Seismic Performance of Masonry Infill Walls Retrofitted With CFRP Sheets

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Synopsis: A significant portion of existing building stock that was constructed prior to the enactment of modern seismic design provisions consists of gravity-load-designed reinforced concrete frames, infilled with unreinforced masonry walls. These structures are susceptible to extensive seismic damage when subjected to strong earthquakes and require retrofitting in order to comply with the provisions of current building codes. Experimental investigation of gravity-load-designed reinforced concrete frames, infilled with concrete block masonry, has been conducted to develop a seismic retrofit strategy that involves the use carbon fiber reinforced polymer (CFRP) sheets. Two half-scale concrete frames, infilled with masonry walls were tested with and without seismic retrofitting. The retrofit technique consisted of CFRP sheets, surface bonded on the masonry wall, while also anchored to the surrounding concrete frame by means of specially developed CFRP anchors. The frame-wall assemblies were tested under constant gravity loads and incrementally increasing lateral deformation reversals. The results indicate that infilled frames without a seismic retrofit develop extensive cracking in the walls and frame elements. The elastic rigidity reduces considerably resulting in softer structure. The failure may occur in non-ductile frame elements, especially in columns. Retrofitting with CFRP sheets controls cracking and increases lateral bracing, improving the elastic capacity of overall structural system. The retrofitted specimen tested in the current investigation showed approximately 300% increase in lateral force resistance, promoting elastic response to earthquake loads as a seismic retrofit strategy. Experimental observations and results are presented in the paper.

Keywords: concrete frames; ductility; earthquakes; fiber reinforced polymer (FRP); infill walls; masonry walls; retrofitting; seismic retrofit

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INTRODUCTION

Unreinforced masonry (URM) elements are used extensively as infill wall panels in reinforced concrete and steel frame structures. URM infills fulfill architectural and other functional requirements, such as forming a significant portion of building envelop, partitioning, temperature and sound barriers, while also providing adequate compartmentalization against fire hazard. Masonry is a locally available building material that has a long history of successful use in the construction industry.

The role of URM in resisting seismic forces has been somewhat misunderstood or ignored in the past due to the brittle nature of the material. Lack of knowledge on its performance under seismic loading has discouraged engineers from relying on the interaction of URM-infills with the enclosing structural system. Therefore, it has become a common practice to ignore the participation of infills in resisting lateral loads. The brittle behavior of masonry elements during past earthquakes has discouraged the use of URM as a seismic resistant structural component, justifiably so. However, previous research has shown the beneficial effects of the interaction between URM infills and structural elements for seismic performance of existing frame buildings. Researchers have concluded that proper use of URM infills in frames could result in significant increases in the strength and stiffness of structures subjected to seismic excitations (Klingner and Bertero 1978, Mehrabi et al. 1996, Bertero and Brokken, 1983). However, the locations of infills in a building must be carefully selected to avoid or minimize torsional effects. Architectural restrictions have to be considered when assigning these locations. Whenever infills are placed in an existing building, the increase in strength demand must be compared with the increase in strength capacity, since infilled frames may attract higher forces as compared to bare frames.

A large number of buildings, particularly those constructed prior to the enforcement of ductile design philosophies of the 1970's, have utilized URM-infilled concrete frames. The concrete frames were primarily designed and detailed to resist gravity loads. It has been a common practice to consider URM infill walls as non-structural elements, though they often interact with the enclosing frame, sometimes creating undesirable effects. These elements may contribute to the increase in strength and stiffness of the overall building, though they are not capable of improving inelastic deformability. Any seismic improvement to be expected from URM infills is limited to the elastic range of material. The elastic behavior may not be ensured during a strong seismic excitation, and the subsequent brittle failure may lead to disastrous consequences. Buildings that were designed for higher seismic forces than those corresponding to the elastic threshold of URM need to be protected during seismic response. One technique to achieve this goal is to strengthen the URM by means of epoxy-bonded carbon fiber reinforced polymer (CFRP) sheets.

