

SEISMIC RETROFIT OF LOAD BEARING MASONRY WALLS WITH SURFACE BONDED FRP SHEETS

by

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ABSTRACT

A large inventory of low rise masonry buildings in Canada and elsewhere in the world were built using unreinforced or partially reinforced load bearing wall. The majority of existing masonry structures is deficient in resisting seismic force demands specified in current building codes. Therefore, they pose significant risk to life safety and economic wellbeing of any major metropolitan centre. Because it is not economically feasible to replace the existing substandard buildings with new and improved structures, seismic retrofitting remains to be an economically viable option.

The effectiveness of surface bonded carbon fiber-reinforced polymer (CFRP) sheets in retrofitting low-rise load bearing masonry walls was investigated in the current research project. The retrofit technique included the enhancements in wall capacity in shear and flexure, as well as anchoring the walls to the supporting elements through appropriate anchorage systems. Both FRP fan type anchors and steel sheet anchors were investigated for elastic and inelastic wall response. One partially reinforced masonry (PRM) wall and one unreinforced masonry (URM) wall were built, instrumented and tested under simulated seismic loading to develop the retrofit technique. The walls were retrofitted with CFRP sheets applied only on one side to represent a frequently encountered constraint in practice. FRP fan anchors and stainless steel sheet anchors were used to connect the vertical FRP sheets to the wall foundation. The walls were tested under constant gravity load and incrementally increasing in-plane deformation reversals. The lateral load capacities of both walls were enhanced significantly. The steel sheet anchors also resulted in some ductility. In addition, some small-scale tests were performed to select appropriate anchor materials. It was concluded that ductile stainless steel sheet anchors would be the best option for brittle URM walls.

Analytical research was conducted to assess the applicability of truss analogy to retrofitted walls. An analytical model was developed and load displacement relationships were generated for the two walls that were retrofitted. The analytical results were compared with those obtained experimentally, indicating good agreement in force resistance for use as a design tool.

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NOTATIONS

AFRP	Aramid fibre reinforced polymer.
A_g	Gross area.
CFRP	Carbon fibre reinforced polymer.
CSA	Canadian standard association.
CT	Cable transducer.
CMU	Concrete masonry unit.
ESRG	Electric resistance strain gauge.
E	Modulus of Elasticity.
FRP	Fibre Reinforced Polymer.
F_a	Acceleration based site coefficient.
f_m	Compressive strength of masonry.
GFRP	Glass fibre reinforced polymer.
HSS	Hollow steel section.
I_E	Earthquake importance factor of the structure.
LVDT	Linear variable displacement transducer.
NBCC	National Building Code of Canada.
PRM	Partially Reinforced Wall.
R_d	Ductility related force modification factor.

S_a	Spectral response acceleration.
URM	Unreinforced Masonry Wall.
V_{diagonal}	Diagonal shear capacity of masonry wall.
V_{sliding}	Sliding shear capacity of masonry wall.
V_{FRP}	FRP shear resistance.
θ	Fibre orientation.
σ	Stress.
σ_y	Yield strength.
ε	Strain.
ϵ_y	Yield strain.
f_m	Compressive strength of masonry.
θ	Fibre orientation.

Chapter 1

Introduction

1.1 General

Masonry is one of the oldest and popular construction methods throughout the world. The majority of existing masonry buildings are unreinforced and un-engineered. Unreinforced masonry (URM) is a brittle construction material with little or no deformability beyond its elastic threshold. Therefore, they suffer significant damage during strong earthquakes. Past earthquakes have demonstrated that URM construction is responsible for significant economic loss and human sufferings (Zhuge, Y. 2011). A large stock of existing masonry buildings is vulnerable to seismic excitations. It is economically not viable to replace the existing, seismically deficient URM buildings with new and improved buildings. Therefore, retrofitting of masonry buildings built prior to the enactment of modern seismic codes remains to be the most viable seismic risk mitigation strategy.

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 mostly unreinforced, with some recently built walls being partially reinforced. Their lateral load resistance is low relative to the seismic force demands specified in the current building code (NBCC 2010). Shear strength of masonry walls are limited to the frictional resistance of bed and head joints or the diagonal tension and compression capacity of URM. Flexural resistance is also limited due to lack of sufficient vertical reinforcement, well anchored into the foundation. Therefore, retrofitting of seismically deficient masonry walls have been adopted as the objective of the current investigation.

Retrofitting of masonry buildings with fibre reinforced polymer sheets has gained popularity during the last decade due to its superior material properties and ease in application. Research

has been conducted to establish effective retrofit strategies for increased in plane and out of plane resistances of load bearing masonry walls. The major advantage of using FRP retrofit systems is that they can be externally applied to existing structural elements without disruption and with relative ease. Because FRP sheets do not increase the mass of a building, dynamic characteristics of the building do not change, and seismic force demands do not increase. In spite of its advantages, the use of FRP in retrofitting URM buildings has been limited due to lack of research, and lack of economically viable and structurally sound retrofit methodologies. There exists specific research needs to establish effective retrofit techniques for improving shear and flexure capacities of load bearing masonry walls with surface bonded FRP.

1.2 Problem Identification

Earthquake resistant buildings are required to have sufficient strength and deformability against seismic effects. Inelastic deformability is desirable to dissipate seismic induced energy. When inelasticity cannot be developed without a significant loss of strength, as in the case of URM, and partially reinforced masonry (PRM) walls, then the buildings need to resist seismic forces elastically. Both URM and PRM load bearing walls do not possess sufficient inelastic deformability. Furthermore, older masonry walls may also lack strength. These walls may be deficient in shear and/or flexure.

Shear strength of masonry walls are limited by the frictional resistance of bed and head joints or diagonal tension capacity of the wall panel. Surface bonded FRP sheets, either oriented in both orthogonal directions, or placed in diagonal directions, covering the entire wall surface, can prevent sliding shear and diagonal tension failures under in-plane seismic forces. When the entire wall panel is covered by FRP, out-of-plane failure may also be prevented. Surface-bonded FRP offers an attractive solution to these deficiencies. A major challenge in such application is to ensure sufficient bond between the FRP composites and the substrates. Therefore, care should be exercised in using surface-bonded sheets.

Another deficiency of URM and PRM walls is the lack of sufficient flexural resistance to in-plane seismic forces. URM walls do not possess tension elements to transfer tensile forces to the adjoining elements or the wall foundation. Load bearing URM walls develop rocking motion

under in-plane seismic forces, with limited wall resistance. PRM walls do have some vertical reinforcement, anchored into the adjoining element or foundation, thereby providing some flexural resistance, but this is often not sufficient to resist seismic force demands. Masonry walls, because of their brittle performance, need sufficiently high lateral force capacity to perform elastically during seismic excitations. The required tension force within the wall panel may be provided by the FRP composite, but the FRP needs to be anchored to the adjoining member to transfer tension. Anchors with sufficient strength, and preferably with ductility, need to be developed for improved flexural resistance.

The surface bonded FRP and the required anchorage system should preferably be applied symmetrically on both sides of the wall panel. However, in practice this approach may not be feasible. Some of the walls in practice may not be accessible from both sides. There is need to develop retrofit techniques that can be implemented on one side of the walls.

The current research is intended to address the research needs identified above, with the objectives and scope stated below.

1.3 Objectives

The primary objective of the present research is to develop innovative retrofit techniques for seismically deficient URM and PRM walls for improved shear and flexural strengths, as well as inelastic wall deformability when possible. The objective also includes the development of anchors suitable for flexural strength enhancement in URM and PRM walls.

1.4 Scope

The scope of the present research project involves experimental and analytical investigations as stated below:

1. Review of existing literature on retrofit techniques for masonry walls by FRP composites and other materials.

2. Small scale anchor tests to establish suitable anchorage techniques for surface bonded FRP retrofit systems to function as tension elements, forming an essential component of flexural resistance.
3. Construction, instrumentation and testing of two large-scale masonry walls (PRM and URM walls) retrofitted with surface-bonded FRP sheets anchored into the concrete foundation.
4. Evaluation of test data and assessment of wall performance.
5. Comparison of retrofitted wall test results with those obtained from companion unretrofitted walls tested earlier as part of an earlier investigation.
6. Development of a truss model for step-by-step truss analysis of the masonry walls tested; and for constructing monotonic force-displacement relationships for the walls to validate the applicability of current analytical techniques for use in retrofit design.
7. Truss analyses of the two retrofitted PRM and URM walls and comparison with experimental results.
8. Presentation of the results of experimental and analytical investigations.

1.5 Thesis Organization

The following provides the outline of the current thesis, and the content of each chapter:

Chapter 1: This chapter introduces the specific research area adopted for the thesis. It discusses research needs and areas that require further research and development. The objectives are identified and the steps followed in the research project are itemized as the scope.

Chapter 2: Chapter 2 provides literature review. Previous research in the area of seismic retrofit of masonry walls is discussed. Both the application of FRP and other techniques are included.

Chapter 3: This chapter presents experimental investigation conducted on small-scale samples to assess performance of different types of anchors as potential candidates for use in subsequent wall tests. The anchors considered include elastic FRP anchors, various combination of FRP and steel, as well as stainless steel sheets for improved strength and ductility.

Chapter 4: Chapter 4 presents the details of two large-scale wall specimens (PRM and URM), retrofitted with surface-mounted FRP for shear strength enhancement, and FRP anchors for flexural strength enhancement and deformability. The chapter includes discussions on the construction methods used, specimen geometry, instrumentation, test set-up, and test procedure.

Chapter 5: This chapter presents the results obtained from the wall tests. Both the observations made during testing, and the relationships obtained by processing recorded numerical data, are presented to assess the performance of walls and the retrofit techniques adopted.

Chapter 6: Chapter 6 includes analytical research in the form of truss analyses of two walls to establish their force-deformation relationships, analytically.

Chapter 7: This chapter presents summary and conclusions of the research project described in the previous 6 chapters. It provides recommendations for use in practice.

Chapter 2

Literature Review

2.1 Introduction

Different approaches and materials have been studied by researchers in the past for masonry wall retrofits. These include the application of interior and exterior coatings, filling of window and door openings, and adding new shear walls or steel braces for drift control. The use of surface-bonded FRP sheets has gained popularity in recent years due to their favourable strength, superior mechanical properties, increased durability, and ease of application. Both shear and flexural strength enhancements are achieved when applied on masonry wall. Most of the available literature covers retrofit techniques to enhance in plane shear capacity of masonry walls and very few cover both flexural and shear strength enhancements.

In this chapter, the characteristics of existing partially reinforced and unreinforced masonry walls and their seismic behaviour are presented first. This is followed by the review of available literature on different retrofit strategies. The literature review includes, experimental and analytical research on shear strengthening of masonry walls, shear and flexural strengthening of masonry walls, elastic and ductile anchors required for different retrofit techniques, and the bond characteristics of FRPs on substrates.

2.2 Seismic Behaviour of Unreinforced Masonry Walls (URM)

Unreinforced masonry walls are made of masonry bricks or blocks, and are designed to resist gravity loads. Unreinforced flexural walls can also resist limited amount of out of plane tensile forces (Drysdale, R.G. 2005). Unreinforced masonry walls are easy to construct and no formwork is required. The blocks are laid on mortar bed in successive courses one above another. A large stock of existing buildings in North America is made of unreinforced masonry wall. As there is no reinforcement present in these types of walls, they do not show any ductility. They show brittle and catastrophic failures in the event of strong earthquakes. Therefore, current

Canadian Standards Association (CSA) design standard S304.1-04, recommends the use of minimum horizontal and vertical reinforcement in walls to be built in moderate to severe seismic zones.

The behaviour of unreinforced walls subjected to seismic loads is affected significantly by their aspect ratio and level of axial force (Hatzinikolas and Korany 2005). The capacity of the wall depends on the strength of masonry units and bed joint mortar. When the axial load is low, then two possible failure modes occur, namely; a) rocking failure or b) sliding failure. During the first failure mode a horizontal crack is developed along the base and the wall starts rocking or overturning around its toe. Crushing of masonry may be observed near the toe due to overturning and associated compressive stresses. The second failure mode occurs if the lateral force exceeds the sliding shear capacity of the mortar along the bed joint. If the axial force is exceeded above a certain limit then the sliding and rocking failure mode is controlled but cracking propagates along wall diagonals due to diagonal tension. Under increasing lateral loads, diagonal tension cracks, associated with shear forces, propagate along the principal stress direction until the wall loses its strength. All of these failure modes are brittle, causing extensive damage during strong earthquakes (Drysdale et al. 2005).

2.3 Seismic Behaviour of Partially Reinforced Masonry Walls (PRM)

The CSA standard S304.1-04 calls for minimum seismic reinforcement for load bearing walls located in seismically active regions with a seismic hazard index $[I_E F_a S_a(0.2)]$ of 0.35 and greater. Accordingly, these walls are required to be reinforced horizontally and vertically with steel having area of $0.002A_g$. Masonry walls which have less reinforcement are termed as partially reinforced or lightly reinforced walls. Partially reinforced walls have little ductility, and hence cannot dissipate seismic-induced energy. The vertical reinforcement is provided to pass through the cells of masonry blocks. These cells are then grouted to develop the required bond between the steel and the masonry wall.

Limited research was conducted on the seismic behaviour of partially reinforced masonry walls. Experimental research is especially scarce. These walls perform similar to non-ductile concrete shear walls with insufficient reinforcement, though masonry walls have nonhomogeneous

material characteristics due to the presence of mortar joints. Concrete shear walls develop diagonal tension cracks in concrete whereas the same cracks in masonry walls initiate in mortar joints (Taghdi et al. 2000a). For seismic design of partially reinforced masonry walls, the ductility related force modification factor R_d in the National Building Code of Canada (NBCC 2010) should be taken as 1.0, implying elastic design with no ductility.

2.4 Literature Review

2.4.1 Experimental Research

This section provides review of previously conducted experimental research on retrofit of masonry walls. The majority of literature reviewed involves the use of surface mounted FRP sheets on masonry walls. FRP-steel hybrid retrofit methods and FRP-to-steel bond are also reviewed, as the current experimental investigation includes steel anchors bonded on FRP sheets.

ElGawady et al. (2007) tested seven, one-half scale, unreinforced masonry walls (URM) before and after retrofitting with fibre reinforced polymer (FRP). The test specimens were built using hollow clay masonry blocks and weak mortar to represent the walls built in central Europe in the mid-20th century. Two different moment-to-shear ratios; 0.5 and 0.7 were used to design the specimens. The main parameter used was FRP axial rigidity, which was defined as the amount of FRP reinforcement times the modulus of elasticity. One sided FRP retrofitting was used. Three walls were tested without the FRP reinforcement. These specimens were damaged through testing and subsequently repaired and retrofitted with FRP before they were re-tested. One virgin specimen was retrofitted and tested. The walls were tested under incrementally increasing cyclic loading. Bi-directional glass FRP (GFRP) and unidirectional aramid FRP (AFRP) were used to retrofit the specimens. Flexural strength was achieved by using steel sheets connected to the foundation with anchor bolts. The test results showed that one sided FRP sheet retrofitting can significantly increase stiffness and lateral load resistance. The increase in lateral strength was found to vary linearly with FRP axial rigidity. FRP sheets controlled cracking and the mode of failure varied with the change of FRP axial rigidity. It was reported that the failure mode became more brittle when the FRP axial rigidity was increased. Energy dissipation capacity of the retrofitted specimens was increased. This was attributed to the friction in masonry bed joints.

Taghdi et al. (2000a) tested four concrete block masonry and two reinforced concrete walls to understand the behaviour of non-ductile low-rise masonry and concrete walls under simulated seismic loading. Among the masonry walls, two were unreinforced and two were lightly reinforced. One wall from each pair was retrofitted with a steel strip system on each side of the wall, placed vertically along wall boundaries and diagonally, as shown in Figure 2.1. The strips were attached to the wall with through-thickness bolts and were anchored to the foundation and the top beam with stiff steel angles and anchor bolts. The test result showed that the steel strip retrofitting system could increase significantly the in-plane strength, ductility and energy dissipation capacity of low-rise masonry and concrete walls. It was reported that in the case of retrofitted URM walls, uniform cracking was observed in the masonry wall first, followed by inelastic buckling of steel strips, eventually leading to masonry crushing. In the case of retrofitted PRM and concrete walls, masonry crushing was delayed until after the yielding of steel strips and internal re-bars occurred. The unretrofitted URM and PRM walls of this research project are used in the current investigation as control specimens, companion to the two retrofitted walls discussed in Chapters 4 and 5. The results are compared in Chapter 5.

Papanicolaou et al. (2010) investigated grid type externally bonded material as a retrofit technique. Textile reinforced mortars (TRM) were used as externally bonded grids. Textile was applied in the form of open fibre mesh by means of mortar. Walls with different aspect ratios and compressive strengths were used. They were tested for both in-plane and out-of-plane loading. Number of grid layers, gravity loads, and the amount and type of bonding material were used as test parameters. The walls were lowered into steel casings, which acted as the anchorage for TRM grids. The effectiveness of TRM overlays with FRP was investigated. For out-of-plane loading, 400% increase in strength and 130% increase in deformability were recorded. The TRM overlay was found to perform better than FRP in deformability enhancement; but the opposite was observed in terms of strength improvement.

Valluzzi et al. (2002) tested 33 small scale masonry panels in diagonal compression. A total of 9 URM and 24 strengthened panels were tested. Nominal size of specimens was (510mm × 510 mm × 51mm). Solid clay brick of (55mm × 250mm 120mm) were used. High tensile strength carbon (CFRP), glass (GFRP), and polyvinyl alcohol (PVAFRP) composites were employed as

retrofit materials. The effectiveness of FRP in different orientations, in terms of grid pattern, was investigated. The effect of reinforcement eccentricity was tested by applying FRP on one side of the specimens. Diagonal FRP orientation was found to be the most effective among all the orientations considered. In one-sided retrofitted walls, most of the damage was observed on the unretrofitted side. The ultimate strength of retrofitted specimens was observed to be lower than that for the companion unretrofitted specimens. FRP debonding was observed in most of the two-sided retrofitted specimens. Proper anchoring was suggested to prevent debonding of FRP from the masonry substrate.

Stratford et al. (2004) tested six masonry panels retrofitted with surface bonded GFRP under static in-plane shear force and vertical pre-load. Three of the specimens had clay units, whereas the other three had concrete units. Single-sided strengthening was considered for practicability. The GFRP sheet used had equal amounts of horizontal and vertical fibers. FRP anchorage was applied at the top and bottom of the wall by wrapping the sheets. The application of GFRP on clay masonry resulted in a strength increase of up to 65%. The strength increase in concrete block wall was about 38%. The stiffness and deformation capacity were not increased by the GFRP. The relatively small strength increase achieved in the concrete wall was attributed to the use of inappropriate mortar that dried up before curing. Furthermore, the concrete bricks were porous. Strengthened specimens failed by rapid propagation of diagonal cracks that were observed on the exposed (un-retrofitted) side. Horizontal cracks were observed in unreinforced panels. It was reported that the GFRP was first fully bonded to masonry, but suffered partial debonding in compression, upon cracking of masonry. It was reported that the partially debonded FRP could resist tension. According to the investigators, the debonding failure in this case was not a peeling failure, as there was no out-of-plane bending. Authors stated that, if initial failure could be avoided, than a ductile failure mode could be achieved in masonry joints. The test results showed that the stress level in FRP at the onset of debonding did not increase with bond length. Authors suggested that the debonding length of FRP could be altered, and the failure mode could be changed if the number and the positions of anchors were selected appropriately.

Fam et al. (2002) tested a large scale reinforced clay brick masonry wall retrofitted with GFRP sheets. At first, the specimen was tested prior to retrofitting under reversed cyclic loading and

then the damaged specimen was repaired and retrofitted with GFRP sheets. When the bottom course of the unretrofitted specimen was substantially damaged the specimen was repaired with horizontal and vertical GFRP sheets. An L-shaped connection with a 45° bidirectional GFRP fabric was used between the wall and the foundation. The corner of the wall and the foundation was rounded to avoid stress concentration. It was reported that with the GFRP retrofitting, the wall restored its original strength and the strength was subsequently exceeded. The displacement capacity of the repaired and retrofitted wall was found to be twice the value observed in the original wall.

Alcaino, P. and Santa-Maria, H. (2008) tested sixteen clay brick masonry walls retrofitted with FRP. FRP strips were used in different configurations. Both sides of the walls were retrofitted with horizontal and diagonal strips. The adhesive used for the FRP installation covered the entire masonry substrate. No special anchorage was used in the tests. The specimens were tested under cycles of in-plane displacements. The test specimens were divided into two categories; i) shear reinforced masonry (SRM) and ii) masonry without shear reinforcement (NSRM). Results showed that the strength increase for SRM walls was higher than that of NSRM walls. Diagonally oriented FRP strips resisted more shear forces than those oriented horizontally. Cracks in retrofitted specimens were found to be equal or smaller than those in un-retrofitted specimens. Multiple strips of FRP performed better than single strips.

Marcari et al. (2007) performed experimental research to determine in-plane shear performance of tuff masonry panels. The panels were strengthened with GFRP and CFRP. Tuff was composed of volcanic particles. It was compacted, cemented or welded into a firm consolidated state. This material is widely used in seismically vulnerable areas, such as Italy, Turkey, Japan and the US. Basically two different orientations of FRP reinforcement were used, consisting of diagonal and orthogonal grid patterns. Specimen dimensions were 1570×1480×400 mm. The specimens were subjected to monotonic shear compression tests. A total of 19 specimens were tested. Four specimens were as-built (without retrofitting), four strengthened with grid pattern GFRP, four strengthened with cross pattern GFRP, four specimens with grid pattern CFRP and four additional specimens with cross pattern CFRP. FRP sheets were applied on both surfaces of the wall. The anchorage of FRP was ensured by providing extra strips of FRP that enclosed the

diagonal and vertical strips at the top and bottom of the walls. This is illustrated in Figure 2.2. All of the as built (un-retrofitted) specimens failed in shear. The low density, cross pattern CFRP retrofitted specimens showed shear dominant failure with a 30% increase in shear strength. The FRP strips buckled in low-density GFRP retrofitted specimens. This limited the strength increase to 17%. In high-density CFRP and GFRP retrofitted specimens, shear-flexure failure mode was observed. The strength increase in CFRP-retrofitted specimens was 53%. This increase was 23% in GFRP-retrofitted specimens. Low-density grid type CFRP and GFRP retrofitted specimens suffered from shear type failures. In the high-density retrofitted specimens however, the failure mode changed to shear-flexure type. For low density CFRP and GFRP specimens the strength increase was 50% and 48%, respectively. High-density CFRP and GFRP resulted in 67% and 62% strength increases, respectively. The researchers concluded that the use of high-density FRP changes the failure mode from axial-rigidity failure to shear or shear-flexure modes. The elastic stiffness of the specimens did not improve by the application of FRP. Strength increase was achieved without an increase in inelastic deformation capacity. GFRP was found to be more compatible with masonry substrates. GFRP rupturing was detected in most cases.

Thanasis et al. (1998) performed systematic analysis of externally bonded FRP-retrofitted masonry walls. Walls were tested for out-of-plane bending, in-plane bending, and in-plane shear under monotonic loading. Uni-directional FRP strips were used instead of two dimensional fabrics covering the entire surface. Mechanical behaviour of FRP was investigated through analytical procedures. It was concluded that out-of-plane bending occurred under low and moderate levels of axial load. The moment capacity increased with an increase in the fraction of FRP covering the surface. For the same coverage of FRP, the moment capacity decreased with increasing axial load. As for in-plane bending, the moment capacity increased with increasing FRP area. For the same FRP area, the moment capacity decreased with increasing axial load. The shear capacity of walls subjected to in-plane forces increased with increasing FRP area, but the shear capacity was attained at a relatively low value of FRP area. Small scale experiments were also performed and ultimate load was compared with theoretically computed data. The theoretical and experimental results were found to be very close. During the in-plane tests, the FRP peeled off due to lack of appropriate anchorage. The authors suggested that a strain limit be established for debonding of FRP for analytical calculations.

Hamid et al. (2004) tested 42 one-third scale unreinforced masonry assemblages under different stress conditions that typically exist in masonry infill and shear walls. The masonry assemblages considered are shown in Figure 2.3. The masonry prisms were loaded in compression with different bed joint orientations under diagonal tension and joint shear. The prisms were retrofitted on both sides with GFRP sheets. It was reported that the laminate significantly increased the load carrying capacity of masonry assemblages. The highest increase was found in direct joint shear assemblages, and the lowest increase was found for 0^0 and 90^0 assemblages. It was found that the unretrofitted specimens failed suddenly in comparison with those retrofitted. The high anisotropy that exists in masonry prisms was reduced due to the use of FRP.

Ozbakkaloglu and Saatcioglu (2009) investigated the tensile behaviour of FRP anchors embedded in concrete. The test parameters considered in their investigation were; length, diameter, and angle of inclination of anchors, as well as the compressive strength of concrete. A total 81 CFRP anchors were tested under direct pullout. The anchors were prepared by cutting fibre strips from FRP sheets, and rolling them into fan-shaped anchors. The anchors were inserted in concrete cylinder and pulled-out to establish their strength. Some anchor samples are shown in Figure 2.4. The failure modes included cone failure of concrete, combined cone-bond failure, anchor rupture, and concrete splitting. It was reported that FRP anchors can effectively prevent or delay the debonding of surface bonded FRP. Assuming uniform bond strength, the researchers calculated the bond strength of FRP anchors. It was concluded that FRP anchors are capable of achieving average bond strength of 10 MPa to 15 MPa. Investigators observed that the depth of pullout cone decreased with increase of embedment length. For shallow embedment length, the failure mode was cone failure. For embedment lengths larger than 50-75 mm, the failure mode was combined cone-bond failure. The average bond strength decreased with increasing anchor diameter. Bond strength also decreased with increasing embedment length. Pull out capacity of anchors decreased with increasing inclination angle. It was found that the concrete strength had a little effect on bond strength. Sufficient amounts of fibres should be provided to avoid failure by FRP rupture. Three models proposed by ACI (1997) and Fuchs et al. (1995), as well as the modified ACI 349-85 (1997) model were discussed for calculating the pull-out capacity. The experimental results were compared with these models and it was found that the Fuch's(1995) model over predicted pull-out capacities in high-strength

concretes. Modified ACI 349 model was close enough for both high-strength and normal-strength concretes. For normal strength concrete the Fuch's results were found to be close to the experimental results.

Prota et al. (2005) tested four under-designed RC square columns retrofitted with GFRP laminates and steel spikes. The specimens were tested under monotonically increasing lateral loads. Four unretrofitted companion specimens were also tested under the same loading condition. GFRP laminates were wrapped around the columns and anchored into the foundation by steel spikes. The spikes consisted of five 3×2 zinc coated steel cords, twisted together to form a spike. At first 300mm deep holes were made in concrete foundation. 700mm×70 mm strips were cut from a roll of steel tape, and then 300 mm portion of the tape was twisted and inserted into the concrete foundation with epoxy. The rest was glued on the column surface. It was reported that the retrofitted specimens showed significant strength enhancement and ductility.

Hall et al. (2002) tested and analysed ductile structural steel connections that can be used in FRP strengthened shear walls for anchorage to their foundations. FRP sheets were applied for flexural strengthening of walls. A hybrid connection of steel and FRP was used. Because of the brittle failure mode of FRP, it was intended to design the connections in such a way that ductile connection failure would be promoted prior to the FRP rupture. In the specimens, the footing-wall connection was simulated. A total of 15 small-scale tests with two masonry units were performed with different configuration of GFRP sheets and steel angles. The angle of the steel plate was rounded into different diameters, and the plate was connected to the foundation with bolts. The position of the bolt and the exterior surface of masonry varied in each specimen. Tensile tests were conducted with the help of hydraulic ram and hydraulic pump. Failure modes observed during tests included GFRP delamination, GFRP composite rupture, glass fibre tearing, and plate yielding, where the plate yielding was the desirable failure mode. The radius of steel angle affected the ultimate strength of connections. Ultimate strength found for the connections was half of the GFRP coupon strength. Displacement ductility factor was recorded to vary between 6.3 and 29.5 with steel yielding occurring prior to the GFRP rupture.

Holberg and Hamilton (2002) tested five URM walls retrofitted with unidirectional GFRP and specially designed ductile anchors as shown in Figure 2.5. The anchors were designed to yield

prior to fibre rupture. Static cyclic tests were performed on four large scale unreinforced concrete block masonry walls. The FRP sheets were configured to increase both flexural and shear strengths. Vertical strips were used for flexural strengthening and diagonal strips were used for shear. Two types of steel connections were used to transfer tensile forces from the FRP sheets to the foundation. Steel angle assembly was used as external connection, and ordinary 3/8 in (9.5 mm diameter) bars were used as internal connection. Steel connections were inserted into the concrete foundation with epoxy after removing four courses of face shells. The cells containing reinforcement were filled with grout. During testing, the GFRP debonded, buckled and finally ruptured above the steel angle plate location, when a steel angle was used as a connection. Reinforcing bars were inserted into the foundation to provide tensile resistance during the rest of the test. During one of the tests, steel reinforcement pulled out from the foundation. In another test, out of plane failure was observed due to the eccentricity of the rebar and the FRP sheet. In the later test, the gap between the rebar and the FRP surface was reduced, and yielding of the rebar was achieved. Unretrofitted and retrofitted wall capacities were calculated from simple mechanics, and showed good agreement with experimental values.

Saatcioglu et al. (2005) tested two half scale reinforced concrete frames with block URM infill walls under simulated seismic loading to understand the interaction of URM infilled walls with the surrounding frames, and to develop a seismic retrofit strategy. Among the two specimens tested for each wall type, the first specimen reflected the pre-1970s construction practice, and the second specimen was retrofitted with CFRP sheets for improved performance. The results of the unretrofitted specimen showed that infill walls can provide significant lateral stiffness during seismic response if the wall is not isolated from the surrounding frame. Infill walls can provide sufficient drift control even after their elastic limit is exceeded. The test wall suffered significant strength and stiffness degradation beyond its elastic limit. Gradual stiffness degradation was observed due to the progressive cracking in masonry units and mortar joints. It was concluded that the infill wall and the reinforced concrete frame were compatible to resist lateral deformations together. At the initial stage, the lateral load was mostly resisted by the infill masonry, and then it transferred the load resistance to the surrounding concrete frame. The wall failed due to the hinging of columns in the reinforcement splice region at the ends. The specimen with CFRP sheets was able to maintain its integrity with cracks completely controlled and

stiffness deterioration reduced, when the sheets were well anchored into the surrounding frame. The specimen was retrofitted with single layer of CFRP sheet on each side of the wall, placed parallel to each diagonal. Surface bonded CFRP sheets were able to improve the overall strength of the structural system by a factor of three times relative to that of the unretrofitted specimen when the sheets were properly anchored to the surrounding frame. The FRP anchors promoted frame-infill interaction and prevented debonding. CFRP sheets ruptured near the corner along the diagonals, and the specimen showed strength degradation. After the CFRP rupture, the overall behaviour of the retrofitted specimen became the same as that of the unretrofitted specimen.

Shalouf (2005) tested four reinforced concrete frames with infill masonry walls retrofitted with diagonal prestressing cables and diagonal CFRP sheets. Three infill walls were built with clay brick masonry, and one with concrete block masonry. The frame with concrete block masonry infill was retrofitted with diagonal CFRP strips, one on each side of the wall, attached to the surrounding frame by specially manufactured CFRP anchors. Among the frames with clay brick masonry walls, one was used as a control specimen and the other two were retrofitted with diagonal prestressing. The specimens were subjected to gravity loading by means of external prestressing. Gravity loading was applied prior to the application of lateral load to simulate dead load and live load. The specimens were tested under in plane lateral load in displacement control mode. The diagonal prestressing was found to be an effective retrofit strategy for increasing lateral load capacity while controlling lateral drift. The surface bonded FRP sheets were also found to be effective in retrofitting lateral drift in non-ductile concrete frames. Lateral load resistance of the retrofitted specimen was found to be 2 to 2.4 times higher than that of the unretrofitted specimen. The CFRP anchors were found to be effective. Inelastic push-over analysis was conducted using software DRAIN-RC, and the results were compared with the force-displacement envelopes obtained experimentally. The comparison showed good agreement between the two sets of results.

Fawzia (2011) tested CFRP bonded steel plates with double straps under tensile loading. It was reported that the adhesive should be carefully chosen to get proper bond between the steel and CFRP sheets. The key mechanical properties for attaining highest bond efficiency from the epoxy resin were reported to be strength, elastic modulus and percentage elongation. Two small

steel plates were grinded by angle grinder and bonded together by epoxy adhesive. Then three layers of CFRP sheets were bonded on each side of the steel plate. The bond length at one end was kept larger than the other to promote failure at one end. An LVDT and strain gauges were fixed to obtain slip between the CFRP and the steel plate, while also measuring shear stress. The distribution of microstrain along the bond length was plotted. It was found that the distribution was highly nonlinear at low levels of loading, and it became linear when the load was increased. Shear stress and slip between CFRP and steel plate was observed to be bilinear. A stress-based model was proposed to compute the ultimate load capacity. The model was found to be in good agreement with experimental results.

2.4.2 Analytical Research

A number of analytical approaches were developed to predict the capacity and behaviour of retrofitted masonry walls. These approaches are presented in this section.

Zhuge (2010) reviewed all existing analytical methods for the calculation of FRP-Retrofitted URM walls subjected to in-plane shear forces. The available expressions were assessed based on the experimental data reported in the literature. A total of seven analytical approaches were reviewed. Accordingly, most design expressions are based on the assumption that total contribution to shear consists of the summation of two term; i) the contribution from uncracked masonry V_m , and ii) the contribution from FRP reinforcement, V_{FRP} .

$$V = V_m + V_{FRP}$$

The analytical approaches were classified into two categories; i) strain based approaches and ii) truss-analogy based approaches.

Triantafillou (1998) suggested an expression which was basically suitable for narrow strips. The contribution of vertical reinforcement was considered to be negligible, and the contribution from horizontal shear reinforcement was considered as the only resisting component. This approach was analogous to stirrup design in reinforced concrete beams.

$$V_{FRP} = \rho_h E_{FRP} \left(r \frac{\epsilon_{FRP,u}}{\gamma_{FRP}} \right) t \times 0.9d \quad (2.1)$$

Where, r is reinforcement efficiency factor, which depends on the exact FRP failure mechanism, i.e., debonding or tensile fracture, ρ_h is the fraction of FRP area relative to wall area, $\varepsilon_{FRP,u}$ is the ultimate tensile strain of FRP. The thickness and the effective depth of masonry wall are denoted by t and d , respectively. γ_{FRP} is partial safety factor for FRP in uniaxial tension.

$$\text{If, } \varepsilon_{FRP,e} = \varepsilon_{FRP,u}$$

$$V_{FRP} = \frac{0.7}{\gamma_{FRP}} \times \rho_h \times E_{FRP} \times \varepsilon_{FRP,e} \times l \times t \quad (2.2)$$

The expression for calculating the effective FRP strain by the same researcher is given below:

$$\varepsilon_{FRP,e} = 0.0119 - 0.0205(\rho_h E_{FRP}) + 0.0104(\rho_h E_{FRP}) + 0.0104(\rho_h E_{FRP})^2 \quad (2.3)$$

This above expression was found by regression analysis of concrete members strengthened with FRP in shear.

Triantafillou and Antonopoulos (2000) improved their earlier model by distinguishing failure modes (FRP debonding or rupture) and the types of FRP materials (CFRP or AFRP). They then suggested the following:

For fully wrapped CFRP:

$$\varepsilon_{FRP,e} = 0.17 \left(\frac{f_c^{2/3}}{E_{FRP} \rho_h} \right)^{0.3} \varepsilon_{FRP,u} \quad (2.4)$$

Side or U-Shaped CFRP jackets:

$$\varepsilon_{FRP,e} = \min \left[0.065 \left(\frac{f_c^{2/3}}{E_{FRP} \rho_h} \right)^{0.56} \times 10^{-3}, 0.17 \left(\frac{f_c^{2/3}}{E_{FRP} \rho_h} \right)^{0.3} \varepsilon_{FRP,u} \right] \quad (2.5)$$

For fully wrapped AFRP:

$$\varepsilon_{FRP,e} = 0.048 \left(\frac{f_c^{2/3}}{E_{FRP} \rho_h} \right)^{0.47} \varepsilon_{FRP,u} \quad (2.6)$$

Where,

f_c = Compressive strength of masonry and $\varepsilon_{FRP,u}$ = 0.015 for CFRP and 0.035 for AFRP.

The orientation of FRP and different retrofitting schemes were not considered in these expressions.

ACI Committee 125 developed an analytical approach for the computation of shear resistance of FRP. The following formulas are given for rectangular walls retrofitted on one side or both sides. The expressions are summarized by the ICC Evaluation Service, Inc (2007).

$$V_{FRP} = 2t_{FRP}f_jH\sin^2\theta \quad (2.7)$$

Where, t_{FRP} =FRP thickness, H =Length of the wall, θ =Fiber orientation.

$f_j=0.004 E_{FRP} \leq 0.75f_{uj}$, f_{uj} =Ultimate tensile strength of composite material (MPa)

When FRP is bonded only on one side with fibre orientation to a have 75° or larger relative to the member axis, then the nominal shear strength enhancement becomes;

$$V_{FRP}=0.75t_{FRP}f_jH\sin^2\theta \quad (2.8)$$

In the above expression, the effective strain is taken as constant and equal to 0.0015 for one sided applications and equal to 0.004 when fully wrapped.

The accepted retrofit scheme specified in recent Chinese Standard (2006) consists of horizontal, diagonal and mixed fibre orientations (combinations of horizontal and diagonal). In this code, V_{FRP} is calculated as shown below:

$$V_{FRP} = \zeta E_f \varepsilon_{fd} \sum_{i=1}^n A_{fi} \cos\theta_i \quad (2.9)$$

Where, ζ is the FRP participation coefficient, which depends on wall's aspect ratio for horizontal FRP retrofitting and the number of FRP strips for diagonal retrofitting. E_f is Young's modulus of FRP and ε_{fd} is the design value for effective strain of FRP, which is equal to $\frac{\varepsilon_{fe}}{\gamma_e}$ where ε_{fe} is the effective strain of FRP varying between 0.001 and 0.002 depending on the end anchoring condition. γ_e is the environmental and partial safety factor. A_{fi} is the cross sectional area of the i th FRP shear reinforcement and θ_i angle of inclination of the i th FRP strip. n is the total number of shear reinforcement. An aspect of the Chinese code that is noteworthy is the consideration of different retrofitting schemes and fibre orientation in design.

A number of researchers suggested using truss analogy for computing wall response. The application of trusses resembles to that for reinforced concrete beam shear models, where beam stirrups are used as tension elements. In the wall analysis, the FRP forms the tension ties.

Nanni and Tumialan (2003) suggested a truss model, which resulted in the following expression for shear resistance of FRP.

$$V_{FRP} = k_f \left(\frac{A_{frp}}{s} \right) f_{fu}^* d \quad (2.10)$$

Where, A_{FRP} is the cross sectional area of FRP shear reinforcement, s is the spacing of shear reinforcement and d is the actual depth of masonry in the direction of shear considered. $k_f = 0.5$ for defining effective stress in FRP, which is limited to 50% of FRP ultimate capacity.

The following assumptions are considered in this model:

- Inclination angle of shear cracks is constant and taken equal to 45°
- Effective strength is reached in all reinforcement intersected by the diagonal crack
- FRP reinforcement carries all the shear demand.

Garbin et al. (2007) improved the above approach and suggested the following expression:

$$V_{FRP} = K_f \left(\frac{A_{FRP}}{s} \right) f_{fe} d \quad (2.11)$$

The tensile strength f_{fu}^* in Eq. (2.10) is replaced by the effective design strength, f_{fe} , where;

$$f_{fe} = k_m f_{fu} = k_m c_E f_{fu}^* \quad (2.12)$$

In the above expression, k_m is the strengthening system factor (usually taken as 0.65) and c_E is the environmental reduction factor. When FRP alone resist the shear force then;

$$V_{FRP} = k_v A_{FRP} f_{fu} \quad (2.13)$$

k_v stands for the orientation angle of fibre with respect to the failure surface. k_v varies between 0.5 and 0.8.

Zhuge et al. (2010) suggested that the spacing term s may be taken equal to the width of FRP $s = w_{FRP}$ when continuous FRP strips are used, or one FRP sheet is placed in the diagonal direction.

The Italian Standard, CNR DT 200 (Italian National Research Council 2004) provided the following expression for FRP strengthening, when the fibres are placed parallel to the mortar joint:

$$V_{FRP} = \frac{1}{\gamma_{Rd}} \frac{0.6 \times d \times A_{fw} \times f_{fd}}{P_f} \quad (2.14)$$

Where, γ_{Rd} is the partial safety factor (1.2 is taken for shear), and d is the distance between the compression side of masonry and the centroid of FRP used for flexural strengthening. A_{fw} is the area of FRP used for shear strengthening in the direction of shear force. P_f is the centre to centre spacing of FRP reinforcement measured orthogonally to the direction of shear force, and f_{fd} is the design strength of FRP reinforcement, which is defined as the smaller of FRP tensile failure strength and debonding strength.

