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PERFORMANCE OF MASONRY WALL RETROFITTED USING CFRP SHEETS AND ANCHORS

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Abstract

Different strategies can be used to repair, rehabilitate and strengthen existing structures. Techniques based on Carbon Fiber Reinforced Polymer (CFRP) materials appear to be innovative alternatives to traditional solutions because of their high tensile strength, light weight, and ease of installation. Due to bending and/or shear effects, CFRP sheets may lose adherence with the concrete surface, however the use of CFRP anchors allows reaching their full tensile capacity. Sliding between courses occurs often when the masonry wall interface has no internal reinforcement. A masonry wall with hollow concrete blocks with internal reinforcement was tested under constant axial load and cyclic lateral loads and rotation was restrained at top and bottom to produce double curvature deformation. The masonry wall length and height were 72in and thickness 7.6in. Diagonal shear cracks, toe crushing of the bottom of both corners of the masonry wall, and sliding at the base were presented. The rehabilitation of the masonry wall was divided into two phases. The first phase consisted of adding a reinforced concrete ring above the base foundation to encase the two badly-damaged bottom courses of CMU, reducing the aspect ratio L/H from 1 to 0.78. The partial retrofitted wall was tested to assess its condition. The second phase consisted of CFRP sheets attached on the wall surface with the strip aligned a diagonal direction to produce a tension brace or tie, being the CFRP sheets attached to the wall surface by epoxy and CFRP anchor which were installed at the ends of the sheets. Each diagonal tie had different amount of CFRP material in order to evaluate two different repair patterns. It was found that the rehabilitated masonry wall was less stiff than the existing elements even though the aspect ratio was reduced because the concrete ring applied at first phase of rehabilitation. However the retrofitted masonry wall had higher shear capacities, increasing more 60% compared with the as-built wall; and the values of drift at peak load were comparable. It was noticed as well considerable sliding displacement occurred at the top of the strengthened wall. Sliding at top was about 50% of the total lateral displacement, shorter that 70% from the as-built wall. Sliding at bottom of the wall was not significant. Performance of the masonry wall and the applied CFRP materials during the laboratory tests are explained.

Keywords: Carbon Fiber Reinforced Polymer, Masonry Wall, Innovative Retrofit, CFRP Anchor

1. Introduction

Masonry walls are frequently damaged by natural disasters such as earthquakes or tsunamis. The pattern of damage commonly seen in walls is shear cracking, and sliding between the wall foundation interfaces or between the courses of the wall. Sliding between courses occurs often when the masonry wall interface has no internal reinforcement. The purpose of this paper is to present innovative retrofit techniques applied to extremely damaged masonry walls, where CFRP sheets, CFRP anchors and CFRP patches are used.

2. Objective

The objective of this program was to evaluate the effectiveness of rehabilitation of a severely damage reinforced masonry wall with CFRP materials to improve the shear capacity of the wall and reduce sliding at the top and bottom interfaces of the wall and the reinforced concrete structure.

3. Experimental Program

3.1 Previous Retrofit

The as-built masonry wall consisted at 9 courses of concrete masonry units (CMU) with a total height of 72 in, and its total length of 71.6in. Measured dimensions of the masonry unit were 15.6x7.6x7.6in and a 1.25in wall thickness; compressive capacity of a grouted 8x8x8in CMU was 3.5ksi; for a grouted prism formed by 2 blocks 3.1ksi, having also compressive strength for grout and mortar between courses were 6ksi and 4ksi respectively. The wall was reinforced internally with bar Grade 60, consisting in #4 bars spaced vertically at 8in through the hollow cores in the concrete blocks and grouted; one #4 horizontal bar was placed between each course of the wall to form a mesh of #4bars @ 8in. This masonry wall was constructed by Ahmadi [1].

The wall called RMW was tested under constant axial load and cycle lateral forces using a test setup which restrain the specimen to rotate at top and bottom resulting in double curvature deformation as shown in Fig.1. The test setup consisted at three hydraulic actuators, L-steel frame formed by welded ASCE structural beam shapes. An extra steel shape and one concrete bed spacers were installed in other to fit the wall on the test setup. Details for the test setup and instruments are in Leborgn[2] and Huaco[3]

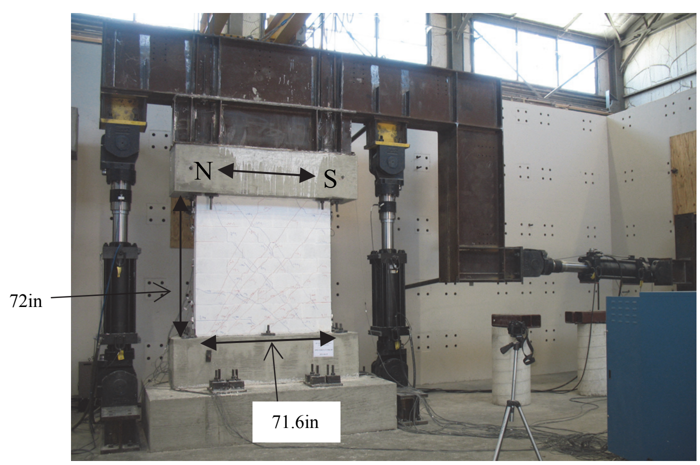

Fig. 1 – Test setup for the fix bottom – top rotation restrained masonry wall RWM [3]

The constant axial load applied was 140kips before begin to push and pull the specimen. After yielding of the vertical bars and later the shear capacity specimen, diagonal cracks formed in the wall and large sliding deformations were noted at the bottom of wall. The crushing occurred in the bottom in two courses of the wall.

