

Seismic Retrofit of Load Bearing URM Walls with Internally Placed Reinforcement and Surface-Bonded FRP Sheets

by

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Abstract

Concrete block masonry is a common building material used worldwide, including Canada. Reinforced masonry buildings, designed according to the requirements of recent building codes, may result in seismically safe structures. However, unreinforced masonry (URM) buildings designed and constructed prior to the development of modern seismic design codes are extremely vulnerable to seismic induced damage. Replacement of older seismically deficient buildings with new and seismically designed structures is economically not feasible in most cases. Therefore, seismic retrofitting of deficient buildings remains to be a viable seismic risk mitigation strategy. Masonry load bearing walls are the most important elements of such buildings, potentially serving as lateral force resisting systems.

A seismic retrofit research program is currently underway at the University of Ottawa, consisting of experimental and analytical components for developing new seismic retrofit systems for unreinforced masonry walls. The research project presented in this thesis forms part of the same overall research program. The experimental component includes design, construction, retrofit and testing of large-scale load bearing masonry walls. Two approaches were developed as retrofit methodologies, both involving reinforcing the walls for strength and deformability. The first approach involves the use of ordinary deformed steel reinforcement as internally added reinforcement to attain reinforced masonry behaviour. The second approach involves the use of internally placed post-tensioning tendons to attain prestressed masonry behaviour. The analytical component of research consists of constructing a Finite Element computer model for nonlinear analysis of walls and conducting a parametric study to assess the significance of retrofit design parameters. The results have led to the development of a conceptual retrofit design framework for the new techniques developed, while utilizing the seismic provisions of the National Building Code of Canada and the relevant CSA material standards.

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1 Introduction

1.1 Background

Masonry construction dates back to the earliest days of civilization. In modern days, a large portion of structures are built using masonry or contain masonry elements. The majority of masonry buildings that are located in seismic regions are constructed before the enactment of modern seismic codes. These buildings were designed to resist vertical gravity loads and lateral wind loads. Therefore, they are often deficient in resisting seismic force demands specified in recent building codes. Unreinforced masonry construction has not been permitted for use in structures situated in moderate to high seismic zones in Canada since 1980 (NBCC 1980; Mitchell et al. 2003). However, after the NBCC 2005 was issued and adopted by the Provinces, seismic design considerations have become mandatory in most of Canada, especially for post-disaster buildings, and those located on soft soil sites (Brzev and Anderson 2009). Retrofitting seismically deficient masonry buildings is often an economically viable approach for seismic risk mitigation.

Masonry buildings are constructed by concrete blocks or clays bricks, which are assembled with mortar. The use of reinforced masonry has become the practice for seismically active regions only recently. Steel reinforcement in masonry provides tensile resistance and ductility that is essential for the dissipation of seismic energy. A large number of existing masonry structures are unreinforced masonry (URM) or lightly/partially reinforced masonry structures. URM construction has significant resistance to gravity loads but remains weak in resisting tensile stresses produced by seismic forces. Therefore, retrofit methodologies are being developed to mitigate seismic risk in these buildings.

Masonry buildings consist of several structural elements. The most vulnerable elements to seismic loads are the load-bearing walls. Loadbearing masonry walls transfer axial loads to the foundation while resisting lateral loads caused by wind or earthquake effects. Lateral loads can be out-of-plane or in-plane. These walls are separated into two types; those under axial load and out-of-plane loading, and others under axial load and in-plane loading. The latter category is traditionally called shear walls. The lateral load resistance of a building is primarily dictated by the in-plane capacity of shear walls. Out-of-plane resistance of URM shear walls is relied on the

tensile strength of masonry and compression arching between the floors. A general rule of thumb used for out-of-plane resistance is to limit the height-to-thickness and length-to-thickness ratios of masonry walls to control tensile stresses so that they do not exceed the masonry capacity threshold. However, these limits are not sufficient in regions of high seismicity. Added reinforcement may be required if tensile demand of the structure is higher than the tensile capacity of masonry.

The failure mode in shear walls depends on load combinations, wall geometry, properties of materials, and details of reinforcement. The four modes of failure observed in masonry shear walls are depicted in Figure 1.1. Accordingly, they include shear failure, sliding failure, rocking failure, and crushing failure (ElGawady, Lestuzzi et al. 2006). Walls with high axial loads and low aspect ratios develop diagonal cracking failure when exposed to high shear forces. Diagonal cracks follow the path of bed and head joints when strong blocks and weak mortars are used (Figure 1.1 (a)). They go through masonry units when the blocks are weak and the mortar is strong. The sliding failure occurs when the shear force exceeds the shear-friction resistance between the masonry units and the mortar joint (Figure 1.1 (b)). This type of failure usually occurs in squat walls with low axial compressive loads and poor-quality mortar, resulting in the sliding of the upper part of the wall relative to the lower part. The combination of low axial load and overturning moment can cause overturning failure, which is accompanied by a crack propagation and overturning of the wall (Figure 1.1(c)).

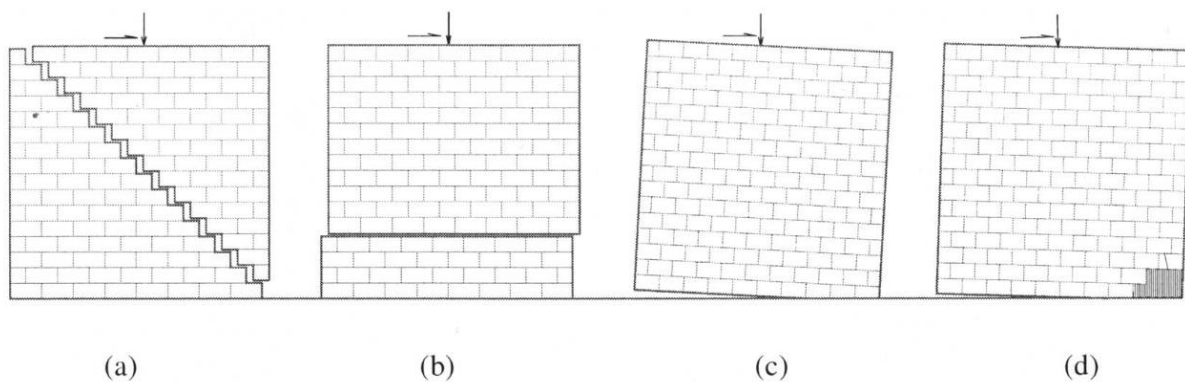


Figure 1.1 In-plane failure modes of a laterally loaded URM wall: (a) Shear failure; (b) Sliding failure; (c) Rocking failure; and (d) Toe crushing (ElGawady et al 2007)

The compression failure of the toe occurs when axial compression is high and shear stress is low (Figure 1.1(d)). This load combination produces tensile stresses in the wall parallel to the bed joints, which forms vertical cracks instead of diagonal cracks.

The wall height to length ratio and axial load level have significant effects on failure modes of shear walls. Energy dissipation and ductility depend on their mode of failure. For instance, rocking failure is a low energy dissipating failure, but sliding failure dissipates large amount of energy and has smaller strength decay. Research is needed to prevent the aforementioned failure modes for improved seismic resistance.

1.2 Objective

The objective of this research project is to develop new retrofit methodologies for seismically deficient masonry walls involving the use of deformed steel reinforcement or post-tensioning strands for flexural strength and ductility enhancements along with surface-bonded carbon fibre reinforced polymer (CFRP) sheets for shear strengthening. The objective also includes development of a reliable analytical modeling approach that could predict structural response of retrofitted masonry walls and a parametric study to assess the significance of design variables. The overarching objective is to develop a conceptual design framework for seismic retrofitting URM walls with internally placed steel reinforcement and surface-bonded CFRP sheets.

1.3 Scope

The scope of research consists of:

- Review of previous research on seismic retrofit of URM load bearing walls.
- Design and construction of two large-scale seismically deficient URM walls, companion to the reference wall that was tested in an earlier phase of the same research program by Taghdi et al. (2000). The wall construction involves URM walls supported by reinforced concrete footings.
- Design and application of retrofit methodologies for walls:
 - Wall 1: Use of internally placed deformed steel reinforcement for flexural capacity enhancement and placement of surface-bonded CFRP sheets for control of diagonal tension associated with shear forces.
 - Wall 2: Use of internally placed 7-wire post-tensioning strands for flexural capacity enhancement, as well as placement of CFRP sheets to control diagonal tension with concrete filling of masonry cells for enhanced diagonal compression capacity.

- Tests of retrofitted masonry walls under constant axial compression and incrementally increasing lateral deformation reversals.
- Data analysis and assessment of failure modes.
- Comparison of retrofitted walls with a companion unretrofitted reference wall.
- Development of analytical models for nonlinear analysis of retrofitted masonry walls and verification of the model against test data.
- Analytical parametric investigation of retrofitted masonry walls to assess the significance of design parameters affecting wall behavior.
- Development of seismic retrofit methodologies for URM walls with internally added re-bars or prestressing strands with surface-bonded CFRP sheets for improved strength and deformability.
- Development of a conceptual retrofit design strategy and design recommendations.

2 Literature Review

2.1 Previous Experimental Research

The Masonry Society of America released a report after the 1994 Northridge earthquake, explaining the considerable progress made in seismic detailing of masonry structures since the 1971 San Fernando earthquake (Bruneau 1995). This report concluded that newer and retrofitted load bearing masonry walls effectively performed with minimum to no damage during the earthquake, while older load bearing unreinforced masonry walls were severely damaged. During the past two decades, many retrofit techniques were developed to improve the behaviour of seismically deficient URM walls. Some of the conventional retrofit techniques for unreinforced and partially reinforced masonry walls are discussed in the following subsections.

2.1.1 Grout or Epoxy Injection as Retrofit Strategy

Strengthening of existing masonry walls by injection of grouts has become a frequent practice (Binda et al. 1994). This method is usually used for filling the cracks and voids that are generated by physical and chemical deterioration in masonry buildings. It restores the original integrity of the wall. The injection of grout into the empty space of multi wythe masonry walls also increases the composite behavior of the wall. The injectability of mortar mix and the injection technique used are key factors in the success of grout or epoxy injection. Cement grout is commonly used for large cracks, while epoxy resin is more suitable for small cracks.

The cement grout injection can be summarized in the following steps (Hamid, El-Sayed et al. 1999):

1. Sealing of the cracked areas by the injection ports.
2. Washing of the cracks and holes to remove dust and debris.
3. Injecting grout with injection pressure of less than 0.1 MPa to fill the cracks from bottom to top of the wall.
4. Using a second grout mix (Cement-based or epoxy) to fill small cracks.

Cement-based injection can restore up to 80% of the original compression strength of masonry walls (Schuller, Atkinson et al. 1994). Retrofitting of masonry walls by epoxy grout

increases lateral resistance of walls from 2 to 4 times the original wall capacity. Epoxy injection also increases wall stiffness by about 10 to 20%.

2.1.2 Surface Treatment for Seismic Retrofitting

Surface treatment is a common method, which has largely developed through experience (El Gavadi et al. 2004). Surface treatment consists of a layer of reinforcement that is installed to the exterior face of the masonry wall, which is covered by a layer of adhesive material like cement-based mortar. The reinforcement can be in the form of steel mesh, fine rods, or steel bars that are located at designated locations. Some of the most conventional surface treatment methods are ferrocement, reinforced plaster, and shotcrete.

Ferrocement consists of a layer of steel mesh that is covered by high strength cement mortar layer. The thickness of mortar varies between 10 to 50 mm. This is an inexpensive retrofitting method for masonry walls that can be applied by unskilled workers. Ferrocement increases in-plane capacity of masonry walls by confining the masonry units after cracking. Static cyclic tests performed by Abrams and Lynch (Abrams and Lynch 2001) showed that ferrocement increases lateral in-plane capacity of masonry walls by a factor of 1.5. Ferrocement also increases out-of-plane capacity of masonry walls by improving arching action.

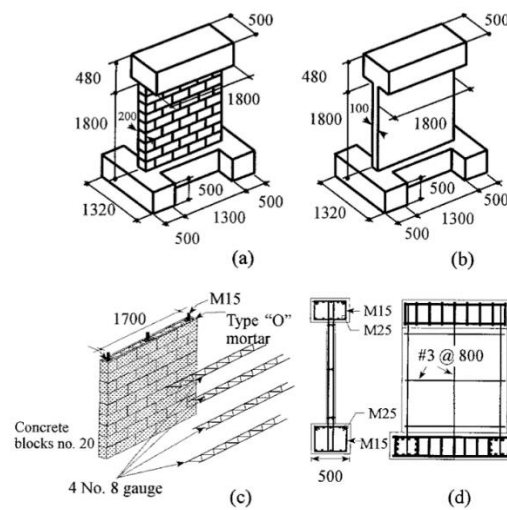
2.1.3 External Reinforcement for Improved Seismic Response

External reinforcement consists of steel plates, FRP sheets, or other types of reinforcements directly attached to the wall. Experimental test results show that this method of strengthening is an effective and convenient way of improving ductility and strength capacity of masonry walls.

2.1.3.1 Use of Diagonal Steel Strips

Taghdi, Bruneau and Saatcioglu (2000a and 2000b) at the University of Ottawa tested four concrete block masonry walls and two reinforced concrete shear walls to evaluate a new retrofitting technique for low-rise masonry and concrete shear walls using steel strips. A fairly detailed description of the research project is presented here as the reference URM wall used in the current investigation was obtained from this study. The walls were labeled as 9, 9R, 10, 10R, 11, and 11R, where single numbers represent as-built walls and the letter R indicates retrofitted walls. All six specimens were designed to resemble low-rise masonry and reinforced concrete walls based on

old seismic design codes. The first and second pairs of walls (9, and 10) were made of masonry units and the third pair of walls (11) was reinforced concrete. Standard concrete masonry blocks of 200 mm nominal size were used in the construction of masonry walls. The aspect ratio of all walls was 1.0 making them critical in shear or rocking failure mode. Figure 2.1 shows the dimensions and reinforcement layout of masonry and reinforced concrete walls. First pair of masonry walls (9 and 9R) had no reinforcement while the second pair of masonry walls (10 and 10R) was partially reinforced. The average compressive strength of grouted and ungrouted masonry prisms was 8.1 MPa and 12.5 MPa respectively and the average concrete compressive strength for reinforced concrete walls was 29 MPa. As it's shown in Figure 2.1, 15M and US #3 rebars with yield strengths of 400 MPa and 642 MPa were used in masonry walls and reinforced concrete walls, respectively. The retrofitting consisted of two diagonally attached 220 mm wide steel strips with 3.81 mm thickness on both sides of the wall. Two 80 mm wide vertical steel strips with 3.81 mm thickness were used as boundary elements on both sides of the walls. The width of steel strips was chosen to have yielding prior to fracturing at bolt locations. The measured yield strength of diagonal strips and vertical strips were 227 MPa and 248 MPa, respectively. Figure 2.2 shows a masonry wall retrofitted by steel strips. Through-thickness 15.9 mm and 9.5 mm diameter A328 structural steel bolts were used to secure diagonal and vertical steel strips, respectively. Steel angles were fixed to the foundation and loading beam by anchor bolts. The steel strips were welded to the angles.



**Figure 2. 2 Dimensions and reinforcement layout of concrete and masonry walls tested
by Taghdi et al (2000a and 2000b)**

A combination of sliding at push and rocking at pull was observed during reversed cyclic testing of the unreinforced masonry wall (Wall 9). Cracking occurred along the bed joint above the second row of blocks which was attributed to the high amount of mortar that was used at the base joint, preventing crack initiation at the base. Sliding at push occurred at a higher force of 64.5 kN while rocking at pull was observed at -58.5 kN of lateral force. Figure 2.3 shows the load-drift response of the wall, illustrating unsymmetrical behaviour of the wall up to about 1 percent drift. The wall demonstrated relatively large deformations with low strength decay before the failure.

The partially reinforced masonry wall (Wall 10) had a symmetrical force-displacement response with fat hysteresis loops. The wall eventually suffered from diagonal crushing of masonry struts due to shear. Shear failure of the wall resulted in early strength degradation and low energy dissipation. Figure 2.4 shows the load-drift response of the wall.



Figure 3.2 A masonry wall retrofitted by steel strips

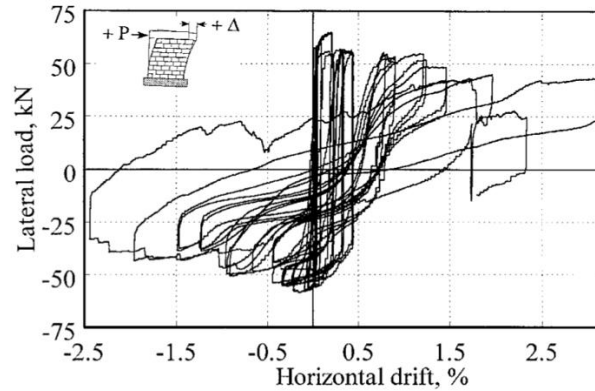


Figure 4.3 Load-drift response of the unreinforced masonry wall

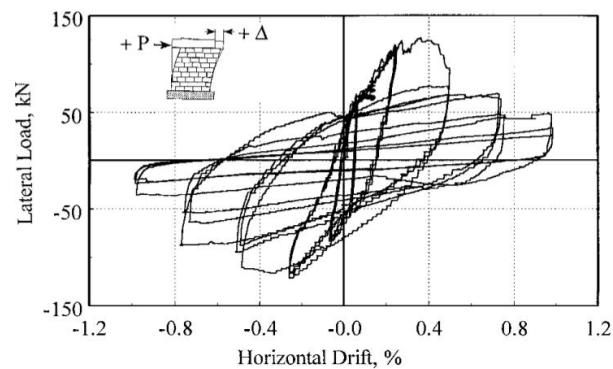


Figure 5.4 Load-drift response of the reinforced masonry wall

The reinforced concrete wall had a symmetrical hysteretic response accompanied by rocking behaviour during the final cycles. The wall had a rigid body rotation behaviour, which was controlled by vertical reinforcement. At the end of the test, the wall corners were severely damaged and the vertical reinforcement that was close to the corners buckled elastically while the vertical reinforcement at the wall center yielded extensively. After the yielding of central reinforcement, the wall started developing strength degradation, which was accompanied by rocking behaviour. All the plastic deformations occurred at the base of the wall and the main body of the wall remained undamaged. Figure 2.5 shows the load-drift response of the wall. All the retrofitted walls had an acceptable seismic behaviour as compared to the non-retrofitted as-built walls. Diagonal steel strips provided symmetrical and stable cyclic behavior with high ductility, stiffness, and energy dissipation.

The retrofitting system for unreinforced masonry wall (9R) was effective in producing uniform cracks in the masonry. The steel strips yielded as the lateral load was increased, which led

to the buckling of strips and crushing of masonry blocks. Wall 9R had maximum lateral load resistance of 4.5 times the resistance of Wall 9. The partially reinforced retrofitted masonry wall (10R) had superior seismic response compared to Wall 9R. The vertical reinforcements and filling the masonry block cells with mortar delayed early buckling of steel strips and improved the energy dissipation of the wall. The presence of retrofitting system dramatically increased the lateral load resistance, ductility, and energy dissipation of Wall 10R compared to Wall 10. The implemented retrofit technique improved seismic behaviour of reinforced concrete shear wall (Wall 11R) relative to unretrofitted Wall 11. Steel strips increased ductility, stiffness, and energy dissipation capacity of Wall 11R.

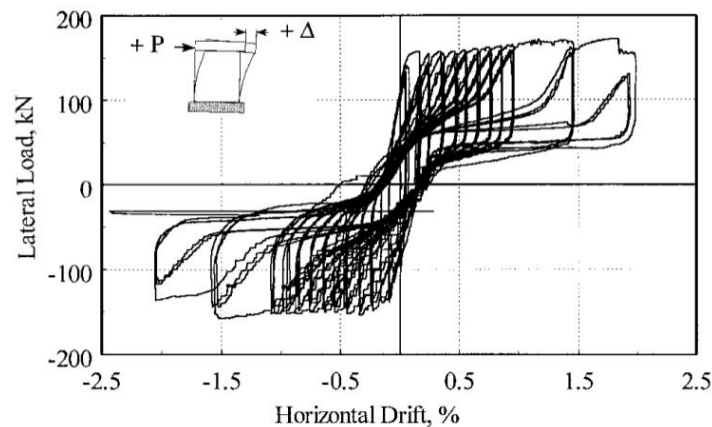


Figure 6.5 Force-drift response of the reinforced concrete wall

2.1.3.2 Use of FRP

The use of fiber reinforced polymers (FRPs) as rehabilitation and strengthening materials is a valid alternative. Appealing characteristics of fiber reinforcement are small thickness, high strength-to-weight ratio, high stiffness, and ease of application (Elvi et al. 2001.). Ehsani and Saadatmanesh (1996) studied the effectiveness of fiber composites in controlling the seismic response of URM walls. The research project included an experimental component which covered three main categories: i) out-of-plane flexural response of masonry beams, ii) in-plane shear response of laboratory specimens, and iii) in-situ shear test of hollow clay tile walls in the San Francisco City Hall building. Small URM beams were tested to evaluate the out-of-plane behaviour of URM walls retrofitted with fiber composites. The URM beams were 1.45 m long, 102 mm high and 216 mm thick. Each beam consisted of 19 solid clay bricks stacked in a single wythe. The URM beams were identified by a combination of four characteristics, each indicating

a property of the beam. The first characteristic was the type of epoxy (Epoxy 1 or Epoxy 2). Epoxy 1 was a two-component epoxy with high energy absorption. Epoxy 2 was similar to Epoxy 1 but with lower viscosity. Both materials had tensile capacities varying between 13.8 and 17.2 MPa, and modulus of elasticities varying between 517 and 862 MPa. The elongation at failure was 15%. The second characteristic was either M or M* which was related to the type of mortar that was used between the bricks. M represented stronger mortar with cement-lime-sand ratio of 1:0.25:3. M* represented weaker mortar that could be found in older buildings with the ratio of materials as 1:0.25:5. The third characteristic was the type of fabric that was used. Three types were considered. The first fabric type was glass fabric with an acrylic polyvinyl finish, having tensile strengths of 249 N/mm and 150 N/mm in strong and weak directions, respectively. The second and third fabric types contained unidirectional E-glass fibers with tensile capacities of 249 N/mm and 150 N/mm, respectively. The fourth characteristic indicated the roughness of the wall and the surface finish. Character F indicated mortar that was flush with the outer surface of the wall, and character S indicated small amount of mortar extruding from the joints.