When debonding becomes a potential failure mode, and the bond length is longer or equal to the optimum length, the design bond strength f_{fd} is expressed as shown below.

$$f_{fd} = \sqrt{\frac{2E_f F_{FK}}{t_f}} \quad (2.15)$$

The corresponding design bond strain is given as;

$$\varepsilon_{fd} = \sqrt{\frac{2F_{FK}}{E_f t_f}} \quad (2.16)$$

Where, E_f is Young's modulus of elasticity for FRP, t_f is FRP thickness, and F_{fk} is the specific fracture energy of FRP strengthened masonry. The latter quantity may be computed by;

$$F_{FK} = c_1 \sqrt{(f_{mk} f_{mtm})} \quad (2.17)$$

Where, c_1 is experimentally determined coefficient and may be taken as 0.015, f_{mk} is the characteristic compression strength of masonry, and f_{mtm} is the average tensile strength of masonry which may be taken as $0.1f_{mk}$.

A database of previous experiments for in-plane tests of masonry walls retrofitted with FRP was compiled and statistical analysis was performed by Zhuge et al.(2010). The results indicated that ACI 125 and CNR- TD (Italian National Research Council 2004) approaches showed better correlation with data than any other approach. The authors reported that the test data generated to date in describing FRP strain distribution does not provide sufficiently conclusive results. Therefore, more research is needed on the topic.

Prota (2008) performed a comparison between the results of available experimental data on FRP strengthened walls and the theoretical approaches suggested by previous researchers. For lateral strength calculation, where the FRP was used parallel to the mortar joint, the formula proposed by CNR DT 200 (CNR 2004) gave good results. For walls with diagonal FRP orientation, a simple truss model with FRP strips providing tension ties and masonry providing compressive struts provided reasonably good results. When the FRP vertical strips were used without any continuity at the ends, then the masonry shear strength was found to remain the same as that of unretrofitted URM walls. When FRP was placed in a grid pattern, the computed lateral strength calculated by the CNR DT 200 (CNR 2004) approach resulted in good estimates of experimental data.

Taghdi et al. (2000b) proposed an indeterminate truss model for low rise walls retrofitted with vertical and diagonal steel strips. The model involves a step-by-step analysis to establish a force displacement relationship. The analytical model validated the experimental results obtained by Taghdi et al. (2000a). Simple and improved truss models were provided where each material of a wall was modelled as a truss member with assigned material properties. A series of elastic push-over analyses was performed where stiffness and strength of each member was reduced gradually. Once a tensile member reached at its yield strength, then that member was replaced by a constant force and the analysis was done with the rest of the members. If a member failed in compression, then it was removed from the analysis. The model gives a force-displacement relationship that could be used for making comparison with experimental result. A lower bound

approach was used to define the wall strength. Finally a design procedure was proposed for the steel strip retrofitting system.

2.5 Summary

Previous research on seismic retrofit of masonry walls was reviewed. Both experimental and analytical research was considered. The following conclusions were obtained:

- Surface-bonded FRP sheets can control the propagation of cracks in walls, increasing the shear strength of masonry walls. Shear retrofitting can be accomplished by diagonal FRP sheets (Valluzi et al. 2002), grid pattern FRP sheets (Marcari et al. 2006) or fully surface covered FRP sheets (ElGawady et al. 2005).
- The application of FRP on single face of masonry walls was found to be a successful retrofit methodology. Valluzi et al. (2002), however, found reduced strength in walls with single sided wall FRP due to the asymmetry created in force resistance and the resulting eccentricity of forces.
- Very few researches conducted experiments on flexural strengthening of masonry walls with surface-bonded FRP. Hall, et al.(2000) and Holberg, A. M. (2002) tested masonry walls with ductile anchorage systems. The difficulty of connecting surface mounted FRP sheets to the adjoining members remains to be a challenge. When steel plates were used as the anchorage system, FRP rupture was observed near the connection region. Holberg et al. (2002) suggested inserting internal steel reinforcement while providing surface-mounted FRP sheets with fibres aligned in the vertical direction
- Limited ductility was observed during rocking of URM walls or yielding of reinforcement in PRM walls. To achieve ductile response with these systems, the surface-mounted FRP sheets should be connected to the foundation or the slab below with ductile materials.
- Analytical approaches were developed to define the contribution of FRP to shear in masonry walls. The capacity of FRP retrofitted masonry walls can be established either by empirical equations or through truss analogy. The empirical approach of the Italian Standard, CNR DT 200 (Italian National Research Council, 2004) appears to provide good estimates of shear strength in FRP retrofitted masonry walls. Truss analogy

provides an excellent analytical tool for establishing flexural and shear strength in FRP retrofitted masonry walls.



Figure 2.1: Unreinforced masonry wall retrofitted with steel strip system (Adapted from Taghdi et al. 2000a)

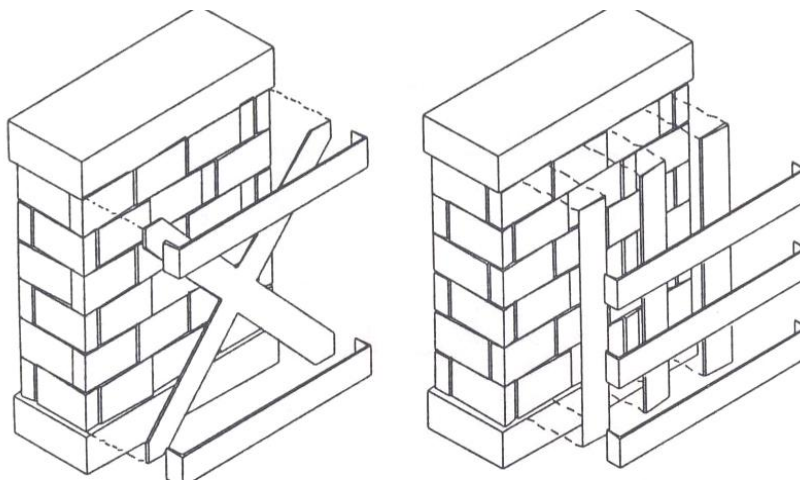


Figure 2.2: U-wrap system for cross layout (left) and grid layout (right) (Adapted from Marcari et al. 2007)

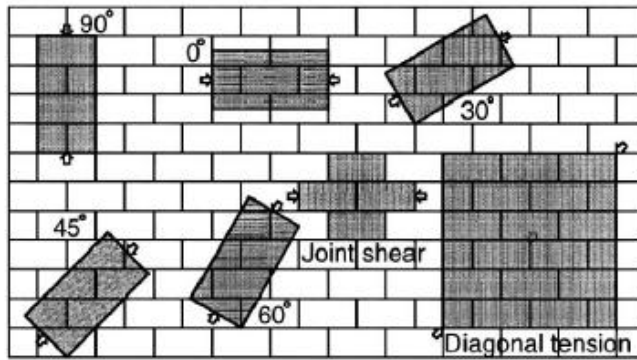


Figure 2.3: Different assemblages as a part of a wall (Adapted from Hamid et al. 2004)



Figure 2.4: FRP anchors placed in a steel barrels as part of a test setup (Adapted from Ozbakkaloglu et al. 2009)

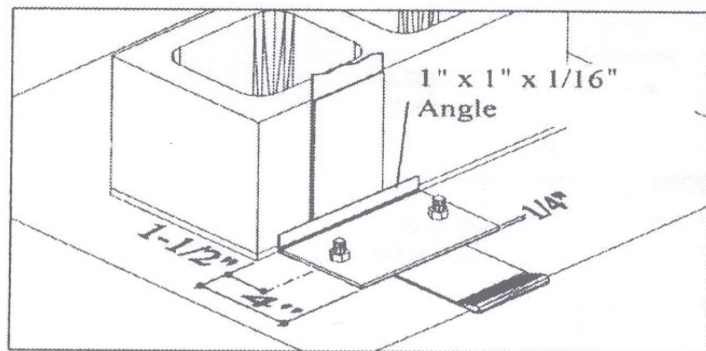


Figure 2.5: Ductile anchor with steel angle (Adapted from Holberg, A. M. 2002)

Chapter 3

Anchor Development

3.1 Introduction

Flexural strengthening of load bearing masonry walls with externally bonded FRP systems requires anchorage of FRP sheets to the adjoining elements (foundation or slabs). Two types of anchorage systems have been developed as part of the current investigation. These consist of, i) elastic (but brittle) FRP anchors for strength enhancement, and ii) ductile inelastic anchorage systems incorporating steel elements for strength and ductility enhancements. Elastic anchors are designed to increase elastic wall capacity without achieving any ductility, whereas ductile anchors are designed to develop yielding prior to the wall FRP capacity. The elastic anchors are produced from CFRP laminates, and the ductile anchors are produced from stainless steel sheets.

The flexural anchor development in the current project involves three stages; i) the application and verification of FRP fan type anchors based on previous research (Ozbakkaloglu and Saatcioglu 2009), ii) the development of new elastic anchors consisting of FRP laminates, and iii) the development of new ductile anchors, consisting of ductile materials. The development of new anchors involves a series of small-scale tests, performed with different materials and different techniques to attain anticipated force levels in critical regions of walls. The details of anchor development, as well as the description of the previously developed and used FRP anchor, are presented in the following sections.

3.2 FRP Fan Type Anchors

FRP anchors are made from fibre reinforced polymer sheets with a mechanism similar to that of adhesive anchors. They were developed by Saatcioglu et al. (2005) at the University of Ottawa, and used in masonry infilled reinforced concrete frame systems. Figure 3.1(a) shows an FRP anchor, which is produced in-house in the Structures Laboratory by cutting and twisting FRP sheets. These anchors have two parts: one part is rolled and tied with strings, and the other part is

kept open for fanning and epoxy gluing on FRP material. The rolled part is inserted in a pre-drilled hole in concrete before it is adhered with epoxy. The application to masonry infilled reinforced concrete frames shows that these fan-type FRP anchors prevent premature delimitation of FRP sheets, improving overall structural performance (Saatcioglu et al., 2005). The same type of anchors were used in the current project as tension elements for transferring forces from the foundation to surface-bonded FRP on masonry walls. Hence, the anchors function as vertical reinforcement. The anchor capacity was adopted from previous research (Ozbakkaloglu and Saatcioglu 2009). The detailed design and application procedure is described in Chapter 4.

Another application of the fan-type anchors involves additional physical anchorage of surface-bonded FRP sheets on the wall surface. Figure 3.1 (b) shows the FRP anchors on the wall and the foundation.

3.3 Elastic Anchors with FRP Laminates

Instead of the fan-type FRP anchors described above, FRP laminates can also be used for flexural anchorage. For the specific problem at hand, which involves flexural strengthening masonry walls, a technique was proposed for inserting FRP laminates into the foundation or slab (adjoining member to which flexural reinforcement is to be anchored) concrete vertically and securing them by injecting epoxy. This involves cutting the concrete and inserting previously cured FRP laminates into the concrete slots in adjoining members, and adhering one end into the concrete and the other end to the surface-bonded FRP sheets on masonry walls by epoxy. The foundation or slab concrete can be cut by a concrete cutter. The cut location should be kept close to the wall to prevent lifting of the laminates from the wall surface during load transfer. Because the foundation concrete essentially provides compression resistance, the cutting of foundation concrete and filling it with epoxy is not of concern in terms of damaging the functionality of concrete in the adjacent member. The performance of this type of anchors was assessed by testing small-scale specimens. Two sets of small-scale tests were performed, as described below.

Three concrete cylinders were used to represent concrete in adjacent elements, to which wall is to be anchored. The cylinders were 100 mm in diameter and 200 mm in length. The cylinder

concrete was cut 100 mm deep, with a 100 mm length and 2 mm slot width. They were cut by a professional concrete cutting company.

Single layer of 600mm ×100 mm FRP sheets were cut to prepare FRP laminates. The sheets were soaked in two-component epoxy and cured for 3 days. Two steel plates were glued at one end of the FRP sheets to facilitate the gripping of specimens by the machine head without damaging the FRP. The cylinders were wrapped with plastic tape before inserting FRP laminates and pouring epoxy to eliminate the draining of the epoxy through the slot that was pre-cut in concrete. Then, the FRP laminates were inserted into the concrete cylinders. The specimens were cured for at least three days. A test set up was manufactured with a steel plate and threaded bars to support the cylinders in the machine, as shown in the Figure 3.2. The steel plate had an opening to allow for the FRP laminate to pass through the steel plate. The setup allowed the application of pull-out force on the FRP laminate. Potential failure modes included the formation of concrete pull out cone, rupturing of FRP sheet, and debonding of FRP laminate from concrete.

In this set of tests, concrete cylinders failed in tension prior to the rupturing of FRP or debonding of anchors. In each case the concrete cylinder failed in tension from one side. The test results are shown in Figure 3.3 to 3.5. The maximum FRP stress was observed to be 380 MPa, 440 MPa and 320 MPa, all of which were lower than the FRP rupturing strength. It was then decided to reduce the laminate strip width to promote FRP rupturing or bond failure prior to tension failure of concrete.

A second set of three small scale tests were performed with a narrow strip width of 25 mm, and using the same size concrete cylinders. The cylinders were placed inside the machine as shown in Figure 3.6 to 3.8. In this series, FRP rupture was observed at 17.5 kN, which was less than the concrete tensile capacity observed in the first test series. The FRP embedment length was kept at 100 mm, which was the same as before. The same steel plates were used but the specimens were kept inside the machine. As before, two steel plates were used at the top end of FRP laminates to facilitate gripping of the ends. Figures 3.6 through 3.8 illustrate the observed failure modes and the bond stress characteristics during testing.

This set of tests were successful in terms of establishing the anchor capacity as FRP sheets developed their tensile capacity and ruptured. The FRP strength was found to be 760 MPa, 700 MPa and 580 MPa for the first, second and third samples, respectively. No FRP bond failure or concrete pull-out cone was observed. The bond stress corresponding to FRP rupture was found from the first, second and third samples as 3.5 MPa, 3.44 MPa and 2.84 MPa. These values suggest that the FRP bond strength in concrete is at least 2.84 MPa.

3.4 Ductile Anchorage Systems

Masonry walls are typically very stiff elements, resulting in short building periods and higher response accelerations during earthquakes. It is not economically viable to design buildings to remain elastic under such high seismic forces. Therefore, CSA 304.1-04 recommends ductile design to reduce seismic design forces. The required ductility in new masonry buildings is introduced through proper seismic design and detailing.

In the case of retrofitting existing buildings, it is desirable to include ductility and energy dissipation capacity so that the ductility related force modification factor becomes higher than 1 ($R_d > 1.0$), and the force demand reduces below the elastic level. At the same time the use of surface bonded FRP on masonry walls has become a viable retrofit strategy. The brittle behaviour of FRP reinforced masonry can be altered to become ductile if combined with ductile materials.

A retrofit methodology was developed in the current research project to enhance ductility and energy dissipation capacity of masonry walls reinforced with surface bonded FRP systems by incorporating ductile anchorage systems for improved flexural strength and ductility. The proposed retrofit methodology includes the use of surface-bonded FRP sheets with fibres in horizontal and vertical directions to control diagonal tension cracks while increasing shear capacity above flexural yielding to prevent brittle shear failure. The longitudinal fibres are intended to serve as flexural reinforcement, but need ductile anchors to transfer tensile forces into the adjacent member. In single storey masonry walls, representing first-storey bearing walls, the adjacent structural element for the anchorage of flexural reinforcement is concrete foundation. Therefore, a compatible anchorage system is needed to connect the longitudinal FRP

sheets on masonry walls to concrete foundation by means of ductile steel elements. These anchors have to possess two important qualities to fulfill the function for which they are designed; i) sufficient ductility beyond flexural yielding, and ii) sufficient bond strength during inelastic response to prevent premature bond failure.

Various types of steel elements were considered to meet the first objective. Different arrangements and combinations of steel and FRP were considered to fulfill the second objective. The anchorage systems considered in the current investigation included the following cases:

- i) Steel sheets
- ii) High-strength wire
- iii) Hard-wire tape
- iv) High-strength ductile steel sheets

In all cases the intent was to cut a narrow slot in foundation concrete immediately below the wall surface, insert the steel anchor and adhere it into concrete with epoxy, while gluing it on the surface mounted wall FRP at the other end by epoxy, and covering it with another layer of FRP. Therefore, bond between steel and concrete, as well as between steel and FRP were of concern, though epoxy glued steel in concrete has had a successful history of use in the construction industry and hence did not pose as much of a challenge as the bond between steel and surface-bonded wall FRP did. Therefore, the emphasis was placed on bond between steel and FRP.

3.4.1 Bond Between Steel Sheets and FRP laminates

Strips of thin and flexible steel sheets were used to investigate the bond characteristics of steel sheets when sandwiched between two FRP sheets. Small-scale samples were tested in direct tension for this purpose. Figure 3.9 shows typical tension pull-out tests conducted. A thickness of 0.6 mm was selected for the steel strip. The strip width was approximately 100 mm. Two steel strips of equal length were connected by sandwiching them between two layers of CFRP laminates with different bond areas. Three different bond areas were used to establish the bond strength between steel and FRP. The bond areas used were 17780 mm^2 ($2 \times 100 \times 88.9$); 26670 mm^2 ($2 \times 150 \times 88.9$) and 35560 mm^2 ($2 \times 200 \times 88.9$). The steel sheets were cut into $300 \text{ mm} \times 88.9 \text{ mm}$ pieces by a punch shear machine. The 88.9 mm width allowed the strips to fit in the universal tensile machine grips. FRP sheets were cut in 200 mm, 300 mm, and 400 mm lengths,

providing sufficient length to connect and cover two steel strips with different bond areas. All the specimens failed through bond failure as illustrated in Figures 3.10 through 3.12. Test results and test observations are shown in the same figures in the form of bond stress versus machine head movement. A significant change in the rate of vertical movement of the machine head under constant bond stress, followed by a drop in resistance signified the bond failure. The bond strength was found to be 0.9MPa, 0.7 MPa, and 0.53 MPa for the first, second and third samples, respectively. A steel sheet without the FRP was also tested to establish the behaviour of steel alone. The steel yielded at about the same load level as the load that caused bond failure in the three samples. Figure 3.13 illustrates the stress vs. machine head position relationship of steel sheets, established experimentally.

To improve bond between steel and FRP the steel sheet was sand blasted in a sandblasting chamber. Two samples with 35560 mm^2 ($2 \times 200 \times 88.9$) bond area were sand blasted and tested in the same manner as the previous smooth steel sheets. In the first test, the FRP slipped from the steel plate due to loss of bond, and in the second test the specimen developed steel yielding near the grip. However, the bond stress in both tests were similar, with average bond stress of 0.58 MPa. This value coincided with steel yield strength, as before. Figure 3.14 shows the improvement in bond behaviour caused by sand blasting, especially in the post yield region.

To further improve bond between FRP and steel sheet anchors the possibility of creating openings in steel and generating FRP-to-FRP contact within these openings between the external FRP laminates was explored. Two test samples were prepared as illustrated in Figure 3.15. Rectangular openings were cut as shown in the figure, and the steel was sandblasted for improved bond. Two samples were tested. Test results are plotted in Figures 3.16 and 3.17. The first strength drop in both samples was initiated by debonding of FRP sheets from the steel plates near the bottom end of samples, followed by the yielding of steel at sections weakened by the openings. In both cases the FRP to steel bond stress was found to be 0.55 MPa.

3.4.2 Curved High Strength Wire

Because the FRP-steel bond strength was found to be small (ranging between 0.5 to 0.9 MPa), the applicability of other types and forms of steel material as ductile anchors was explored. First, the use of high-strength steel wire, curved in FRP for improved bond, was tried. This type of anchor was obtained by sandwiching the steel wire in CFRP sheets, as shown in Figure 3.18. The behaviour of the wire was established through a coupon test, as presented in Figure 3.19. This technique for improving bond between steel and FRP did not work well. The steel debonded from the FRP laminate as shown in the Figure 3.18 (b). Because the wire has a round diameter, as opposed to a flat surface of the steel sheet used earlier, desired contact between the wire and the FRP could not be achieved, and debonding started during the test in this region, with a tendency of the wire to straighten itself in tension within the FRP laminate. Even though the steel wire itself showed ductile behaviour, as illustrated in Figure 3.19, no further testing was conducted, and the idea was dismissed for potential development of ductile anchors.

3.4.3 Hardwire Tape for Anchorage

The possibility of using hardwire-tape as a ductile anchor was explored by investigating bond strength between this type of steel material and FRP. Hardwire is made from ultra-high strength twisted steel wires. It is used as floor reinforcement, historical building restorations and retrofits, boating industry, strengthening of bridges and blast retrofitting. The material can be used with epoxy and externally applied to interior or exterior surface of a structure. It is available in the market as a single end roving, or unidirectional tape. The hardwire tape is available in different densities in the market. The high density $3 \times 2 - 20 - 12$ tape was used in the present study. This tape had 20 wires per inch. Coupon tests were conducted to understand the behaviour of the material. The properties observed from direct tensile test is shown in Figure 3.20. The material was of high strength but did not show any ductility. The wires ruptured one by one, shortly after peak resistance. Therefore no further testing was conducted, and the material was dismissed as a potential ductile anchor.

3.4.4 Steel Mesh for Anchorage

The use of FRP-steel hybrid system was investigated, consisting of carbon fibres and steel mesh. A diamond shape steel mesh plate was cut into 90 mm×300mm coupons. The idea was to get FRP to FRP contact through the grid openings, in addition to FRP to steel bond, because the FRP to FRP bond is known to be very high. A pre-cured system was designed to improve bond stresses between the steel mesh and the FRP sheet. A wooden box was prepared and filled with fine sand as shown in the Figure 3.21, to manufacture the composite material. A polyethylene sheet was laid on top of the sand so that the FRP sheets would not adhere to the sand. This is shown in Figure 3.21(a). Then an FRP sheet with the same size as the steel mesh was placed on the polyethylene sheet. The steel was applied thereafter and sandwiched in between two FRP sheets. Two samples were prepared in the same sand box. In one test one steel mesh plate was sandwiched between two single layer FRP sheets, and in the other test two steel mesh plates were sandwiched in between three FRP sheets. The composite coupons were put under weight as shown in the Figure 3.21(d) so that the layers would be compressed together. The steel mesh that was used in coupons had a tendency to elongate as shown in the Figure 3.22. It was believed that in a sandwiched position, and with the help of FRP laminates, it would not behave the same. Instead of performing direct bond test, the effectiveness of composite material was tested under direct tension as shown in Figure 3.23. The test results showed that the steel mesh sandwiched between the FRP sheets performed as fully composite material, and the FRP laminates protected the steel from distorting. There was a problem in the gripping area and premature failure occurred in this region. The strength was found to be 460 MPa and 270 MPa for the first and second coupons, respectively. These tests provided evidence that FRP and steel mesh can be used as a composite material for flexural anchorage. The expected ductile behaviour was not observed as the specimens failed prematurely near the grips. Further research is needed to fully develop this type of anchors.

3.5 Selection of a Suitable Anchor for Use in Masonry Walls

Based on the results of the above series of tests, it was decided to use steel sheets as ductile flexural anchors. Though the bond strength of FRP on steel was not very high, the steel sheet was found to be the most compatible anchor that could be secured in concrete and yet bonded on

surface bonded FRP. However, instead of regular steel sheets, high-strength stainless steel sheets were found to be significantly more ductile. An 18 gauge (1.4 mm thick) 4 ft×8ft (1.2 m by 2.4 m) mill finished stainless steel sheet was obtained from a local supplier to manufacture the anchors. The plate thickness was selected to ensure yielding in steel prior to FRP rupture. The coupons were cut into 300mm×90mm sizes with the help of a shear machine. They were sand blasted and tested under direct tension. The stress-strain relationship of the stainless steel sheets are shown in Figures 3.26 and 3.27. The material shows 60% elongation as shown in Figure 3.27. This remarkable elongation capacity made stainless steel an ideal candidate for use as a ductile anchor.

It was decided to investigate the performance of stainless steel anchors prior to their application to a masonry wall retrofit. Small-scale masonry samples were prepared for this purpose. Figure 3.28 illustrates the test samples under preparation. The masonry samples consisted of one and a half masonry blocks, vertically positioned on top of each other, reinforced together vertically by a No: 20M re-bar and placed on top of a concrete foundation as shown in Figure 3.28. The hollow blocks were then filled with fresh concrete as shown in Figure 3.28 (b). Four threaded bars were positioned in the foundation and protruded towards the bottom of the foundation for attachment to the machine during the pull-out test. The foundation was cut immediately below the blocks by a professional concrete cutter, 200 mm deep, to be able to insert a steel sheet anchor, as shown in Figure 3.28 (b). Small-scale stainless steel ductile anchors were prepared. The geometrical details of the anchor are shown in Figure 3.29.

The masonry samples were retrofitted on one side to simulate the large-scale concrete block wall retrofit planned for the next phase of research. Two FRP sheets were used to retrofit the blocks, similar to the planned masonry wall retrofit. One FRP sheet was applied horizontally and the other was applied vertically. The FRP sheets were extended to other two surfaces of masonry blocks for proper bonding. FRP sheets were applied according to the standard procedure described in Chapter 4. The steel plate was designed to yield prior to bond failure. The bond strength between FRP and steel was taken as 0.5 MPa (as obtained from earlier tests) in design calculations.

Five strain gauges were attached on one face of the steel plate as indicated in Figure 3.29. Only one strain gauge was attached to the retrofitted surface of the specimen to understand the stress transfer from the steel plate to the FRP sheet. One strain gauge SG-1 was placed inside the foundation to understand the yield penetration. SG-2, SG-3 and SG-4 were placed at the same level in the critical section.

The steel anchor was placed in two steps. In the first step, the plate was installed in the concrete foundation, and in the second step it was attached to masonry by means of FRP layers. It was first attached to the existing FRP on masonry by epoxy. Because the steel plate was 1.2 mm thick, it was necessary to clamp it against the specimen, as shown in Figure 3.30(b). Once the epoxy was cured, then one FRP layer with fibres oriented horizontally and another layer with fibres oriented vertically were applied onto the specimen. Figure 30 (c) illustrates the placement of FRP sheets.

A test set up was designed to test the masonry block samples in the universal testing machine as shown in Figure 3.30(d). A 500mm×500mm×25mm steel plate was used to secure the sample in the machine. The plate was attached to the machine, as well as the specimen. A lateral support was used to support the specimen as it was retrofitted on one side only, and could rotate under direct tension due to asymmetry. Two cable transducers and strain gauges were connected to a data acquisition system. The specimen was tested under cyclic loading, varying between tension and compression to simulate the anchorage region of a full-scale masonry wall on concrete foundation. The load increased in 10 kN increments in both push and pull directions. The steel anchor performed well both in tension and compression, up to 40 kN force. No debonding of steel anchor from FRP or buckling of steel sheet in compression was observed. During 40 kN pull, some cracking noise was noticed, generating from the FRP in the steel bonding region. The specimen and the anchor continued functioning well up to 60 kN in tension. During 60 kN pull, the specimen started lifting up from the unretrofitted side. During 60 kN in compression the anchor started showing signs of buckling. Another factor that contributed to buckling was the rotating of the specimen in tension towards the anchor. At 80 kN pull, tensile failure of concrete was observed on the unretrofitted side as illustrated in Figure 3.31(b). The test results are shown in Figure 3.32. The figure shows that the steel plate started yielding at 20 kN in tension,

corresponding to 0.1% strain. Strain hardening was observed starting at 0.3% strain. The test indicated that the steel plate yielded prior to anchor bond failure in concrete or on FRP, which indicated its suitability for use in the masonry wall test. In compression the specimen showed buckling, and eventually debonding within the FRP covered region, indicating a need for clamping the anchors to masonry by means of rivets or bolts to reduce the buckling length when used in wall retrofit.



Figure 3.1: Application of FRP fan type anchors

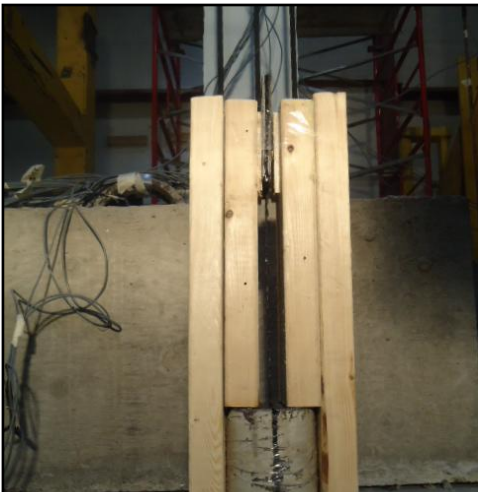


Figure 3.2: Test of FRP laminates inserted in concrete cylinders

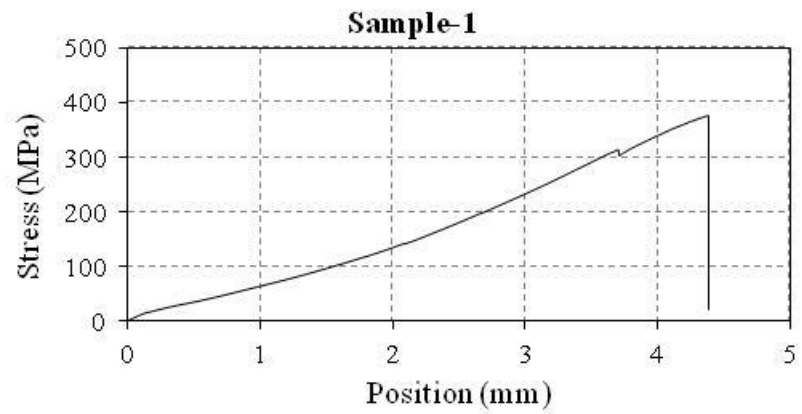


Figure 3.3: Stress on FRP laminate inserted in concrete vs. machine head position

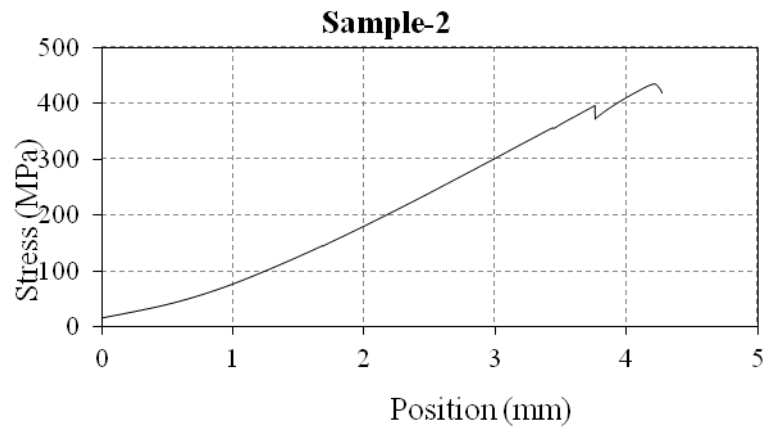


Figure 3.4: Stress on FRP laminate inserted in concrete vs. machine head position

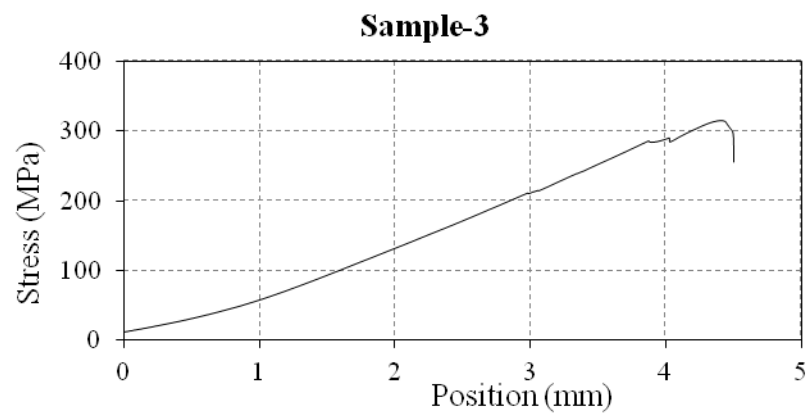


Figure 3.5: Stress on FRP laminate inserted in concrete vs. machine head position

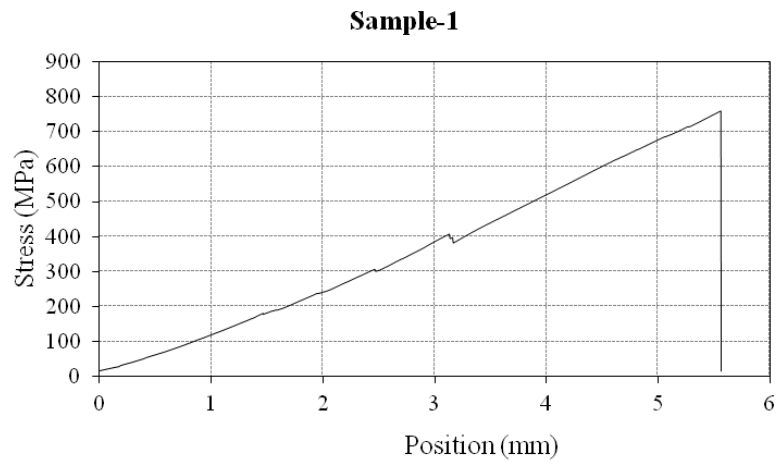


Figure 3.6: Stress on a 25 mm FRP laminate in concrete vs. machine head position

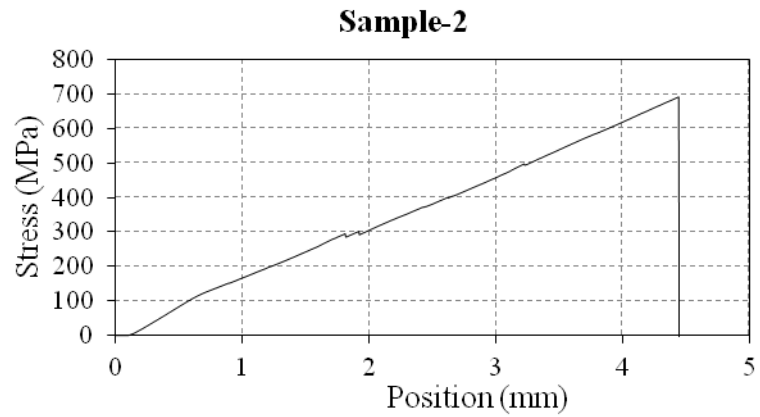


Figure 3.7: Stress on a 25 mm FRP laminate in concrete vs. machine head position

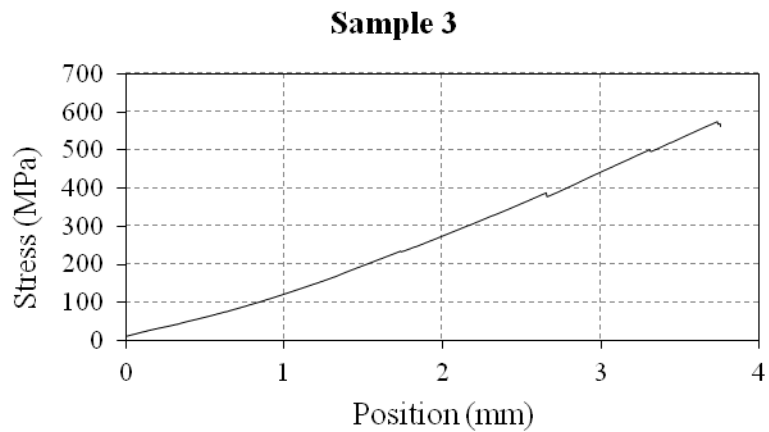


Figure 3.8: Stress on a 25 mm FRP laminate in concrete vs. machine head position



(a) Sample 1



(b) Sample 3

Figure 3.9: Test setup for pull-out bond tests

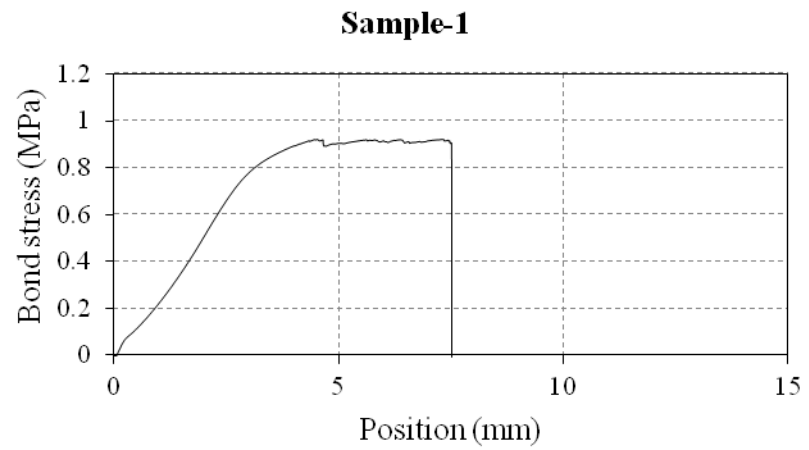


Figure 3.10: FRP-steel bond stress vs. machine head position

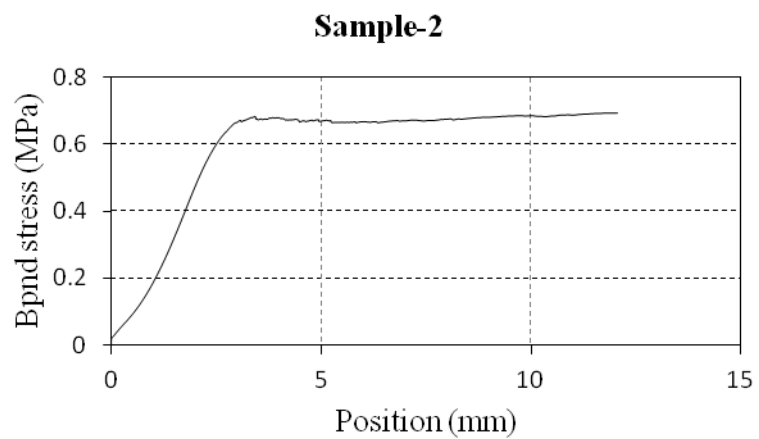


Figure 3.11: FRP-steel bond stress vs. machine head position

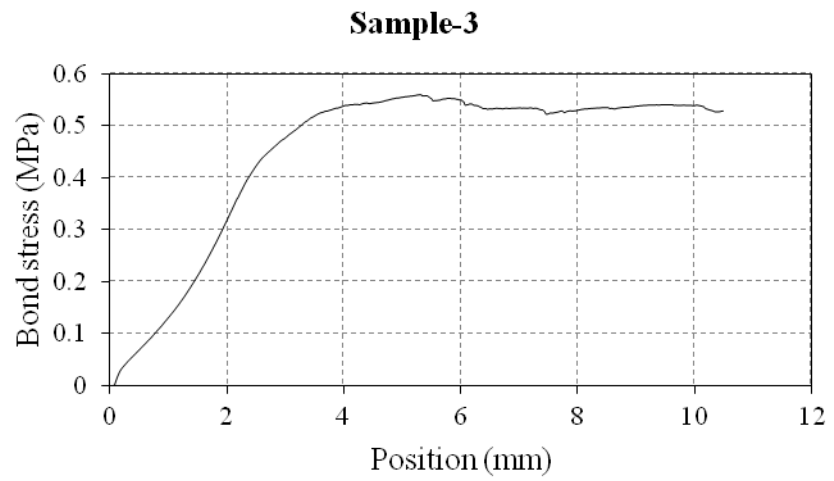


Figure 3.12: FRP-steel bond stress vs. machine head position

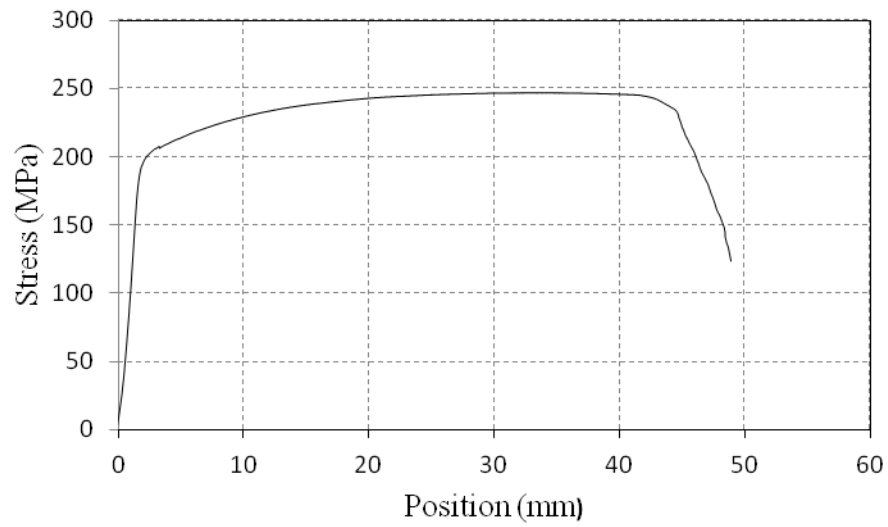


Figure 3.13: Stress on steel sheet vs. machine head position

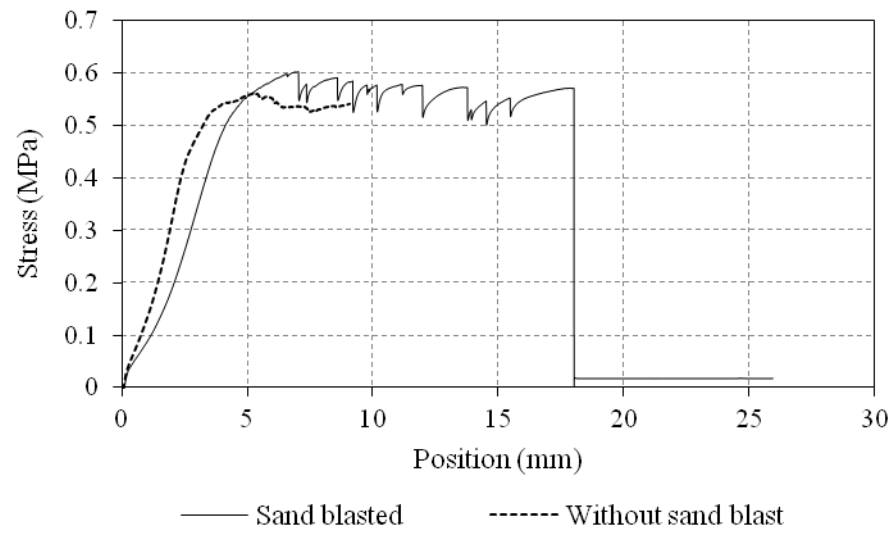
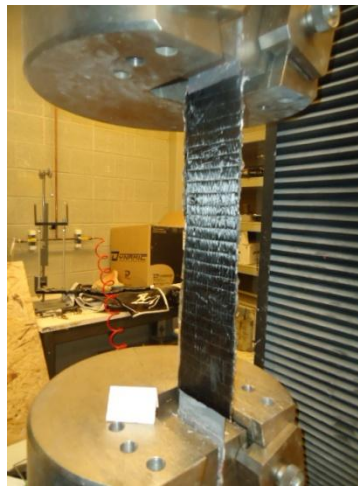