The vertical bars in the wall were bent due the high shear deformation. After the wall began to crush, the specimen was not able to support the 140kips axial load producing a collapse at 1.4 in. lateral deformation (2% drift ratio). The shear capacity degraded from maximum presented 175kips to 80kips loading north and from 171kips to 61kips loading south, less than half the peak capacity. Sliding at the base was the predominant feature of behavior at large deformations, and represented more than 70% of the total deformation of the specimen Sliding at the top was less than 1% of the total deformation of the specimen. The maximum displacement to north without the sliding is 0.39 in. (0.54% drift ratio) and 0.43 in. (0.6% drift ratio) to south direction. Fig.2 shows the damaged wall and failure patterns such as the diagonal cracks and the crushing on inferior corner toes.

Fig. 2 – Final condition of the masonry wall, diagonal cracks crossing the field of the wall and sliding at the base (left) and toe crushing of the inferior right corner of the masonry wall (right) [3]

3.2 Retrofit Process and Laboratory Test

The rehabilitation of the masonry wall was divided in two phases. The first phase consisted of adding a reinforced concrete ring above the base foundation to encase the two badly-damaged bottom courses of CMU, therefore this concrete ring reduced the height of the wall. In the second phase, CFRP sheets were attached to the wall along the diagonals to produce a tension brace or tie, and. CFRP anchors were installed to at the ends of the sheets. It can be seen in Fig.3 the scheme for two phases of the repair process including the location of the concrete ring for the 1st Phase, and the CFRP diagonal sheets, the anchors and patches for 2nd Phase.

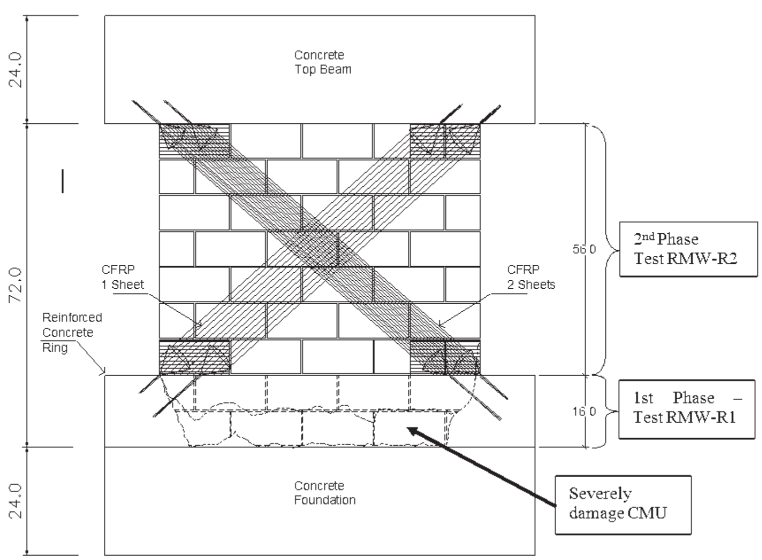

Fig. 3 – Details for the retrofit of the masonry wall [3]

3.2 First Phase of the Masonry Wall Retrofit

3.2.1 Application of the Concrete Ring

A reinforced concrete beam was cast to encase the original base and two bottom CMU courses that were severely damaged. Some damage to the end CMU wall in third course of the wall was repaired with a mortar patch. The concrete beam or ring also eliminated sliding at the base. The concrete ring reduced the clear height of the masonry wall from 72 in. to 56 in., and the aspect radio from 1 to 0.78. Internal reinforcement was provided in the concrete ring as shown in Fig.4 to connect the concrete foundation of the original masonry wall the new concrete ring. Eight #5 Grade 60 steel bars were anchored into holes drilled in the concrete foundation. Also 4 #5 bars were placed horizontally through the 2nd course level in order to tie the masonry wall to the concrete. The application process for this 1st Phase is shown at Fig.5, having cleaned the two first CMU courses with the drilled holes, then the internal reinforcement applied being ready for the pouring of concrete for the ring.

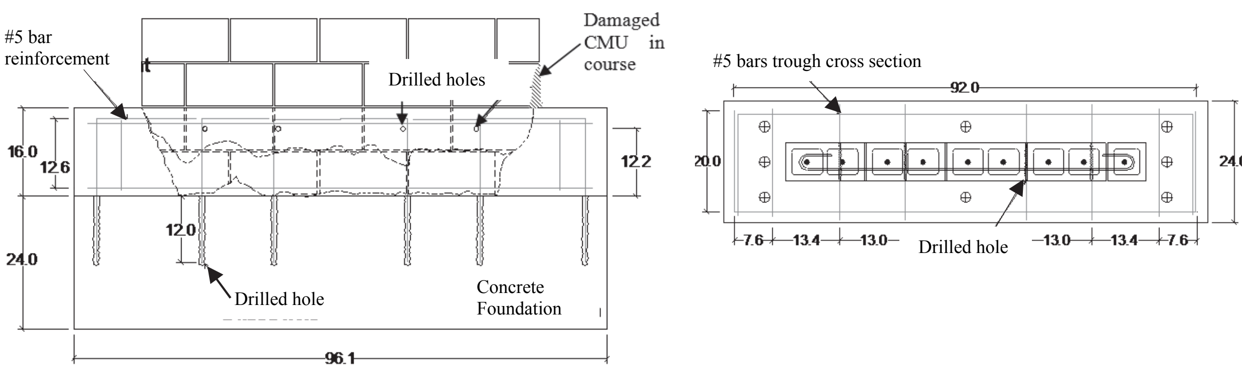

Fig. 4 – Internal reinforcement for the concrete ring [3]