Figure 2.6 shows the load-deflection response of all retrofitted masonry beams. Specimen 2M1S shows the weakest behaviour among the specimens, which was attributed to the weak fabric that was used. The specimen failed at 3.114 kN due to the tensile failure of fabric at 7 mm deflection. A comparison between the load-deflection responses of specimens 1M2S and 1M3F clearly shows the influence of fabric strength on load resistance. The response of specimens 1M3F and 2M3F indicates that Epoxy 2 showed superior behavior with higher ductility relative to Epoxy 1. Specimen 1M2S-1 was constructed with old reclaimed bricks to evaluate the effect of the retrofit technique on old buildings. The comparison of force-deformation response of specimens 1M2S-1 and 1M2S1 indicates differences between new and old bricks. Specimen 1M2S-1 had softer response with less load resistance, which can be attributed to its lower compressive capacity. The effect of mortar type had a minor effect on the load carrying capacity. However, Specimen 1M*2S with weaker mortar showed softer response relative to Specimen 1M2S.

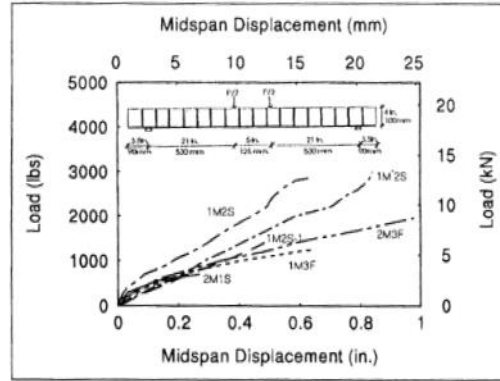


Figure 7.6 Load-deflection response of retrofitted masonry beams (Ehsani and Saadatmanesh 1996)

Ehsani and Saadatmanesh (1997) also studied different modes of failures for FRP attached masonry by testing 37 clay brick specimens. Direct shear tests were performed on masonry specimens. The masonry specimens consisted of three clay bricks that were covered by composite fibers on both sides. Masonry units were separated by lubricated spacers to measure the shear capacity of composite fibers without the mortar contribution. Figure 2.7 shows the force-deflection response of two specimens, retrofitted with a fabric having a weight of 339 g/m^2 and equal amounts of fibers in horizontal and vertical directions. The solid line represents the behavior of a specimen with horizontal and vertical fibers positioned perpendicular and parallel to the axis of loading, respectively. The dashed line represents the response of a similar specimen in which horizontal and vertical fibers made 45 and 135 degrees with the loading axis. The specimen with 45-135 degree orientations showed stiffer behaviour, because the fibers were in direct tension where the stiffness was the largest.

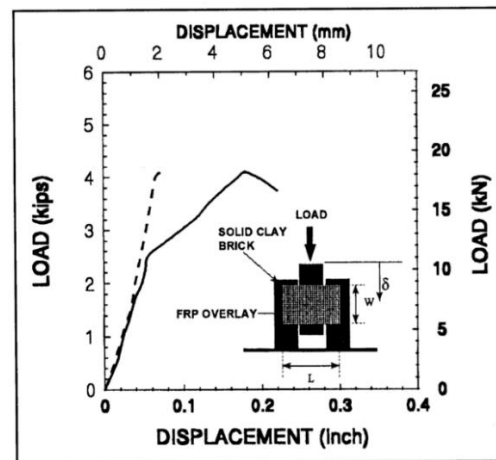


Figure 8.7 Force-deflection responses of direct shear tests

In-situ tests of hollow clay tile walls in San Francisco City Hall building were performed to study the effectiveness of proposed method on shear capacity of the walls. The average in-plane shear capacity of unretrofitted walls was 0.47 MPa. Three masonry walls were retrofitted by attaching 610 g/m² fabric on both sides of the wall. The average shear strength of three retrofitted walls was 1.63 MPa, which was 3.5 times larger than the unretrofitted wall. The failure mode in all the retrofitted walls was due to the crushing of clay tiles. The test results indicated that the retrofit of unreinforced masonry walls with composite fabrics was quite effective in terms of increasing shear capacity, flexural capacity, and ductility.

Thaniasis and Triantafillou (1998) established analytical and experimental study on short-term strength of FRP-retrofitted masonry walls under monotonic out-of-plane bending, in-plane bending, and in-plane shear in the presence of axial load. The out-of-plane and in-plane bending capacity as well as the shear capacity of FRP retrofitted walls were formulated based on an analytical study. Interaction diagrams were also provided for use in design. Twelve identical small wallettes were tested statically in four-point bending to evaluate the effect of FRP retrofit on masonry walls. Each wallette had dimensions of 900 x 120 x 400 mm and they consisted of 20 perforated clay units, stacked in a single wythe. The wall geometry and test setups for out-of-plane and in-plane bending tests are shown in Figure 2.8 and 2.9.

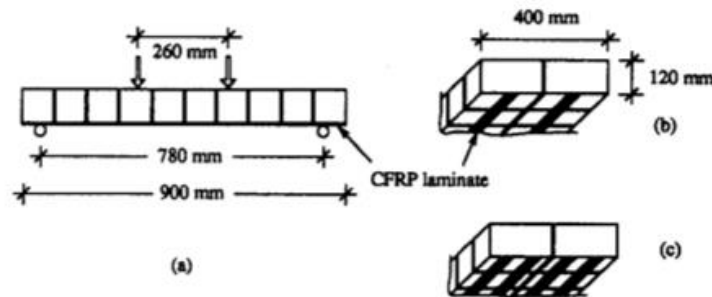


Figure 9.8 The specimen test setup for out-of-plane loading (Thaniasis and Triantafillou 1998)

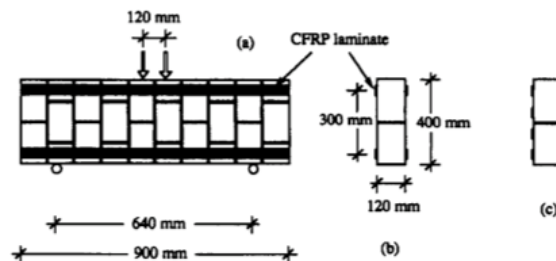


Figure 10.9 The specimen test setup for In-plane loading (Thaniasis and Triantafillou 1998)

The wallettes tested in out-of-plane bending were retrofitted with either two or four strips of CFRP laminates bonded on tension side of the walls (Figures 2.8(b) and (c)). Strengthening of the wallettes for in-plane bending was done by either two or four strips of CFRP laminates attached on both sides of the specimens (Figures 2.9(b) and (c)). Those tested in out-of-plane bending failed by the crushing of masonry in the compression zone. Figure 2.10 shows the load versus mid-span deformation relationships for four specimens. Two specimens were strengthened with two strips of CFRP that were labeled as O-2a and O-2b and two specimens were strengthened by four strips of CFRP which were labeled as O-4a and O-4b. The test results showed significant improvements in load carrying capacities of walls compared with the two unretrofitted specimens that had 4.05 kN and 4.7 kN capacity. The agreement between experimental and analytical results was acceptable with 15% deviation.

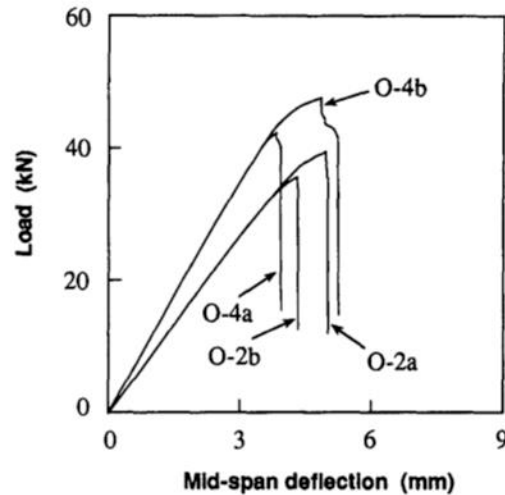


Figure 11.10 Load-deformation response of four specimens tested in out-of-plane

The wallettes subjected to in-plane bending had a premature failure due to the peeling-off of the FRP laminates. This behaviour was attributed to the short bond development length in the specimens. Table 2.1 provides a summary of test results for two unretrofitted and four retrofitted samples, indicating significant improvements in the load carrying capacity of wallettes. The agreement between the analytical and experimental results was acceptable with 27% and 12% error for the specimens with two and four strips of CFRP, respectively.

Table 2.1 Summary of test results for unretrofitted and retrofitted masonry specimens

Specimen	Experimental failure load (kN)
Unretrofitted 1	8.1
Unretrofitted 2	14.2
1-2a: Two strips of CFRP at each side	81.0
1-2b	88.7
1-4a: Four strips of CFRP at each side	152.0
1-4b	141.5

Galal et al. (2011) investigated the effect of CFRP wrapping on structural behaviour of reinforced masonry columns. Six reinforced masonry columns (390x390x1400 mm) were tested under constant axial force and cyclic reversals. Table 2.2 shows the properties of the tested specimens with their designations.

Table 2.2 Specimen properties with their designations

Reinforced masonry column: Control Specimen	RMC-0
Reinforced masonry column with 1 layer of CFRP wrap	RMC-CW-1
Reinforced masonry column with 2 layers of CFRP wrap	RMC-CW-2
Reinforced masonry column with 3 layers of CFRP wrap	RMC-CW-3
Reinforced masonry column with 1 layer of CFRP strip	RMC-CW-1S
Reinforced masonry column with 1 layer of CFRP wrap at plastic hinge	RMC-CW-1P

The specimens were representative of the lower half of 2.8 m reinforced masonry columns that were constructed on reinforced concrete footings. Figure 2.11 shows the details of as-built and retrofitted specimens. As it is shown at Figure 2.11 (c), the columns were reinforced by four 15M bars that were doweled inside the footing. Steel stirrups with a 4.75 mm diameter were placed between the masonry joints at a spacing of 200 mm. The hollow cells of masonry blocks were filled with coarse grout. The surface of masonry was prepared before the installation of FRP wraps and all grooves and joints were filled with cementitious-based plaster to provide a flat surface. The CFRP wraps were applied with 150 mm overlaps following the ISIS Canada Design Manual (2008).

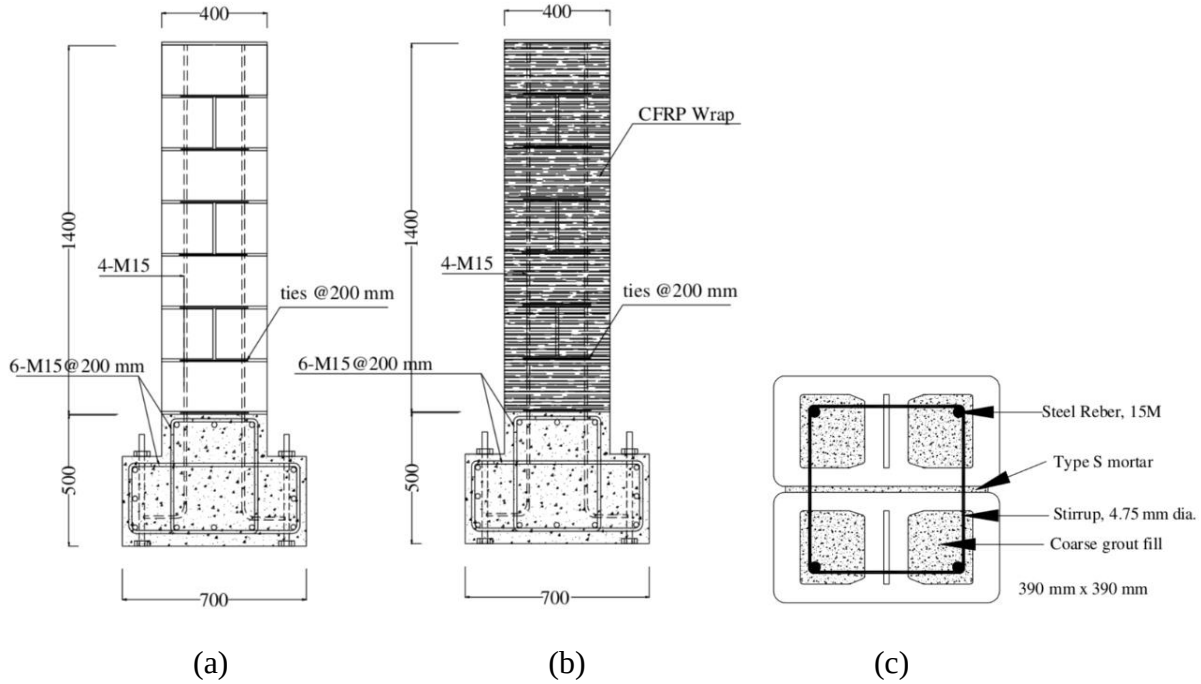


Figure 2.12 (a) Specimen RMC-0 (b) Specimen RMC-CW-1 (c) Cross-section of masonry column

Constant axial load of 200 kN was applied to the columns prior to lateral displacement reversals. Figure 2.12 shows the envelopes of hysteresis loops of tested columns. The retrofitted column with 3 layers of CFRP (RMC-CW-3) had the highest improvement in lateral load and displacement ductility capacities as compared to the other specimens. The lateral load capacity improvements for RMC-CW-1, RMC-CW-2, and RMC-CW-3 were 30%, 46%, and 53%, respectively. Moreover, the improvements in the lateral displacement ductility ratios for RMC-CW-1, RMC-CW-2, and RMC-CW-3 were 115%, 260%, and 273%, respectively. The strengthened columns with one layer of CFRP in their plastic hinge regions (RMC-CW-1p) or 1 layer of FRP strips, also demonstrated improved structural behaviour relative to the control specimen.

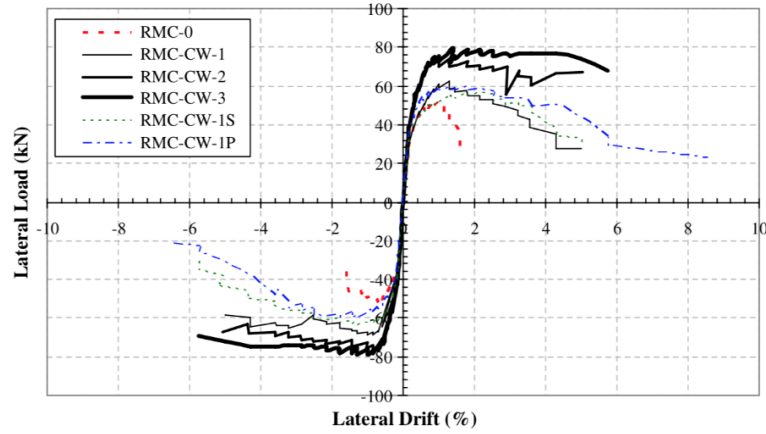


Figure 2.13 Envelopes of hysteretic loops of tested columns

Galal et al. (2017) experimentally investigated the effect of CFRP confinement on the uniaxial compressive stress-strain behaviour of block masonry columns. Nineteen half-scale masonry block specimens were constructed and tested with different configurations. Masonry block specimens consisted of two masonry blocks in thickness and five masonry blocks in height (Figure 2.13(a)). The blocks were connected together by 5 mm mortar. The investigated parameters were the number of CFRP layers and the corner radius of columns. Figure 2.13 (b) shows the strengthening stages with CFRP sheets.

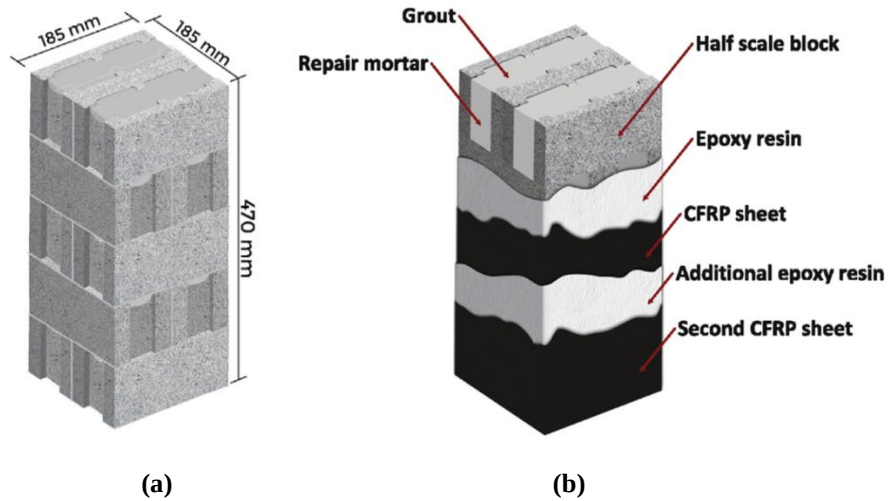


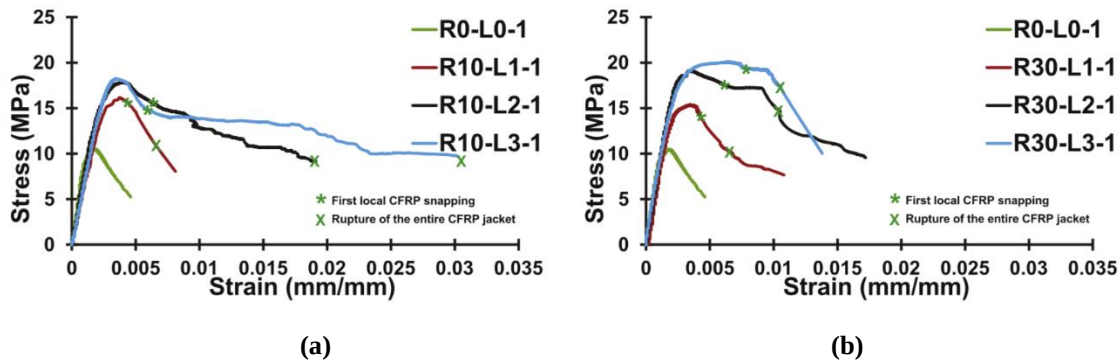
Figure 2.14 (a) Dimensions of fully grouted Prism, (b) Strengthening Steps

Table 2.3 includes the properties of all the tested specimens. Specimen R0-L0 was the as-built prism with no CFRP, and other specimens had 1 to 3 layers of CFRP with 10 mm or 30 mm corner radius. Three or two samples were tested from each set for averaging purposes.

Table 2.3 Properties of the specimens

Set	Corner radius (mm)	Number of CFRP layers	Number of Specimens
R0-L0	-	-	3
R10-L1	10	1	3
R10-L2	10	2	3
R10-L3	10	3	2
R30-L1	30	1	3
R30-L2	30	2	3
R30-L3	30	3	2

Figure 2.14 shows the axial stress-strain curves for one specimen from each set. Figure 2.14(a) is for specimens with 10 mm radius corners and Figure 2.14(b) is for specimens with 30 mm radius corners. The unconfined specimens demonstrated brittle failure with sudden drop in capacity. However, all the strengthened specimens had higher compressive capacity and improved post-peak behaviour compared to the unreinforced specimens. The structural response of masonry prisms improved by increasing the number of CFRP layers and also the radius of the corners. The implemented CFRP jacketing has dramatically improved the strain capacity of masonry prisms. The improvements in ultimate axial load and strain capacities were 79% and 281%, respectively.

**Figure 2.15 Axial stress-strain curves for (a) prisms rounded with 10 mm, (b) prisms rounded with 30 mm**

ElGawady et al. (2007) evaluated strengthening of URM walls by fiber-reinforced polymers (FRP). Seven half-scale masonry walls were tested, consisting of three unretrofitted as-built walls, three FRP repaired walls, and one undamaged retrofitted wall. All the specimens were tested under a constant gravity load with incrementally increasing in-plane loading cycles. The masonry walls were constructed with 0.5 and 0.7 aspect ratios. The test variable was FRP axial rigidity, defined as the amount of FRP multiply by its modulus of elasticity. Test results showed that lateral strength

and stiffness of retrofitted walls had direct relationship to FRP axial rigidity. However, retrofitted walls with high axial rigidity showed brittle failure. The test specimens were intended to simulate the mid-20th century buildings in central Europe. Table 2.4 provides the description of each specimen. The specimens with S and M letters at the beginning of their names represented moment/shear ratio as 0.7 and 0.5, respectively. Two types of FRP were used in the experimental study. Their properties are listed in Table 2.5.

The masonry walls were constructed on reinforced concrete foundations that were connected to the laboratory strong floor. The test setup is shown in Figure 2.15. An axial load of 30 kN was applied by two posttensioning cables. The self-weight of the reinforced concrete loading beam, and the masonry wall accounted for 9 kN, which produced a total normal stress of 0.35 MPa in the wall. The horizontal load was applied manually by means of two hydraulics jacks and hand pumps. The reference specimen S2-REFE-ST with effective moment-to-shear ratio of 0.67 had a lateral strength of 35.5 kN at 0.86% drift. The failure mode was a combination of rocking, sliding, and toe crushing.

