Seismic Retrofit of Load Bearing Masonry Walls by FRP Sheets and Anchors

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ABSTRACT
Most of the old masonry buildings are made of either unreinforced or partially reinforced walls. These walls suffer significant damage in the event of a strong earthquake. Therefore, retrofitting of masonry buildings built prior to the enactment of modern seismic codes plays an important role in seismic risk mitigation. The effectiveness of carbon fiber-reinforced polymer (FRP) sheets and anchors in retrofitting low-rise load bearing masonry walls has been investigated experimentally and the results are reported in this paper. A partially-reinforced concrete block masonry wall specimen was designed and built as representative of a single-story low-rise masonry wall. The wall was retrofitted with carbon FRP sheets and anchors to enhance lateral load capacity in flexure and shear. The retrofit methodology was applied only on one side of the wall to represent a frequently encountered constraint in practice, where the wall may not be accessible from both sides for strengthening. FRP anchors, developed at the University of Ottawa, were used to prevent premature debonding of FRP sheets. Additional FRP anchors were used to anchor the FRP sheets to the foundation for flexural strength enhancement. The wall was tested under constant gravity load and incrementally increasing in-plane deformation reversals. The test results showed that epoxy bonded FRP sheets could be used effectively for flexural and shear strengthening masonry load bearing walls. The FRP-retrofitted specimen resisted higher lateral forces and developed higher drift levels in comparison with the unretrofitted control specimen.

Keywords: Masonry Wall, Retrofitting, Fiber Reinforced Polymer (FRP), Earthquake, Anchor

1. INTRODUCTION

Single-story masonry buildings such as schools, hospitals and shopping centers, are common in North America. Masonry walls in these buildings are mostly designed for gravity loads, but they also act as a lateral load resisting system. These walls are either unreinforced (URM) or partially reinforced, and their lateral load resistance is low relative to the design force levels required by the current building code (NBCC 2010). Shear strength of masonry walls are limited to the frictional resistance of bed and head joints. Flexural resistance is also limited due to the lack of sufficient vertical reinforcement, well anchored into the foundation. Therefore, retrofitting of seismically deficient masonry walls has been adopted as the objective of the current investigation.

Different approaches and materials were used by researchers in the past for retrofitting masonry walls. These include the application of surface mounted interior and exterior FRP sheets, filling of window and door openings, and adding new shear walls or steel braces for drift control. Marcari et al. (2006), Valluzi et al. (2002), Alcaino et al. (2007), and Stratford et al. (2004) investigated shear behaviour of epoxy-bonded FRP sheets on masonry walls. Different configurations, consisting of diagonal strips, grids, and entire surface coverage were considered to improve shear behavior of masonry walls. Overall strength of the walls was increased by these retrofit techniques. Tinazzi et al (2000) used FRP rods in Masonry joints as a retrofit strategy. The use of surface-bonded FRP sheets has gained popularity in recent years due to high material strength, superior mechanical properties, increased durability, and ease in application.

Previous researchers applied FRP either on one or both sides of the masonry wall. In places where masonry walls are not easily accessible from the exterior of the building, it may be preferable to retrofit them from the interior. The internal FRP can then be covered with dry walls or other architectural materials. ElGawady et al. (2007) tested a wall that was retrofitted on one side, and reported strength enhancement. Very few researchers addressed the issue of flexural strengthening of masonry walls. The success of flexural strengthening depends on proper anchoring of overlaying FRP sheets to the foundation or slab. Triantafillou (1998) and Fam et al. (2002) addressed the issue of FRP sheet anchoring to the foundation. Saatcioglu et al. (2005) used fan type of FRP anchors for infilled walls, which controlled premature debonding. Hall et al. (1999) and Holberg et al. (2002) used a ductile anchorage system for FRP retrofitting. In the present study, fan-type FRP anchors were used to increase flexural strength.