Two half-scale reinforced concrete frame-concrete block infill assemblies were tested under simulated seismic loading to develop a seismic retrofit strategy for these structures. The first specimen was built to reflect the majority of existing buildings constructed prior to 1970's, with a gravity-load designed frame. The second specimen was retrofitted with CFRP sheets for improved seismic resistance. The experimental program and the results are presented in the paper.

EXPERIMENTAL PROGRAM

Test specimens

Two identical specimens were constructed at the Structures Laboratory of the University of Ottawa for testing. The frames were designed following the requirements of ACI 318-1963 to represent an older building prior to the enactment of modern seismic design requirements. The columns had a 250 mm square section and 8-#15 (16 mm diameter) deformed bars, resulting in 2.56% reinforcement ratio. The longitudinal column reinforcement was lap spliced just above the foundation, with 24 bar-diameter lap lengths, which conformed to the requirements of ACI 318-1963 for compression members. Column ties consisted of 6.35 mm (#2 US) diameter smooth wire with 90-degree bends with 6 bar diameter extensions. The ties were spaced at 125 mm (1/2 the column dimension), providing virtually no confinement to column concrete. The beams, 250 mm wide and 350 mm deep, were reinforced with 3-#15 top bars and 2-#15 bottom bars, resulting in 0.77% and 0.54% reinforcement ratios, respectively. The beam stirrups were in the form of closed hoops with 90-degree bends, following the same detailing as that for column ties with 125 mm spacing. Figures 1 and 2 illustrate the details of the frame members.

Concrete was supplied by a local ready-mix company for both frames. Once the foundation of each specimen was cast, the reinforcement cage for frame was assembled. The formwork for frames consisted of 19 mm plywood boards wrapped with plastic to preserve the moisture in concrete during curing, and to facilitate removal of the formwork. The formwork was removed after 3 days. The specimens were moist cured for another 3 to 4 days after the removal of formwork. A professional contractor was hired to build the masonry infills to implement the actual practice in industry. Figure 3 illustrates the construction process. The infill walls were not connected to the frames by means of anchors. Masonry prisms were prepared and mortar samples were taken to verify that it met the requirements of Mortar Type N with compressive strength of at least 5.2 MPa at 28 days (Drysdale et al., 1994).

One of the specimens was retrofitted after testing as the as-built specimen, and its performance was used to assess the retrofit requirements for the subsequent specimen. Two aspects of the retrofit strategy employed were of concern; i) the amount and arrangement of CFRP sheets and ii) the possibility of delamination of sheets and measures against them. After carefully examining the results of the first test, it was decided to use one sheet per face parallel to each of the two diagonals, resulting in two sheets per wall face. The CFRP sheets were placed diagonally to increase their efficiency since their primary function was to resist diagonal tension.

Specially designed CFRP anchors were developed and used to minimize/eliminate the delamination of CFRP sheets from the surface of the wall. This was done by drilling holes in frame members adjacent to the wall, with approximately 45-degree inclination towards the centre of the frame elements, and inserting the anchors to be epoxy glued into concrete. The anchors were produced in the Laboratory by twisting strips of CFRP sheets and folding into two, as illustrated in Figure 4. A hammer drill was used to make approximately 50 mm deep, 12 mm diameter holes in columns and beams for the FRP anchors (Figure 5). The anchors were placed and epoxy glued after the application of surface bonded CFRP sheets. Wooden pieces were inserted into the anchor holes during the placement of CFRP sheets, as guides, and also to avoid filling of the holes with epoxy. This is illustrated in Figure 6.

The recommendations of the supplier were followed to place the sheets as described below:

- Masonry substrate was prepared by removing loose concrete and dust to provide good adhesion between CFRP and masonry substrate.
- Epoxy primer coat was applied by using a roller.
- Epoxy putty filler was applied on the surface to fill mortar joints as much as possible, as well as any imperfections on the surface. This was done using a trowel. Figure 3(b) shows a wall specimen after the application of epoxy primer and filler putty.
- The first coat of epoxy resin (saturant) was applied diagonally over the primed and puttied surface using a roller.
- Fibers were saturated with epoxy and the first layer of fibers (sheet) was placed over the masonry surface.
- The second coat of saturant was applied over the first layer of sheet.
- The second saturated layer of sheet was placed over the wall with fibers running diagonally, perpendicular to the fibers of the first layer.
- Final coat of saturant was applied as illustrated in Figure 3(c).
- The CFRP anchors were carefully embedded through two layers of fiber sheets into the holes in the frame and bonded to the CFRP sheets with epoxy resin as illustrated in Figure 7.