Table 2.4 Summary of test specimens tested by ElGavadi et al (2007)

Specimen	Description
S2-REFE-ST	Reference specimen
S2-WRAP-G-F-ST	Specimen S2-REFE-ST after retrofitting with one layer of GFRP
M2-REFE1-ST	Reference specimen
M2-WRAP-G-F-ST	Specimen M2-REFE1-ST after retrofitting with one layer of GFRP
M2-2WRAP-G-F-ST	Virgin specimen retrofitted directly after construction with two layers of GFRP
M2-REFE2-ST	Reference specimen
M2-WRAP-A-F-ST	Specimen M2-REFE2-ST after retrofitting with two layers of AFRP

Table 2.5 FRP types and properties

Commercial name	Fiber Orientation	FRP Type	Wrap (gr/m ²)	f _t (MPa)	E (GPa)
SikaWrap-300A	Unidirectional	Fabrics of Aramid	300	2,880	100
SikaWrap-300G 0/90	Bidirectional	Fabrics of glass	145	2,400	70

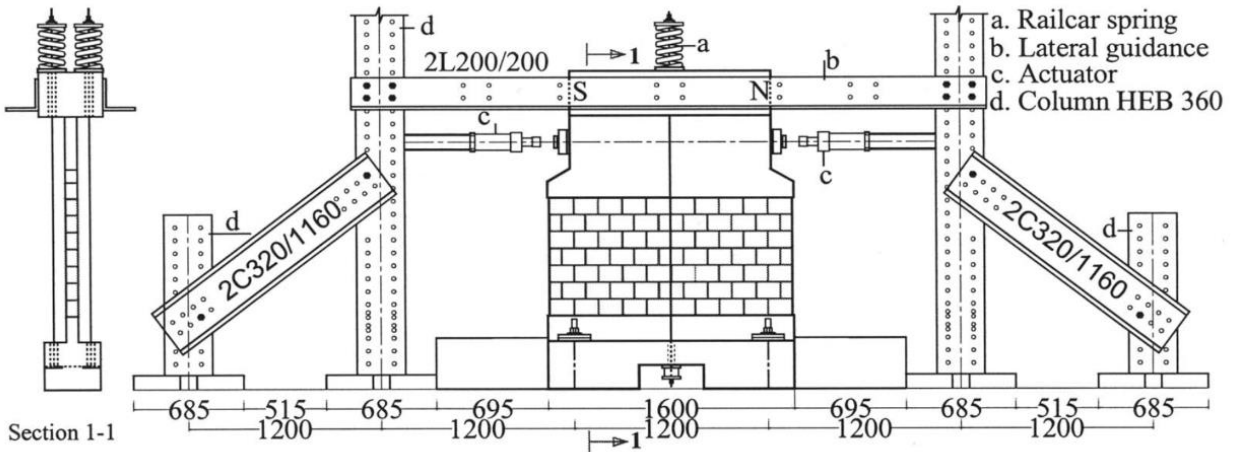


Figure 16.15 Test setup that was used by ElGavadi (2007) for masonry wall specimens

After testing specimen S2-REFE-ST, the wall was repaired by replacing three completely damaged bricks with the similar type of bricks and mortar. The specimen was then covered with one layer of glass fiber-reinforced polymer (GFRP) with an axial rigidity of 0.05 GPa. The repaired wall was tested as specimen S2-WRAP-G-F-ST. The specimen had a shear-flexural mode of failure with a lateral strength of 50.8 kN, which was equivalent to 1.4 times the original resistance of the wall. The FRP application was successful in keeping the masonry bricks together despite the heavy damage to the specimen.

Specimens M2-REFE1-ST and M2-REFE2-ST were both reference specimens with a moment-to-shear ratio of 0.5. Specimen M2-REFE1-ST was tested up to 0.08% drift and 25.1 kN lateral resistance, and then it was decided to improve the wall strength by increasing the posttensioning force. After increasing the axial load, the test continued, but sliding and rocking of reinforced concrete led to diagonal cracks that passed through the masonry units. Specimen M2-REFE2-ST showed several cracks up to 0.08% drift with a lateral strength of 28.2 kN. After the development of cracks, the specimen developed sliding, which resulted in 2.25% drift ratio.

Specimen M2-WRAP-G-F-ST was similar to specimen M2-REFE1-ST, which was retrofitted by one layer of GFRP with an axial rigidity of 0.05 GPa. The specimen developed initial cracks horizontally along the first joint when subjected to a lateral load of 59.8 kN. The cycling continued until the specimen reached its ultimate lateral capacity of 69.7 kN at a lateral drift ratio of 0.3%. At this point, tearing of the GFRP was observed over the entire wall length. This changed

the failure mode from rocking to sliding. The sliding continued up to 5.5% drift at which time the test was stopped.

Specimen M2-WRAP-A-F-ST was a retrofitted version of specimen M2-REFE2-ST with two layers of aramid fiber-reinforced polymer (AFRP). The first layer of FRP had fibers that were oriented vertically, and the second FRP layer had fibers oriented horizontally. Each layer had an axial rigidity of 0.28 GPa. The failure mode of the wall was shear in one direction and flexural in the other. The specimen failed at an ultimate lateral load capacity of 170 kN, corresponding to 2.44% lateral drift ratio.

Specimen M2-2WRAP-G-F-ST was a retrofitted version of the as-built specimen with a moment-to-shear ratio of 0.5. The retrofitting consisted of two layers of GFRP with an axial rigidity of 0.1 GPa. The specimen showed maximum lateral resistance of 95.2 kN at a drift ratio of 0.5%. At the end of the test, the FRP was torn over the wall length and the wall toes were heavily damaged.

The experimental research showed that FRP retrofitting was quite effective in increasing the in-plane lateral strength and stiffness of URM walls. The lateral strength of retrofitted masonry wall was almost linearly proportional to the FRP axial rigidity. The energy dissipation of retrofitted walls was higher than the reference walls. However, most of the energy dissipation was due to the friction between the masonry units, rather than deformations in FRP sheets. FRP sheets improved the seismic response of masonry walls by keeping the masonry units together.

Willis and Yang (2009) repaired four severely damaged full-scale masonry walls with FRP strips, and tested them in two-way bending. These walls were part of a group of eight walls that were tested earlier to evaluate the out-of-plane behaviour of masonry walls. Three of the four were rectangular and one was square. Wall configurations are shown in Figure 2.16. An axial load of 0.1 MPa was applied on Walls 2 and 4, but Walls 1 and 3 were tested without any axial load. The masonry units were clay bricks with two rows of five cores having dimensions of 230 x 110 x 76 mm.

Vertical strips of FRP were used to retrofit four masonry walls. Air bags were used to apply monotonic static out-of-plane loading. The effectiveness of CFRP and GFRP were studied and compared. The specimen properties and the test results are listed in Table 2.6. The application of FRP strips improved the load carrying capacity and ductility of masonry walls. The retrofitted

walls with CFRP showed a stiffer and less ductile response compared to the walls that were retrofitted by GFRP. This was attributed to the mechanical properties of CFRP, which was stiffer than those of GFRP.

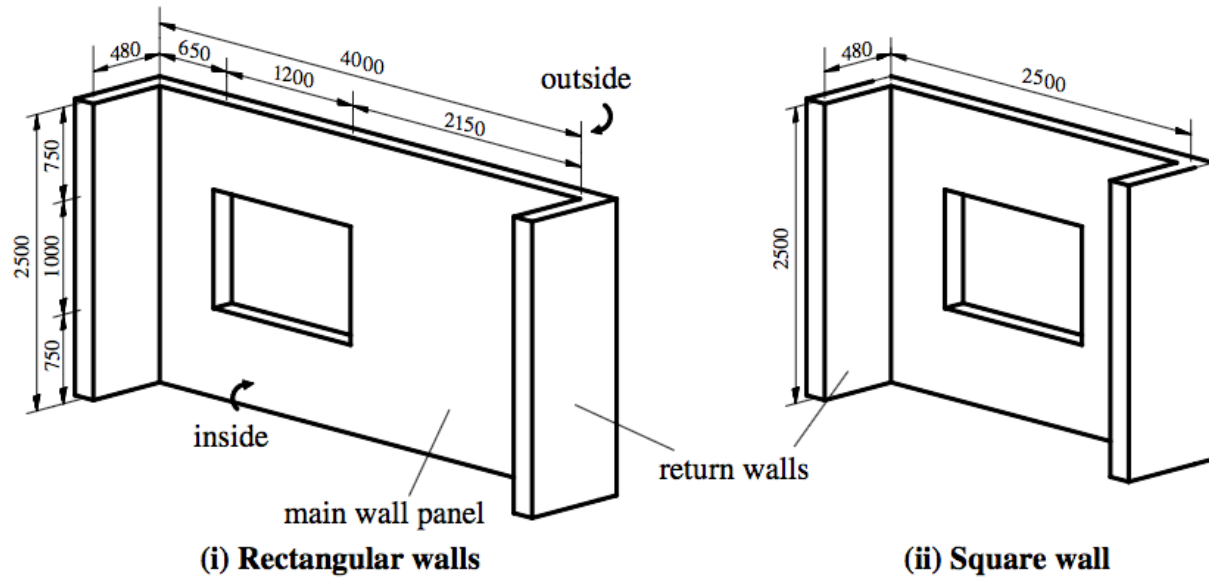


Figure 2.17 Rectangular and square masonry wall configurations (Willis and Yang 2009)

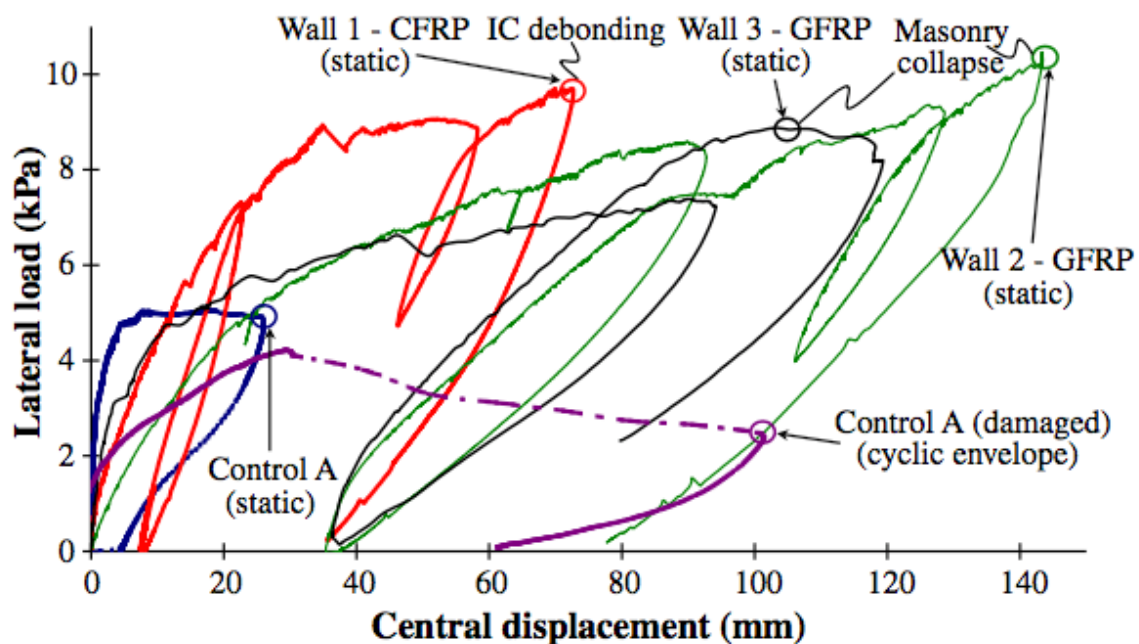


Figure 2.18 Load-deformation response of rectangular masonry walls

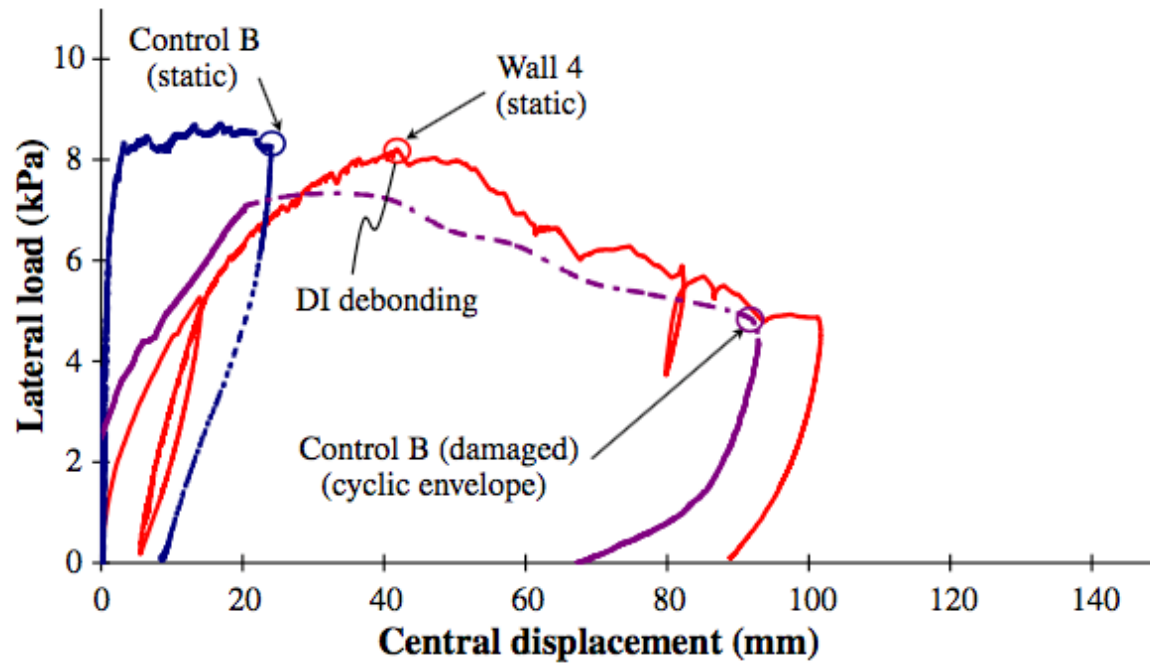


Figure 19.18 Load-deformation response of square masonry walls

Table 2.6 Wall specimen properties and test results

Specimen	Height (m)	Length (m)	Retrofitting Technique	FRP Strips Spacing (mm)	Maximum Load (kPa)	Maximum Displacement (mm)
Control A	2.5	4.0	-	-	5.1	25.2
Wall 1	2.5	4.0	Externally Bonded CFRP	650	9.7	72.7
Wall 2	2.5	4.0	Externally Bonded CFRP	650	10.5	143.5
Wall 3	2.5	4.0	Externally Bonded CFRP	500	12.1	119.4
Control B	2.5	2.5	-	-	8.7	26.5
Wall 4	2.5	2.5	Near-Surface Mounted	-	8.2	101.7

The load carrying capacity and rigidity of the masonry walls increased by the reduction of FRP strip spacing. The crack patterns were similar in retrofitted and control walls. The load deformation response of rectangular and square walls is shown in Figures 2.17 and 2.18.

Arifuzzaman (2013) investigated retrofitting of masonry walls by FRP sheets with new types of anchorage systems. A partially reinforced masonry (PRM) wall and an unreinforced masonry (URM) wall were constructed, retrofitted and tested to evaluate the new retrofit strategies. The new retrofit methods consisted of surface-bonded FRP sheets on one side of masonry walls, which were connecting to the foundation by means of FRP anchors or newly developed stainless-steel ductile sheet anchors. Two types of anchorage systems were developed: i) elastic anchors made from CFRP laminates for strength enhancement, and ii) ductile inelastic anchorage systems made from stainless steel sheets. Prior to the selection of the anchor type, a series of small-scale tests were performed with different materials and different setups to find the best-suited option for the anchorage system.

The elastic anchors consisted of FRP laminates that were rolled, folded over, and tied together by a string on one side (to be inserted into the predrilled holes), and flared out and epoxy glued to the wall on the other side. The process of anchoring FRP covered wall to the foundation for flexural reinforcement involved drilling of the concrete foundation near the footing-wall interface, and inserting folded part of the anchor inside the holes in concrete. The holes were then filled with epoxy. The flared ends of the anchors were adhered to the surface-bonded FRP sheets on the wall by means of epoxy. Figure 2.19 shows the elastic anchors with FRP laminates.



Figure 20.19 FRP anchors

The performance of elastic anchors was assessed by two sets of small-scale specimens. Bond between the FRP laminates and concrete was established through pull-out tests that consisted of 600 mm x 100 mm FRP sheets that were glued by epoxy in grooved concrete cylinders. The embedded length of FRP inside the concrete was 100mm. Figure 2.20 shows the setup of pull-out test for FRP laminates inside the concrete.



Figure 2.21 Pull-out test setup for FRP laminates

Three potential failure modes were anticipated: i) formation of concrete pull out cone, ii) rupturing of the FRP sheet, and iii) debonding of the FRP laminate from concrete. The test results justified the use of FRP anchors in terms of bond to develop full capacity in tension.

The use of the second type of anchor, i.e., ductile steel sheet anchor, was intended to provide ductility to FRP retrofitted masonry walls. The anchors were intended to transfer tensile forces in FRP sheets to the foundation. A list of possible products was investigated first prior to deciding on the ductile anchor sheets. These products included steel sheets, high-strength wire, hard-wire tape, and high strength ductile stainless-steel sheets. The bond characteristics of each product with FRP was evaluated by a series of small-scale specimens tested in tension.

The stainless-steel sheet was chosen as the preferred material based on its performance during tests. It was used as an anchor because of its higher ductility compared to regular steel sheets. Figure 2.21 shows the stress-strain relationship of stainless steel sheet that was obtained from a coupon test. The material showed 60% elongation as shown in Figure 2.22.

Small-scale masonry samples were tested to investigate the performance of stainless steel anchors further. The masonry samples consisted of one and a half masonry blocks that were placed on top of each other vertically with a 20M reinforcing bar inserted.

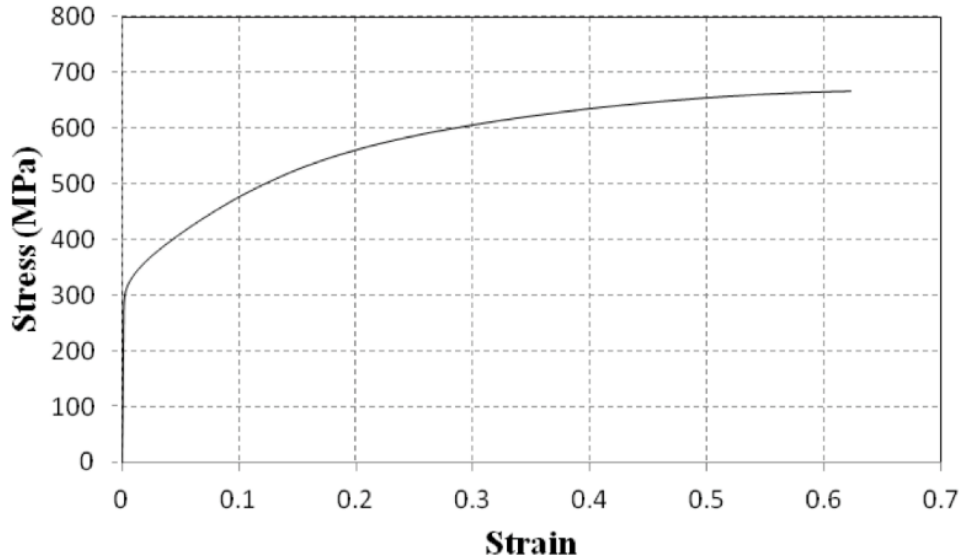


Figure 2.22 Stress-strain relationship of stainless-steel sheet



Figure 2.23 Elongation in stainless steel sheet (60%)

The hollow concrete blocks were filled with concrete and then the whole assembly was placed on top of a small concrete foundation. The small concrete foundations were cut adjacent to the masonry blocks to provide locations for steel anchors. Horizontal and vertical FRP sheets were applied to retrofitted masonry blocks. Stainless steel anchors were installed in three stages, first the bottom end of the anchor was placed inside the foundation and glued with epoxy. Then, the top end of the anchor was glued to the FRP surface by epoxy and clamped to the masonry till it cured. Finally, FRP sheets were epoxy glued horizontally and then wrapped around the masonry blocks vertically to cover the steel anchor. The steel plates were designed to yield prior to bond failure. Figure 2.23 shows different stages of sample preparation.

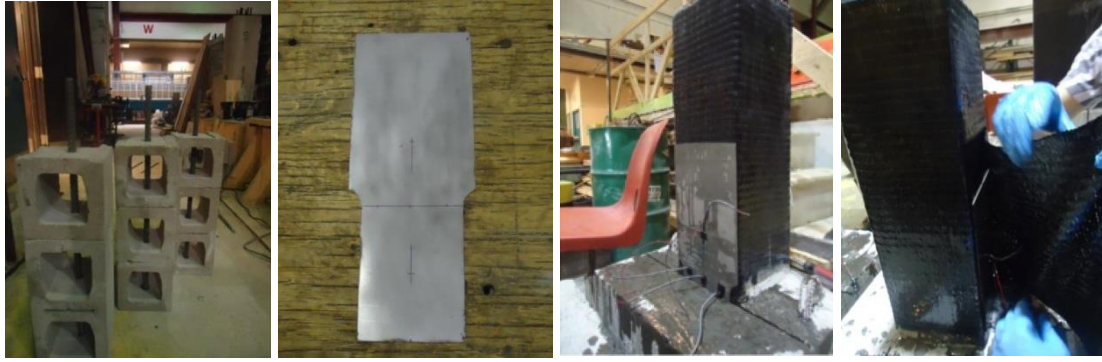


Figure 2.24 Preparation steps of test specimens for steel anchors

Universal testing machine was used to test the masonry samples. The specimens were tested under cycling loading of pull and push with 10 kN load increments. The steel plates yielded prior to bond failure, which showed the effectiveness of these anchors. The buckling of steel plate in compression in the FRP covered regions indicated the need for using bolts to clamp the anchors while reducing the buckling length.