2. RETROFIT CONCEPT

The objective of the experimental program was to develop a retrofit technique for increasing in-plane wall capacity in shear and flexure, while also enhancing out-of-plane failure of block masonry during seismic excitations. Diagonal shear cracks that form within the wall panel are controlled by surface-bonded FRP sheets placed in single layer in horizontal direction and another single layer in the vertical direction. The major challenge is the enhancement of flexural capacity, which requires anchorage of FRP sheets placed on the wall panel to the foundation or the slab that the wall is built on. In the current research program this has been achieved through the use of fan-type FRP anchors. These anchors functioned as vertical flexural reinforcement, and transferred the tension force to the surface-bonded FRP sheets on the wall panel.

The retrofit strategy employed represents a global strengthening technique against in-plane shear forces. The additional FRP reinforcement is intended to increase elastic wall capacity, promoting elastic response, as neither the unreinforced masonry nor the FRP sheets have ductility for energy dissipation. Hence, the amount of FRP is dependent on the deficiency in elastic seismic force levels. This implies that the retrofit strategy implemented should be designed using a ductility related force modification factor of $R_d = 1.0$ for elastic response.

The details of the retrofit approach are illustrated through discussions of the retrofitted wall test presented in the experimental program.

3. EXPERIMENTAL PROGRAM

3.1 Test Specimen

A large-scale load bearing masonry wall was designed and built in the Structures Laboratory of the University of Ottawa as a companion specimen to the control specimen tested in an earlier phase of the investigation (Taghdi et al. 2000). Figure 1 illustrates the geometry and reinforcement details of the test specimen. The wall length and wall height were 2000 mm and 1800 mm, respectively. Two 15M longitudinal steel reinforcement (16 mm diameter) were used to make the wall partially reinforced. These two reinforcing bars were anchored in the foundation and the top loading beam. The half masonry cells, where the reinforcement bars were placed, were grouted. Two masonry cells at the ends were also grouted as part of the usual practice for improving compression resistance at the ends of reinforced walls. A certified mason was hired for construction of the wall. The wall was cured for at least 28 days and an over-reinforced concrete top beam was constructed on top of the wall for the purpose of applying simulated seismic forces, while also representing a rigid floor system. The beam cross section was 400mm wide and 500mm deep. The wall was designed to represent a single-story old masonry building. The size and material properties of the test specimen was kept the same as that for the control wall specimen tested earlier by Taghdi et al. (2000).

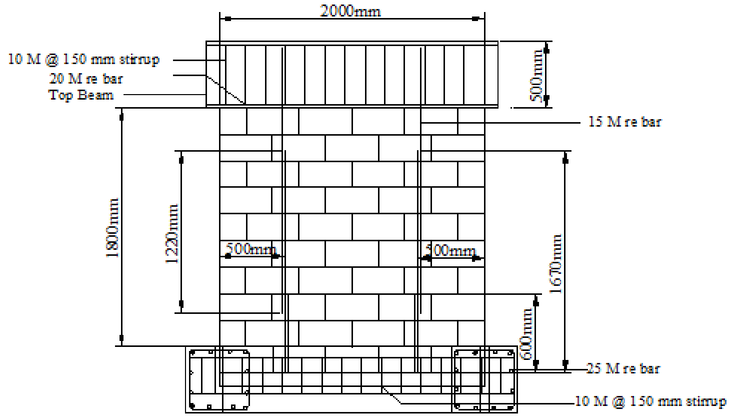

Figure 1. Geometric details of the masonry wall specimen

The wall was built on an I-shaped concrete foundation. The foundation was over reinforced to avoid any damage during testing. The foundation was designed to accommodate four large holes (approximately 100 mm in diameter each) to fit the holes in the Laboratory strong floor for securing the foundation during testing. The foundation concrete was supplied by a local ready-mix company. The concrete strength that was ordered, was 30 MPa. After casting, the foundation cured at least for 28 days.