Figure 3.14: Improvements in bond stress behaviour of FRP on sandblasted steel sheets



(a) Preparation of samples



(b) Sample 1



(c) Sample 2

Figure 3.15: Preparation and testing of FRP laminate steel sheet composite anchors with openings in steel for improved bond



Specimen after bond failure

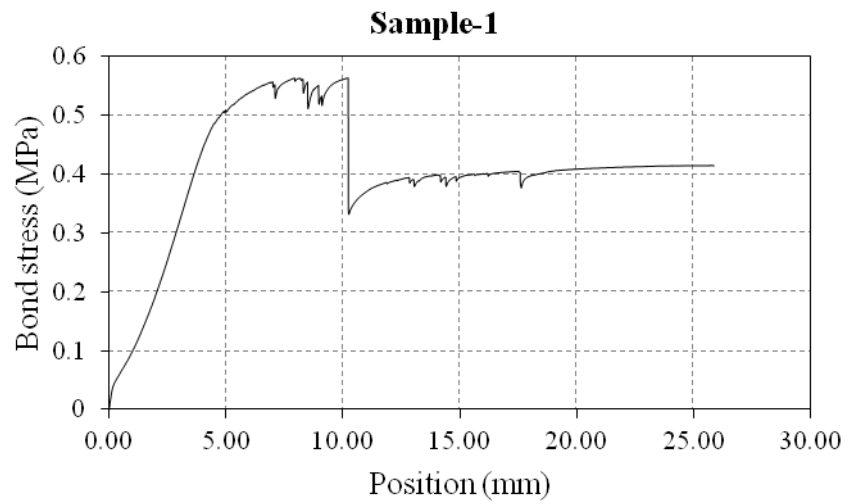


Figure 3.16: Behaviour of bond stress between FRP laminates and steel sheets with openings



Specimen after bond failure

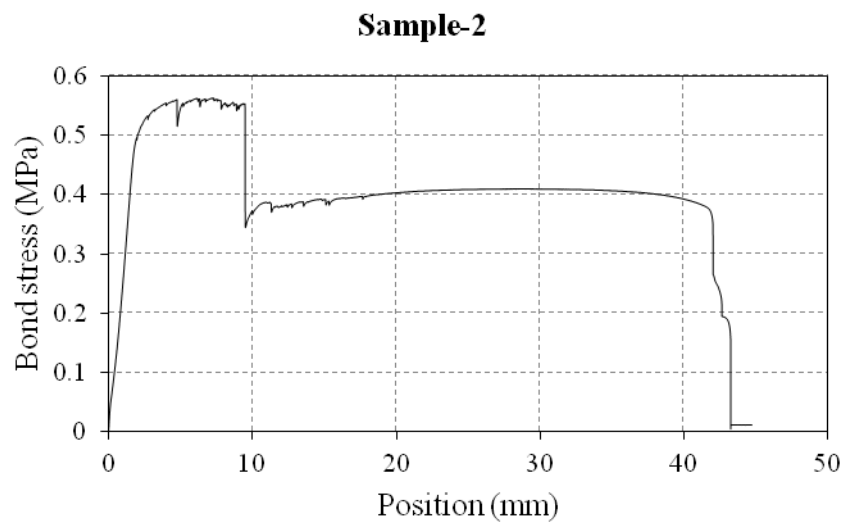


Figure 3.17: Behaviour of bond stress between FRP laminates and steel sheets with openings



(a) Curved wire sandwiched in CFRP sheets



(b) Bond failure surrounding the wire

Figure 3.18: Steel wire anchor embedded in CFRP

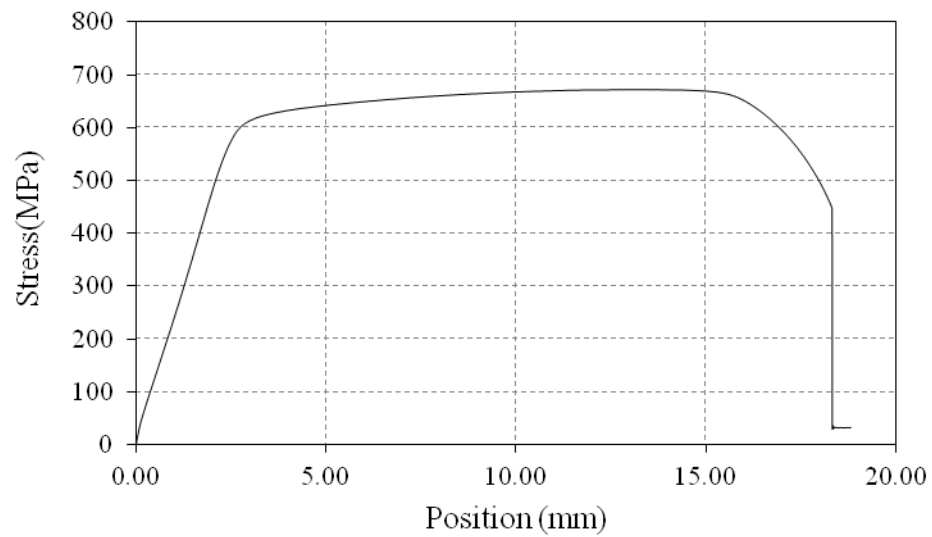


Figure 3.19: Coupon test result for high strength 6 mm wire

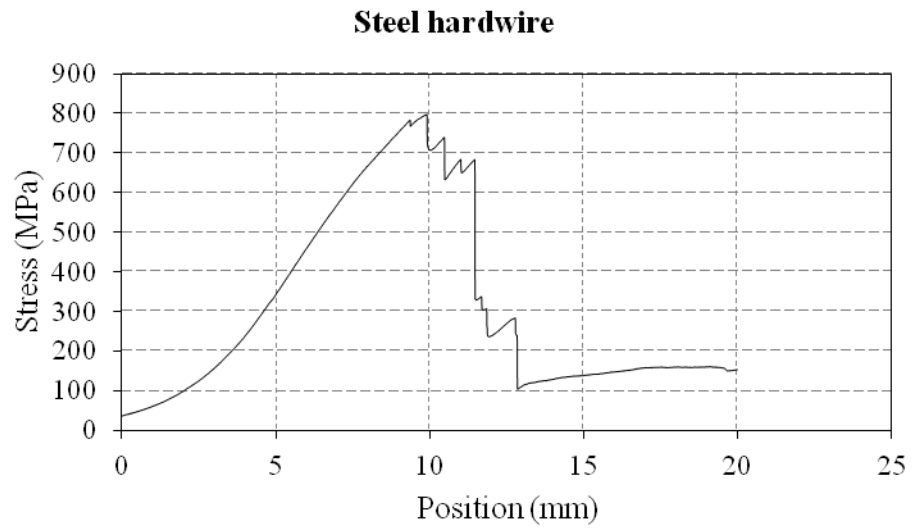


Figure 3.20: Behaviour of hardwire tape



(a) Sand box



(b) FRP and steel mesh layers



(c) Covering sand



(d) Weights to clamp the layers

Figure 3.21: Preparation of FRP-steel mesh hybrid anchors

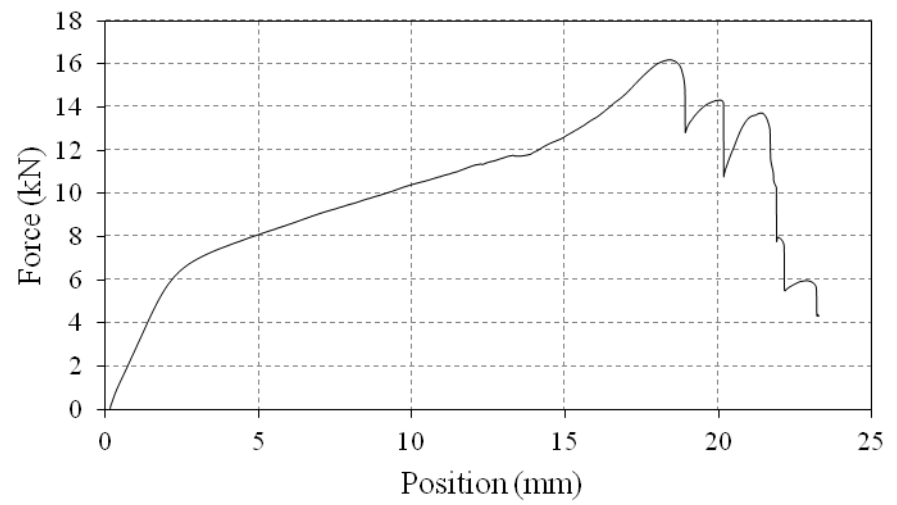


Figure 3.22: coupon test result for steel mesh



(a) Test of a FRP-steel mesh composite anchor



(b) Failure in the gripping area

Figure 3.23: Tests of FRP-steel mesh composite anchors

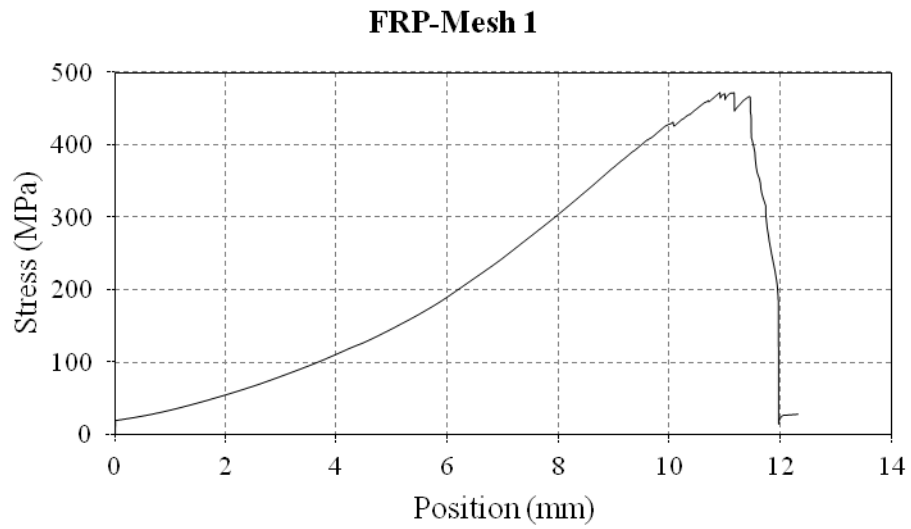


Figure 3.24: Stress on FRP-steel mesh composite anchor vs. machine head position
(One layer of steel mesh)

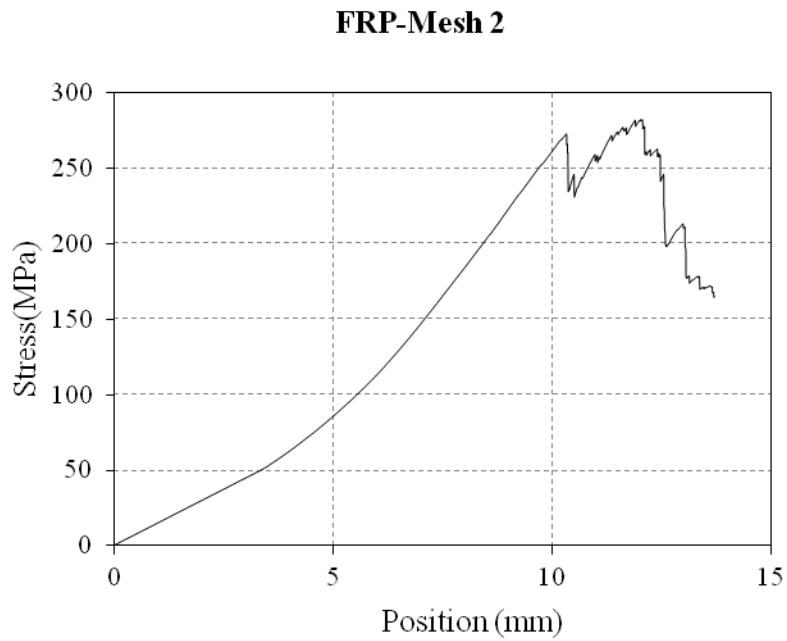


Figure 3.25: Stress on FRP-steel mesh composite anchor vs. machine head position
(Two layers of steel mesh)



Figure 3.26: Elongation of stainless steel sheet

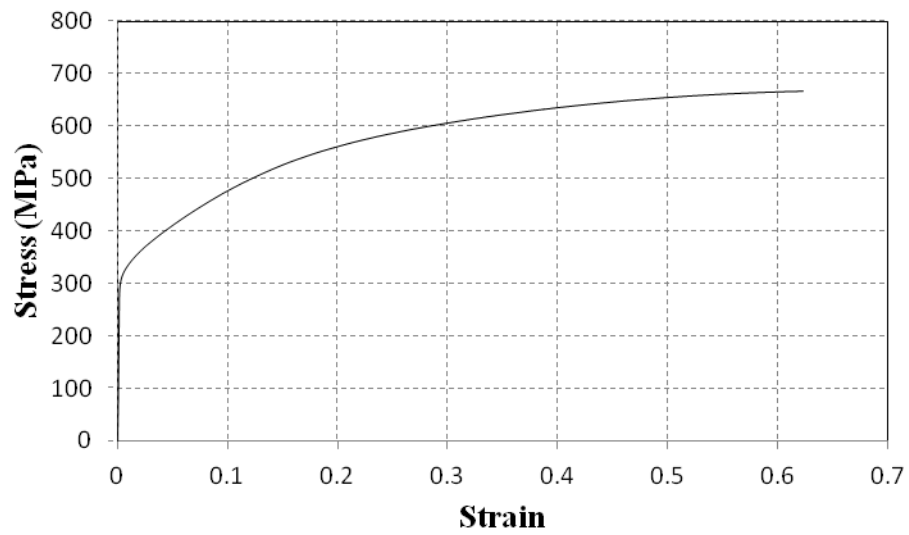


Figure 3.27: Stress-strain relationship of stainless steel sheet



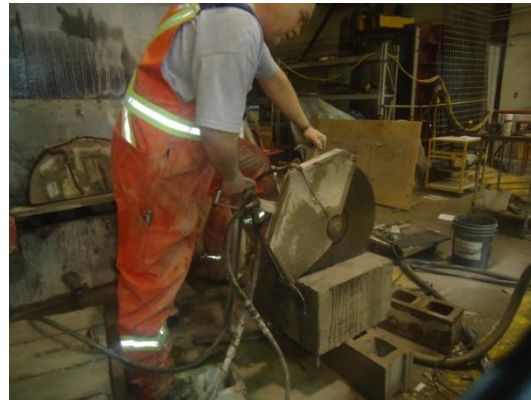
(a) Masonry blocks with #20M rebar



(b) Masonry blocks filled with wet concrete



(c) Preparation of concrete footing



(d) Cutting concrete footing for anchor placement

Figure 3.28: Preparation of masonry samples for bond test

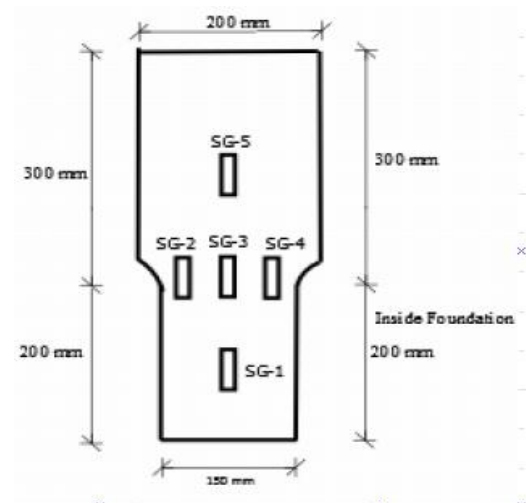
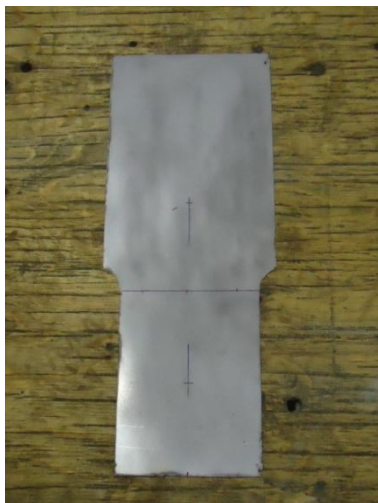


Figure 3.29: Small-scale stainless steel anchor and locations of strain gauges



(a) Placement of anchor



(b) Clamping for adhesion

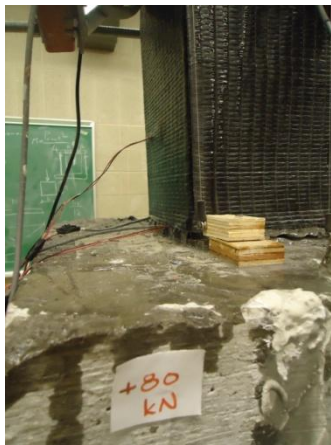


(c) Placement of FRP on anchor



(d) Block sample ready for testing

Figure 3.30: Placement of stainless steel anchors on a masonry sample



(a) Block sample under 80 kN of pull force



(b) Concrete splitting crack of block

Figure 3.31: Performance of a masonry block sample with a stainless steel anchor

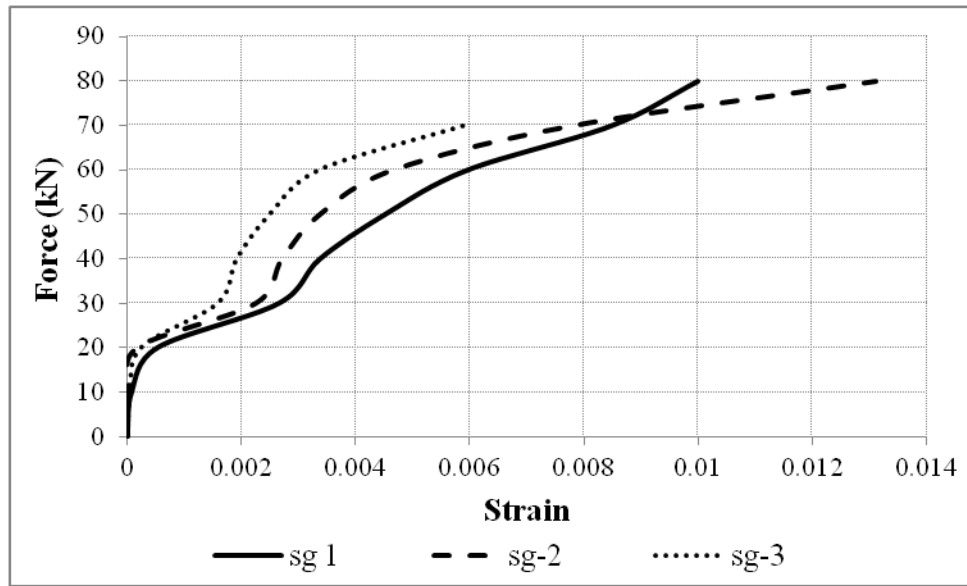


Figure 3.32: Performance of stainless steel anchor

Chapter 4

Experimental Program

4.1 Introduction

Research needs for improved seismic retrofit techniques for low-rise load bearing masonry walls have been well established as indicated in Chapter 1. These walls can be deficient in flexure and shear, as well as in ductility and related energy dissipation capacity. A combined experimental and analytical research program is underway at the University of Ottawa for developing economically viable and structurally sound seismic retrofit methodologies for load bearing masonry walls. This Chapter provides the highlights of experimental research that involves the development of a surface-bonded FRP retrofit methodology for masonry walls. The experimental program consists of one partially reinforced and another unreinforced masonry load bearing walls, both deficient against seismic forces.

Previous research has shown that diagonal shear cracks that form within masonry wall panels under in plane shear force reversals can be controlled by surface bonded FRP sheets (Elgawdy et al. 2007). Diagonal orientation of FRP sheets has been suggested by Valluzi et al. (2002) for shear strength enhancement. Use of surface-bonded carbon FRP on one side of walls is investigated in the current research project. The application of FRP on one side, with full surface coverage, was selected for both shear and flexural strength enhancement as accessibility from both sides of the wall may not be possible in practice. Full surface retrofitting has an additional advantage of controlling out of plane damage during earthquakes. FRP sheets with fibres in both horizontal and vertical directions control shear cracking, while the longitudinal fibres may act as flexural reinforcement when well anchored.

The enhancement of flexural capacity requires the anchorage of FRP sheets placed on the wall panel to the foundation or beams/slabs that support the wall. Two different types of anchors were used in the current research project for ductility and/or strength enhancements. The FRP fan type anchors were used to connect the vertical FRP sheets on the partially reinforced masonry wall to the foundation. In this case the FRP reinforcement was used to increase the elastic wall capacity

for elastic response. A ductile anchorage system was introduced to the unreinforced masonry wall, as a result of the experimental research outlined in Chapter 3, for connecting FRP sheets to the foundation. In this case stainless steel sheet anchors were epoxy bonded to the wall FRP at one end and secured into the foundation concrete at the other end. The details of the anchor selection process are presented in Chapter 3.

This chapter provides the details of the wall test program that forms the principle component of the current research project. It involves retrofitting and testing two large-scale concrete masonry walls, one partially reinforced, the other unreinforced, under simulated seismic loading. The retrofit technique investigated consists of surface bonded carbon FRP sheets with either FRP or stainless steel anchors applied only on one side of the walls.

4.2 Test Specimens

One partially reinforced and one unreinforced large-scale load bearing masonry walls were designed, built and tested in the Structures Laboratory of the University of Ottawa. The walls were retrofitted with carbon FRP sheets and subjected to constant axial load and incrementally increasing lateral load reversals simulating seismic effects. The walls were companion to those tested in an earlier phase without the retrofits (Taghdi et al. 1998).

4.2.1 Partially Reinforced Masonry Wall (PRM)

The partially reinforced masonry wall (PRM) refers to those walls which do not have enough vertical and horizontal reinforcement required by the current seismic design guidelines (CSA S304.1-4). A partially reinforced wall was built by a certified local mason in the Structures Laboratory on top of a 100mm×100 mm hollow steel section. It was subsequently embedded in an I-shaped reinforced concrete foundation that was built to secure the wall to the laboratory strong floor.

4.2.1.1 Configuration of PRM Wall

Figure 4.1 illustrates the geometric and reinforcement details of the PRM test specimen. A certified mason was hired for the construction of the wall so that it represented the usual construction practise. The wall panel was built to have a square geometry with a 2.0 m length and height. It was built on a HSS steel section. This specimen was originally intended to be tested under blast-induced lateral shock waves using the University of Ottawa Blast Simulator (Shock Tube). However, it was later decided to be included as part of the current research program. Therefore, it was built into a reinforced concrete foundation wall subsequently for seismic testing. One course of the wall was inserted into the concrete foundation in the process of integrating the wall with the foundation as shown in Figure 4.2. This resulted in the final wall height to be 1800 mm. Two 15M longitudinal steel reinforcement (16mm diameter) were used to make the wall partially reinforced. These two reinforcing bars were anchored into the foundation and to the top loading beam. The amount of reinforcements used in the wall was less than the minimum seismic reinforcement required by CSA S304.1-04 in clause 10.15.2. 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 capacity at the ends of reinforced walls. Truss type joint reinforcement was placed at every other course of masonry block in the mortar joint as part of common construction practise for maintaining vertical alignment of the wall. The wall was cured for at least 28 days and was integrated in the foundation when the foundation was ready.

4.2.1.2 Foundation

An I shaped concrete foundation was built to provide a rigid base for testing. The foundation was heavily reinforced and secured on the laboratory strong floor by high strength bolts. Figure 4.3 shows the reinforcement details of the foundation. The wall was constructed on a hollow steel section before the construction of the foundation for scheduling purposes. The two vertical No: 15M wall reinforcement were welded to the steel section. The foundation was designed in such a way so that the wall can be accommodated within the foundation with the hollow steel section. To ensure the integrity of the wall with the foundation it was decided to lower one course of masonry block with the steel section inside, into the foundation as shown in Figure 4.2. Four

plastic pipes were placed in the foundation to accommodate bolts for connecting the specimen to the laboratory strong floor. Four steel hooks were placed in the foundation concrete for lifting the specimen. Three HSS sections were placed in the foundation, perpendicular to the wall plane, protruding out of the foundation for subsequent application of axial load through prestressing strands. Ready mix concrete was used to cast the foundation. It was cured by spraying water and by placing wet burlap for seven days.

4.2.1.3 Load Transfer Beam

A rigid concrete top beam was built for transferring lateral forces from a horizontally positioned actuator. This top beam acted as a diaphragm for transferring the lateral load to the wall. Before the construction of the top beam, the existing No: 15M bars were extended from the wall as shown in Figures 4.1 and 4.4 by using mechanical couplers. This extension was done to ensure continuity between the wall and the top beam. The beam was heavily reinforced as illustrated in Figure 4.1 and 4.5. Wooden formwork was built and placed on the wall, while being supported from the foundation as shown in Figure 4.6. During the placement of concrete in the beam formwork, expandable foams were injected into the hollow blocks so that no concrete is poured into masonry cells (See Figure 4.4). Dowel bars were inserted in the top beam, connecting the wall to the beam, in addition to the extension of the wall reinforcement into the beam, to ensure that horizontal sliding of the top beam during loading is completely eliminated. In addition, four threaded bars were placed horizontally in the beam at each end, for connecting the actuator. The beam was cast using ready mix concrete supplied by a local company. It was cured for seven days before the formwork was opened. The partially reinforced test specimen after construction is shown in Figure 4.7.

4.2.2 Unreinforced Masonry Wall (URM)

An unreinforced load bearing concrete block masonry (URM) wall was constructed for retrofitting and testing. The wall was built after the testing of the first (PRM) wall. Hence, the same reinforced concrete foundation was re-used after repairing the top, as shown in Figure 4.8. The following subsections provide the details of the URM wall.

4.2.2.1 Configuration of URM Wall

The geometrical configuration of URM wall was kept same as the PRM wall. The height and the length was 2.0 m. This time a precast hollow top beam was constructed to transfer the lateral load to the specimen. One and a half course of the wall was inserted into the hollow section of the top beam as shown in Figure 4.9, reducing the clear wall height to 1.7 m, while maintaining the shear span (wall height to the point of application of lateral force) at 2.0 m.

The only reinforcement used was the standard horizontal truss type reinforcement placed in every other mortar joint, as per standard construction practice. The first mortar joints were fully mortared and then the first course was laid. For the other courses only the face shells were mortared. Figure 4.10 shows the URM wall during construction. Usually, grouting is not required for unreinforced masonry walls. However, because the wall was built for retrofitting with FRP reinforcement, which would provide tensile capacity for the wall, increasing compression demand at the ends (wall toe), two cells at each end were grouted after the construction of the wall as part of the retrofit strategy. Ordinary sand and cement (1:3) were used as grout and poured from the top of the wall. In practice where the cells may not be accessible from the top, this can be done by opening some units, and injecting grout by pressure.

4.2.2.2 Foundation

The same foundation that was used for the PRM wall was also used for the URM wall. Before the construction of the wall, the foundation was repaired by removing the masonry blocks that had been inserted during the PRM wall test, and replaced by fresh concrete, as illustrated in Figure 4.8. Some vertical reinforcing bars were placed along the length of the groove to prevent possible cone failure from pull out of anchors that would be inserted as part of retrofitting. The fresh concrete was mixed in the laboratory and poured into the foundation.

4.2.2.3 Load Transfer Top Beam

A precast hollow beam was constructed for the URM wall and placed on top of the wall after the wall was retrofitted with FRP. This hollow precast beam was designed and constructed for transfer of lateral load, with the intention of re-using in future tests.

The geometric details of the beam are shown in Figure 4.11. The groove of the beam accommodated the wall. The top beam was over designed to prevent premature failure prior to the failure of the wall. Three plastic pipes were placed inside the beam for pouring concrete to fill up any gap that may exist between the beam and the wall. The beam was casted by using ready mix concrete. It had four rods on each side for connecting the actuator. The beam also had two hooks on the top to facilitate lifting and handling. The beam was lifted by crane and placed on top of the URM wall which as shown in Figure 4.12. There was a 20mm gap between the hollow part of the beam and the wall on either side. After placing the beam these gaps were filled by inserting wooden pieces from the bottom and pouring grout from the top through the plastic pipes that had been cast in concrete. Figure 4.13 shows the URM wall after the placement of the top beam.

4.3 Retrofitting

Both PRM and URM walls were retrofitted with surface-bonded carbon FRP sheets and anchors for improved bond. The following sections provide the details of FRP and anchor applications for each wall.

4.3.1 Application of Surface-bonded FRP

Carbon FRP sheets were placed on one side of each wall for shear and flexural strength enhancement. The application of FRP was specifically selected to be on one side only because of potential difficulties in practice associated with retrofitting both sides. One layer of carbon FRP sheet was placed in each direction with fibres aligned in horizontal and vertical directions. This pattern of external reinforcement is effective against controlling diagonal tension cracks caused by shear force reversals. The vertical fibres also act as flexural reinforcement when anchored in the supporting element (in this case the concrete foundation). This pattern of fibres is also effective in preventing out-of-plane failure of block masonry during seismic excitations. The FRP sheets were applied following the wet layup procedure, covering the entire wall surface on one side of the wall. The wall surface was first prepared prior to the application of CFRP sheets. 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, as shown in Figure 4.14(a).
- 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.
- v. Sanding the surface by sand paper after two full days of curing as shown in Figure 4.14(b).

Once the wall surface was ready for the application of FRP sheets, the sheets were cut into 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 as shown in Figure 4.14(c).
- ii. Soaking of the first FRP layer with epoxy as shown in Figure 4.14(d).
- iii. Application of the horizontal layer of FRP, saturated in epoxy, with fibres parallel to the bed joint as shown in Figure 4.14(e).
- iv. Removal of extra epoxy and air pockets by means of a ribbed steel roller.
- v. Application of the vertical layer of FRP, saturated in epoxy, with fibres perpendicular to the bed as shown in Figure 4.14(f).

4.3.2 Installation of Anchors

Lateral load capacity of masonry walls in shear can be increased by FRP sheets epoxy bonded to the wall surface. The capacity of the wall in flexure can only be increased if the FRP sheets are connected to the foundation with proper anchorage. Two special types of anchors, one with FRP and the other with stainless steel sheet, have been introduced in the present research work as flexural anchors. FRP anchors were designed to increase elastic force resistance whereas steel sheet anchors were designed to enhance both the strength and ductility. Both types of anchors functioned as vertical flexural reinforcement and transferred tensile forces to the surface-bonded FRP sheets on the wall panel. In the PRM, additional FRP anchors were installed in the wall surface to prevent debonding of FRP sheets from the wall.

4.3.2.1 Design and Installation of FRP Anchors

Carbon FRP anchors, developed at the University of Ottawa (Ozbakkaloglu and Saatcioglu 2009), were used to prevent debonding of FRP sheets from the PRM wall panel, as well as anchoring the FRP sheets to the concrete foundation. The anchors were manufactured in-house at the University of Ottawa's Structures Laboratory by cutting 200mm × 250mm pieces from a roll of FRP sheet. The cut FRP pieces were rolled by hand to form anchors as illustrated in Figure 4.15. A 100 mm portion of the twisted FRP was tightened with a string for easy insertion into the element. The 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 (wall or foundation) to which the FRP anchors were to be inserted for anchorage. After making the holes, two-component epoxy, thickened by silica, was injected into the holes with a 60 ml injection syringe. Then, the rolled parts of anchors were saturated with epoxy and inserted into the holes. Finally, the fan portions were secured to the wall surface with epoxy.

The FRP anchors were used at two different strategic locations in the PRM wall. The first location was wall bottom edge, near its ends, where the anchors would be most effective in forming tension reinforcement. First a long sheet of CFRP was cut and applied along the bottom edge following the same wet layup method employed for the wall FRP with fibres aligned in the vertical direction. 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 along the foundation and top loading beam interface were rounded with cement mortar as illustrated in Figure 4.16. A hand held hammer drill was used to open holes near the wall end to secure the FRP anchors. The holes were 75 mm in depth and diameters were little larger than the anchor diameter. Figure 4.17 depicts the anchors in place. These anchors were designed to take the required tensile forces for increased moment capacity established by the retrofit design.

The number of anchors was determined by establishing the tension force required for the additional moment capacity. The wall was analyzed using a truss analysis computer program described in Chapter 6, which computes forces in tension and compression elements in the wall

after establishing a strut and tie model for the wall. The required tension force in the anchors at one end was calculated as 100 kN for PRM wall. This force demand was used, along with the design procedure recommended by Ozbakkaloglu and Saatcioglu (2009) to establish the number and the size of FRP anchors. Accordingly, if the anchor embedment length is greater than 50-75mm then the most common failure mode is combined cone and bond failure.

$$F_u = F_{cone} + F_{bond} \quad (4.1)$$

Concrete cone capacity can be calculated as per ACI 349-85 expression. This expression correlated well with anchor test results (Ozbakkaloglu and Saatcioglu 2009), and is given below:

$$(F_{cone})_{ModACI} = \pi(f_{ct}')_{exp} L_c^2 \left(1 + \frac{d_h}{L_c}\right) \quad (4.2)$$

Where;

f_{ct}' = 28 day tensile strength of concrete = 3MPa (taken as the 10 % of the compressive strength of foundation concrete).

L_c = depth of concrete pullout cone. It was reported that the concrete pull out cone decreased with increasing embedment length (Ozbakkaloglu and Saatcioglu 2009). The average measured depth found from the experiment was 5 mm for 100 mm long anchors.

d_h = diameter of the FRP rolled anchor. It was found that the anchor diameter becomes 16 mm for 220 mm wide FRP sheets. As 200 mm wide FRP sheets were used for making the FRP anchors, the diameter was taken as 16 mm.

From Eq. (4.2)

$$(F_{cone})_{ModACI} = 3.14 \times 3 \times 5^2 \times \left(1 + \frac{16}{5}\right) = 984N = 0.98 kN$$

$$F_{bond} = \pi \times d \times L_b \times \tau \quad (4.3)$$

Where;

L_b = bond length (embedment length less the cone depth) = 100 - 5 = 95 mm

τ = average bond strength between FRP anchor and concrete = 10 MPa

$$F_{bond} = 3.14 \times 159 \times 95 \times 10 = 47430 kN$$

$$F_u = F_{cone} + F_{bond}$$

$$= 47.4 + 0.98 = 48.4kN$$

Anchor design force in each side of the wall =100 kN (From the truss program)

$$\text{Number of anchor required} = \frac{100}{48.4} = 2.07$$

However, anticipating difficulties associated with providing the full 100 mm embedment length and accounting for the fact that a masonry course was embedded in the foundation in the critical region, a total of 4 flexural anchors were provided for conservatism. Figure 4.18 shows further details of the fan type FRP anchor used to enhance flexural resistance.

The second location of FRP anchors was the wall surface to ensure sufficient bond between the surface-bonded FRP sheets and the masonry substrate. These anchors would provide physical bond, over and above the chemical bond provided by the epoxy. These anchors were placed along the 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 4.19 illustrates the locations of fan type FRP anchors.

4.3.2.2 Design and Installation of Ductile Anchor

For the URM wall, which did not have any reinforcement, ductile anchors made with stainless steel, developed in Chapter 3, were preferred over other anchors investigated in Chapter 3. The target lateral force capacity of the retrofitted wall was set as three times the capacity of the unretrofitted companion specimen tested by Taghdi et al. (2000). The ideal anchors would have sufficient yield strength and ductility for energy dissipation, while maintaining bond in concrete and on wall FRP. Therefore, the most ductile material tested in Chapter 3, stainless steel sheet with 60% elongation capacity, was selected as the anchor material. The anchor shape was selected such that the critical steel-to-FRP region would have increased surface area to maintain bond, and the flexure critical section at wall-foundation interface would have a reduced cross-sectional area to promote yielding prior to debonding. Figure 4.20 shows the geometric details of ductile steel sheet anchors. Figure 4.21 shows the positions of anchors on the wall. The narrower

section was intended to control tensile capacity, and was expected to yield prior to debonding. To ensure proper bond between the steel plate and the wall FRP a layer of FRP sheet, with fibres aligned vertically, was applied on top of each anchor. Another layer of horizontal FRP sheet was applied to prevent out of plane buckling. Applying statics and wall geometry, the anchors were designed as follows:

$$\sigma_{steel} = 300 \text{ MPa (Yield stress from coupon tests)}$$

$$\sigma_{usteel} = 620 \text{ MPa (Ultimate strength from coupon tests)}$$

Retrofitted lateral wall capacity, $F = 150 \text{ kN}$ (Demand)

From Figure 4.21 taking moment at the centerline of the right anchor

$$150 \times 2.05 = 34 \times 1.075 + 34 \times 0.7 + 34 \times 0.325 + P \times 1.4$$

$P = 168 \text{ kN}$, where P = Anchor design force

Steel sheet thickness = 1.2 mm

$$\sigma = \frac{F}{A} = \frac{F}{b \times 1.2} = \frac{168000}{b \times 1.2} = 550 \text{ MPa (Taken as 90% of the ultimate strength)}$$

$b = 254 \text{ mm}$, say $b = 300 \text{ mm}$ (b = width of the anchor at foundation edge)

The geometric details of the anchor is shown in Figure 4.20

Check for debonding:

Bond strength between steel and FRP was found 0.5 MPa. (From the bond tests described in Chapter 3)

$$\sigma_{debonding} = \frac{F}{A}$$

$$400 \times h = \frac{168000}{1.5 \times 0.5} \quad h = \text{Height of the anchor}$$

$h = 560 \text{ mm}$, say, $h = 600 \text{ mm}$

use 400 mm× 600 mm steel sheet

Check:

$$\sigma_{debonding} = \frac{F}{A} = \frac{168000}{1.5 \times 600 \times 400} = 0.46 \text{ MPa} < 0.5 \text{ MPa, ok}$$

Therefore, 400 mm × 650 mm provides sufficient surface area for bond strength.

Stainless steel anchors were cut from a #8 gauge 4 ft× 8 ft steel sheet. The edges were smoothed by a grinder. Two anchors were then sand blasted in a sand blasting chamber. A sample anchor is shown in Figure 4.22 with five strain gauge positions for monitoring strains during the wall test. Gauge locations were selected to assess the behaviour of anchors, yield penetration into the foundation, if applicable, and stress transformation from the steel plate to the FRP sheet. Additional strain gauges were placed on the FRP and the prestressing strands used to apply the gravity loading. Strain gauges on anchors are numbered from 1-10. Strain gauges placed at wall-footing interface location were expected to monitor the level of inelasticity in anchors. The three gauges along the longitudinal axis were intended to capture the strain profile, while the gauge at the foundation end of the anchor would detect yield penetration into the foundation, if any.

The anchors were placed in the foundation by cutting a slot in concrete and securing the anchors by means of epoxy. Two locations near the end of the foundation were cut by a professional concrete cutting company as illustrated in Figure 4.24. The width of the cut was 6 mm as this was the thinnest blade of the supplier. The cut location was kept as close as possible to the wall so that the steel anchor can be inserted vertically without much inclination. The slots were filled with two component epoxy as shown in Figures 4.25(a) and 4.25(b). Then the steel sheets were soaked with epoxy and inserted into the concrete slots as shown in Figure 4.25. Because the steel sheets had some rigidity due to their thickness of 1.2 mm, they had to be pushed towards the wall FRP and clamped for proper adhesion as shown in the Figure 4.26(a). Clamping was done with C clamps and pieces of 2 in by 4 in wood. The sheet anchors were cured for two days and clamps were removed as shown in Figure 4.26(b). Two FRP sheets of 750 mm × 600 mm were applied over the steel anchors in wet layup procedure, the first with fibres oriented in the vertical direction, and the other layer positioned with fibres in the horizontal direction. This is shown in

Figures 4.26 (c) and 4.26 (d). These two FRP sheets gave additional bond by providing another surface to the steel, while also increasing the tensile capacity of the wall.

