksi)</th>
<th>Yield Strain</th>
<th>Ultimate Stress (ksi)</th>
<th>Ultimate Strain</th>
<th>Young's Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4&quot; x 1/2&quot;</td>
<td>53.5</td>
<td>0.00168</td>
<td>83.3</td>
<td>0.168</td>
<td>32,000</td>
</tr>
<tr>
<td>6&quot; x 1/2&quot;</td>
<td>53.1</td>
<td>0.00178</td>
<td>80.7</td>
<td>0.167</td>
<td>31,000</td>
</tr>
</tbody>
</table>

(1 in. = 25.4 mm; 1 ksi = 6.895 MPa)
Using these results, it was decided that the steel properties used for the connector plates were:

- Yield Strength = 50 ksi (345 MPa)
- Ultimate Strength = 80 ksi (552 MPa)
- Young’s Modulus = 30,000 ksi (207 GPa)

Poisson’s ratio for the steel was assumed to be 0.3. The relationship between Poisson’s ratio ($\nu$), Young’s Modulus ($E$), and the Shear Modulus ($G$) is given by the equation:

$$G = \frac{E}{2(1 + \nu)} = 11,540 \text{ ksi (80 GPa)}$$

The full stress-strain curves for each tension specimen can be found in Appendix A3.

### 3.2.2 Reinforcing Bars In Masonry Wall Specimens

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 three samples of the #5 (16 mm) bars in the masonry wall specimens. The tension test results are summarized in Table 3-3.

![Steel Reinforcing Bar Tension Test Setup](image)
Table 3-3: Grade 60 Steel Reinforcing Bar Tension Test Results

<table>
<thead>
<tr>
<th>Specimen No.</th>
<th>Yield Stress (ksi)</th>
<th>Yield Strain</th>
<th>Ultimate Stress (ksi)</th>
<th>Ultimate Strain</th>
<th>Young's Modulus (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>69.2</td>
<td>0.0023</td>
<td>102.33</td>
<td>*</td>
<td>30,498</td>
</tr>
<tr>
<td>2</td>
<td>68.8</td>
<td>0.0020</td>
<td>101.34</td>
<td>0.0965</td>
<td>34,404</td>
</tr>
<tr>
<td>3</td>
<td>68.8</td>
<td>0.0028</td>
<td>101.44</td>
<td>0.146</td>
<td>25,004</td>
</tr>
<tr>
<td>Average</td>
<td>68.93</td>
<td>0.0024</td>
<td>101.70</td>
<td>0.121</td>
<td>29,969</td>
</tr>
</tbody>
</table>

*Unknown - Extensometer slip before ultimate stress achieved

(1 ksi = 6.895 MPa)

Using these results, it was decided that the steel properties used for the reinforcing bars were:

Yield Strength = 68 ksi (469 MPa)
Ultimate Strength = 100 ksi (690 MPa)
Young’s Modulus = 30,000 ksi (207 GPa)

Poisson’s ratio for the steel was assumed to be 0.3. The relationship between Poisson’s ratio ($\nu$), Young’s Modulus (E), and the Shear Modulus (G) is given by the equation:

$$G = \frac{E}{2 + (1 + \nu)} = 11,540 \text{ ksi (80 GPa)}$$

A sample stress-strain curve for specimen no. 3 can be found in Appendix A3.
4 Approach

4.1 Masonry Wall Specimen Connector Plate Testing

4.1.1 Experimental Setup

In the preceding phases of connector testing, Goodnight used the test setup shown in Figure 4-1 [Goodnight et al, 2011] and Ozaki-Train used the test setup shown in Figure 4-2 [Ozaki-Train et al, 2011]. Goodnight’s setup was used to test a single link or fuse connector plate, while Ozaki-Train’s setup tests a pair of link or fuse connectors. Both of these test programs were vital for development of the link and fuse connectors to be tested on the full-scale subassemblies with masonry walls.

![Figure 4-1: Single connector plate test setup [Goodnight et al, 2011]](image)

![Figure 4-2: Double side plate connector test setup [Ozaki-Train et al, 2011]](image)
Many of the features used on these test setups were also included on the masonry wall subassembly testing, except on a larger scale. The full test setup is shown in Figures 4-3 and 4-4, as well as Appendix A1. A steel A-frame was bolted to the structures laboratory strong-floor. This frame was used to support the MTS hydraulic actuator and was laterally braced by a structural steel HSS member to the structures laboratory wall. Two steel intermediate bracing supports were also bolted to the structures laboratory strong-floor. A #8 steel reinforcing bar was placed along the floor on each side of the frame and was welded to the A-frame base, bracing support bases, and the masonry wall specimen base. This was used to prevent the wall specimen from sliding while load was being applied to the wall by the actuator. In addition, two steel plates were welded in between the two bars near the wall specimen to restrain the bars from buckling, as shown in Figure 4-4.

Figure 4-3: Full view of experimental test setup
4.1.1.1 Masonry Wall Specimens

Three masonry wall specimens were constructed using standard 8” x 8” x 16” CMU blocks. Each wall specimen was 6’-8” long and 2’-8” high (i.e. five blocks long and four courses high), simulating a portion of a small-scale CMU infill wall on the second story of a typical building structure. The CMU blocks were constructed on top of a 6-inch concrete slab, which was cast above a W24x76 steel beam. This beam was bolted to the structures laboratory strong-floor in three locations (i.e. a 2”-diameter high strength threaded rod at the two ends and a ¾”-diameter threaded rod in the center) to securely anchor the wall specimens during testing. At each rod location, stiffener plates were welded to the beam to strengthen the beam web.

One of the three wall specimens was fully-grouted, meaning that all of the cells were filled with grout. The other two specimens were partially-grouted, meaning that only cells containing steel reinforcing bars were grouted. All three of the specimens were reinforced identically with a total of four vertical #5 (16 mm) bars and one horizontal #5 (16 mm) bar with 180° hooks at each end. A vertical bar was centered in each of the two end cells in addition to #5 bars at 24 inches (610 mm) on center vertically in between, as shown in Figure 4-5. All four vertical bars were welded to the steel beam at the base of the specimen and were vertically continuous through the concrete layer and CMU blocks. The horizontal bar was centered in the second course from the top, to develop a bond beam. This course was entirely grouted in all three wall specimens. Three ¾”-diameter holes were drilled through the bond beam 1’-1” from the
top of the wall, centered in accordance with the center of the fuse and link connector plate locations, as shown in Figure 4-6. During testing, ¾"-diameter x 10"-long, high-strength A490 (minimum tensile strength of 150 KSI and minimum yield strength of 120 KSI) through-bolts were used to secure each pair of connector plates on each side of the wall as shown in Figure 4-7.

Figure 4-5: Masonry wall specimen rebar layout

Figure 4-6: Holes in masonry wall for through bolts

Figure 4-7: ¾"-ø A490 through bolt
4.1.1.2 Steel Beam Above Wall Specimens

Shear load was transferred from the hydraulic actuator through a W18x40 steel beam, simulating the horizontal beam of a steel frame, as shown in Figure 4-8. This beam was laterally braced to the structures laboratory wall, as shown in Figure 4-4, to prevent the beam from moving or rotating transverse to the wall specimen while being loaded. Because these tests are for Type I Hybrid Masonry connectors, the beam was supported 1" above the CMU wall specimen in order to avoid axial load transfer onto the wall. A ½” steel plate was welded to the end of the beam to allow for a clevis connection to the actuator. Two pin-ended vertical load rods were used to support the steel beam 1” above the wall specimen and to maintain the beam horizontal during translation, also shown in Figure 4-4. These load rods were attached to load cells (55-kip axial capacity) to record overturning uplift and compression forces due to the lateral force in the beam. A negative reading from the load rods indicates compression in the rods.

On each side of the W18x40 steel beam, three 8” wide x ¾” thick steel side plates were attached along the top and bottom flange of the beam with 7/16” fillet welds across the 8” width of the plates. These plates (6 total) were used as side plates for bolting the top of the fuse and link connector plates to the steel top beam. The locations of the side plates along the beam were determined by centering each plate with the center of the fuse and link connector plate locations as described previously. Six 1-1/16”-diameter holes were drilled in each side plate (i.e. two rows of three holes each) to match the two holes in the 4” fuse connector plate and the four holes in the 6” fuse and bolted link connector plates. The plate surfaces were cleaned with a grinder for the slip critical connection of the fuse or link connector plates.

Figure 4-8: Steel side plates on steel beam
4.1.2 Tapered Fuse Connector Specimens

Two different types of tapered fuse side connector plates were used for the full-scale masonry wall subassembly tests. The first connector plate was a ½” thick, bolted 4 inch (102 mm) tapered fuse connector plate similar to one previously tested by Goodnight, as depicted in Figure 4-9. This connector plate had an aspect ratio of 3:1 (i.e. the length of the fuse is three times larger than the widest portion of the fuse) and was designed to yield across the entire fuse length at the same time. As shown, this connector required two 1 inch (25.4 mm) diameter A490SC bolts to secure the connector plate to the side plate. The second side connector plate tested was a ½” thick, bolted 6 inch (152 mm) tapered fuse connector plate similar to one previously tested by Ozaki-Train, as depicted in Figure 4-10. This connector plate had an aspect ratio of 2.4:1 and was designed to yield across the entire fuse length at the same time. As shown, this connector required four 1 inch (25.4 mm) diameter A490SC bolts to secure the connector plate to the side plate.

These tapered fuse connector plates were chosen for the masonry subassembly tests based on their performance during previous testing on the single and double connector plate test setups. One significant difference between the connector plates...
used during this phase of testing and the previous phases of testing was the fabrication method used to produce the connector plates. Goodnight and Ozaki-Train used an end-mill to fabricate all bolt holes, slotted holes, and fuse sections, while all of the connector plates for the full scale subassembly tests were produced with a C&C plasma cutting table. The plasma cutting table was used due to the large number of connector plates that were required for these tests and the efficiency of the plasma cutting table to cut the same shape repeatedly. Additionally, the plasma cutting table fabrication method is a more realistic cutting method in the construction industry, as milling each connector plate would be impractical and expensive.

These connector plates were bolted to side plates on the beam above the wall specimen using either two (for the 4 inch [102 mm] fuse connector) or four (for the 6 inch [152 mm] fuse connector) A490SC bolts. All of the bolted connections were slip critical to prevent undesired slipping of the bolted connections. This required the removal of mill scale from the contact surface with the side plate prior to bolting. Load indicating squirter-type washers were used to ensure that a slip critical connection was developed. These washers indicate when the proper level of tension has been achieved by squirting orange paint out of seven of the eight protrusions on the back face of the washer, as shown in Figure 4-11. The specification for the load indicating washers is included in Appendix A1.

\[\text{Figure 4-11: Load indicating washer at slip critical bolted connection}\]

4.1.3 Experimental Test Specimens

All test specimens followed a similar naming convention to that which was developed during the single and double connector plate testing phases of the hybrid masonry project [Goodnight et al, 2011 and Ozaki-Train et al, 2011]. The naming convention was altered so that it would be applicable to the full scale subassembly tests. Since these tests were for type 1 masonry wall tests, ‘HT1’ was used as the prefix before the specimen name with either a ‘P’ or ‘F’ before it representing either a partially grouted or fully grouted wall specimen, respectively. All of the connector plates were bolted to side plates on the beam above the wall specimen using either two or four A490 bolts, as described in Section 4.1.2. In addition, since these tests involved multiple pairs of connector plates, the total number of connector plates used for each test specimen was added to the specimen name with a ‘B’ (for bolted connector plate) followed by the number of plates used on the test specimen. One of the test specimens was tested with a unique loading routine, which was noted with ‘RC’ in the specimen name to distinguish that a rapid cycling routine was used during testing. The naming convention is fully documented in Table 4-1, which shows the names developed for the five tests performed during this phase of research.
4.1.4 Test Instrumentation

Various instruments were used to obtain vital data during testing. Two different instrumentation layouts are shown in Figures 4-12 and 4-13. An MTS actuator capable of applying loads of ±300 kips (1335 kN) and displacements of ±15 inches (381 mm) provided the load being applied to the beam above the masonry wall specimen. The displacement of the connector plates was measured by a string potentiometer located on the beam above the masonry specimen and connected to the east edge of the wall panel to measure relative horizontal displacement between the beam and wall. Two load cells capable of recording ±30 kips (133.5 kN) were located on vertical load rods, which measured the vertical forces required to keep the beam horizontal above the wall specimen. The sum of these forces is equal and opposite to the vertical load in the connector plates. In addition, four linear variable differential transformers (LVDT) were used to measure any potential slip at specific joint locations on the wall specimen. The first LVDT was positioned horizontally on the front face of the wall specimen between the steel base beam and the concrete slab. The second LVDT was positioned horizontally on the front face of the wall specimen between the concrete slab and the bottom course of the masonry wall panel. The third and fourth LVDT were placed vertically on each end of the wall specimen between the base beam and concrete slab.