The main research program consisted of two large-scale concrete masonry walls retrofitted and tested under simulated seismic loading. The walls were companion to the as-built walls tested earlier by Taghdi et al. (1998). The first wall was a partially reinforced masonry wall (PRM) and the second wall was an unreinforced masonry (URM) wall. The FRP sheets were applied on one side of the masonry wall to simulate a practical case of having access only to one side of the wall. Both PRM and URM walls were retrofitted by surface-bonded carbon FRP sheets for flexural and shear enhancements. Two layers of FRP sheets were surface bonded. The first layer was oriented to have the fibers in the horizontal direction. The second layer was applied on top of the first layer with its fibers aligned in the vertical direction. Figure 2.24 shows the application process of FRP sheets on masonry walls.

The partially reinforced masonry wall had two 15M longitudinal reinforcement that were anchored to the foundation and the top loading beam. The concrete cells where the reinforcement passed through were filled with concrete. Two boundary cells at both ends of the wall were also grouted to increase compressive capacity of the wall. Truss type reinforcements were also used horizontally at mortar joints to maintain vertical alignment of the walls. The height of the wall was 1.8 m and the shear span was 2.0 m. The retrofit consisted of two layers of carbon FRP sheets that were epoxy bonded in horizontal and vertical directions, on one side of the wall, as described

earlier. The vertical FRP sheet extended to the foundation and epoxy glued to the foundation concrete. In addition, the FRP anchors were used to prevent debonding of the FRP sheet from the foundation.



Figure 2.25 Application of FRP sheets on masonry wall

The URM wall was built using concrete blocks without any longitudinal reinforcement. Truss type reinforcement was placed horizontally at mortar joints. The two boundary cells were grouted at both ends to provide additional compressive strength, which was required after retrofitting the wall. This time a different type of anchor system was used to connect the FRP sheets to the foundation. Ductile anchors made from stainless steel sheets that had been evaluated earlier was applied. Two vertical slots were cut in the foundation at two locations at wall ends, adjacent to the wall. The steel sheet anchors were epoxy glued inside the slots while the portions above were epoxy glued to the wall FRP. Figure 2.25 shows the FRP and steel anchors on PRM and URM walls.



Figure 2.26 Installed FRP and ductile steel sheet anchors on PRM and URM walls, respectively

The specimens were cycled three times at each drift level. The retrofitted PRM wall showed elastic response up to 0.5% lateral drift, when the wall reached its maximum lateral load resistance. The maximum lateral load resistance of the wall was 165 kN in push and 174 kN in pull directions. The failure was triggered by debonding of the FRP sheets from the foundation near the extreme tension end. At 0.75% drift level, more anchor pull-out occurred and the lateral load resistance dropped to 138 kN in the push direction. Drop in load resistance in the pull direction was observed at 1.0% drift level. Figure 2.26 shows the hysteretic response of the retrofitted PRM wall.

The comparison of unretrofitted and retrofitted walls shows considerable improvement in the seismic behavior of retrofitted wall. The lateral load capacity of the wall was increased by 40% to 50% depending on the direction of loading. Improvements in energy dissipation was also observed due to the application of FRP as a retrofit strategy. The retrofitted URM wall had elastic behavior up to 0.25% drift ratio. Significant yielding was observed in the steel sheet anchors up to this drift level.

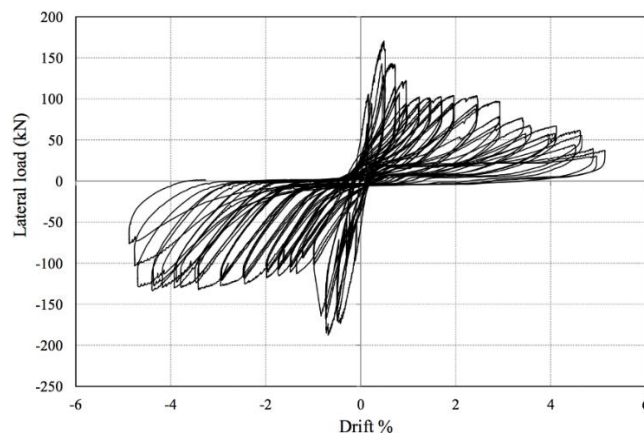


Figure 27.26 Hysteretic response of the retrofitted PRM wall

The peak load resistances of 224 kN and 211 kN were attained at 0.5% drift ratio in push and pull directions, respectively. The force resistances remained essentially the same throughout the experiment. During 1.5% drift cycles the wall started twisting along two diagonal cracks, which made the experiment unsafe to be continued in the push direction. The deformation cycles continued in the pull direction up to 3.5% drift ratio, which led to a significant damage. Figure 2.27 shows the force-drift response of retrofitted URM wall. The comparison of retrofitted wall response with the unretrofitted wall tested by Taghdi et al. (1998) indicated 4-fold increase in lateral load capacity. The applied retrofit system provided higher ductility and energy dissipation

as compared to the as-built wall. A comparison between the retrofitted URM wall by steel anchors and retrofitted PRM wall by FRP anchors indicated the superiority of steel anchors over FRP anchors. Even though the PRM wall had longitudinal steel reinforcement, it had lower lateral load capacity when compared with the URM wall. The lateral load capacity of the PRM wall dropped significantly throughout the test while URM wall maintained its load capacity until the end of the test. However, URM wall experienced an out of plane twist during the test, which may be attributed to the mild nature of the steel compared to the brittle nature of the FRP sheets.

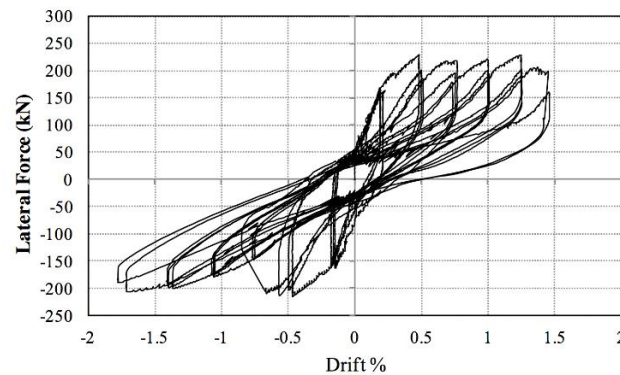


Figure 2.28 Hysteretic response of the retrofitted URM wall

Ingham and Mahmood (2011) performed diagonal compression tests on seventeen FRP retrofitted solid clay brick wallettes with different FRP types and patterns. The results were compared with those of five companion unretrofitted walls. Table 2.7 and Figure 2.28 show the wallette dimensions and FRP retrofit details. Test materials were selected to replicate the old masonry URM structures in New Zealand. Clay bricks were obtained from old buildings and the ASTM type O mortar was used which consisted of one-part cement, two-parts hydrated lime, and nine-parts sand by volume. The test setup consisted of a steel I-beam as the foundation for the wall. Diagonal force was applied by means of a manually operated hydraulic jack to simulate diagonal compression. Figure 2.29 shows the details of the test setup.

The failure mode in unretrofitted walls consisted of diagonal cracks accompanied by sliding on stepped cracks. The FRP retrofits improved the shear capacity of masonry walls up to 325%. Figure 2.30 and 2.31 show the crack patterns and shear stress (ν) versus percentage drift (δ) response for the FRP retrofitted walls. The thick lines in Figure 2.31 show the retrofitted wall response and the thin lines show the response of unretrofitted walls. The application of FRP on

URM resulted in improved shear capacity and pseuductility, especially for walls that were vulnerable to diagonal tension failure. The summary of observations is listed as follows:

- The application of horizontal full surface fabrics or diagonal CFRP plates is recommended as the best combination for improved pseuductility.
- The use of FRP on one side of the wall was effective in improving shear capacity. However, a large out of plane deformation was observed.
- CFRP plate showed higher debonding compared to other materials.
- Horizontal FRP application was not effective in controlling the sliding deformation on walls with a weak mortar. On the other hand, vertical or diagonal FRP retrofit was able to mitigate this mode of failure.

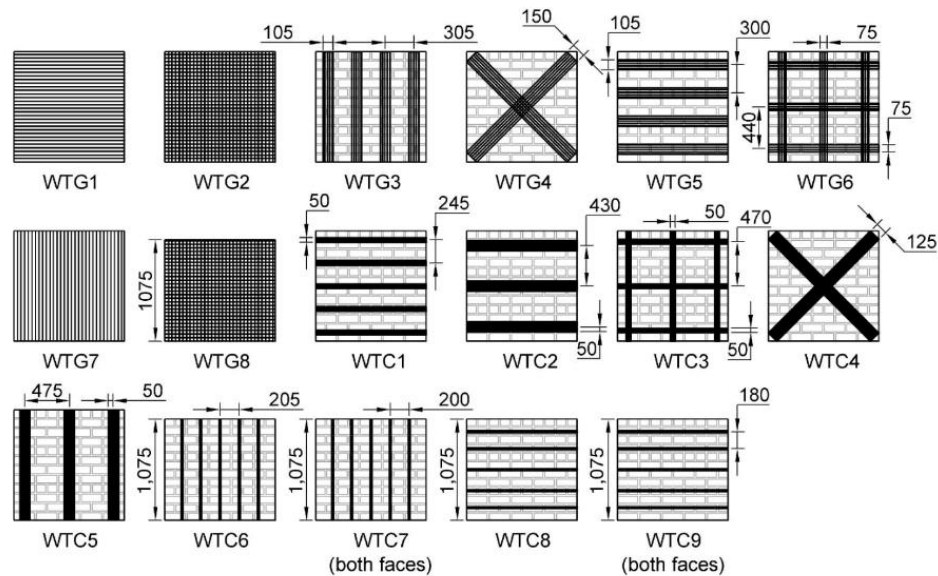


Figure 2.29 FRP retrofitted solid clay brick wallettes with different FRP types and patterns

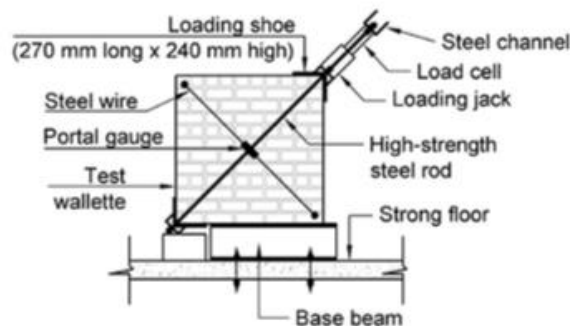


Figure 30.29 Shear test setup for clay brick wallette

Table 2.7 Masonry wallette dimensions and retrofit details

Stage	Wallette	Wallette dimension			Number of layers shown within parameters	Pattern
		L (mm)	H (mm)	T (mm)		
1	AP2	1,170	1,170	225	-	-
	AP4				-	-
	WTG1				Full surface GFRP fabric (2)	HL
	WTG2				Full surface GFRP fabric (2)	HL+V
	WTG3				Four 150 mm wide strips cut out of GFRP fabric (2)	HL
	WTG4				Four 150 mm wide strips cut out of GFRP fabric (2)	D
	WTG5				Four 150 mm wide strips cut out of GFRP fabric (2)	V
2	AP6	1,170	1,170	225	-	-
	AP7				-	-
	WTC1				Five 50 mm wide CFRP plates(1)	HL
	WTC2				Five 50 mm wide (pairs of two) CFRP Plates(1)	HL
	WTC3				Five 50 mm wide CFRP plates(1)	G
	WTC4				Five 125 mm wide CFRP plates(1)	D
	WTC5				Six 50 mm wide CFRP plates (1)	V
	WTC6				Six 75 mm wide strips cut out of GFRP	G
	WTC7				Two full surface GFRP fabrics (2)	V
3	AP8	1,170	1,075	225	-	-
	WTC6				Five 1.2 mm x 15 mm NSM bars (1)	V
	WTC6				Five 1.2 mm x 15 mm NSM bars (1)	V
	WTC8				Five 1.2 mm x 15 mm NSM bars (1)	HL
	WTC9				Five 1.2 mm x15 mm NSM bars (1)	HL

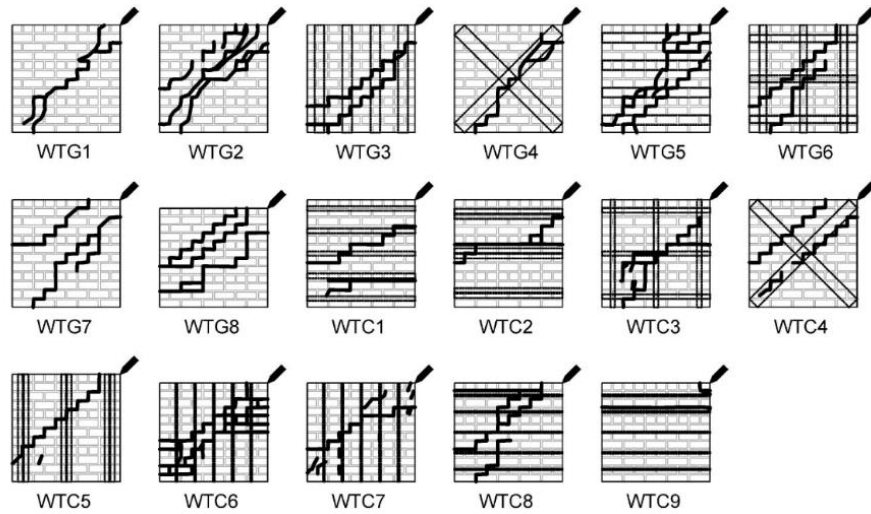


Figure 31.30 Crack patterns of FRP retrofitted masonry walls

Aiello and Sciolti (2006) investigated bond characteristics between FRP sheets and typical natural stone used in stone masonry structures in southern Italy. Two types of CFRP sheets and two types of masonry blocks were tested in their study. Bond stresses at the interface were measured and the effects of geometry and bond length were evaluated. The experimental investigations were carried out on Naples tuff and Lecce stone specimens reinforced by glued FRP on their surface. The test variables were bond length, type of stone, strengthening procedure, specimen geometry and type of bond test. The test results mainly depended on the type of stone, the position of the bonded length, and the geometry of the specimen.

Two types of bond tests were performed to evaluate the shear capacity of bonded interface of FRP and stone. The first type was a specially designed test, while the second one was performed by a commercially available machine. The first type of test was a double lap shear test, which consisted of two sheets of reinforcement bonded on two opposite sides of ashlar. The test was performed in the displacement mode with a speed of 0.2 mm/min. Figure 2.32 illustrates the test apparatus for the double lap shear test. The variables of double lap shear test consisted of specimen geometry, bond length and position, and the strengthening typology. Two ashlar sizes were used as illustrated in Figure 2.33 with 100 mm and 200 mm widths. Bond lengths of 70 mm and 150 mm were tested on both sizes of specimens. Transverse FRP strip anchors were also tested on specimens with 150 mm bond length. The effect of bond length position was studied on specimens with 100 mm length and 150 mm bond length. Figure 2.34 illustrates the details of specimens with different bond length positions. The free ends of FRP sheets were embedded between two steel

plates to prevent premature failure. The second type test was performed by a commercially available testing machine, called Direct Shear Strength Test (DSS-test). The mean bond strength of CFRP strip, bonded to ashlar surface was measured by DSS-test.

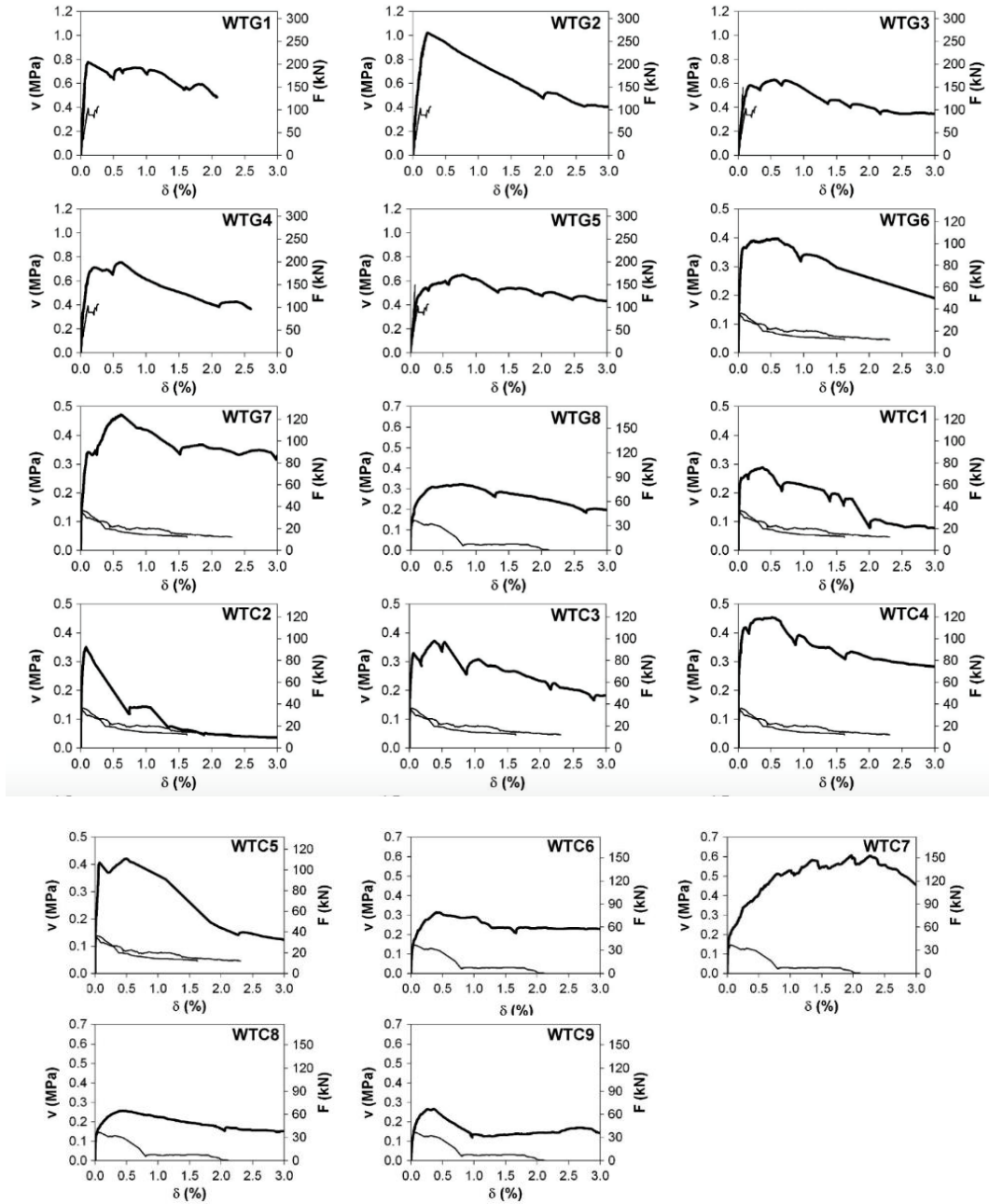


Figure 32.31 Shear stress versus percentage drift response for masonry walls

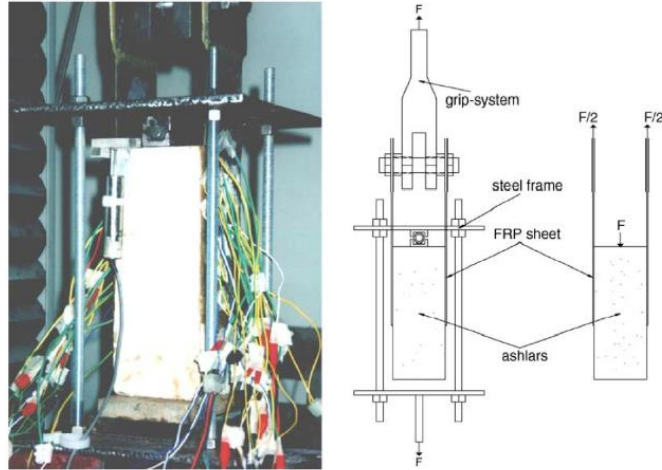


Figure 2.33 Test setup of double lap shear test

The test results are summarized below:

- The ultimate bond strength was mainly influenced by the properties of substrate. The flexural strength of the substrate was an important factor. The Leccese stone had 50% higher maximum bond stress at the loaded end compared to the Naples tuff.
- The failure mode of Leccese stone specimens was accompanied by delamination and detachment of a thin layer of stone while the failure mode of Naples tuff specimens took place with crushing of a thick layer of stone.
- Naples tuff had a higher transfer length when compared with Leccese stone. This behavior was justified by the rough fracture surface of Naples tuff, which produced interlocking mechanism at the surface and stress transfer over a wider area.

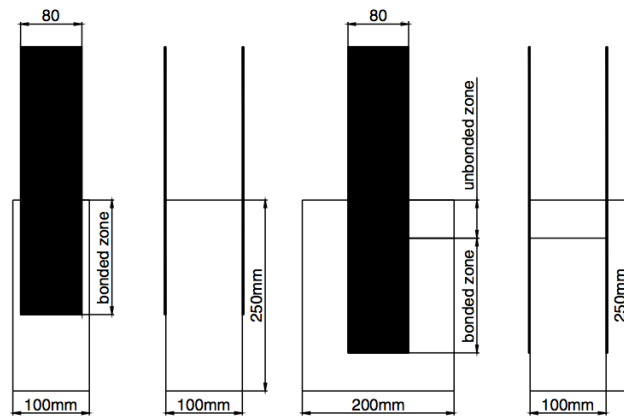


Figure 2.34 Specimens geometry for double lap shear test

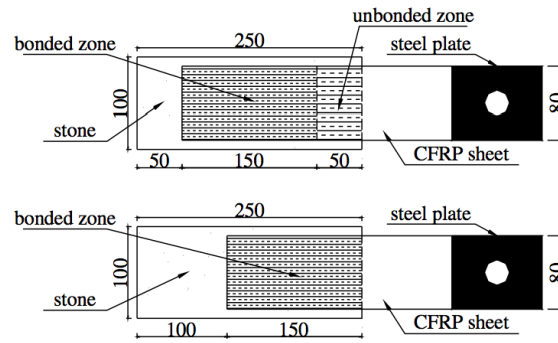


Figure 2.35 Bond length positions in double lap shear test

Aiello et al. (2007, 2009) also investigated the mechanical behaviour of FRP confined circular and square masonry columns which was very effective in terms of strength and ductility.