During the development steel sheet anchors as described in Chapter 3, it was observed that the stainless steel sheets would develop buckling in compression during later stages of cyclic loading. Therefore, it was decided to use six drill-through bolts in each sheet anchor for additional support to reduce the buckling length. This is shown in Figure 4.27. The critical buckling load of the steel sheet anchor was calculated, and the spacing of the bolts were determined as follows:

Unsupported length of the steel plate, from Euler's buckling formula: $F = \frac{\pi^2 \times EI}{(KL)^2}$

Where;

E = materials modulus of elasticity

I = moment of inertia

K = factor related with the end support conditions

L = unsupported length

Bolts can be used in two columns

$$\frac{168 \times 1000}{2} = \frac{\pi^2 \times 2 \times 10^5 \times 1.2^3 \times 400}{12 \times L^2}$$

$$L = 37 \text{ mm}$$

Six bolts were used with a vertical spacing of less than 37 mm.

4.4 Material Properties

Materials used in wall tests consisted of concrete block masonry, mortar and reinforcing steel, with concrete used in the foundation and the top loading beam. In addition carbon FRP and stainless steel were used as retrofit materials.

Standard 8 in hollow concrete block masonry units (CMU) were used for the construction of both walls. The gross and net areas of each block were; 74100mm^2 and 39719mm^2 , respectively. The blocks were 53.6 % solid. The approximate weight was 17 kg. The average compressive strength was found from standard compressive strength tests of units to be 12.74 MPa. Table 4.3 shows the results of three tests conducted in the Structures Laboratory. The mortar used was of Type N with compressive strength of 9 MPa.

Deformed reinforcing bars were used in the foundation, top beam and as partial reinforcement in the PRM wall. The reinforcement used in the PRM wall consisted of two No.15M bars with yield strength of 400 MPa and a corresponding yield strain of 0.2%.

Concrete for the foundation and the two top beams were received from a local ready mix concrete company. Standard cylinder tests were conducted in the laboratory and the average 28 day compressive strength for the foundation, PRM top beam and URM top beam were found to be 28.6 MPa, 28.1 MPa and 41.6 MPa, respectively. The cylinder test results are given in Table 4.4 to Table 4.6.

The FRP sheets used for wall retrofitting had carbon fibers with tensile strength of 3790 MPa and elastic modulus of 230 GPa. The tensile rupturing strain was 1.7%. The density of the fibre material was reported by the manufacturer to be 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%. The laminate strength was found to be 780 MPa from the coupon test that was conducted. Figure 4.28 shows the stress strain relationship of the FRP laminates. The same FRP was used to manufacture the fan-type anchors used in the PRM walls.

The ductile steel anchors used in the URM wall consisted of stainless steel sheets. Steel coupons taken from the material, with dimensions of $90\text{mm} \times 300\text{ mm}$, showed yield strength of 300 MPa at 0.2% strain and ultimate strength of 650 MPa at 60% strain. The stress strain of the material is presented in Chapter 3, Figure 3.27.

4.5 Axial Load

Both PRM and URM walls were subjected to constant axial load as load bearing walls during testing. The level of axial load was 100 kN for each wall. This level of axial compression represents the tributary floor area that typically acts on a single story load bearing Masonry wall (Taghdi 1998). It was applied by means of 8 mm-diameter prestressing strands on the PRM wall and 10 mm-diameter prestressing strands on the URM wall. The applied level of force in each strand was monitored through strain gauges placed on the strands. A coupon of a pre-stressing strand was calibrated by placing a strain gauge on the coupon and applying tension by a universal testing machine. Each strand was targeted to be stressed up to 17 kN. The strands were tensioned between two sets of hollow steel sections which were placed on top of the load transfer beam, as well as cast in the foundation. The cross sectional dimensions of the hollow sections used for the PRM wall were 50mm x 50mm x 6mm. The hollow sections placed on the URM top beam were of 100mm x 100mm x 10mm. The purpose of using three hollow sections and three sets of prestressing strands was to distribute the gravity load along the beam length. Two hand held hydraulic jacks were used to apply the prestressing force. The strands were locked in using standard cylindrical anchors and wedges. Strain gauges were attached at middle height of each tendon and connected to strain indicators. The prestressing was applied two strands at a time at each location, one strand on each side of the wall, while balancing stresses in each strand. Tables 4.1 and 4.2 present the prestressing loads applied.

4.6 Test Setup and Instrumentation

The walls were highly instrumented before testing. The instrumentation was done to measure displacement, rotation and strain measurements. Three types of sensors were used; i) electric resistance strain gauges (SG), ii) linear variable displacement transducers (LVDT) and iii) cable transducers (CT).

Strain gauges were primarily used to understand the flow of forces in surface-bonded FRP. For the PRM wall, the positions of strain gauges are shown in Figure 4.29. They were attached either at wall ends or along the diagonals. The strain gauges used on the URM wall were primarily intended to measure the strain conditions on and around the steel anchors. A total of ten strain gauges were placed, as shown in Figure 4.23;

Linear variable displacement transducers, with a stroke capacity of ± 25 mm, were placed at different locations of PRM and URM walls to measure the uplift of the wall from the foundation, sliding of the wall, and the uplift of the foundation from the strong floor. LVDT positions for the walls are shown in Figures 4.30 and 4.31 for the PRM and URM walls, respectively.

Cable transducers (CT) were the main instruments for displacement measurements. The stroke capacity of CTs used ranged from 300mm to 1000mm. CT locations for the PRM and URM walls are shown in Figures 4.30 and 4.31, respectively.

All the instruments, as well as the actuator used for the application of lateral loads were connected to a data acquisition system for continuous data recording.

Both wall specimens were retrofitted on one side only. This created asymmetry in resistance. Therefore, it was important to provide lateral restraint to be able to apply in-plane forces. A special type of lateral support was designed and fabricated for this purpose, consisting of steel sections. The lateral support was attached to a steel frame, which was secured on the laboratory strong floor. The support had a steel attachment with guide rails and rollers to ensure in-plane movement under lateral loading. Figure 4.32 shows the test set-up used for testing the PRM wall. Same test set-up was used for the URM wall. Figures 4.33 and 4.34 show the actual testing of the PRM and URM walls.

4.7 Lateral Load Program

A servo controlled 1000 kN capacity MTS actuator was used to apply in plane lateral cyclic loading. The actuator was used in the displacement control mode to impose incrementally increasing, slowly applied lateral deformation reversals. It was supported by a frame that consisted of a 457mm deep channel section, secured to the laboratory strong floor. An A frame was attached to the top of the channel section. The other end of the actuator was connected to the wall with a thick (50.8 mm) steel loading plate. The steel plate section was slightly larger than the cross sectional area of the top beam. Another small plate was connected to the large plate. This small plate had four holes and the bolts from the top loading beam went through these holes and they were tightened with nuts and wrenches. The large plate had four more holes. Another steel plate of the same thickness (51 mm) was attached at the other end of the top beam. Four

dywidag bars were connected to the end plates. The function of these bars was to transfer push and pull from the actuator to the top load transfer beam. The dywidag bars were tightly connected to the plates. For additional security, the holes of the plates were filled with thickened epoxy. The test setup allowed the test specimen to be tested as a cantilever wall.

Each specimen was subjected to incrementally increasing lateral displacement reversal under constant axial load. Lateral load was applied completely in the displacement control mode throughout the test. The specimens were cycled three times at each displacement level. The displacement was computed as a story drift, defined as the top displacement (at the point of application of lateral load) divided by the height of the wall (measured to the top of the foundation). The specimens were first cycled three times at 0.25% drift level. The subsequent displacement cycles consisted of three cycles at each displacement level where the displacement level increased in increments of 0.25% drift. Figures 4.35 and 4.36 show the loading program imposed on the walls. The test continued until the specimens were substantially damaged and the lateral load resistance dropped more than 50% of the maximum resistance.

Table 4.1: Prestressing cable load for partially reinforced wall (PRM)

Position	NW	SW	NM	SM	NE	SE
Load on each cable (kN)	12.98	24.85	14.2	18.18	16.14	15.5
Both Side (kN)	37.83		32.38		31.64	
Total (kN)	101.85					
NW= North west, SW= South west, NM= North middle, SM= South middle, NE= North east, SE= South east.						

Table 4.2: Prestressing cable load for unreinforced masonry wall (URM)

Position	NW	SW	NM	SM	NE	SE
Load on each cable	19.47	19.8	15.54	16.18	15.05	18.69
Both Side (kN)	39.27		31.72		33.74	
Total (kN)	104.73					
NW= North West, SW= South west, NM= North middle, SM= South middle, NE= North east, SE= South east.						

Table 4.3: Compressive strength of masonry unit.

Masonry Unit	Load (N)	Area (mm ²)	Strength (MPa)	Average Strength (MPa)
1	600000	39719	15.1	14.7
2	568641	39719	14.31	

Table 4.4: Compressive strength of foundation concrete from cylinder test.

Sample	Load (N)	Area (mm ²)	Strength (MPa)	Average Strength (MPa)
1	255080	7850	32.49	28.55
2	193200	7850	24.61	

Table 4.5: Compressive strength of top beam concrete from cylinder test (PRM)

Sample	Load (N)	Area (mm ²)	Strength (MPa)	Average Strength (MPa)
1	220320	7850	28.06	28.06

Table 4.6: Compressive strength of top beam concrete from cylinder test (URM)

Sample	Load (kN)	Area (mm ²)	Strength (MPa)	Average Strength (MPa)
1	341.3	7850	43.4	41.61
2	276.6	7850	35.2	
3	362.1	7850	46.1	

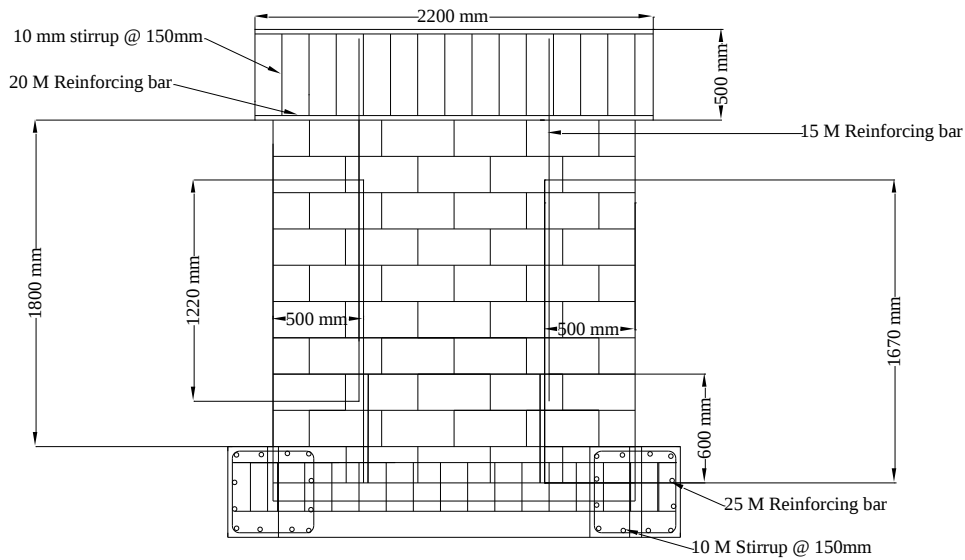


Figure 4.1: Geometric and reinforcement details of the partially reinforced masonry wall specimen



Figure 4.2: Integration of wall with reinforced concrete foundation

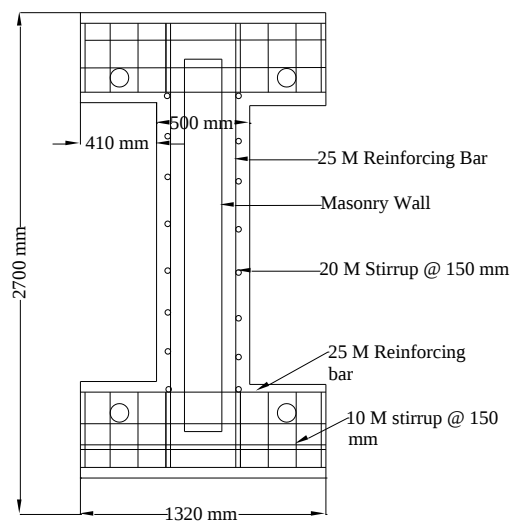


Figure 4.3: Reinforcement details of the foundation (Plan view)



Figure 4.4: Wall reinforcement extending into the top beam (PRM Wall)



Figure 4.5: Reinforcement cage in top beam for PRM wall



Figure 4.6: Formwork for top beam (PRM Wall)



Figure 4.7: PRM Wall after construction



Figure 4.8: Repair of wall foundation

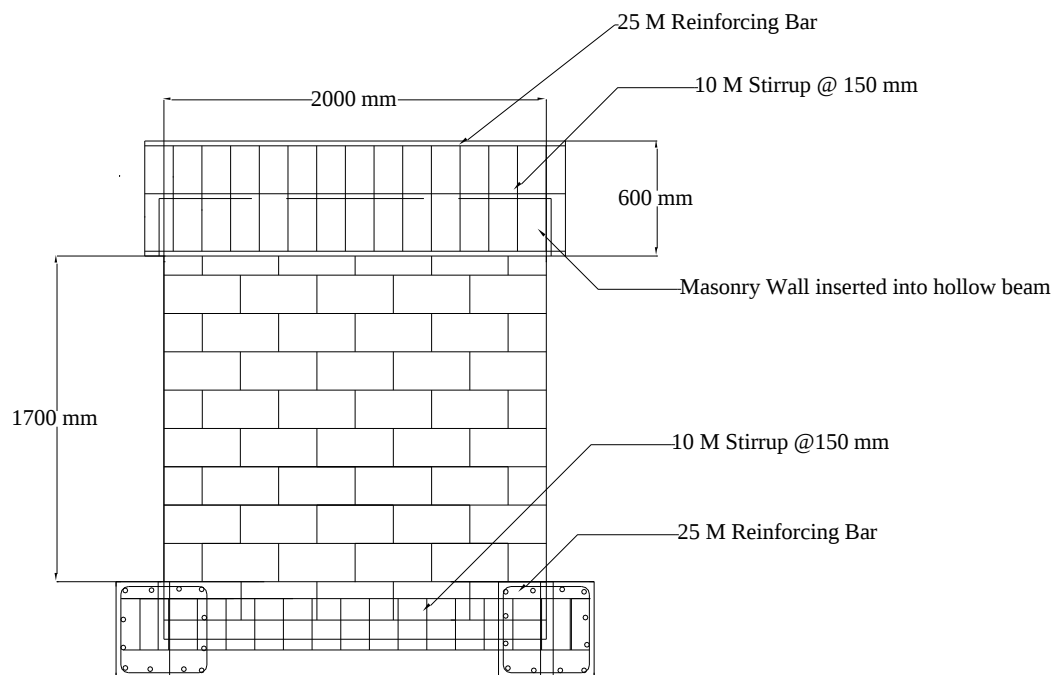


Figure 4.9: Geometric details of URM wall



Figure 4.10: URM wall during construction

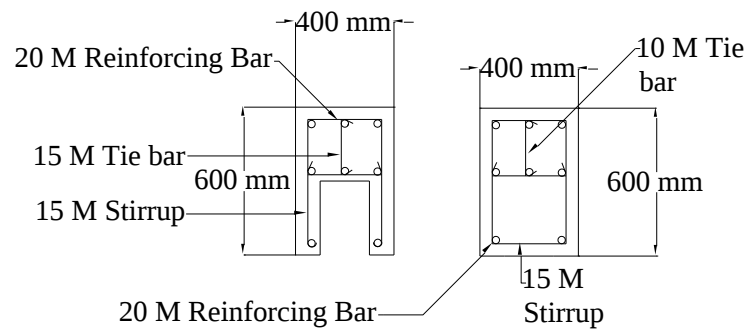


Figure 4.11: Cross sectional details of the precast top loading beam



Figure 4.12: The placement of precast top beam on the wall



Figure 4.13: URM wall after the placement of top beam



(a) Surface preparation with epoxy putty



(b) Smoothing of putty with sand paper



(c) Soaking of wall surface with epoxy



(d) Saturation of FRP sheet with epoxy



(e) Application of horizontal fibres



(f) Application of vertical fibres

Figure 4.14: Surface preparation and application of FRP sheets



Figure 4.15: FRP anchors



Figure 4.16: Rounded wall footing interface



Figure 4.17: FRP anchors after installation on the wall

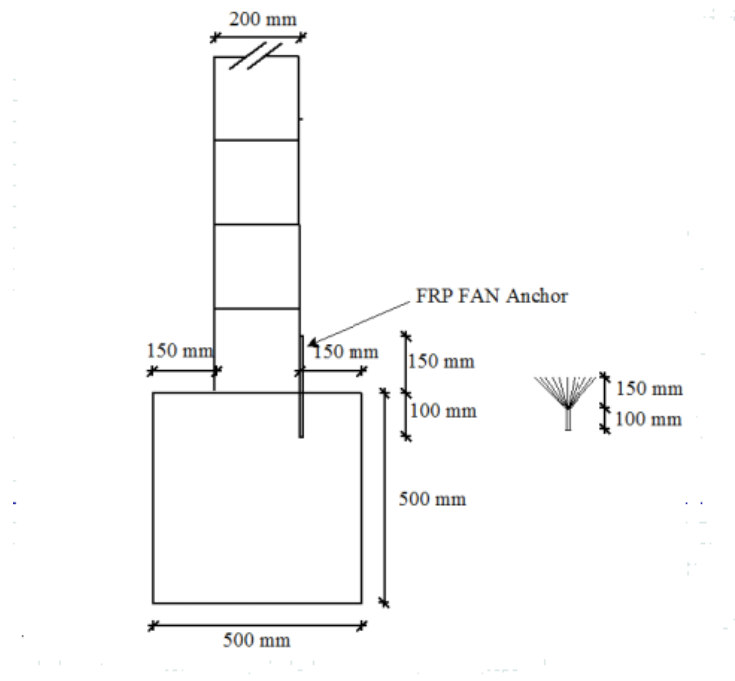


Figure 4.18: FRP anchor inserted into the foundation

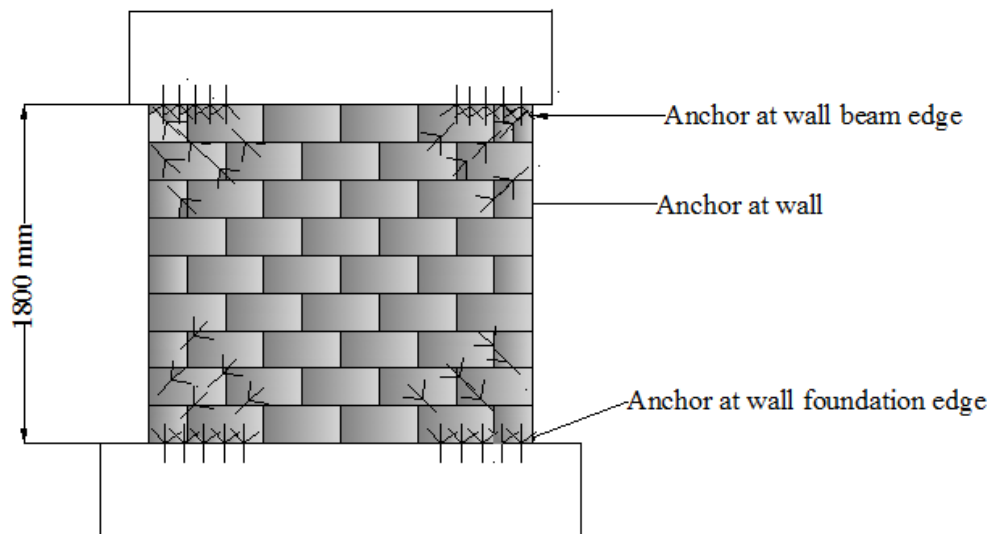


Figure 4.19: FRP anchor orientation for wall surface, foundation and beam edge.

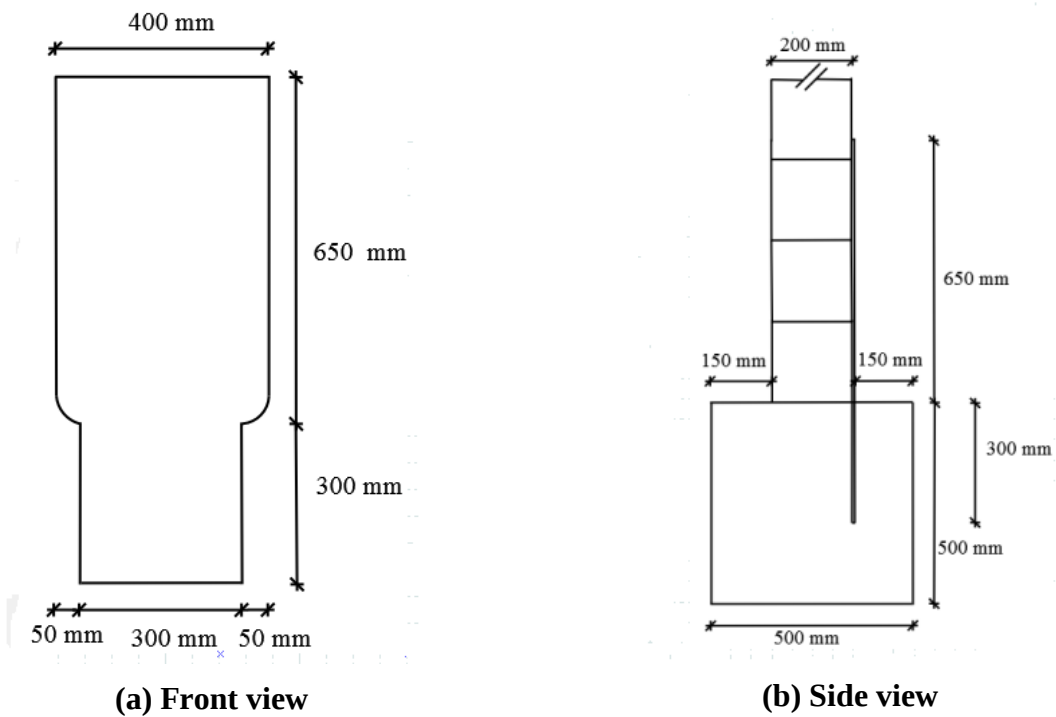


Figure 4.20: Geometric details of stainless steel sheet anchors

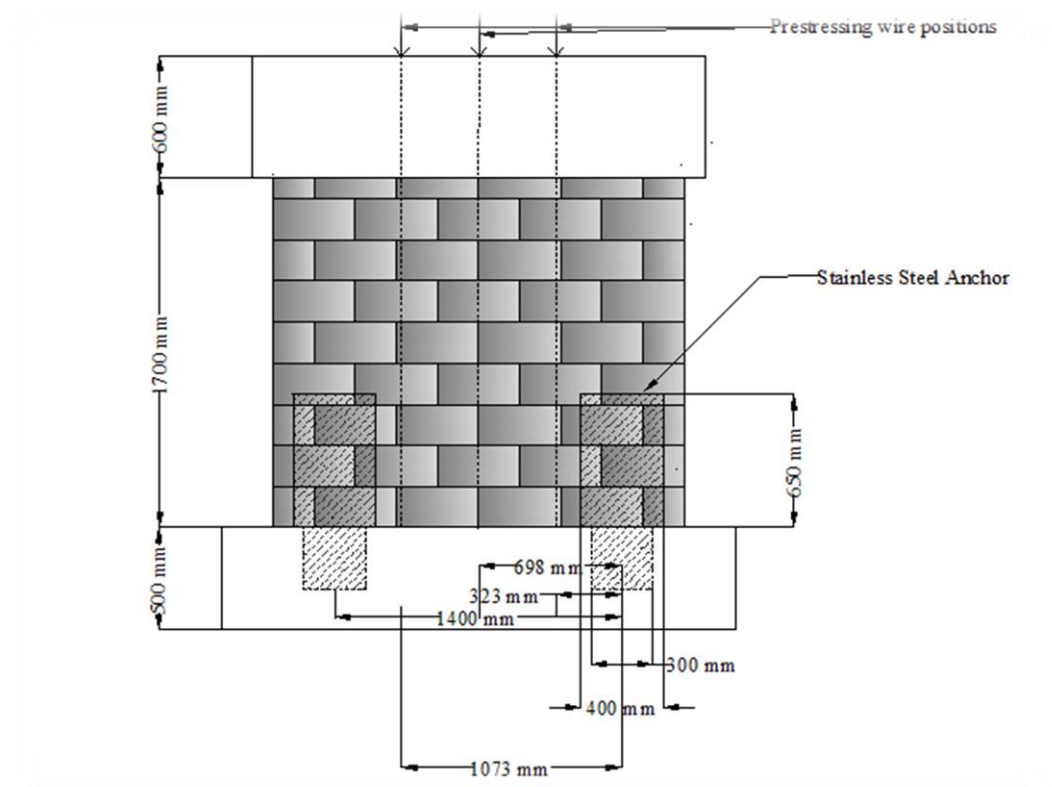


Figure 4.21: Positions of steel anchors in URM wall

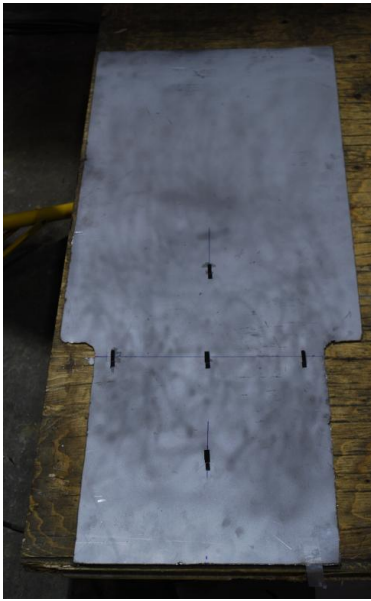


Figure 4.22: Stainless steel sheet anchor

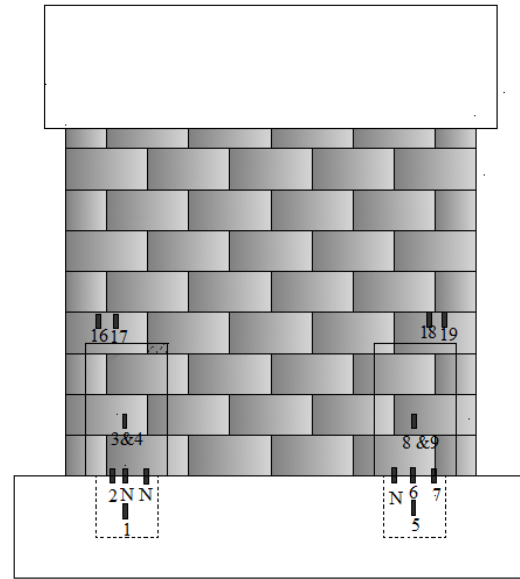


Figure 4.23: Strain gauge positions on URM wall



Figure 4.24: Cutting of foundation concrete for anchor placement



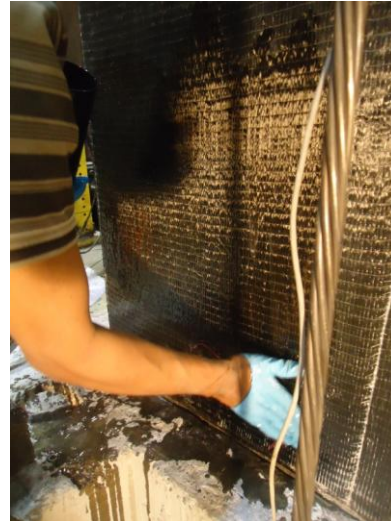
(a) Pouring of epoxy into the concrete slot



(b) After the placement of epoxy in concrete



(c) Soaking of the anchor in epoxy



(d) Saturation of wall FRP with epoxy



(e) Placement of the anchor in concrete



(f) After the placement of anchor

Figure 4.25: Preparation and placement of steel sheet anchors



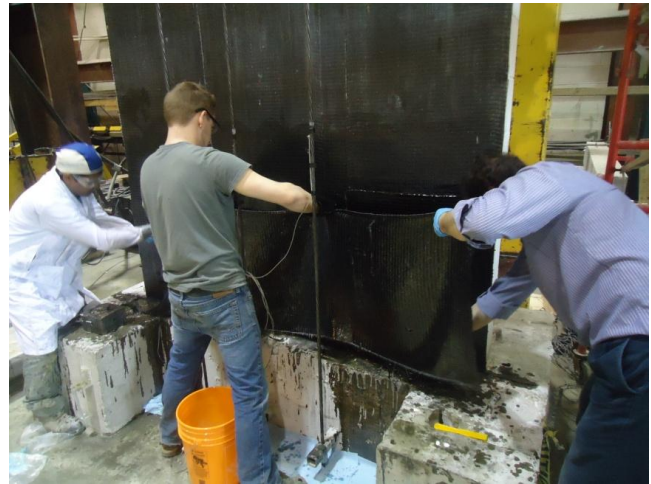
(a) Clamping of anchors



(b) Exposed anchors after adhesion



(c) First layer of FRP with vertical fibres



(d) Second layer of FRP with horizontal fibres

Figure 4.26: Securing of steel sheet anchors with layers of FRP

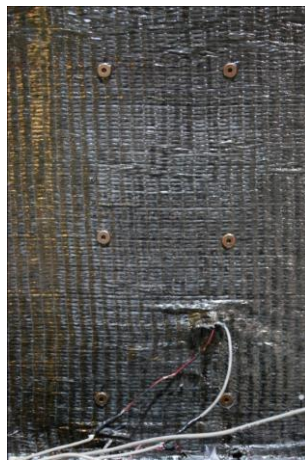


Figure 4.27: Threaded bars inserted into the wall through the steel anchor

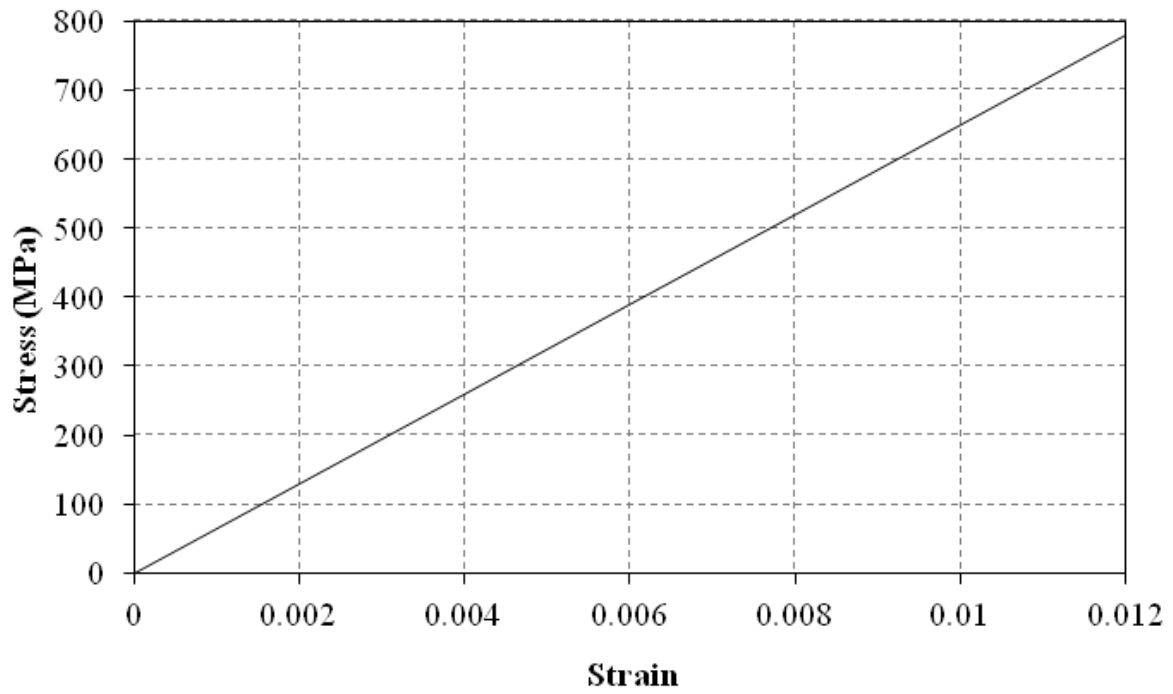


Figure 4.28: Stress strain relationship of FRP laminates

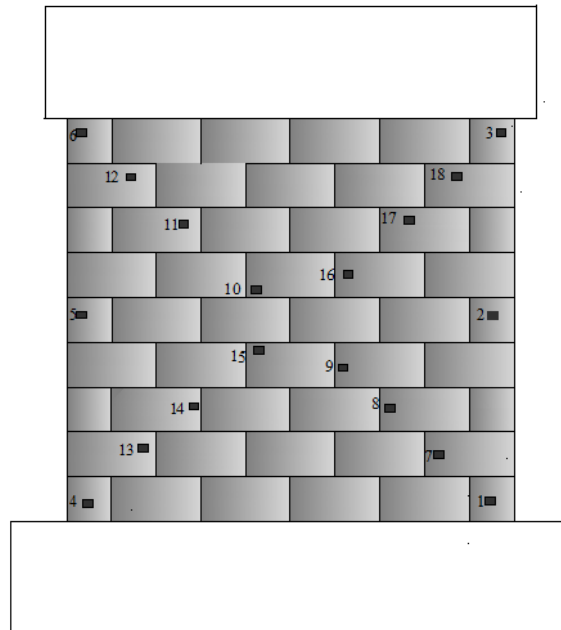


Figure 4.29: Strain gauge positions for PRM wall

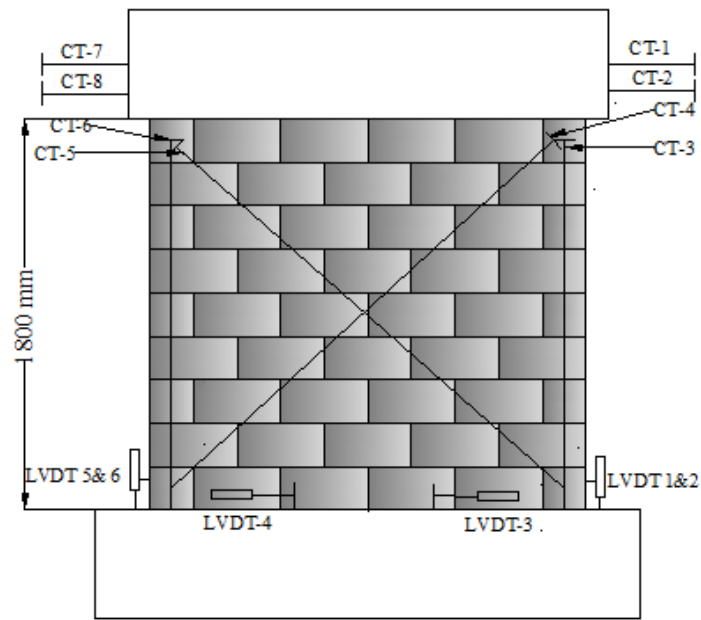


Figure 4.30: LVDT and CT positions for the PRM wall

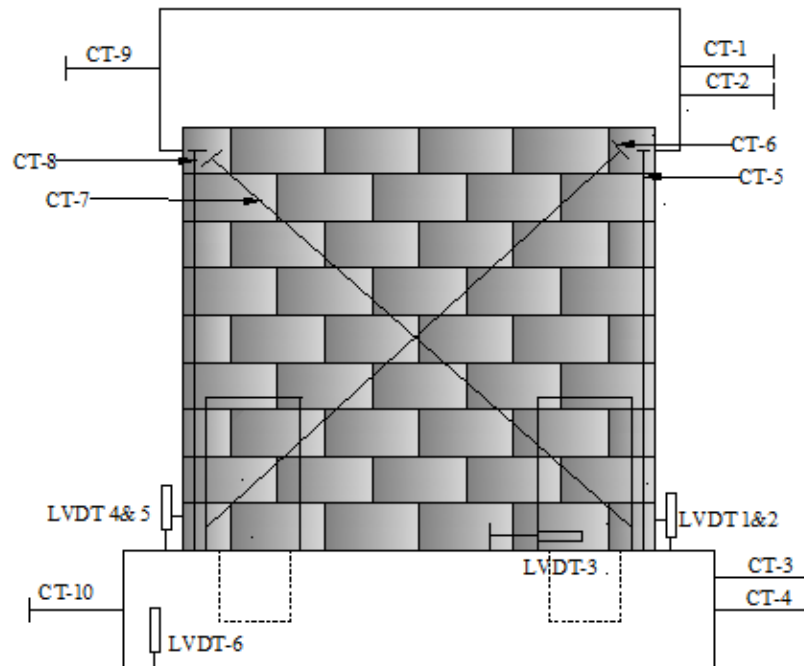


Figure 4.31: LVDT and CT positions for the URM wall

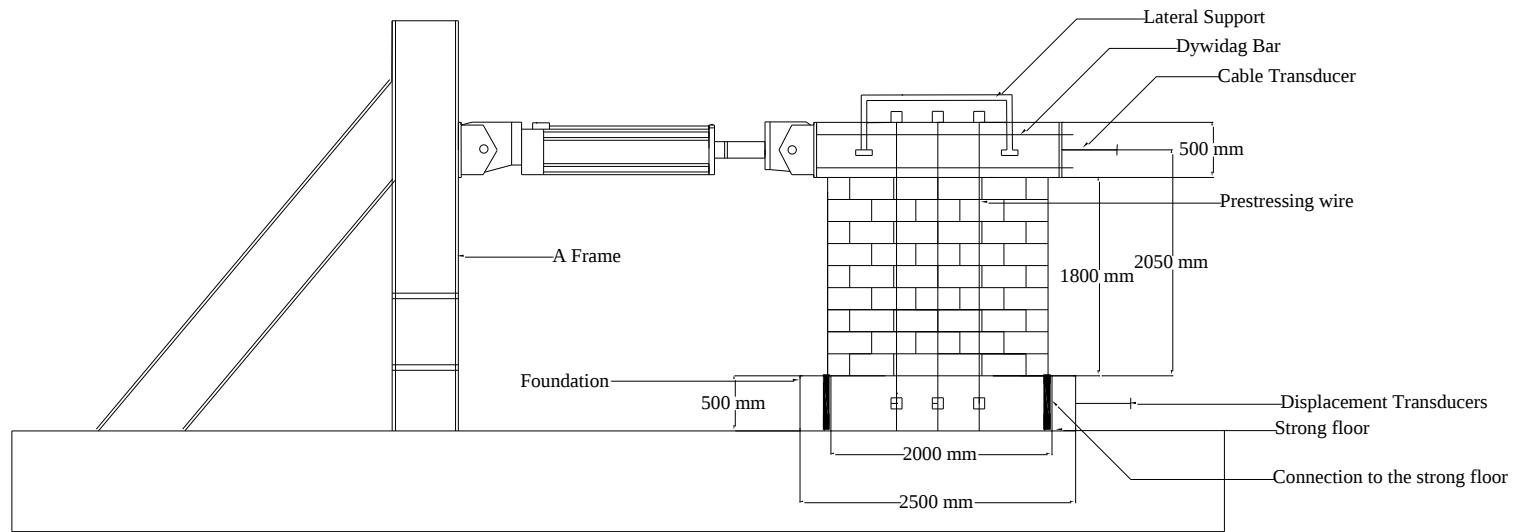


Figure 4.32: Test setup for the partially reinforced masonry wall



Figure 4.33: PRM wall prior to testing

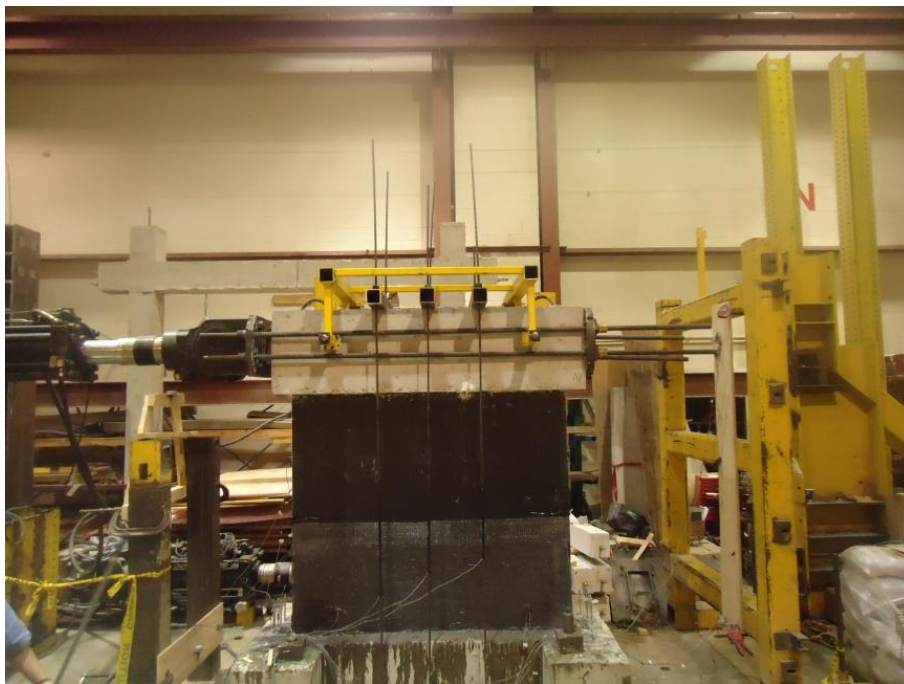


Figure 4.34: URM wall prior to testing

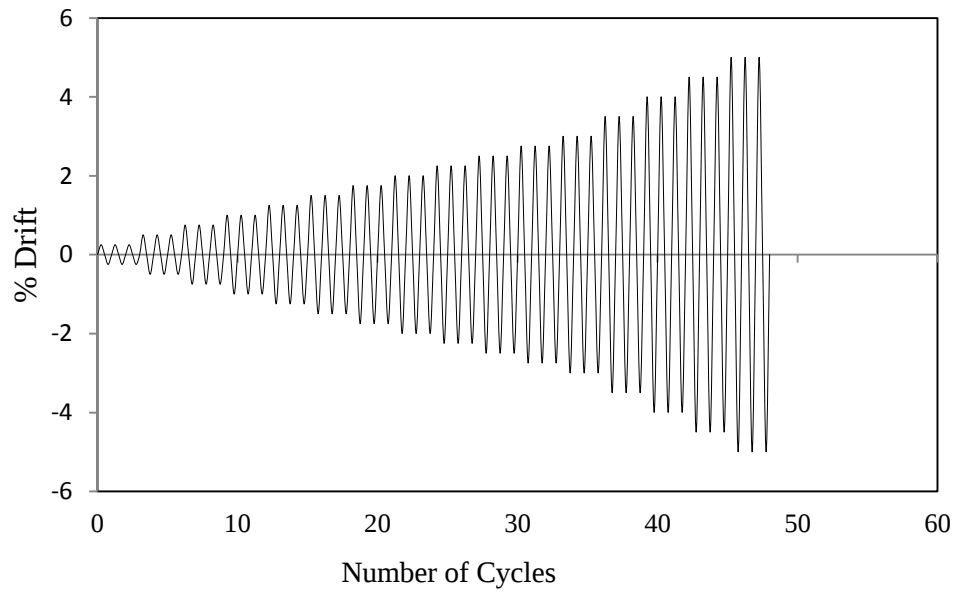


Figure 4.35: Loading history for the PRM wall

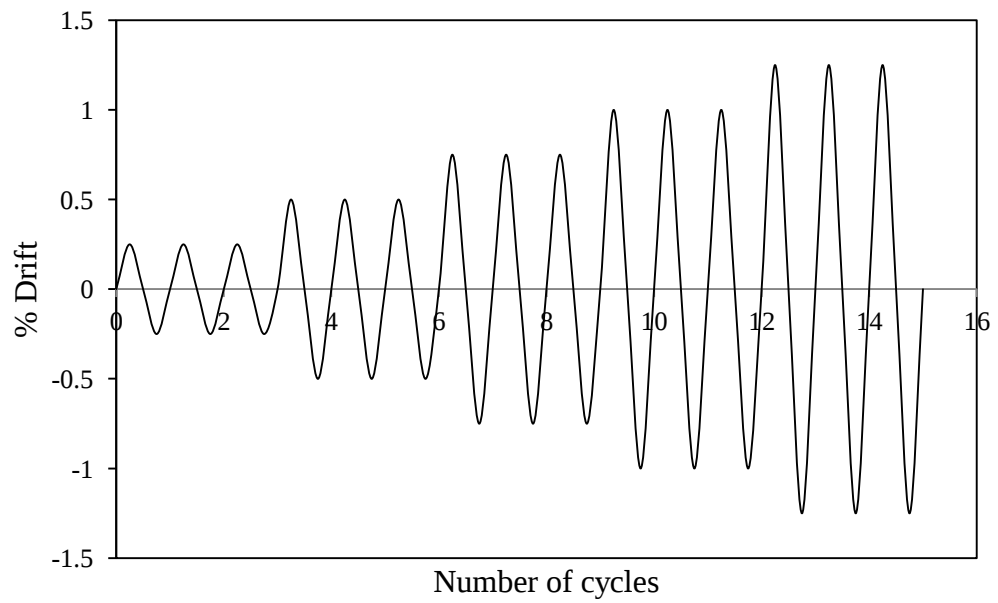


Figure 4.36: Loading history for the URM wall

Chapter 5

Test Results

5.1 Introduction

The results of two large scale masonry wall tests are presented in this chapter. The details of the test specimens, instrumentation and test procedure are presented in Chapter 4. This chapter includes observations made during testing in terms of the progression of cracking and damage to walls, as well as the recorded data. The test results recorded by the data acquisition system are plotted in the form of hysteretic force-deformation relationships. Strain gauge readings are also plotted as hysteretic force-strain relationships.