Table 4-1: Test Specimen Naming Convention Summary

<table>
<thead>
<tr>
<th>XHT1</th>
<th>-</th>
<th>PX</th>
<th>TX</th>
<th>FTX</th>
<th>BX</th>
<th>-</th>
<th>X</th>
<th>Specimen Designation</th>
</tr>
</thead>
<tbody>
<tr>
<td>PHT1</td>
<td>P4</td>
<td>T4</td>
<td>F3</td>
<td>B4</td>
<td></td>
<td></td>
<td></td>
<td>PHT1-P4_T4_FT3_B4</td>
</tr>
<tr>
<td>PHT1</td>
<td>P4</td>
<td>T4</td>
<td>F3</td>
<td>B6</td>
<td></td>
<td></td>
<td></td>
<td>PHT1-P4_T4_FT3_B6</td>
</tr>
<tr>
<td>PHT1</td>
<td>P4</td>
<td>T4</td>
<td>F3</td>
<td>B6</td>
<td></td>
<td></td>
<td>RC</td>
<td>PHT1-P4_T4_FT3_B6-RC</td>
</tr>
<tr>
<td>PHT1</td>
<td>P6</td>
<td>T4</td>
<td>F3</td>
<td>B4</td>
<td></td>
<td></td>
<td></td>
<td>PHT1-P6_T4_FT2.4_B4</td>
</tr>
<tr>
<td>FHT1</td>
<td>P6</td>
<td>T4</td>
<td>F3</td>
<td>B4</td>
<td></td>
<td></td>
<td></td>
<td>FHT1-P6_T4_FT2.4_B4</td>
</tr>
</tbody>
</table>

Note: Fuses of type ‘FT’ are tapered fuses and the associated number is the aspect ratio of the fuse, based on fuse length divided by the widest fuse dimension. ‘RC’ indicates a rapid cycling routine; all others were tested with a standard cycling routine.

(1in = 25.4 mm, 1kip = 4.45 kN)
Before testing PHT1-P6_T4_FT2.4_B4, four additional string potentiometers (i.e. two diagonal on the front face of the wall panel and one vertical on each end of the wall panel) were added to the wall specimen, as shown in Figure 4-13. This was done because the load transferred into the wall by the larger connector plates was expected to damage the wall. The intent of these sensors was to monitor the deformation in the wall specimen. After this specimen failed, two additional diagonal string potentiometers were added to the back face of the wall panel. Also, the two vertical LVDTs were removed to add steel collars at the end of the concrete slab, as shown in Figure 4-14. These collars were added to prevent the concrete slab from cracking at the boundary of the wall specimen, which is not an expected failure mode in real applications because the concrete slab below the masonry wall would be continuous preventing breakout of the slab.
1 – MTS hydraulic actuator
2 – String Potentiometer
3 – Load rod (Typical of 2)
4 – LVDT (Typical of 4)
5 – Pulley
6 – Added String Potentiometers (Typical of 4)

Figure 4-13: Instrumentation layout for PHT1-P6_T4_FT2.4_B4

Figure 4-14: Welded steel collar at boundary of concrete slab for FHT1-P6_T4_FT2.4_B4
4.1.5 Procedure – Theoretical masonry shear wall strength capacity

Theoretical values for the strength of the partially and fully grouted masonry shear walls were calculated for comparison with the experimental results and providing approximate estimates for the capacity of the wall specimens. The material properties discussed in Section 3 were used while calculating the theoretical strength values to obtain more accurate estimates for the strengths of the walls. Three failure capacities for the masonry shear walls were calculated: bending moment failure, shear failure, and shear friction failure. Spreadsheets showing these calculations are included in Appendices A4 and A5.

4.1.5.1 Bending moment capacity

The first step of the calculation was to assume a location of the neutral axis, \( c \) (i.e. the distance from the edge of the wall to the neutral axis on the compression end of the wall). From this assumed value, the depth of the compression stress block was calculated using the following equation:

\[
a = 0.85c
\]

Then, the tension force in the wall was calculated using the following equation, where the area of tension steel, \( A_s \), is the three vertical #5 bars in the wall which would theoretically be in tension:

\[
T = A_s f_y
\]

Where,

\[
A_s = 3 \times \frac{\pi (5/8)^2}{4}
\]

\[
f_y = 68 \text{ ksi} \quad \text{(from the material property tests)}
\]

The compression force in the wall is theoretically calculated using the equation:

\[
C = 0.85 f_m' b a
\]

Where,

\[
f_m' = \begin{cases} 1.54 \text{ ksi} \quad \text{(partially grouted specimen)} \\ 1.77 \text{ ksi} \quad \text{(fully grouted specimen)} \end{cases}
\]

\[
b = 7.625 \text{ in}
\]

\[
a = 0.85c \quad \text{(as calculated previously)}
\]

The goal seek tool in Microsoft Excel was used to set the wall tension force, \( T \), and compression force, \( C \), equal to each other by changing the assumed neutral axis location, \( c \). The resulting neutral axis depths were,

\[
c = \begin{cases} 7.38 \text{ in} \quad \text{(partially grouted wall specimen)} \\ 6.42 \text{ in} \quad \text{(fully grouted wall specimen)} \end{cases}
\]

The next step was to calculate the nominal moment of the wall, \( M_0 \), using the equation:
\[ M_0 = \Sigma T_0 \text{(Tension Distance)} - C \text{(Compression Distance)} \]

Where,

\[ T_0 = A_{s,1}f_y = \frac{\pi}{4} \left( \frac{5}{8} \text{in} \right)^2 \times 68 \text{ksi} = 20.86 \text{ kips} \]

\[ Tension \ Distance = \Sigma (28 \text{ in} + 52 \text{ in} + 76 \text{ in}) = 156 \text{ in} \]

\[ C = 55.22 \text{ kips} \text{ (as calculated previously)} \]

\[ Compression \ Distance = \frac{d}{2} \]

Therefore,

\[ M_0 = 254.86 \text{ kip} - \text{ft.} \quad \text{(for partially grouted specimen)} \]

\[ = 256.98 \text{ kip} - \text{ft.} \quad \text{(for fully grouted specimen)} \]

The nominal moment, \( M_0 \), was divided by the height at which the shear load will be transferred into the wall from the connector plates to determine the maximum amount of shear load that the wall could theoretically resist, \( V_u \), before failing in bending moment failure:

\[ V_u = \frac{M_0}{h} = \frac{254.86 \text{ kips ft}}{1.67 \text{ ft}} = 153 \text{ kips} \quad \text{(for partially grouted specimen)} \]

\[ = \frac{256.98 \text{ kips ft}}{1.67 \text{ ft}} = 154 \text{ kips} \quad \text{(for fully grouted specimen)} \]

This theoretical maximum shear value due to bending moment capacity of approximately 150 kips is applicable for both the partially grouted and fully grouted masonry shear wall because the bending moment capacity is only dependent on the vertical rebar size and locations and compression zone, which are similar in both wall specimens. The only difference in the calculation is the masonry compressive strength from the material properties testing.

### 4.1.5.2 Shear capacity

The first step in this calculation was to determine the height to length ratio, of the wall, where length is also referred to as the depth of the wall and is denoted as ‘d’ in the calculation:

\[ \frac{M_{ul}}{Vd} = \frac{Vh}{Vd} = \frac{h}{d} = \frac{1.67 \text{ ft}}{(6.67 \text{ ft} - 0.33 \text{ ft})} = 0.26 \]

The next step was to determine the equivalent wall thickness, \( t_e \), which is based on the amount of solid masonry wall or grout for each wall specimen. For the partially grouted wall specimen, due to partial grouting in masonry cells at 24 inches on center, an equivalent wall thickness of 5.2 inches was used instead of the solid wall thickness of 7.625 inches. For the fully grouted wall specimen, the equivalent wall thickness is 7.625 inches because all of the cells are fully grouted. This equivalent wall thickness was used to calculate the shear area, \( A_{m_v} \):
The shear coefficient, $C_d$, was calculated using the equation:

$$C_d = 1.6 \left(1 - \frac{M_y}{V_d}\right) + 1.2 = 1.6(1 - 0.26) + 1.2 = 2.38$$

This shear coefficient was used to determine the nominal shear capacity of the masonry wall specimens, $V_m$, with the equation:

$$V_m = C_d A_{mv} \sqrt{f_m}$$

$$= (2.38)(416.21 \text{ sq. in.}) \sqrt{1,540 \text{ psi}} = 38.86 \text{ kips} \; \text{(partially grouted wall specimen)}$$

$$= (2.38)(610.31 \text{ sq. in.}) \sqrt{1,770 \text{ psi}} = 61.09 \text{ kips} \; \text{(fully grouted wall specimen)}$$

These shear capacities are smaller values than the shear capacity based on the bending moment strength of the wall. Therefore, the ultimate shear load capacity is based on the shear capacity of the masonry wall.

### 4.1.5.3 Shear friction capacity

Before testing the wall specimens, the shear strength of the wall was thought to control the wall specimen failure. However, test results for both the partially grouted and fully grouted wall specimens confirmed that shear friction failure between the bottom course of the masonry and concrete slab was the failure mode of the walls. The current MSJC does not include a calculation for the shear friction failure of masonry. Therefore, the shear friction failure calculation from the *Building Code Requirements for Structural Concrete* (ACI 318-08) was used with approximate assumptions to make the calculation applicable for masonry.

The shear friction failure is dependent on the area of reinforcement crossing the shear friction plane, which is the area of the four vertical #5 bars crossing the masonry and concrete joint. In addition, the shear friction failure is dependent on a friction coefficient, $\mu$. Since the masonry wall panel was constructed on the concrete slab after the slab had cured, a friction coefficient of 0.6$\lambda$ was used to represent the friction coefficient between “concrete placed against hardened concrete not intentionally roughened”. [ACI, 2008]. The weight factor of the masonry, $\lambda$, used was 0.75 for lightweight concrete, which was assumed to be similar to masonry. The theoretical shear friction capacity was calculated using the following equation [ACI, 2008]:

$$V_n = A_{vf} f_y \mu$$

Where,

$$A_{vf} = 4 \times \frac{\pi (5/8 \text{ in.})^2}{4} = 1.23 \text{ in}^2$$

$$f_y = 68 \text{ ksi}$$
\[ \mu = 0.6 \lambda = 0.6 \times 0.75 = 0.45 \]

Therefore,
\[ V_h = 37.55 \text{kips} \]

This theoretical shear friction value is applicable for both the partially grouted and fully grouted masonry shear wall because the vertical rebar size and locations are identical in both wall specimens.

