FRP Repair Methods for Unreinforced Masonry Buildings Subject to Cyclic Loading

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Synopsis: Unreinforced masonry building specimens were evaluated under cyclic lateral loading. Various fiber reinforced polymer (FRP) composite configurations were used to repair and retrofit the masonry structures. In the first phase, three different composite systems were used to repair pre-damaged masonry structures. These systems included: a wet lay-up woven glass fabric; a near surface mounted (NSM) extruded carbon FRP plate; and a glass FRP grid attached via a high elongation polyurea resin. Retesting of the repaired structures revealed increases as much as 700% in terms of energy dissipation and 300% in terms of pseudo-ductility. The second phase involved retrofitting similar undamaged building specimens with FRP composites. Significant increases in strength, ductility and energy dissipation were observed. The seismic performance of each structure was increased with the addition of a minimal amount of composite material as compared to the unreinforced structures.

Keywords: FRP; near surface mounted; repair; retrofit techniques; seismic loading; unreinforced masonry; wet lay-up

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RESEARCH SIGNIFICANCE

The research presented herein describes a methodology for the repair and retrofit of unreinforced masonry (URM) buildings using fiber reinforced polymer (FRP) composites. This methodology can be directly applied to structures that have been damaged due to significant lateral loading or structures that are substandard and need to be retrofitted and, as seen in this paper, can significantly enhance the strength and performance characteristics of these structures. As compared to traditional repair/retrofit methods, this new methodology is appreciably more cost-effective as well as less intrusive.

INTRODUCTION

Fiber reinforced polymer (FRP) composites have been extensively investigated in the last decade and a half throughout the world. The interaction of FRP materials with concrete and masonry has been studied under a variety of loading conditions. With regard to masonry, FRP composites have been used to strengthen unreinforced masonry shear and flexural walls with resounding success. The extent of this research has been limited, with few exceptions, to component testing.

The seismic capacity of unreinforced masonry (URM) shear walls is minimal. Experimental studies (Eshani et al., 1996) proved that using FRP composites could enhance the seismic performance of unreinforced masonry shear walls. The dynamic response of URM shear walls has been investigated as well (Al-Chaar et al., 1999). In another study, the performance of URM shear walls subjected to cyclic lateral loads were greatly enhanced when CFRP composite laminates were used to retrofit the masonry components (Gergely et al., 2000).

The capacity of unreinforced masonry flexural walls has been shown to have drastically increased by using FRP composites (Albert, Elwi and Cheng 2001). In this investigation, both glass and carbon FRP laminates were used. In a similar investigation, flexural walls strengthened with FRP materials resulted in significantly increased flexural capacities, provided that shear was controlled at the support (Hamoush et al., 2001). In a similar study, the capacities of GFRP flexural walls were compared to design equations that resulted in an overprediction of no more than 20% (Hamilton and Dolan, 2001).

The performance of unreinforced masonry infill walls has been explored as well. The capacity and performance of these infill walls can be significantly enhanced by FRP composite retrofit (Silva et al., 2001). Infill walls, subjected to both on and off-axis loading, were shown to have moderate to significant strength increases when retrofitted with glass FRP (Hamid et al., 2005).

However, few investigations focused on the entire masonry building system subjected to quasi-static lateral loading. One such project evaluated a full-scale masonry building system with flexible diaphragms, that utilized several different FRP composite laminates, as well as post-tensioning, to enhance the performance of the system (Moon et al., 2002). It was found that the system-wide performance was greatly enhanced under cyclic lateral loading. It has also been shown that the dynamic capacity of URM walls reinforced with FRP composites can be significantly improved with failure occurring in the masonry units, rather than the mortar joints (Marshall, Sweeney and Trovillion, 1999).

In general, the experimental evaluation of full scale building models is quite expensive and testing of individual components seems to be a much more cost-effective solution. The results of these component tests, however, may be somewhat skewed in some instances and there may exist idiosyncrasies that may not be accurately captured by component testing. In a full scale building model, redistribution of lateral forces and changes in end conditions (i.e. fixed to cantilever) can be achieved when cracking occurs. These cannot be captured accurately with component testing. Also, with component testing, out-of-plane rotations may occur that would not normally occur in a full scale building model. It may be that component testing is applicable for obtaining general performance characteristics of a particular FRP system but, ultimately, the system must be implemented on a full scale structure to attain an accurate response.

The goal of the present research was to investigate several repairs <u>and</u> retrofit methods for existing unreinforced masonry structures, with rigid diaphragms, subjected to quasi-static lateral loading. In support of this effort, large-scale structural tests were performed at UNC Charlotte, and the small component tests were performed at NC A&T.

