SEISMIC REHABILITATION OF UNREINFORCED MASONRY SHEAR WALLS

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ABSTRACT

One of the challenging tasks of current engineering practice is to systemically determine the deficiencies of existing structures and then provide rational and economical solutions to these problems. A rational understanding of the performance characteristics of structures and their components is essential to efficiently accomplish this task. The purpose of this paper is to investigate the performance characteristics of both plain and rehabilitated unreinforced old masonry (URM) shear walls to in-plane loads.

A series of full-scale shear walls were tested under harmonic deformation cycles at quasi-static loading rates. The deformation level was progressively increased in each test until there was a significant loss in the force carrying capacity of the shear wall. The walls were constructed using reclaimed solid clay bricks (units compressive strength of 28MPa [4050psi]) to represent the early twentieth century construction characteristics. Aspect ratio, height-to-length, was held constant at approximately 0.5 for each wall. Type S mortar, having cement:lime:sand ratio of 1:3½:4½ was used in constructing the walls. The vertical stress was maintained constant throughout each test and was set equal to 0.62Mpa [90psi] and 0.90Mpa [130psi].

Two plain (non-rehabilitated) and a rehabilitated wall were tested as part of this program. The plain wall after being tested was repaired for further testing. The center-core technique was used to rehabilitate the second wall. Performance parameters were deduced from the experiments in the context of the FEMA 273 Rehabilitation Guidelines (FEMA, 1997). The performance parameters of interest were strength, stiffness, and energy absorption and deformation capacities.

This paper summarizes the preliminary results of the shear test series. The results showed that sliding as well as flexural strength of plain walls were enhanced by the center-core method. The strain history of the embedded reinforcing bars suggested that walls rehabilitated with this method can be treated as reinforced masonry walls.

Key words: unreinforced brick masonry, shear strength, ductility, center-core, rehabilitation

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INTRODUCTION

Comparing past seismic events with more recent ones, one might argue that the impact of earthquakes on civilizations and communities shows an increasing trend. This might be attributable to an increase in the rate of occurrence of seismic events, a change in the spatial distribution of these events, or an expansion of communities into more seismic prone areas. Whichever is the reason, more and more people are affected by the undesirable consequences of earthquakes. This issue becomes more important in regions like the New Madrid seismic zone in the Central United States, where the rate of occurrence of major seismic events is on the order of hundred years, (Nuttli, 1995). Even though recent regulations and seismic codes enforce seismic design in these regions, there are old structures, which were designed before the seismic regulations took place, directly or indirectly threatening lives. One possible way to reduce this threat is to improve the seismic behavior of these vulnerable structures through rehabilitation measures. A rational understanding of the performance characteristics of structures and their components for different rehabilitation techniques is essential to efficiently accomplish this task. The purpose of this paper is to investigate the performance characteristics of both unretrofitted and rehabilitated unreinforced old masonry (URM) shear walls to in-plane loads.

TEST SETUP AND MATERIAL PROPERTIES

A series of full-scale masonry brick walls were tested under harmonic deformation cycles at quasi-static loading rates. Figure 1 shows a typical wall in the loading rig. The loading rig consists of a post-tensioned concrete masonry reaction wall, a concrete foundation pad (305x1524x5180mm, [12x60x204in]), a concrete loading beam (457x457x4290mm, [18x18x169in]) and a pair of 490kN [110kip] capacity servo-hydraulic actuators. For the first test, the vertical compressive forces were applied with