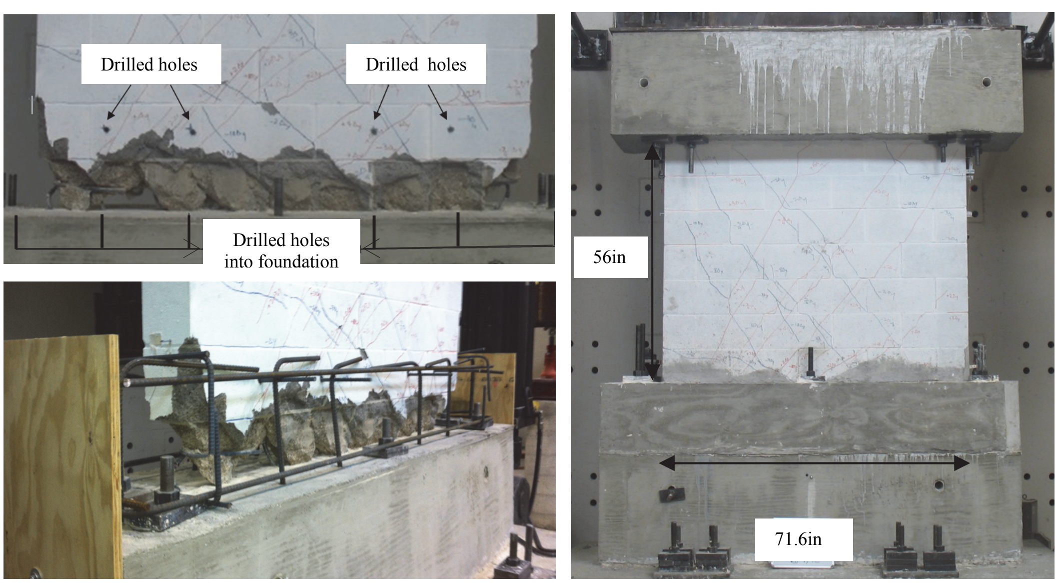

Fig. 5 – Application of the concrete ring [3]

3.2.2 Test of the retrofitted masonry wall at First Phase – RMW-R1

After the 140kips axial load was applied, lateral deformations to 0.045in were imposed in both directions. Diagonal cracks opened to a width of 0.020in. The measured lateral load was 70kips. For the second cycle of the test, the existing cracks opened to 0.025 in. and load reached was 84 kips in the south direction and 99 kips to the north. The peak displacement in both directions was 0.073 in. In the third cycle, the lateral load reached 119 kips at 0.10 in. south displacement and 118kips for 0.12in to north displacement. The diagonal cracks opened to 0.030 in. The behavior of the specimen was nearly linear, the stiffness in both directions was observed similar. It can be concluded that the pattern of damage for both directions was the same. The stiffness calculated was 1060kip/in for pushing (north) displacement and 1190 kip/in for pulling displacement (south).

The retrofitted masonry wall was tested again in order to review the parameters obtained and to also check the sliding deformation between the concrete beam and the CMUs. The test consisted of one lateral cycle displacement to 1.25 in. in both directions. The maximum lateral loads were 121 and 119kips in the north and south directions. The lateral stiffness measured was 970 kip/in and 1003 kip/in for north and south direction respectively. Load-deformation is plotted in Fig.9. The test was stopped when the specimen was returned to its initial position. After the axial load applied was removed, the specimen movement to the north direction by 0.066in and the lateral load dropped to 23 kips. The specimen was pulled 0.066 in. south to its initial position.

It was observed that the actual stiffness of RMW-R1 is less compared from the test of the as-built specimen (RMW), even when the masonry wall retrofitted had lower aspect ratio (from 1 for the as-built case to 0.78 for the retrofit case). The diagonal shear cracks represented important keys to work for rehabilitation purposes as well. Results of the test indicated that the secant stiffness loading north was similar than loading south direction indicating that the level of damage was the same for both directions of loading. The secant stiffness

3.3 Second Phase of the Masonry Wall Retrofit

3.3.1 Application of CFRP sheets, anchors and patches

CFRP sheets were attached to the wall along the diagonals to produce a tension brace or tie. One diagonal tie consisted in 9”wide two layers of CFRP sheets at one direction and 12”in wide-one layer for the other direction. CFRP anchors were installed to at the ends of the sheets. The repaired wall was tested using the same deformation pattern and constant axial load than the as-build specimen. The angle of inclination of the diagonal CFRP strips was about 40°.

The Two 9in. wide layers of CFRP diagonal ties provided a tension brace produced for loading to the north. The CFRP diagonal ties provided tension resistance to the strengthened masonry wall. The total amount of CFRP material was 4 strips 9 in. wide and 0.04 in. thick, with a total transverse area of 4x9x0.04= 1.44sq.in. Both CFRP diagonal ties were also attached by a total of 4 CFRP anchors in their extremes and wrapped by U-patches. The one 12in. wide layer of CFRP diagonal ties provided a tension brace for loading to the south. The total amount of CFRP material was 2 strips of 12 in. wide and 0.04in. thick, having a total transversal area of 2x12x0.04=0.96sq.in. being 50% less than the other diagonal ties. The CFRP diagonal ties were attached the base and the upper load beam by a total of 4 CFRP anchors. Fig. 6 shows dimensions of the installed CFRP materials to each face of RMW-R1.