EXPERIMENTAL DESIGN

Several full-scale unreinforced masonry buildings were constructed. Due to size constraints that existed within the load frame, each structure was constructed with the dimensions 2.48 m (8') in height, 3.25 m (10'8") in width, and 4.47 m (14'8") in length (see Figure 1). The shear walls on each side were perforated with two 82 cm (32") wide standard doorway openings in two structures, resulting in a symmetric structural configuration. An asymmetric configuration was also tested, and in this case only one shear wall was perforated with the standard doorway opening.

A rigid roof diaphragm, designed as a deep beam, was used to prevent any independent movement that may occur in the walls with a flexible diaphragm. This roof diaphragm was placed atop a mortar bed and secured to the structure utilizing 32 anchorage points evenly spaced around the perimeter of the bond beam. Dywidag Threadbar® reinforcing bars were used to transfer the applied lateral load through the roof panel via a hydraulic piston. A gravity load simulator was used to apply a 667 kN (150 Kips) vertical load to the structure, simulating the weight of two additional floors. The loading mechanism was comprised of wide-flange steel members, and this simulator was designed such that the gravity load would be maintained under an applied cyclic lateral load, i.e. the entire gravity load system was allowed to translate, together with the building, under lateral load while maintaining the applied gravity load. This vertical load was monitored throughout the test both by the hydraulic system pressure transducer and by strain measuring instruments.

Each masonry building was constructed atop reusable concrete foundations. These foundations had removable dowel bars that extended 41 cm (16") above the top of the foundation, and these bars were grouted in the masonry wall. The dowel bars were used to prevent any overturning or sliding that may have occurred at the foundation level, and forced the failure to occur in the shear wall panels. In addition to the bottom 41 cm (16"), the top 20 cm (8") bond beam was also grouted with a 27.6 MPa (4000 psi) grout. This was done to allow anchorage for the roof diaphragm and foundation elements. The remainder of the masonry walls was ungrouted.

The testing procedure for each structure was identical: after the 667 kN (150 Kips) gravity load had been applied, the lateral load was applied at a rate of 3.5 kN (800 pounds) per second, with a load step of 44.5 kN (10 Kips). Each load step was comprised of three push-pull cycles. In the first phase, after failure of the URM building was achieved, the lateral load was removed, followed by the release of the gravity load. Following the removal of the instrumentation, the structure was evaluated and an appropriate FRP composite repair technique was selected. Following the FRP repair, the structure was retested. In the case of the retrofitted structures (Phase II), the FRP composite was applied without pre-testing the building. During the loading process, an array of instrumentation was used, which included displacement transducers, strain transducers, strain gages, linear potentiometers, pressure transducers and load cells.

EXPERIMENTAL RESULTS

CMU-Configuration I

Two symmetric CMU structures were fabricated and tested in this configuration. Both structures, as previously stated, were unreinforced. One structure was damaged and subsequently repaired using a glass FRP composite system and retested. The second structure was retrofitted using an identical composite system, although the layout was altered.

The performance of the unreinforced (baseline) structure remained linear until a lateral load level of approximately 267 kN (60 Kips) (see Figure 2). At this point, microcracking began to occur in the bed and head joints, which developed into a more extensive diagonal step cracking pattern as the load was increased. The cracking in the large pier was limited to the mortar joints, while in the small pier diagonal tension cracking was observed in the masonry units and the mortar joints. Failure occurred at an applied lateral load level of 400 kN (90 kips) with an associated lateral displacement of 6.35 mm (0.25"). The damage associated with this load level and displacement was not catastrophic, and the structure continued to withstand the gravity load at failure.

For both testing phases, the composite system used to strengthen the symmetric CMU building specimens was SikaWrap Hex 100G glass fiber woven fabric in conjunction with Hexcel 306 resin. This is an externally applied wet lay-up GFRP composite system that can be applied to one or both surfaces of a wall, depending on the site conditions and the performance requirements. In this project, all the FRP systems were applied only to the exterior surface of the shear walls.