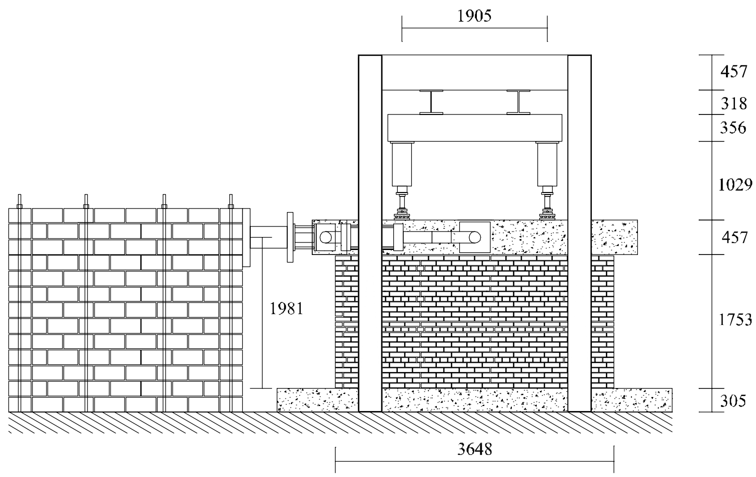

Figure 1 Loading Rig and The Masonry Test Specimen

a series of four pairs of hydraulic jacks that were driven by motorized pumps. Due to the slow reaction time and low accuracy in this method of control, the hydraulic jacks were replaced by a pair of 670kN [150kip] homemade servo-hydraulic jacks that were operated by an Instron 8500^{+} control tower. The hydraulic jacks in the initial setup were reacted by high strength 25.4mm [1.0in] diameter all threaded rods that were screwed into the foundation pad. The servo-hyraulic jacks in the second setup were pushing against a pair of transverse beams that are supported by an external steel frame as shown in Figure 2.

The vertical stress was maintained constant throughout each test and was set equal to 0.62Mpa [90psi] for the first wall and 0.90Mpa [130psi] for the second and third walls. The test specimens were intended to emulate a cantilevered wall fixed at the foundation level and free at the centerline of the top concrete beam.

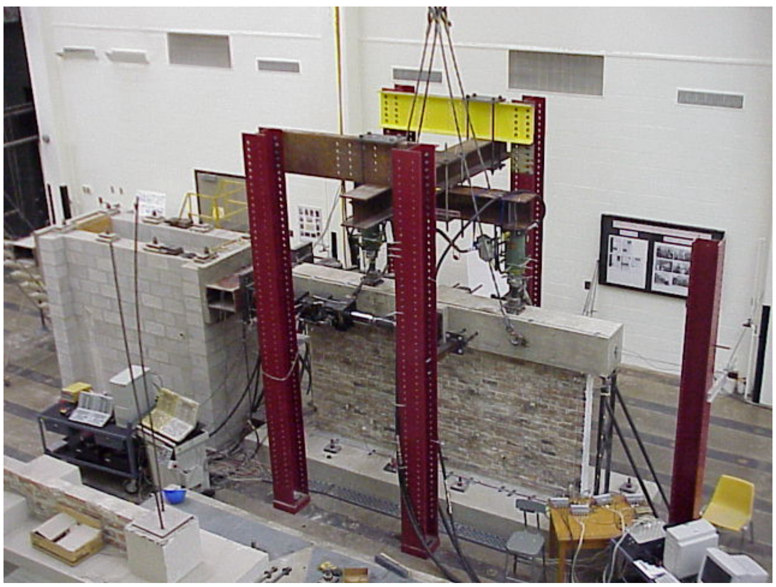

Figure 2 Test Wall in Loading Rig

Horizontal forces were applied at the center of the top concrete beam, which was 1980mm [78in] above the concrete foundation surface. The horizontal actuators were operating under deformation-controlled mode, applying three equivalent harmonic displacement cycles at quasi-static rates with gradually increasing amplitudes until a significant loss in the force carrying capacity of the wall was achieved. The deformations were measured at the same level the horizontal forces were applied. A fixed reference column outside the loading rig served as the datum for these measurements.