2.1.3.3 Use of Polyurea

Polyurea is a type of an elastomer that is derived from the reaction product of an isocyanate component and a synthetic resin blend component through step-growth polymerization. Polyurea spray technology was recently developed as a coating system. Polyurea spray coatings are very suitable for applications in construction applications. The fast curing makes it possible to use it when only very short disturbance periods are allowed. (Broekaret 2003). Polyurea is a fast curing (even at very low temperatures) compound that provides water and humidity resistance, chemical resistance, and physical properties such as high hardness, flexibility, and tear strength. These properties make polyuria spray coating a suitable product for many applications in industry. Some of the conventional polyurea applications include roof coatings, pipe protection, tank coating, and truck bed liners. (Broekaert 2003). The application of polyurea has recently expanded to include retrofit of buildings and structures. The following literature review focuses on external application of polyurea to improve flexural and shear capacities of non-ductile masonry walls.

Tanizawa et al. (2009) developed a new retrofit material for infill wall systems. This new material was made from class-c fly ash and wood fibers as raw materials. They studied out-of-plane behavior of URM infill walls made from the new material. As part of the project, an external strengthening system, consisting of elastomeric polyurea surface coating and chopped E-glass fibers, was tested. The test set-up consisted of two steel shoes at the corners of the specimen that were connected together by steel bars. Diagonal in-plane load was applied by a hydraulic jack through steel shoes, as shown in Figure 2.35. The diagonal compressive shortening and diagonal

tensile expansion of the walls were measured by two LVDTs.

Five masonry walls were constructed by using the new masonry units with masonry unit dimension of 102 x 67 x 241 mm. The wall dimensions were 1219 x 1219 x 102 mm. The first masonry wall (P2-1) was representative of unretrofitted as-built wall, while the other four walls were tested as retrofitted walls. Wall P2-2 was retrofitted by polyurea surface coating on the entire surface of one side of the wall. Wall P2-3 was retrofitted similar to wall P2-2, but on both sides of the wall. Walls P2-4 and P2-5 were retrofitted by polyurea and chopped E-glass fibers on double sides of the walls. Table 2.8 shows the summary of test results, including the compressive strength of masonry f_{mt} (based on ASTM C 1314-07) and the maximum applied load.



(a) Wall P2-1

(b) Wall P2-3

Figure 2.36 In-plane load tests of the alternative URM walls

The tests indicated that the application of polyurea increased the in-plane load capacity of the wall by a factor of 2.52 and 5.86 depending on whether polyurea was applied on one or both sides of the wall, respectively. The application of polyurea and E-glass fibers on both sides of Walls P2-4 and P2-5 increased the in-plane load capacity by 7.14 and 6.94 times that of the as-built wall. Shear stress - shear strain relationships for walls were calculated based on ASTM 519-07 and are depicted in Figure 2.36. The relationship for Wall P2-2 is not complete since out-of-plane deformations occurred during the test and the diagonal expansion of the wall was not measured. The maximum shear stress of Walls P2-3, P2-4, and P2-5 were 6, 7 and 7 times that for wall P2-1, respectively. The ductility index for each wall was computed as the area under the shear stress-strain curve. The ductility index of Walls P2-3, P2-4, and P2-5 were found to be 17, 28, and

36 times that for the as-built wall, respectively.

Six unreinforced concrete masonry and five clay brick masonry walls were tested by Silva et al. (2008) to evaluate the retrofit of walls by horizontal and vertical GFRP grid reinforced polyurea strips. The retrofit variables for each wall included number of strips, direction of layout, and the strengthened sides of walls (one side or both sides). The test setups consisted of two steel bars that were connected to the corners of the walls on both sides by steel shoes. Two hydraulic jacks were connected to the steel shoes to apply in-plane loads. Diagonal compressive and tensile deformations were recorded by LVDTs. The aspect ratio of all walls tested was 1.0. Concrete masonry block walls had dimensions of 64 x 64 x 6 in, and fully grouted clay brick walls had dimensions of 48 x 48 x 3-5/8 in. All the walls were built with running bond pattern. Table 2.9 illustrate the strengthening scheme.

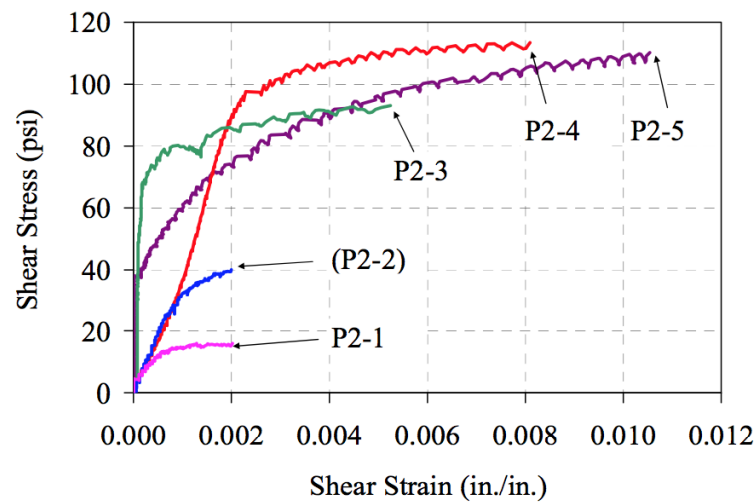


Figure 2.37 The shear stress and shear strain

Table 2.8 Summary of test results

Wall	Compression Strength of Masonry f_{mt} (MPa)	Maximum Applied Load (kN)	Retrofit Scheme
P2-1	4.7	19.19	As-built
P2-2	4.3	48.38	One side, polyurea
P2-3	5.4	112.4	Double sides, polyurea
P2-4	3.6	137.0	Double sides, polyurea and E-glass
P2-5	4.5	133.2	Double sides, Polyurea and E-glass

Table 2.9 Strengthening Schemes

Wall	Type	Directional Layout	Sides being strengthened
WC0a	Concrete Masonry	None	-
WC0b	Concrete Masonry	None	-
WC1	Concrete Masonry	Horizontal	Single
WC2	Concrete Masonry	Horizontal	Double
WC3	Concrete Masonry	Vertical	Single
WC4	Concrete Masonry	Vertical	Double
WK0	Clay Bricks	None	-
WK1	Clay Bricks	Horizontal	Single
WK2	Clay Bricks	Horizontal	Double
WK3	Clay Bricks	Vertical	Single
WK4	Clay Bricks	Vertical	Double

The application process of GFRP reinforced polyurea was as follows:

1. Covering of the wall by thick plastic sheets are not to be sprayed by polyurea.
2. Spraying of polyurea in designated areas.
3. Attachment of GFRP grid on top of the sprayed polyurea prior to curing.
4. Spraying of second layer of polyurea over the GFRP grid.

The overall thickness of each GFRP reinforced polyurea strip was 6.35 mm ($\frac{1}{4}$ in). Each strip was 114 mm ($4\frac{1}{2}$ in) wide. The distances between polyurea strips were 400 mm (16 in) and 300 mm (12 in) for the concrete block walls and clay brick walls, respectively. Table 2.10 shows the experimental results for the walls. In this Table, F represents the diagonal failure load that was applied by hydraulic jacks, and V represents the horizontal component of F. The as-built walls, WC0a, WC0b, and WK0 had typical shear friction failure mode. Wall WC1 also failed in a typical shear friction mode, similar to the as-built walls. However, horizontally reinforced polyurea on one side of the wall increased the shear capacity. The cracks were initially observed on non-retrofitted side of the walls that were strengthened on one side. Wall WC2 initially cracked like shear friction failure, but as the load increased it failed in sliding shear mode. This result indicated that the GFRP horizontal strips with polyurea were effective in controlling the shear friction failure, but they were not effective in restricting the sliding shear failure.

Table 2.10 Experimental results of the tested walls

Wall	Type	Failure mode	F (kips)	V (kips)
WC0a	Concrete Masonry	Shear friction	24.3	17.2
WC0b	Concrete Masonry	Shear friction	38.1	26.9
WC1	Concrete Masonry	Shear friction	43.0	30.4
WC2	Concrete Masonry	Shear slide and friction	53.5	37.8
WC3	Concrete Masonry	Shear friction	34.8	24.6
WC4	Concrete Masonry	Shear friction	50.6	35.8
Wall 5	Concrete Masonry	Shear slide		29.7
WL3	Concrete Masonry	Shear friction		33.1
WK0	Clay Bricks	Shear friction	42.0	12.9
WK1	Clay Bricks	Shear slide and friction	46.8	23.7
WK2	Clay Bricks	Shear slide	18.2	25.7
WK3	Clay Bricks	Shear friction and tension	29.8	21.1
WK4	Clay Bricks	Tension with localized crushing	33.4	23.6
SL1	Clay Bricks	Shear friction	29.2	20.7

Wall WC3 failed in the shear friction mode. The vertical strips of reinforced polyurea increased the shear capacity of the wall compared to the as-built walls. Vertical strips were less effective than horizontal strips in increasing shear capacity. Wall WC4 also failed in the shear friction mode and retrofitting the wall on both sides did not change the mode of failure (unlike the horizontal strip case). The increase in the number of vertical strips increased the shear capacity of the wall compared to Wall WC3. The increase in shear capacity due to retrofitting on both sides was more than double the shear capacity of the wall with only one side retrofitted.

Greene and Myers (2012) investigated bond behavior between polyurea and concrete for retrofit applications. The failure modes and direct tensile pull-off strengths of four different systems were examined. These systems included XS 100 and XS 430 polyurea materials, with and without E-glass fibers. The effectiveness of primed surface on bond capacity between the polyurea and concrete was also studied. Eight concrete specimens were cast with compressive capacity of 27.6 MPa as substrate surface. A 5 mm layer of each polyurea system was applied on two concrete

specimens. The concrete surface remained untreated to provide the worst case scenario. One side of concrete specimens was unprimed to be compared with the primed side of the specimens. The DYNA Z pull-off tester with digital manometer (Figure 2.37) was used to measure the bond capacity of polyurea and concrete interface. Twelve pull-off holes were cored to the depth of 6.4 mm on polyurea coatings. The pull-off discs adhered at their place by a heavy-duty epoxy. In order to provide better attachment, the polyurea surface was sanded prior to the application of epoxy.

Four modes of failure were observed during the tests. Failure mode 1 consisted of failure within the concrete substrate. This was the most desirable mode of failure, which shows that bond capacity was higher than the concrete tensile capacity. Failure mode 2 was a combination of failure in concrete and the bond failure in concrete and polyurea interface, which was the most observed failure type during the tests. Failure mode 3 consisted of failures that occurred exclusively in polyurea. The XS 100 system with fibers had the highest rate of failure in mode 3. This can be attributed to discontinuity within the system due to the fibers. Failure mode 4, the most undesirable type of failure, consisted of complete removal of polyurea from the concrete surface at the interface. This failure was limited to number of systems with XS 100 without the fibers.



Figure 2.38 DYNA Z pull-off tester with digital manometer

The average stress of pull-off test results is presented in Table 2.11. Side 1 in all specimens had a polyurea coating without the primer. Sides 1 and 3 were top surfaces of the specimens and they were troweled during the finishing process. Sides 2 and 4 were cast against the form on the bottom face. The test results showed that the application of primer improved the bond capacity of polyurea on concrete. The highest average pull-off strength was observed in the XS 430 system with no fibers present. The addition of E-glass fibers reduced the tensile bond capacity of each polyurea system. The percentage of concrete in pull-off tests is a good indicator of bond

effectiveness. Figure 2.38 compares the pull-off stress level with the percentage of concrete that pulled off. The XS 430 system had a higher percentage of concrete as compared to XS 100 system.

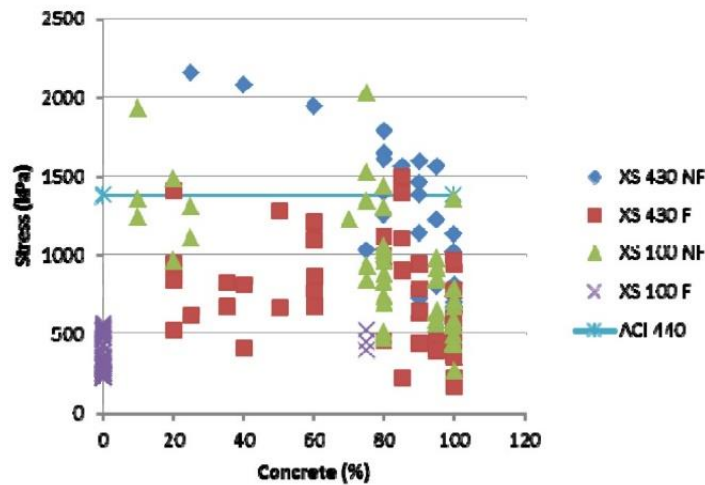


Figure 2.39 Comparison of the pull-off stress level with the percentage of concrete that has pulled off

Table 2.11 The average stress of pull-off test results

	XS 430 NF				XS 430 F			
	Side 1	Side 2	Side 3	Side 4	Side 1	Side 2	Side 3	Side 4
Average Stress (kPa)	525	1702	764	1342	501	830	1000	543
	XS 100 NF				XS 100 F			
x	Side 1	Side 2	Side 3	Side 4	Side 1	Side 2	Side 3	Side 4
Average Stress (kPa)	831	868	531	1409	365	395	321	409

Myers et al. (2008) studied the long-term durability of polyurea systems exposed to harsh environments. The effect of freeze-thaw and de-icing agents on polyurea confined concrete was examined through experimental research. The compressive strength, stiffness, and durability of polyurea sprayed cylinders subjected to accelerated environmental conditioning were compared with unconditioned cylinders with and without the polyurea confinement. Three concrete mixes were used to evaluate the effect of concrete type on conditioning of polyurea coated concrete. The concrete design mixes were high-strength lightweight concrete (LWC), high-strength high-performance concrete (HSC), and normal-strength concrete (NSC). Sixteen 4 in. diameter, 8 in

high cylinders were cast from each concrete. Each group of sixteen cylinders consisted of four control cylinders (C), four cylinders with polyurea confinement (PC), four polyurea confined cylinders with two-week freeze-thaw cycles (PSD), and four polyurea confined cylinders with four-week freeze-thaw cycles (PLD). The thickness of epoxy-based polyurea was 1.6 mm, which was applied by skilled technicians. The stress strain properties of polyurea, based on coupon tests, were obtained as presented in Table 2.12. The polyurea behaved as almost a bi-linear material with a linear-elastic region and a plastic region.

Table 2.12 Stress strain properties of polyurea

Stress at Full Yield (MPa)	Modulus of Elasticity (GPa)	Strain at Full Yield	Ultimate Stress (MPa)
4.00	0.083	0.145	6.89

The freeze-thaw cycles were chosen based on the ASTM C 666 freeze-thaw (F-T) temperature profile shown in Figure 2.39. De-icing was the second environmental mechanism that was simulated by spraying a salt solution of 7.5% by weight on the specimens three times a week to saturate the specimen surface. The presence of salt solution during the freeze-thaw cycles can affect the concrete strength and also the bond between concrete and polyurea. The researchers reported that four-week exposure cycle was equivalent to two decades of field exposure in central Missouri, USA.

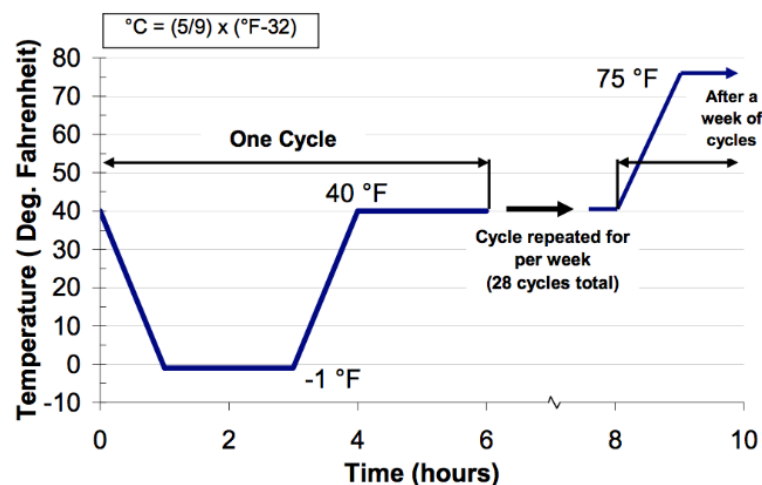


Figure 40.39 The freeze-thaw cycles

One cylinder in each group of four was tested to ultimate compression failure. The remaining specimens were loaded to 50% of the failure capacity of the first specimen and then unloaded to establish the modulus of elasticity. Following two cycles at 50% of capacity, the cylinders were loaded to their ultimate capacity. The instrumentation used consisted of a compressometer to measure axial strains. Two strain gauges were also attached to the cylinders in vertical and horizontal directions. The experimental results were evaluated in terms of modulus of elasticity, axial compressive strength, hoop strain, and bond retention. The polyurea increased the modulus of elasticity of specimens by 2.8%, 13.2%, and 8.8% for the NSC, LWC, and HSC respectively. The increase in modulus of elasticity by polyurea was more effective in concrete mixes with higher compressive strength. Polyurea provided a modest elastic confinement that had little effect on cylinder stiffness. Modulus of elasticity of concrete decreased due to prolonged conditioning. Not only the application of polyurea did not increase the compressive strength of concrete, but also a small loss of strength was observed. This can be attributed to either the normal variance for concrete or the polyurea effect on strength gain of concrete. Since a primer was not applied in this research study, polyurea may have absorbed the moisture of concrete and affected the process of concrete strength gain. A significant loss of strength was observed in HSC and LWC specimens that were subjected to four week conditioning (PLD). The following figures show the effect of conditioning on different types of concrete.

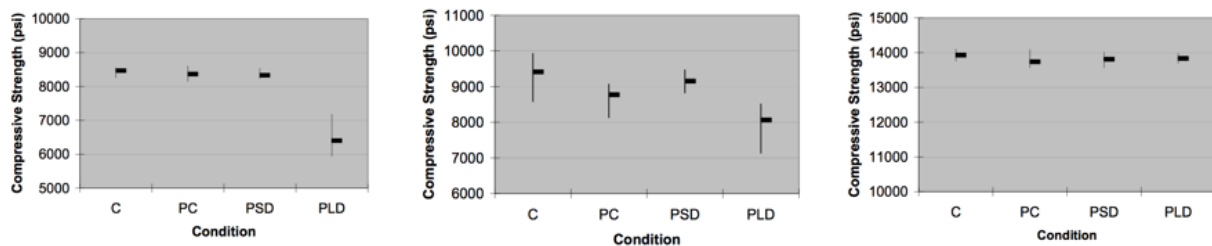


Figure 2.41 The effect of conditioning on different types of concrete

The Polyurea coat showed very high ductility in the hoop direction. The environmental conditioning had little effect on the stress-strain relationship of polyurea. However, a significant loss of hoop ductility was observed in NSC and LWC concrete mixes due to the exposure to PLD conditioning. The polyurea coating demonstrated strong bond with concrete. Figure 2.41 shows that the polyurea bond with concrete remained even after testing. Moreover, the bond between polyurea and concrete remained unaffected after conditioning. Polyurea coating was able to keep

concrete fragments in place even after the failure of the core concrete.



Figure 2.42 A concrete cylinder protected by polyurea

2.1.3.4 Prestressing

Ingham (2001) investigated the application of post-tensioning in masonry structures. The main focus was placed on in-plane seismic response of post-tensioned concrete masonry (PMC) walls with unbounded tendons. The researchers studied the effect of aspect ratio, thickness, and tendon layout on seismic behavior of single-storey masonry walls by testing eight walls. Table 2.13 shows the properties of all the wall specimens tested.

Table 2.13 Properties of wall specimens tested by Ingham et al (2001)

Wall	Length (m)	Thickness (mm)	Self-Weight (kN)	Masonry Strength (kN)	Number of Tendons	Initial Stress (MPa)	Axial Force Ratio
1.FG:L3.0-W20-P3	3.0	190	50	13.3	3	468	0.083
2.FG:L3.0-W15-P3	3.0	140	41	15.1	3	555	0.115
3.FG:L3.0-W15-P2C	3.0	140	41	20.61	2	757	0.077
4. FG:L3.0_W15-P2E	3.0	140	41	17.8	2	757	0.089
5. FG:L1.8-W15-P2	1.8	140	31	20.5	2	534	0.092
6. FG:L1.8-W15-P3	1.8	140	31	18.4	3	614	0.167
7. PG:L3.0-W15-P2	3.0	140	32	15.5	2	176	0.070
8. UG:L1.8-W10-P2	1.8	90	10	15.0	2	308	0.162

The first two letters in the specimen label indicated the grout level: FG fully grouted, PG partially grouted, UG un-grouted. All the masonry walls were prestressed with two or three vertically placed threaded rods. The test set-up consisted of reusable foundation and loading beam. A portion of prestressing force was considered as axial force. Figure 2.42 shows the test set-up of a strengthened masonry wall with three prestressing threaded rods.

