LARGE-SCALE TESTING OF HYBRID MASONRY

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ABSTRACT

The hybrid masonry lateral-force resisting system is comprised of masonry panels within a steel frame. This recently-introduced system is currently the subject of a comprehensive NSF-sponsored research program investigating seismic behavior, design and performance. This paper is concerned with large-scale experiments that are being conducted at the University of Illinois at Urbana-Champaign to demonstrate the efficacy of hybrid masonry for resisting seismic loads. Two-story one-bay configurations are being tested in the George E. Brown, Jr. Network for Earthquake Engineering Simulation Multi-Axial Full-Scale Sub-Structured Testing and Simulation facility. Initial results indicate that hybrid masonry is a viable option for seismic regions in addition to its current use in wind-dominated regions. Different connection methods between the masonry panels and steel frame were studied to explore the effect on hysteretic behavior. Steel energy dissipating fuses and link plates designed to focus inelasticity in the masonry panels were used to transmit forces from steel to masonry. Future testing will involve shear studs to transfer forces at steel-masonry interfaces. This experimental data will be used along with numerical simulations of hybrid masonry system seismic response to verify the proposed design and analysis methodology. This paper presents the experimental background and methodology and initial findings from the large-scale tests.

KEYWORDS: hybrid masonry, large-scale testing, seismic, lateral forces, reinforced concrete masonry, steel frames

INTRODUCTION

Seismic structural engineering requires efficiency, robustness and ductility for building design. A relatively new approach for meeting these goals is hybrid masonry, a system that takes advantage of the typically unused ductility and lateral strength of reinforced masonry panels set within steel frame structural systems. In typical construction practice, exterior masonry cavity walls as well as interior masonry partition walls are only utilized as architectural elements in the building system. In such a system, steel bracing can often interfere with the placement of masonry, creating more complicated construction conditions. The basic idea behind hybrid masonry is to use reinforced masonry panels to replace steel braces and resist lateral forces. Several connection types between masonry and steel provide the design engineer freedom to tailor the strength and ductility of the system. Type I hybrid masonry uses steel plates to transfer the lateral forces from the steel frame to the masonry. Type II hybrid masonry uses shear studs on the underside of the beams to transfer the lateral forces. Shear studs on the beams and the columns in Type III hybrid masonry transfer forces in the horizontal and vertical directions between the masonry and steel.

Simple representations of the different types compared to conventional masonry infill are shown in Figure 1, Figure 2, Figure 3 and Figure 4.