The walls were constructed using reclaimed solid clay bricks (units compressive strength of 28MPa [4050psi]) to represent the early twentieth century construction characteristics. Aspect ratio, height-to-length, was held constant at approximately 0.5

for each wall. Type S mortar, having cement:lime:sand ratio of 1:3½:4½ was used in constructing the walls. The results of five prism and six mortar cube tests indicated an average prism compressive strength of 8.5MPa [1240psi] and an average mortar compressive strength of 17.6MPa [2550psi]. From the same data, the average elastic modulus of the masonry prisms in compression was computed as 4500MPa [654ksi]. Bond-wrench and in-place shove tests were performed to get the flexural tensile and the mortar-joint sliding shear strength of the masonry assemblages. The average values are 0.3MPa [44psi] and 2.45MPa [360psi], respectively.

THE REHABILITATION METHOD

General

The effectiveness of the center-core rehabilitation technique was examined in this test series. The method consists of reinforcing an existing masonry wall by placing conventional reinforcing bars into pre-drilled cores and filling these cores with grout to provide the bond between the reinforcement and the surrounding masonry (Lizundia, et. al. 1997 and Breiholz, 2000). The walls rehabilitated with this technique are treated as reinforced masonry walls provided that the bars have sufficient development length (Lizundia, et. al. 1997).

Coring is done either by wet or dry process. Though the wet process was the preferred method, the dry process became more popular as the enhancements in drill bit technology made it more practical. With the current equipment technology, 75 to 150mm cores can be drilled through the entire height of a two or three story masonry wall (Abrams, 2000).

The grout material can be cement, sand/epoxy or sand/polyester mix. Research have shown that the strength as well as the flow characteristics of epoxy and polyester grouts are better than cement grouts (Breiholz, 2000 and Lizundia, et. al. 1997).

The main advantage of the center-core rehabilitation technique is that the application does not affect the appearance of the wall surface and the implementation can be done without interrupting the building function (Abrams 2000).

Test Wall

Four 75mm [3in] vertical cores were drilled through dry process into the test wall. The cores were extended 260mm [10in] into the concrete foundation to provide 16 bar diameters of development length for the reinforcing bars. The cores were symmetrically oriented along the wall centerline and were equally spaced at 1120mm [44in].

The reinforcement amount is selected based on the minimum requirements given in UBC 1997, sec 2108.2.5.2. In view of this document, 16mm [#5] conventional reinforcing bars, having vertical reinforcement ratio, ρ_{sv}, of 0.01%, were selected. The bars were placed at the center of each core and strain gauged at the base of the wall.

Sand/Polyester (also known as Orthophtalic Polyester Resin – Sand) mix was used for grouting purposes. The mix had a sand-to-polyester volumetric ratio of 1.5:1.0. The materials were mixed using a motorized steel blade until the desired homogeneity was reached in the mix. After that step, catalyst, DDM-9, was added to the grout mix (10cc per 1 liter of mix [2.5cubic inches per 1 gallon of mix]). Preliminary cylinder tests showed that the grout mix had an average compressive strength of 85.6MPa [12.4ksi] and an average compressive elastic modulus of 6190MPa [897ksi].

OBSERVED WALL BEHAVIOR UNDER CYCLIC LOADING

Plain Wall Behavior

Two control test specimens were tested under two different vertical compressive stresses, 0.62MPa [90psi] and 0.90MPa [130psi]. The general response behavior was similar for both walls. The initial elastic response was followed by a flexural crack at the bottom bed-joint that extended along the total base length for higher drift levels. As the flexural crack length became longer, walls started to slide along the base joint. The lateral force capacity of each wall was bounded by the surface friction at the base level. The limiting state was reached when the deterioration at the toe region reached a level at which the out-of-plane stability of each wall was altered and the walls moved out-of-plane.