### 4.1.6 Procedure – Double side plate connector testing

Prior to testing, the bolted connector plate specimens were installed on the side plates of the steel beam above the masonry wall specimens using an impact wrench. All of the connector plates were bolted with A490 load indicating squirter-type washers in conjunction with the 1 inch (25.4 mm) diameter A490 bolts to develop a slip critical connection. Each bolt was tightened with the impact wrench until at least seven of the eight protrusions on the load indicating washer had squirted out paint, indicating the proper tension had been achieved. The specification for the load indicating washers is included in Appendix A2. Once the connector plates were bolted to the side plates, the \( \frac{3}{4} \) inch diameter through bolt was used to secure each pair of connector plates to the masonry wall. These bolts were not tensioned so as to allow rotation of the bottom of the connector. A layer of whitewash was painted onto the outside of each connector to observe flaking of the whitewash during yielding and to improve visibility of cracks as they appeared.

A laptop was used to collect data from a National Instruments (NI) Data Acquisitions system (DAQ). The NI-DAQ recorded the load rod, string potentiometer, and LVDT data as well as the actuator load and displacement. The NI-DAQ data were recorded in comma separated value (csv) format on the laptop using the program NI Measurements and Automation.

Once the MTS control system for the actuator and the NI-DAQ were running, the front of the actuator was attached to the steel beam. 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 one of the connector plates to observe the connector plate behavior.

The specimens were loaded in displacement control under a constant strain rate of \( 0.2 \varepsilon \text{/min} \) \( (30 \mu \varepsilon /\text{sec}) \) according to specified loading rates in ASTM A370 and E8 [ASTM, 2008]. The ATC-24 single-specimen testing program was used as a starting point for developing the testing routine for the connector tests at UHM. The test routine put each specimen through a set of three displacement cycles at each level of displacement, as shown in Figure 4-15. A single displacement cycle began with the actuator in the original zero position, progressed with an extension of the actuator to the maximum positive displacement level for that cycle followed by a retraction of the actuator back past neutral to the maximum negative displacement level for that cycle, then concluded with the actuator returning to the zero-displacement position.
The displacement levels for each set of three cycles were the same as those used in previous single and double connector plate testing routines. For the single-connector plate tests, appropriate displacement levels were chosen by calculating the yield strain for a typical connector plate and the corresponding lateral deflection [Goodnight et al, 2011]. The early displacement levels were chosen so as to slowly approach the yield point of the specimen, ensuring that at least three cycles occurred prior to yielding as shown in Figure 4-15. The displacements were increased according to a stepped routine as documented in Table 4-2. The frequencies for loading decrease with increases in the displacement to maintain the constant strain rate of $30 \mu \varepsilon$/sec. Each specimen was tested using the displacement routine in Table 4-2 until failure occurred. Failure for these tests was defined as the cycle during which the maximum load recorded was less than 80% of the peak load during testing.
Table 4-2: Displacement Routine for Double Side Plate Connector Tests

<table>
<thead>
<tr>
<th>Deformation Step Number</th>
<th>Displacement (in)</th>
<th>Frequency (Hz)</th>
<th># of Cycles</th>
<th>Total Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.8333</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>0.015</td>
<td>0.5556</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>0.03</td>
<td>0.2778</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>4</td>
<td>0.06</td>
<td>0.1389</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>0.0833</td>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>0.25</td>
<td>0.0333</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>7</td>
<td>0.5</td>
<td>0.0167</td>
<td>3</td>
<td>21</td>
</tr>
<tr>
<td>8</td>
<td>0.75</td>
<td>0.0111</td>
<td>3</td>
<td>24</td>
</tr>
<tr>
<td>9</td>
<td>1</td>
<td>0.0083</td>
<td>3</td>
<td>27</td>
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<tr>
<td>10</td>
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<td>0.0067</td>
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<td>0.0048</td>
<td>3</td>
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<td>13</td>
<td>2</td>
<td>0.0042</td>
<td>3</td>
<td>39</td>
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<tr>
<td>14</td>
<td>2.25</td>
<td>0.0037</td>
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<td>0.0033</td>
<td>3</td>
<td>45</td>
</tr>
<tr>
<td>16</td>
<td>2.75</td>
<td>0.003</td>
<td>3</td>
<td>48</td>
</tr>
<tr>
<td>17</td>
<td>3</td>
<td>0.0028</td>
<td>3</td>
<td>51</td>
</tr>
</tbody>
</table>

As previously mentioned, specimen PHT1-P4_T4_FT3_B6-RC was tested following a slightly different testing routine. The routine shown in Table 4-2 was altered to determine whether fatigue would play a significant role in the failure mode of the fuse connectors. In addition, this test was done to demonstrate the capability of fuse connectors to dissipate large amounts of seismic energy. The test routine for this specimen is shown in Table 4-3. Thirty additional cycles were added at each displacement representing approximately 0.5%, 1.0%, and 1.5% drift limits (i.e. 0.5 inch, 1.25 inches, and 1.75 inches, respectively). These drift percentages were calculated based on the projected height of the full-scale tests to be performed at UIUC of 9.75 feet (2.972 m). The additional cycles were performed at increased frequencies to decrease testing time and simulate actual seismic loading frequencies. Due to deformation of the test setup, the displacements being applied by the actuator were larger than the actual internal displacement of the fuse, so the testing routine was changed to accommodate the actual internal displacement from the string potentiometer, rather than that recorded by the actuator. This change in test routine is reflected in Table 4-3.
4.2 Future Hybrid Masonry Testing

During the next phase of testing, bolt push out tests similar to those previously performed by Goodnight will be conducted. These tests will differ from Goodnight’s previous tests in several ways. The first difference will be that the test will be performed on a full-scale masonry wall panel with identical dimensions to the experiments performed in this phase of testing. This will change the manner in which the masonry wall is supported during loading to correspond with the actual behavior of a through bolt breakout failure for Type I hybrid masonry, thus producing more relevant results than those previously obtained. The second difference will be that different edge distances and interior spacing will be tested to determine a proper edge distance and spacing for design purposes. The third difference will be that the load transfer into the masonry wall will be through link connector plates to observe their behavior while transferring lateral loads into the masonry wall. The test setup for these tests is shown in Figure 4-16.

### Table 4-3: Displacement Routine for Rapid Cyclic (RC) Connector Test

<table>
<thead>
<tr>
<th>Deformation Step Number</th>
<th>Actuator Input Displacement (in)</th>
<th>Actual Internal Connector Displacement (in)</th>
<th>Frequency (Hz)</th>
<th># of Cycles</th>
<th>Total Cycles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.01</td>
<td>0.01</td>
<td>0.8333</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>2</td>
<td>0.015</td>
<td>0.015</td>
<td>0.5556</td>
<td>3</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>0.03</td>
<td>0.03</td>
<td>0.2778</td>
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<td>9</td>
</tr>
<tr>
<td>4</td>
<td>0.06</td>
<td>0.06</td>
<td>0.1389</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>5</td>
<td>0.1</td>
<td>0.1</td>
<td>0.0833</td>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td>6</td>
<td>0.25</td>
<td>0.25</td>
<td>0.0333</td>
<td>3</td>
<td>18</td>
</tr>
<tr>
<td>7</td>
<td>0.5</td>
<td>0.5</td>
<td>0.0167</td>
<td>3</td>
<td>21</td>
</tr>
<tr>
<td>8</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>30</td>
<td>51</td>
</tr>
<tr>
<td>9</td>
<td>0.75</td>
<td>0.75</td>
<td>0.0111</td>
<td>3</td>
<td>54</td>
</tr>
<tr>
<td>10</td>
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<td>1</td>
<td>0.0083</td>
<td>3</td>
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</tr>
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<td>11</td>
<td>1.25</td>
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<td>0.0067</td>
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<td>60</td>
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<td>1.5</td>
<td>1.25</td>
<td>0.1</td>
<td>30</td>
<td>93</td>
</tr>
<tr>
<td>14</td>
<td>1.75</td>
<td>1.5</td>
<td>0.0048</td>
<td>3</td>
<td>96</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>1.75</td>
<td>0.0042</td>
<td>3</td>
<td>99</td>
</tr>
<tr>
<td>16</td>
<td>2</td>
<td>1.75</td>
<td>0.06</td>
<td>30</td>
<td>129</td>
</tr>
<tr>
<td>17</td>
<td>2.25</td>
<td>2</td>
<td>0.0037</td>
<td>3</td>
<td>132</td>
</tr>
<tr>
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<td>2.75</td>
<td>2.5</td>
<td>0.003</td>
<td>3</td>
<td>138</td>
</tr>
<tr>
<td>20</td>
<td>3</td>
<td>2.75</td>
<td>0.0028</td>
<td>3</td>
<td>141</td>
</tr>
</tbody>
</table>
Test results obtained during this phase of research will be used to finalize the design of Type I connector plates for implementation in the UIUC full-scale hybrid masonry panel tests. The conceptual test assembly of the full-scale, two-story hybrid masonry tests to be performed by UIUC is depicted in Figure 4-17.
5 Results

5.1 Masonry Wall Specimens

5.1.1 Theoretical and Expected Strength Capacity

In order to determine the expected strength capacity of the masonry wall specimens, calculations for each wall’s moment and shear strength capacities were performed. This was done in order to anticipate the approximate load at which the partially grouted and fully grouted specimens were expected to fail.

For both wall specimens, the shear strength capacity of the wall governed over the moment capacity of the wall (i.e. the shear strength capacity had a lower value than the moment capacity), which was due to the wall’s squat geometry. If the wall specimens were taller, similar to the wall specimens to be tested at UIUC, then the overturning moment on the wall may have governed over the shear strength value of the wall. The moment capacities for the partially grouted wall specimen and fully grouted wall specimen were both calculated as approximately 250 kip-ft. (339 kN-m). Therefore, the shear load required to cause a moment capacity failure was determined to be approximately 150 kips (667.5 kN). As mentioned previously, these strength values for the two different wall specimens are similar because the moment capacity is dependent on the dimensions and reinforcing of the wall, which are identical for both the partially grouted and fully grouted specimens.

The shear strength of the wall specimens, on the other hand, are based on shear area of the wall, which differs between the partially grouted and fully grouted specimen. The shear strength capacity of the partially grouted and fully grouted wall specimens were calculated as approximately 39 kips (175 kN) and 61 kips (271 kN), respectively.

Another possible mode of failure was a breakout failure of the masonry from the through bolted connection. Goodnight performed bolt pushout tests to determine what load would be required for the through bolted connection into the masonry to cause shear failure near the boundary of the wall. From these tests, it was determined that for a bolt located in the second cell from the end of the wall, a load exceeding 20 kips (89 kN) could potentially cause this breakout failure. Because of these tests, the boundary connector plates were placed three cells away from the edge of the wall, rather than the two cells that were used as the edge distance for the bolt pushout testing. Also, during the design of the connector plates used in these tests, the connector plate load capacity was limited to just below the breakout capacity of the wall specimens and this failure mode was not expected to govern during testing.

One of the failure conditions which was not considered until all tests were completed was shear friction failure of the wall between the masonry and concrete layer of the specimens. The masonry design code, ACI 530-08, provides no guidance for computing shear friction between masonry and concrete components. After performing a shear friction capacity calculation on the wall specimen based on ACI 318-08 guidelines, it was determined that approximately 37.5 kips (167 kN) of shear load being transferred into the masonry would theoretically cause slippage between the masonry and concrete layers. This value is based on the vertical reinforcing in the wall specimen.
and the friction coefficient between the masonry and concrete, which were identical for the two wall specimens.