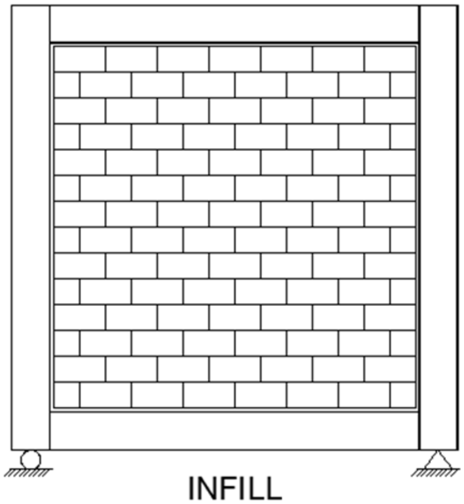


Figure 1: Generic steel frame with masonry infill, no gaps

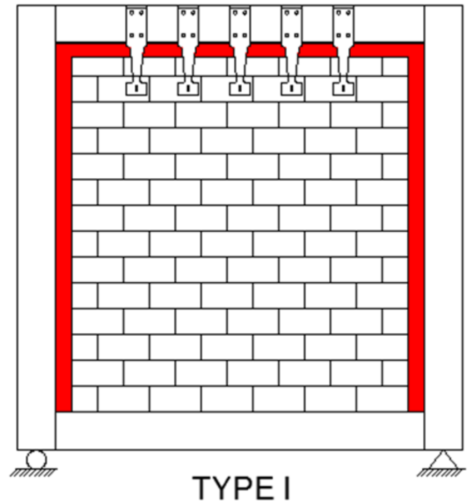


Figure 2: Generic steel frame with Type I hybrid masonry, gaps shown in red

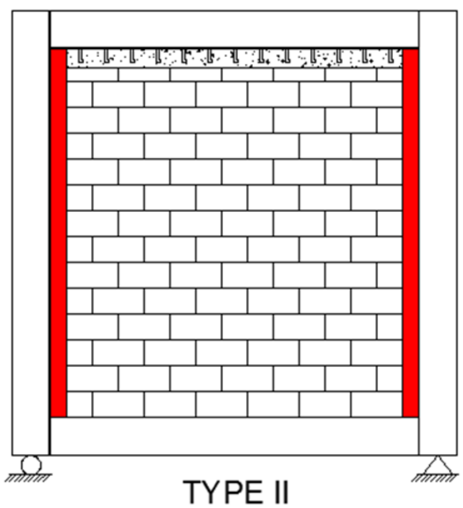


Figure 3: Generic steel frame with Type II hybrid masonry, gaps shown in red

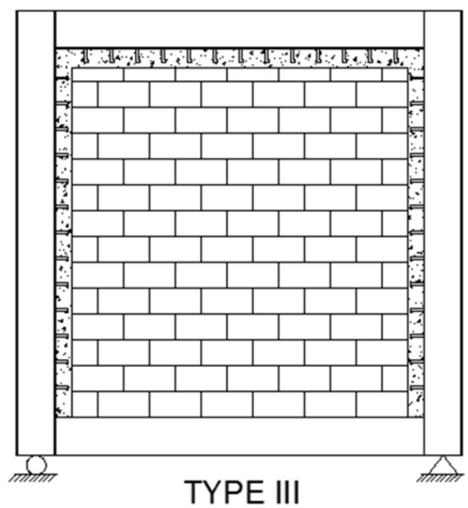


Figure 4: Generic steel frame with Type III hybrid masonry, no gaps

In addition to the three types of steel-to-masonry connections, two types of reinforcement anchorage between the bottom of a masonry panel and the supporting beam have been proposed [1]. When "a" is added to the hybrid masonry type designation, this indicates that the steel reinforcement has been fully developed and anchored at the base of the masonry panel. Conversely, "b" indicates that the reinforcement is not anchored. A basic summary of hybrid masonry load paths is provided in NCMA Tek 14-09A [1].

With the development and implementation of hybrid masonry as a lateral force resisting system for low seismic regions, extension to high seismic regions is a logical next step for exploration. As related papers describe [2–5], hybrid masonry is a relatively new system and current work is examining it extensively. Building off of feasibility studies, exploratory designs, and component testing, a large-scale experimental program was developed and is in progress at the University of Illinois at Urbana-Champaign. This paper presents the initial results for this testing program.

EXPERIMENTAL PROGRAM

Tests are being conducted at the Multi-Axial Full-Scale Sub-Structured Testing and Simulation (MUST-SIM) facility which is part of the George E. Brown, Jr. Network for Earthquake Engineering Simulation (NEES). A strong floor and 30 foot (9.1m) tall L-shaped reaction wall allow for a two-story, one-bay, full-scale hybrid masonry wall system to be tested. Loading and Boundary Condition Boxes (LBCBs) allow for 6 degrees of freedom to be controlled with mixed-mode displacement and force commands. Quasi-static loading protocols are being used, with constant vertical load to represent gravity effects and cyclic horizontal displacement imposed to simulate seismic demands. Instrumentation for the large-scale experimental setup consists of a K-600 dynamic coordinate measuring system with 132 LED targets, 116 strain gages, 49 linear transducers and 7 digital cameras.

As shown in Table 1, the three tests that have been conducted to date have panel height-to-length aspect ratios of 1.0. Both steel fuses and strong connector plates (link plates) have been explored for Type I. The same masonry panels were used for Tests 1 to 3. Figure 5 shows the test specimen before the first test, which employed the smaller fuse size. Both fuse sizes are designed to reach peak strength before the masonry panel experiences significant inelastic behavior. Link plates are designed to be stronger than the masonry panels so that inelastic behavior is focused in the masonry panels. An in-depth explanation of the fuses and link plates can be found in [6].

Table 1: Test Matrix

Test	h/L	# of LBCBs	Hybrid Type	Steel/Masonry Connection
1	1	1	I	4” (15.2 cm) Fuses
2	1	1	I	6” (10.2 cm) Fuses
3	1	1	I	Link Plates