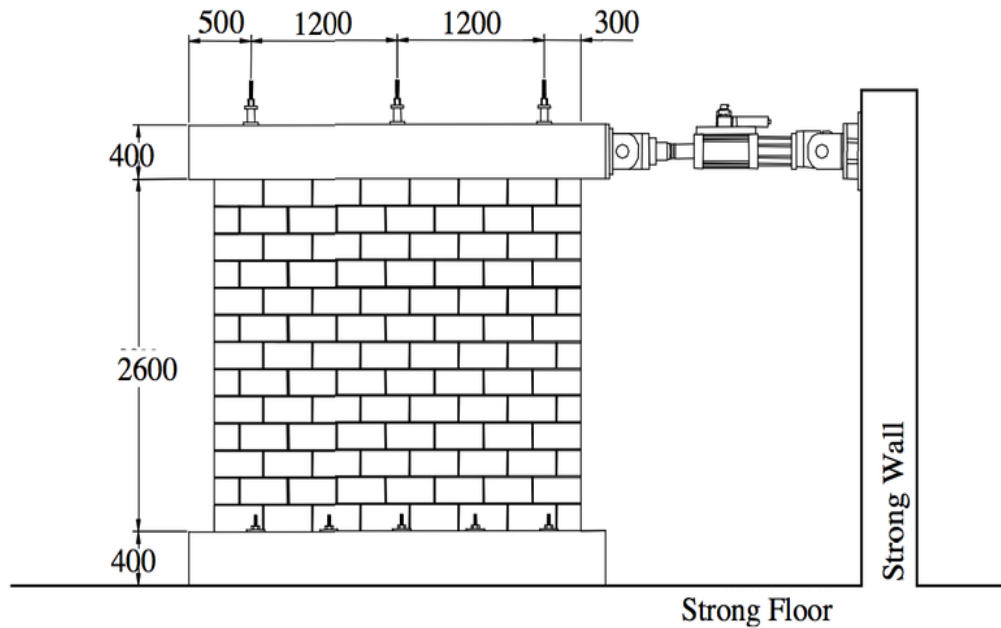


Figure 2.43 Test set-up of prestressed masonry walls tested by Ingham et al. (2001)

Table 2.14 presents the test results of all eight specimens that were tested by cyclic displacement reversals. Flexural and shear capacities of the walls were analytically calculated based on the New Zealand Masonry Code and are expressed as V_f and V_s in the Table. V_{max} is the maximum lateral force and d_{vmax} is the corresponding displacement. d_u is the ultimate recorded displacement, which is the wall displacement at 80% decay in V_{max} . The ultimate drift capacity of the wall was $g_u = d_u / h_e$.

Table 2.14 Test results of all specimens tested by Inham et al (2001)

Wall	Predictions		Test Results				
	$V_f(kN)$	$V_s(kN)$	$V_{max}(kN)$	$d_{vmax}(mm)$	$d_u(mm)$	$g_u(\%)$	Behaviour
1.FG:L3.0-W20-P3	336	567	561	29.2	>38	>1.4	Rocking, compr. Toe crushing
2.FG:L3.0-W15-P3	367	504	465	5.3	40.4	0.73	Rocking, compr. Toe crushing
3.FG:L3.0-W15-P2C	356	542	373	12.2	26.3	0.93	Rocking, compr. Toe crushing
4. FG:L3.0_W15-P2E	256	515	373	8.4	28.3	1.01	Rocking, compr. Toe crushing
5. FG:L1.8-W15-P2	154	332	178	25.3	37.9	1.35	Rocking, compr. Toe crushing
6. FG:L1.8-W15-P3	218	355	266	14.1	14.1	0.50	Rocking, incl. shear cr., Shear f.
7. PG:L3.0-W15-P2	91	180	120	3.9	10.0	0.36	Inclined shear cracking, Shear f.
8. UG:L1.8-W10-P2	73	140	102	7.7	7.7	0.28	Inclined shear cracking, Shear f.

All the fully grouted walls demonstrated rocking behavior initiated by the development of a single large crack along the wall-footing construction joint. Localized masonry crushing in the compression toe region was the dominant failure mode of fully grouted walls. Distributed flexural cracks were not observed in any of the tests. According to the author, the fully grouted walls, strengthened with unbounded post-tensioning, provided non-linear elastic behaviour prior to toe crushing. Non-linear elastic behaviour refers to the self-centering behaviour of the walls, which is also depicted in force-displacement hysteretic graphs. All the graphs show pinching of hysteresis loops with little energy dissipation. Figure 2.43 shows the extent of wall damage and the force-displacement response of wall FG:L3.0-W15-P2C.

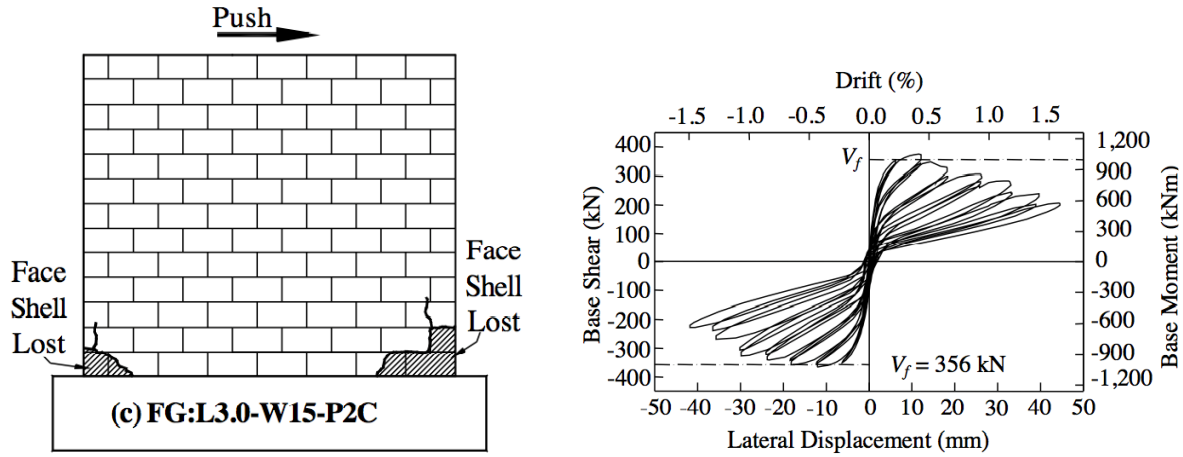


Figure 2.44 Wall FG:L3.0-W15-P2C; failure mode and force-deformation response

The prestressing force in the tendons (sum of forces in three tendons) was measured and plotted relative to the lateral displacement of the wall. It was observed that the first major loss of tension in tendons occurred in the wall displacement range of ± 8 mm to ± 12 mm, which corresponded to the first tendon yield in tension. Partially grouted and un-grouted walls exhibited shear dominant failure signified by diagonally extending cracks. These walls, initially demonstrated flexural damage and rocking behavior for the load level of up to V_f , followed by diagonal shear cracks. Diagonal shear cracking was accompanied by significant strength degradation, resulting in wall failure. Figure 2.44 shows the extent of damage as well as the force-displacement response of wall PG:L3.0-W15-P2.

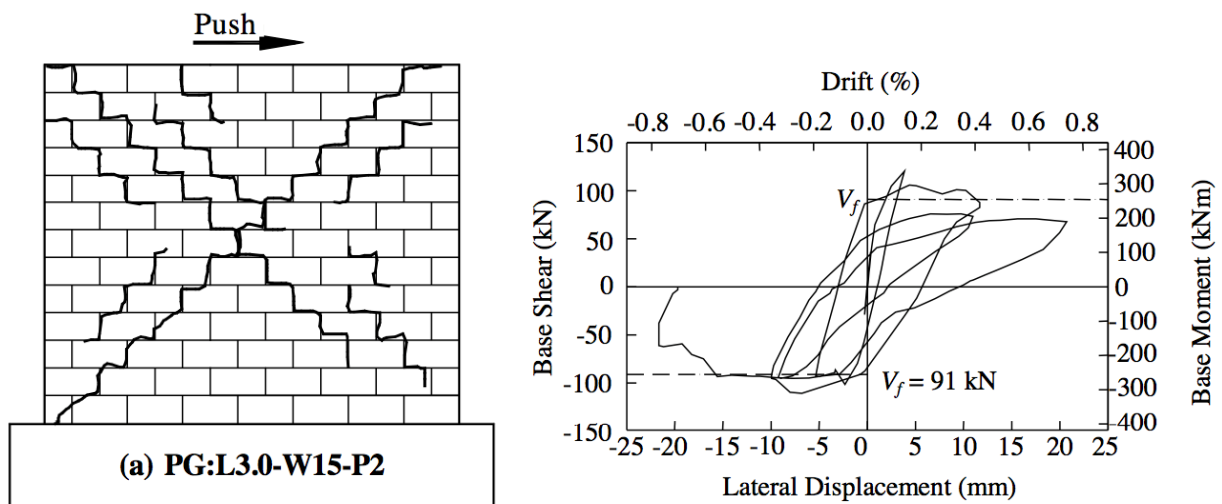


Figure 2.45 Specimen PG:L3.0-W15-P2 failure mode and force-displacement response

The results indicated that the wall dimension, reinforcement ratio, and initial prestressing level have direct effects on the flexural strength (maximum moment) of walls. The flexural strength was higher for longer walls and for those with higher reinforcement ratios. Walls with lower initial prestressing developed higher over-strength due to the reserved elastic capacity in the tendons. Based on the experimental results, the wall dimension and reinforcement ratio affected the ultimate wall drift capacity. An increase in wall length and reinforcement ratio reduced the drift capacity of the wall. The test results suggest a level of initial prestressing of 25%-50% of the tendon yield strength to avoid yielding of the strands while improving drift capacity. The measured vertical strain in masonry associated with nominal flexural strength ranged between 0.0025 and 0.0039. The strain values corresponding to the first sign of toe region distress, which may be considered as ultimate flexural condition, ranged between 0.0067 and 0.0104. The axial load provided by prestressing tendons improved the shear capacity of masonry walls. This effect was further verified by observing that the embedded horizontal reinforcement in wall FG:L3.0-W20-P3 was not engaged at all.

Laursen and Ingham (2004) tested two unbounded post-tensioned concrete masonry (PCM) cantilever walls under in-plane simulated seismic loading to evaluate the use of PCM in actual practice. The prestressing strands were unbonded over the entire length of the wall to ensure maximum elongation of strands. The wall critical corners were also strengthened to improve seismic behaviour of walls. The specimen dimensions and properties are presented in Table 2.15. Confinement plates were embedded in all the wall bed joints up to the height of 1000 mm from the bottom. Half height masonry blocks were used within the lower part of the walls at the first floor level to double the number of confinement plates in the potential plastic hinge region. Three prestressing strands, located at the center and ± 400 mm distance from the center, were used to prestress each wall.

Table 2.15 Wall specimens tested by Ingham et al (2004)

	Dimensions (mm)			Total Weight (kN)	Masonry Strength (MPa)	Strands	Initial Prestress (MPa)	Axial Load
Wall	h_e	l_w	b_w	N_t	f'_m	Number	f_{ps}	Ratio
S3-1	5,250	2,400	140	840	17.9	3	984	0.136
S3-2	5,250	2,400	140	346	14	3	1017	0.135

The level of prestressing for each wall was chosen so as not to yield prior to reaching the drift demand of 2%, which was equivalent to 105 mm of wall displacement. This resulted in the level of prestressing of $0.55f_{pu}$. Horizontal shear reinforcement was also used with 400 mm spacing. Figure 2.45 shows the wall specimen details.

The walls were subjected to in-plane loading by a 400 kN loading jack connected to the loading beam. Table 2.16 includes a summary of wall behavior. V_{max} and d_{vmax} are the maximum recorded lateral force and displacement, respectively. The predicted flexural (V_f) and shear (V_s) capacities of walls are also presented in the Table. The ultimate lateral displacement is defined by d_u , which is the wall displacement after 80% loss in maximum lateral force relative to V_{max} . The drift capacity of the wall at maximum load resistance and ultimate failure point are presented as γ_{vmax} and γ_u . Both walls demonstrated rocking response accompanied by a large single crack along the wall and foundation bed joint. Similar cracks were also observed at the interface of the wall at the first storey and second storey slabs and also at the loading level. Figure 2.46 shows the force-displacement response of both walls. It can be seen that the walls returned to their original alignment even after large deformations. This phenomenon is referred to as nonlinear elastic response by the author. The small area enclosed between loading and unloading reversals shown in Figure 2.46 indicates low energy dissipation.

Table 2.16 Wall specimen behaviour tested by Ingham et al (2004)

Wall	V_s (kN)	ΔP (kN)	V_f (kN)	Direction	V_{max} (kN)	d_{vmax} (mm)	γ_{vmax} (%)	d_u (mm)	γ_u (%)
S3-1	165	21	673	Push	208	49.5	0.94	79.2	1.51
				Pull	-192	-36.6	0.70	-67.2	1.28
S3-2	129	21	647	Push	174	65	1.24	79.8	1.52
				Pull	-156	-54.4	1.04	-93.5	1.78

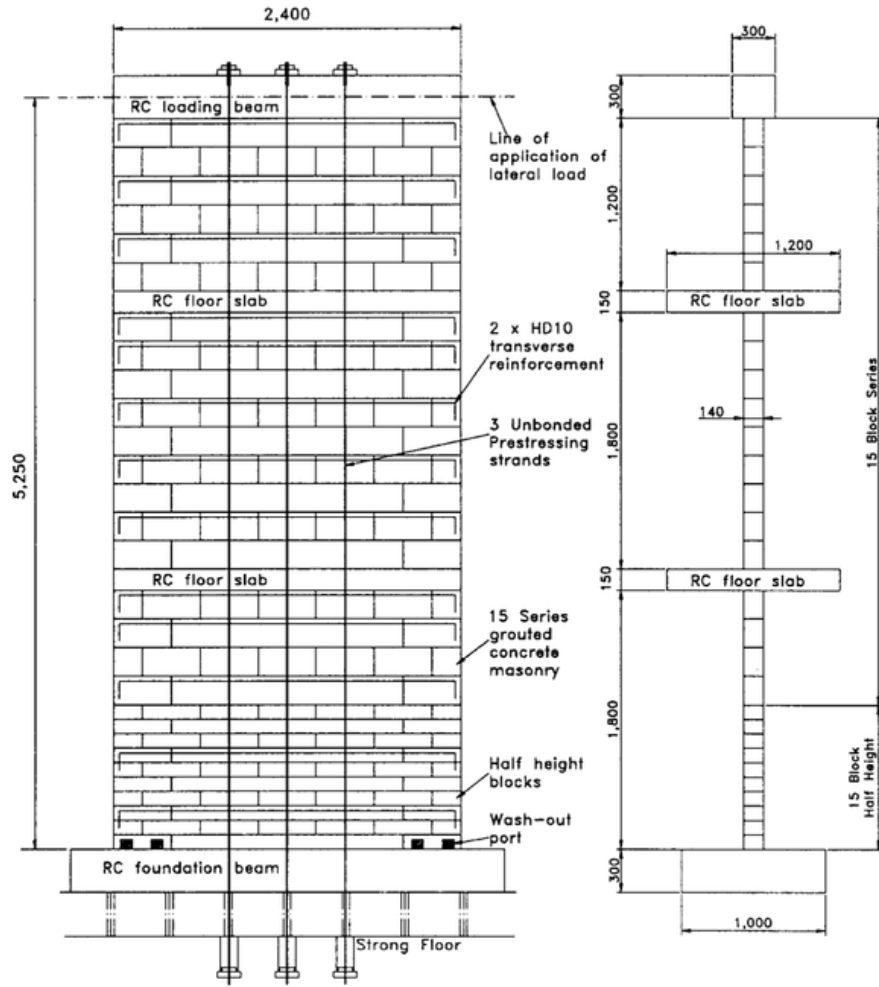


Figure 2.46 Wall specimen details tested by Ingham et al (2004)

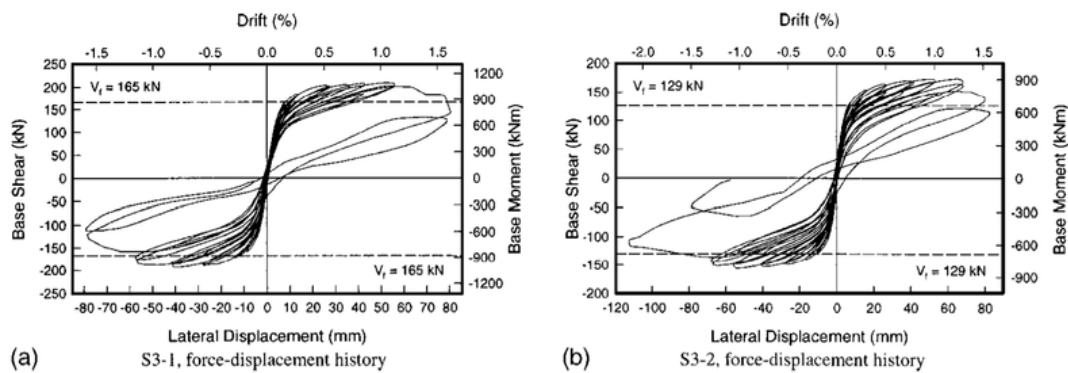


Figure 2.47 Force deformation response of wall specimens S3-1 and S3-2 tested by Ingham et al (2004)

Rosenboom and Kowalsky (2004) tested five large-scale cantilever clay brick masonry walls to investigate the in-plane reversed cyclic behaviour of post-tensioned masonry walls with

different details. All five walls were double wythe walls with 1.22 m length, 205 mm thickness and 2.44 m height. Figure 2.47 shows the wall properties, test setup, and instrumentation. All three walls were post-tensioned with three 25 mm diameter dywidag bars with equal spacing using a target total force of 1000 kN. The prestressing force was based on 2% drift ratio limit under design level earthquake. The first test specimen (Figure 2.47(b)) contained a fully grouted wall cavity with unbonded dywidag bars without any additional reinforcement. Second test specimen (Figure 2.47(c)) was similar to the first, but it was confined in the compression toe region by means of galvanized steel plates that were added to the mortar bed joints in the bottom five-course masonry. The third specimen (Figure 2.47(d)) was also similar to the first specimen, but it was reinforced by four longitudinal #4 US Grade 60 steel re-bars; two at each end. The fourth specimen (Figure 2.47(e)) was also similar to the first specimen, but with bonded dywidag bars. Test specimen five was not grouted and the dywidag bars were not laterally restrained.

The first specimen demonstrated rocking behaviour, which was consistent with the observations made by (Laursen and Ingham 2004). A horizontal crack formed at the wall-footing interface at 0.2% lateral drift. No further flexural crack was observed until 0.5% drift ratio. Toe crushing initiated at 0.5% drift and it propagated towards the wall core at higher drift ratios. The wall reached its maximum lateral force capacity at 1.75% drift ratio beyond which it experienced a gradual strength decay up to 6.5% drift ratio. Two main response characteristics of the wall were self-centering behaviour with zero residual displacement and limited energy dissipation due to rocking.

The second specimen demonstrated identical response compared to the first wall up to 0.5% drift ratio when initial crushing has begun. However, the confinement plates improved the behaviour dramatically and helped the wall maintain its integrity and strength up to 11% drift ratio. The wall failed due to the crushing of toe coupled with the development of a compression strut outside the confined region.

The third wall specimen demonstrated similar initial response as the first specimen, but the energy dissipation was improved due to the placement of mild longitudinal steel bars. There was no significant strength degradation up to 5% drift ratio.

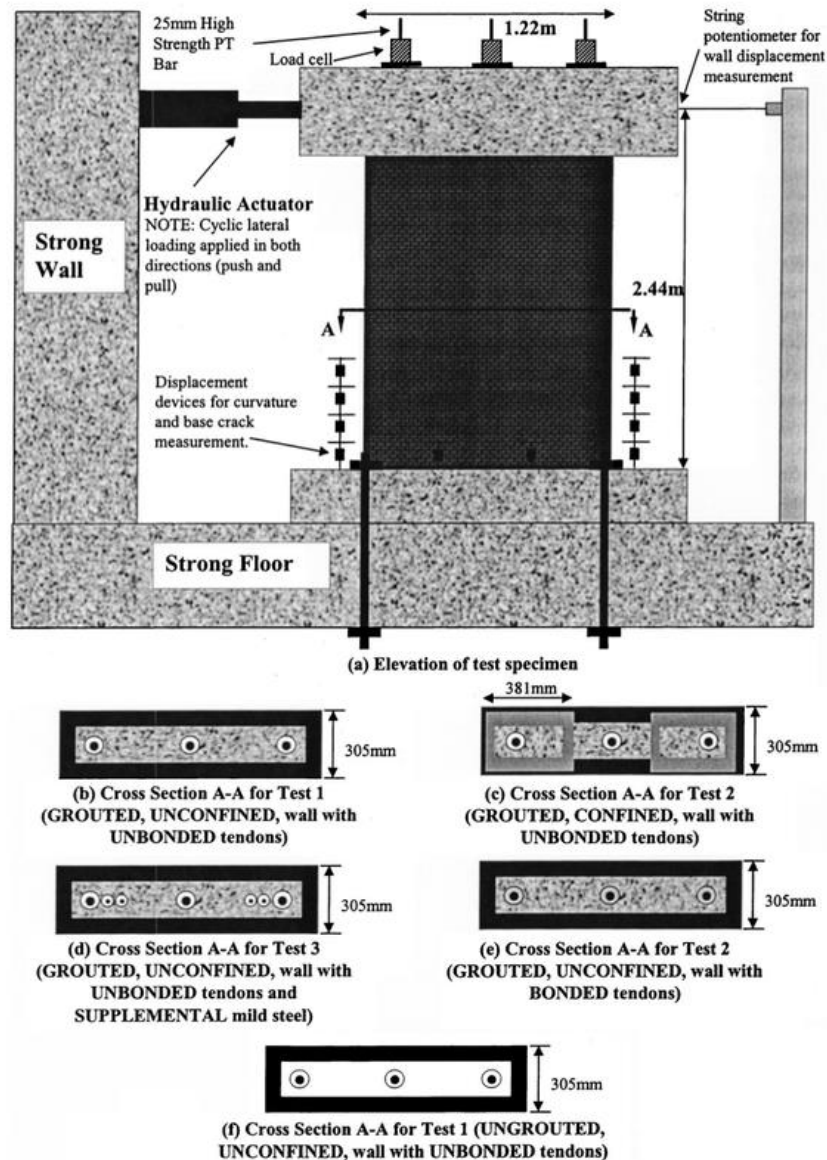


Figure 2.48 The wall specimen properties, test setup, and instrumentation tested by Kowalsky et al. 2004

The fourth specimen demonstrated a very poor performance, which was mainly attributed to the bond stress between the post-tensioning cables and the surrounding grout. The bond stress initiated vertical cracks at 1.75% drift ratio, which led to a significant strength degradation at 3% drift ratio. Although there was higher energy dissipation compared to the first wall, the performance was considered poor because of the level of damage, reduced ductility, and quick loss of strength.