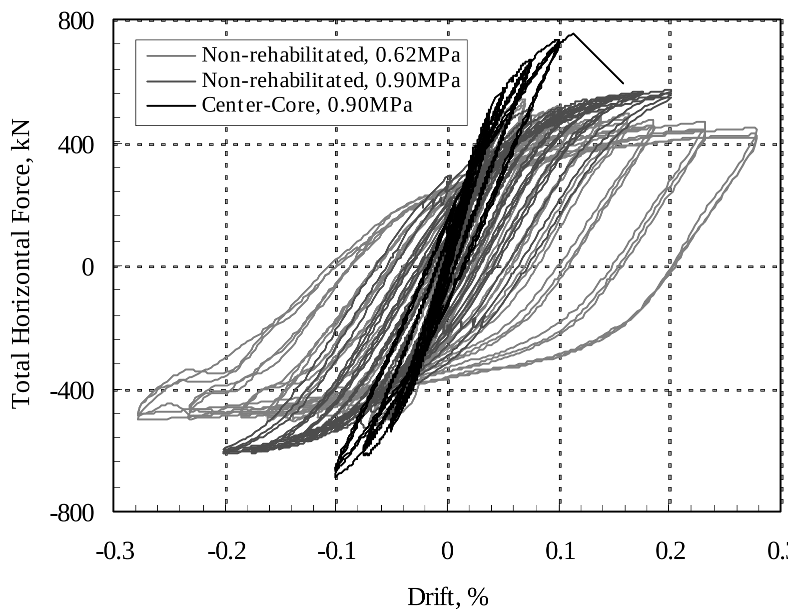

Figure 3 Measured Force-Deflection Curves for Masonry Wall Specimens

The first plain wall specimen, 1S, had a constant vertical compressive stress of 0.62MPa [90psi]. The behavior was linear-elastic up to a drift level of 0.04% at which a flexural crack took place at the base joint. The lateral force was approximately 74% of the ultimate lateral force capacity of the specimen. With increasing drift levels the flexural crack extended and eventually joined with the crack developing from the other side of the base. At approximately 0.1% drift, the wall started to slide causing unrecoverable deformations along the wall base, Figure 4. Figure 3 shows the full force-displacement response history of the wall. As can be seen, sliding shear response becomes more dominant, more dissipated energy within loops, with increasing drift levels. Other than providing a good energy dissipation mechanism, the sliding shear behavior greatly enhanced the deformation capacity of the wall. Lateral drifts as large as seven times the initial flexural cracking drift, were observed without loss of lateral force capacity. High

Figure 4 Sliding Along the Bottom Bed-Joint, Specimen 2S

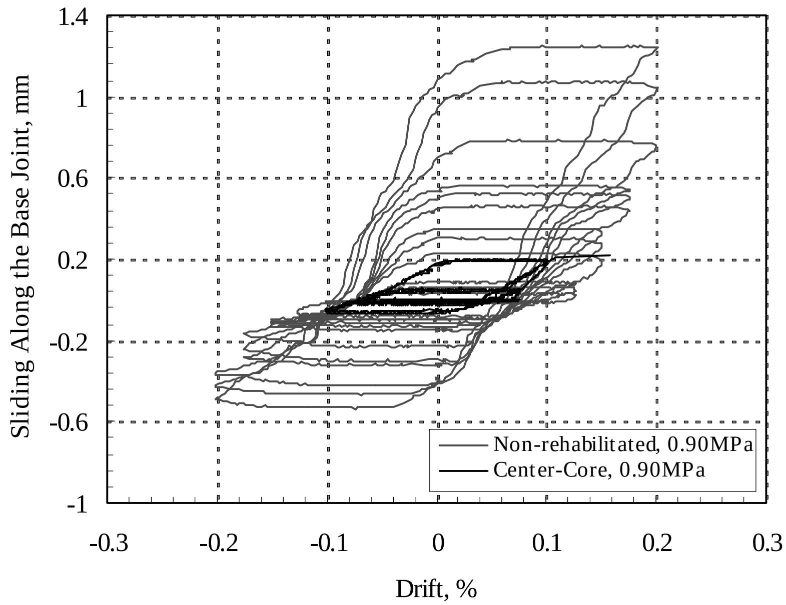

Figure 5 Bottom Bed-Joint Displacement at Different Drift Levels (2Sand 3S)

post-cracking strength is due to the friction acting along the base joint and therefore suggest that the vertical compressive force plays a significant role on ductility. The limit state was reached at a drift level of 0.3%. The limit state mode can be stated as toe-crushing together with an out-of-plane failure, Figure 6.