- A ribbed steel roller was used to remove excess saturant from the sheets and to improve surface bond by removing entrapped air pockets.

Material properties

CFRP Composite – A carbon fiber composite system was used to retrofit the masonry wall, consisting of carbon fibers and epoxy resin. The thickness of carbon fiber sheet was 0.165 mm/ply, which was increased to 0.8 mm/ply when impregnated with epoxy resin. The stress-strain relationship of composite material was established by coupon tests and showed linear-elastic behavior up to the rupturing strength in tension. The capacity of the composite material was established to be approximately 785 MPa based on the fiber content corresponding to 0.8mm/ply. This observation was found to be in agreement with the 3800 MPa fiber strength reported by the manufacturer. The maximum tensile strain of 1.67% specified by the manufacturer was also in agreement with the values recorded during coupon tests. The above values translate into the elastic modulus of 47,000 MPa for the composite material, which is in line with 227,000 MPa reported for the modulus of elasticity of carbon fibers alone.

Concrete and Reinforcing Steel – Concrete was cast in two different batches. The 28-day concrete strengths for unretrofitted and retrofitted frames were established by standard cylinder tests to be approximately 42 MPa, and 38 MPa, respectively.

The stress-strain relationships of reinforcement were determined by performing standard coupon tests. The yield strength of transverse reinforcement (bar diameter = 6.35 mm) used in both frames was approximately 400 MPa. The stress-strain relationship for longitudinal reinforcement (#15 re-bar) in both frames indicated a yielding strength of approximately 425 MPa.

Test setup, instrumentation, and loading program

The specimens were secured to the laboratory strong floor using four threaded steel rods to provide full fixity. Vertical loads were applied on the columns and top beam to simulate gravity loading. This was done by means of externally post-tensioned cables anchored to steel loading assemblies. The loading assemblies were built with hollow structural steel (HSS) sections (75mm x 75mm x 9.5mm). Three HSS sections were welded together to apply axial loads on columns. These assemblies were positioned at the top of columns and underneath the foundation. Each cable was loaded using a hydraulic jack to strain to a value corresponding to 100 kN, generating a total load on each column of 400 kN. The vertical loads on the beam were applied in a similar manner. However, a single HSS was sufficient for each concentrated vertical load on the beam. Three concentrated loads were applied at an equal spacing on the beam, each having a magnitude of 40 kN to simulate distributed gravity load. An MTS hydraulic actuator, with a force capacity of 1000 kN and a stroke capacity of 500 mm was used to apply cyclic in-plane lateral loading. The actuator was connected to a steel reaction frame at one end and to the top beam of the infilled frame at the other end to apply horizontal load during the pushing phase of loading. Four high-strength Dywidag bars were placed externally along the top beam and connected to a loading plate at the far end of the top beam to facilitate pulling of the specimen during lateral load reversals. The test setup is illustrated in Figure 8.

An MTS temposonic LVDT, with a range of 250 mm, was used to measure lateral displacements of the frame along the longitudinal centre-line of the beam. The LVDT was mounted on a metal frame that was connected to the foundation of the specimen. Electric resistance strain gages were placed on re-bars inside the concrete frame. Strain gages were also placed on the post-tensioning cables used to apply vertical loads. These gages were all connected to the same data acquisition system. The MTS actuator was controlled with a computer system that recorded the magnitude of the lateral load, displacements from the actuator's internal LVDT, and displacement readings from the temposonic LVDT.

Lateral loading was applied by the MTS actuator in deformation control mode, as the increments of lateral drift ratios were increased. Three cycles of lateral displacements were applied at each deformation level until failure. The failure was assumed to occur when the strength of specimen was reduced by more than 50% of its ultimate resistance.