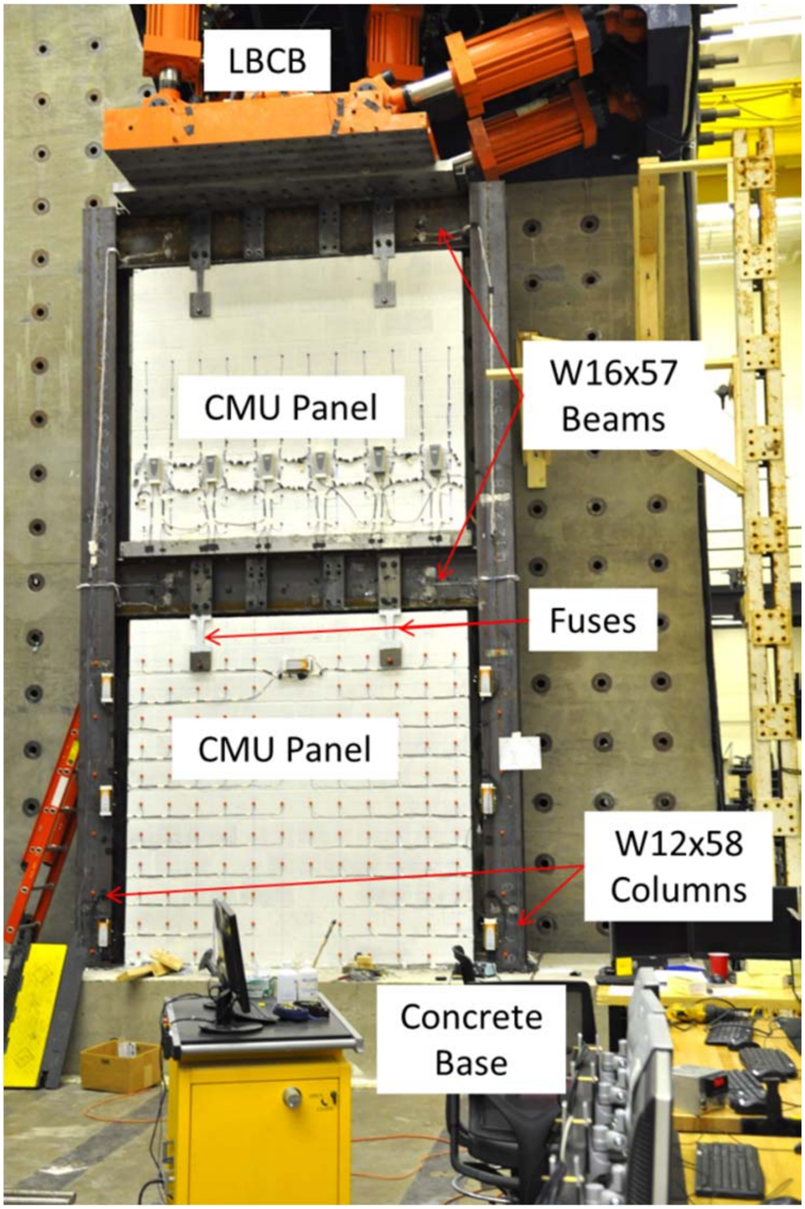


Figure 5: Large-scale hybrid masonry test specimen

Preliminary testing was conducted in the ⅕-scale laboratory that is part of the MUST-SIM facility. These tests were designed to verify experimental procedures and loading protocols while using a similar but less expensive small-scale specimen. The small-scale laboratory consists of a 70 in (178cm) tall aluminum strong wall that is L-shaped in plan. Loading is applied with small-scale LBCBs that have a force capacity in the lateral direction of ±2 kips (8.9kN) and displacement capacity that is 20% of the large LBCB displacements. The small-scale test specimen (Figure 6) consisted of components similar to those planned for the large-scale experiments. Although the fuse plates in this test were expected to yield, the first story masonry panel failed in shear due to over-strength in the fuses (Figure 7).