The second plain test wall, 2S, had a vertical compressive stress of 0.90MPa [130psi]. The general response behavior was very similar to the first specimen except that the higher vertical compressive stress level resulted in higher lateral force capacity, Figure 3. The initial flexural crack occurred at approximately 0.04% drift, corresponding to a lateral force level of approximately 65% of the ultimate lateral strength. Similar to the first specimen, this specimen also slid along the bottom bed-joint, Figure 4. The sliding started at a drift level of approximately 0.13%, Figure 5. As can be seen, the amount of sliding gets larger as the number of repetitions of the cycle increases for the same drift level. This indicates that the sliding surface roughness deteriorates with increasing cycle repetitions. At approximately 0.2% drift level the wall reached its limit state. The mode of failure can be represented as toe-crushing together with out-of-plane failure. The ultimate drift level is approximately five times the drift level at the initial flexural cracking took place.

Figure 6 Toe-Crushing at the Ultimate Stage, Specimen 2S

Rehabilitated Wall Behavior

The center-core rehabilitation technique was selected to improve the sliding shear strength and the flexural capacity of the wall specimens. The vertical compressive stress was set equal to 0.90MPa [130psi] for direct comparison purposes. The specimen had an initial cracking drift that was similar to the previous specimens. The force level corresponding to this drift level was approximately 66% of the ultimate lateral strength. The outer reinforcing bars yielded at approximately 0.06% drift level, indicating that the cores were able to develop the full yield strength of the reinforcing bars, Figure 7. This suggests that the behavior of the specimen can be represented by that of a reinforced masonry wall. Examination of Figure 7 shows a horizontal offset in the curve corresponding to sliding taking place between the core and the surrounding masonry in the East corner reinforcement beyond 0.07% drift. A similar offset can be noticed for the West corner reinforcement but in the vertical direction. This indicates a permanent plastic deformation in the bar.

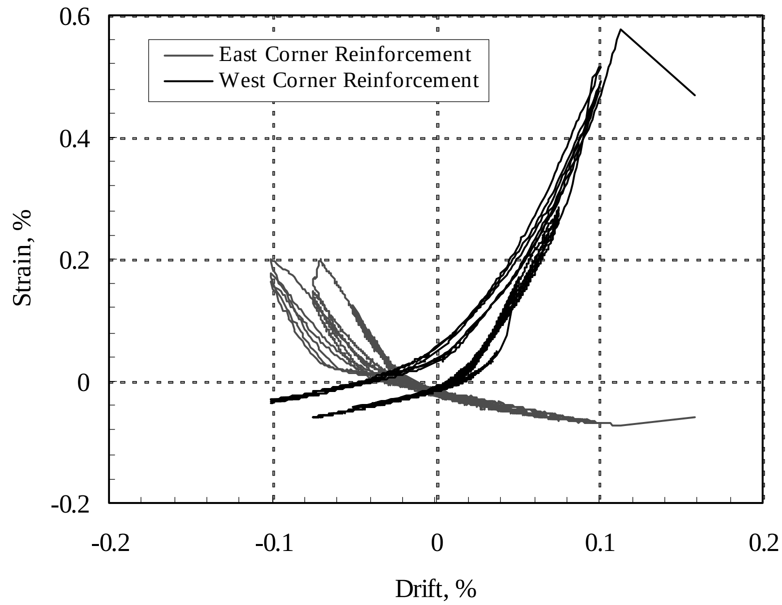

Figure 7 Strain Variation at Corner Reinforcements in Specimen 3S

The sliding shear capacity was improved due to the presence of the cores that acted as shear keys and reacted the shear developed at the base through dowel action, Figure 5. Moreover, the presence of the vertical reinforcement resulted in smaller flexural cracks that further enhanced the sliding shear capacity through better particle interlock mechanism. Though the energy dissipation characteristics of shear sliding behavior was not observed in this specimen, energy was dissipated due to yielding of the reinforcing bars. Since the bars yielded at an early drift level the energy dissipation started at early stages of the response. The ultimate drift reached was 0.113% at which the top loading beam lost bond with the specimen.