5.2 Observed Behaviour of Partially Reinforced Wall (PRM)

The partially reinforced masonry wall (PRM), which was retrofitted with surface mounted CFRP sheets on one side and anchored in the concrete foundation by means of FRP anchors, was tested following the loading history presented in Chapter 4 as shown in Figure 4.35. Accordingly, a constant gravity load was first applied through vertical prestressing. The wall was then subjected to incrementally increasing lateral deformation reversals until significant strength decay was observed.

Initial deformation reversals with maximum displacements ranging between $\pm 0.25\%$ horizontal drift ratio resulted in very few diagonal hairline cracks. The cracks were observed and marked on the unretrofitted (exposed) surface, because the other side was covered with FRP sheets as part of the retrofit strategy implemented. The initial hairline cracking is illustrated in Figure 5.1(a). The specimen resisted 105 kN in the push direction (loading towards East) and 100 kN in the pull direction. No other damage was observed at this early load stage.

During the first 0.5% drift cycle, the peak load resistance in the push and pull directions were increased to 165 kN and 174 kN, respectively. During the second cycle at this deformation level FRP de-bonding sound was heard near the foundation, on the west side, coming from the bottom

angle-shaped FRP strip that was used to provide additional chemical bond for connecting the wall to the foundation, in addition to the physical bond provided through FRP anchors. At this deformation level this angle-shaped FRP sheet started de-bonding and separating from the foundation concrete. The resistance during the second push cycle at 0.5% drift ratio dropped to 125 kN as the FRP anchors started pulling out from the foundation concrete, forming pull-out cones. Figure 5.1(b) shows the FRP fan anchors and the separation of the FRP from the foundation concrete. The diagonal cracks on the wall started widening and propagating through the mortar joints as illustrated in Figure 5.1(c). During the first cycle of 0.75% drift level, additional FRP anchor-pullout sound was heard, indicating the failure of the remaining anchors, accompanied by the separation of FRP from the wall foundation. The separation between the FRP and the foundation can be clearly seen in Figure 5.1(d). Under the pull force at this deformation level the diagonal crack propagated towards the east corner and became horizontal following the mortar bed just below the top course of blocks. This is illustrated in Figure 5.1(e). The pull force resistance was slightly higher than the push force. This was attributed to the difference in anchor strength on each side, caused by the imperfections during installation. The force resistance during pushing started to drop due to the loss of anchors. During the 0.75% cycles the anchor pullout continued as the FRP de-bonded from the foundation. The load resistance at this deformation level was recorded to be 138 kN and 182 kN in push and pull directions, respectively. During the second cycle of 0.75% drift, the load resistance in the pull direction dropped to 166 kN and it further reduced to 156 kN during subsequent cycles. The crushing of the corner masonry block at the east end intensified as the wall lost part of its tension elements (FRP anchors) as depicted in Figure 5.1(f).

The separation and lifting up of the FRP from the foundation continued during 1% drift cycles. LVDT-2, which was attached to the wall on the east side, became unusable as the supporting masonry block split apart. More crushing of masonry was observed during the second cycle of 1% drift on the east side. The second course of masonry blocks from the foundation showed separation on the east side, while the blocks on the west side remained intact. During the third cycle, the dislodging of the bottom block in the east corner became more apparent as shown in Figure 5.2(a). The peak load resistance at this deformation level was 120 kN in the push direction, and 107 kN in the pull direction. The failure of the anchors became visible through the opening between the FRP and the foundation, as shown in Figure 5.2(b). The separation of FRP

from the foundation increased during the subsequent drift level at 1.25%, and the corner crushing at the east end intensified as illustrated in Figures 5.2(c) and 5.2(d), respectively. The peak load resistance at this deformation level was 100 kN in push and 110 kN in pull.

During the 1.5% drift cycles the wall started to rock as a rigid body, with the blocks attached together by the wall CFRP. Figures 5.2(e) and 5.2(f) show the wall during rocking. At this deformation level LVDT-1, LVDT-2 and LVDT-3 were completely detached from the specimen. During the second cycle, the separation of the wall from its foundation reached to a maximum vertical uplift of 12 mm. The peak load resistance at this drift level was 100 kN in push, and 109 kN in pull. At 1.75% drift level, vertical cracks were formed along the vertical joints. The wall continued rocking as illustrated in Figures 5.3(a) and 5.3(b), with separation of the wall along the foundation extending up to 900 mm from the west end. The wall did not experience masonry crushing on the west end as can be seen in Figure 5.3(a). The peak loads attained at 1.75% drift were 100 kN and 114 kN during push and pull, respectively. The wall exhibited similar rocking behaviour without any change in load resistance during the 2% drift cycles, except for increased separation of the wall from its foundation. This is shown in Figure 5.3(c). Because the wall behaviour remained unchanged, it was decided to increase deformations in increments of 0.5% drift ratio beyond this deformation level. The performance remained the same during the first cycle at 2.5% drift. The second cycle resulted in the extreme prestressing strand becoming loose and detaching from its lock. The peak load resistance in push and pull were 102 kN and 120 kN, respectively. The wall continued rocking during 3% drift cycles. Some of the instrumentation was dislodged due to wall damage, and the prestressing strands used to apply the gravity loads became loose. Figure 5.3(d) shows damage near the east end. Peak load resistances were recorded to be 94 kN in push and 126 kN in pull directions. The wall continued experiencing increased damage during 3.5% drift cycles. The crushing of masonry near the east end became excessive. The overlap between the wall FRP and the angle-shaped FRP piece near the bottom showed delamination, as illustrated in Figure 5.3(e). Peak load resistances were 73 kN in push and 122 kN in pull directions. The rocking behaviour continued as the corner crushing started to occur at the west end, as depicted in Figure 5.3(f).

During 4% drift cycles the masonry crushing became excessive at the east end, and more pronounced at the west end. Peak load resistances in push and pull were 66 kN and 126 kN,

respectively. The separation of the wall from its foundation increased on both sides as shown in Figure 5.4(a). Masonry deterioration continued during the 4.5% drift cycles, with peak load resistances dropping to 59 kN and 129 kN in push and pull directions, respectively.

The east end of the wall was damaged completely during the 5% drift cycles, as depicted in Figure 5.4(b). The peak resistance at this load stage dropped to 38 kN in push because of the loss of compression resistance. It was recorded to be 122 kN in the pull direction. As the masonry crushed substantially on both east and west sides, a decision was made to stop the test.

5.3 Hysteretic Behaviour of PRM Wall and Comparison with Unretrofitted PRM Wall

The hysteretic relationship of the PRM wall, retrofitted with single-side surface-mounted CFRP sheets, and anchored to the concrete foundation by means of FRP anchors, is shown in Figure 5.5. The hysteresis loops indicate elastic response up to 0.5% lateral drift. The wall reached its maximum lateral force resistance at this deformation level, developing 165 kN in the push direction during the first cycle, and 174 kN in the pull direction during the second cycle. The lateral force capacity was limited by de-bonding of the angle-shaped FRP sheets from the foundation concrete, followed by the pull-out of the FRP anchors near the extreme tension fibre. Pull-out of the remaining anchors occurred during the subsequent 0.75% drift cycles, and the load resistance gradually dropped to 138 kN in the push direction. The load resistance in the pull direction remained high at this deformation level, reaching 182 kN. At 1% lateral drift the anchors on the east side also started pulling out and causing a drop in load resistance to 120 kN in the pull direction. During subsequent deformation reversals the load resistance remained around 100 kN until 2.75% drift, during which the wall exhibited rocking behaviour with some flexural resistance provided by the internal # 15M reinforcement. Beyond the 2.75% drift ratio, further drop in load resistance was observed due to the crushing of masonry on the east side. The wall gradually lost flexural compression resistance and suffered from severe strength degradation up to 5% lateral drift when the test was stopped. The behaviour in the pull direction was slightly different as the crushing of the west toe occurred at a later stage. The compression resistance of masonry and tension resistance of #15M bar continued providing some moment resistance until the masonry crushing occurred at the west end, shortly after 4% lateral drift.

The surface bonded FRP on the south face of the wall controlled diagonal cracking, and prevented shear failure of the wall. There was no de-bonding of FRP observed on the wall, except for the delamination of the angle-shaped FRP piece from the wall FRP at 3.5% lateral drift. The wall failed due to the compression crushing of masonry. The gradual crushing of the bed mortar along the foundation and the rocking response resulted in the pinching of hysteresis loops.

A companion partially reinforced masonry wall was tested in an earlier investigation at the University of Ottawa by Taghdi Bruneau and Saatcioglu (2000). The wall had the same overall dimension, consisting of a square masonry wall panel with 1800 mm length and height. It was reinforced with 3 # 15M bars as opposed to the wall tested in the current investigation, which had 2 #15 M bars. It was subjected to the same level of constant axial compression (100 kN). The details of the unretrofitted specimen are shown in Figure 5.6. The wall developed lateral load capacity of 120 kN, and suffered from shear failure. Wide diagonal tension cracks were formed, followed by the crushing of diagonal compression struts at 0.6% drift. The longitudinal reinforcement did not develop its yield strength and the wall failed prematurely in shear prior to developing its flexural capacity. The wall lost 60% of its capacity at 0.8% lateral drift when the test was discontinued.

The comparison of unretrofitted and retrofitted PRM walls indicate a clear change in behaviour due to the FRP retrofit. The flexural capacity of the retrofitted wall was developed and the shear failure was completely eliminated by the surface mounted CFRP. The lateral load capacity was increased by 40% to 50% depending on the direction of loading. The increase in load capacity was controlled by the capacity of the anchors. The FRP anchors, which were placed to transfer tension forces from the wall FRP to the foundation, provided improvements in flexural resistance but failed by pulling out of concrete prior to developing their rupturing capacity. This was attributed to the insufficient anchor depth provided. However, they provided 40% to 50% increase in capacity, proving that the retrofit strategy employed can be implemented for flexural strengthening. It is also important to note that the surface bonded CFRP sheets fulfilled their functions and prevented shear failure, which was the mode of failure observed in the unretrofitted wall specimen.

The retrofitted PRM wall was instrumented to measure the vertical movement of the wall, both in terms of the movement of the top beam relative to the foundation, and the vertical uplift of the wall panel along the wall foundation joint. Figure 5.7 indicates that the maximum vertical uplift of the top beam on the wall was approximately 60 mm at each end, when the wall end is subjected to maximum tensile forces. Figure 5.8 indicates that about one half of the vertical uplift of the top beam occurred between the wall panel and the foundation. Furthermore, displacement measurements were taken along each diagonal, as shown in Figure 5.9. The diagonal tension and compression deformations ranged between 4 mm to 7mm, reconfirming the effectiveness of surface-bonded FRP in controlling diagonal tension cracks. Sliding shear displacement along the wall base was monitored and measured to vary between 1 mm to 4mm, as depicted in Figure 5.10. In addition, strains on the CFRP were recorded at selected locations. Figure 5.11 indicates that vertical tensile strains in FRP were small and controlled by the failure of bond on foundation and the pull out of the FRP anchors. The maximum value recorded at the east end, near the critical section was only 0.0002.

5.4 Observed Behaviour in Unreinforced Masonry Wall (URM)

The unreinforced masonry wall (URM) was retrofitted with surface-mounted CFRP sheets on one side, with ductile steel sheet anchors secured in the foundation concrete. It was tested by following the loading history presented in Chapter 4 as displayed in Figure 4.36. Accordingly, a constant gravity load was first applied on the wall through vertical prestressing. The wall was then subjected to incrementally increasing lateral deformation reversals until significant strength decay was observed.

Initial three deformation reversals were applied with maximum displacements ranging between $\pm 0.25\%$ horizontal drift ratio. The wall showed completely elastic behaviour during these initial cycles, without any sign of distress. Figures 5.12(a) and 5.12(b) illustrate the wall during this early stage of testing. However, significant yielding of the steel sheet anchors was observed during these deformation cycles, starting at approximately 100 kN lateral load. The maximum lateral load resistance was recorded to be 160 kN and 164 kN in the push and pull directions, respectively. At the peak load of this deformation level one of the strain gauges on the east anchor, at wall footing interface, recorded 1% tensile strain indicating significant yielding at the location of the gauge. At 0.5 % drift, no shear cracks were observed. The wall started acting as a

rigid body with a crack forming at the wall-foundation interface. The interface crack was visible up to approximately half the wall length. The behaviour of the wall is presented in Figures 5.12(c) and 5.12(d). The peak load resistance at 0.5% lateral drift increased to 224 kN and 211 kN during the push and pull cycles, respectively. The strain gauges on the anchors were lost at this stage. However, the steel anchors continued performing well during 0.75% drift cycles and continued experiencing inelastic deformation cycles. The overall view of the wall at this deformation stage is shown in the Figure 5.12(e). Minor local de-bonding of FRP was observed on the east side, below the bottom row of threaded bars. However, there was no sign of debonding on the west side. The maximum load resistances at this drift level were 215 kN and 220 kN in pull and push, respectively. There were no shear cracks or corner crushing observed at this deformation level. During the first pull to 0.75% drift, a piece of concrete within the foundation cover became loose and lifted up along with the extending steel sheet anchor in tension. This resulted in a vertical movement of the east end as shown in Figure 5.12(f), and caused a drop in lateral force resistance from 220 kN to 128 kN. The load was recovered to its previous level after two cycles, indicating that the effect was to reduce wall stiffness in this direction, but not the strength. Maximum lateral force during the push cycles remained unchanged.

Diagonal shear cracks started forming at 1% lateral drift. This is shown in Figure 5.13(a). The shear crack initiated from the west side in between sixth and seventh course of masonry blocks. This is explained by the presence of an extra layer of CFRP placed over the steel sheet anchors below the level at which the cracks have formed. Shear cracks have not formed along the opposite diagonal. Buckling of the steel sheet anchor was observed on the east side during push cycles. This is shown in Figure 5.13(b). The maximum load resistance at 1% drift was observed to be 218 kN and 178 kN in the push and pull directions, respectively.

At 1.25 % drift level, additional diagonal cracks were formed along the same diagonal as before. Figure 5.13(c) shows the propagation of diagonal cracks. The steel sheet anchors on both sides showed buckling in compression between the bottom row of blocks and the foundation. This resulted in the widening of the interface crack, which reached a maximum crack width of 25 mm. There were no shear cracks observed along the opposite diagonal. The peak load resistances

were recorded as 227 kN and 199 kN in the push and pull directions, respectively. The strength loss that was observed in the previous cycle has recovered at this deformation level.

During the 1.5% drift cycles the wall began to twist when deformed towards the push direction. This is shown in Figures 5.14(a) through 5.14(c). Shear cracks were observed along both diagonals. The steel sheet anchors developed extensive elongation in tension and subsequent buckling in compression. The wall twisted along two diagonal cracks. However, it continued resisting load while pulling. As it was not safe to continue testing in the push direction, it was decided to continue the test by applying pull cycles between the incrementally increasing lateral drift and zero load. The pull cycles continued between 2% and 3.5 % lateral drift with 0.5 % drift ratio increments. The capacity in the pull direction was almost 200 kN at each drift level. The damage observed during these cycles is illustrated in Figure 5.15. The masonry was damaged significantly, The steel anchors elongated significantly due to the high elongation characteristics of this type of steel, and buckled in compression.

5.5 Hysteretic Behaviour of URM Wall and Comparison with Unretrofitted URM Wall

The hysteretic relationship for the URM wall, retrofitted with one-side surface-mounted CFRP sheets, and anchored to the concrete foundation by means of steel sheet anchors, is shown in Figure 5.16. The hysteresis loops indicate elastic response up to 0.25% lateral drift. The onset of yielding of steel anchors occurred as the wall was loaded towards 0.5% drift. This is indicated on the hysteretic behaviour with a significant change in slope of the load-drift relationship. However, the lateral resistance continued increasing up to 0.5% drift, when the wall reached its capacity, and developed 224 kN and 211 kN during the push and pull cycles, respectively. Subsequent cycles developed similar load resistance up to 1.5% drift ratio, but showed progressive softening of the wall as evidenced by reduction in slopes of hysteresis loops. There was sudden drop in load resistance during the first pull cycle to 0.75% lateral drift when the cover concrete of the footing on the east side pulled out attached to the anchor. This loss in strength was subsequently recovered during the pull cycles at 1% drift. Unloading branches of hysteresis loops showed pinching, because of the buckling of steel sheet anchors in compression after yielding in tension. However there was no strength decay up to about 1.5% lateral drift. The

wall maintained its diagonal tension resistance until the toe concrete started crushing under combined flexural and diagonal compression stresses. The asymmetry in wall resistance, created by applying the retrofit materials (both CFRP and steel sheet anchors) on one side of the wall only, resulted in twisting of this URM wall, as the bottom layer of masonry remained connected to the foundation, and the wall started developing a crack along the mortar joint between the bottom and the second layer of concrete blocks. The test beyond 1.5% drift continued by applying deformation cycles only in the pull direction. Figure 5.17 illustrates the hysteretic relationship with the additional cycles in the pull direction, up to 4.5% drift level, signifying that the anchors continue providing tensile resistance even at this high inelastic deformation level.

A companion URM wall was tested by Taghdi et al (2000) in an earlier phase of the same research program at the University of Ottawa without retrofit. The geometrical details of the unretrofitted URM wall are shown in Figure 5.18(a). Both the previously tested unretrofitted wall and the current retrofitted wall had similar geometry and material properties. They were both tested under a constant gravity load of 100 kN. The hysteretic relationship of the unretrofitted wall is shown in Figure 5.18(b). The behaviour of this wall was dominated by rocking, and the capacity was limited to 60 kN. The wall experienced sliding in one direction, and never recovered from this shift along the base joint. In comparison; the lateral load capacity of the retrofitted specimen increased almost four times due to the presence of steel anchors, which provided transfer of tensile forces to the foundation. The comparison indicates that the steel sheet anchors provided significant strength and deformability to the URM wall tested, and hence can be used as an effective seismic retrofit strategy for otherwise brittle URM construction.

The retrofitted URM wall was instrumented to measure the vertical movement of the wall, both in terms of the movement of the top beam relative to the foundation, and the vertical uplift of the wall panel along the wall foundation joint. Figure 5.19(a) indicates that the maximum vertical uplift of the top beam on the wall was approximately 60 mm at the east end when the wall end was subjected to the maximum tensile force. The LVDT readings on the west side stopped recording during testing, and hence the hysteretic relationship shown in Figure 5.19(b) is regarded as incomplete. Figure 5.20 indicates that about one half of the vertical uplift of the top beam occurred between the wall panel and the foundation. Furthermore, displacement

measurements were taken along each diagonal, as shown in Figure 5.21. The cable transducers worked intermittently, and did not produce continuous readings. However they did indicate the maximum displacement recorded varied between 4 mm to 5mm, reconfirming the effectiveness of surface-bonded FRP in controlling diagonal tension cracks. Sliding shear displacement along the wall base was monitored and measured to vary between 2 mm to 6mm until failure, as depicted in Figure 5.22. In addition, strains on the CFRP were recorded at selected locations. Figure 5.23 and 5.24 indicate that vertical tensile strains in FRP reached a maximum strain of about 0.002, which proves that the steel sheet anchors fulfilled their functions as tension transfer elements and transferred tensile forces to the surface-bonded FRP on the wall. The readings of a strain gauge placed on the east end steel-sheet anchor are shown in Figure 5.25. This strain gauge was placed at the critical section between the wall panel and the foundation where the cross-sectional area was intentionally reduced to promote yielding. The figure indicates a maximum strain value of 1.65% at 0.25% drift level before it stopped recording strains due to the onset of buckling and excessive deformations in this critical region. However, these readings confirm the visual observations, indicating excessive inelastic deformations in the anchor associated with ductile behaviour of wall in flexure, while the surface bonded FRP prevented shear failure.



(a) Hairline diagonal cracks at 0.25% drift



(b) FRP fan anchors at 0.5% drift



(c) Diagonal cracks at 0.5% drift



(d) Separation of FRP at 0.5% drift



(e) Propagation of diagonal crack; 0.75% drift



(f) Corner crushing, east end, 0.75% drift

Figure 5.1: Behaviour of PRM wall up to 0.75% drift ratio



(a) Dislodged masonry block; 1.0% drift



(b) Failure of FRP anchors; 1.0% drift



(c) Separation of FRP; 1.25% drift



(d) East corner crushing; 1.25% drift



(e) Onset of crushing, west end; 1.5% drift



(f) Performance of anchors at 1.5% drift

Figure 5.2: Performance of PRM wall between 1.0% and 1.5% drift ratios



(a) West end at 1.75% drift



(b) Wall rocking at 1.75% drift



(c) FRP Separation at 2.0% drift



(d) Crushing of east end; 3.0% drift



(e) FRP delamination at 3.5% drift



(f) Lifting of west end; 3.5% drift

Figure 5.3: Performance of PRM wall between 1.75% and 3.5% drift ratios



(a) Lifting of west end at 4% drift



(b) Crushing of east end at 5% drift

Figure 5.4: Performance of PRM wall at 4% and 5% drift ratios

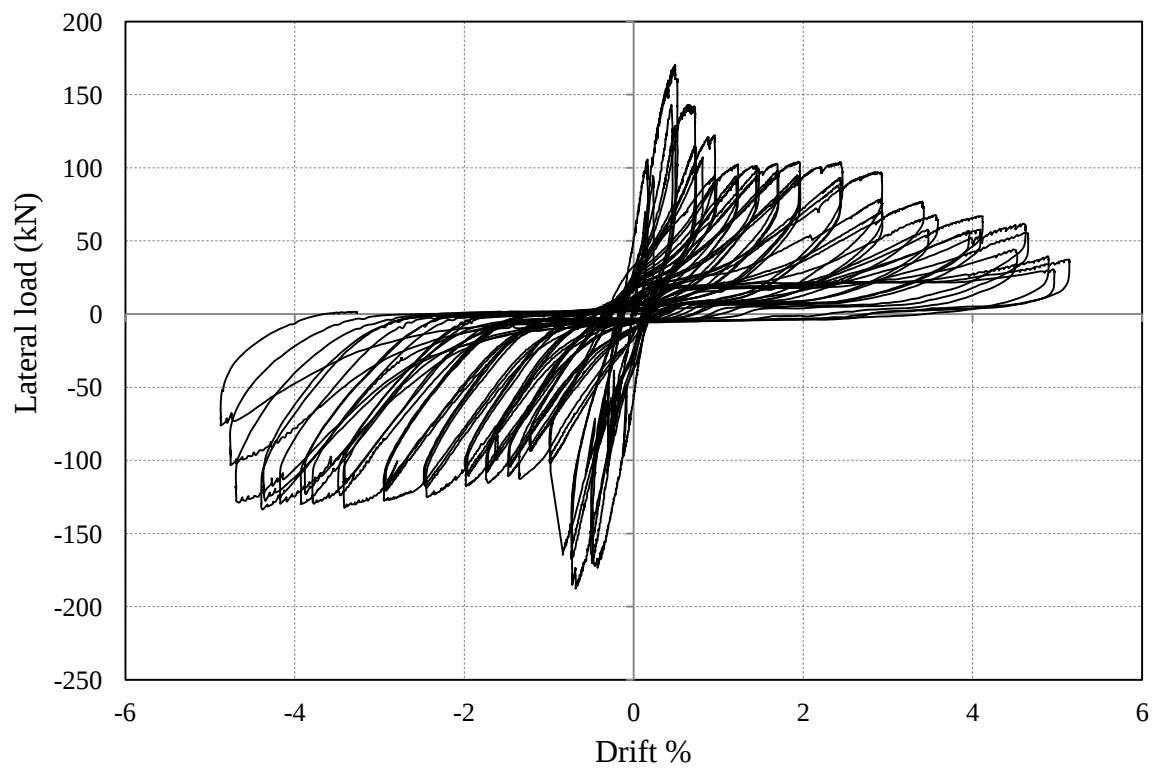
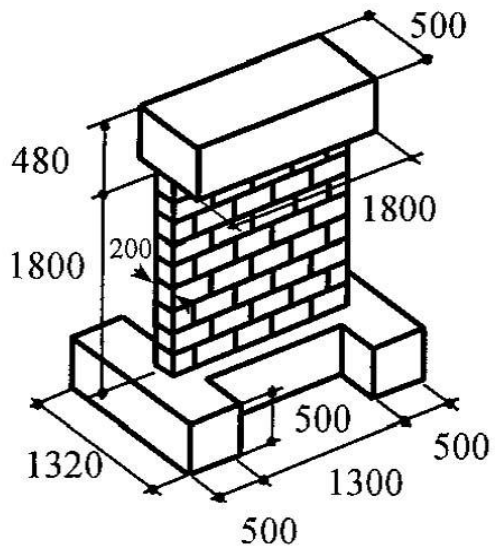
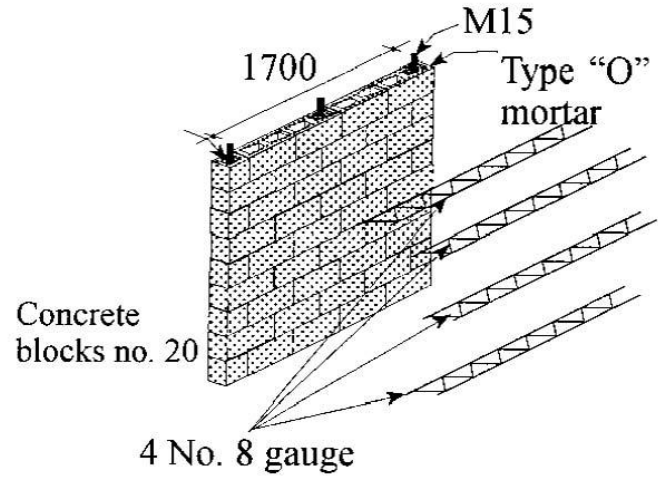


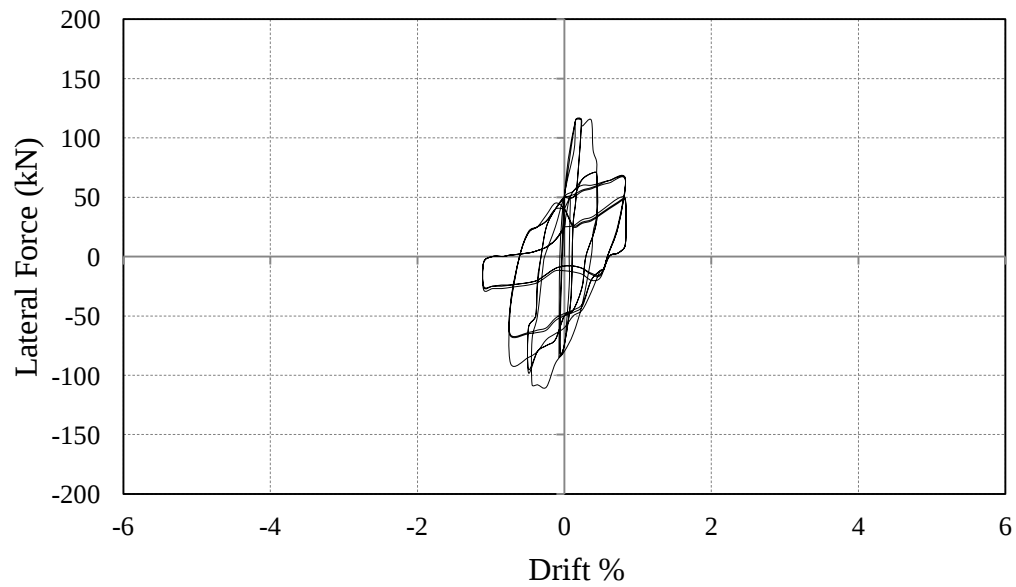
Figure 5.5: Hysteretic load-displacement relationship of PRM wall



(a) Geometric details (mm)

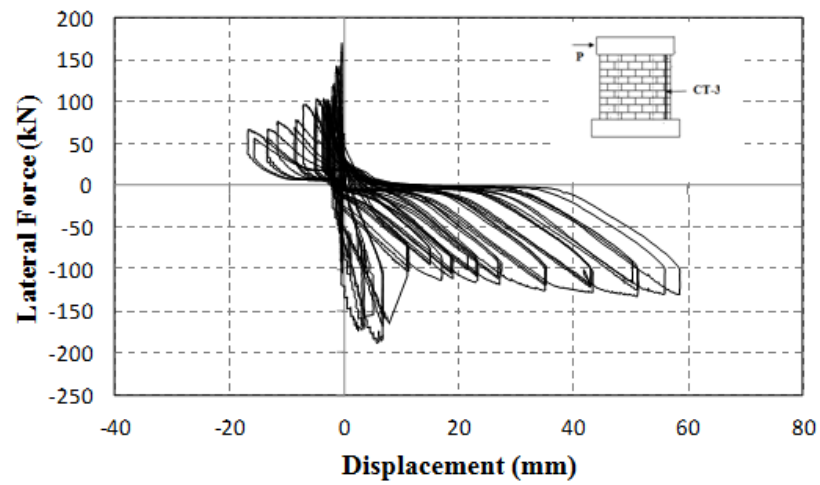


(b) Reinforcement details

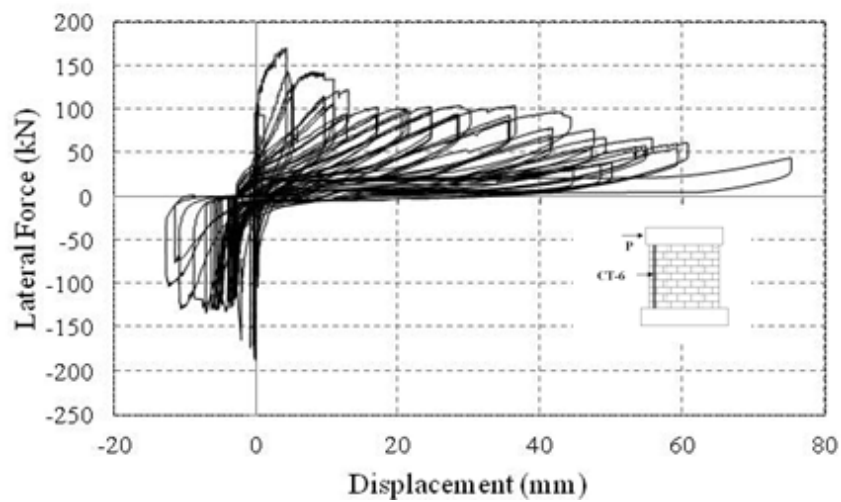


(c) Hysteretic relationship

Figure 5.6: Unretrofitted PRM Wall Test (Taghdi et al. 2000)

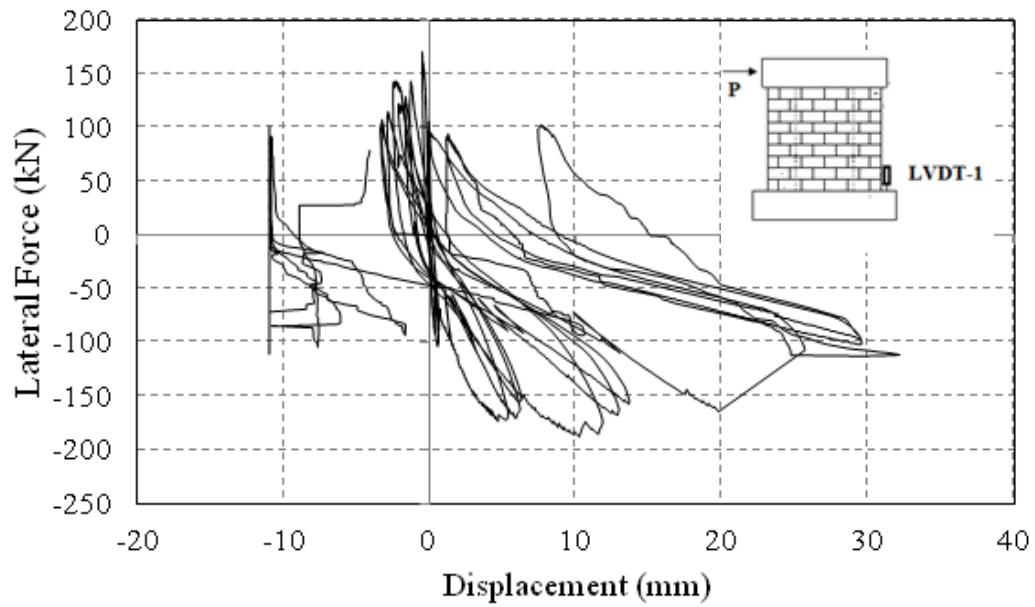


(a) Lateral force vs. vertical deflection of top beam relative to foundation at east end

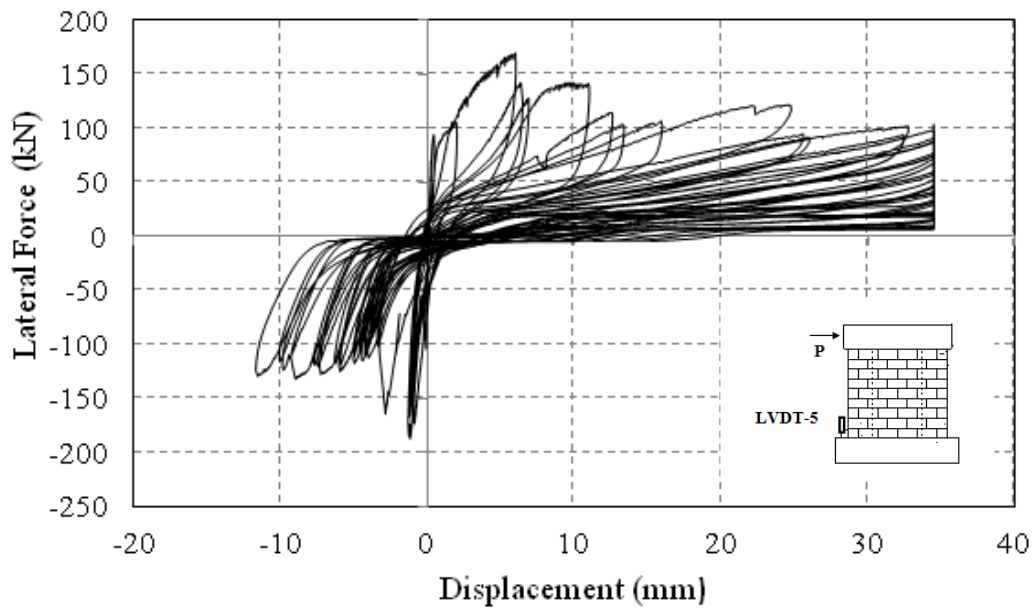


(b) Lateral force vs. vertical deflection of top beam relative to foundation at west end