The composite layout for the repaired structure was determined using a strut-and-tie model. The strut and tie model was developed from a simple free body diagram (FBD) as seen in Figure 3. In this figure, only the FBD for the large pier is shown. The axially applied load, P, and the applied lateral load, V, are known and this allows for the determination of the internal forces of the pier. The axial load, P, as previously specified was 667 kN (150 Kips) distributed evenly along the length of the shear walls. The applied lateral load was set to a level of 778.4 kN (175 kips) with an equal dispersion between the two shear walls. Distribution factors, based on rigidity calculations, were used to determine the lateral force applied to each pier. The internal shear forces, V1 and V2, characterize the shear strength of the GFRP laminate as determined through small component testing and the additional shear capacity achieved through friction and the applied axial load, respectively. The tensile forces, T and T2, represent solely the tensile strength of the GFRP laminate as the damaged unreinforced masonry substrate cannot hold significant tensile forces. From this, the quantity of GFRP composite needed to maintain rotational equilibrium was determined and yielded the number of 30 cm (12") layers needed to repair the structure. It was determined that two 1.016 mm (0.04") thick layers would be used throughout the majority of the structure, with the exception of the flexure-dominated small piers, where three vertical layers were used (see Figure 4), in addition to the diagonal laminates. The composite layout for the retrofitted symmetric CMU structure was determined using a similar procedure although the laminate configuration differed. It was comprised of only vertical layers, a somewhat less efficient but simpler layout to implement. The layout called for three 25 cm (10”) strips in the large piers, evenly spaced across the pier, and two in the small piers. Each laminate was comprised of two layers of glass fiber.

As expected, the performance of the repaired structure was far superior to the unreinforced test. The lateral load reached a maximum level of 667 kN (150 kips), and the associated displacement in the push cycle had a magnitude of 22.35 mm (0.88”). At failure, the FRP laminate received little to no damage. The majority of the damage occurred in the large pier and was localized to the bottom of the doorway, as anticipated. The mode of failure was masonry substrate failure. The nature of the composite layout was such that significant forces from the flexural and shear reinforcement were transmitted through this region in a pull cycle, during which the structure failed. The tensile forces in the diagonal and vertical composite laminates created a moment about the dowel bar embedded with the CMU, a load which the masonry units could not withstand. A possible way to avoid this would be to wrap this region with the same fabric, or provide some other anchoring means to prevent the splitting that occurred. Later tests proved that when this region is protected, the structure will likely reach higher lateral loads.

The retrofitted CMU structure shown in Figure 5 performed well under the cyclic lateral load. The structure resisted approximately 35% more load than the URM baseline building, with a maximum applied load of 547 kN (123 kips). The displacement accompanying that load level was 12.2 mm (0.48”) in push. Surprisingly, the stiffness of the structure did not significantly degrade until the lateral load reached a level of 400 kN (90 kips). The mode of failure observed was diagonal tension cracking in all piers .

Hysteretic and backbone curves were developed for the undamaged, repaired and retrofitted tests using guidelines set forth be the Federal Emergency Management Agency Document 356 (FEMA, 2000). The envelope created defines the general behavior of the structure in terms of applied load versus displacement. A comparison of the backbone curves from the unreinforced and the repaired tests shows that the FRP reinforced system had significantly more lateral capacity, but also had an increased pseudo-ductility (see Figure 6). Repairing the damaged CMU building with the glass FRP laminate increased the strength of the system by 67% beyond the capacity of the URM baseline building, and increased the energy dissipation (strain energy) of the system by nearly 550%, based on the compression quadrant of the curve. The repair of the damaged URM building with the GFRP laminate resulted in not only a stronger system, but also a much more ductile one as well. The performance of the retrofitted structure (using less composite material with a different layout), although not as “spectacular” as the repaired structure, did show remarkable increases in terms of both strength and energy dissipation. A 35% increase in strength and a 220% increase in energy dissipation were observed.

The maximum tensile strain in the GFRP laminate measured during the test of the repaired structure and had a magnitude of approximately 0.54%. This value was recorded in the flexure-dominated pier at the location of the lintel. The maximum strain measured in the retrofitted structure was approximately 0.8%. This occurred in the small pier at the level of the grouted section at the base of the structure. These strain levels are significantly less than the ultimate tensile strain of 2.1% of the GFRP laminate. Again, this indicates that the failure mode was based on the masonry substrate rather than the capacity of the composite laminate.

Brick-Configuration I

Similar to the previously discussed buildings, two symmetric two-wythe brick structures were evaluated for seismic performance. Near-surface mounted (NSM) systems were used for the brick buildings, provided by Hughes Brothers, Inc. Aslan 500 #2 carbon FRP bars were used in the repaired structure, while Aslan 100 #3 glass FRP rods was used in the retrofitted structure. The epoxy resin used in both structures was Unitex® Pro-Poxy 300, as recommended by the NSM system developer. The installation of the NSM rods involved cutting a vertical groove into the shear walls at the desired locations, placing an epoxy bed into the groove, placing the coated rod/bar into the groove and filling the remaining void with epoxy resin. The grooves were cut such that they passed through head joints, and from a short distance, the composite application resembled expansion joints.