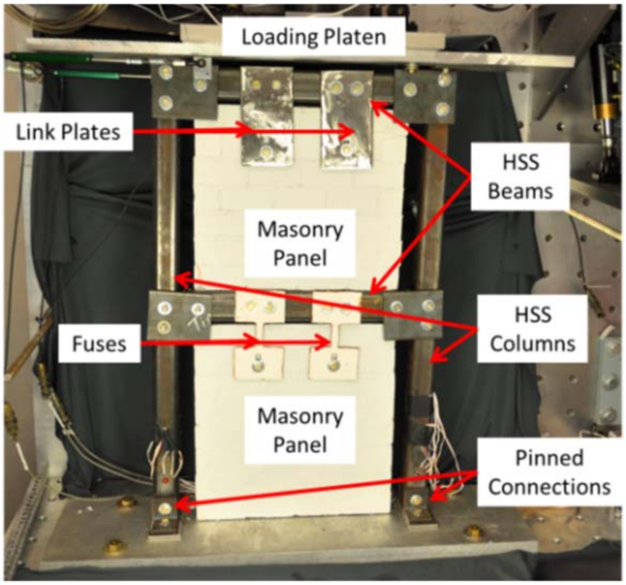


Figure 6: Small-scale hybrid masonry test specimen

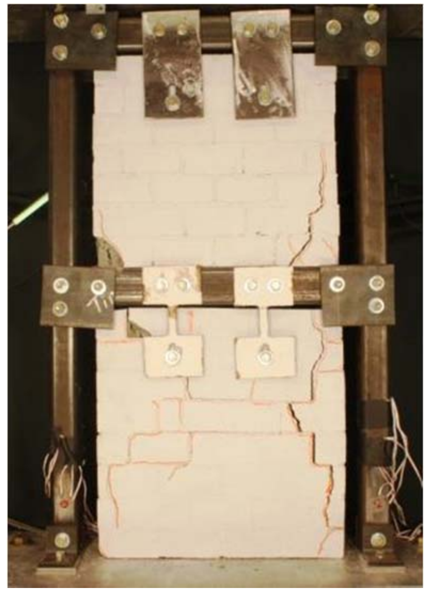


Figure 7: Masonry shear failure mechanism in small-scale specimen

TEST SPECIMENS

The test specimens were designed using specially reinforced masonry panels within typical steel framing. Shear connections, which have high rotation capacity and low moment capacity, were chosen for the steel beam-column joints. Since primary energy dissipation is intended to occur in fuse plates or masonry panels, the steel frame was designed to remain nominally in the elastic range. Steel section sizes were chosen conservatively so that members could be used for multiple test specimens. Overall dimensions were chosen to maximize the test specimen size within the laboratory constraints. The test specimens were designed using current-practice documents: the American Institute of Steel Construction (AISC) *Specification for Structural Steel Buildings* [7] and the Masonry Standards Joint Committee's (MSJC) *Building Code Requirements and Specification for Masonry Structures* [8]. For a more detailed description of load paths and design methodology, see Gregor et al. [9].

W12x58 columns and W16x57 beams were selected for all test specimens. Double angle connections consisting of 2L4x4x½ connect the beams to columns, providing sufficient strength and ample end rotation in the beam to approximate a pinned-end condition. Beams and columns are ASTM A992 steel and angles are ASTM A36 steel. Fully grouted 8 in (20cm) concrete masonry blocks are used for the masonry panels. The first story panel consists of 12 courses of blocks, with 6 blocks in each course. Through reinforcement sizing and placement, the walls conform to the MSJC Code provisions for specially reinforced masonry under seismic design categories E and F. Uncoated deformed #5 bars (ASTM A615 Grade 60 steel) are used for both vertical and horizontal reinforcement. Four bars are used in the vertical direction and three in the horizontal direction in the first story panel. The second story panel has double the number of vertical and horizontal bars to force the majority of damage into the first story masonry panel. A drawing of the specimen is shown in Figure 8. Masonry properties from ancillary tests are summarized in Table 2.

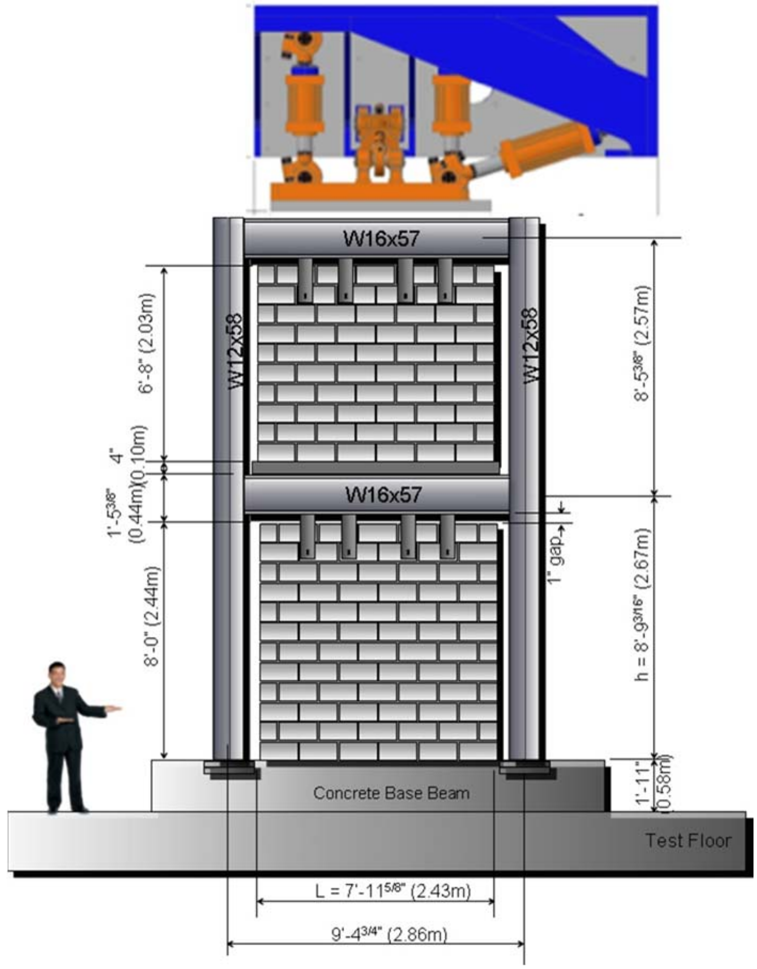


Figure 8. Elevation of large-scale hybrid masonry test specimen

Table 2: Material Properties

	F'_m (psi/MPa)	Grout f'_g (psi/MPa)
Block Strength	3400/23	-
Ungrouted Prism	3200/22	-
Grouted Prism	4000/27.6	3800/26.2