Comparison of member behavior

Table 1 summarizes test results of the masonry wall specimens. The results are presented in light of the FEMA 273 document for performance-based seismic rehabilitation. The comparison of the preliminary test results is given in terms of the performance parameters indicated in that document.

Figure 8 shows the envelope curve for each specimen. Comparison of the curves in the elastic range indicates that specimens 1S and 2S have very similar initial elastic stiffnesses. However, being reinforced and grouted with stiffer material than the surrounding masonry, the third specimen shows a higher initial elastic stiffness than the first two specimens. The initial elastic stiffnesses can be approximated as 540,000kN/m [3080kip/in], 592,000kN/m [3376kip/in], and 663,000kN/m [3780kip/in] for 1S, 2S and

3S, respectively. Similar to the initial elastic stiffness, the unloading stiffness is higher for the rehabilitated specimen, Figure 3. Early stages of the unloading response for specimens 1S and 2S reflects the small stiffness associated with the crack closing behavior. Unlike the first two specimens, the presence of cores and reinforcing bars reacted the unloading motion thus increased the unloading stiffness for specimen 3S.

Table 1. Summary of Test Results

Wall ID	Aspect Ratio, h/L	Vertical Comp. Stress MPa [ksi]	Lateral Ultimate Strength kN [kip]	Ultimate Drift Level %	Brief Response History
1S	0.52	0.62 [90]	543 [122]	0.279	Flex. Crack → Base Sliding → Toe Crushing + Out-Of-Plane Failure
2S	0.52	0.90 [130]	614 [138]	0.200	Flex. Crack → Base Sliding → Toe Crushing + Out-Of-Plane Failure
3S	0.52	0.90 [130]	752 [169]	0.113	Flex. Crack → Yielding → Contact Failure of the Top Beam

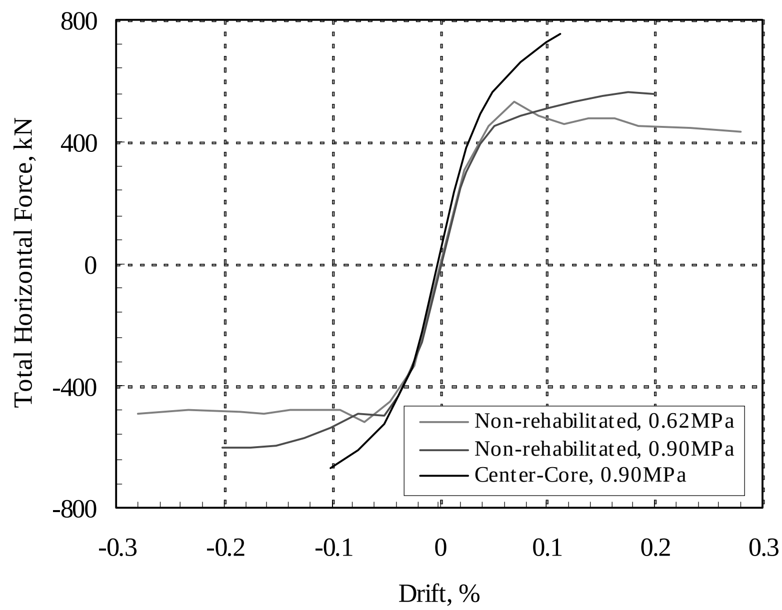

Figure 8 Envelope Force-Deflection Curves for 1S, 2S and 3S

Lateral shear strength is enhanced by increasing vertical compressive stresses and application of the center-core rehabilitation method. Comparison of two plain walls, 1S and 2S, suggests that the strength increased proportional to the increase in the vertical compressive stress level at the same drift. This observation is directly attributable to external equilibrium of applied forces, (Abrams, 1992). Reduced flexural crack length and width also increased the effective stiffness and improved the particle interlock mechanism. A similar observation between the rehabilitated and the plain wall, 3S and

2S, reveals that the sliding shear as well as the flexural strength of the wall were enhanced. Lateral strength was 20% more than that of the plain wall.