The baseline brick building depicted a well-balanced behavior between the push and pull cycles. Also, the linear portion of the curve was seen until a lateral load level of 222 kN (50 Kips). At the peak failure load, the damage to the masonry system was nearly catastrophic (see Figure 7). The large piers exhibited massive diagonal tension cracks as well as bed joint sliding in several locations. The entire front half of the structure moved away from the remaining portions, leaving the structure in two pieces. As a result, the lateral stiffness of the structure was near zero, although the gravity load was maintained at failure.

Due to the damage incurred, it was debated whether there is a good reason to spend the effort repairing the structure. In a real repair project these walls would have been demolished, and rebuilt. However, for research purposes, it was decided to repair the building and retest it. Due to the extent of the movement, the mortar joints were repointed before the CFRP composite NSM bars were installed. A total of sixteen bars were placed vertically in the shear walls, with 5 rods installed in each of the large piers, and the remainder divided between the small piers. The response of the repaired structure to the laterally applied load was similar to the unreinforced test (see Figure 8). The stiffness of the structure began to diminish at the load level of 222 kN (50 kips). At the load level of 400 kN (90 kips), the original failure surface of the unreinforced building had reappeared and the CFRP NSM bars then carried the lateral load, mostly through tension and dowel action. Failure occurred at a load level of 490 kN (110 kips) with an associated lateral displacement of 16.51 mm (0.65"). The peak load level was nearly identical to the URM brick results, and the energy dissipation for the repaired building was significantly improved. This was a remarkable result, considering the fact that the URM building was catastrophically damaged and only minimal NSM reinforcement was used.

The retrofitted clay masonry structure received GFRP NSM rods, as compared to the CFRP bars used for the repaired building. Based on the results of small component tests performed, it was decided to use only fourteen rods in the retrofitted configuration, with four rods in each of the large piers and three in each of the small piers. Fewer rods were used to promote cracking in the shear walls and thereby increase ductility. The lateral load level achieved was 534 kN (120 kips), which represents an increase of approximately 10% over the unreinforced structure. The associated lateral displacement was 5.11 mm (0.201"), which represented only a nominal increase as compared to the URM building results.

There was a notable increase in the energy dissipation of the repaired system as compared to the unreinforced test, especially in the pull cycle. The added CFRP composite bars embedded within the shear walls not only fully returned the capacity of the shear walls, but the energy dissipation of the repaired structure were far superior to the unreinforced structure. The amount of energy that could be dissipated was increased by a factor of nearly 3, as compared to the baseline structure which had a calculated strain energy of 1652 Nm (14.62 kip-in) based on the tension quadrant of the backbone curve. The presence of the CFRP bars provided an increase in the ductility and also greatly enhanced the performance of the structure.

The retrofitted structure showed increases in terms of ductility and energy dissipation. As previously mentioned, the GFRP composite system used was based on small component testing as was anticipated to provide nominal gains. Despite this, a 70% increase in ductility and 100% increase in strain energy, based on the tension quadrant of the backbone curve.

The strain measured during the tests of both the repaired and retrofitted structures were such that the ultimate tensile strain of the GFRP rods and CFRP bars was reached at failure. Prior to failure, the strain measurements were approximately 0.5% and 0.3% in the CFRP bars and GFRP rods, respectively. The strain levels are below the ultimate capacities of 1.5% and 1.9% for the CFRP bars and GFRP rods, respectively. In the case of the repaired structure, the strain level of 0.5% was maintained, while the structure was degrading and a clear slip plane had developed. Significant dowel action was observed in the CFRP bars. The mode of failure for the retrofitted structure was shearing of the GFRP rods (see Figure 9).

CMU-Configuration II-Phase I

The asymmetric CMU structure was repaired with a GFRP grid system. One layer of TechFab MeC-Grid® G15000-BX1 glass grid, with an opening size of roughly 12.7 mm x 6.35 mm (0.5" x 0.25"), was placed in a polyurea matrix provided by Bondo Corporation on each shear wall. The layout of the GFRP grid system was of a strut-and-tie configuration.