5.1.2 Experimental Results

During testing of the 6 inch (152 mm) tapered fuse connectors, both the partially grouted and fully grouted masonry wall specimens failed in shear friction at the joint between the bottom course of the masonry wall and the concrete slab below. The shear load which caused both walls to fail was approximately 36 kips (160 kN). Since the concrete was cast and cured as a level surface before the masonry wall specimens were built on top of it, the friction between the masonry and concrete was limited. Once shear friction failure began to occur, the masonry wall panel started to slide back and forth while being loaded. However, the reinforcing bars in the wall panel resisted the loads as the wall was sliding, causing tension cracks to form near the bar locations in the wall. These tension cracks intensified until the wall specimens started to lose strength. The failure of each wall specimen will be described in further detail in Section 5.2.2 as individual specimens will be evaluated and narratives provided describing each test outcome individually.

5.2 Tapered Fuse Connector Plates

5.2.1 Single and Double Connector Plate Test Results

Prior to this set of testing, connector plates with similar dimensions were tested using a steel test frame to determine the behavior of different fuse connectors. However, the objectives of these tests were to determine the behavior of an individual connector or pair of connectors. Goodnight tested a single 4 inch (102 mm) tapered fuse connector and Ozaki-Train tested a pair of 6 inch (152 mm) tapered fuse connectors. These results were used to determine the strength and energy dissipation capabilities of these two types of connectors. In Table 5-1, a summary of the results found during Goodnight and Ozaki-Train’s tests are shown. This set of results will be used as a point of comparison with results during tests on the masonry wall specimens. This will help to determine the interaction of the steel fuses with the masonry wall and the relationship between the results obtained from the single connector plate testing to that obtained from testing multiple pairs of connector plates.
Table 5-1: Critical values from specimen testing routine

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Point</th>
<th>Maximum Load (w/all fuses intact)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Displ. (in)</td>
<td>Load (k)</td>
</tr>
<tr>
<td></td>
<td>(+)</td>
<td>(-)</td>
</tr>
<tr>
<td>P4_T4_FT3*</td>
<td>0.17</td>
<td>0.25</td>
</tr>
<tr>
<td>(One pair)</td>
<td>3.2</td>
<td>3.0</td>
</tr>
<tr>
<td>(Two pairs)</td>
<td>6.4</td>
<td>6.0</td>
</tr>
<tr>
<td>(Three pairs)</td>
<td>9.6</td>
<td>9.0</td>
</tr>
<tr>
<td>P6_T4_FT2.4**</td>
<td>0.15</td>
<td>0.13</td>
</tr>
<tr>
<td>(Two pairs)</td>
<td>22.0</td>
<td>21.6</td>
</tr>
</tbody>
</table>

* Test was performed on a single connector plate [Goodnight et al, 2011]. Load values multiplied by 2 to represent a pair of connector plates, by 4 to represent two pairs of connector plates, and by 6 to represent three pairs of connector plates, respectively.

** Test was performed on a pair of connector plates [Ozaki-Train et al, 2011]. Values shown are for a pair of connector plates. Load values multiplied by 2 to represent two pairs of connector plates.

(1in = 25.4 mm, 1kip = 4.45 kN)

5.2.2 Load-Displacement Hysteretic Diagrams and Specimen Narratives

Load values obtained from the MTS actuator and relative internal displacement of the connector plates obtained from a string potentiometer were plotted to create a hysteresis plot for each specimen. This plot is shown along with a narrative discussing any significant events that occurred during testing. Displacements were also obtained from LVDT sensors at different locations on the wall specimens, as discussed in Section 4.1.4. These displacements were plotted separately against both test time and relative internal displacement of the connector plates to determine if any type of slippage is occurring between materials (e.g. the joint between the bottom of the masonry wall and the concrete layer) during the test, and if so, when and where it occurred. These plots are shown in Appendices A6 and A7.

5.2.2.1 PHT1-P4_T4_FT3_B4

PHT1-P4_T4_FT3_B4 was the first test performed on the partially grouted wall specimen. Failure of the fuse connector plates within the fuse section of the plates were consistent with previous tests run individually by Goodnight, as shown in Figure 5-1. However, while Goodnight’s single connector remained intact for numerous cycles at ±3 inch (76 mm) displacement, all four of the fuse connector plates failed at or just before the ±2.75 inch (70 mm) displacement cycles.
Yielding of the plates occurred along the length of the fuse on both sides of the wall at approximately 7 kips (28.5 kN) and ±0.18 inch (4.6 mm) displacement. This was made visible by flaking of the whitewash layer during this displacement cycle and verified with the change in load-displacement slope on the hysteretic plot, shown in Figure 5-2. Fractures along the edge of the fuse section of the connector plates started to appear during the ±2.5 inch (63.5 mm) displacement cycles, as shown in Figure 5-3. Finally, failure in all of the fuse connectors occurred during the 46th and 47th cycle of testing (±2.75 inch [70 mm] displacement). The maximum load achieved was approximately 13.5 kips (60 kN) during the ±2.25 inch (57 mm) displacement cycles. Based on video taken during the testing and the LVDT sensor data, little to no slippage of the test setup occurred during testing.
Figure 5-2: Load-Displacement hysteretic diagram for PHT1-P4_T4_FT3_B4
5.2.2.2 PHT1-P4_T4_FT3_B6

PHT1-P4_T4_FT3_B6 was the second test performed on the partially grouted wall specimen. Similar to the previous test and Goodnight's single connector test, failure of the fuse connector plates occurred within the fuse section of the plates. Additionally, similar to the previous test, failure of the connector plates occurred at or before the ±3 inch (76 mm) displacement cycles, while Goodnight's test showed the connector plate's ability to perform for numerous cycles at ±3 inch displacement. Unlike the first test, one of the fuse connectors had a slight fabrication error, which led to premature failure of that fuse connector. The fabrication error was a 1/16 inch diameter groove along the edge of the fuse which occurred during plasma cutting of the fuse, shown in Figure 5-4. Prior to testing, it was thought that the small imperfection would not cause premature failure of the fuse, but the slight error played a fairly significant role in the early failure of the fuse, causing a crack prematurely at the location of the groove, shown in Figure 5-5. However, after the early failure of this fuse connector, the other five connector plates remained intact and resisted their expected loads until failure, providing relevant test results.
Flaking of the whitewash layer and a change in slope of the load-displacement hysteretic plot (shown in Figure 5-6) verified that yielding of the connector plates occurred at approximately 9.6 kips (43 kN) at ±0.16 inch (4 mm) displacement. The fuse connector plate with the fabrication error failed during the 41st cycle of testing at ±2.25 inch (57 mm) displacement. The maximum load achieved with all six connector plates was 19 kips (85 kN) and occurred at this displacement cycle. Fractures along the edge of the other fuse connector plates started to occur with the subsequent ±2.50 inch (63.5 mm) displacement cycles. Finally, failure occurred in all of the fuse connectors during cycle 48 through 50 of testing (±2.75 inch [70 mm] and ±3.00 inch [76 mm] displacement). The maximum load achieved with the remaining five connector plates was approximately 16 kips (71 kN), which is 84% of the peak load, during the ±2.50 inch (63.5 mm) displacement cycles. Similar to the first test, no evidence of significant slippage within the test setup was indicated from either the video footage of the test or the LVDT sensor data.
5.2.2.3 PHT1-P4_T4_FT3_B4-RC

After the first two tests on the partially grouted masonry specimen, it was observed that the holes in the wall were slightly elongated from previous testing. However, it was determined that this would not significantly impact future testing with the wall specimen because the masonry damage was limited to the area around the holes. The third test performed on the partially grouted wall specimen (PHT1-P4_T4_FT3_B6-RC) was to determine the 4-inch (102 mm) fuse connectors’ ability to dissipate large amounts of energy by repeating rapid cycles at specific displacement levels of the test, as discussed previously in Section 4.1.6.

Although testing was performed using a different displacement-frequency procedure than the previous two tests, the fuse connectors failed within the fuse section of the plates, similar to the previous tests and Goodnight’s single connector test. However, due to the difference in the testing procedure, the fuses did not perform in the same manner as the previous tests or Goodnight’s single connector test. While the connectors remained intact until the ±2.75 inch (70 mm) and ±3 inch (76 mm) displacement cycles previously, the connectors failed during the rapid cycling displacement level of ±2 inches (51 mm).

Similar to PHT1-P4_T4_FT3_B6, yielding of the plates occurred along the length of the fuse on both sides at approximately 9.6 kips (43 kN) at the ±0.16 inch (4 mm)
displacement, which was made visible by flaking of the whitewash layer during testing and verified with the change in load-displacement slope on the hysteretic plot, shown in Figure 5-7. Fractures along the edge of the fuse section of the connector plates started to appear during the ±2 inch (51 mm) displacement level during the 3 cycles at the routine frequency of 0.0042 Hertz, which occurred just before the ±2 inch (51 mm) rapid cycles. All of the fuse connectors failed within the first ten cycles during the ±2 inch (51 mm) rapid cycles. The maximum load achieved was approximately 17.5 kips (78 kN) during the ±1.75 inch (44.5 mm) displacement cycles.

Again, based on video taken during the testing and the LVDT sensor data, little to no slippage of the test setup occurred during testing. However, during testing, the A-frame (which supports the actuator) was observed lifting approximately 0.25 inch (6.5 mm) off the ground during the pulling portion of the cycles due to bending of the channel at the bottom of the frame. This slight lifting of the test setup caused the displacements recorded by the actuator during testing to be inaccurate by approximately 0.25 inch (6.5 mm). It did not affect the fuse deformation recorded by the string potentiometer. The test setup was modified prior to the next test by placing a 1 inch thick steel plate washer across the bottom of the A-frame and anchor bolted with an impact wrench to minimize the lifting action of the frame.

Figure 5-7: Load-Displacement hysteretic diagram for PHT1-P4_T4_FT3_B6-RC
5.2.2.4 PHT1-P6_T4_FT2.4_B4

PHT1-P6_T4_FT2.4_B4 was the fourth test performed using the partially grouted wall specimen, but the first test featuring the 6 inch (152 mm) fuse connector plates. As previously discussed in Section 4.1.4, four string potentiometers were added to the masonry wall to determine the amount of damage the wall sustained during the test due to the relatively large loads expected from these fuse connector plates. Based on previous tests by Ozaki-Train on these fuse connectors, the maximum expected load was more than double of what the previous test had achieved.

Failure of this test occurred due to shear-friction failure between the bottom course of masonry and the concrete slab below. The shear-friction failure started to occur during the ±1.25 inch (32 mm) displacement cycles of the test. Cracks near the reinforcement locations of the wall specimen started to appear on both faces of the wall, indicating the beginning stages of wall failure. By observing the movement of the wall during testing, it was determined that the wall had failed in shear-friction failure causing the masonry portion of the wall to drag from side to side along the concrete slab as load was being transferred through the connector plates. This movement caused the wall to crack vertically in the locations of the vertical bars because the bars were resisting the movement of the wall, causing the masonry along each bar to be loaded in tension. Due to masonry’s relatively low tensile strength, the wall started to lose strength capacity and cracks began to form. The wall cracking continued to progress until the wall lost almost all strength capacity during the last of the ±1.75 inch (44.5 mm) displacement cycles, after a total of 36 cycles, as shown in Figure 5-8.