The specimen was retrofitted with two layers of Carbon Fiber Reinforced Polymer (CFRP) sheets, one in each direction, covering the entire surface of the wall. This is illustrated in Fig. 2. CFRP sheets were applied in only one side of the specimen. The first layer had fiber orientation parallel to bed joints and the second layer had fibers oriented 90^0 with the horizontal. They were applied on one side of the wall, while the other side remained with exposed block masonry. CFRP was applied using the wet layup procedure. The wall surface was first prepared prior to the application of CFRP. The preparation involved; i) surface cleaning by wire brush, followed by air pressure to remove loose mortar, ii) application of putty consisting of two-component epoxy and silica fume to cover head and bed joints and to smoothen the wall surface, iii) removal of any extra putty by a plastic putty knife, iv) after curing for a day, inspection of the surface and covering any noticeable air bubbles with putty using the same plastic knife and finally v) sanding the surface by sand paper after two full days of curing.

Figure 2. The Masonry wall retrofitted on one side.

Once the wall surface was ready for the application of CFRP sheets, the sheets were cut to required sizes and applied on the wall surface. The application involved the following steps:

i. Application of a layer of two-component epoxy on the surface.
ii. Application of the first layer of CFRP whose fibres were parallel to the bed joint, saturated in epoxy.
iii. Removal of extra epoxy and air pockets by means of a ribbed steel roller.
iv. Application of another layer of epoxy prior to the placement of the next layer.
v. Application of the second layer of CFRP perpendicular to the bed joint after saturating it in epoxy and following step iii.

3.2 Material Properties

Standard hollow concrete block masonry units (CMU) with 390 mm × 190 mm × 190 mm dimensions were used to build the wall. The weight of each block was 17 kg and its compressive strength was 17 MPa as established by standard compressive strength tests. The gross and net areas of each block were 74,100 mm^2 and 39,719 mm^2, respectively. The blocks were 54% solid. Type S mortar, with a compressive strength of 9 MPa was used to build the wall. The same mortar mix (masonry cement / sand = 1:3) was used to grout the end cells, as well as the cells that contained reinforcement. The longitudinal reinforcing steel had 400 MPa yield strength and 0.2% yield strain. Truss type joint reinforcement was used in every other mortar joint to prevent vertical cracking, as part of the standard construction practice.

The FRP sheets consisted of carbon fibers with a tensile strength of 3790 MPa and an elastic modulus of 230 GPa. The tensile rupturing strain was 1.7%. The density of the fibre material was reported by the manufacturer as 1.74 gm/cm^3. The epoxy strength was 72.4 MPa with a tensile modulus of 3180 MPa, resulting in 5% strain at ultimate. The composite laminate thickness was 1.0 mm, with a tensile strength of 834 MPa and elastic modulus of 82 GPa. The elongation at rupture of the laminate was 0.85%.

3.3 Anchoring Technique

CFRP anchors, developed at the University of Ottawa (Ozbakkaloglu and Saatcioglu 2009), were used to secure the CFRP sheets on the wall, as well as anchoring them in the foundation. The anchors were manufactured in-house by cutting required 200mm×250mm pieces from a roll of CFRP sheet. The cut FRP pieces were rolled by hand to form the anchors, as illustrated in Fig. 3(a). A 100mm length of the twisted FRP was tightened with a thread. Remaining 150 mm was kept free for fanning. The application involved a two-step process. First holes were drilled at strategic locations of the element to which the FRP was to be anchored (wall or foundation). After making the holes, two-component epoxy, thickened by silica, was injected into the holes with a 60ml injection syringe. Then, the rolled part of an anchor was saturated with epoxy and inserted into the hole. Finally, the fan portion was secured to the wall surface with epoxy.