The undamaged, unreinforced structure failed under an applied lateral load of 445 kN (100 kips). The associated lateral displacement was 6.35 mm (0.25"). Due to the asymmetry of the structure, torsion was induced. The torsional displacement was 3.81

mm (0.15”) with a of 2.5 mm (0.1”) displacement variant between the shear walls. The failure mechanism for the structure was diagonal tension cracking that occurred in all piers starting at an applied lateral load of 89 kN (20 kips). The tension failure was primarily localized in the mortar joints.

The GFRP grid system drastically enhanced the performance of the damaged structure. At the point of failure, the lateral displacement was three times higher and the lateral load had increased by 25%. The flexible nature of the polyurea resin allowed the structure to achieve even more rotation under the applied lateral load. At failure, extensive diagonal tension cracking occurred in a push cycle (see Figure 10). Composite failure was observed in one of the diagonal elements, where the diagonal tension force clearly exceeded the capacity of the glass FRP grid system.

A comparison of the backbone curves, for both the unreinforced and the repaired structures (see Figure 11) reveals that the repaired structure significantly outperformed the unreinforced structure with respect to both strength and deformation. The energy associated with the cyclic lateral force was drastically increased with only one layer of the GFRP composite grid system. The associated strain energies are quite impressive, with an increase of approximately 425%, based on the compression quadrant of the hysteretic curve.

SUMMARY OF EXPERIMENTAL RESULTS

Several masonry structures were load tested under a constant gravity load and increasing cyclic lateral load. It has been shown that a minimal amount of FRP composite can drastically improve the performance of unreinforced masonry systems in both repair and retrofit applications. GFRP wet lay-up, GFRP and CFRP near surface mounted rods and bars, and GFRP grid systems were tested. An increase in strength, pseudo-ductility and energy dissipation were observed in all cases. Due to the pre-damaged (and significantly cracked) nature of the repaired buildings, these structures outperformed the retrofitted ones in terms of energy dissipation. It is important to note that, although substantial damage was produced in all building specimens, the gravity load was not lost during any testing. Since the applied gravity load modeled two additional floors, it was imperative that it be maintained, even at failure.

The externally applied GFRP wet lay-up composite applied to the symmetric CMU structures resulted in a more ductile and stronger system, as compared to the unreinforced configuration. The addition of the GFRP laminate resulted in a 67% increase and a 541% increase in strength and energy dissipation calculated by integration of the backbone curves, respectively (see Table 1). Failure modes in all cases were diagonal tension cracking in all piers. Since masonry substrate failure governed in all cases, the level of strain measured was much lower than the ultimate tensile of the GFRP laminate, ranging from 0.5% to 0.8% and resulting in a safety factor of 3-4 against composite rupture.

The catastrophically damaged symmetric brick structure was repaired using

CFRP NSM bars and regained 100% of the strength of the system. The addition of the composite bars increased the lateral displacement of the system by a factor of nearly 3.5 (see Table 2), resulting in a 300% increase in strain energy dissipation. The retrofitted structure received GFRP NSM rods, which also increased the performance of the building specimen, although not to the same degree. As with the previous structure, the method of failure was diagonal tension cracking, present in all piers of the structure. The measured strain values in both the CFRP bars and GFRP NSM rods ranged from 0.3% for retrofitted system to 0.5% for the repaired system prior to masonry failure.

Similar results were observed for the asymmetric CMU structure as compared with symmetric CMU structure. Increases of 420% in energy dissipation and 270% were measured during the test of the repaired structure. The degree of rotation of the structure was increased due to the application of the polyurea-GFRP grid composite system. Similar to the other structures, diagonal tension was the observed failure mode. At failure, composite rupture was observed at one location where the tension force exceeded the capacity of the GFRP grid.

Future research may include optimization of the FRP systems used and exploration of new systems. Since only two building configurations were examined, the possibility for a broader investigation exists as well as development of an anchoring methodology, if a positive foundation connection does not exist.

CONCLUSIONS

The experimental program has shown that FRP composite laminates can restore and significantly enhance the seismic performance of unreinforced masonry structures, in a damaged and undamaged configuration. Multiple FRP systems were utilized and all were well below ultimate strain values for each respective laminate at failure of the masonry building specimens.

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REFERENCES

Albert, M., Elwi, A. and Cheng, J.J., 2001, "Strengthening of Unreinforced Masonry Walls Using FRPs," *Journal of Composites for Construction*, May, V. 5, No. 2, pp. 76-84.

Al-Chaar, G.K. and Hassan, H.A., 1999, May 10-12, "Seismic Testing and Dynamic

Analysis of Masonry Bearing and Shear Walls Retrofitted with Overlay Composite," *Proceeding of ICE '99*, Cincinnati, OH, 1999 (Session 4-B), pp. 1-10.