EXPERIMENTAL RESULTS

Hybrid masonry Test 1 employed two pairs of fuse plates in each story. Initial tests were conducted with the fuses disconnected to characterize the contribution of the steel frame to lateral load resistance. Under cyclic loading up to a deflection of 0.75" (19 mm), the initial elastic frame stiffness was 4.2 kips/inch (735.5 N/mm) with respect to force and displacement at the top of the test structure. Testing of only the frame between Tests 1 and 2 as well as between Tests 2 and 3 showed that the stiffness of the steel frame reduced to 2.9 kips/in (508 N/mm) after the initial test. With the fuses connected to the steel frame and the masonry, the initial elastic hybrid masonry stiffness was 23 kips/inch (4 MN/m) The specimen was subjected to a constant 80 kip (356 kN) vertical load and cyclically displaced as described in Table 3 and Table 4. The resulting applied lateral force (base shear) vs. top of structure deflection curve for each test is shown in Figure 9.

Table 3: Loading protocol for hybrid masonry Tests 1 and 2

Deformation Number	Displacement Target (in/cm)	% Drift	# of Cycles	Total Cycles
1	0.108/0.27	0.05	2	2
2	0.216/0.55	0.10	2	4
3	0.54/1.37	0.25	2	6
4	0.81/2.06	0.38	2	8
5	1.08/2.74	0.50	2	10
6	1.62/4.11	0.75	2	12
7	2.16/5.49	1.00	2	14
8	3.24/8.23	1.50	2	16
9	4.32/10.97	2.00	2	18
10	6.48/16.46	3.00	2	20

Table 4: Loading protocol for hybrid masonry Test 3

Deformation Number	Displacement Target (in/cm)	% Drift	# of Cycles	Total Cycles
1	0.108/0.27	0.05%	1	1
1	0.162/0.41	0.08%	1	2
1	0.1215/0.31	0.06%	2	4
2	0.216/0.55	0.10%	1	5
2	0.162/0.41	0.08%	2	7
3	0.432/1.10	0.20%	1	8
3	0.324/0.82	0.15%	2	10
4	0.648/1.65	0.30%	1	11
4	0.486/1.23	0.23%	2	13
5	0.864/2.19	0.40%	1	14
5	0.648/1.65	0.30%	2	16
6	1.512/3.84	0.70%	1	17
6	1.134/2.88	0.53%	2	19
7	2.16/5.49	1.00%	1	20
7	1.62/4.11	0.75%	2	22
8	3.24/8.23	1.50%	1	23
8	2.43/6.17	1.13%	2	25
9	4.32/10.97	2.00%	1	26
9	3.24/8.23	1.50%	2	28