In addition to the shear-friction failure at the ±1.25 inch (32 mm) displacement cycles, the overturning vertical load at the two boundaries of the wall caused cracking to occur at the two ends of the concrete slab below the masonry wall, as shown in Figure 5-9. This type of failure was not representative of a real structure because the concrete below the wall would be a continuous concrete slab, which would not crack in this manner. This issue was mitigated during the next test by welding steel collars at each end of the concrete layer to simulate a continuous slab.
Figure 5-8: PHT1-P6_T4_FT2.4_B4: Failure of wall specimen

Figure 5-9: PHT1-P6_T4_FT2.4_B4: Crack at boundary of concrete layer

Figure 5-10: PHT1-P6_T4_FT2.4_B4: Fuse connector plate during testing
The fuse connector plates (shown in Figure 5-10) performed as expected during testing. Flaking of the whitewash layer and a change in slope of the load-displacement hysteretic plot (shown in Figure 5-11) verified that yielding of the connector plates occurred at approximately 20 kips (89 kN) at ±0.16 inch (4 mm) displacement. The maximum load achieved was approximately 36 kips (160 kN) and occurred at the ±1.75 inch (44.5 mm) displacement cycle. The fuse connector plates did not have any visible cracking along the edges of the fuse sections indicating that the connector plates could have resisted higher loads if the masonry wall had not failed. Based on video taken during the testing and the LVDT sensor data, the only slippage of the test setup occurred between the masonry and concrete slab.

![Graph](image)

Figure 5-11: Load-Displacement hysteretic diagram for PHT1-P6_T4_FT2.4_B4

5.2.2.5 FHT1-P6_T4_FT2.4_B4

Due to the failure of the partially grouted wall specimen with the 6 inch (152 mm) fuse connector plates, the same test was repeated on the fully grouted wall specimen, with the expectation that the fully grouted wall specimen would be able to resist more load than the partially grouted wall specimen. Also, due to the nature of the failure from
the previous test, two more string potentiometers were added to the new masonry wall to monitor the wall’s behavior during testing.

Unfortunately, the fully grouted wall specimen performed similar to the partially grouted wall specimen, also failing due to shear-friction at approximately the same load (37 kips [165 kN]). Also, similar to the previous test, the shear-friction failure started to occur during the ±1.25 inch (32 mm) displacement cycles of the test. However, rather than vertical tension cracks appearing at all of the vertical rebar locations of the wall, the cracks only initially appeared at the two end locations of the rebar, as shown in Figure 5-12. Then, after the two outside edges of the wall completely spalled off, the two inside bars were left to resist the back-and-forth movement of the masonry sliding on the concrete. This shear-friction slip between the masonry and concrete slab caused cracking near the inside bar locations on both faces of the wall until both inside vertical bars sheared at the joint, as shown in Figure 5-13.

In contrast with the partially grouted wall specimen, this wall specimen did not start to lose strength until approximately the 42nd cycle at ±2.25 inch (57 mm) displacement. However, the maximum load of 37 kips (165 kN), which was reached during the ±1.75 inch (44.5 mm) displacement cycle, did not increase as the displacement increased.

Figure 5-12: FHT1-P6_T4_FT2.4_B4: Cracking at two ends of wall specimen
Figure 5-13: FHT1-P6_T4_FT2.4_B4: Failure of masonry wall specimen

Similar to the previous test, the fuse connector plates performed as expected during testing. Flaking of the whitewash layer and a change in slope of the load-displacement hysteretic plot (shown in Figure 5-14) verified that yielding of the connector plates occurred at approximately 20 kips (89 kN) at ±0.20 inch (5.1 mm) cycle. The hysteretic plot shows the relative displacement of the connector plates, but these values do not match the displacement input values for the actuator cycling due to the sliding of the masonry (i.e. when the masonry slides the displacement of the fuses do not match the displacement of the actuator). Since shearing of the two inside bars occurred, the sliding was significantly larger in this test and the displacement difference between the fuses and actuator was made very apparent. Again, based on video taken during the testing and the LVDT sensor data, the only slippage of the test setup occurred between the masonry and concrete slab.
5.2.3 Additional Test Result Diagrams

In addition to measuring shear force and deformation experienced by the connectors, each test also recorded the vertical force in the connectors using load rods on either end of the test-setup. The sum of forces in both load rods were calculated and plotted against the horizontal displacement, and superimposed on the corresponding hysteresis for shear load. Note that the vertical loads shown are those for the connector plates, which are equal and opposite of the combined vertical loads in the load rods. As mentioned earlier, LVDT sensors were also placed at various joint locations along the wall to determine any failure or slip of the wall specimen during testing. Displacements recorded from these sensors were plotted separately against time and horizontal displacement to determine when and where, if any slip occurred, during the testing procedure. An example of the three different plots are shown in Figure 5-15, while the plots for all of the test specimens are shown in Appendices A5 through A7.
Based on the vertical forces recorded by the load rods, the vertical loads within the connector plates were relatively small. For the 4 inch fuse specimens, the maximum vertical load on each pair of fuse connectors was approximately 1.6 kips (7.1 kN). In addition, none of the tests involving the 4 inch fuse connectors experienced any type of slip during testing as indicated by the LVDT plots. The plots showed the LVDTs recording vibrations during testing, but all of the displacements recorded were below ±0.01 inch (0.25 mm) displacement, indicating that no movement occurred within the wall specimen during testing.

For the 6 inch (152 mm) fuse specimens, the vertical loads recorded were slightly higher than the loads recorded for the 4 inch (102 mm) fuse connectors. The maximum vertical load on each pair of 6 inch (152 mm) fuse connectors was approximately 5 kips (22 kN). However, the increase in the vertical loads is approximately equivalent to the increase in shear capacity between the 6 inch fuse connectors and 4 inch (102 mm) fuse connectors (i.e. the 6 inch [152 mm] fuse connector plates carry approximately 2.5 times more shear load than the 4 inch [102 mm] fuse connector plates, while the vertical load in the 6 inch [152 mm] fuse connector plates is approximately 3 times higher than

**Figure 5-15: PHT1-P4_T4_FT3_B4: (a) Hysteresis w/ vertical force diagram; (b) LVDT displacement vs. time; (c) LVDT displacement hysteresis (clockwise from top left)**

(1 in = 25.4 mm, 1 kip = 4.45 kN)
the 4 inch [102 mm] fuse connector). Due to the shear friction failure during the two tests involving the 6 inch fuse connector plates, the LVDT plots show that large displacements occurred between the masonry and concrete slab. Similar to the 4 inch (102 mm) fuse test, all other LVDTs did not record any type of displacement occurring on the wall specimen because the concrete slab never separated from the steel base beam during any of the tests. Additionally, during the 6 inch (152 mm) fuse tests, string potentiometers on the masonry wall specimen showed displacement across the masonry wall and vertical displacement between the masonry and concrete layer at the two ends of the wall. The displacements from the string potentiometers were plotted separately against time and relative connector plate displacement. An example of the two different plots are shown in Figure 5-16, while the plots for the two test specimens involving the 6 inch (152 mm) fuse connector plates are shown in Appendix A8.

![Figure 5-16: PHT1-P6_T4_FT2.4_B4: (a) String potentiometer displacement vs. time (left); (b) String potentiometer displacement hysteresis (1 in = 25.4 mm)](image)

Although the string potentiometers were placed on the wall specimen to measure yielding of the vertical rebar in the wall, the early wall failure due to shear friction prevented the rebar from yielding. Therefore, the string potentiometers displacement results were used to detect when and where major displacements occurred during testing due to the shear friction failure. As indicated by both the LVDT plots and string potentiometer plots, the partially grouted and fully grouted wall specimens started to fail during the ±1.25 inch (32 mm) displacement cycles. The displacements recorded after this cycle became progressively larger as the masonry started to crack at the bar locations and the friction area between the bottom masonry course and concrete began to decrease.

### 5.2.4 Critical values from hysteretic plots

The hysteretic plots were used to determine several critical values, namely yield point, maximum load, and maximum displacement for each specimen. These values are summarized in Table 5-2, which includes the force and displacement levels for each critical value.
Table 5-2: Critical values from specimen testing routine

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Point</th>
<th>Maximum Load</th>
<th>Maximum Displacement (w/all fuses intact)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Displ. (in)</td>
<td>Load (k)</td>
<td>Displ. (in)</td>
</tr>
<tr>
<td></td>
<td>(+) (-) (+) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-) (+) (-)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B4</td>
<td>0.18 0.17</td>
<td>7.3 6.3</td>
<td>2.2 2.4</td>
</tr>
<tr>
<td>(per pair)</td>
<td>3.65 3.15</td>
<td>6.75 6.75</td>
<td>6.75 6.75</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6*</td>
<td>0.16 0.18</td>
<td>9.6 9.4</td>
<td>1.8 1.8</td>
</tr>
<tr>
<td>(per pair)</td>
<td>3.20 3.13</td>
<td>6.2 6.13</td>
<td>5.6 5.4</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6-RC**</td>
<td>0.16 0.2</td>
<td>9.6 9.0</td>
<td>1.5 1.8</td>
</tr>
<tr>
<td>(per pair)</td>
<td>3.20 3.00</td>
<td>5.73 5.83</td>
<td>5.73 5.83</td>
</tr>
<tr>
<td>PHT1-P6_T4_FT2.4_B4***</td>
<td>0.16 0.16</td>
<td>20.0 20.4</td>
<td>1.0 1.8</td>
</tr>
<tr>
<td>(per pair)</td>
<td>10.0 10.2</td>
<td>17.75 18.55</td>
<td>17.75 18.55</td>
</tr>
<tr>
<td>FHT1-P6_T4_FT2.4_B4***</td>
<td>0.20 0.20</td>
<td>20.0 20.0</td>
<td>1.2 1.0</td>
</tr>
<tr>
<td>(per pair)</td>
<td>10.0 10.0</td>
<td>18.75 17.9</td>
<td>18.75 17.9</td>
</tr>
</tbody>
</table>

*Early failure of one fuse due to fabrication error - Maximum displacement values with only 5 fuses intact
**Failure caused by fatigue due to rapid cycling
***Limited by failure of wall specimen; fuse connector plates still intact after testing completed

(1 in = 25.4 mm, 1 kip = 4.45 kN)

As noted in Table 5-2, certain values are not representative of the behavior expected from previous testing. During PHT1-P4_T4_FT3_B6, one of the fuses failed prematurely due to a fabrication error causing the maximum displacement values to only be representative of 5 connector plates rather than 6 connector plates. Similarly, PHT1-P4_T4_FT3_B6-RC does not present representative maximum displacement values because the fuses connectors were fatigued during the rapid cycles when they most likely could have reached higher displacements if the standard testing routine was followed. In addition, both of the 6 inch (152 mm) fuse connector plate tests are not representative of the fuse connector plate strength or maximum displacement because the shear-friction failure of the wall specimen prevented the fuse connectors from failing. As previously mentioned, these values will be used for test result comparisons to determine if testing two or more pairs of connector plates in series would provide approximately similar behavior to the tests previously performed with only one or two connector plates. Figure 5-17 shows the hysteretic backbone curves of the five test specimens to compare the hysteretic characteristics of each test specimen. The hysteretic backbone curve represents the maximum load achieved at the first of each different displacement cycle.
Figure 5-17: Hysteretic Backbone Curve Comparison
6 Analysis

6.1 Comparison of Expected Results vs. Experimental Results

The results obtained during testing of the connector plate and wall specimens provided valuable information and brought attention to vital aspects of hybrid masonry connections that will need to be addressed. In order to properly analyze the results found during these tests, comparisons will be made with previous testing results and theoretical results.