Ehsani, M.R. and Saadatmanesh, H., 1996, "Seismic Retrofit of URM Walls with Fiber Composites," *The Masonry Society*, December, V. 14, No. 2, pp. 63-72.

Federal Emergency Management Agency, November 2000, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356, Section 2.8.3 pg 2-28, Washington, D.C.

Gergely, J., and Young, D.T., October 2001, "Masonry Wall Retrofitted with CFRP Materials," *Proceedings of the 1st International Conference on Composites in Construction*, Porto, Portugal, 2001, Figueiras et al (eds), Swets and Zeitlinger, pp. 565-569.

Hamid, A.A., El-Dakhakni, W.W., Hakam, Z.H. and Elgaaly, M., 2005, "Behavior of Composite Unreinforced Masonry – Fiber-Reinforced Polymer Wall Assemblages Under In-Plane Loading," *Journal of Composite for Construction*, V. 5, No. 1, pp. 73-83.

Hamilton III, H.R. and Dolan, C.W., 2001, "Flexural Capacities of Glass FRP Strengthened Concrete Masonry Walls," *Journal of Composites for Construction*, V. 5, No. 3, pp. 139-145.

Hamoush, S., McGinley, M., Mlakar, P., Scott, D. and Murray, K., 2001, "Out-of-plane Strengthening of Masonry Walls with Reinforced Composites," *Journal of Composites for Construction*, August, V. 5, No. 2, pp. 139-145.

Marshall, O.S, Sweeney, S.C. and Trovillion, J.C., 1999, "Seismic Rehabilitation of Unreinforced Masonry Walls," *Proceedings of the Fourth International Symposium on Fiber Reinforcement for Reinforced Concrete Structures*, Batlimore, MD, 1999 (SP188-26), American Concrete Institute, pp. 287-295.

Moon, F.L., Yi, T., Leon, R.T. and Kahn, L.F., 2002, September 9-13, "Seismic Strengthening of Unreinforced Masonry Structures with FRP Overlays and Post-tensioning," *Proceeding of the 12th European Conference on Earthquake Engineering*, London, England, 2002 (Paper Reference 123), pp. 10.

Silva, P., Myers, J., Belarbi, A., Tumialan, G., El-Domiaty, K., and Nanni, A., 2001, July 4-6, "Performance of Infill URM Wall Systems Retrofitted with FRP Rods and Laminates to Resist In-Plane and Out-of-Plane Loads", *Proceeding of Structural Faults and Repairs*, London, UK, 2001, pp. 1-12.

Table 1 – Energy dissipation (ε-energy) comparisons throughout experimental testing. (Units: N-m [k-in])

Structure	Side	Unreinforced	Repaired	% Increase	Retrofitted	% Increase
Brick-Symm	Comp.	1365 [12.08]	3384 [29.95]	148	1845 [16.33]	35
	Tension	1652 [14.62]	6538 [57.87]	296	3342 [29.58]	102
CMU-Symm	Comp.	1711 [15.14]	10960 [97.0]	541	5511 [48.78]	222
	Tension	1249 [11.05]	6843 [60.57]	448	2616 [23.15]	110
CMU-Asymm*	Comp.	1685 [14.91]	8779 [77.7]	421	---	---
	Tension	2281 [20.19]	9711 [85.95]	326	---	---

*Based on displacement measurements taken from perforated shear wall.

Table 2 – Pseudo-ductility comparisons of unreinforced, repaired and retrofitted structures. (Units: mm [in])

Structure	Side	Unreinforced	Repaired	% Increase	Retrofitted	% Increase
Brick-Symm	Comp.	4.43 [.18]	8.63 [.34]	95	5.29 [.21]	19
	Tension	5.02 [.20]	17.29 [.68]	244	8.63 [.34]	72
CMU-Symm	Comp.	6.05 [.24]	22.34 [.88]	269	12.28 [.48]	101
	Tension	4.65 [.18]	14.27 [.56]	207	7.21 [.28]	58
CMU-Asymm*	Comp.	5.49 [.22]	20.40 [.80]	272	---	---
	Tension	7.09 [.28]	20.0 [.79]	182	---	---

*Based on displacement measurements taken from perforated shear wall.