Figure 5.7: Total Vertical Movement of PRM Wall

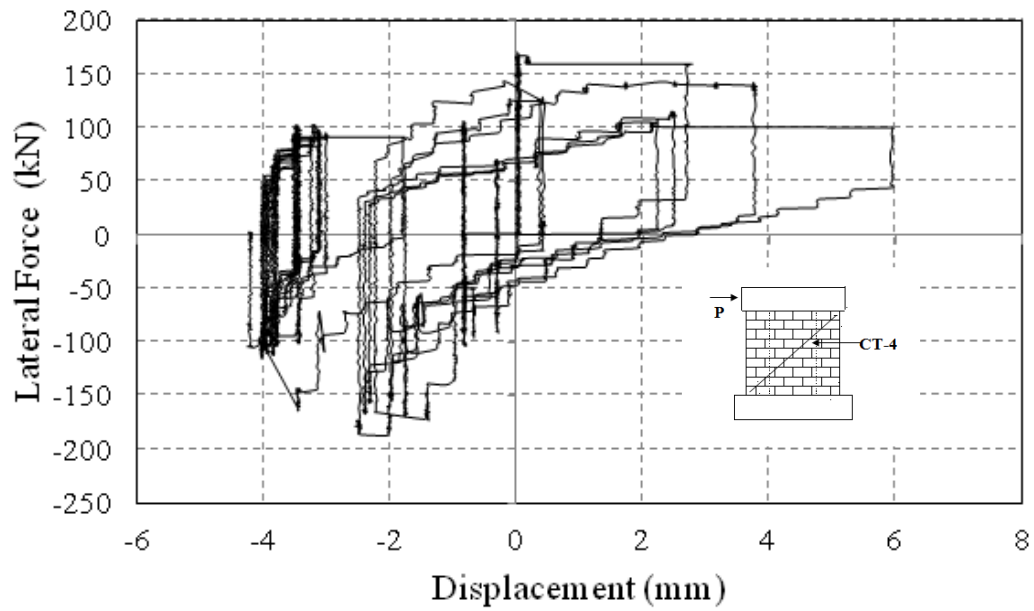


(a) Lateral force vs. wall uplift relative to foundation at east end

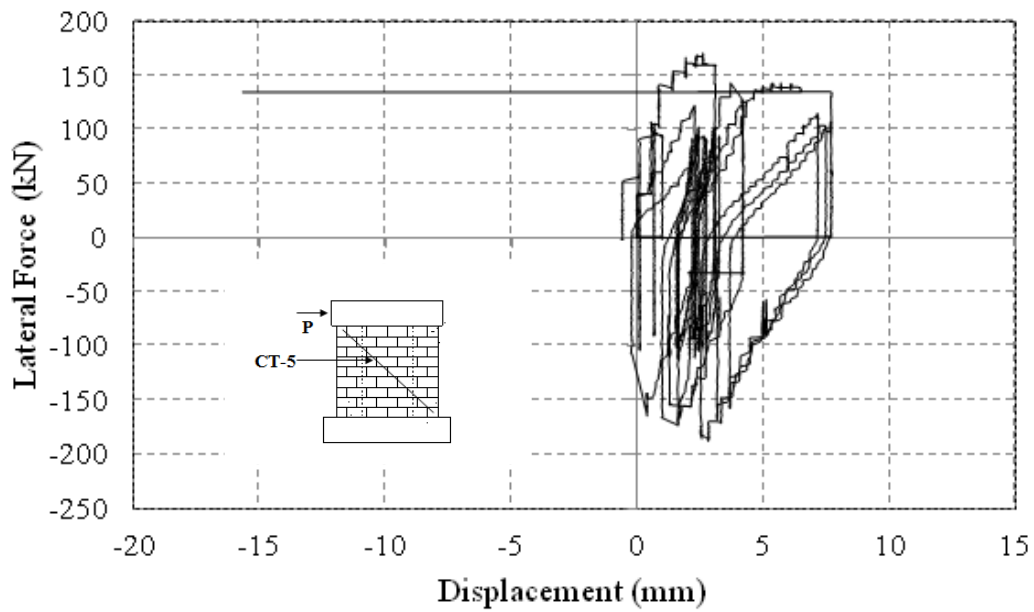


(b) Lateral force vs. wall uplift relative to foundation at west end

Figure 5.8: Uplift of PRM Wall Panel from Foundation



(a) Lateral force vs. diagonal deformation; lower west end to upper east end



(b) Lateral force vs. diagonal deformation; lower east end to upper west end

Figure 5.9: Deformations along wall diagonal; PRM Wall

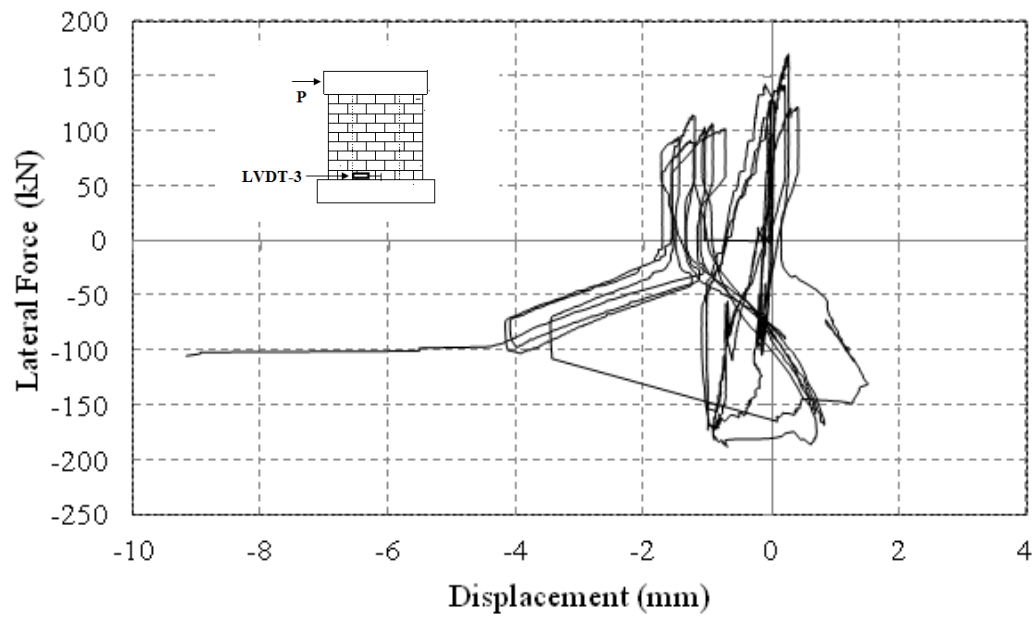


Figure 5.10: Sliding shear along wall-foundation joint; PRM Wall

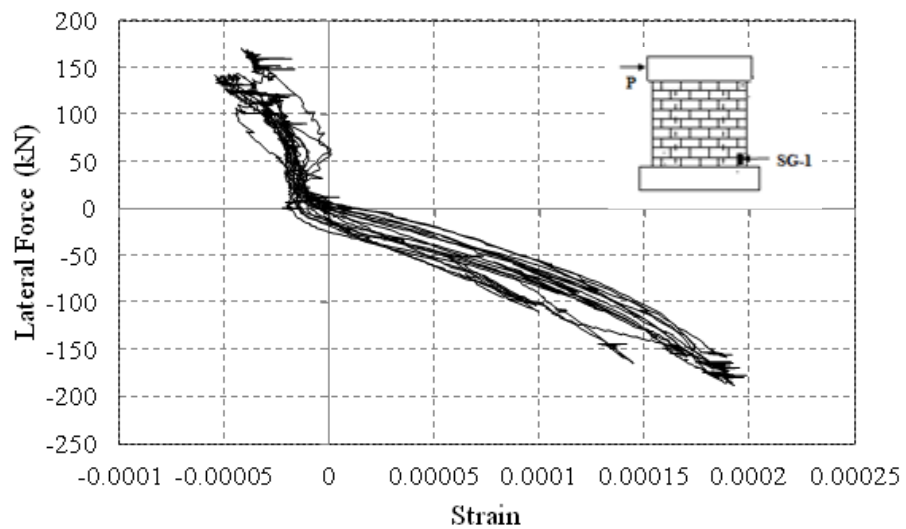


Figure 5.11: Lateral force vs longitudinal strain in wall FRP at the east end, near wall-foundation interface (PRM)



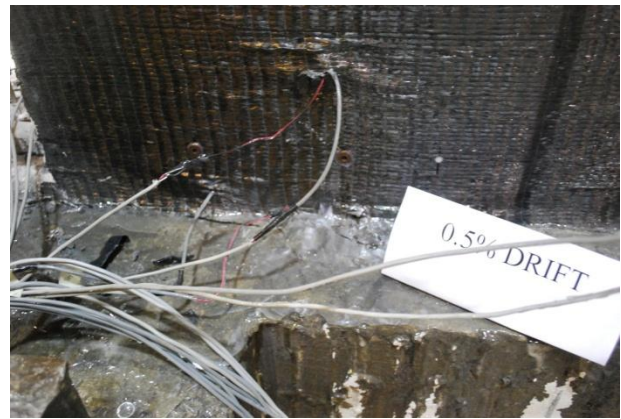
(a) Unretrofitted side of wall at 0.25% drift



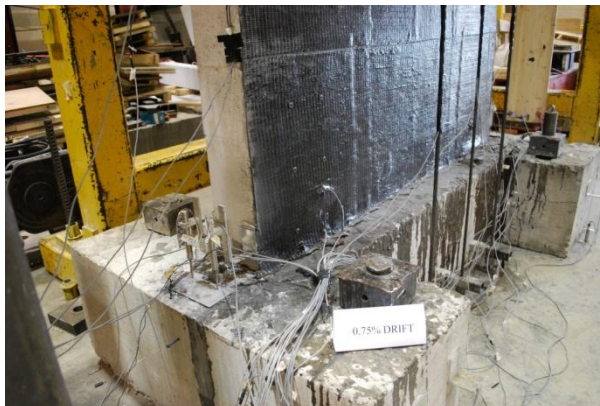
(b) Retrofitted side of wall at 0.25% drift



(c) Unretrofitted side of wall at 0.5% drift



(d) View of the anchor region at 0.5% drift



(e) Wall at 0.75% drift



(f) Lifting of wall east end at 0.75% drift

Figure 5.12: Performance of URM wall during 0.25% and 0.75% drift cycles



(a) Diagonal cracks at 1.0% drift



(b) Buckling of steel sheet anchor; 1.0% drift



(c) Diagonal cracks at 1.25% drift



(d) Separation of a block at 1.25% drift



(e) Buckling of steel sheet anchor; 1.25% drift



(f) Debonding of CFRP near the wall-footing interface; 1.25% drift

Figure 5.13: Performance of URM wall during 1.00% and 1.25% drift cycles



(a) Diagonal cracks at 1.5% drift



(b) Twisting of the wall at 1.5% drift



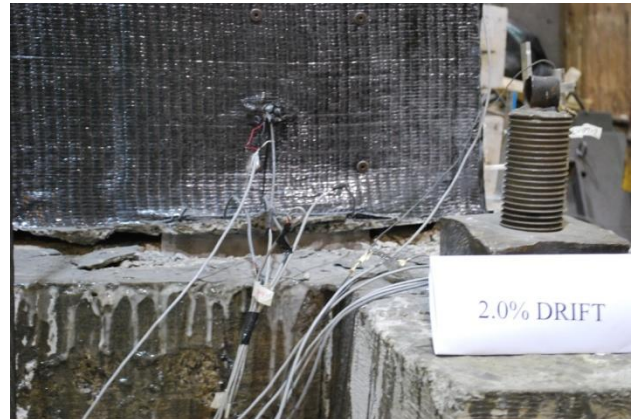
**(c) Twisting of wall at the east corner;
1.5% drift**



**(d) Extension in tension and buckling in
compression of steel sheet anchors; 1.5%
drift**



**(e) Close-up view of anchor buckling;
1.5% drift**

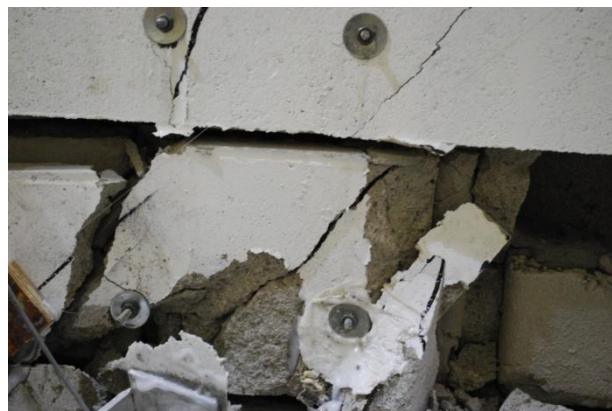


(f) Buckling of steel sheet anchors at 2% drift

Figure 5.14: Performance of URM wall during 1.5% and 2.0% drift cycles



(a) Wall damage at east end; 2.5% drift



(b) Close-up view, wall damage; 2.5% drift



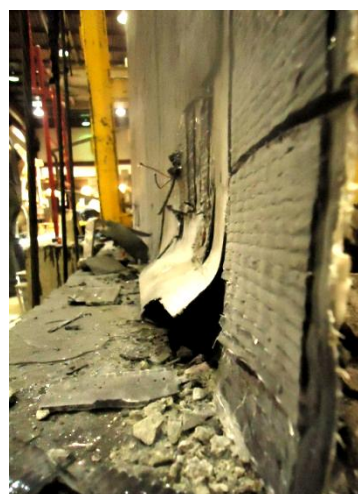
(c) Wall-foundation interface; 2.5% drift



(d) Wall-foundation interface; 3.0% drift



**(e) Buckled steel sheet anchor after test
(front view)**



**(f) Buckled steel sheet anchor after test
(side view)**

Figure 5.15: Performance of URM wall between 2.5% drift and end of test

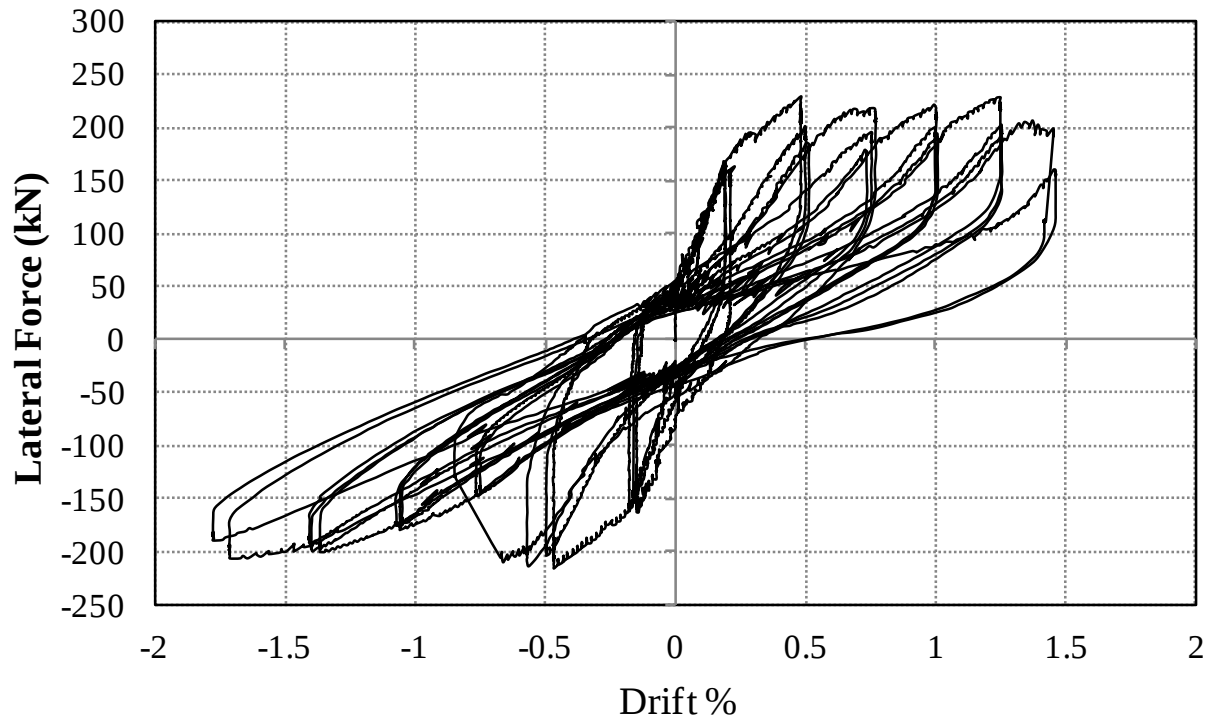


Figure 5.16: Hysteretic force-drift relationship of retrofitted URM wall

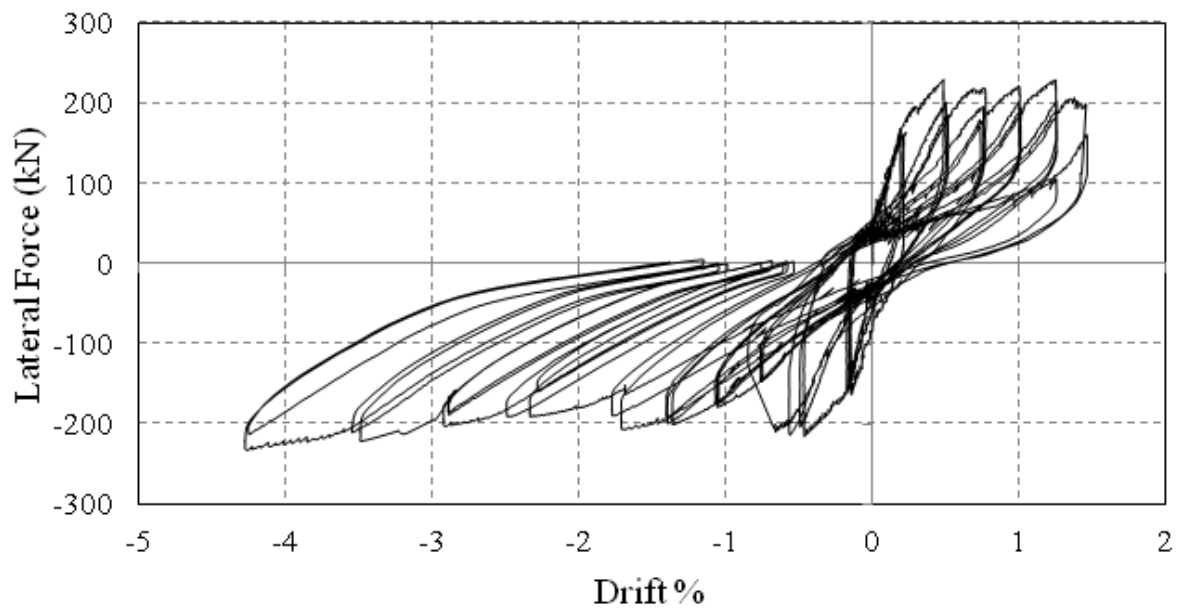
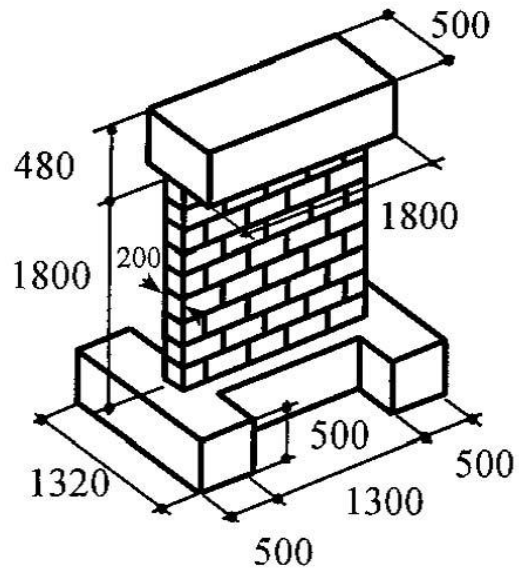
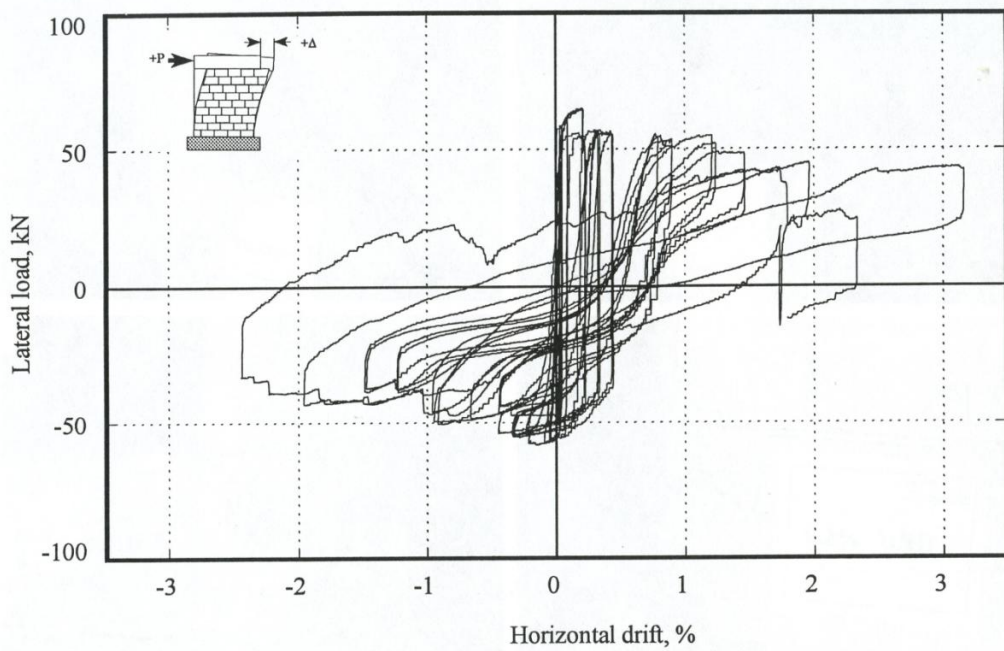


Figure 5.17: Hysteretic force displacement relationship of retrofitted URM wall

(With additional pull cycles)



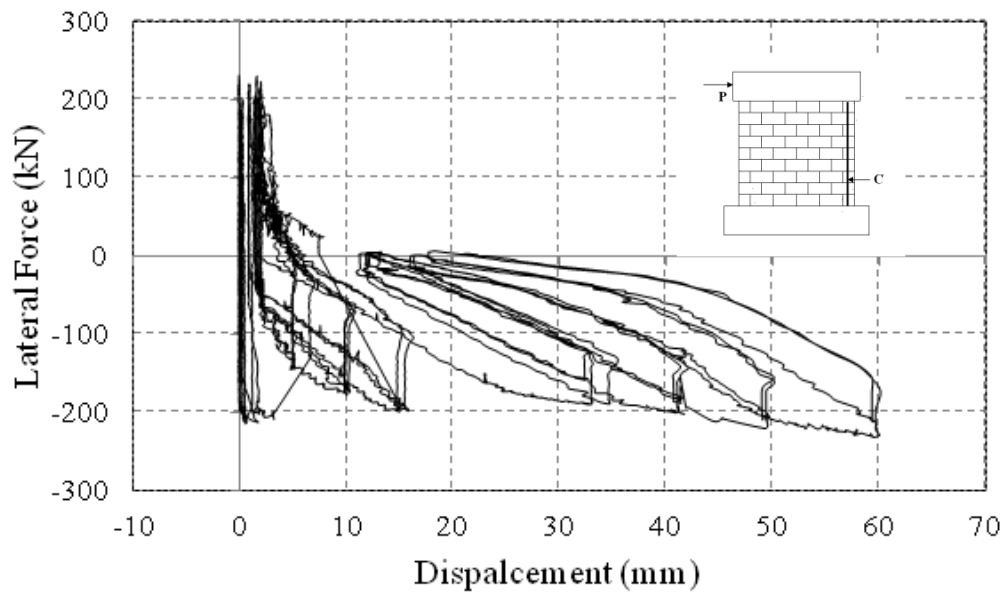
(a) Geometric details (in mm)



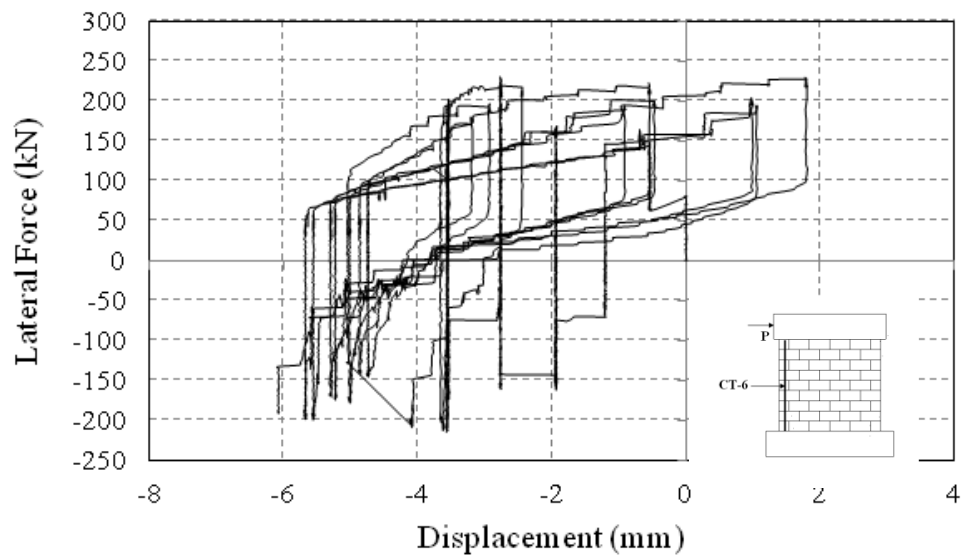
(b) Hysteretic force-drift relationship

Figure 5.18: Hysteretic force displacement relationship of unretrofitted URM wall

(Taghdi et al. 2000)

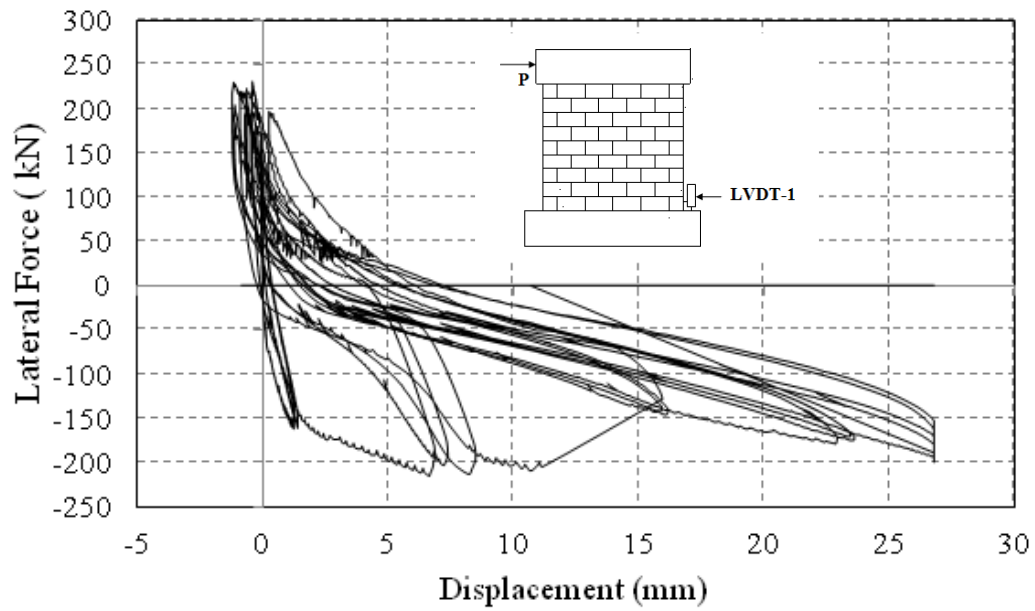


(a) Lateral force vs. vertical deflection of top beam relative to foundation at east end

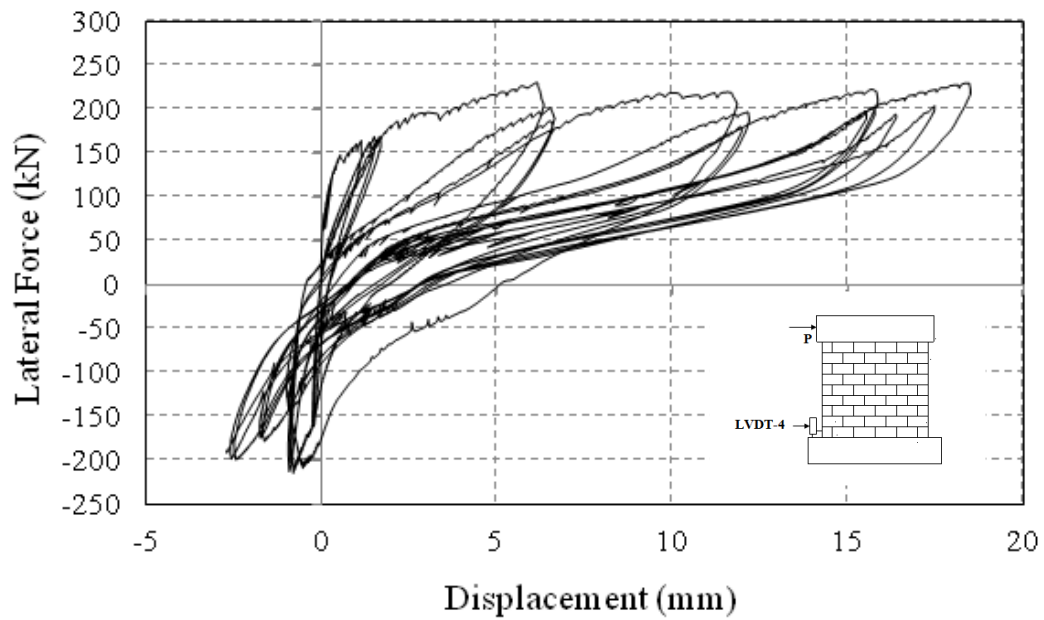


(a) Lateral force vs. vertical deflection of top beam relative to foundation at west end

Figure 5.19: Total vertical movement of URM wall

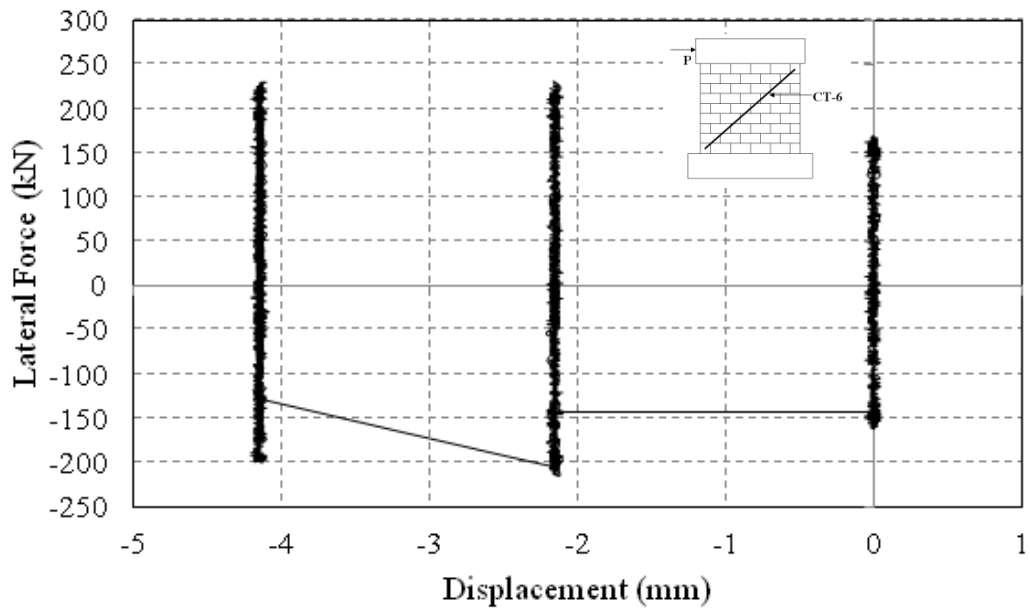


(b) Lateral force vs. wall uplift relative to foundation at west end

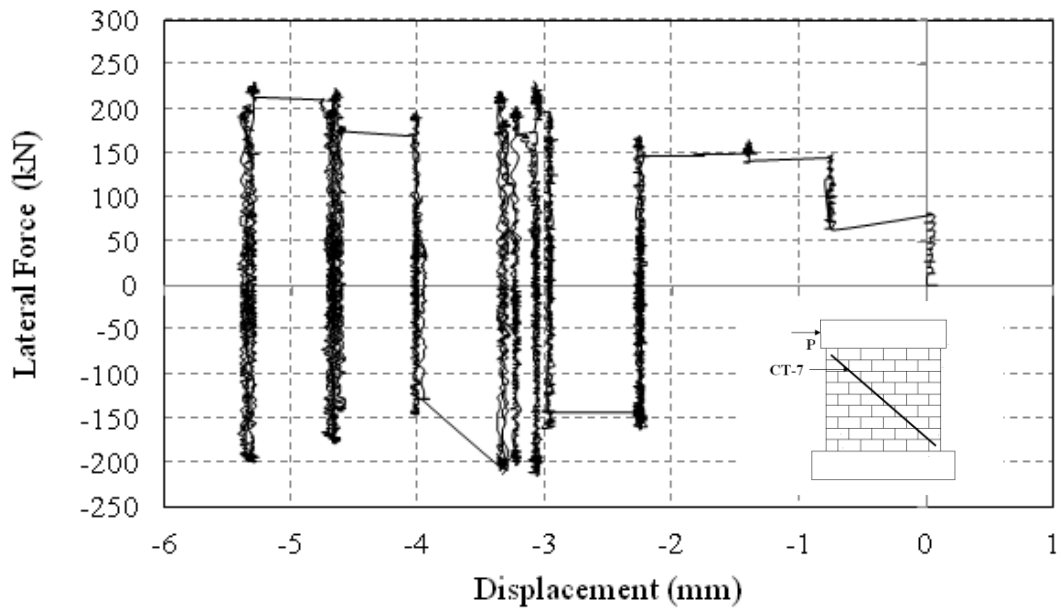


(b) Lateral force vs. wall uplift relative to foundation at west end

Figure 5.20: Uplift of URM wall panel from foundation



(a) Lateral force vs. diagonal deformation; lower west end to upper east end



(b) Lateral force vs. diagonal deformation; lower east end to upper west end

Figure 5.21: Deformations along wall diagonal; URM wall

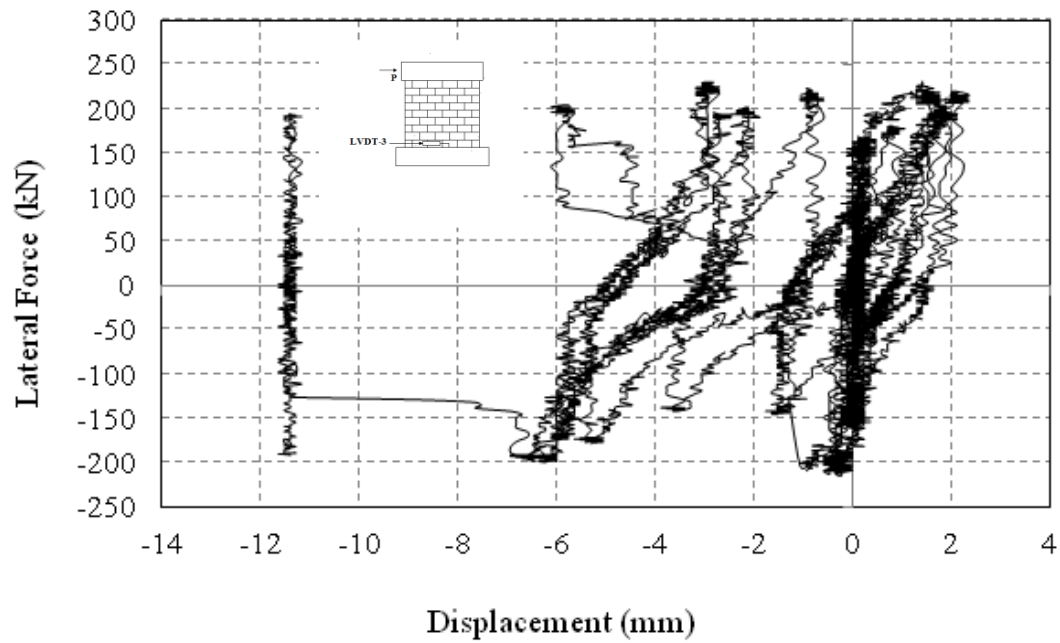


Figure 5.22: Sliding shear along wall-foundation joint; URM wall

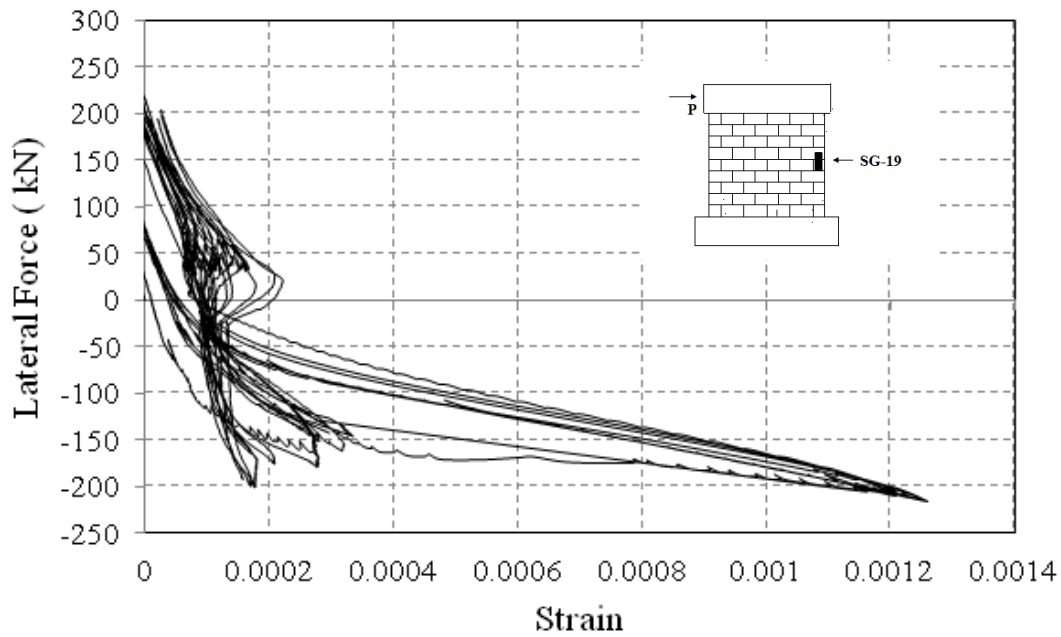


Figure 5.23: Lateral force vs longitudinal strain in wall FRP at the east end, mid height of wall (URM)

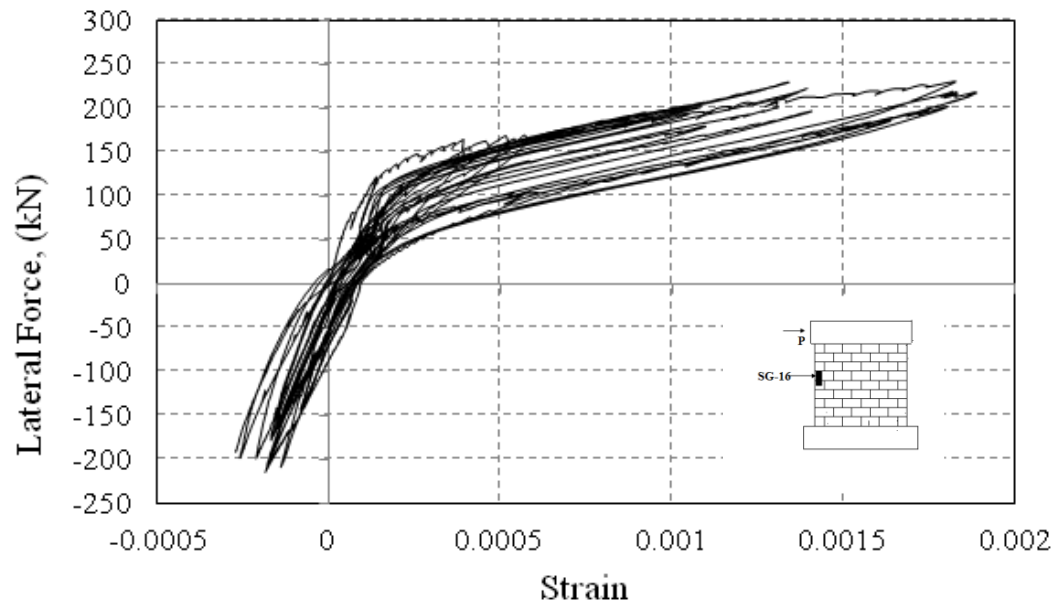


Figure 5.24: Lateral force vs longitudinal strain in wall FRP at the west end, mid-height of wall (URM)

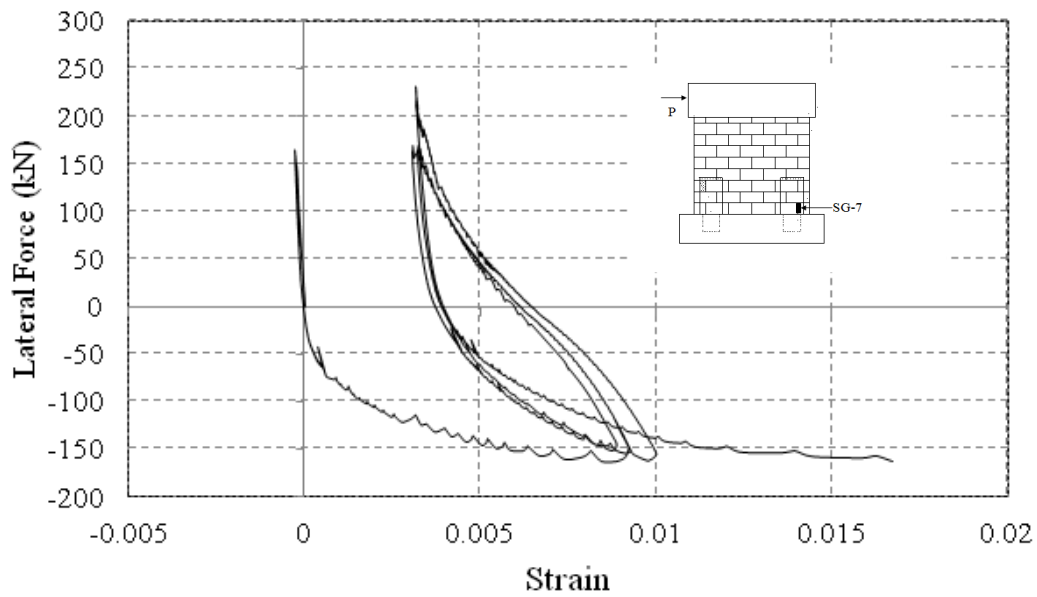


Figure 5.25: Lateral force vs strain in east-end stainless steel sheet anchor at wall-foundation interface (URM Wall)

Chapter 6

Analytical Research

6.1 Introduction

The analytical phase of the current research project consists of modelling and analysis of the two masonry walls tested using truss analogy. Non-linear structural analysis was conducted under incrementally increasing monotonic loading (push-over analysis) to establish the envelopes of hysteretic force-deformation relationships recorded during testing. The walls were modelled as trusses, where each element was modelled either as a strut or a tie. Non-linear material properties were assigned to members as obtained from material testing. Lateral load was increased incrementally with material properties updated at the end of each force increment to reflect material nonlinearity. Nodal deformations and internal member forces were calculated at the end of each load increment. The results are plotted in the form of non-linear force-deformation relationships and compared with experimental envelopes of hysteretic relationships.