TEST RESULTS

The infilled frames tests indicated a high level of participation of the walls to frame response. The walls were able to stiffen the frames significantly, especially during the initial stages of loading. The unretrofitted wall experienced gradual stiffness degradation under increasing levels of deformation reversals. Progressive cracking of masonry units and the mortar joints led to the dissipation of energy, without affecting the strength of frame. The lateral drift was controlled by the stiffening effect of wall. Of particular interest was the simultaneous degradation of strength and stiffness of the wall and the frame, contrary to the common design assumption that the masonry walls would disintegrate early in seismic response, leaving the frames as the only structural system to resist earthquakes. This may be the characteristics of the wall system tested in the current investigation, as the walls were constructed to be fully in contact with the frames. Nevertheless, this observation does indicate that the two materials, i.e., reinforced concrete and infill masonry are compatible in resisting lateral deformations. During loading, the initial resistance was provided mostly by the walls, and the load resistance was gradually transferred to the frames through progressive cracking and softening of the walls. The eventual failure of the unretrofitted specimen was caused by the hinging of columns within the reinforcement splice regions near the ends, while a significant portion of the cracked infill wall remained intact. Figure 9 shows the hysteretic force-lateral drift relationship for the unretrofitted frame-wall assembly. The relationship indicates that a peak load of 273 kN was attained in the direction of first load excursion at approximately 0.25% lateral drift ratio and remained constant up to about 1% drift, though there was substantial stiffness degradation during each cycle of loading. Gradual strength decay was observed after 1% drift, and the assembly failed due to the failure of reinforced columns in their reinforcement splice region at about 2% lateral drift.

The retrofitted specimen showed a substantial increase in elastic rigidity and strength. The initial slope of the force-deformation relationship was high, corresponding to that of uncracked wall stiffness and remained at the uncracked stiffness value until the specimen approached its peak resistance. It was clear that the CFRP sheets controlled cracking and helped improve the rigidity of the wall. The specimen resisted a peak load of 784 kN in the direction of first load excursion, indicating an improvement of approximately a factor of 3. The peak load was attained at approximately 0.3% lateral drift ratio. The CFRP sheets maintained their integrity until after the peak resistance was reached. There was no delamination observed throughout the test. However, the CFRP sheets started to rupture gradually, near the opposite corners in diagonal tension. This resulted in strength decay. By about 0.5% lateral drift, approximately 25% of the peak load resistance was lost. The load resistance continued to drop during subsequent deformation reversals and the resistance dropped down to the level experienced by the unretrofitted specimen at approximately 1% drift ratio. The behavior beyond this level was similar to that of the earlier unretrofitted specimen, and the failure occurred at 2% drift ratio. The hysteretic relationship recorded during the test is illustrated in Figure 10. Further details of the experimental program, including additional data can be found elsewhere (Serrato and Saatcioglu 2004).

CONCLUSION

The following conclusions can be drawn from the experimental research reported in this paper:

- Unreinforced masonry infill walls in reinforced concrete frame structures can provide significant lateral stiffness during seismic response provided that they are not isolated from the surrounding frame elements. These walls can lead to sufficient drift control until after their elastic limit is exceeded. Beyond this level, the walls suffer from significant strength and stiffness deterioration. The overall strength of infilled frames is governed by that of the bare frame.

- Surface bonded CFRP sheets placed parallel to wall diagonals and sufficiently anchored to the surrounding frame can control cracking and resulting stiffness deterioration in unreinforced infill masonry walls. This results in continued in-plane lateral bracing of the wall during seismic response.

- Surface bonded CFRP sheets placed parallel to wall diagonals and sufficiently anchored to the surrounding frame can result in significant increase in the strength of overall structural system. However, they do not contribute to ductility. Upon failure of CFRP, the wall strength reduces to a level that is equivalent to that of unretrofitted wall. The strength of the specimen tested in this investigation increased by a factor of three due to the use of a single layer of CFRP sheet on each side of the wall parallel to each diagonal.

- The CFRP anchors used in securing the sheets to the surrounding frame can function effectively in preventing the delamination of sheets, and promote frame-infill interaction during seismic response.

- Seismic retrofit design of CFRP covered masonry walls should be based on elastic force limits rather than the principles of ductility and energy dissipation.