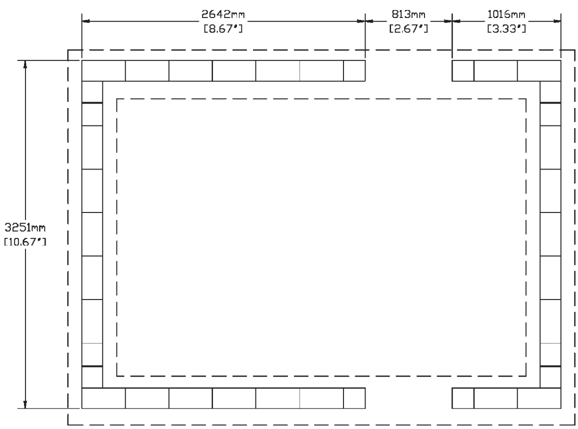

Figure 1 – Plan view of the symmetric configuration (asymmetric not shown).

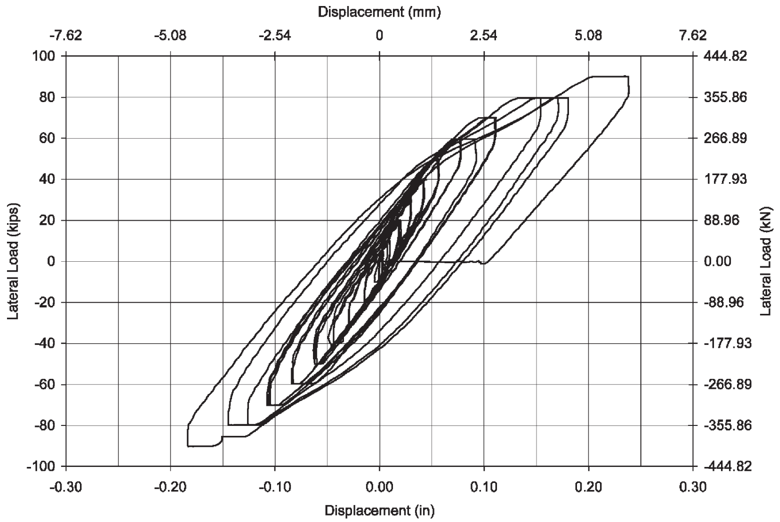

Figure 2 – Hysteretic curve for the unreinforced, symmetric CMU structure.

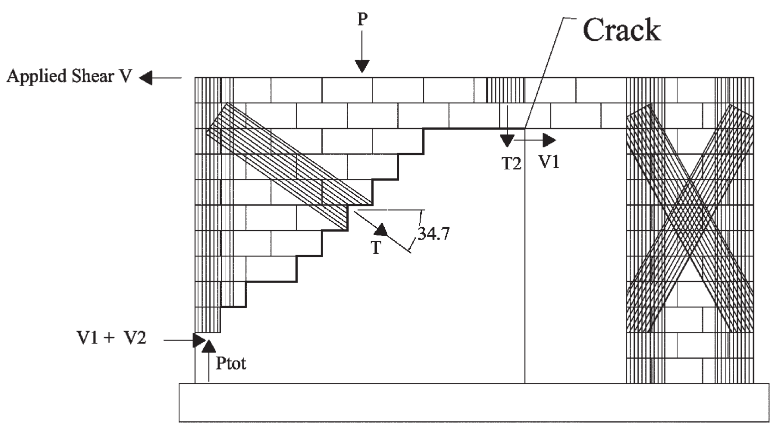

Figure 3 – Free body diagram from determination of composite layout using a strut and tie configuration.

Figure 4 – GFRP composite layout for the repaired symmetric CMU structure.

Figure 5 – Damage sustained during experimental load testing of the symmetric retrofitted CMU structure.

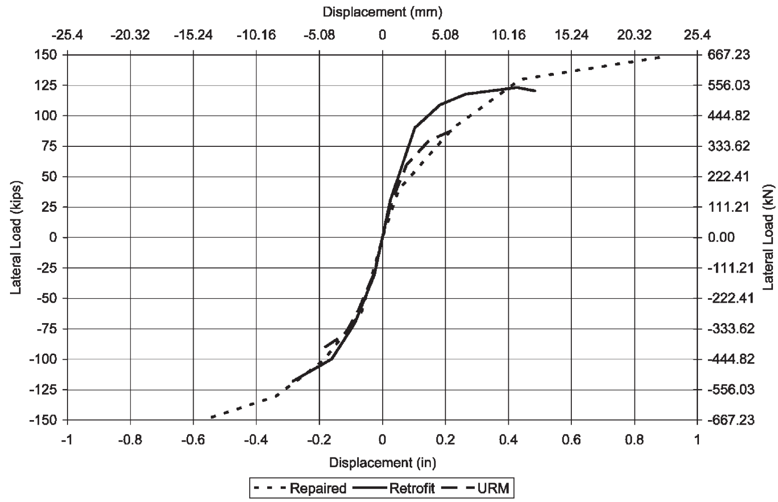

Figure 6 – Idealized backbone curve for the unreinforced, repaired and retrofitted symmetric CMU structures.