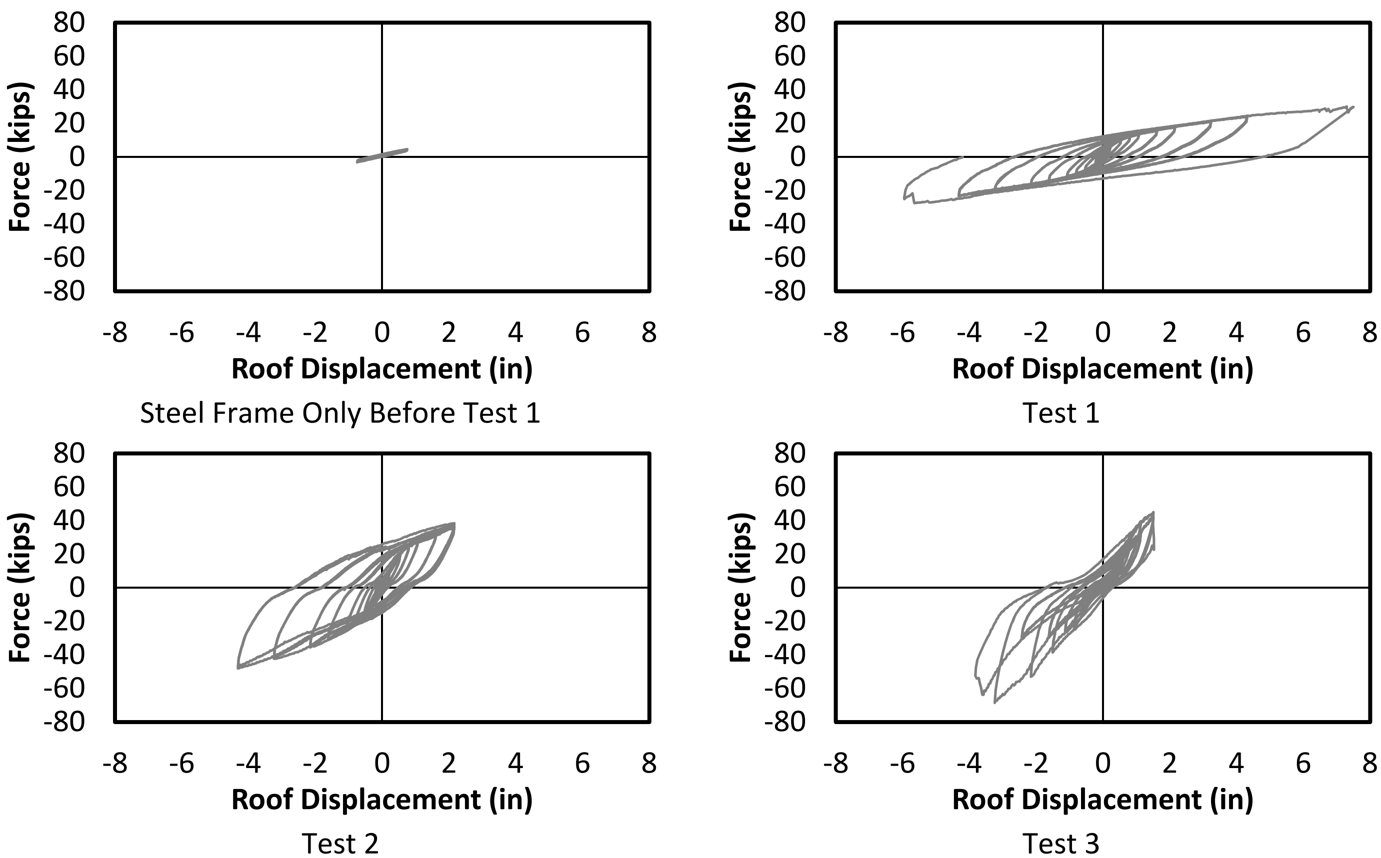


Figure 9: Base shear vs. top of structure deflection

The last cycle for Test 1 was chosen in an attempt to fracture the fuses, but they exhibited remarkable ductility and did not fracture. Testing was stopped to avoid damaging either the frame or test setup. During Test 2, the left most vertical rebar was not providing anchorage to the wall and was clearly not coupled into the base beam. Test 2 and Test 3 were adjusted to asymmetric loading accordingly. A maximum load of 30 kips (133 kN) was reached at a displacement of 7.5” (19 cm) as seen in Figure 9 for Test 1. Test 2 reached a peak load of 47 kips (209 kN) at 4.25” (10.8 cm). A maximum load of 66 kips (293 kN) was reached at a displacement of 3.14” (8.0 cm) during Test 3. Figure 10 illustrates the deformed shape of the system at these maximum forces and displacement points. Greater drift can be seen in the second story for Test 1 due to the rotational restraint of the column bases, which stiffen the first story. The difference in story deflection is reduced for Test 2 due to the increased stiffness provided by the wider fuses. This effect can be seen in Figure 11, where the fuses are shown for Tests 1 and 2.