Unlike the strength improvement, the increase in the vertical compressive stress had an adverse effect on the overall deformation capacity of the wall specimens. The increase in vertical compressive stress level was compensated by an increase in the compressive stresses at the toe region of the wall. This redistribution and rescaling of base stresses resulted in reaching the limit state at an earlier drift level. Comparison of the plain and the rehabilitated wall may not yield realistic results owing to the fact that the bond between the loading beam and the third specimen failed before reaching a similar drift level as the plain specimen. However, the fact that the reinforcing bars yielded, suggests a ductile behavior.

Plain and rehabilitated walls had a different energy dissipation mechanism. For plain walls most of the energy was dissipated through the sliding shear mechanism. As can be seen from Figure 3, this mechanism initiated at a certain drift level and advanced further during the subsequent drift levels. Since the siding shear capacity was greatly enhanced by the rehabilitation technique, the energy dissipation attributable to this mechanism was relatively low for the rehabilitated specimen. However, yielding in the reinforcing bars and slip at the core interface introduced other means of energy dissipation measures. Unlike the sliding shear mechanism, energy dissipation due to yielding started at an earlier drift level because drift levels smaller than drift levels resulting in sliding, generate yielding in the reinforcing bars. Different energy dissipation characteristics, loop fatness, for small drift levels can be seen in Figure 3.

CONCLUSIONS

This paper has presented the preliminary test results of recent unreinforced clay brick masonry shear wall test series that investigates the performance characteristics of rehabilitated and plain walls. The walls were tested under harmonic deformation cycles at quasi-static loading rates. The specimens were constructed by using reclaimed brick to represent early twentieth century construction quality.

Results shoed that unreinforced masonry walls can behave during earthquakes with a substantial amount of inelastic deformation capacity and energy dissipation. The walls possessed higher post-cracking strength than their strength at initial cracking (as high as 50% was observed). Similarly, deformation capacities were larger than the initial cracking deformation capacity, (more than a factor of five was observed). Both performance parameters were highly influenced by the vertical compressive stress level. Higher stress levels resulted in higher lateral strength but lower deformation capacities.

The behavior of the plain wall with Type S mortar was dominated by sliding shear behavior, for which the lateral strength of the wall was limited by the amount of friction developed at the wall base. The center-core rehabilitation technique greatly enhanced (improvement of as much as 20% was observed) the sliding shear capacity through dowel action. In addition, smaller flexural cracks were observed with the center-core technique.

Friction due to sliding shear was the primary energy dissipation mechanism for the plain walls. Reinforcement yielding and core sliding contributed to energy dissipation mechanism for the center-cored wall. Both energy dissipation mechanisms became more pronounced for higher drift levels, though yielding took place at a smaller drift level than the drift level at which the sliding shear behavior started.

The deformation and strength limit states presented in this study were representative of single wall behavior and might underestimate the wall behavior in an actual building structure, in which surrounding members enhance the response by confining and restraining the wall.

ACKNOWLEDGEMENTS

Research on rehabilitation of unreinforced masonry shear walls was done as part of project ST-6 of the Mid-America Earthquake Center. This work was supported primarily by the Earthquake Engineering Research Centers Program of the National Science Foundation under Award Number EEC-9701785. Author would like to thank Mr. Shaun Franklin, Mr. Jaret Lynch and Newmark Civil Engineering Laboratory technical staff for their assistance with this research.

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