The following sections provide an overview of the analysis method used in an Excel-based computer software, as well as sample analyses. The comparisons of analytically generated horizontal force-horizontal displacement relationships with those recorded during testing are presented.

6.2 Overview of the Analysis Method

A matrix method of structural analysis was employed, using the stiffness matrix approach to conduct truss analyses. This section provides an overview of the procedure followed. Accordingly, the following basic concepts of mechanics were employed:

Equilibrium: External loads, expressed in the form of a force matrix [P] at a joint are related to internal resisting forces [F] at the ends of members meeting at the joint. This is done by the static equilibrium matrix, [E].

$$[P] = [E] [F] \quad (6.1)$$

The resisting force matrix $[F]$ consists of axial forces in truss members, and moments and axial forces in moment-resisting frame elements. Because the truss analogy is employed in the current investigation, the matrix $[F]$ only consists of axial forces, either in tension or compression.

Compatibility: Joint displacements expressed as matrix $[D]$ are related to member-end displacements δ for members meeting at that joint. This is done through compatibility matrix, $[C]$.

$$[\delta] = [C] [D] \quad (6.2)$$

Load-Deformation Relationship: Internal forces $[F]$ and member end displacements $[\delta]$ are related through stiffness matrix $[S]$, which is a function of modulus elasticity (E), member length (L), and member cross-sectional area (A) and/or sectional moment of inertia (I) for moment resisting elements. This relationship is shown below:

$$[F] = [S] [\delta] \quad (6.3)$$

The stiffness matrix $[S]$ involves; AE/L for trusses, and EI/L for moment-resisting frames. The solution of any structural analysis problem requires the generation of $[P]$, $[E]$, $[C]$ and $[S]$ matrices.

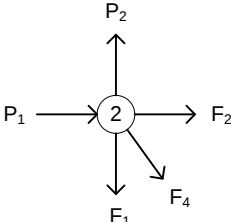
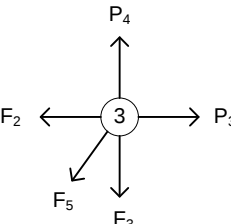
6.2.1 Generation of Required Matrices

The construction of $[P]$, $[E]$, $[C]$ and $[S]$ matrices, defined in Eqs. (6.1) through (6.3), is illustrated by following a sample truss analysis. The truss shown in Figure 6.1 is considered for this purpose. External forces and reactions acting on each joint are illustrated in Figure 6.2 as P and R , respectively. The number of members in the sample truss is 5 ($m = 5$), the number of joints is 4 ($j=4$) and the number of possible reaction forces is 4 ($r = 4$). The degree of indeterminacy (n), as the number of required conditions over and above those provided by the equilibrium conditions, can be computed as shown below:

$$n = m + r - 2j = 5 + 4 - 2(4) = 1 \text{ (indeterminate to 1st degree)} \quad (6.4)$$

The number of possible joint movements (degrees of freedom – DOF - horizontal and vertical displacements at joints 2 and 3) and the number of potential external joint forces (q) are each equal to 4. Because the structure is a truss, the loads can only be applied at the joints.

The equilibrium matrix [E] can be constructed by writing the equations of equilibrium at joints 2 and 3, as illustrated below. Figure 6.3 illustrates internal and external forces.

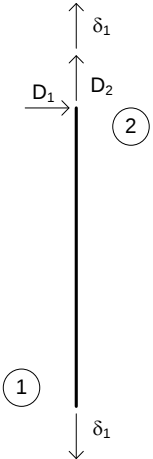
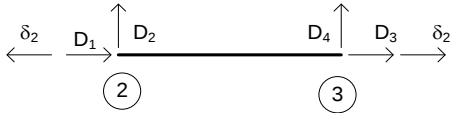
Joint 2:		$\Sigma F_x = 0 = P_1 + F_2 + 3/5 F_4$ $P_1 = -F_2 - 3/5 F_4$ $\Sigma F_y = 0 = P_2 - F_1 + 4/5 F_4$ $P_2 = F_1 - 4/5 F_4$
Joint 3:		$\Sigma F_x = 0 = P_3 - F_2 - 3/5 F_5$ $P_3 = F_2 + 3/5 F_5$ $\Sigma F_y = 0 = P_4 - F_3 - 4/5 F_5$ $P_4 = F_3 + 4/5 F_5$

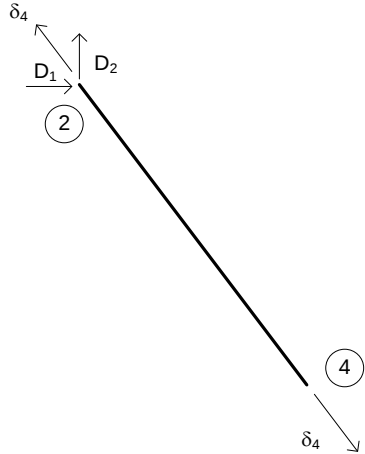
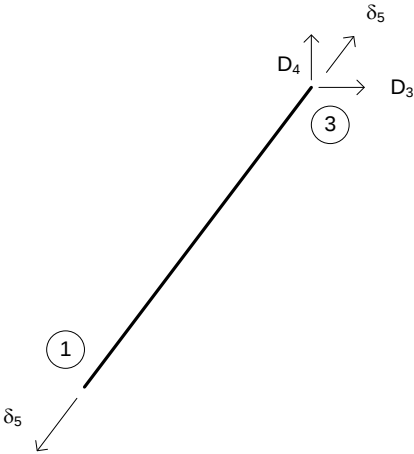
$$[P] = [E] [F] \quad (6.5)$$

$$(4 \times 1) \quad (4 \times 5) \quad (5 \times 1)$$

$$\begin{vmatrix} P_1 \\ P_2 \\ P_3 \\ P_4 \end{vmatrix} = \begin{vmatrix} 0 & -1 & 0 & -0.6 & 0 \\ 1 & 0 & 0 & 0.8 & 0 \\ 0 & 1 & 0 & 0 & 0.6 \\ 0 & 0 & 1 & 0 & 0.8 \end{vmatrix} \begin{vmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \\ F_5 \end{vmatrix} \quad (6.6)$$

The compatibility matrix $[C]$ can be established by dis-assembling the structure as shown below, and relating internal displacements to external displacement. External and internal deformations are shown in Figure 6.4.

Member 1		$\delta_1 = D_2$
Member 2		$\delta_2 = D_3 - D_1$
Member 3		$\delta_3 = D_4$

Member 4		$\delta_4 = -0.6D_1 + 0.8D_2$
Member 5		$\delta_5 = 0.6D_3 + 0.8D_4$

$$\begin{array}{ccc}
 [\delta] & = & [C] \quad [D] \\
 (5 \times 1) & & (5 \times 4) \quad (4 \times 1)
 \end{array} \tag{6.7}$$

$$\begin{array}{c}
 \delta_1 \\
 \delta_2 \\
 \delta_3 \\
 \delta_4 \\
 \delta_5
 \end{array}
 =
 \begin{array}{cccc}
 0 & 1 & 0 & 0 \\
 -1 & 0 & 1 & 0 \\
 0 & 0 & 0 & 1 \\
 -0.6 & 0.8 & 0 & 0 \\
 0 & 0 & 0.6 & 0.8
 \end{array}
 \begin{array}{c}
 D_1 \\
 D_2 \\
 D_3 \\
 D_4
 \end{array} \tag{6.8}$$

It can be observed from Eqs. (6.1) and (6.2) that the compatibility matrix is the transpose of the equilibrium matrix; $[C] = [E]^T$.

The member stiffness matrix $[S]$ can be formed by establishing the relationship between axial force and axial deformation. From Hooke's law:

$$\sigma = E \epsilon \quad (6.9)$$

$$\sigma = F / A \quad (6.10)$$

$$\epsilon = \delta / L \quad (6.11)$$

Therefore,

$$(F / A) = E (\delta / L) = S \delta \quad (6.12)$$

Or,

$$F = (E A / L) \delta = S \delta \quad (6.13)$$

For an individual member,

$$F_i = (E_i A_i / L_i) \delta_i = S_i \delta_i \quad (6.14)$$

Where, σ =Member stress, E =Modulus of elasticity, ϵ = Strain, δ = Internal Deformation, L = Length of the member, A = Cross sectional area of the member. The relationship between member forces and member deformation can then be written as:

$$[F] = [S] [\delta] \quad (6.15)$$

$$(5 \times 1) \quad (5 \times 5) \quad (5 \times 1)$$

$$\begin{vmatrix} F_1 \\ F_2 \\ F_3 \\ F_4 \\ F_5 \end{vmatrix} = \begin{vmatrix} E_1 A_1 / L_1 & 0 & 0 & 0 & 0 \\ 0 & E_2 A_2 / L_2 & 0 & 0 & 0 \\ 0 & 0 & E_3 A_3 / L_3 & 0 & 0 \\ 0 & 0 & 0 & E_4 A_4 / L_4 & 0 \\ 0 & 0 & 0 & 0 & E_5 A_5 / L_5 \end{vmatrix} \begin{vmatrix} \delta_1 \\ \delta_2 \\ \delta_3 \\ \delta_4 \\ \delta_5 \end{vmatrix} \quad (6.16)$$

6.2.2 Matrix operations to determine member forces and displacements

A series of matrix operations are performed to compute member forces and displacements. The member forces are then used to assess if the members remain elastic, develop inelasticity, or fail. Corresponding displacements at each step of analysis are computed to establish the force-deformation relationship. Substituting Eqs. (6.5) and (6.7) into Eq. (6.9);

$$[F] = [S] [C] [D] \quad (6.17)$$

Because, $[C] = [E]^T$ and $[F] = [S] [E]^T [D]$, substituting these matrices into Eq. (6.5) results;

$$[P] = [E] [S] [E]^T [D] \quad (6.18)$$

The global stiffness matrix for the entire structure can be defined as;

$$[S^*] = [E] [S] [E]^T \quad (6.19)$$

From Eqs. (6.18) and (6.19);

$$[P] = [S^*] [D] \quad (6.20)$$

The following external deformation values can be computed;

$$[D] = [S^*]^{-1} [P] \quad (6.21)$$

For internal forces, substituting (6.21) and $[C] = [E]^T$ into (6.17), one can obtain;

$$[F] = [S] [E]^T [S^*]^{-1} [P] \quad (6.22)$$

Internal deformations can be obtained from Eq. (6.15);

$$[\delta] = [F] [S]^{-1} \quad (6.23)$$

6.2.3 Excel-Based Computer Software for Non-Linear Analysis of Trusses

A truss analysis program was developed using Microsoft Excel spreadsheet software following the procedure described in the preceding sections. The software allows incremental load analysis under monotonically increasing lateral forces to conduct non-linear push-over analysis. The input consists of structural geometry and non-linear material properties. The nodal coordinates and

member data, including node-to-node connectivity and material properties are first specified as shown in Table 6.1. Coordinates of nodes are entered along with their degrees of freedom to identify supports or free standing nodes. Also, the loads in horizontal and vertical directions are entered. Members are “connected” between two nodes by the user, as shown in the Table 6.2. Member material properties, such as cross-sectional area and non-linear stress-strain relationships in tension and compression are entered as illustrated in Tables 6.3 and 6.4. Also, the initial modulus of elasticity for each member is entered. For each load increment, external node displacements, and internal member forces and deformations are computed. The current state of strains and corresponding stresses (under 70 kN of lateral force as indicated in Table 6.1) are displayed for each member, as shown in Table 6.5.

The strain is calculated as the ratio δ/L and the corresponding stress is calculated from the material stress-strain relationship. Meanwhile, the instantaneous modulus of elasticity, E , is calculated from the current stress-strain values (σ/ϵ if elastic). The value if E changes with material nonlinearity. The new value of instantaneous E is displayed (as shown in Table 6.6) to be used in the next step of analysis under the next increment of load. The program also displays the remaining strength in the member as illustrated in Table 6.7. The analysis continues until the “Next E to be used” and “ E used in calculation” match. Then, the external displacement value becomes the true value. The same iterative procedure is continued until one or more members fail or the structure becomes unstable, ie. no merging of modulus of elasticity is attained. The load and displacement values are recorded in the Global Response Table. The output is displayed graphically in terms of undeformed and deformed shapes of the structure. This is illustrated in Figure 6.5. The program also displays external and internal deformations, as well as member forces as shown in the Tables 6.8 to 6.10. Finally the equilibrium check and member stresses are displayed as shown in Tables 6.10 & 6.11.

6.3 Modeling and Analysis of Test Specimens

Push-over analysis, as described above, was conducted for each of the two retrofitted masonry walls. A load increment of 5 kN was used and the top horizontal displacement was computed at the end of each force increment. This provides lateral force-lateral displacement relationship for

each wall. These relationships are then compared with the envelopes of the hysteretic force-displacement relationships obtained experimentally.

The masonry wall was modeled as inclined struts. Taghdi et al. (2000b) performed a sensitivity analysis for masonry walls and investigated the effect of masonry strut width on the accuracy of results. The researchers suggested using a range of strut widths for inclined struts, changing between one-sixth and one-half of the horizontal strut length. It was also suggested to use one-tenth of the wall length as the width of the vertical strut. In the current study, one-sixth of the horizontal strut length was used as the width of inclined struts, and one-tenth of the horizontal wall length was used for the vertical strut. The strut thickness was taken equal to the wall thickness.

6.3.1 Retrofitted Partially Reinforced Masonry (PRM) Wall

The partially reinforced masonry wall (PRM), tested as part of the current research project, was modeled as shown in Figure 6.6. Non-linear material properties are presented in Table 6.13 and 6.14. Member assignments, cross-sectional areas and initial modulus of elasticity are presented in Table 6.15. Masonry struts, reinforcing bars, FRP laminat and the FRP tension anchors were transformed into truss elements with respective effective areas. Internal reinforcing bars (#15M) were modeled as vertical ties with the actual bar cross sectional area (200 mm^2). The vertical and diagonal tension ties were taken as an element representing the tensile characteristics of FRP anchors. There were four FRP anchors placed in the extreme tension zone, and an element representing all four anchors, placed at the centroid of the four anchors was used with FRP elastic modulus. Material properties were assigned from laboratory tests. Axial forces were applied as external forces on the nodes. Masonry tensile capacity was ignored.

The results of the retrofitted partially reinforced wall obtained from truss analysis are compared with the envelop curve of experimentally recorded hysteretic relationship in Figure 6.7. Table 6.16 and Table 6.17 list top horizontal displacement and all external and member forces at each step of analysis corresponding to incrementally increasing horizontal force applied at node 1. The ultimate capacity obtained from the analysis was 170 kN and the experimental capacity found from the test was 182 kN. The analytical stiffness obtained from the truss analysis showed excellent correlation within the elastic range of deformations until approximately 50 kN of

lateral force. Beyond this level of force resistance, the analytical results showed more rigid response, with smaller horizontal displacements than those recorded during testing. This difference is attributed to the fact that masonry cracking under monotonically increasing load was ignored in the analysis, whereas the wall during testing experienced softening of the masonry under reversed cyclic loading. Furthermore, the elastic modulus used for the FRP anchors was assumed to be the same as that obtained for surface-bonded FRP sheets in the direction of fibres. The anchors were manufactured by twisting the FRP sheets, with substantial inclination of fibres, which would reduce the elastic modulus of the material. This reduction in the elastic modulus of anchors was not accounted for in the analysis, potentially contributing to the relatively more rigid response obtained. The wall capacity was governed by the pull-out cone failure of FRP anchors in both the analysis and the test. This stage of loading corresponded to a total anchor force resistance of approximately 100 kN. The wall reached its maximum lateral load capacity at this stage and experienced strength and stiffness degradation in subsequent deformation reversals. The analysis was stopped at this stage, as the truss analysis was not intended to reflect behavior beyond the peak load. Table 6.18 shows stresses in truss members, indicating that the masonry struts were approaching the crushing value as the anchors pull-out in tension.

6.3.2 Retrofitted Unreinforced Masonry (URM) Wall

The retrofitted unreinforced masonry wall tested in the current research project was modeled as shown in the Figure 6.8. Nonlinear material properties are given in Table 6.19 and 6.20. Cross-sectional and material properties for the truss model are presented in Table 6.21. Masonry, top reinforced concrete beam, and steel sheet anchors were transformed into truss members and formed the truss model analyzed using the software described in this chapter. Steel anchors were modeled using the actual steel area used in the wall. Because the capacity of steel sheet anchors governed the capacity of the tension element (M8), the same anchor properties were assigned to the vertical tension element. The properties of compression struts were modeled in the same manner as they were done previously for the PRM wall. Axial forces were applied externally on specific nodes.

The analysis results for the retrofitted URM wall are presented in Figure 6.9, and Table 6.22 in the form of lateral force lateral displacement relationships. The ultimate capacity computed in the analysis was found to be 205 kN. The capacity recorded during testing was 225 kN. Tables 6.23 and 6.24 show the analysis results in terms of member forces and stresses. They indicate that the wall experienced the crushing of masonry at its toe when the maximum force resistance was attained. This is consistent with experimental observations. The computed response, however, was observed to be softer than what was measured during the experiment. The difference may be attributed to the inaccuracies associated with the rigidity of masonry struts used in the model. The strut width of $1/6^{\text{th}}$ the horizontal strut length used may underestimate the actual effective width, which may be as high as $1/2$ the horizontal strut length according to some previous researchers (Taghdi et al. 2000b). Furthermore, the stiffening effect of surface-bonded FRP on masonry in compression was assumed to be negligible, whereas this effect may increase the strut rigidity. As well, the analytical model ignored the presence of the steel sheet anchor in compression, though this element may also resist compression prior to buckling, and may contribute to the lateral load resistance significantly.

Table 6.1: Node assignment in the truss example

	Node Exist?	Coordinates		Degrees of Freedom		Loading	
Node No.	Y=1 N=0	X (mm)	Y (mm)	X (mm)	Y (mm)	Px (N)	Py (N)
N1	1	0	0	0	0	0	0
N2	1	0	300	1	1	70000	0
N3	1	300	300	1	1	0	0
N4	1	300	0	0	0	0	0

Table 6.2: Member assignment in the truss example.

	Member Exist?	Connectivity			
Member No.	Y=1 N=0	Start Node	End Node	Material	Area (mm^2)
M1	1	1	2	Steel	100
M2	1	2	3	Steel	100
M3	1	3	4	Steel	100
M4	1	2	4	Steel	100
M5	1	1	3	Steel	100

Table 6.3: Stress-strain relationship of the material in tension in the truss example.

	Strain (ϵ_y)	Stress (σ_y)	Strain (ϵ_{sh})	Stress (σ_{sh})	Strain (ϵ_u)	Stress (σ_u)
Member No.		MPa		MPa		MPa
M1	0.002	400	0.15	400	0.15	400
M2	0.002	400	0.15	400	0.15	400
M3	0.002	400	0.15	400	0.15	400
M4	0.002	400	0.15	400	0.15	400
M5	0.002	400	0.15	400	0.15	400

Table 6.4: Stress - strain relationship of the material in tension in the truss example.

	Strain (ε_y)	Stress (σ_y)	Strain (ε_{sh})	Stress (σ_{sh})	Strain (ε_u)	Stress (σ_u)
Member No.		MPa		MPa		MPa
M1	0.002	400	0.15	400	0.15	400
M2	0.002	400	0.15	400	0.15	400
M3	0.002	400	0.15	400	0.15	400
M4	0.002	400	0.15	400	0.15	400
M5	0.002	400	0.15	400	0.15	400

Table 6.5: Current state of stress shown in the program for the truss example

Current State (Tension+Compression)			
Member No.	ε	σ	Status
M1	0.195%	390	Resisting
M2	-0.155%	-310	Resisting
M3	-0.155%	-310	Resisting
M4	-0.276%	-400	Crushed in Compression
M5	0.219%	400	Yielded in Tension

Table 6.6: Initial and instantaneous modulus of elasticity

Member No.	E used in calculation	Use this E
M1	200000	200000
M2	200000	200000
M3	200000	200000
M4	200000	144888
M5	200000	182733

Table 6.7: Stresses left in each member

Member No.	Tension (MPa)	Compression (MPa)
M1	10	---
M2	---	-90
M3	---	-90
M4	---	0
M5	0	---

Table 6.8: External deformation at each node

Node No.		Deformation (mm)
2	DOFx-2	2.242
2	DOFy-2	0.586
3	DOFx-3	1.778
3	DOFy-3	-0.464

Table 6.9: Internal deformation in each member

Member No.	Deformation (mm)
M1	0.586
M2	-0.464
M3	-0.464
M4	-1.171
M5	0.586

Table 6.10: Internal forces in each member

Member No.	Force (N)	State
M1	39043	T
M2	-30956	C
M3	-30956	C
M4	-55215	C
M5	43779	T

Table 6.11: Reactions and equilibrium check

Node No.	Reaction & DOF	Reaction (N)			
N1	Rx-1	-30956	$\Sigma F_x =$	0.000	OK
	Ry-1	-70000	$\Sigma F_y =$	0.000	OK
N2	DOFx-2	70000			
	DOFy-2	0.000			
N3	DOFx-3	0.000			
	DOFy-3	0.000			
N4	Rx-4	-39043.039			
	Ry-4	70000.000			

Table 6.12: Member stresses

Member	Stress (MPa)	Designation
M1	390.430	T
M2	-309.570	C
M3	-309.570	C
M4	-552.152	C
M5	437.798	T

Table 6.13: Stress -strain relationship of materials in tension for the PRM wall model

Material	ε_y	σ_y	ε_{sh}	σ_{sh}	ε_u	σ_u
	(mm/mm)	(Mpa)	(mm/mm)	(MPa)	mm/mm	(MPa)
Steel	0.002	400	0.15	400	0.15	400
FRP	0.012	780	0.012	780	0.012	780
Masonry	0.00006	0.8	0.00006	0.8	0.00006	0.8
Concrete	0.0001	2.86	0.0001	2.86	0.0001	2.86

Table 6.14: Stress -strain relationship of materials in compression for the PRM wall model

Material	ε_y	σ_y	ε_{sh}	σ_{sh}	ε_u	σ_u
	(mm/mm)	(Mpa)	(mm/mm)	(MPa)	mm/mm	(MPa)
Steel	0.002	400	0.15	400	0.15	400
Masonry	0.0006	8	0.0006	8	0.0006	8
Concrete	0.001	28.6	0.001	28.6	0.001	28.6

Table 6.15: Model input for the retrofitted PRM wall

Member	Material	Cross-sectional Area (mm ²)	Initial Modulus of Elasticity (MPa)
1	Concrete	200000	21400
2	Concrete	200000	21400
3	Concrete	200000	21400
4	Concrete	200000	21400
5	Concrete	200000	21400
6	Concrete	200000	21400
7	FRP	793	70000
8	Steel	200	200000
9	Steel	200	200000
10	Masonry	40000	12500
11	Masonry	22620	12500
12	Masonry	17593	12500
13	Masonry	16022	12500
14	Masonry	11310	12500
15	Masonry	6598	12500
16	Masonry	5026	12500
17	FRP	200	70000

Table 6.16 Model output for the PRM Wall

Load Step	Lateral Force (kN)	Top Displacement (mm)	Member Condition
1	0	0	
2	5	0	
3	10	0	
4	15	0	
5	20	0	
6	25	0	
7	30	0	
8	35	0	
9	40	0	
10	45	0	
11	50	0.06	
12	55	0.27	
13	60	0.49	
14	65	0.7	
15	70	0.92	
16	75	1.13	
17	80	1.35	
18	85	1.56	
19	90	1.78	
20	95	1.99	

21	100	2.21	
22	105	2.43	
23	110	2.64	
24	115	2.86	
25	120	3.07	
26	125	3.29	
27	130	3.5	
28	135	3.72	
29	140	3.93	
30	145	4.15	
31	150	4.36	
32	155	4.58	
33	160	4.79	
34	165	5.01	
35	170	5.226	Anchors pulled Out

Table 6.17: Forces in the Members of PRM Truss model

F	M7	M8	M9	M10	M11	M12	M13	M14	M15	M16	M17
(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
5	-31.64	-13.95	-4.48	3.81	42.57	-45.06	-36.19	-33.08	17.03	4.57	-6.48
10	-27.99	-12.35	-3.97	3.26	37.67	-45.06	-36.19	-33.08	15.07	4.05	-5.74
15	-24.35	-10.74	-3.45	2.714	32.76	-45.06	-36.19	-33.08	13.11	3.52	-4.99
20	-20.71	-9.14	-2.94	2.16	27.86	-45.06	-36.19	-33.08	11.16	3	-4.25
25	-17.06	-7.53	-2.43	1.61	22.96	-45.06	-36.19	-33.08	9.2	2.47	-3.51
30	-13.42	-5.94	-1.91	1.06	18.05	-45.06	-36.19	-33.08	7.25	1.95	-2.77
35	-9.78	-4.33	-1.40	.511	13.15	-45.06	-36.19	-33.08	5.29	1.43	-2.03
40	-6.13	-2.73	-0.89	-.038	8.25	-45.06	-36.19	-33.08	3.33	0.9	-1.29
45	-2.49	-1.13	-.37	-0.59	3.35	-45.06	-36.19	-33.08	1.38	0.38	-0.55
50	1.15	0.47	0.14	-1.14	-1.55	-45.06	-36.19	-33.08	-0.58	-0.14	0.19
55	4.79	2.06	0.65	-1.7	-6.45	-45.06	-36.19	-33.08	-2.53	-.67	.93
60	8.44	3.68	1.17	-2.24	-11.39	-45.06	-36.19	-33.08	-4.49	-1.19	1.67
65	12.08	5.28	1.68	-2.79	-16.26	-45.06	-36.19	-33.08	-6.45	-1.72	2.41
70	15.73	6.88	2.19	-3.34	-21.16	-45.06	-36.19	-33.08	-8.4	-2.24	3.15
75	19.37	8.49	2.71	-3.89	-26.078	-45.06	-36.19	-33.08	-10.36	-2.77	3.89
80	23.02	10.09	3.23	-4.44	-30.97	-45.06	-36.19	-33.08	-12.32	-3.29	4.63
85	26.66	11.69	3.74	-4.99	-35.87	-45.06	-36.19	-33.08	-14.27	-3.81	5.37
90	30.3	13.29	4.25	-5.54	-40.77	-45.06	-36.19	-33.08	-16.23	-4.24	6.11
95	32.95	14.89	4.77	-6.09	-45.68	-45.06	-36.19	-33.08	-18.19	-4.86	6.85
100	37.59	16.5	5.28	-6.65	-50.58	-45.06	-36.19	-33.08	-20.14	-5.39	7.60
105	41.24	18.10	5.79	-7.20	-55.48	-45.06	-36.19	-33.08	-22.09	-5.91	8.34
110	44.88	19.71	6.31	-7.75	-60.38	-45.06	-36.19	-33.08	-24.06	-6.44	9.08
115	48.53	21.31	6.83	-8.30	-65.29	-45.06	-36.19	-33.08	-26.01	-6.96	9.82
120	52.17	22.91	7.34	-8.85	-70.19	-45.06	-36.19	-33.08	-27.97	-7.48	10.56

125	55.82	24.52	7.85	-9.4	-75.09	-45.06	-36.19	-33.08	-29.92	-8	11.30
130	59.46	26.12	8.37	-9.95	-79.99	-45.06	-36.19	-33.08	-31.88	-8.53	12.04
135	63.1	27.72	8.88	-10.5	-84.89	-45.06	-36.19	-33.08	-33.84	-9.06	12.78
140	66.75	29.32	9.39	-	-89.8	-45.06	-36.19	-33.08	-35.79	-9.58	13.52
145	70.39	30.93	9.9	-	-94.7	-45.06	-36.19	-33.08	-37.75	-10.11	14.26
150	74.04	32.53	10.42	-	-99.61	-45.06	-36.19	-33.08	-39.71	-10.63	15
155	77.68	34.13	10.94	-12.7	-104.51	-45.06	-36.19	-33.08	-41.66	-11.15	15.75
160	81.32	35.73	11.45	-	-109.41	-45.06	-36.19	-33.08	-43.62	-11.68	16.48
165	84.97	37.34	11.97	-13.8	-114.31	-45.06	-36.19	-33.08	-45.57	-12.2	17.23
170	88.61	38.94	12.48	-	-119.21	-45.06	-36.19	-33.08	-47.53	-12.73	17.97

Table 6.18: Stresses in The Members of PRM Truss Model

F	M7	M8	M9	M10	M11	M12	M13	M14	M15	M16	M17
(kN)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
5	-39.9	-69.75	-22.43	0.09	1.88	-2.56	-2.26	-2.92	2.58	0.91	-32.39
10	-35.31	-61.74	-19.86	0.08	1.67	-2.56	-2.26	-2.92	2.28	0.81	-28.69
15	-30.71	-53.73	-17.29	0.07	1.45	-2.56	-2.26	-2.92	1.99	0.7	-24.99
20	-26.12	-45.71	-14.72	0.05	1.23	-2.56	-2.26	-2.92	1.69	0.6	-21.28
25	-21.52	-37.69	-12.14	0.04	1.02	-2.56	-2.26	-2.92	1.39	0.49	-17.58
30	-16.93	-29.69	-9.58	0.03	0.79	-2.56	-2.26	-2.92	1.09	0.39	-13.87
35	-12.33	-21.67	-7	0.01	0.58	-2.56	-2.26	-2.92	0.8	0.28	-10.17
40	-7.74	-13.66	-4.43	0	0.37	-2.56	-2.26	-2.92	0.51	0.18	-6.47
45	-3.14	-5.64	-1.86	-0.01	0.15	-2.56	-2.26	-2.92	0.21	0.08	-2.76
50	-1.46	2.37	0.71	-0.03	-0.07	-2.56	-2.26	-2.92	-0.09	-0.03	0.94
55	6.05	10.38	3.28	-0.04	-0.28	-2.56	-2.26	-2.92	-0.38	-0.13	4.64
60	10.65	18.4	5.85	-0.06	-0.5	-2.56	-2.26	-2.92	-0.68	-0.24	8.35

65	15.24	26.41	8.42	-0.07	-0.72	-2.56	-2.26	-2.92	-0.98	-0.34	12.05
70	19.84	34.43	10.99	-0.08	-0.94	-2.56	-2.26	-2.92	-1.17	-0.05	15.76
75	24.43	42.44	13.56	-0.09	-1.15	-2.56	-2.26	-2.92	-1.57	-0.55	19.46
80	29.03	50.45	16.13	-0.11	-1.37	-2.56	-2.26	-2.92	-1.88	-0.66	23.17
85	33.62	58.47	18.7	-0.13	-1.57	-2.56	-2.26	-2.92	-2.16	-0.76	26.87
90	38.22	66.48	21.27	-0.14	-1.8	-2.56	-2.26	-2.92	-2.46	-0.86	30.57
95	42.81	74.49	23.84	-0.15	-2.02	-2.56	-2.26	-2.92	-2.76	-0.97	34.28
100	47.41	82.51	26.41	-0.17	-2.24	-2.56	-2.26	-2.92	-3.05	-1.07	37.98
105	52	90.52	28.99	-0.18	-2.45	-2.56	-2.26	-2.92	-3.35	-1.18	41.67
110	56.6	98.53	31.56	-0.19	-2.67	-2.56	-2.26	-2.92	-3.65	-1.28	45.39
115	61.19	106.55	34.13	-0.21	-2.89	-2.56	-2.26	-2.92	-3.94	-1.39	49.09
120	65.79	114.56	36.69	-0.22	-3.1	-2.56	-2.26	-2.92	-4.24	-1.49	52.79
125	70.39	122.58	39.27	-0.24	-3.32	-2.56	-2.26	-2.92	-4.54	-1.59	56.5
130	74.98	130.59	41.84	-0.25	-3.54	-2.56	-2.26	-2.92	-4.83	-1.7	60.21
135	79.58	138.6	44.41	-0.26	-3.75	-2.56	-2.26	-2.92	-5.13	-1.8	63.91
140	84.17	146.62	46.98	-0.28	-3.97	-2.56	-2.26	-2.92	-5.43	-1.91	67.61
145	88.77	154.63	49.55	-0.29	-4.19	-2.56	-2.26	-2.92	-5.72	-2.01	71.32
150	93.36	162.64	52.12	-0.31	-4.4	-2.56	-2.26	-2.92	-6.02	-2.115	75.021
155	97.96	170.66	54.69	-0.32	-4.6	-2.56	-2.26	-2.92	-6.31	-2.22	78.72
160	102.55	178.67	57.26	-0.33	-4.84	-2.56	-2.26	-2.92	-6.61	-2.32	82.43
165	107.15	186.68	59.83	-0.35	-5.05	-2.56	-2.26	-2.92	-6.91	-2.43	86.13
170	111.74	194.69	62.4	-0.36	-5.27	-2.56	-2.26	-2.92	-7.2	-2.53	89.84
175	116.34	202.72	64.97	-0.37	-5.49	-2.56	-2.26	-2.92	-7.5	-2.64	93.54

Table 6.19: Stress -strain relationship of materials in tension for the URM wall model

Material	ε_y	σ_y	e_{sh}	σ_{sh}	ε_u	σ_u
	(mm/mm)	(Mpa)	(mm/mm)	(Mpa)	mm/mm	(Mpa)
Stainless Steel	0.002	300	0.15	550	0.15	660
Masonry	0.00006	0.8	0.00006	0.8	0.00006	0.8
Concrete	0.00016	4.1	0.00016	4.1	0.00016	4.1

Table 6.20: Stress -strain relationship of materials in compression for the URM wall model

Material	ε_y	σ_y	e_{sh}	σ_{sh}	ε_u	σ_u
	(mm/mm)	(Mpa)	(mm/mm)	(Mpa)	mm/mm	(Mpa)
Stainless Steel	0.002	300	0.15	550	0.6	660
Masonry	0.0006	8	0.0006	8	0.0006	8
Concrete	0.0016	41	0.0016	41	0.0016	41

Table 6.21: Model input for the retrofitted URM wall

Member	Material	Cross-sectional Area (mm ²)	Initial Modulus of Elasticity (MPa)
1	Concrete	200000	25000
2	Concrete	200000	25000
3	Concrete	200000	25000
4	Concrete	200000	25000
5	Concrete	200000	25000
6	Stainless Steel	270	12500
7	Stainless Steel	120	150000
8	Stainless Steel	360	150000
9	Masonry	40000	12500
10	Masonry	21363	12500

11	Masonry	17279	12500
12	Masonry	12566	12500
13	Masonry	7854	12500
14	Masonry	3770	12500

Table 6.22: Model output for the URM wall

Load Step	Lateral Force (kN)	Top Displacement (mm)	Member Condition
1	0	0	
2	5	0	
3	10	0	
4	15	0	
5	20	0	
6	25	0	
7	30	0	
8	35	0	
9	40	0	
10	45	0	
11	50	0	
12	55	0	
13	60	0	
14	65	0	
15	70	0	
16	75	0	
17	80	0.29	
18	85	0.66	
19	90	1.01	
20	95	1.37	
21	100	1.73	
22	105	2.08	
23	110	2.44	

24	115	2.8	
25	120	3.15	
26	125	3.51	
27	130	3.87	
28	135	4.22	
29	140	4.58	
30	145	4.94	
31	150	5.29	
32	155	5.65	
33	160	6	
34	165	6.39	Member-6 Yielded
35	170	6.99	
36	175	7.82	
37	180	8.99	Member-7 Yielded
38	185	10.78	
39	190	13.37	
40	195	17.14	M-14 Crushed
41	200	22.49	
42	205	29.88	

Table 6.23: Forces in the members of URM truss model

F	M6	M7	M8	M9	M10	M11	M12	M13	M14
(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)	(kN)
5	-64.76	-22.1	-9.36	15.84	84.96	-48.4	-35.78	-35.62	9.47
10	-60.16	-20.55	-8.71	14.66	78.95	-48.4	-35.78	-35.62	8.8
15	-55.58	-18.99	-8.04	13.47	72.95	-48.4	-35.78	-35.62	8.13
20	-51	-17.43	-7.38	12.28	66.94	-48.4	-35.78	-35.62	7.46
25	-46.42	-15.87	-6.72	11.09	60.93	-48.4	-35.78	-35.62	6.79
30	-41.85	-14.31	-6.06	9.9	54.92	-48.4	-35.78	-35.62	6.12
35	-37.27	-12.75	-5.39	8.71	48.92	-48.4	-35.78	-35.62	5.46

40	-32.69	-11.19	-4.74	7.53	42.91	-48.4	-35.78	-35.62	4.79
45	-28.11	-9.63	-4.07	6.33	36.9	-48.4	-35.78	-35.62	4.12
50	-23.54	-8.07	-3.41	5.15	30.89	-48.4	-35.78	-35.62	3.45
55	-18.96	-6.51	-2.75	3.96	24.89	-48.4	-35.78	-35.62	2.78
60	-14.38	-4.95	-2.09	2.77	18.88	-48.4	-35.78	-35.62	2.11
65	-9.81	-3.39	-1.43	1.58	12.87	-48.4	-35.78	-35.62	1.44
70	-5.23	-1.8	-0.76	0.396	6.87	-48.4	-35.78	-35.62	0.77
75	-0.65	-0.27	-0.11	-0.79	0.86	-48.4	-35.78	-35.62	0.11
80	3.92	1.28	0.55	-1.98	-5.14	-48.4	-35.78	-35.62	-0.56
85	8.49	2.84	1.21	-3.17	-11.15	-48.4	-35.78	-35.62	-1.22
90	13.07	4.41	1.87	-4.36	-17.16	-48.4	-35.78	-35.62	-1.89
95	17.65	5.96	2.54	-5.54	-23.17	-48.4	-35.78	-35.62	-2.56
100	22.23	7.52	3.2	-6.7	-29.17	-48.4	-35.78	-35.62	-3.23
105	26.8	9.08	3.86	-7.92	-35.18	-48.4	-35.78	-35.62	-3.91
110	31.38	10.64	4.52	-9.11	-41.19	-48.4	-35.78	-35.62	-4.57
115	35.96	12.2	5.18	-10.29	-47.19	-48.4	-35.78	-35.62	-5.24
120	40.54	13.76	5.85	-11.49	-53.2	-48.4	-35.78	-35.62	-5.91
125	45.11	15.32	6.5	-12.67	-59.21	-48.4	-35.78	-35.62	-6.5
130	49.69	16.88	7.16	-13.84	-65.21	-48.4	-35.78	-35.62	-7.25
135	54.27	18.44	7.83	-15.05	-71.22	-48.4	-35.78	-35.62	-7.92
140	58.84	20.03	8.49	-16.24	-77.23	-48.4	-35.78	-35.62	-8.58
145	63.42	21.56	9.15	-17.43	-83.24	-48.4	-35.78	-35.62	-9.25
150	68	23.12	9.81	-18.61	-89.25	-48.4	-35.78	-35.62	-9.92
155	72.57	24.68	10.47	-19.8	-95.25	-48.4	-35.78	-35.62	-10.59
160	77.15	26.24	11.13	-20.99	-101.26	-48.4	-35.78	-35.62	-11.26
165	81.61	27.94	11.86	-22.29	-107.11	-48.4	-35.78	-35.62	-11.99
170	85.29	30.57	12.97	-24.29	-111.94	-48.4	-35.78	-35.62	-13.12
175	88.18	34.15	14.49	-27.01	-115.74	-48.4	-35.78	-35.62	-14.65

180	91.75	36.67	16.68	-28.94	-120.43	-48.4	-35.78	-35.62	-16.86
185	96.36	37.59	19.97	-29.64	-126.46	-48.4	-35.78	-35.62	-20.2
190	100.95	38.16	24.8	-30.08	-132.49	-48.4	-35.78	-35.62	-25.07
195	105.21	38.69	31.77	-30.47	-138.07	-48.4	-35.78	-35.62	-32.13
200	108.93	39.22	41.68	-30.88	-142.96	-48.4	-35.78	-35.62	-42.15
205	111.95	39.8	55.38	-31.32	-146.93	-48.4	-35.78	-35.62	-56