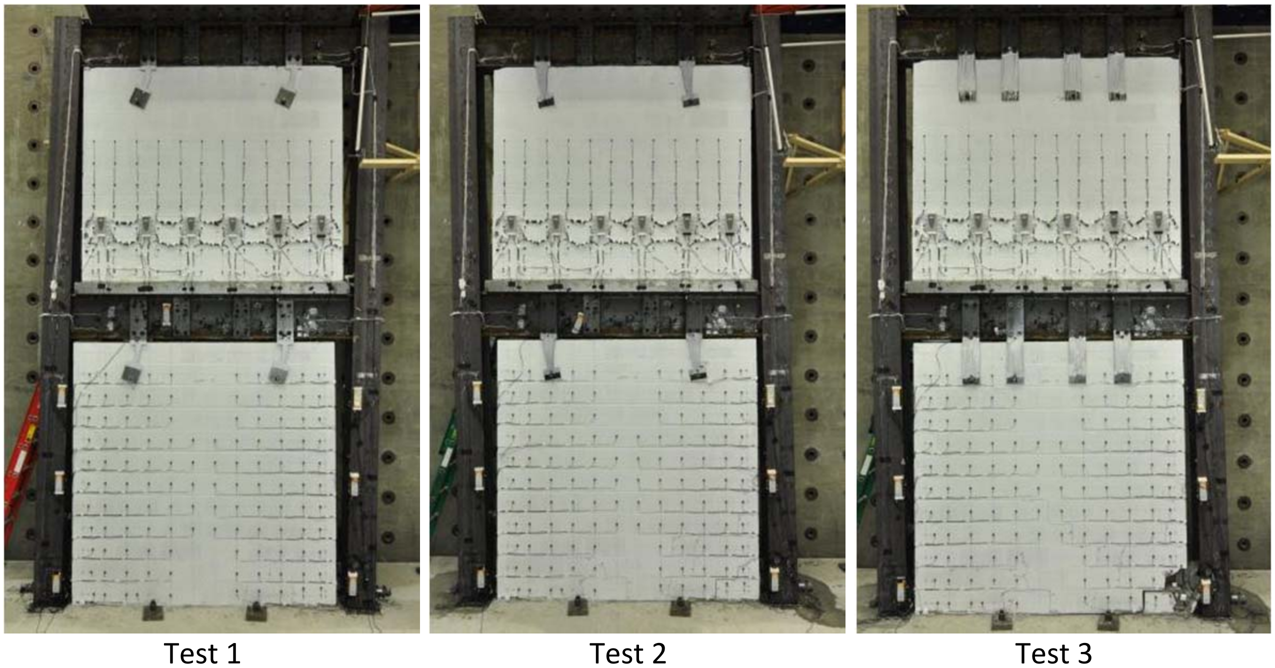


Figure 10. Test structures at maximum lateral deflection

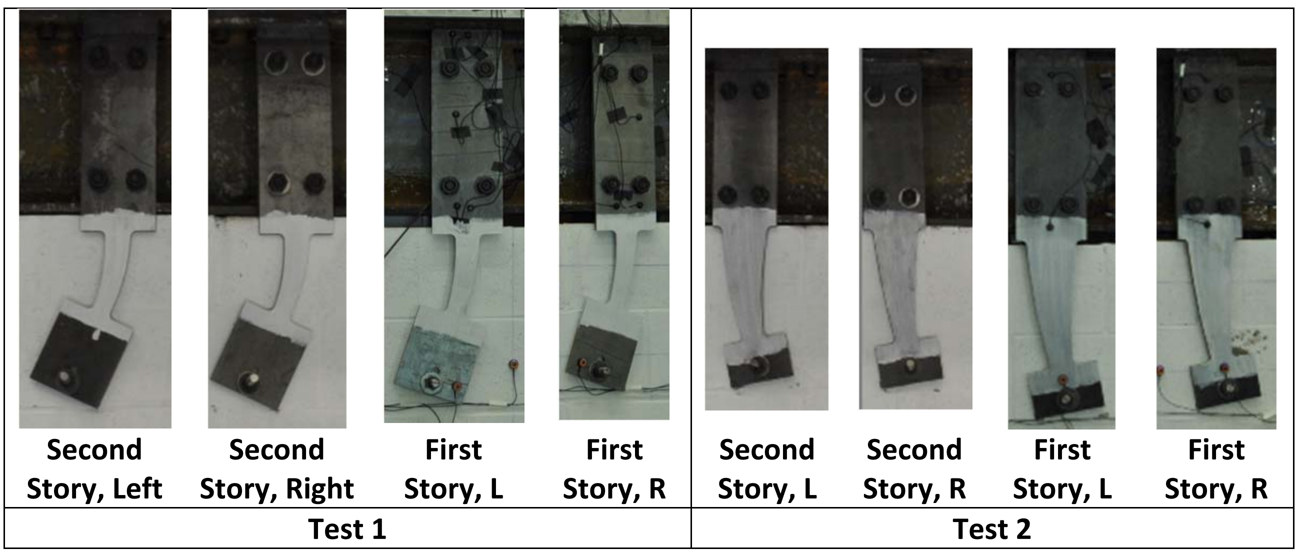
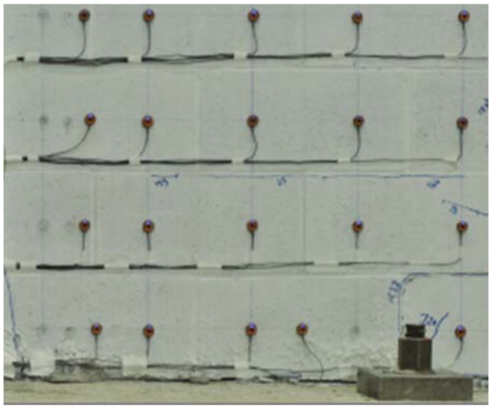
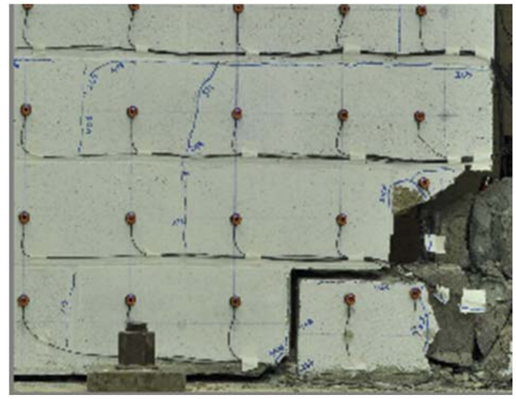