In the current experimental program the anchors were used at two different strategic locations:

i. Wall bottom edge, near its ends, where they would be most effective in forming tension reinforcement. First a long sheet of CFRP was cut and secured along the bottom edge, following the same wet layup method employed for the wall FRP. This sheet was placed along the entire wall-foundation interface, overlapping with the wall surface on one side and the foundation concrete on the other side, covering the entire wall-foundation joint by forming an FRP angle. A similar FRP sheet was also placed along the top wall-beam joint. To avoid sharp corners, the edge of the wall at foundation and top loading beam interface were rounded with cement mortar as illustrated in Fig. 3(b). A hand held hammer drill was used to open holes near the wall end to securing the FRP anchors. The holes were 75 mm in depth and diameters were little larger than the anchor diameter. Figure 3(c) depicts the anchors in place. These anchors were designed to take required tensile forces for the incremental increase in moment capacity established by retrofit design.

ii. Wall surface to prevent debonding of CFRP sheets from the wall. These anchors were placed along wall diagonal to ensure bond when diagonal tension forces are developed under reversed cyclic loading. The anchor capacity was developed experimentally by Ozbakkaloglu and Saatcioglu (2009) as a function of anchor inclination. They were placed with 15 degree inclination with the surface to be effective in resisting tension when the respective wall region was subjected to diagonal tension. The anchor fans were positioned to transfer tensile stresses in anchors to the wall FRP. Figure 3(c) illustrates the anchors in place.

(a) (b) (c)

Figure 3. (a) FRP fan anchor (b) Rounding of wall-foundation joint prior to FRP application (c) FRP anchors in place.

3.4 Test Setup, Instrumentation and Loading Program

The test setup was designed to apply incrementally-increased in-plane cyclic loading. Gravity load was simulated through axial prestressing of the beam relative to the foundation. The test setup is illustrated in Fig. 4.

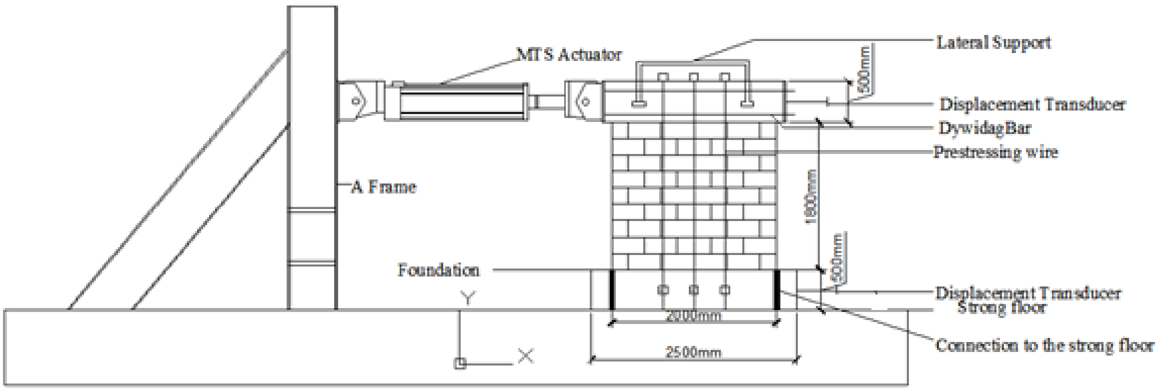

Figure 4.Test setup.

A total of 100 kN of axial load was applied prior to testing by means of six 8mm-diamater prestressing strands. The appropriate level of load per strand was ensured through strain gauges placed on the strands. A pre-stressing cable was calibrated for this purpose, by placing a strain gauge on the cable and applying tension by a universal testing machine. Each strand was stressed to 17 kN. The strands were tensioned between the three hollow steel sections (50mm x 50mm x 6mm) of 915 mm length above the loading beam and another set of equal geometry that had been cast in the foundation concrete were inserted into the foundation before concrete casting. The use of three sets of hollow sections, distributed along the beam length helped simulate distributed gravity loading.