Table 6.24: Stress in the members of URM truss model

F	M6	M7	M8	M9	M10	M11	M12	M13	M14
(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)	(MPa)
5	-239.76	- 184.24	-26.02	0.4	3.98	-4.92	-2.85	-4.54	2.51
10	-222.81	- 171.25	-24.18	0.37	3.7	-4.92	-2.85	-4.54	2.34
15	-205.86	- 158.25	-22.3	0.34	3.41	-4.92	-2.85	-4.54	2.16
20	-188.91	- 145.25	-20.5	0.31	3.13	-4.92	-2.85	-4.54	1.98
25	-171.95	- 132.25	-18.67	0.28	2.85	-4.92	-2.85	-4.54	1.8
30	-155	- 119.26	-16.83	0.25	2.57	-4.92	-2.85	-4.54	1.63
35	-138.05	- 106.26	-15	0.22	2.29	-4.92	-2.85	-4.54	1.45
40	-121.1	-93.26	-13.16	0.19	2.01	-4.92	-2.85	-4.54	1.27
45	-104.14	-80.26	-11.32	0.16	1.73	-4.92	-2.85	-4.54	1.09
50	-87.19	-67.26	-9.48	0.13	1.45	-4.92	-2.85	-4.54	0.92
55	-70.24	-54.27	-7.65	0.1	1.17	-4.92	-2.85	-4.54	0.74
60	-53.29	-41.27	-5.81	0.07	0.88	-4.92	-2.85	-4.54	0.56
65	-36.33	-28.27	-3.97	0.04	0.6	-4.92	-2.85	-4.54	0.38
70	-19.38	-15.27	-2.13	0.01	0.32	-4.92	-2.85	-4.54	0.21
75	-2.43	-2.28	-0.3	-0.02	0.04	-4.92	-2.85	-4.54	0.03
80	14.52	10.72	1.54	-0.05	-0.24	-4.92	-2.85	-4.54	-0.15
85	31.48	-0.33	3.38	-0.08	-0.52	-4.92	-2.85	-4.54	-4.54

90	48.43	36.72	5.22	-0.11	-0.8	-4.92	-2.85	-4.54	-0.5
95	65.38	49.72	7.05	-0.14	-1.08	-4.92	-2.85	-4.54	-0.68
100	82.34	62.71	8.89	-0.17	-1.37	-4.92	-2.85	-4.54	-0.86
105	99.29	75.71	10.73	-0.2	-1.65	-4.92	-2.85	-4.54	-1.04
110	116.24	88.71	12.57	-0.23	-1.93	-4.92	-2.85	-4.54	-1.21
115	133.19	101.71	14.4	-0.26	-2.21	-4.92	-2.85	-4.54	-1.39
120	150.15	114.71	16.24	-0.29	-2.49	-4.92	-2.85	-4.54	-1.57
125	167.1	127.7	18.08	-0.32	-2.77	-4.92	-2.85	-4.54	-1.75
130	184.05	140.7	19.91	-0.35	-3.05	-4.92	-2.85	-4.54	-1.92
135	201	153.7	21.75	-0.38	-3.33	-4.92	-2.85	-4.54	-2.1
140	217.96	166.7	23.59	-0.41	-3.62	-4.92	-2.85	-4.54	-2.28
145	234.91	179.69	25.43	-0.44	-3.9	-4.92	-2.85	-4.54	-2.46
150	251.86	192.69	27.26	-0.47	-4.18	-4.92	-2.85	-4.54	-2.63
155	268.81	205.69	29.1	-0.5	-4.46	-4.92	-2.85	-4.54	-2.81
160	285.77	218.69	30.94	-0.52	-4.74	-4.92	-2.85	-4.54	-2.99
165	302.27	232.88	32.95	-0.56	-5.01	-4.92	-2.85	-4.54	-3.18
170	315.9	254.8	36.05	-0.61	-5.24	-4.92	-2.85	-4.54	-3.48
175	326.62	284.57	40.25	-0.68	-5.42	-4.92	-2.85	-4.54	-3.89
180	339.85	305.63	46.34	-0.72	-5.64	-4.92	-2.85	-4.54	-4.47
185	356.89	313.28	55.49	-0.74	-5.92	-4.92	-2.85	-4.54	-5.36
190	373.9	318.07	68.9	-0.75	-6.2	-4.92	-2.85	-4.54	-6.65
195	389.66	322.41	88.26	-0.76	-6.46	-4.92	-2.85	-4.54	-8.52
200	403.45	326.89	115.79	-0.77	-6.69	-4.92	-2.85	-4.54	-8.52
205	414.65	331.69	153.86	-0.78	-6.88	-4.92	-2.85	-4.54	-8.52

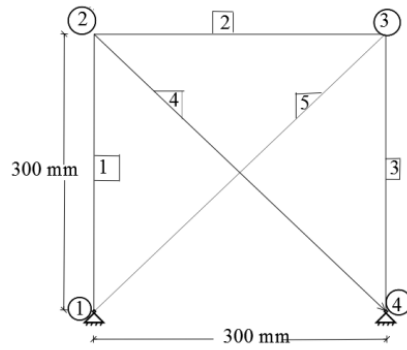


Figure 6.1: Truss example

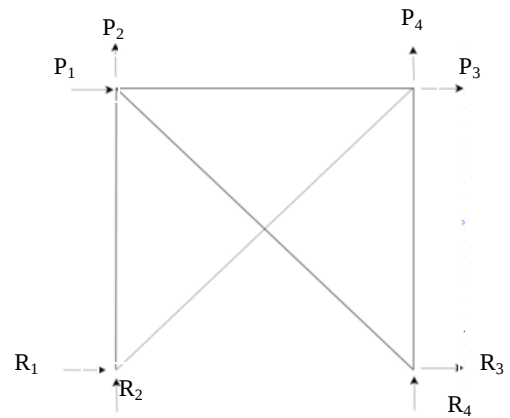


Figure 6.2: External joint and reinforcement components

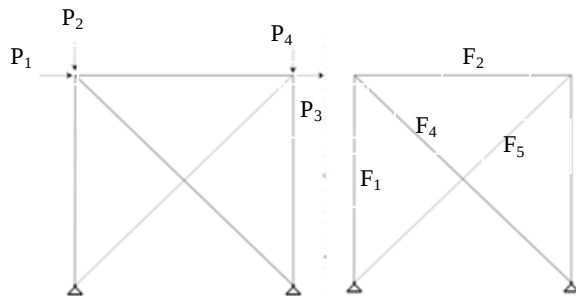


Figure 6.3: Member forces

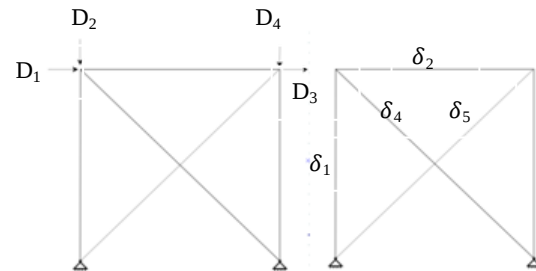


Figure 6.4: External and internal deformations

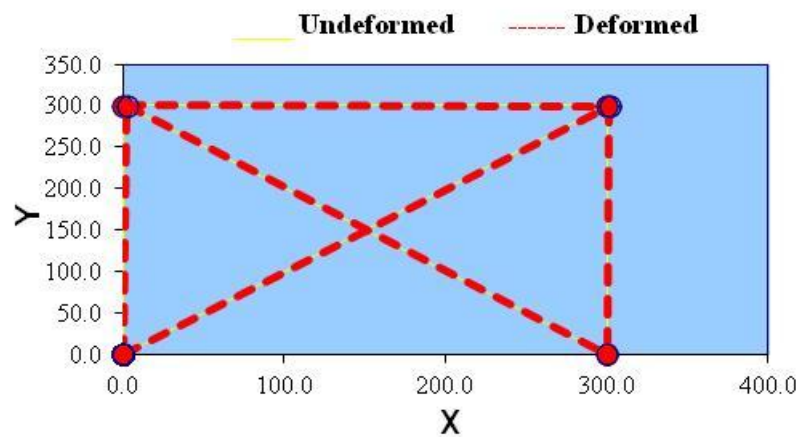
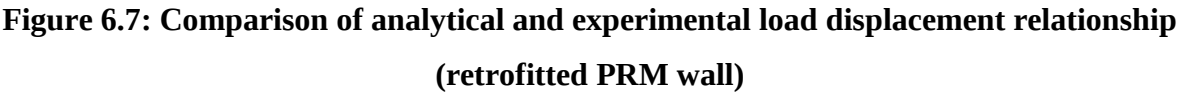


Figure 6.5: Deformed and undeformed shape of the truss example



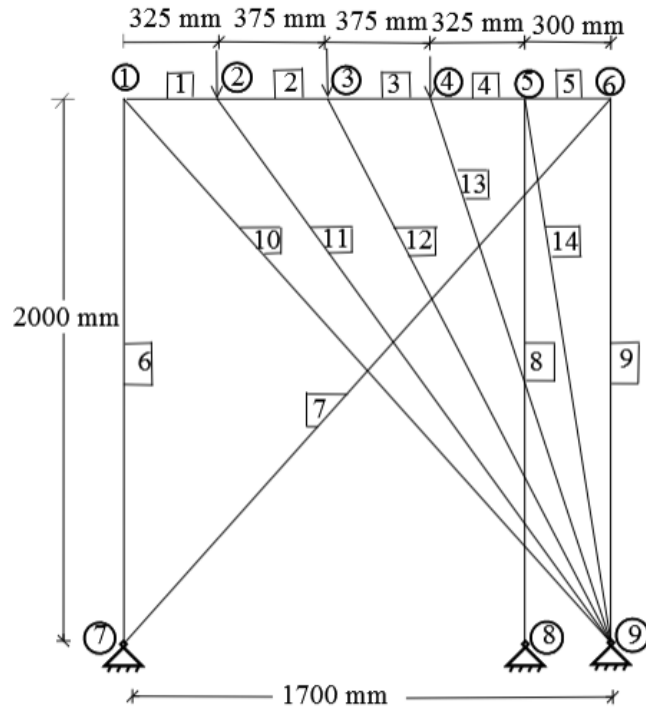


Figure 6.8: Modeling of the retrofitted unreinforced masonry wall

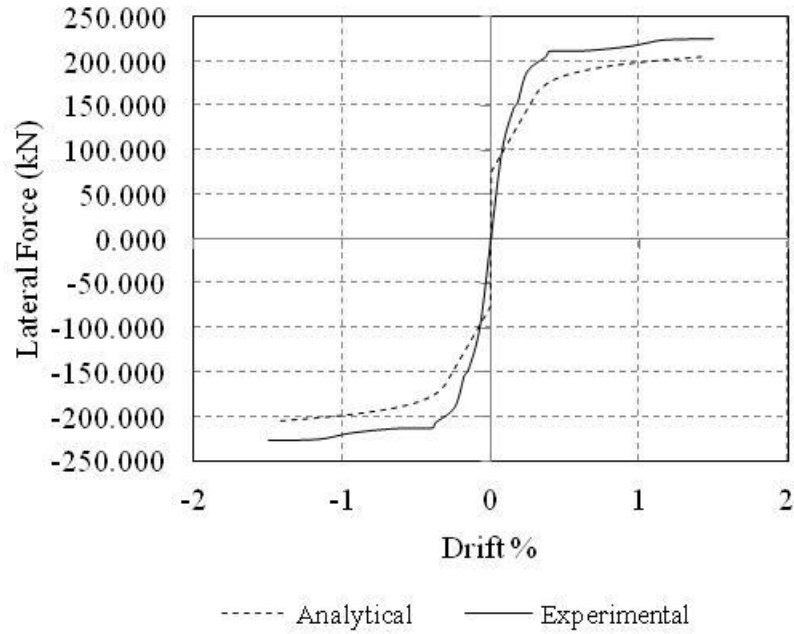


Figure 6.9: Comparison of analytical and experimental force-displacement relationship (retrofitted URM wall)

Chapter 7

Conclusions

7.1 Summary

A large inventory of existing masonry buildings in Canada and around the world were built with unreinforced or inadequately reinforced walls. Lateral load capacities of these buildings are often much less than seismic force demands indicated by current building codes. When subjected to strong earthquakes, these walls sustain significant damage, sometimes suffering from total collapses. Therefore, retrofitting existing substandard masonry buildings remains to be an effective seismic risk mitigation strategy. The use of FRP in retrofitting masonry structures has emerged as a viable methodology. Two major challenges remain to be resolved; i) providing sufficient bond between surface bonded FRP and the substrate, as well as the bond between the FRP sheets and the adjoining members for tension force transfer, and ii) ductility of a system that consists of brittle masonry and brittle FRP. The former challenge can be resolved through the use of proper anchors, and the latter challenge can be handled either performing elastic design or using ductile anchors as flexural tension elements.

The effectiveness of the surface bonded CFRP retrofitting technique for low rise masonry walls was investigated in the current research project, with a view of developing effective anchors for elastic and inelastic response. The FRP sheets were used for diagonal tension control and shear strength enhancement, whereas FRP or steel anchors were used as tension elements for improved flexural strength. Both PRM and URM walls were considered. The FRP sheets were applied only on one side of the walls to represent a frequently encountered constrain in practise. Two different types of anchors were used for elastic and inelastic behaviour. FRP fan type anchors were used in the PRM wall to promote strength enhancement in the elastic range of deformations. Steel sheet anchors were used in the URM wall for ductile response. A series of small scale tests were performed to select the appropriate anchor type.

Large scale wall tests were conducted under simulated seismic loading. The loading consisted of a constant gravity load and incrementally increasing lateral deformation reversals. The FRP

retrofitting changed the mode of behaviour from shear to flexure. The FRP Fan Anchors resulted in elastic improvement in wall flexural capacity. Ductile steel sheet anchors resulted in both strength and ductility. Truss models were constructed for both walls and analyses of the walls were conducted. The conclusions drawn from the research project are presented in the following section.

7.2 Conclusions

The following conclusions can be drawn from the current research work reported in this thesis:

- Surface bonded CFRP sheets can be used as an effective seismic retrofit technique for load bearing masonry walls. The CFRP-retrofitted specimens, with surface-bonded FRP sheets placed in each of the two orthogonal directions, are able to withstand shear force reversals without significant shear distress. The FRP sheets control diagonal tension and horizontal sliding shear cracks in both PRM and URM walls.
- Surface-bonded CFRP sheets can be used to increase the flexural capacity of walls if properly anchored to their foundations (or slabs below). The anchors must be designed to have sufficient tension capacity to transfer tensile forces from the supporting element to the FRP on the wall panel.
- CFRP fan type anchors can be used to improve bond between FRP composites and the wall substrate. They can also be used as tension elements for flexural strength enhancement. FRP anchors show elastic behaviour until failure. They either develop concrete pullout cone failure or rupture capacity in tension. These anchors must be properly designed to resist flexural tensile forces required for flexural strength enhancement. Retrofit strategy involving the use of FRP anchors must be based on elastic response and sufficient elastic capacity to resist seismic force demands.
- Ductile response can be attained in masonry walls when surface-bonded FRP composites are anchored to the supporting elements by means of steel sheet anchors. The stainless steel sheets used as anchors in the current investigation were able to develop significant ductility, and enabled the walls to dissipate seismic energy. The anchors can be bonded to concrete elements by placing them in pre-cut slots, and adhering them with epoxy. This enables sufficient tensile capacity, while preventing the sheets from buckling inside the

concrete when subjected to compression. The same anchors can be epoxy bonded on FRP for sufficient tensile resistance, but must be bolted on the wall to reduce the buckling length under compression force reversals.

- The surface-bonded FRP sheets on masonry walls, anchored to concrete foundation resulted in approximately 50% increase in the lateral load resistance of the PRM Wall tested in the current investigation. The load increase was as high as four times the unretrofitted capacity in the URM Wall.
- Seismic retrofit design of PRM and URM load bearing walls can be implemented by following the steps outlined below:
 - Provide sufficient layers of FRP to prevent diagonal tension failure in the wall panel through surface-bonded sheets. This can be done by assuming a 45-degree shear crack and computing the force resistance provided by the fibres in the plane of applied shear forces while ensuring that the strain limit for bond is not exceeded. The increase in diagonal tension resistance must be limited to shear force associated with compression crushing of diagonal struts.
 - Provide sufficient flexural resistance by means of FRP anchors to ensure elastic behaviour under code specified seismic force demands, or provide ductile steel anchors to develop sufficient inelastic deformability to resist seismic forces, reduced by appropriate force reduction factors. The level of ductility and associated reduction in design force levels depend on ductility available in the type of anchors considered.
- Masonry walls can be retrofitted only on one side with FRP composites, provided that sufficient lateral bracing is present in the structure.
- Masonry load bearing walls retrofitted with surface-bonded FRP sheets well anchored to the supporting elements can be analysed using simple truss models. The models provided sufficiently accurate force capacities to be used in design. However, the computation of displacements requires reasonably accurate modelling of element rigidities.
- The steel anchors tested were shown to develop a minimum of 1.0 MPa bond between the steel and FRP surfaces, when epoxy was used as resin. This bond stress can be used for designing epoxy-adhered steel anchors.

- FRP composites can be used in the form of pre-cured sheet anchors, similar to the steel sheet anchors tested in the current investigation, to develop elastic tensile forces for flexural strengthening of walls.

7.3 Recommendations for Future Work

Based on the observations made during the combined experimental and analytical study conducted in the current research project, the following additional work is recommended:

- Additional wall tests should be conducted with the proposed retrofit and anchor technologies investigated in the current research project to generate more test data prior to developing complete design guidelines.
- Further experimental study should be undertaken to establish the bond characteristics between FRP and steel materials. This may be done by using small-scale samples, as well as large scale wall panels. The experimental program may include the use of different resins for improved bond.
- Additional tests are required to establish the requirements of threaded rods used to connect the steel sheet anchors to masonry walls for prevention of buckling in compression.
- Improved truss analysis is suggested with considerations of inelastic material properties within the post-peak range, incorporating appropriate strength and stiffness decay in members.
- Finite element modeling and analysis should be performed to understand the behaviour of masonry walls with and without FRP retrofits, with emphasis placed on bond characteristics of surface mounted FRP sheets, as well as FRP and steel anchors.

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APPENDIX A

Additional force strain relationship for the PRM wall

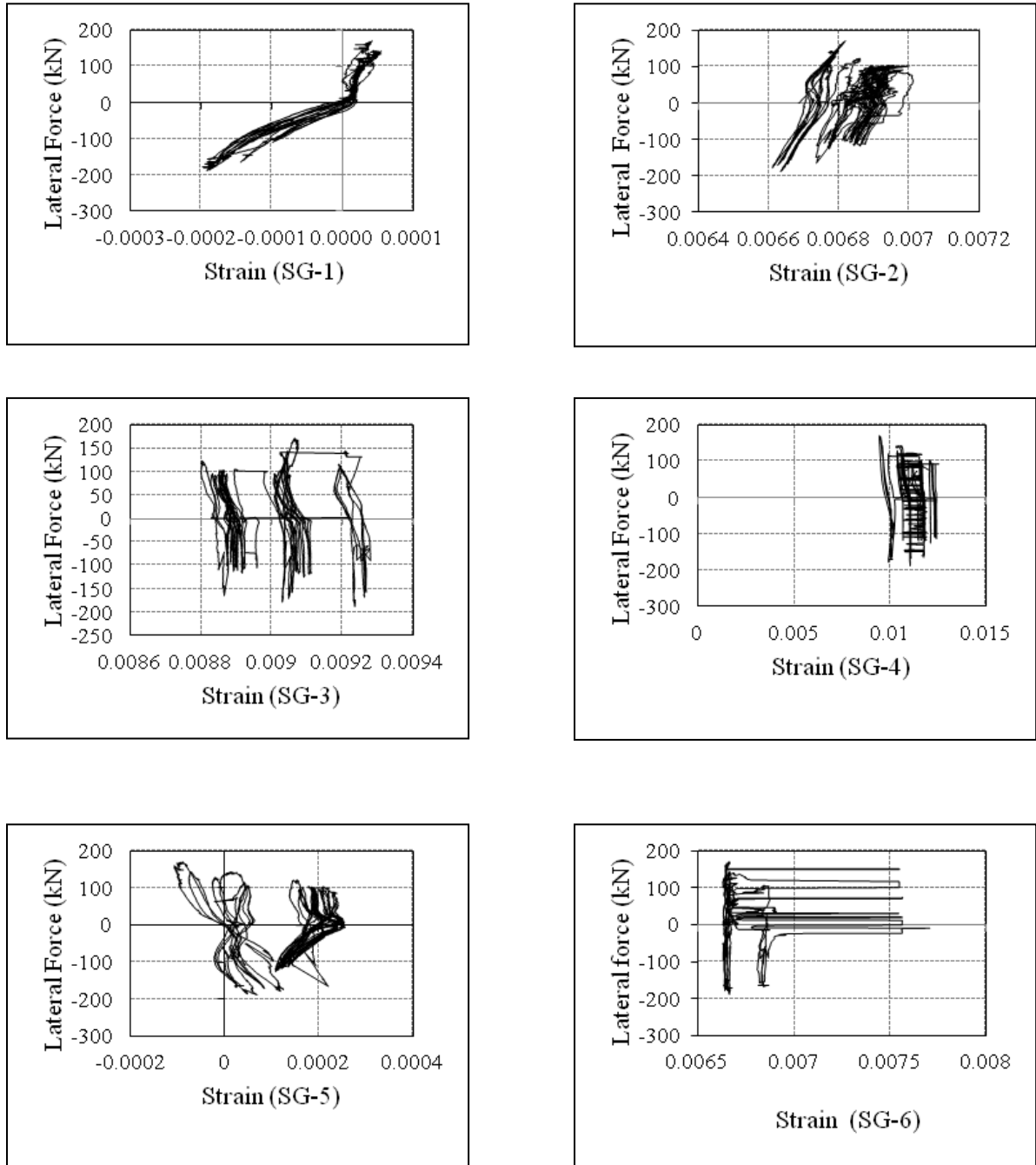


Figure A 1: Lateral force vs. strain in FRP for the PRM wall (SG-1 to SG-6)

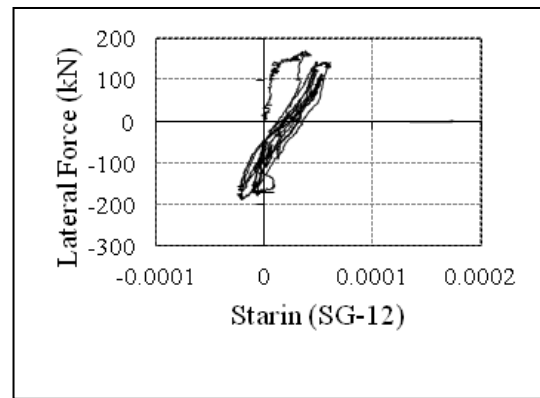
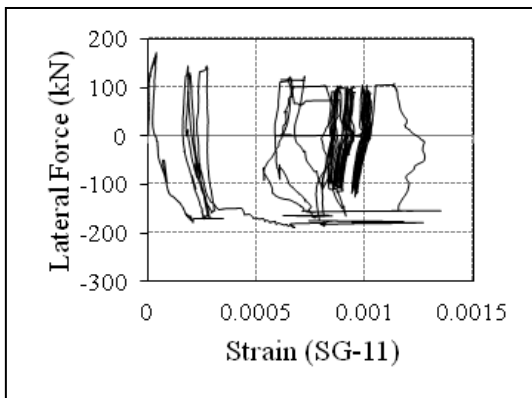
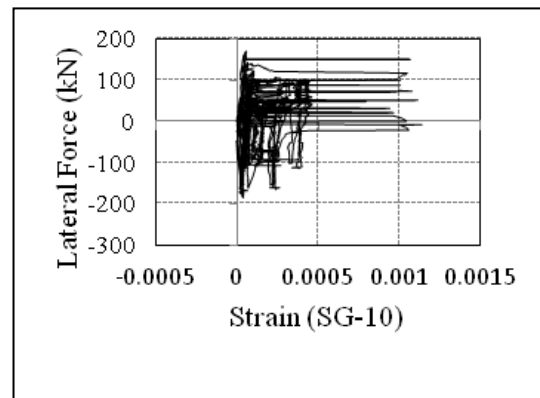
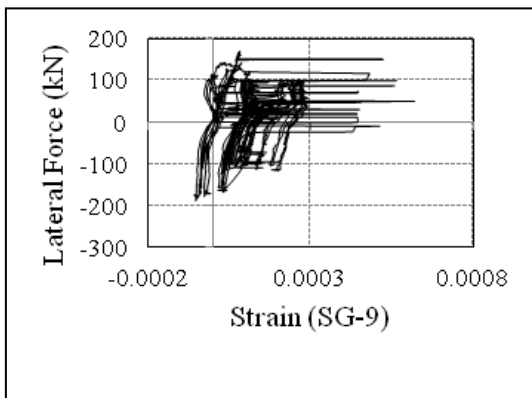
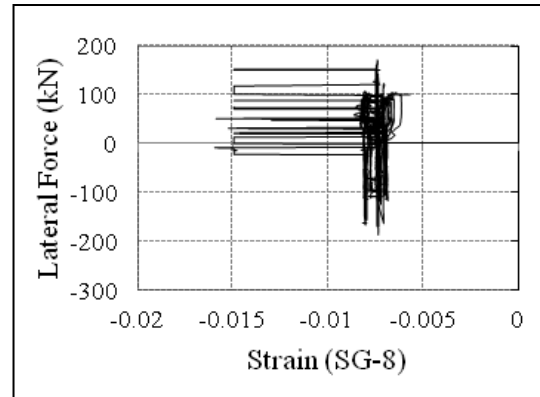
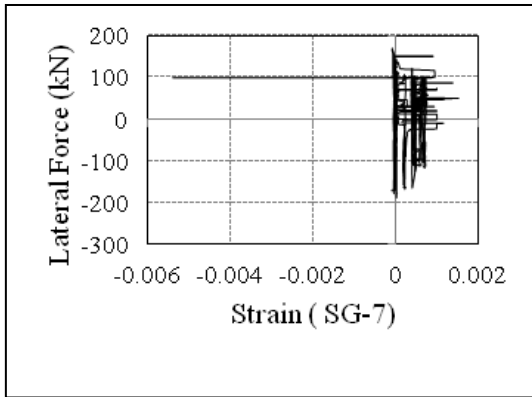


Figure A 2: Lateral force vs. strain in FRP for the PRM wall (SG-7 to SG-12)

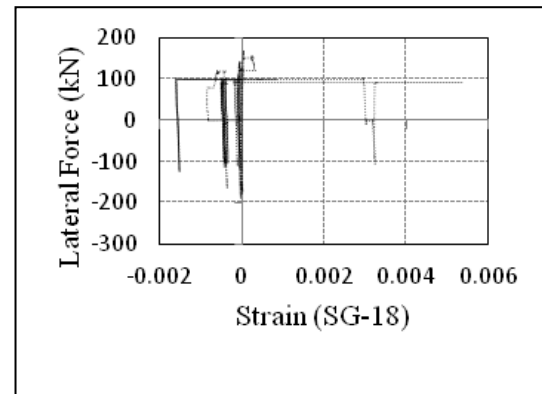
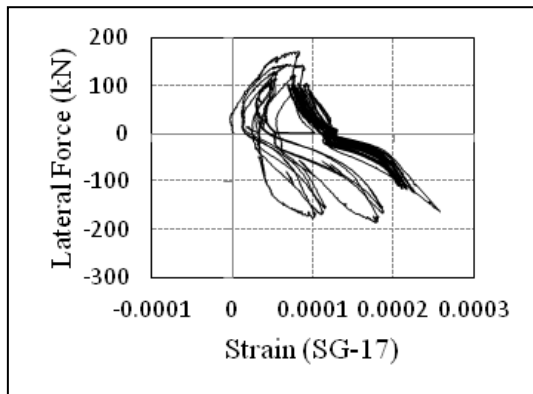
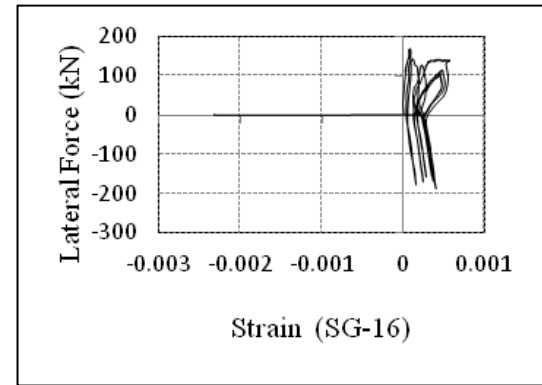
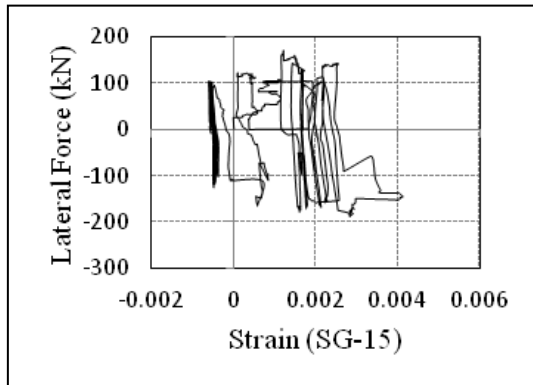
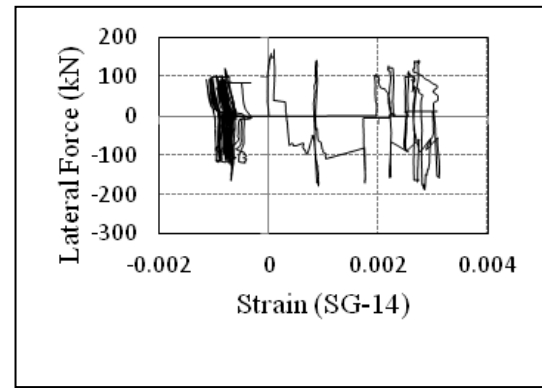
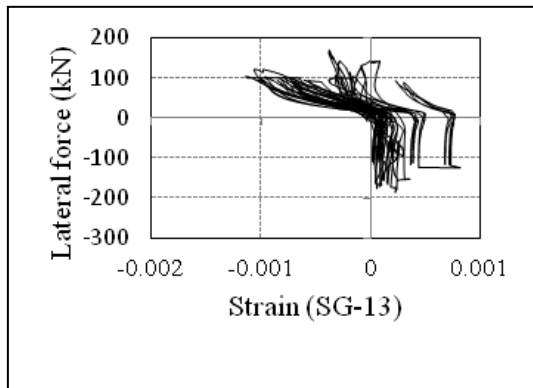


Figure A 3: Lateral force vs. strain in FRP for the PRM wall (SG-13 to SG-18)

APPENDIX B

Additional Force-strain relationship in the URM wall

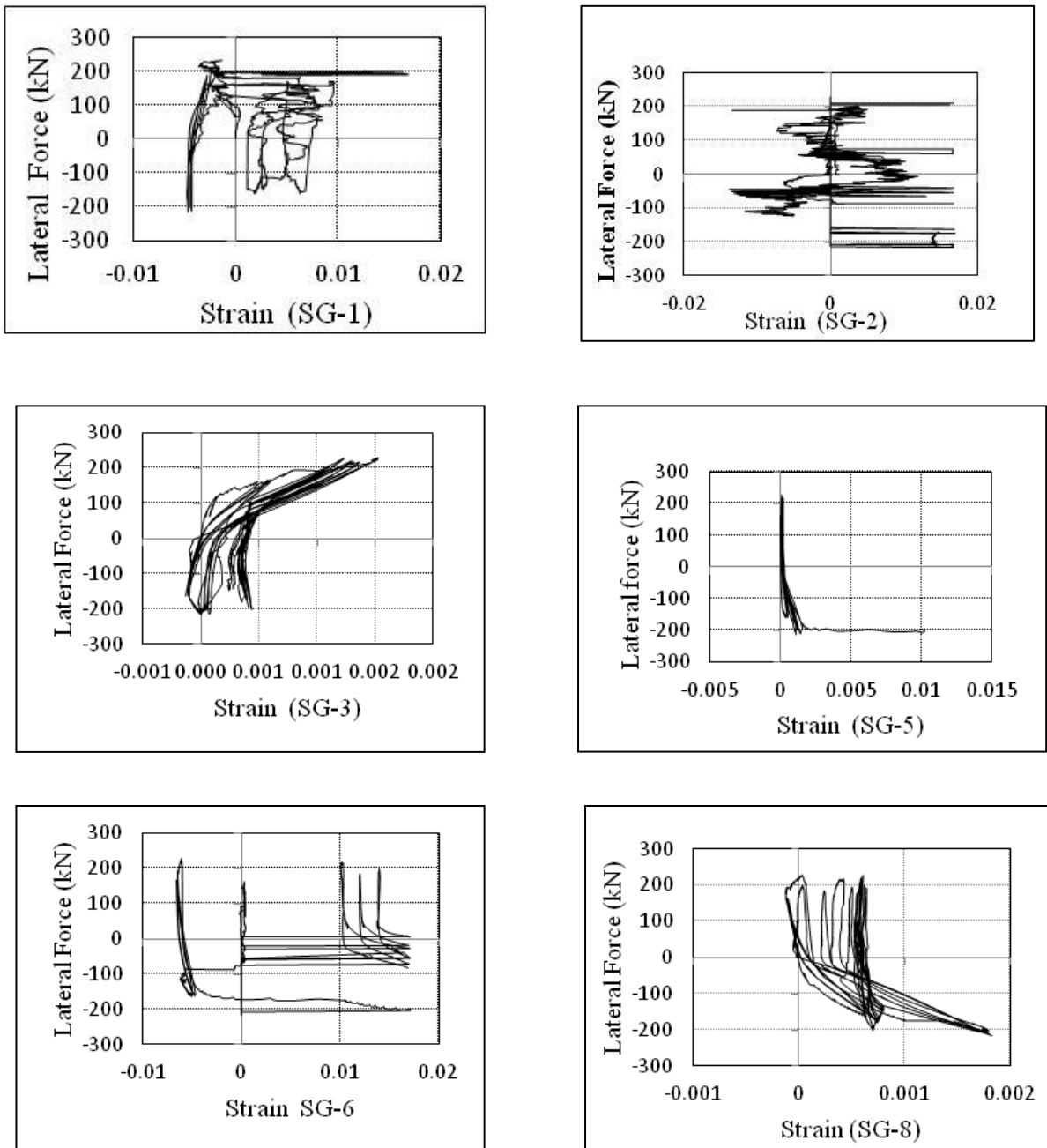


Figure B1: Lateral force vs. strain relationship in steel anchor in the Unreinforced Masonry Wall (URM)

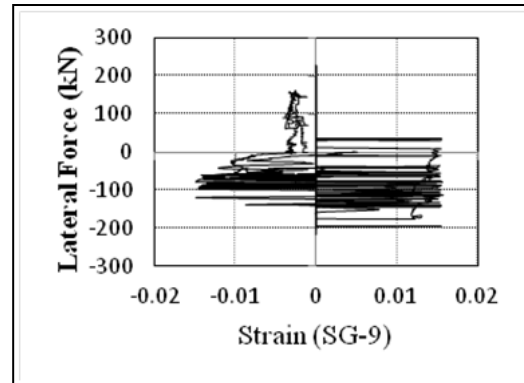
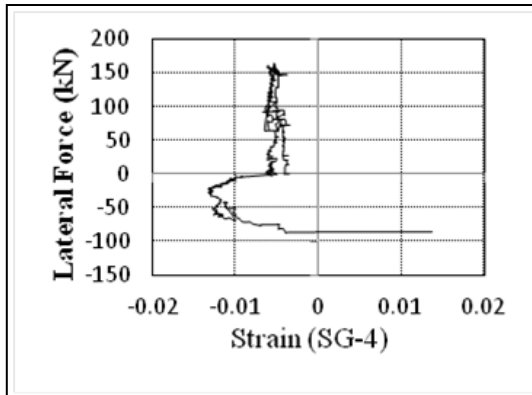


Figure B2: Lateral force vs. strain in FRP (Underneath steel anchor)

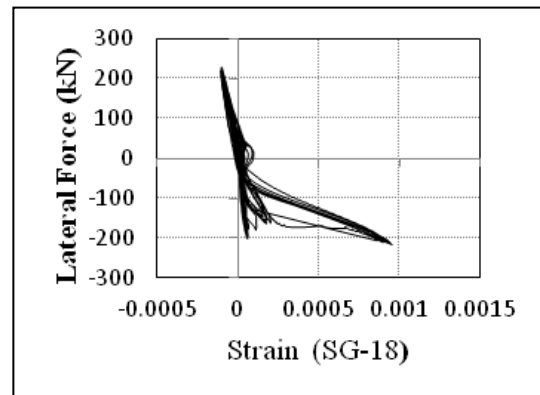
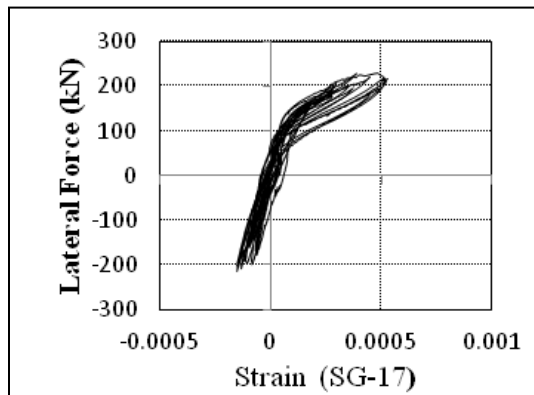


Figure B3: Lateral force vs. strain in wall FRP

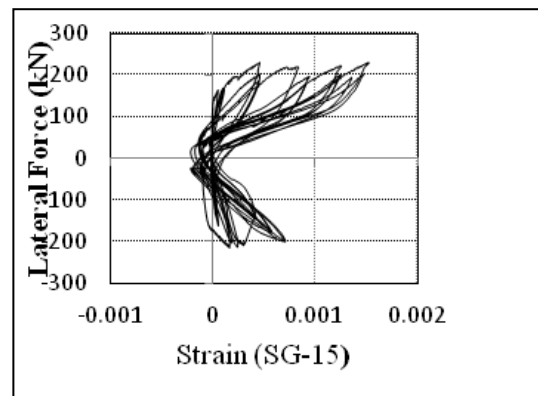
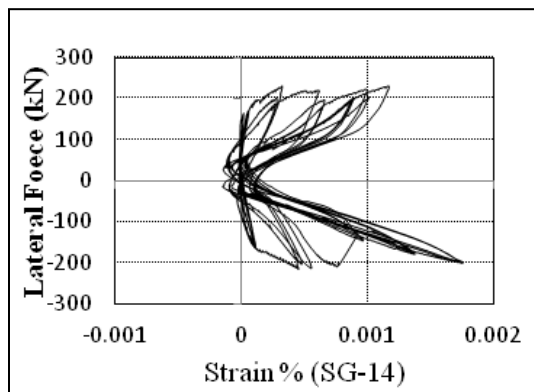
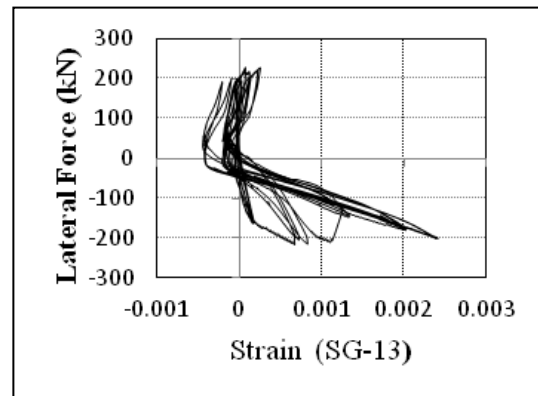
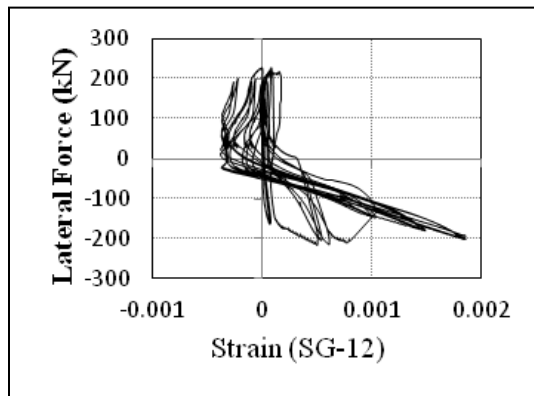
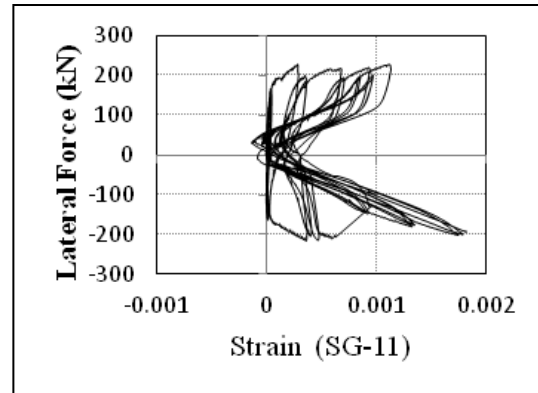
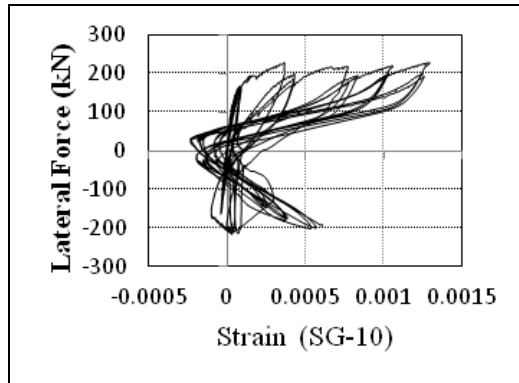


Figure B4: Lateral force vs. strain in prestressing wires