The calculation of the resistance capacity of the CFRP sheets as a tension tie and CFRP anchors under pulling and shear forces are explained at Ozdemir and Akyuz [4] and Huaco[3]. The force in the tension tie was transferred from the CFRP sheet to an anchor that extended into the supporting members at the top and bottom of the wall. For the CFRP anchors, resistance was determined by the tension strength of the anchor and adherence between the interior surface of the hole and the anchor.

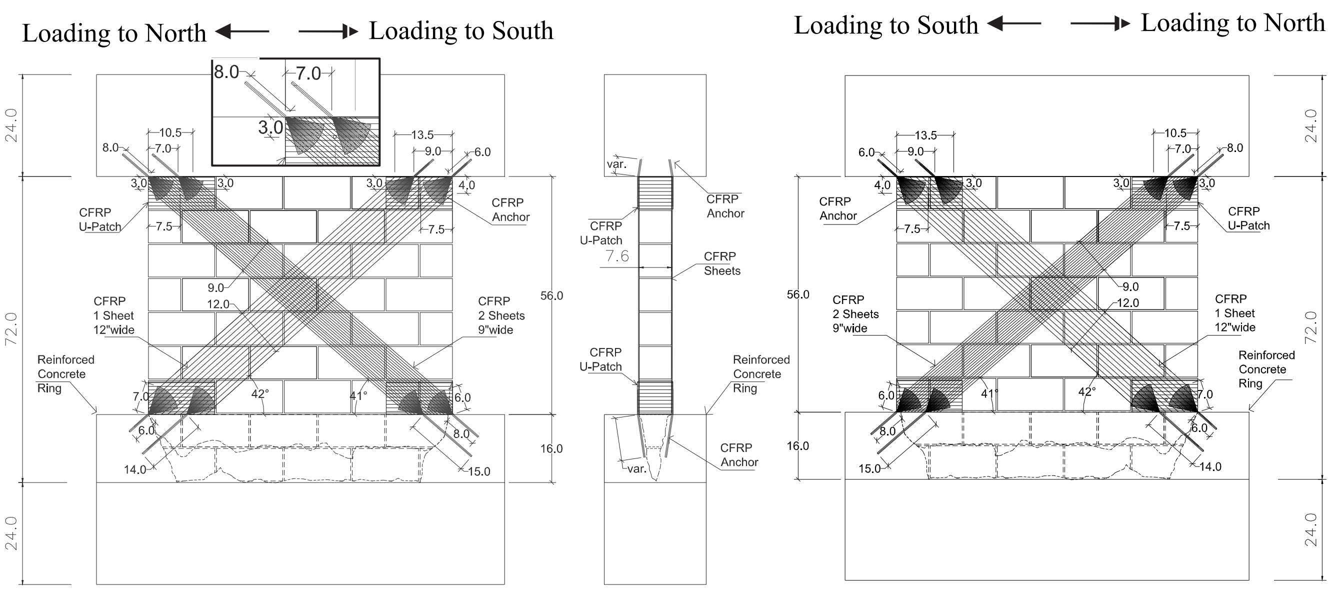

Fig. 6 – Front, side and back view (left, center and right respectively) of the details of the application of the CFRP materials.

For optimal use of CFRP materials, it is very important that strict quality control be carried out in the application of the CFRP. Without high quality installation, the capacity of the CFRP is compromised and may not reach the capacity desired. See Fig.7 for installing process of the CFRP material. Grinding the surface for an appropriate adherence of the CFRP sheet to the CMU surface (Fig.7.a), drilling the holes for the application of the CFRP anchor (Fig.7.b) and the installation of the CFRP diagonal sheets (Fig.7.c), the installed CFRP anchors previous the application of the patch (Fig.7.d) and the final shape of the retrofitted wall (Fig.7.e) are shown. During the application of CFRP material a roller was used, after the CFRP sheets were placed the roller was used again to ensure saturation of the epoxy into the CFRP sheets. The roller also removed air pockets between the surface of the wall and the layers of CFRP. The CFRP anchors were installed after the diagonal CFRP sheets were installed. The internal surface of the drilled holes was saturated with epoxy prior to inserting the CFRP anchors. For the holes into the concrete ring, half of the hole was filled with epoxy. However for the holes into the top concrete beam, the surface was saturated with epoxy. CFRP strips for the anchors also were saturated with epoxy before installing them into the holes. A small aluminum bar was used to push the CFRP anchor into the hole. The remaining portion of the anchor was spread out forming the triangle shaped fan shown in Fig.7.d, and the roller was used to coat the fan with epoxy then the CFRP patches were placed at each inferiors and superior corners covering the applied anchors.