The specimen was instrumented to measure displacements, rotations and strains. Three types of sensors were used; i) electric resistance strain gauge (ERSG), ii) linear variable displacement transducers (LVDT) and iii) cable transducers (CT). Displacements were measured mainly with CTs. The load was recorded by the MTS controller. The positions of transducers to record the force displacement relationship are shown in Fig. 4. An extra set of displacement transducers were attached on the opposite side of the wall as a backup. Foundation displacements were monitored and recorded during testing. Diagonal transducers were attached to measure shear distortions on the wall panel. Two vertical transducers were placed to measure vertical displacement of the wall. LVDT's were attached to measure potential rotation and slippage of the wall relative to the foundation. Strain gauges were placed at different locations on FRP to record the strain in FRP. All sensors were attached to a data acquisition system. The data acquisition system was also connected to the actuator load cell.

A servo controlled 1000 kN capacity MTS actuator was used to apply the in-plane cyclic loading as shown in Figure 4. The actuator was used in displacement-controlled mode. The actuator was supported with an "A" frame that consisted of steel I sections, as illustrated in Fig. 4. The other end of the actuator was connected to the wall with a thick steel loading plate of 50.8mm. Another steel plate of same thickness was attached at the other end of the top beam. The two end plates were connected with four high-strength post-tensioning bars. These bars were effective during the pull phase of loading. The setup allowed the wall to be tested as a cantilever wall.

Lateral (out-of-plane) restraint was very important in this experiment. Because the specimen was retrofitted on one side of the wall and lateral movements were anticipated during testing. A special type of lateral support was designed and fabricated, consisting of a frame secured on the laboratory wall and a steel attachment with guiding rails and attached rollers to ensure in-plane loading. This is shown in Fig. 4. The lateral support was built from different size steel hollow sections.

The specimen was subjected to incrementally increasing lateral deformation reversals, accompanied by constant initial axial compression. Lateral load was applied in the displacement controlled mode throughout the test. The specimen was cycled three times in each displacement level. The displacement was computed as a story drift, defined as top displacement divided by the height of the wall. The test started by cycling the wall three times at 0.25% drift ratio. It was then cycled at gradually increasing drift cycles, as illustrated in Fig. 5, continuing with three cycles at 0.5% drift, followed by 0.75%, 1% etc. The test continued until the specimen was damaged substantially and the lateral load resistance dropped very significantly.

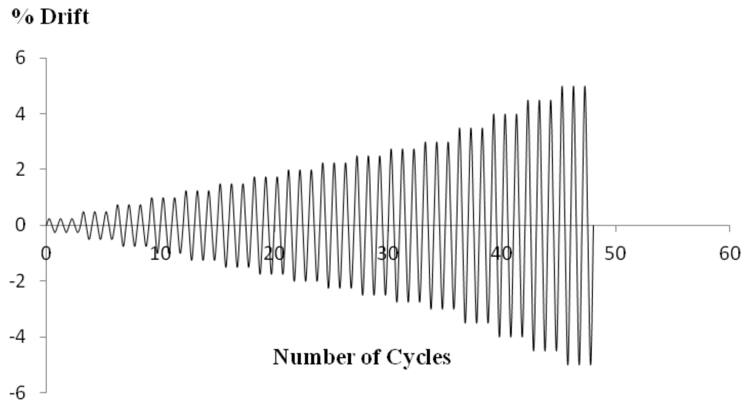

Figure 5. Loading history in displacement controlled mode

4. TEST RESULTS

4.1 Observations Made During Testing

The wall retrofitted with CFRP sheets on one side showed much better performance than the companion unretrofitted wall tested earlier by Taghdi et al. (2000). The peak lateral load resistance increased significantly, developing 1.5 times the capacity of the unretrofitted wall. Shear performance was also improved with the use of FRP sheets on one sides, controlling diagonal shear cracking effectively. The specimen did develop initial diagonal shear cracks at 0.5% lateral drift level, but the cracks did not propagate appreciably due to the presence of CFRP sheets. Figure 6(a) shows the crack pattern at the end of 0.75% drift ratio. CFRP sheets helped ensure the integrity of masonry blocks until the very end of the test, at which time toe crushing was observed at approximately 4% lateral drift.