Figure 11. Test 1 and Test 2 fuses at maximum lateral deflection



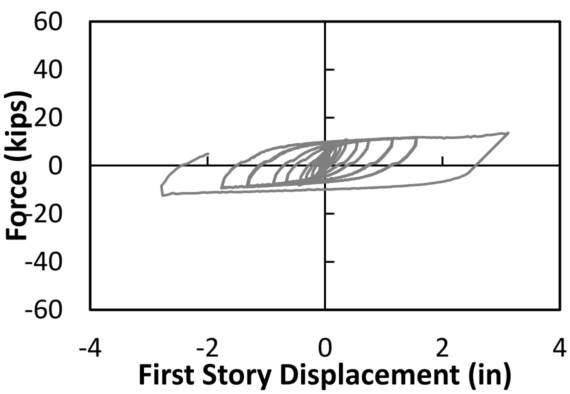
Left



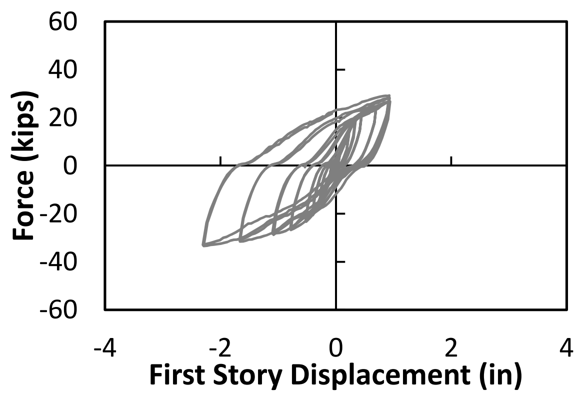
Right

Figure 12. Test 3 first story toe cracking/crushing damage at maximum lateral deflection

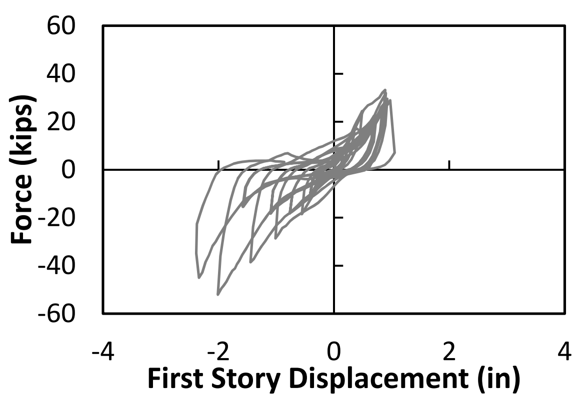
Strain gage data was used to calculate the moment at the bases of the column and just below the first story beam, and these moment gradients were used to calculate the shear forces at the bases of the columns. Taking the difference between the lateral load measured in the LBCB and the estimated first-story column shear provides an estimate of the shear in the connecting elements as well as the first-story masonry panel. The resulting force-displacement response is shown in Figure 13.



Test 1 Fuses



Test 2 Fuses



Test 3 Masonry Panel

Figure 13: Shear force in first story yielding components vs. first story displacement

Initial fuse yield occurred around 0.4” (1.0 cm) and 0.3” (0.76 cm) first story deflection and 11 kips (48 kN) and 17.5 kips (78kN) lateral force in the fuses for Test 1 and Test 2, respectively. Test 3 showed a maximum force of 49 kips (218 kN) in the masonry panel at 2” (5.1 cm) lateral displacement, which was in line with force expectations for the masonry flexural capacity. Although detailed data analysis is still underway, initial evaluation indicates that the fuse behaviour was consistent with component tests conducted at the University of Hawaii [10].

CONCLUDING REMARKS

Initial large-scale testing of hybrid masonry has begun to explore the potential of this new structural system. The first tests of a two-story hybrid masonry Type I specimen demonstrated robust cycle performance with good ductility. Further testing will thoroughly examine the three different types of hybrid masonry to provide valuable new data on system behavior and to validate and refine seismic design procedures. Expectations are that the full experimental program, along with design studies and numerical simulations, will provide a sound basis for implementing hybrid masonry as a new lateral force resisting system in high seismic regions.

ACKNOWLEDGEMENTS

The National Science Foundation has supported this work under Grant No. CMMI 0936464, as part of the George E. Brown, Jr. Network for Earthquake Engineering Simulation. Additional support has also been provided by the American Institute of Steel Construction, the National Concrete Masonry Association, and the International Masonry Institute.

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