Figure 7 – Damaged incurred during experimental testing of the symmetric unreinforced brick structure.

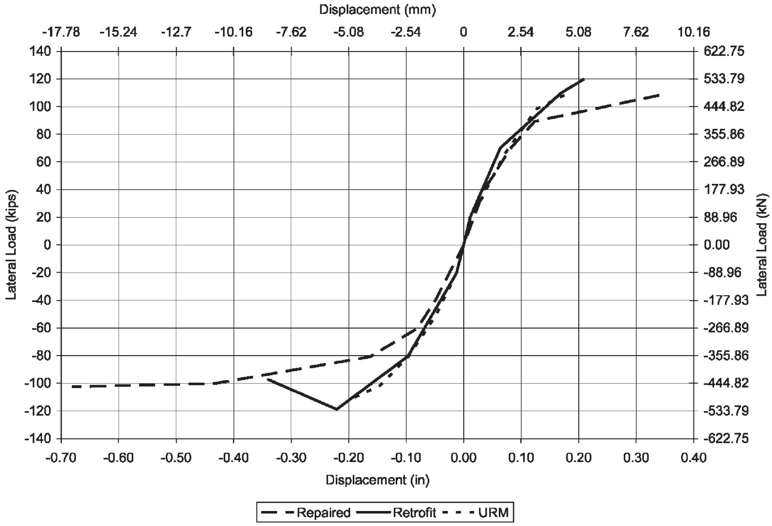

Figure 8 – Backbone curves for unreinforced, repaired and retrofitted symmetric brick structures.

Figure 9 – Diagonal tension failure and GFRP NSM rod rupture of the symmetric retrofitted brick structure.

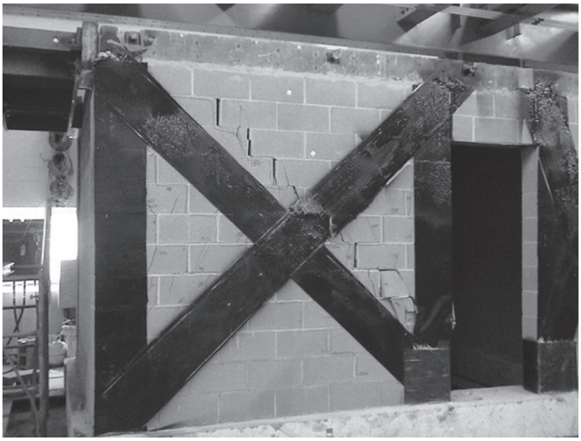

Figure 10 – Damaged induced by an applied 556 kN (125 kips) lateral force to a repaired asymmetric CMU structure.

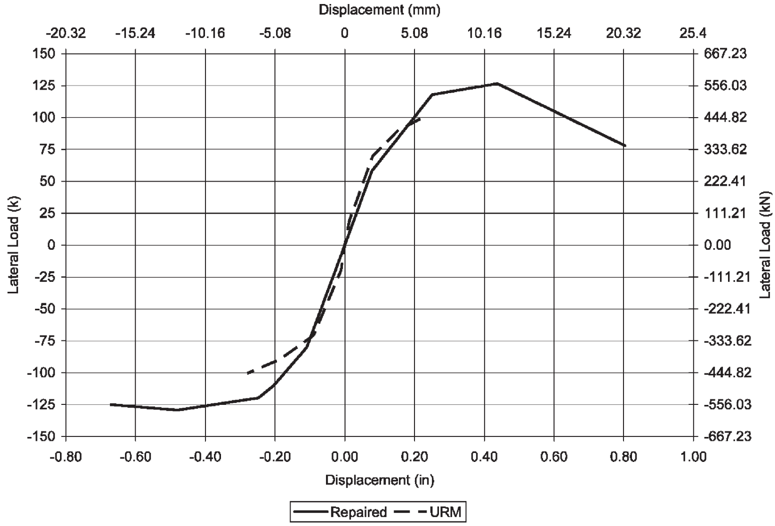

Figure 11 – Backbone curve comparison for asymmetric CMU structure in unreinforced and repaired condition.