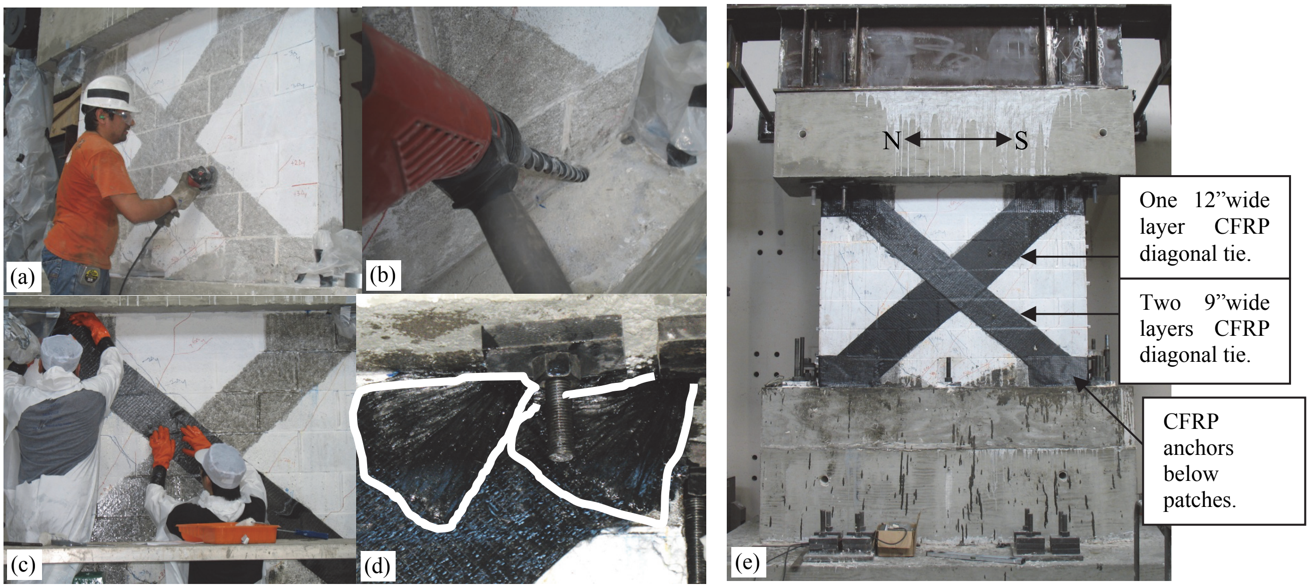

Fig. 7 – Installation process of the CFRP materials to the damaged masonry wall.

3.3.1 Test of the retrofitted masonry wall at Second Phase – RMW-R2

The axial load applied during the experiment was 140kips. Protocol of load was following the procedures of FEMA 461 until 0.80% drift ratio for loading to north direction, which is above the maximum allowed drift recommended in the ASCE-07-10 seismic design provisions for masonry walls.

The retrofitted masonry wall behaved linearly for to deflection of 0.11 in. in both direction of loading as and no sliding was observed. The measured load was 136kips to north and 130kips to the south. The measured secant linear stiffness to replace the peak load was 1236kip/in for the north direction and 1.182 kip/in for the south direction. The existing crack widths did not increase. No pinching was observed. However, in sudden pops indicated the beginning of the debonding of the CFRP strips.

Nonlinear response was noted at about 0.12 in. lateral deformation in both directions, with a gradual degradation of the stiffness, besides more 'popping-in' was heard indicating further debonding of the CFRP sheets.

The maximum lateral load of 284kips under loading to north was reached at a displacement of 0.45 in. When the specimen was loaded to south, the test was stopped because the maximum tension load of the lateral actuator was reached measuring at 191kips and the lateral displacement was 0.23 in. Next cycle of loading significant sliding at top of the masonry wall was noted. No sliding between the concrete ring and the CMUs of the masonry wall was observed. Loading in the south was reversed when the capacity of the actuator (200kips) was reached. Larger displacements to south occurred in each cycle due slipping at the top of the wall.

At second last loading cycle, the lateral load varied suddenly. This variation was due to a drop axial from 140kips to about 100kips. The axial load was increased to 140kips and the test continued. The maximum lateral displacement to the south was 1.04 in. and the shear force measured was 191kips. There was a gradual degradation of stiffness and shear capacity under loading to the north due to sliding effects, debonding of the CFRP material, and yielding of the reinforcement in the wall. At next and last cycle failure occurred loading to north. The top CFRP anchors located at the north end of the wall fractured. Detail of this mechanism of failure by the anchor rupture is explained on further paragraph. The maximum displacement reached was 1.76 in. and the shear measured was 128kips. Before failure, there was a drop in the axial load of 20kips when the CFRP anchor ruptured, producing a drop in the shear force from 204kips to 128kips and an increase in the lateral displacement from 1.40 to 1.76 in. due sliding at the top of the masonry wall.

The existing crack in the bottom-left corner of the wall opened when the specimen was loaded to the south. This crack was located close to the end of the CFRP tension tie as it can be seen at Fig.8. Additionally, Fig.8-right shows the existing crack located on the bottom-right corner close to the edge of the U-wrap. The crack opened when the specimen failed loading to north.

Fig.9 shows the comparisons of shear force vs. drift ratio responses for specimens RMW, RMW-R1 and RMW-R2 at linear range where the 1st Phase of retrofit was tested, seeing that reduced stiffness for both retrofitted phases. Fig10 shows the responses for RMW and RMW-R2 including the behavior due sliding showing the increasing of shear force capacity and the comparable lateral deformation.

Fig. 8 – Existing cracks opened after reach maximum south (left) and north (right) lateral displacement.