The fifth specimen demonstrated the poorest performance of all. Diagonal shear failure initiated at 1% drift ratio, which was attributed to lack of a grouted core to stabilize diagonal

compression struts in the wall. Rocking response did not take place and a very brittle shear failure was observed without any energy dissipation. Figure 2.48 shows the force-deformation response of all 5 wall-specimens.

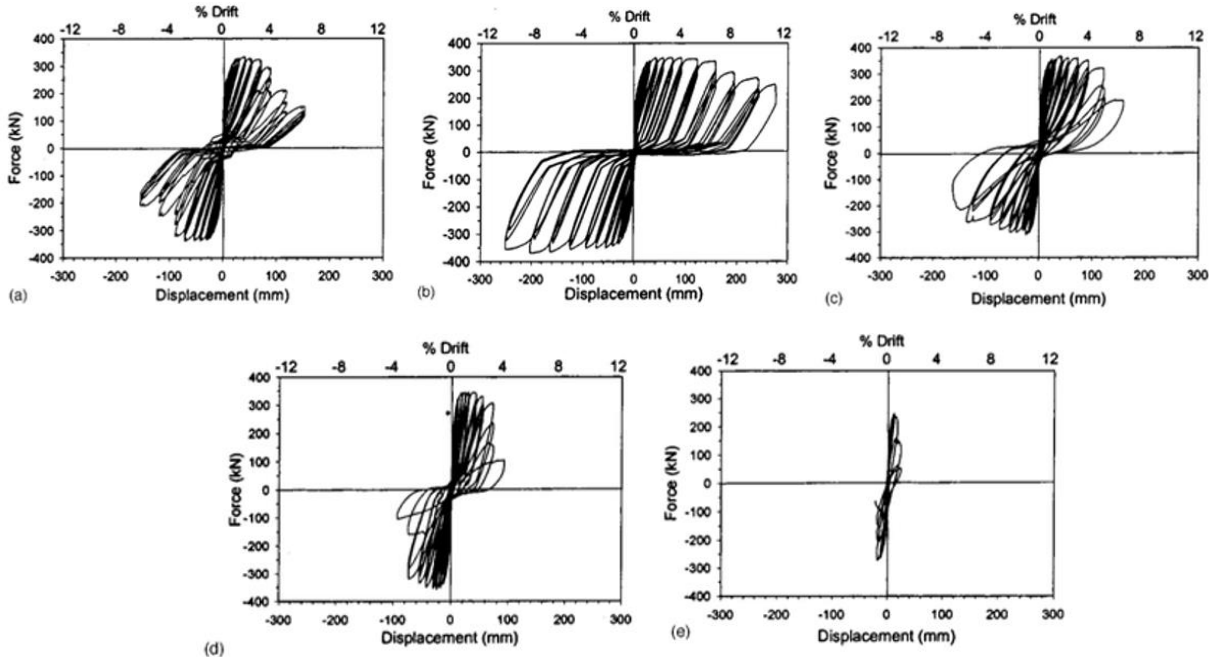


Figure 2.49 Force-displacement hysteretic responses tested by Kowalsky et al. (2004): (a) test 1: grouted, unbonded, unconfined; (b) test 2: grouted, unbonded, confined; (c) test 3: grouted, unbonded, unconfined, mild steel; (d) test 4: grouted, bonded, unconfined; and (e) test 5: ungrouted, unbonded, unconfined

The comparison of five experimental test results justifies the application of following configurations to improve seismic response of posttensioned masonry walls: (1) the presence of a grouted cavity; (2) confinement in the toe compression region; (3) the presence of longitudinal reinforcement steel; (4) unbounded posttensioned ducts.

Levy and Spira (1973) tested three prestressed masonry wall specimens to evaluate design aspects of prestressed masonry and loss in prestressing. First two samples (A1, A2) were wall foundation specimens that were tested in the laboratory. Specimen A1 was mainly developed to study the loss of prestressing force in masonry walls. Figure 2.49 shows the wall dimensions and properties of specimens A1 and A2.

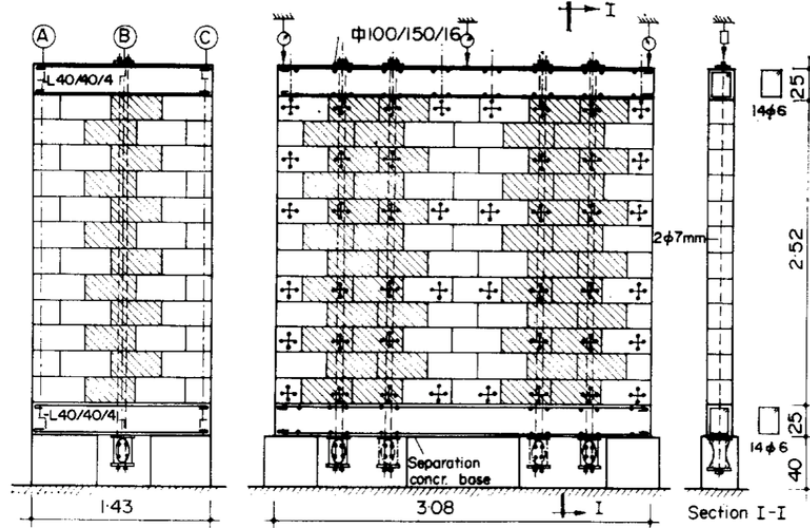


Figure 50.49 Wall specimens tested by Levy (1973) (A1 on left A2 on right)

A prestressing force of 8 tons was applied at the centre of the wall for a period of 7 months and the length changes in the wall at locations A, B, and C, shown in Figure 2.49, were measured once a week. The prestressing forces in tendons were also measured once every three weeks. Figure 2.50 shows the vertical displacement changes along axes A, B, and C as a function of time. As it is shown in this figure, displacement changes converge to zero after 120 days. 74% of the displacements were attributed to shrinkage and the rest to creep.

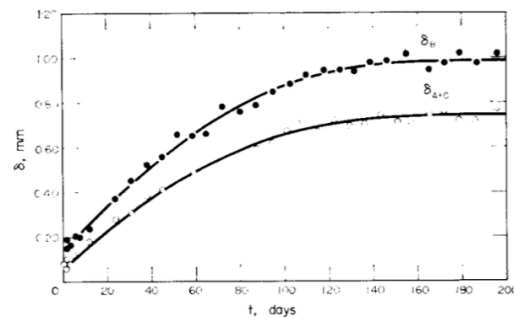


Figure 51.50 Vertical displacement changes at axis A, B, and C by time

Figure 2.51 shows the variation of prestressing force in tendons with time. Prestressing force changes also converged to zero after 120 days. The total measured prestress loss was less than 15%. The author recommended the application of well-aged blocks to reduce the effect of shrinkage while also suggesting not prestressing freshly built masonry.

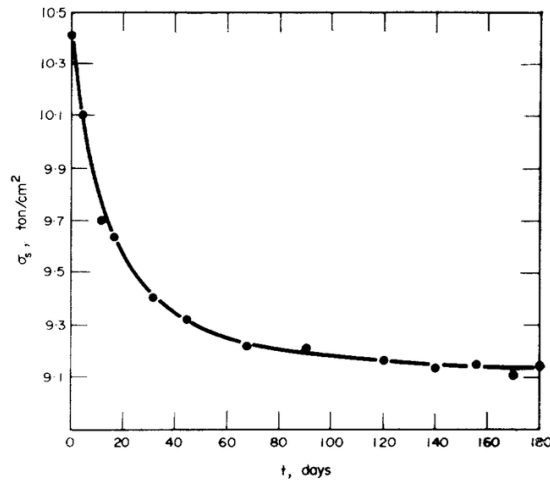


Figure 52.53 Variation of prestressing force in tendons by time

Specimen A2, shown in Figure 2.49, was built to study the strains in masonry and reinforcement, and the deflections of the top beam. Two loading scenarios were evaluated:

- Two pairs of high strength strands were provided on either sides of the wall, with each pair located at 62 cm from the centre.
- The same two pairs of high strength strands were provided at 103 cm from the centre.

Figures 2.52 and 2.53 show test results for loading ‘a’ and ‘b’ described above. The dotted lines indicate the predicted strains and the solid lines indicate the measured strains. The difference between the experimental and predicted results is attributed to; i) partial restraint of the bottom beam in supporting concrete blocks, ii) low level of loading in the experiment and iii) lack of accuracy in measurements.

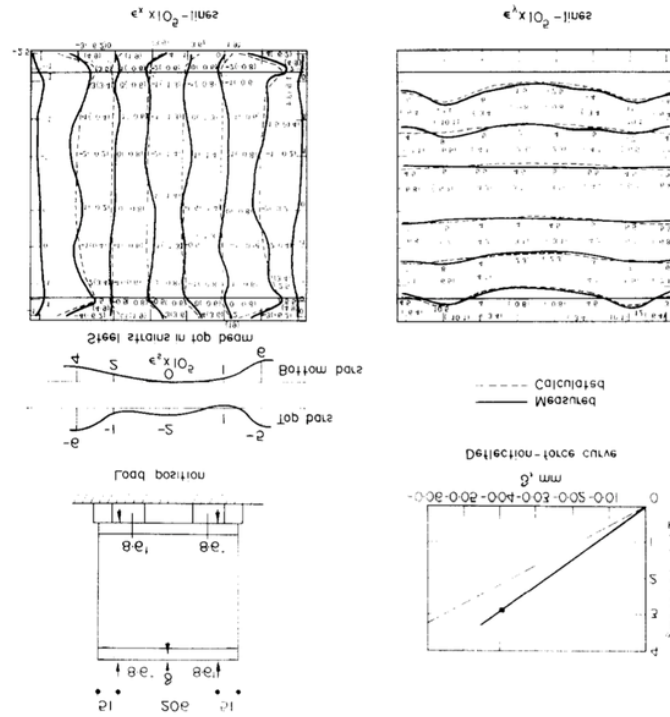


Figure 54.52 Test results of loading (a) by Levy (1973)

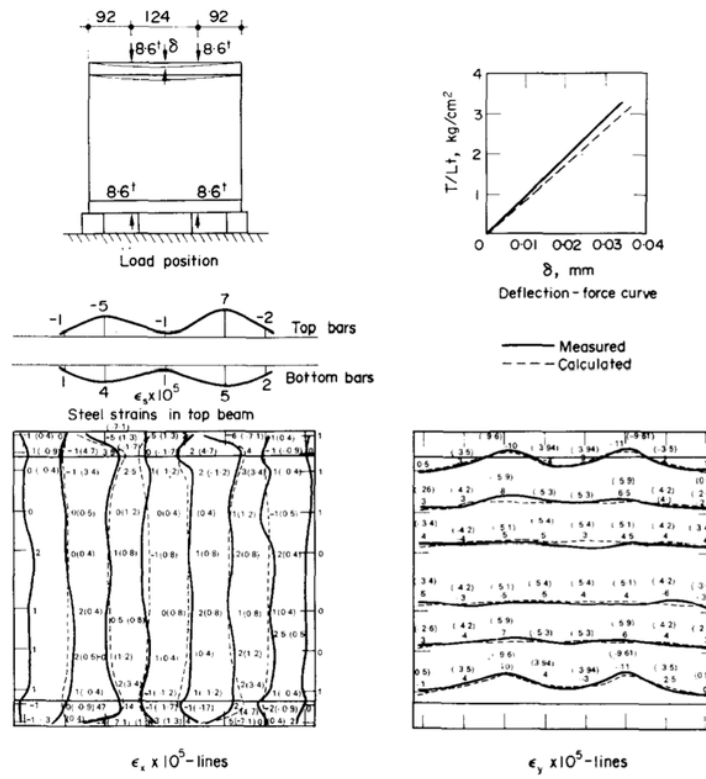


Figure 55.53 Test results of loading (b) by Levy (1973)

2.2 Review of Literature on Analytical Research

There is considerable demand for the prediction of in-plane seismic response of masonry walls for use in design and analysis. Research on masonry panels subject to lateral loading has focused on searching for accurate and reliable analytical techniques to predict failure loads and failure patterns for the designer (Zhou et al. 2006.). An ideal method for estimating structural response of masonry walls is one that does not consume too much effort while maintaining reasonable accuracy. Some of the most conventional analytical models that are reviewed in this section include; i) finite element models, ii) truss model iii) equivalent frame models, and iv) macro-element discretization models.

Taghdi et al. (2000b) studied the mechanism of load resistance in masonry and reinforced concrete shear walls retrofitted with a special system of vertical and diagonal steel strips. Truss models were developed and evaluated by comparing experimental and analytical results. The simplest model was an indeterminate truss model consisting of five elements. Figure 2.54 shows the simple truss model with a short description of each element.

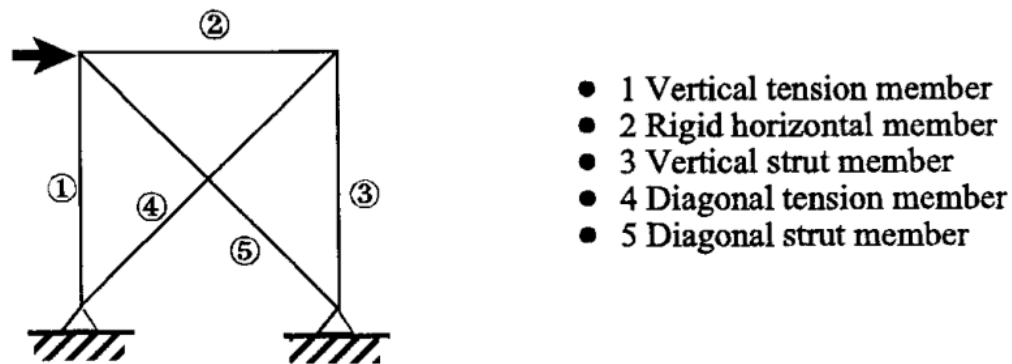


Figure 56.54 Simple truss model to analyze masonry walls

A series of elastic pushover analyses was performed on the simple truss model to draw the force-displacement response of a wall. The analysis consisted of a step-by-step approach to determine the total lateral displacement and force at yield point of each truss member in the model. Even though the simple truss model was relatively effective in capturing the inelastic behavior of the retrofitted wall, a more comprehensive truss model with more truss elements was also evaluated. Figure 2.55 shows the improved truss model with additional truss elements to predict the inelastic deformation of retrofitted walls more accurately.

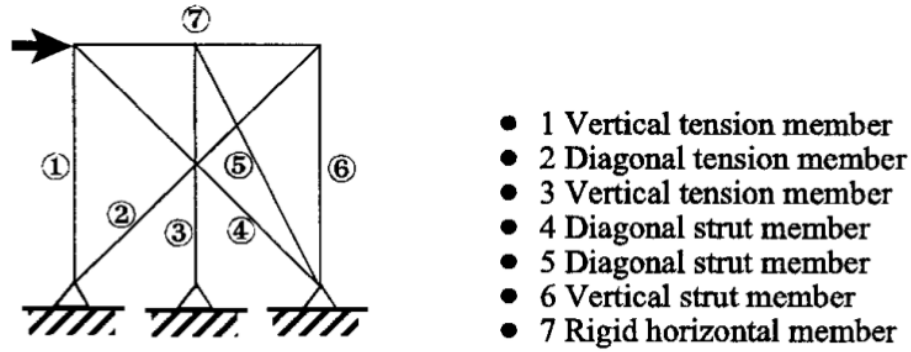


Figure 57.55 Improved truss model to analyze masonry wall

An additional vertical member (truss member 3) was introduced to take into account the effect of vertical reinforcements in the middle of the wall. More vertical members can be located in the model based on the reinforcement layout. Moreover, a diagonal member (truss member 5) was added to model the additional compression strut in the wall. The added truss members led to more data points to define force-displacement response. Figure 2.56 shows the yielding sequence of tension members in simple and improved truss models. The researchers indicated that the ultimate strength of retrofitted walls can be estimated using a lower bound theory from the theory of plasticity (Chen and Toma 1995). Accordingly, every load computed on the basis of a stable and admissible system is not higher than the true ultimate load of the system. Figure 2.57 shows an assumed equilibrium for a retrofitted reinforced concrete shear wall tested by Taghdi et al. (2000a), where A_d is the area of diagonal steel strips in tension, A_v is the area of vertical steel strip and A_{si} , and A_{shi} are the areas of vertical and horizontal reinforcing steel.

It was reported that the ultimate horizontal lateral load capacity, V_u could be calculated by taking moment about point O. The required area of diagonal steel strips A_d , and vertical steel strips A_v could be calculated based on the original unretrofitted capacity of the wall V_{uo} , and the new desired strength of the wall V_u . The details of these quantities are shown illustrated in Equation 1 to 4.

The comparison of experimental results from three retrofitted walls by Taghdi (1998) with analytical results calculated by simple and improved truss models justified the effectiveness of applied analytical methods.

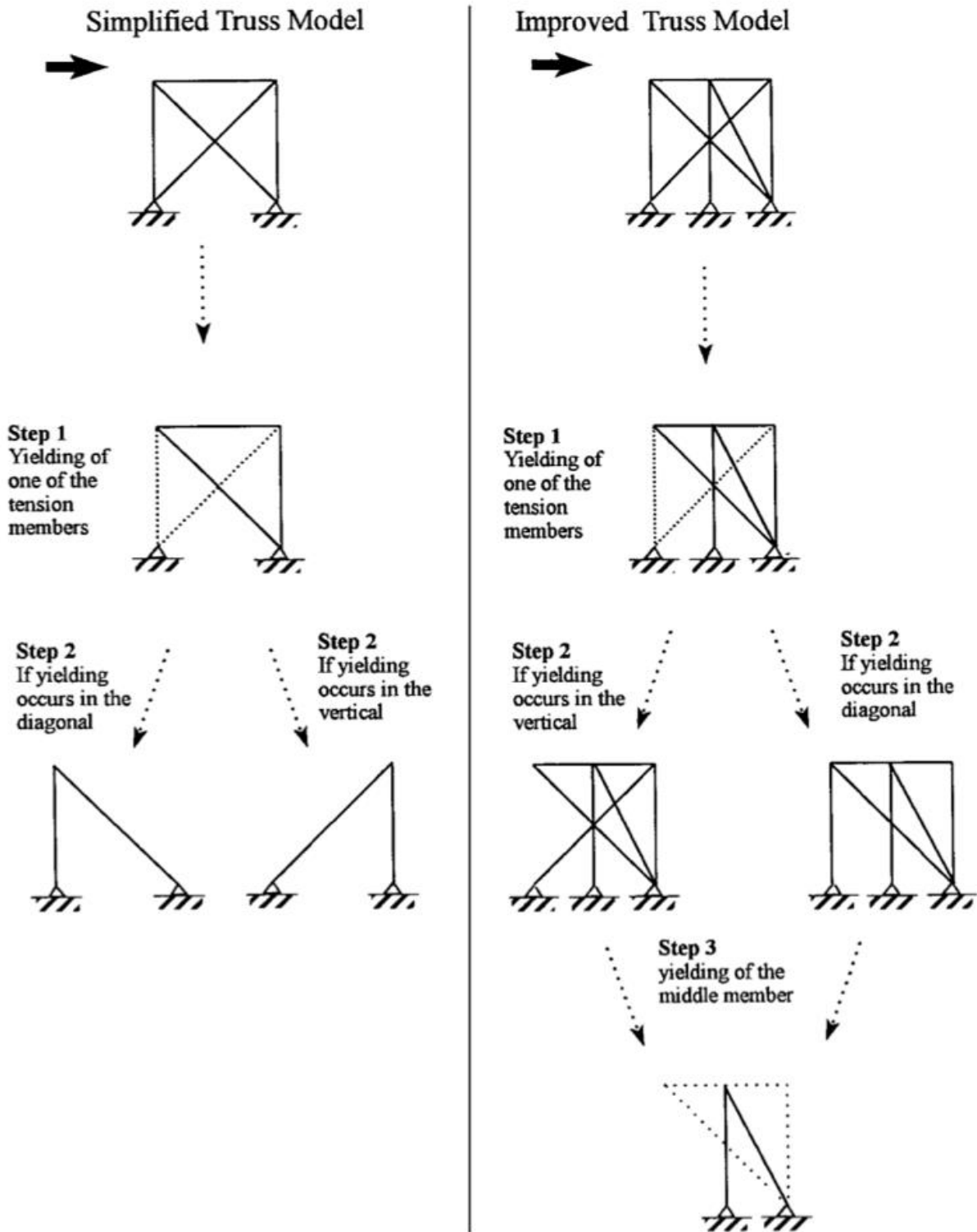


Figure 58.56 Yielding sequence of tension members in truss models

$$V_u = \frac{A_v f_{up} d_v + (A_d f_{yp} d_v \sin \theta + 0.5 P d_v + \sum A_{shi} f_{yh} h_i + \sum A_{si} f_y d_i)}{H} \quad (1)$$

$$V_u - V_{uo} = A_d f_{yp} \cos \theta \quad (2)$$

$$A_d = \frac{V_u - V_{uo}}{f_{yp} \cos \theta} \quad (3)$$

By substituting (3) into (1):

$$A_v = \frac{V_u H - (V_u - V_{uo}) \tan \theta d_v - 0.5 P d_v - \sum A_{si} f_y d_i}{f_{yp} d_v} \quad (4)$$

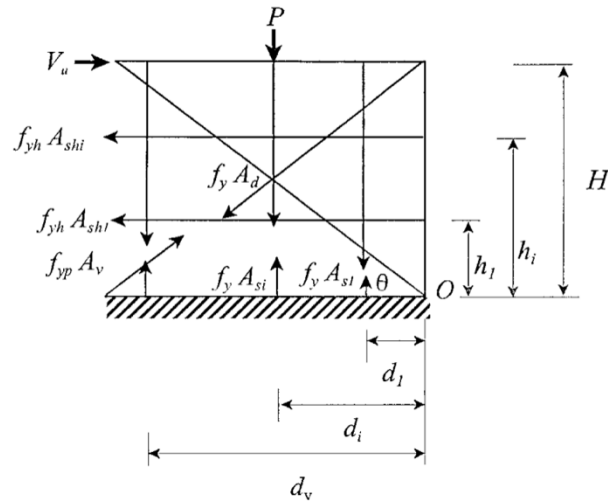


Figure 59.57 Force equilibrium for retrofitted RC shear wall

Equivalent frame model shown satisfactory results particularly for reinforced concrete structures (Belmouden et al. 2009). Pasticier et al. (2008) evaluated the effectiveness of equivalent frame modeling technique for seismic analysis of masonry buildings using SAP2000 software package. Static pushover analyses and time-history analyses were performed on walls that had

been analyzed by other researchers. Plastic hinges and non-linear links provided in SAP2000 were used to create lumped plasticity model for non-linear analysis. The plastic hinges were used in pushover analysis and non-linear links were used in time-history analysis. The non-linear link allowed the user to define cyclic behaviour of elements and hence was suitable for time-history analysis. Figure 2.58 shows the building models used for static pushover and time-history analyses. The piers were modeled by Elastoplastic elements with two rocking hinges and one shear hinge in the middle. The spandrels were also modeled by elastoplastic elements and a single shear hinge placed in the middle. The “multilinear-plastic pivot” non-linear link was used for piers and spandrels as shown in Figure 2.58(b) for dynamic analysis. A single shear hinge was defined at the middle of piers to reduce the computational complexity of time-history analyses. The results indicated that the proposed model in SAP2000 could be used for pushover analysis of masonry structures with regular geometry. It was suggested to calculate the strength of piers based on gravity load, without the application of any horizontal load.