6.1.1 Masonry Wall Specimens

For the wall specimens, CMU through-bolt test results and theoretical values for wall strength will be compared to test results obtained from this research. During the experimental testing, both the partially grouted and fully grouted masonry wall specimens failed at approximately 36 kips (160 kN) in shear failure between the bottom course of the masonry wall panel and the concrete slab below. Although this was an unexpected failure, there is only a 4% difference between the theoretical nominal shear friction capacity and experimental result of the masonry-concrete shear friction capacity. Both the moment and shear capacity of the wall were not relevant to the failure mode of the wall because the shear friction failure controlled (i.e. was the smallest strength value) from the wall strength calculations. However, as previously discussed, this was only the case because the height at which the load was being transferred into the wall was relatively low compared to the wall’s length, resulting in a higher moment capacity than the typical wall used for hybrid masonry.

From the shear friction failure mode, it is clear that this type of failure should be designed for when designing hybrid masonry connections. One potential solution to this issue would be to roughen the top of the concrete layer before the wall panel is built to increase the friction coefficient at this joint. Another possible solution to increase the shear friction capacity would be to have more and/or larger vertical reinforcing bars to increase the shear friction reinforcing area. This would allow for the fuse connector plates to resist higher loads and (depending on the height of the wall) potentially cause the vertical bars to yield, resulting in a more ductile and less catastrophic failure mode.

Another failure mode which did not occur before the shear friction failure was masonry breakout at the location of the through-bolt connection. Previous bolt push out tests performed by Goodnight showed that when the through-bolts are located two cells away from the edge of the wall panel, approximately 20 kips would be required to cause a breakout failure. Due to the shear friction failure, the 6 inch (152 mm) fuse connector plates only reached a load of 18 kips per pair, which was not high enough to cause breakout failure. In addition, due to the relatively low breakout capacity from Goodnight’s testing, the through-bolts were placed three cells away from the edge of the wall panel. Therefore, the breakout capacity of the wall panel was most likely larger than the 20 kips obtained from Goodnight’s testing. Additional tests will be performed to obtain a better estimate of the breakout value with different edge distances and the proper testing restraints.
6.1.2 Tapered Fuse Connector Plates

For the fuse connector plates, previous test results on either single or double connector plates (shown in Table 6-1) will be compared to test results obtained from this phase of the research. Hysteretic value comparisons for the different test specimens are shown in Tables 6-2 through 6-4. There are many similarities that can be observed from the comparisons, but there are also some differences that stand out between previous test results and the recent test results. These similarities and differences will be described in detail for each specimen. In addition, hysteretic diagrams and vertical load diagrams will be compared for each specimen.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Yield Point</th>
<th>Maximum Load</th>
<th>Maximum Displacement (w/all fuses intact)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Displ. (in)</td>
<td>Load (k)</td>
<td>Displ. (in)</td>
</tr>
<tr>
<td></td>
<td>(+)</td>
<td>(-)</td>
<td>(+)</td>
</tr>
<tr>
<td>P4_T4_FT3*</td>
<td>0.17</td>
<td>1.6</td>
<td>1.5</td>
</tr>
<tr>
<td>(One pair)</td>
<td>6.4</td>
<td>6.4</td>
<td>12.8</td>
</tr>
<tr>
<td>(Two pairs)</td>
<td>9.6</td>
<td>9.0</td>
<td>19.2</td>
</tr>
<tr>
<td>P6_T4_FT2.4**</td>
<td>0.15</td>
<td>11.0</td>
<td>10.8</td>
</tr>
<tr>
<td>(Two pairs)</td>
<td>22.0</td>
<td>21.6</td>
<td>42.2</td>
</tr>
</tbody>
</table>

* Test was performed on a single connector plate [Goodnight, 2011]. Load values multiplied by 2 to represent a pair of connector plates, by 4 to represent two pairs of connector plates, and by 6 to represent three pairs of connector plates, respectively.

** Test was performed on a pair of connector plates [Ozaki-Train, 2011]. Values shown are for a pair of connector plates. Load values multiplied by 2 to represent two pairs of connector plates.

(1 inch = 25.4 mm, 1 kip = 4.45 kN)

Table 6-2: Prior Test vs. Experimental Yield Displacements and Loads

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Prior Test Yield Point</th>
<th>Experimental Yield Point</th>
<th>% Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Displ. (in)</td>
<td>Load (k)</td>
<td>Displ. (in)</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B4</td>
<td>0.17</td>
<td>6.4</td>
<td>0.18</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6</td>
<td>0.17</td>
<td>9.6</td>
<td>0.16</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6-RC</td>
<td>0.17</td>
<td>9.6</td>
<td>0.16</td>
</tr>
<tr>
<td>PHT1-P6_T4_FT2.4_B4</td>
<td>0.15</td>
<td>22.0</td>
<td>0.16</td>
</tr>
<tr>
<td>FHT1-P6_T4_FT2.4_B4</td>
<td>0.15</td>
<td>22.0</td>
<td>0.20</td>
</tr>
</tbody>
</table>
6.1.2.1 PHT1-P4_T4_FT3_B4

For this comparison, load results from Goodnight’s single connector tests on the 4 inch (102 mm) tapered fuse connector plate were multiplied by 4 to obtain values comparable to the two pairs of connector plates on the wall specimen. As shown in Tables 6-2 through 6-4, the load values for the yield point, maximum load, and maximum displacement load are all relatively similar to results obtained from Goodnight’s testing. This similarity in load verifies the assumption that this fuse connector has the ability to resist approximately the same amount of load when used as a series of multiple connector plates as it does as a single connector plate. It was concluded that the loads obtained from Goodnight’s single connector plate testing can simply be multiplied by the number of connector plates to determine the approximate load that multiple connector plates can resist. Similarly, the yield displacement for the single connector plate was approximately the same as the yield displacement of the series of connectors showing that multiple connectors possess the same or similar yielding properties as a single connector plate. This also verifies even distribution of load between all of the connector plates.
On the contrary, the comparison between the displacement at the maximum load and maximum displacement for the two different phases of testing produced fairly different results. While Goodnight’s single connector test showed that each connector could resist load for numerous cycles at ±3 inch (76 mm) displacement, the two pairs of connectors all failed prior to reaching the ±3 inch (76 mm) displacement cycle. This difference in the fuses’ displacement capacity could potentially be due to the connector plate fabrication method and/or a difference in the test setup.

While Goodnight fabricated the connector for the single connector plate testing by milling, all of the connectors in this phase of testing were fabricated by means of a plasma cutting table as discussed in Section 4.1.2. Despite the plasma cut edge of the fuses being ground slightly to eliminate the rough and brittle edge of the fuse, the extreme heat produced near the surface of the cut edge during cutting may have caused premature cracking earlier in the test leading to earlier failure. In addition, Goodnight’s tests showed the fuses failing in lateral torsional buckling. However, during this phase of testing, the masonry wall prevented the fuse connectors from this type of torsional buckling, which could have concentrated the load as shear, rather than torsion. In Goodnight’s tests, the torsional buckling of the fuse sections may have allowed the connector plates to accommodate the larger displacements because the applied shear load was split between both shear and torsion across the fuse section.

![Hysteretic diagram comparison for PHT1-P4_T4_FT3_B4](image)

**Figure 6-1: Hysteretic diagram comparison for PHT1-P4_T4_FT3_B4**

The similarities and differences between the two tests can also be seen in Figure 6-1, which shows a comparison between the two hysteretic diagrams. Similarly, vertical load diagrams are compared in Figure 6-2, which shows that vertical loads during testing were vastly different between the single connector plate test and wall
tests. This difference in vertical loads is due to the lateral torsional buckling resistance of the fuses during the wall tests, as discussed previously. During lateral torsional buckling of Goodnight’s single connector plate testing, the slotted hole of the connector plate tended to twist and cause the slotted hole to grip the bolted connection causing large vertical load transfer through the connector plate. During the wall tests, the lack of torsion near the bolt allowed the connectors to stay fairly straight allowing the connectors to move freely in the vertical direction. The lack of vertical loads found during the wall tests were a positive result because in Type I hybrid masonry, vertical load transfer to the wall should not occur. Also, this could potentially cause the masonry wall to fail quickly if the connector plates go into vertical tension due to masonry’s inability to carry tensile loads.

Figure 6-2: Vertical load diagram comparison for PHT1-P4_T4_FT3_B4

6.1.2.2 PHT1-P4_T4_FT3_B6

Similar to PHT1-P4_T4_FT3_B4, load results from Goodnight’s single connector tests on the 4 inch (102 mm) tapered fuse connector plate were multiplied by 6 to obtain values comparable to the three pairs of connector plates on the wall specimen. The comparisons between the single connector plate test results and the wall test results are very similar to PHT1-P4_T4_FT3_B4. The yield displacement, yield load, and maximum load were very similar, showing even load distribution between the connector plates and the ability of multiple connector plates to carry approximately the same amount of maximum load as a single connector plate multiplied by the number of connector plates being loaded. One discrepancy in the comparison was the difference in load at maximum displacement, which occurred due to a fabrication error as
discussed in Section 5.1.2.2 of this report. However, the fabrication error led to an observation in the results that is interesting to note.

Due to the premature crack which formed because of a fabrication error in one of the fuses, this fuse failed earlier during the testing routine than the other connector plates. However, the five remaining plates continued to perform, resisting displacements and loads in a similar manner to how they were performing previous to the premature failure of the flawed connector. While the load dropped after the failure of the connector, this result shows that even if there is a premature failure of one of the connector plates, the other connector plates in the series will continue to perform as expected and can continue to displace and resist loads as they are designed for.

The differences in displacements allowed by the single connector plate tests and the wall tests were similar to PHT-P4_T4_FT3_B4 as well. The reasons for these differences are suspected to be due to fabrication method and test setup, as discussed in the previous section of the report. The vertical load in the connector plates also differed between the two phases of testing in a similar manner to the previous test. Hysteretic diagram and vertical load diagram comparisons are shown in Figures 6-3 and 6-4, respectively.

![Hysteretic diagram comparison for PHT1-P4_T4_FT3_B6](image)

**Figure 6-3: Hysteretic diagram comparison for PHT1-P4_T4_FT3_B6**
6.1.2.3 PHT1-P4_T4_FT3_B6-RC

Similar to PHT1-P4_T4FT3_B4, load results from Goodnight’s single connector tests on the 4 inch (102 mm) tapered fuse connector plate were multiplied by 6 to obtain values comparable to the three pairs of connector plates on the wall specimen. This comparison was mainly performed to determine the similarities between tests for the load resistance of the connector plates. As shown in Table 6-2, the yield displacement and yield load between the two tests are very similar. Also, as shown in Tables 6-3 and 6-4, the maximum load occurred at the maximum displacement, resulting in maximum load values that are relatively similar to the single connector plate tests. The displacement values between the two tests were expected to be different for this specimen because of the rapid cycling of the fuses.

The displacement at the maximum load and maximum displacement were the same, meaning that the load resistance of the connectors did not begin to drop before the connectors failed. As expected, the maximum displacement achieved was much lower than the single connector plate tests due to the 30 rapid cycles each at approximately 0.5% and 1.0% drift. The early failure of the fuses due to the rapid cycles shows that fatigue does have an effect on the amount of displacement that the fuse connectors can achieve. Although the previously discussed plasma cut preparation and test setup could be potential causes for the premature failure, it is much more likely that the fuses failed due to fatigue. However, despite the early failure of the fuses during this test, the fuses were able to perform for numerous cycles past their yield point, which is a positive result.