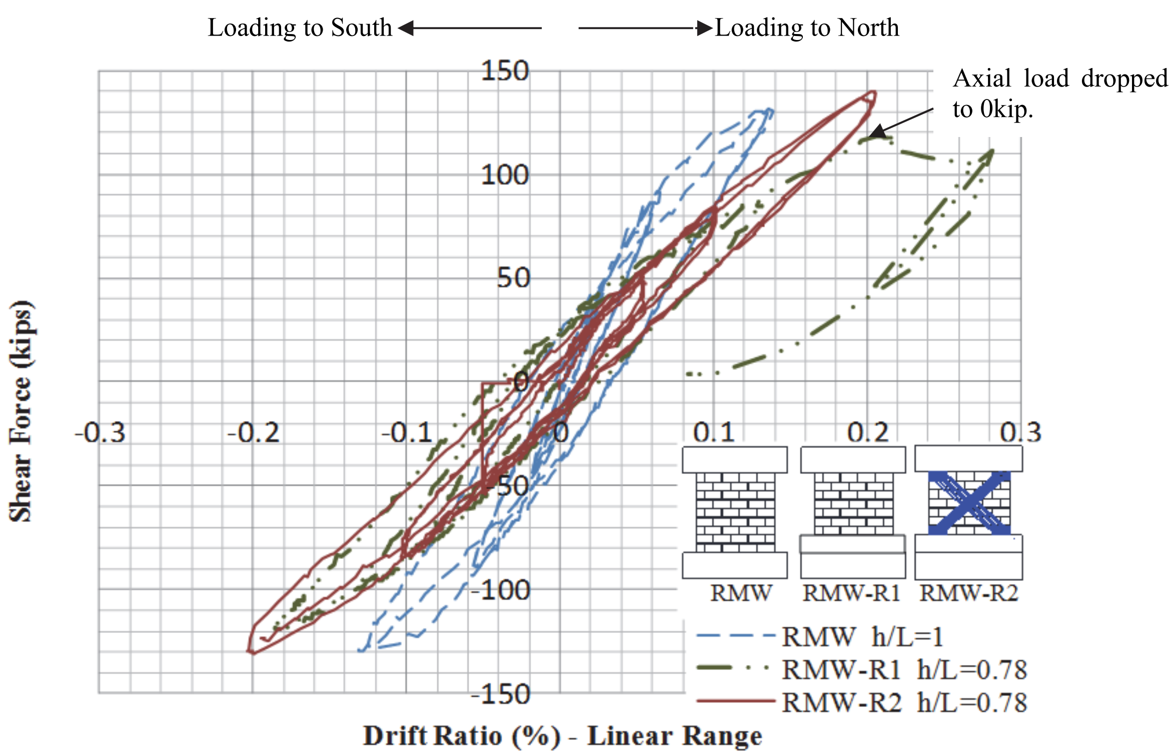

Fig. 9 – Shear force vs. drift ratio on linear range of RMW, RMW-R1 and RMW-2

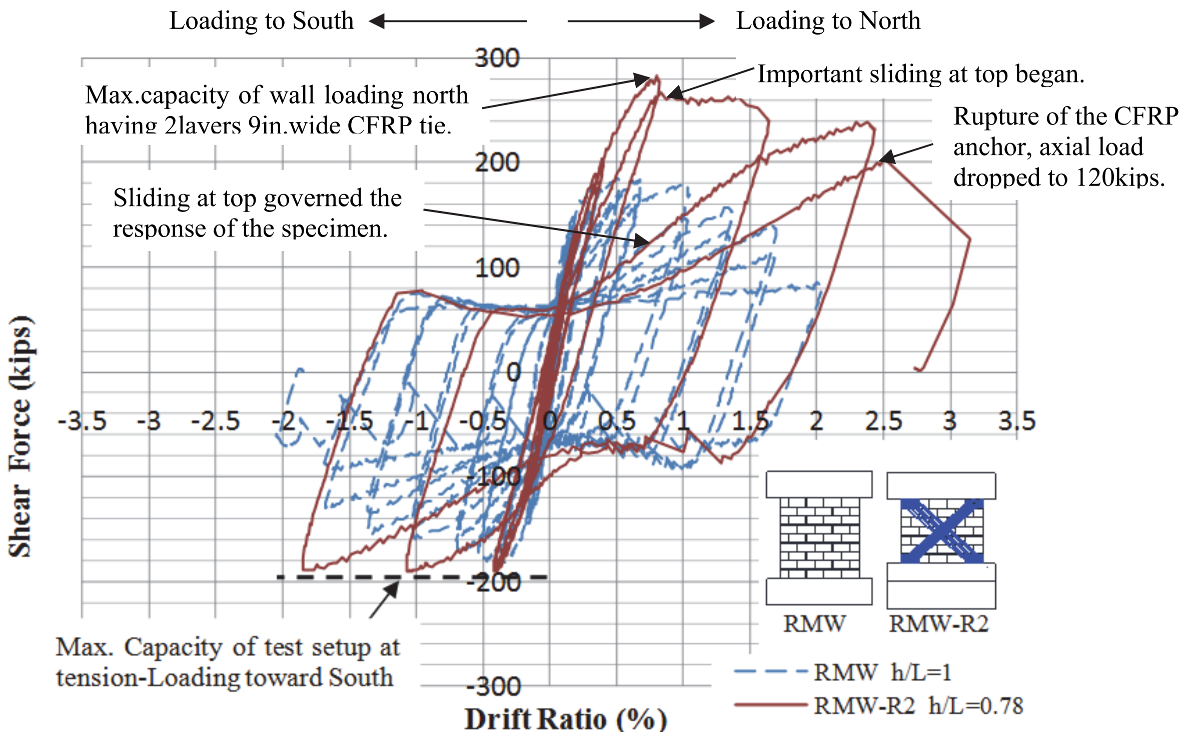

Fig. 10 – Shear force vs. drift ratio of RMW and RMW-R2.