The vertical FRP anchors, anchoring the wall to the foundation, served their intended function and increased wall resistance approximately 50% until they failed through the formation of pull-out cones. The anchors started failing gradually during the cycles at 0.75% lateral drift ratio. Figure 6(b) and 6(c) show the separation of FRP from the foundation, initially due to the elastic elongation of anchors, followed by the anchor failure. The drop in load resistance due to the anchor failure was 37% of peak load. After the initial strength degradation, the wall continued resisting lateral load due to the presence of vertical reinforcement and the presence of CFRP eliminating the diagonal tension failure. Beyond the anchorage failure, the stiffness of the wall was maintained with little degradation as compared to the companion unretrofitted specimen. Overall energy dissipation was found higher in the retrofitted specimen. One-sided retrofitting was successful and did control diagonal cracking, eliminating diagonal tension failure. The wall showed rocking behaviour after 1.5% lateral drift. Some ductility was observed at this stage due to the presence of vertical reinforcement. As the rocking behaviour continued, the wall suffered masonry crushing at its toes. This is illustrated in Fig. 6(d). No sliding or debonding of CFRP layers was observed. The anchors connecting the FRP to the wall panel remained intact until the end of the test.

(a) (b) (c) (d)

Figure 6. (a) Initial diagonal crack, (b) FRP sheet connected to the foundation prior to the ancor pull-out cone failure, (c) FRP anchorage failure and the seperation of FRP from the foundation, (d) corner crushing of masonry

4.2 Hysteretic Behaviour

The hysteretic behviour of unretrofitted and retrofitted walls are shown in Figs. 7 and 8. The unretrofitted wall that was tested by Taghdi et. al. (2000) in the same laboratory showed inferior performance than the retrofitted specimen. The peak load resistance was limited to 120 kN. The wall survived up to 0.8% drift level. The wall experienced extensive diagonal crushing and a major load drop at 0.6% drift level due to the masonry crushing. The wall also showed significant drop in its stiffness beyond initial peak load resistance.

The retrofitted specimen showed higher load resistance and energy dissipation than that of the unrerofitted specimen. The retrofitted specimen showed higher stiffness with only a few diagonal cracks developing in the wall. In the push and pull directions, the specimen resisted 100 kN of load at 0.25% drift ratio. During the subsequent drift level (0.5% drift) CFRP sheets started lifting up from the foundation as the anchors were extending elastically. The lateral load resistance in push was 165 kN, and in pull was 174 kN.

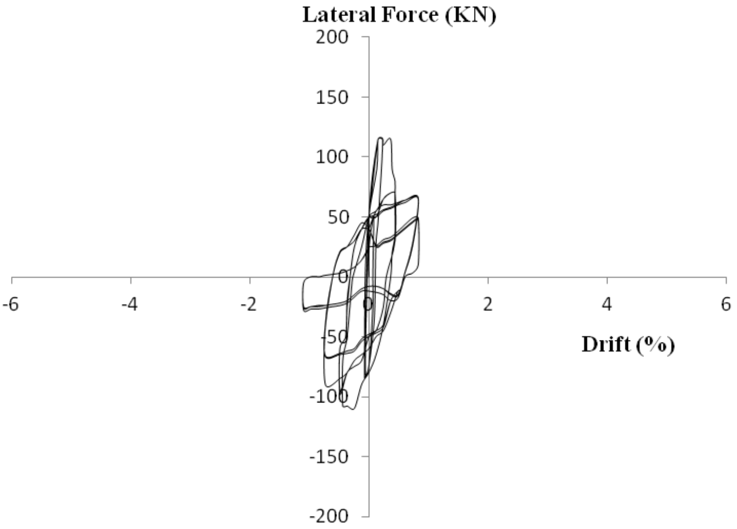

Figure 7. Hysteretic Load-displacement relationship for un-retrofitted companion specimen, Taghdi (2000)