The vertical load in the connector plates also differed between the two phases of testing in a similar manner to the previous tests. Hysteretic diagram and vertical load diagram comparisons are shown in Figures 6-5 and 6-6.
Figure 6-5: Hysteretic diagram comparison for PHT1-P4_T4_FT3_B6-RC

Figure 6-6: Vertical load diagram comparison for PHT1-P4_T4_FT3_B6-RC
6.1.2.4 PHT1-P6_T4_FT2.4_B4 and FHT1-P6_T4_FT2.4_B4

For this comparison, load results from Ozaki-Train’s double connector tests on the 6 inch (152 mm) tapered fuse connector plate were multiplied by 2 to obtain values comparable to the two pairs of connector plates on the wall specimen. As shown in Tables 6-2 through 6-4, the load value for the yield point and yield displacement are both relatively similar to results obtained from Ozaki-Train’s testing. The yield displacement for FHT1-P6_T4_FT2.4_B4 was somewhat different from Ozaki-Train’s results, but the yield load is similar, so this could be a result of a difference in test setups. As previously mentioned, this shows that the fuse connector plates are resisting the shear loads equally, allowing all plates to yield at approximately the same time across the entire fuse section of the plates.

Due to the failure of the wall specimen prior to the failure of the connector plate, the maximum displacement and maximum load comparisons are somewhat irrelevant. However, despite the failure of the wall specimen, the fuses reached a maximum load that was only 16% (for the partially grouted wall specimen) and 11% (for the fully grouted wall specimen) less than the load achieved during Ozaki-Train’s double connector testing. This shows that the fuse was properly transferring shear loads into the wall specimen and had the wall been strong enough to resist these forces, the connector plates would have most likely had a similar behavior to the 4 inch (102 mm) tapered fuse tests. In addition, the displacement values obtained from the wall tests are for the displacement of the connector plates, which are significantly smaller than the double connector plate tests due to the shear friction sliding of the masonry specimen.

The vertical load in the connector plates also differed between the two phases of testing in a similar manner to the previous tests. Hysteretic diagram and vertical load diagram comparisons are shown in Figures 6-7 through 6-10. An interesting comparison is shown in Figure 6-9, which compares the hysteretic diagrams for the double connector plate testing, partially grouted wall specimen, and fully grouted wall specimen. Unfortunately, the fully grouted wall specimen did not resist much more load than the partially grouted wall specimen, as their hysteretic diagrams were very similar. This is because shear friction is dependent on the area of the vertical reinforcing in the wall which was identical for the two wall specimens.
Figure 6-7: Hysteretic diagram comparison for PHT1-P6_T4_FT2.4_B4

Figure 6-8: Vertical load diagram comparison for PHT1-P6_T4_FT2.4_B4
Figure 6-9: Hysteretic diagram comparison for FHT1-P6_T4_FT2.4_B4

Figure 6-10: Vertical load diagram comparison for FHT1-P6_T4_FT2.4_B4
6.2 Fuse Connector Energy Dissipation

One of the major advantages to using fuse connector plates rather than link connector plates is their ability to dissipate energy in the structural system. This allows the system to resist larger displacements and ultimately adds ductility to the system. In order to measure the ductility of the connector plates on the wall specimens during this phase of testing, the cumulative displacement resisted by each set of connector plates and the energy dissipated during the testing procedure were calculated.

Cumulative displacement was calculated by evaluating the change in actuator displacement, Δa. In order to filter out signal “noise” inherent in collecting sensor data, the data was examined and it was determined that ±0.0005 inch (0.13 mm) was an appropriate value to represent this displacement signal “noise”. If |Δa| > 0.0005 inch (0.13 mm), then the cumulative displacement was incremented from the change in the displacement potentiometer measurement as follows:

\[ \delta_{\text{cumulative(new)}} = \delta_{\text{cumulative(old)}} + |\Delta a| \]

Energy dissipated (EDIS) was calculated as the approximate area enclosed by the load-displacement curve. The following equation shows this calculation:

\[ EDIS = \Delta a \times \frac{\text{Current Load} + \text{Previous Load}}{2} \]

If |Δa| > 0.0005 inch (0.13 mm), then the cumulative energy dissipated was incremented:

\[ \text{EDIS}_{\text{cumulative(new)}} = \text{EDIS}_{\text{cumulative(old)}} + \text{EDIS} \]

Table 6-5 compares the total cumulative displacement and energy dissipated for each of the specimens tested.
Table 6-5: Energy Dissipation Behavior of Side plate Connectors

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Cumulative Displacement (in)</th>
<th>Energy Dissipated (k-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PHT1-P4_T4_FT3_B4</td>
<td>159.7</td>
<td>960.0</td>
</tr>
<tr>
<td>(per pair)</td>
<td></td>
<td>480.0</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6*</td>
<td>182.6</td>
<td>1360.7</td>
</tr>
<tr>
<td>(per pair)</td>
<td></td>
<td>453.6</td>
</tr>
<tr>
<td>PHT1-P4_T4_FT3_B6-RC***</td>
<td>310.3</td>
<td>1995.3</td>
</tr>
<tr>
<td>(per pair)</td>
<td></td>
<td>665.1</td>
</tr>
<tr>
<td>PHT1-P6_T4_FT2.4_B4***</td>
<td>60.8</td>
<td>514.6</td>
</tr>
<tr>
<td>(per pair)</td>
<td></td>
<td>257.3</td>
</tr>
<tr>
<td>FHT1-P6_T4_FT2.4_B4***</td>
<td>80.6</td>
<td>854.5</td>
</tr>
<tr>
<td>(per pair)</td>
<td></td>
<td>427.3</td>
</tr>
</tbody>
</table>

* Early failure of one fuse due to fabrication error
** Rapid cycling
*** Limited by failure of wall specimen; fuse connector plates still intact after testing completed

(1 in = 25.4 mm, 1 k-in = 113 J)

6.2.1 P4_T4_FT3

The 4 inch (102 mm) tapered fuse connector tests verified that these fuse connector plates can dissipate large amounts of energy. PHT1-P4_T4_FT3_B4 achieved a cumulative displacement of approximately 160 inches (4,064 mm), while dissipating 960 kip-inch (108,480 Joules) or 480 kip-inch (54,240 J) per pair of connectors. Similarly, PHT1-P4_T4_FT3_B6 achieved a cumulative displacement of approximately 180 inches (4,572 mm), while dissipating 1,360 kip-inch (153,680 Joules) or 450 kip-inch (50,850 Joules) per pair of connectors. Interestingly, the rapid cycling at lower displacements along with the regular testing procedure of PHT1-P4_T4_FT3_B6-RC achieved a higher cumulative displacement of approximately 310 inches (7,874 mm), while dissipating approximately 2,000 kip-inches (226,000 Joules) or 666 kip-inches (75,258 Joules) per pair of connectors. This result shows that the fuse connector plates have the potential to dissipate even larger amounts of energy if the displacements that they absorb are limited. All of the cumulative displacements and energy dissipated values achieved by these fuse connector tests are significantly lower than that achieved during Goodnight’s single connector test. However, as discussed earlier, Goodnight’s single connector was able to cycle numerous times at ±3 inch (76 mm) displacement, while these connectors failed before reaching this displacement level. The values obtained during this phase of testing are much more realistic values for what can be expected in real structural applications because the test setup and fabrication method are similar to what is expected to be used in the construction industry.

6.2.2 P6_T4_FT2.4

During Ozaki-Train’s testing of the 6 inch (152 mm) tapered fuse double connector test, these fuse connector plates displayed the potential to dissipate large amounts of
displacements and energy dissipated. During this phase of testing, however, due to failure of the wall specimens, the connector plates were not able to displace to their full potential. This failure of the wall specimen led to very low cumulative displacement and dissipated energy values, which should not be taken as representative for these connector plates. The main cause of the lack of energy dissipated was the sliding of the masonry on the concrete slab causing the connector plates to reach a maximum displacement of only 1 inch (25 mm).
7 Design Procedure

Based on the hybrid masonry connector development research performed to date, the following is the recommended procedure for designing a Type I hybrid masonry fuse connector plate:

1) Perform structural analysis of the building system subjected to all combinations of factored loads to determine the shear demand at each hybrid masonry shear wall.

2) Determine the maximum number of connectors allowed ($n_{\text{max}}$) based on minimum through bolt edge distance of 20 inches (508 mm) and spacing of 24 inches (610 mm).

3) Verify that the ultimate shear load into the wall is not greater than the masonry breakout capacity for a through bolted connection:

$$V_u \leq (0.65)(20 \text{ k})(n_{\text{max}})$$

If not, then either more Type I hybrid masonry shear walls must be added to the lateral resisting system or a Type II or III hybrid masonry shear wall system must be used.

4) Determine the vertical distance from the through bolt to the lower slip critical bolts or bottom of weld attaching the connector plate to the side plate, $L_b$, as shown in Figure 7-1.
5) Select a trial connector plate thickness, \( t \), not less than \( \frac{1}{2} \)-inch.

6) Based on \( L_b \), \( L_{top} \) should be selected such that:

\[
L_b - L_{top} \geq 2 \text{ inches (51 mm)}
\]

This is a conservative minimum to ensure that the connection to the side-plate does not become the weak point for the connector. Likewise, \( L_{bottom} \) should be selected such that the slotted hole does not become a weak point for the connector:

\[
L_{bottom} \geq 2.5 \text{ inches (63 mm)}.
\]

7) Determine \( d_{top} \) from the following;

\[
V_n = F_u S_x = (F_u) \frac{tb_t^2}{6}
\]
If \( d_{top} \) is greater than 6 inches (152 mm), then select a larger fuse connector thickness, \( t \), and repeat steps 6 and 7.

8) Determine \( d_{bottom} \) from the following:

\[
\frac{d_{top}}{d_{bottom}} = \frac{\sqrt{L_{top}}}{\sqrt{L_{bottom}}}
\]

9) Select a plate width to suit the fuse dimension from the following:

\( w_{min} = d_{top} + 1" \)

10) If a slip critical bolted connection is used to secure the connector plate to the side plate, the slip critical bolted connection must be designed with the seismic over strength factor, \( \Omega \), and the dimensional factors as shown in Figure 7-1:

\[
\phi V_1 L_a \geq \Omega V_n (L_a + L_b)
\]

\[
\phi V_1 \geq \Omega V_n \left(1 + \frac{L_b}{L_a}\right)
\]

11) Given the factored in-plane shear demand, verify that the fuse or link connectors can resist the shear from the following:

\[
V_u \leq \phi n V_n
\]

Where,

\( \Phi = 0.9 \)

\( n = \) Number of pairs of fuse or link connectors

\( V_n = \) Nominal shear capacity of one pair of fuse or link connectors

\( = 0.6F_{uA} \) (Check buckling limits and shear \( V_n \))

12) Verify that the connector plates can resist out-of-plane wind and seismic loads.
8 Conclusions / Recommendations

The following conclusions and recommendations are presented for Type I hybrid masonry wall specimens and fuse connectors, based on the testing performed in this project.

- Tapered fuse connectors were successful at transferring cyclic lateral load between the steel beam and masonry wall components of the Type I hybrid masonry system. However, if the fuse capacity exceeds that of the masonry wall, then premature masonry failure may reduce the effectiveness of the Type I fuse system.

- The tapered fuses used in this study were able to dissipate large amounts of seismic energy before failure. Provided the masonry wall is designed appropriately, the replaceable fuse connectors should not damage the masonry wall panels of the structure. The damaged connectors can then be replaced, allowing for relatively quick and economical repair after a seismic event.

- In order for the fuse connectors to serve as the seismic fuse, the masonry wall specimen must be designed appropriately to resist bending, shear, or shear friction failure. Current masonry code guidelines do not provide design capacities for shear friction.
  
  o Additional tests are recommended on masonry wall specimens in which the concrete slab is roughened prior to construction of the masonry wall panel above to increase the shear friction capacity at this joint.