CFRP diagonal strips one 12" wide layer CFRP diagonal strips worked as axial tension brace when the masonry wall was loaded to the south. Debonding of the CFRP diagonal strips appeared at the beginning of the non linear behavior of the wall. It was applied 4 strain gages per CFRP tie, two at half top and two at half bottom of the ties. The maximum strain was reached in tension, 0.0023 and 0.0036 for the front and back face of the wall respectively. Compression strains reached 0.002 (20% of the maximum tensile strain capacity). This was measured on the lower strain gage applied on the CFRP diagonal strip. The strain deformation in compression was larger for the bottom part of the CFRP diagonal strip, indicating that there was a major concentration of stress at the bottom tie. It is observed that the strain gages near the bottom exhibited larger strain than the gages near the top. Buckling of the CFRP tie was observed too (Fig.11).The sliding observed was 0.88in and the CFRP anchors close to the rupture. It was observed that CFRP anchor carried out load from sliding.

CFRP diagonal strips two 9" wide layers worked as axial tension punctual brace when the masonry wall was loaded to the north. After the CFRP 12"wide started to debond, some noise was heard indicating the beginning of the debonding for that CFRP two 9"tie. The CFRP anchors transferred tension from the CFRP diagonal strips to the top and bottom support of the wall. The maximum strain measured was 0.0052 on the front face of specimen which is slightly above 50% of the strain capacity of the CFRP according to the producer's specification. The maximum strain deformation measured on the CFRP strip on the back face was 0.0039. Compressive strains in the CFRP strips were measured under loading to the south. At the second last cycle of the test, the CFRP strip buckled .The maximum strain in compression prior to buckling was 0.0016. However, when the buckling occurred CFRP on front face of the wall, the compressive strain was 0.0027 and 0.0014 on the back face of the wall. It was observed also that the strain measured was longer on the bottom part of the diagonal ties, having concentration of compression stress on this region. Buckling of the CFRP tie was noticed as well (Fig.11). Failure was due to rupture of the CFRP anchors installed on the top north corner of the masonry wall (Fig.12). Those CFRP anchors provided restraint against the sliding between the wall line and the top concrete beam. The CFRP anchors carried the axial load from the debonded CFRP diagonal ties (2 x two 9in.CFRP sheets), and a tension due to the correspondent diagonal component of the force produced during sliding. The sliding measured at failure was 1.24 in.

It was noticed during the test that the CFRP anchor provided resistance against the sliding at top of wall. There are two types of forces applied to the anchor. They are: pulling loads due the tension load from the diagonal ties, and the sliding forces due the lateral load from the top concrete beam. Each of these forces is divided by the correspondent number of anchors. Using the parallel and perpendicular forces component respectively to the direction of the anchor, the CFRP anchor resists two types of load: pulling load (parallel to the anchor's direction) and shear force (perpendicular to the anchor's direction). It is not considered the dowel effect on the sliding.

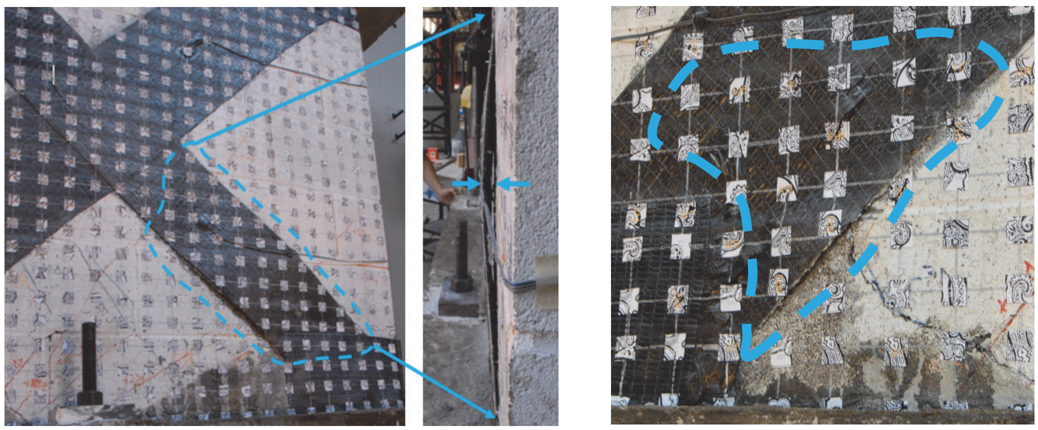

Fig. 11 – Buckling presented on 9in.wide CFRP sheets (left), and on 12 in.wide CFRP sheet (right)

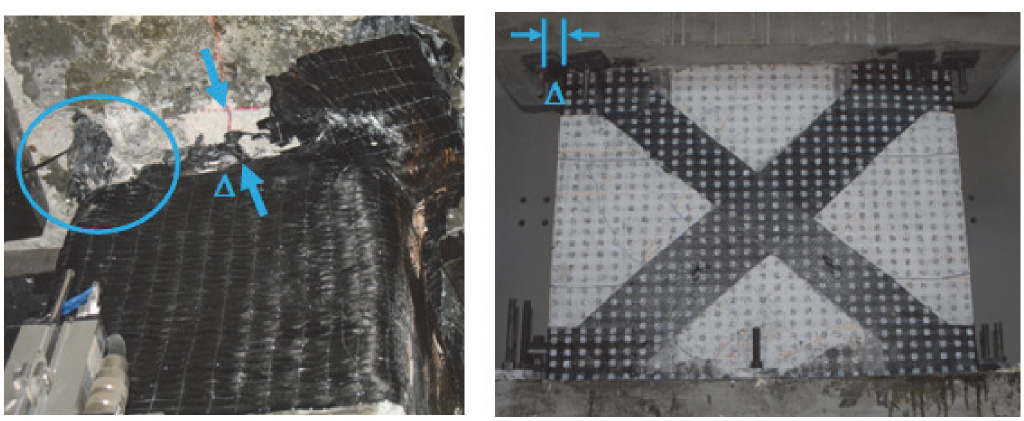

Fig. 12 – Amount of slip following rupture of CFRP anchors on 9in wide CFRP sheet ties. Same pattern presented on 12in wide CFRP sheet ties without rupture of corresponding CFRP anchors.