ACKNOWLEDGEMENTS

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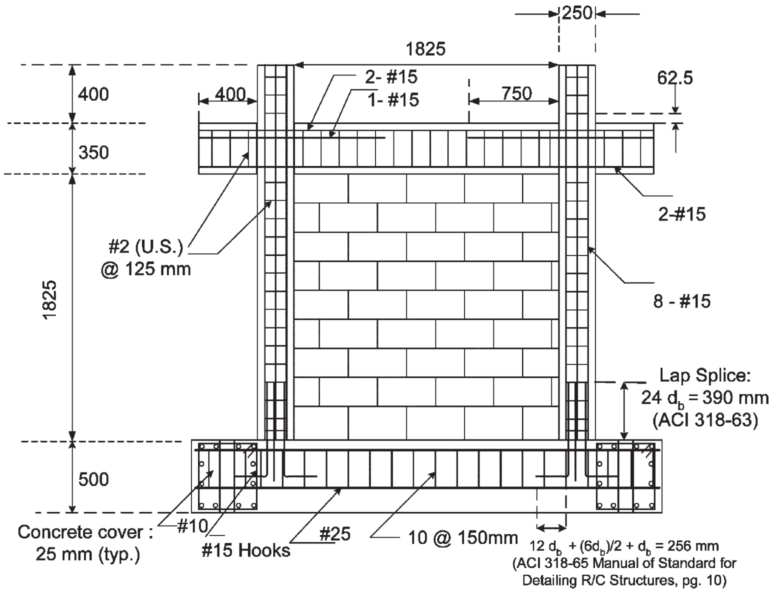
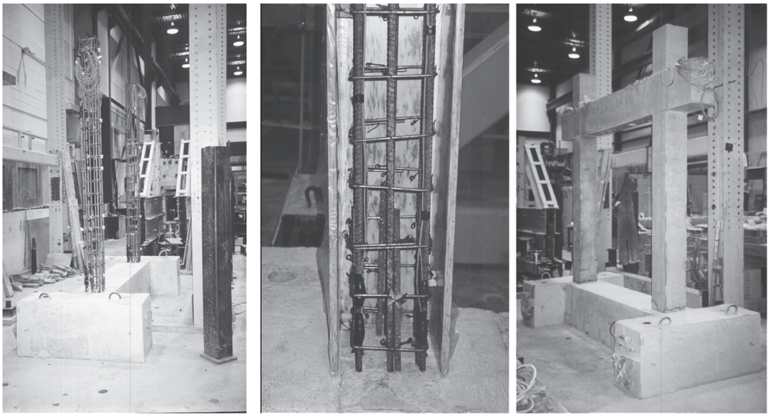
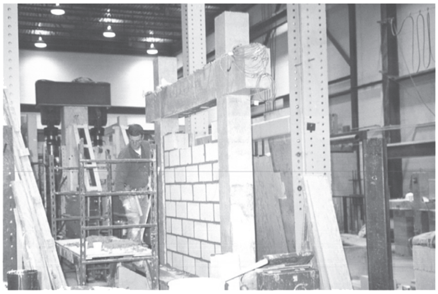