- During prior testing of single plate fuses and connectors, lateral torsional buckling lead to friction between the fuse and the through bolt. This resulted in large vertical forces developing in the fuse. However, during the masonry wall panel tests, the presence of the wall limited the extent of fuse buckling, thereby reducing the vertical load transferred by friction.

- The load transfer capacity of multiple pairs of fuse connector plates through bolted on a single masonry wall panel can be approximated by designing a single connector plate and multiplying the single plate’s capacity by the number of connector plates on the wall panel. However, the individual fuse connectors were able to survive larger lateral displacement cycles than those in the wall test. This may be the result of different fabrication techniques used for the fuses, or the lateral torsional buckling of the individual plates.

  o Additional testing is recommended for fuse connectors to determine the effects of a plasma-cut fuse vs. a milled fuse and fuse behavior during testing when lateral-torsional buckling is allowed vs. when lateral-torsional buckling is restrained.
• Each fuse connector should be inspected for any type of fabrication error that could potentially lead to a premature failure of the fuse and a decrease of the system’s lateral resistance capacity. However, if a fuse connector does fail prematurely, the remaining fuse connectors in the system will continue to resist their respective loads and behave similarly until failure.

• The test specimen subjected to multiple high frequency post-yield cycles was able to absorb more energy than the equivalent specimen subjected to increasing static displacement cycles. However, it did not achieve the same ultimate ductility as the static test specimen.
9 References


APPENDICES
A1: Graphic Representation of Test Setup
A2: DTI Washers Product Specification

Fastenal Product Standard: FNL.DTI.A490.SQRT.P

Direct Tension Indicator, Type A490, Squirter®, 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.

<table>
<thead>
<tr>
<th>Size</th>
<th>Inside Diameter</th>
<th>Protr. Tang Dia.</th>
<th>Outside Dia.</th>
<th>Number of Protrusions (Equally Spaced)</th>
<th>Thickness Without Protrusion</th>
<th>Thickness With Protrusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2</td>
<td>.523</td>
<td>.527</td>
<td>.788</td>
<td>1.355</td>
<td>1.375</td>
<td>5</td>
</tr>
<tr>
<td>5/8</td>
<td>.654</td>
<td>.658</td>
<td>.956</td>
<td>1.605</td>
<td>1.625</td>
<td>5</td>
</tr>
<tr>
<td>3/4</td>
<td>.786</td>
<td>.790</td>
<td>1.125</td>
<td>1.730</td>
<td>1.750</td>
<td>6</td>
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<td>7/8</td>
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<td>.921</td>
<td>1.294</td>
<td>1.980</td>
<td>2.000</td>
<td>6</td>
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<tr>
<td>1</td>
<td>1.048</td>
<td>1.052</td>
<td>1.463</td>
<td>2.230</td>
<td>2.250</td>
<td>7</td>
</tr>
<tr>
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<td>1.183</td>
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<td>2.500</td>
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<tr>
<td>1 1/4</td>
<td>1.311</td>
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<td>1.800</td>
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<td>2.750</td>
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<td>1 3/8</td>
<td>1.442</td>
<td>1.446</td>
<td>1.969</td>
<td>2.980</td>
<td>3.000</td>
<td>8</td>
</tr>
<tr>
<td>1 1/2</td>
<td>1.573</td>
<td>1.577</td>
<td>2.138</td>
<td>3.230</td>
<td>3.250</td>
<td>9</td>
</tr>
</tbody>
</table>

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.

Page 1 of 1

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October 28, 2008
A3: Tension Test Stress-Strain Curves

½” Plate (from 4” wide stock) [Ozaki-Train et al, 2011]

![Stress-Strain Curve](image)

Avg width (in): 1.0007
Avg thickness (in): 0.4938
Cross-section area (in^2): 0.4941

Final elongation (in): 10.09375
% elongation: 26.172

Approximated Values:

<table>
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<tr>
<th></th>
<th>Strain</th>
<th>Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield</td>
<td>0.00168</td>
<td>53.447</td>
</tr>
<tr>
<td>Ultimate</td>
<td>0.16771</td>
<td>83.293</td>
</tr>
<tr>
<td>Failure</td>
<td>0.23785</td>
<td>63.491</td>
</tr>
</tbody>
</table>

E ~ 32,000 ksi
$\frac{1}{2}''$ Plate (from 6'' wide stock) [Ozaki-Train et al, 2011]

Avg width (in): 0.9977
Avg thickness (in): 0.4992
Cross-section area (in$^2$): 0.4980

Final elongation (in): 10.0625
% elongation: 25.781

Approximated Values:

<table>
<thead>
<tr>
<th></th>
<th>Strain</th>
<th>Stress (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield</td>
<td>0.00178</td>
<td>53.076</td>
</tr>
<tr>
<td>Ultimate</td>
<td>0.16679</td>
<td>80.696</td>
</tr>
<tr>
<td>Failure</td>
<td>0.23369</td>
<td>61.575</td>
</tr>
</tbody>
</table>

E ~ 31,000 ksi
#5 Reinforcing Bar Specimens

Bar Diameter (in): 0.625
Cross-Sectional Area (in.$^2$): 0.3068
### SHEAR STRENGTH OF PARTIALLY GROUTED CMU WALL SPECIMEN

Given:
- $f_m' = 1,540$ PSI
- $f_y = 68,000$ PSI
- Wall thickness = 7.625 in.
- Vert. Reinf. = 4 - #5 bars; $A_s = 1.227$ sq. in.
- Wall height = 2.67 ft.
- Horiz. Reinf. = 1 - #3 bar; $A_s = 0.110$ sq. in.
- Wall length = 6.67 ft.
- Dead Load = Weight of CMU = (135 PCF)(8/12 ft.)(2.67 ft.)(6.67 ft.) = 1,600 lbs. = 1.6 kips

#### 1) Moment Design

**A)** Nominal Axial Load, $P_o$

$$A_e = (Wall \ thickness)(Wall \ length) = 610.31 \text{ sq. in.}$$

$$P_o = \frac{[0.85(f_m')(A_e - A_s) + f_y(A_s)]}{1000} = 83.97 \text{ kips}$$

**B)** Factored Axial Load, $P_u$

$$P_u = 1.92 \text{ kips}$$

Check $P_u \leq \phi (0.8)P_o$

$$1.92 \text{ kips} \leq (0.65)(0.8)(P_o) = 43.66 \text{ kips}$$

**C)** Nominal Moment Strength, $M_o$

Assumed location for $NA$; $c = 7.38$ in.

Max. allowable CMU strain = 0.003

Depth of compression stress block = $a = 0.85c = 6.27$ in.

$x = 0.5a = 3.14$ in.

Tension force = $T = (A_s)(f_y) = 62.59 \text{ kips}$

Compression force = $C = (A_s)(f_s)+0.85(f_m')(b)(a) = 62.59 \text{ kips}$

Force difference (Goal Seek to equal 0) = $T-C = 0 \text{ kips}$

Sum of moments about left edge of wall:

$$M_o = T(Distance) - C(Distance) = 3,023.64 \text{ kip-in.} = 251.97 \text{ kip-ft.}$$

$$M_u = \phi M_o = (0.90)(M_o) = 226.77 \text{ kip-ft.}$$

$$V_u = \frac{M_o}{(1.67 \text{ ft.})} = 151.18 \text{ kips}$$

(Does not govern)

#### 2) Shear Design

**A)** Shear Of Masonry

$$M_u/(V_d) = 0.26$$

**B)** Total Shear

$$V_n = V_u = 38.86 \text{ kips}$$

(Does not govern)

#### 3) Shear Friction

$$A_v = 1.23 \text{ sq. in.}$$

$$\mu = 0.0.6 \times (0.75) = 0.45$$

$$V_n = (A_v)(\mu) = 37.55 \text{ kips}$$

<--------- SHEAR FRICTION FAILURE GOVERNS
A5: Fully Grouted Masonry Wall Bending Moment, Shear, and Shear Friction Capacity Calculations Spreadsheet

### SHEAR STRENGTH OF FULLY GROUTED CMU WALL SPECIMEN

**Given:**
- $f_{m'} = 1,770$ PSI
- $f_y = 68,000$ PSI
- Wall thickness = 7.625 in.
- Vert. Reinf. = 4 - #5 bars; $A_s = 1.227$ sq. in.
- Wall height = 2.67 ft.
- Horiz. Reinf. = 1 - #3 bar; $A_s = 0.110$ sq. in.
- Wall length = 6.67 ft.

**Dead Load**
- Weight of CMU = (135 PCF)(8/12 ft.)(2.67 ft.)(6.67 ft.) = 1,600 lbs. = 1.6 kips

**Dead Load Factor** = 1.2
- Factored Dead Load = (1.2)(Dead Load) = 1.92 kips

**Seismic Load Factor** = 1.0

1) **Moment Design**

A) **Nominal Axial Load, $P_o$**
- $A_e = (Wall\ thickness)(Wall\ length) = 610.31$ sq. in.
- $P_o = \left[0.85(f_{m'})(A_e-A_s)+(f_y)(A_s)\right]/1000 = 83.97$ kips

B) **Factored Axial Load, $P_u$**
- $P_u = 1.92$ kips
- Check $P_u \leq \phi(0.8)P_o$
  - \(\text{OK}\)

C) **Nominal Moment Strength, $M_o$**
- Assumed location for NA; $c = 6.42$ in.
- Max. allowable CMU strain = 0.003
- Depth of compression stress block = $a = 0.85c = 5.46$ in.
- $x = 0.5a = 2.73$ in.
- Tension force = $T = (A_s)(f_y) = 62.59$ kips
- Compression force = $C = (A_s)(f_{s'})+0.85(f_{m'})(b)(a) = 62.59$ kips
- Force difference (Goal Seek to equal 0) = $T-C = 0$ kips
- Sum of moments about left edge of wall:
  - $M_o = T(Distance) - C(Distance) = 3,053.64$ kip-in. = 254.47 kip-ft.
- $M_u = \phi M_o = (0.90)(M_o) = 229.02$ kip-ft.
- $V_u = (M_o)/(1.67 ft.) = 152.68$ kips (Does not govern)

D) **Check Cracking Moment**
- Section modulus = $S = 8,141.47$ cub. in.
- $f_r = 4(f_{m'}^{1/2}) = 168.29$ PSI
- $M_{cr} = S(P/A+f_r) = 1,391.43$ kip-in. = 254.47 kip-ft.

2) **Shear Design**

A) **Shear Of Masonry**
- $M_v/(V_d) = 0.26$
- $A_{mv} = (Wall\ thickness)(Wall\ length) = 610.31$ sq. in.
- $C_v = 1.6(1-M/(V_d)+1.2 = 2.38$
- $V_n = [(C_v)(A_{mv})(f_{m'}^{1/2})]/1000 = 61.09$ kips

Note: Ignore the shear strength of the horizontal shear reinforcement because the location of the horizontal rebar in the wall is outside the shear plane

B) **Total Shear**
- $V_v = V_n = 61.09$ kips (Does not govern)
- $V_v = \phi V_n = (0.8)V_n = 48.87$ kips

3) **Shear Friction**
- $A_{vf} = 1.23$ sq. in.
- $\mu = 0.6a = (0.6)(0.75) = 0.45$
- $V_u = (A_{vf})(f_{y})\mu = 37.55$ kips

---

*\(\text{SHEAR FRICTION FAILURE GOVERNS}\)*
A5: Vertical Load vs. Shear Load Hysteretic Plots
A6: Time vs. LVDT Displacement Plots
A7: LVDT Displacement Hysteretic Plots
A8: String Potentiometer Displacement Plots