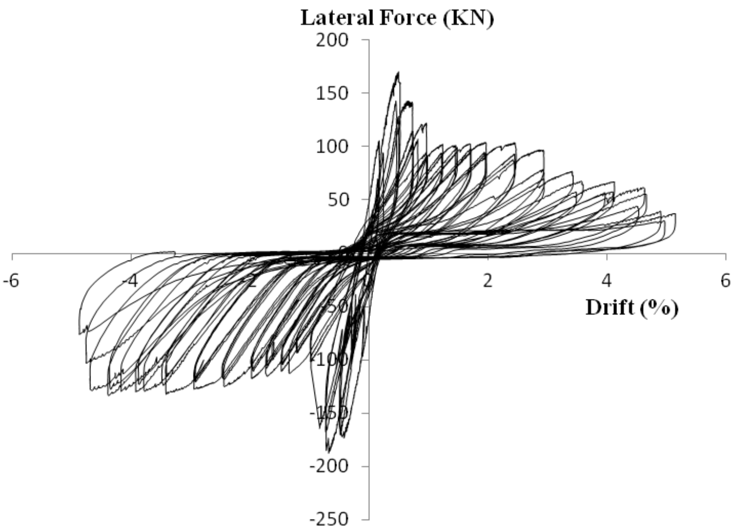

Figure 8. Hysteretic force-displacement relationship for retrofitted specimen.

CFRP anchors placed near the ends of the wall, anchoring the wall to the foundation, functioned as designed up to 0.75% lateral drift ratio. During the 0.75% drift cycles, the anchors gradually started pulling out by forming pull-out cones of concrete, and the FRP overlap on the foundation started separating from the concrete. The load resistance in the push direction dropped to 138 kN, but the pull direction continued resisting 182 kN. The load resistance in pull dropped to 120 kN at 1% drift level. FRP anchors failed similar to those that failed earlier on the other side by, pulling out of the foundation. The peak load drop can be attributed to this pull out cone failure. During the subsequent deformation cycles, the load resistance remained around 100 kN until 2.75% drift ratio was achieved. Another load drop was observed after the 2.75% drift level. This drop was attributed to toe crushing of masonry at the end of the wall, on the east side. The load resistance was observed to remain constant before the crushing of masonry. Yielding of vertical No 15M reinforcement kept the load resistance at the same level during many deformation cycles within the inelastic range. The wall started to rock at 1.5% drift level. Pull side behaviour was better than that of the push side due to the crushing of masonry, which initiated sooner in the push direction. The wall maintained its load resistance after the failure of the bottom anchors until the 4.5% drift level is attained. There was no shear sliding and CFRP debonding from the wall panel until the end of the test. The FRP retrofitting also helped reduce stiffness degradation as can be seen in the hysteretic relationship.

5. CONCLUSIONS

The following conclusions can be drawn from the current experimental study.

- Surface bonded CFRP sheets can be used as an effective retrofit technique for load bearing masonry walls. The CFRP-retrofitted specimen, with a single layer of CFRP in each direction, anchored to the foundation, resisted 1.5 times the lateral force resisted by the companion specimen without the retrofit.
- CFRP sheets prevent diagonal shear failure and improve wall shear capacity.
- Surface-bonded CFRP sheets can increase the flexural capacity of walls if properly anchored to their foundations (or slabs). CFRP fan type anchors, inserted in the foundation, must be designed to have sufficient embedment length and fibre area to achieve the required tension force and associated flexural capacity.
- Retrofitting masonry walls only on one-side with surface-bonded FRP prove to be effective, improving overall wall behaviour under simulated seismic loading.
- FRP fan type anchors can be used to ensure proper bonding of FRP sheets to masonry walls.

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