Figure 1 Details of the frame-wall assemblies tested

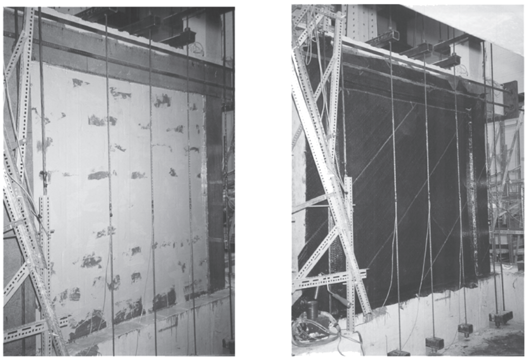


(a)Column cages (b)Spliced bars (c)Frame after casting

Figure 2 Frames during construction



(a) Masonry wall under construction



(b) Application of filler putty c) Wall retrofitting with FRP sheets

Figure 3 Infill walls under construction

Figure 4 CFRP anchor

Figure 5 Anchor holes

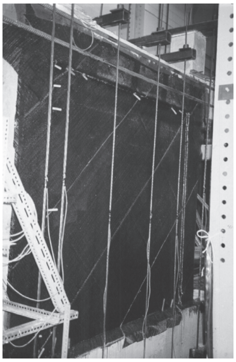


Figure 6 Wooden guides

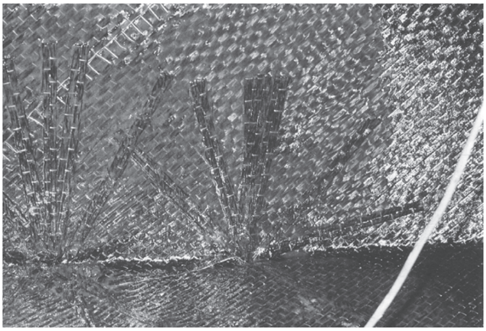


Figure 7 CFRP anchors in place

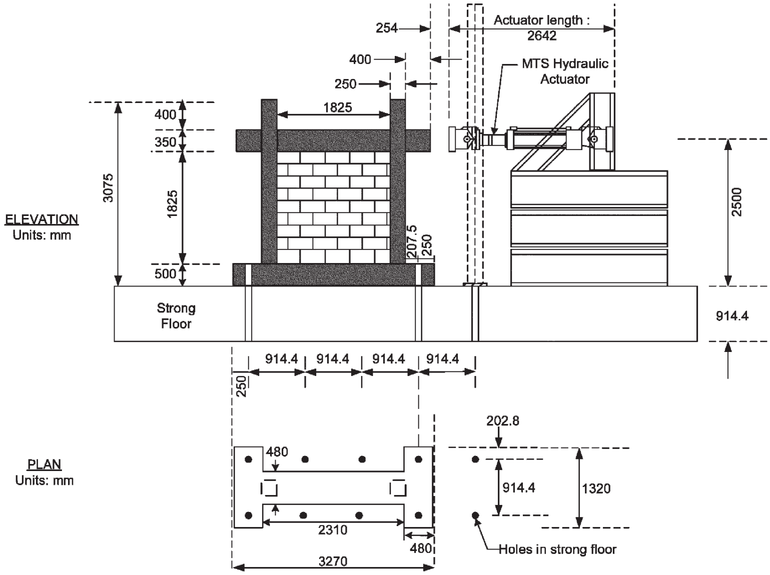


Figure 8 Test setup

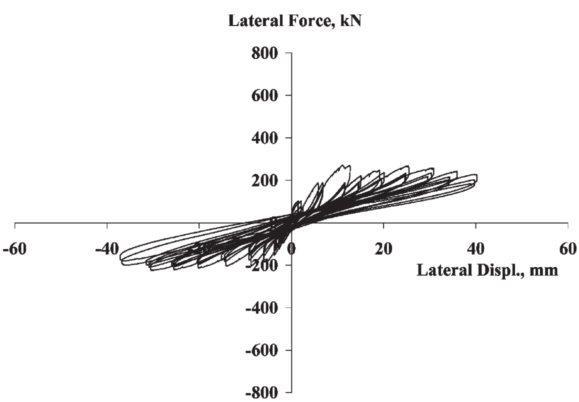


Figure 9 Hysteretic force-displacement relationship for unretrofitted specimen

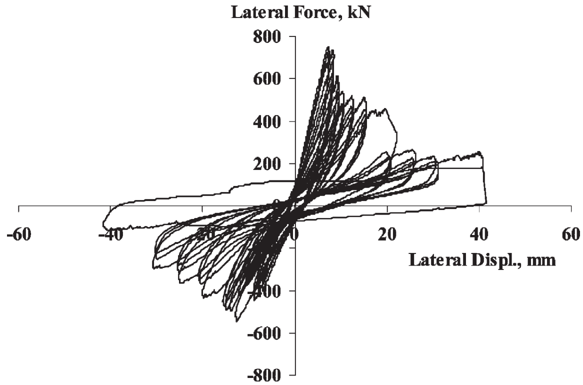


Figure 10 Hysteretic force-displacement relationship for retrofitted specimen