Fig.13 shows the response of the specimen without sliding at bottom and top of the masonry wall. The overall nonlinear response of the specimen was governed primarily by sliding at the top. Considerable sliding displacement occurred at the top of the strengthened wall. Sliding at top was about 50% of the total lateral displacement. Sliding at bottom of the wall was not significant. The maximum shear capacity prior to sliding was 284kips at 0.8% drift. Failure occurred due to rupture of the CFRP anchors installed at the top of the masonry wall at a lateral load 128kips and the maximum drift ratio was 3.1%.

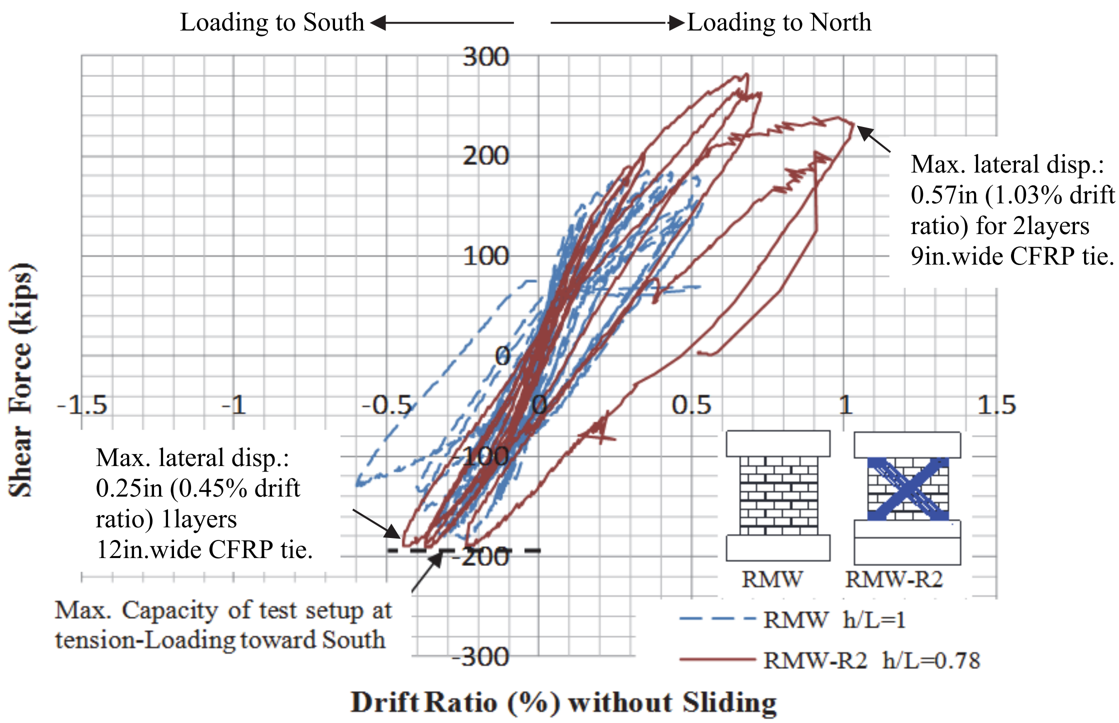

Fig. 13 – Shear force vs. drift (without sliding) of RMW and RMW-R2.

5. Conclusions

The research represents a unique project for repair of severely damaged masonry wall efficiently and with attractive cost-time benefits. The proposed rehabilitation methods can be implemented rapidly and may cost less than others traditional techniques.

The proposed rehabilitation methods performed well. Higher shear capacity and larger displacement were reached using the methodologies proposed in this research. Besides sliding provides energy dissipation that allows the structure remain properly. However actual shear capacity of the wall cannot be predicted because sliding. CFRP anchors predict a no occurrence of sliding until the anchor resistance level. Above that force, sliding may occur.

CFRP anchors had a very important role for the resistance of the masonry wall against its sliding at top. The structure may not collapse however non-structural elements may be damaged rendering the structure unusable. Structures such as hospitals that have many pipes and equipment installations on their walls would be most vulnerable. A possible solution to avoid wall sliding is the addition of CFRP anchors. The response of the strengthened wall demonstrated that development of the required tension force in the diagonal CFRP brace can be achieved, and the prevention of sliding depends entirely on the CFRP anchors.

The ultimate drift ratio of the test for the retrofitted masonry wall, an acceptable drift of 0.70% prior to sliding was reached correspond for essential facilities defined in ASCE07-10 occupancy category IV.

5. References

[1] Ahmadi (2012).: Displacement-based Seismic Design and Tools for Reinforced Masonry Shear-Wall Structures. *Ph.D. Disertation The University of Texas at Austin. USA,* 692pp.

[2] LeBorgne M (2012).: Modeling The Post Shear Failure Behavior Of Reinforced Concrete Columns. *Ph.D. Disertation The University of Texas at Austin. USA,* 301pp.

[3] Huaco, G (2013): Procedures to Rehabilitate Extremely Damaged Concrete Members using Innovative Materials and Devices. *Ph.D. Disertation The University of Texas at Austin. USA,* 649pp

[4] Akyuz, O. and Ozdemir, G., (2004). "Mechanical Properties of CFRP Anchorages", *13th World Conference on Earthquake Engineering,* Vancouver, B.C., Canada.