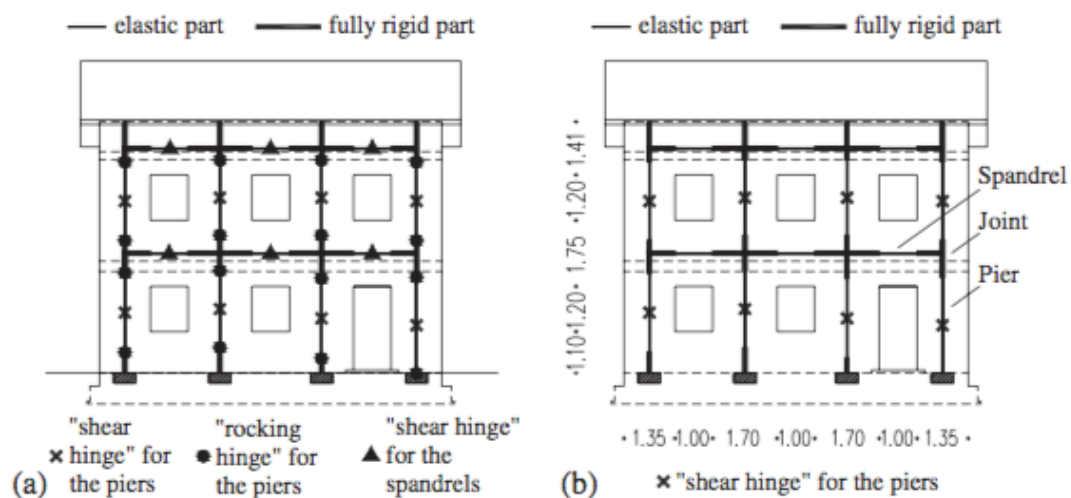


Figure 60.58 Equivalent frame model for static pushover analysis

The non-linear time-history analysis using the proposed model indicated that the structural response depended on the type of earthquake ground motion used. Different earthquake records with the same peak ground acceleration produced different results for the same structure.

Mahmood and Ingham (2011) investigated the effectiveness of equivalent frame approach as an analytical model for perforated unreinforced masonry walls. Non-linear finite element program SAP2000 was used for this purpose and the analysis was performed in linear and nonlinear steps. Six substructures and a two-storey perforated wall were modeled using the

equivalent frame technique. These structures were also constructed and tested as part of their experimental research. Figure 2.59 shows the tested substructures with their crack patterns.

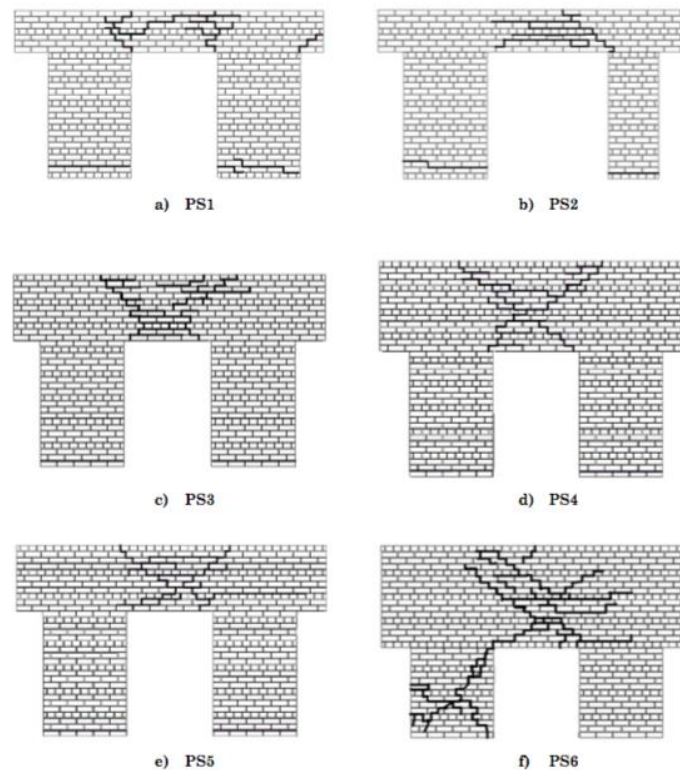


Figure 61.59 Masonry walls tested by Mahmood and Ingham et al. (2011)

Linear models were generated to determine mode shapes and fundamental periods for the samples. Moreover, linear frame models were used as the basis of non-linear equivalent frame models. Material properties were defined using the material type ‘other’ and material directional symmetry ‘anisotropic’. The anisotropic behaviour was selected to define the shear modulus directly, instead of considering it having isotropic behavior as a function of the Poisson’s ratio. The effective height of the piers was determined by Dolce (1989) model. Modification factors for moment of inertia and shear area were obtained through calibration of linear models based on experimental results. These factors are presented in Table 2.17. Figure 2.60 shows the frame layout for the two-storey perforated wall, as well as the first mode response of the structure as obtained from SAP2000 analysis.

Table 2.17 Modification factors for moment of inertia and shear area

	Moment of Inertia	Shear Area
Piers	$I_{eff} = 0.65I_g$	$A_{veff} = 0.8A_v$
Spandrel	$I_{eff} = 0.4I_g$	$A_{veff} = A_v$

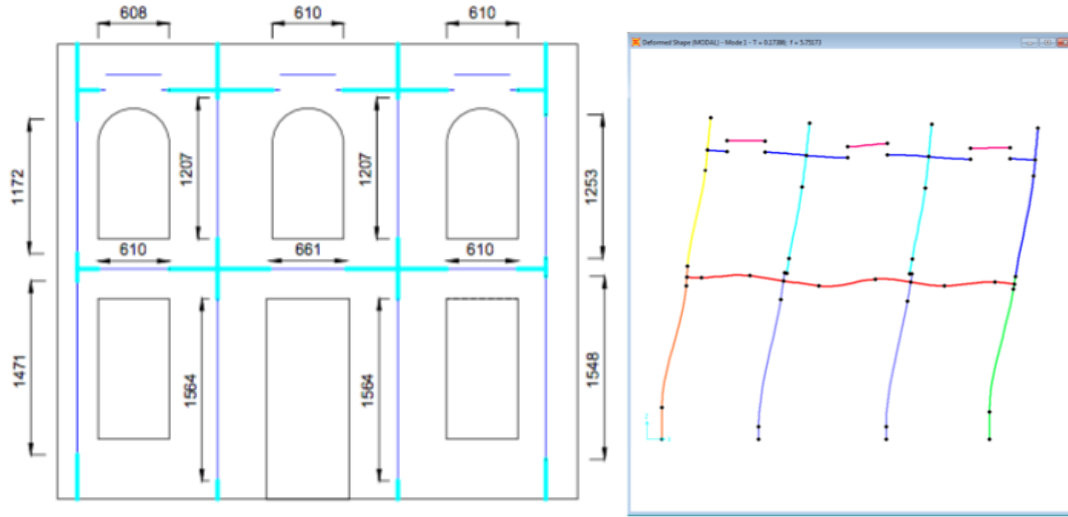
**Figure 62.60 Frame layout for the two storey perforated wall and the first mode of the structure according to SAP2000 model**

Table 2.18 compares the elastic stiffness of six substructures that were captured by the SAP2000 model and the experimental results. The predictions were satisfactory for the first five substructures, except for substructure PS6, which resulted in an underestimation of elastic stiffness by 65%. It was suggested to use an effectively rigid spandrel with high moment of inertia for substructure PS6. Concentrated plasticity approach was implemented to convert the linear equivalent frame model to non-linear model. Flexural plastic hinges were located at the end of each effective pier and spandrel lengths, and shear plastic hinges were located at the center of the elements. Figure 2.61 demonstrates typical hinge positions for Substructure PS1. SAP2000 was capable of defining force-displacement, moment-rotation or moment-curvature relationships for both hinges and links, but hinges were more suitable for pushover analysis. However, it is recommended to use non-linear links for time-history analysis due to the wide range of available hysteretic rules and damping parameters. The analytical and experimental results matched perfectly for all the samples, except for substructure PS5. The equivalent frame model for substructure PS5 correctly predicted flexural response, spandrel shear failure, and also linear

portion of the pushover cure. However, it showed strength decay in the plastic range of deformations due to the shear failure of spandrels. The experimental backbone curve showed strength increase in the plastic portion. This behavior was attributed to an increase in axial load due to the rocking of the sample. The same mechanism was prevented in other tests by placing springs in series to monitor the axial load.

Table 2.18 Comparison between experimental and analytical elastic stiffness

Substructure	K_{iM} (kN/m)	K_{iSap} (kN/m)	K_{iSap}/K_{iM}
PS1	37173	35702	0.96
PS2	26713	22763	0.85
PS3	37441	38491	1.03
PS4	31266	30432	0.97
PS5	29259	29922	1.02
PS6	148530	52604	0.35

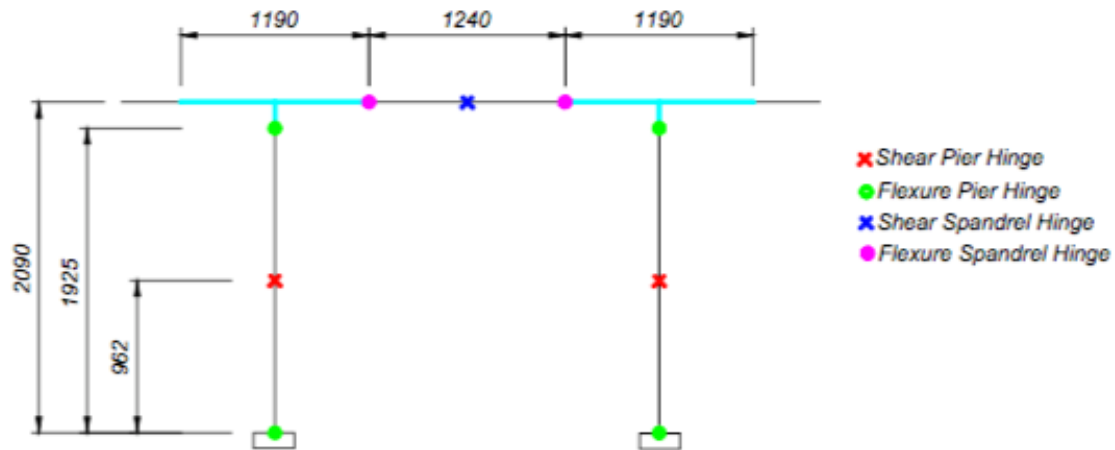


Figure 63.61 Equivalent frame model for PS1 substructure with typical hinge locations

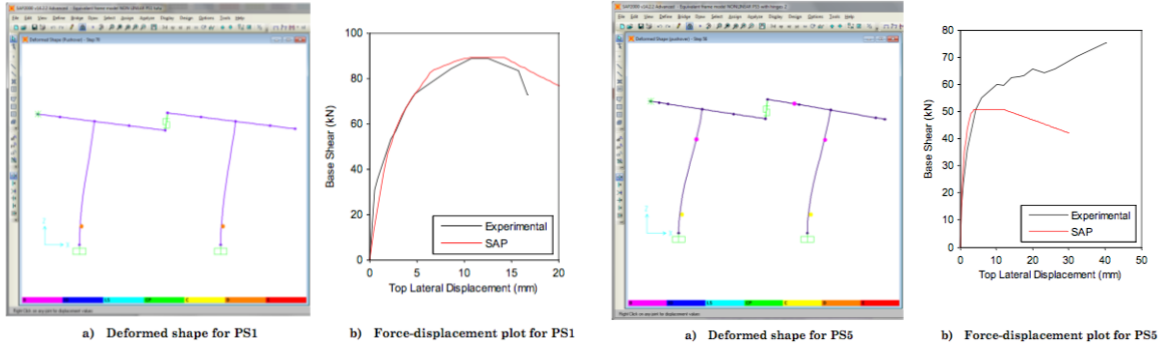


Figure 64.62 Deflected shape of the Sap2000 models for PS1 (left) and PS1 (right) with force-deformation graph of experimental vs. analytical results

Caliò et al. (2012) developed a new discrete-element model, based on the concept of macro-element discretization for the simulation of in-plane behavior of masonry buildings. Caliò et al. (2014) also investigated the same macro-element modeling approach on infilled frame structures. The proposed model approximated the in-plane nonlinear response of masonry walls by an equivalent discrete element. The model was able to produce collapse response of masonry walls subjected to earthquakes. The effectiveness of proposed model was evaluated by comparing the nonlinear incremental static analysis of masonry structures with available theoretical and experimental results. Figure 2.63 shows the mechanical scheme of a discrete-element model. This model consisted of four rigid elements connected by four hinges and two diagonal nonlinear springs. The interaction between different panels was modeled by a discrete distribution of nonlinear springs as interface elements. Each interface consisted of nonlinear springs parallel to the panel side in addition to a longitudinal spring.

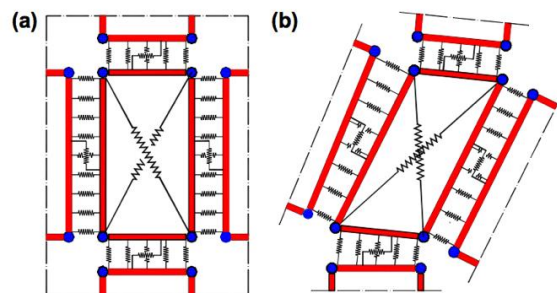


Figure 65.63 Discrete-element model for analyzing masonry walls

The discrete-element model was capable of simulating common collapse mechanisms, i.e., flexural failure, diagonal shear failure, and sliding shear failure. Figure 2.64 shows simple mechanical simulation of collapse mechanisms. The researchers indicated that the discrete element

model could be improved by using smaller size macro-element mesh to capture more accurate results and better simulation of the collapse mechanism. Figure 2.65 shows a masonry wall with door and window, which is modeled by discrete-element model with different mesh resolutions.

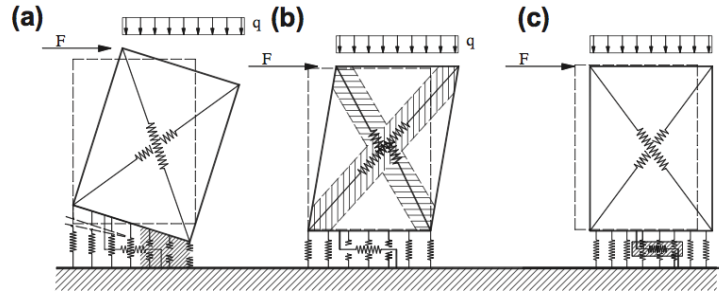


Figure 66.64 Simulation of different collapse mechanisms by discrete-element model

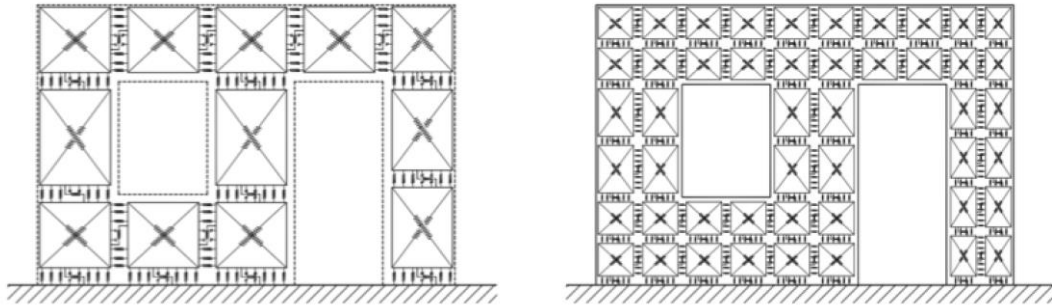


Figure 67.65 Discrete-element model of a masonry wall with door and window with two different resolutions

The calibration of nonlinear springs was done by applying a specific constitutive law for the masonry media based on a simplified fiber model. An elasto-plastic behavior with limited deformability was considered in the calibration process. The horizontal and vertical interfaces were calibrated separately due to the orthotropic nature of masonry. The proposed discrete model was evaluated by comparing the nonlinear response of masonry walls and structures with available experimental and numerical test results from previous studies. Unreinforced brick masonry was used in the research program. Two case studies with available test results were considered. The first series of numerical simulations were performed on masonry panels with different geometric ratios. The second series of simulations were performed on a large-scale masonry model built and tested at the University of Pavia.

The brick masonry panels that were considered for the first case study were tested under constant axial load and lateral load reversals. The first panel was a squat wall with dimensions of

1000 mm x 1350 mm and suffered from diagonal shear failure. The second wall was a 1000 mm x 2000 mm slender wall, which developed rocking behavior. The wall thickness was 250 mm in both cases.

The large-scale masonry model was a two-storey prototype brick masonry building tested by Magenes and Calvi (1994) at the University of Pavia. The plan of the building was 6.0 m by 4.4 m, and the height was 6.4 m. The building consisted of four two-wythe solid brick walls with 250 mm wall thickness. The two masonry walls in the longitudinal direction had openings, while the other two transverse walls had no openings. The lateral cyclic loading was applied at floor levels on the walls in the longitudinal direction.

The agreements of numerical results and experimental results were satisfactory in terms of cyclic response, strength, stiffness, energy dissipation, and damage distribution. Figures 2.63 through 2.69 show the comparison between the analytical and experimental response of specimens. Pantò et al. (2016), investigated macro-element discretization approach for three dimensional seismic assessment of historical masonry churches.

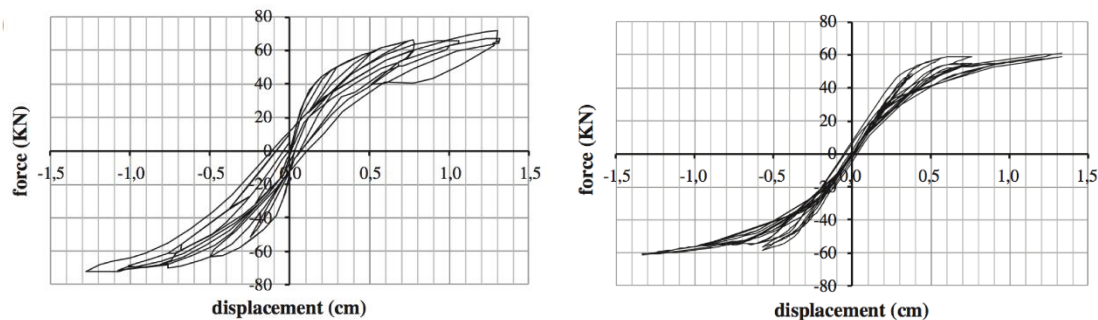


Figure 68.66 (Flexural Panel) Left: Experimental Right: Numerical

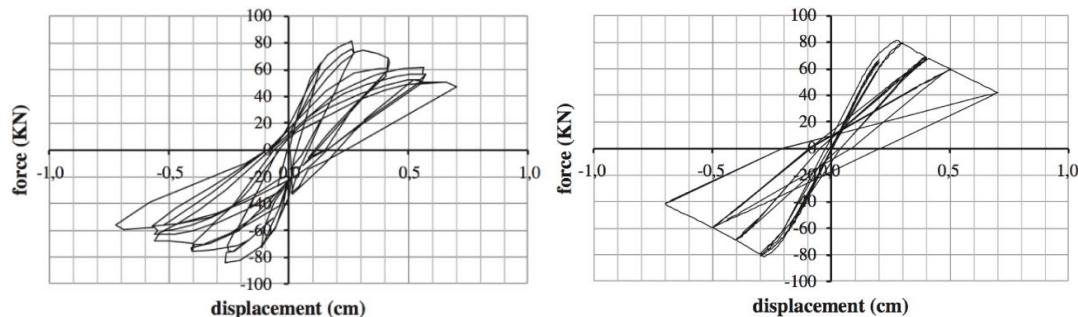


Figure 69.67 (Shear Panel) Left: Experimental Right: Numerical

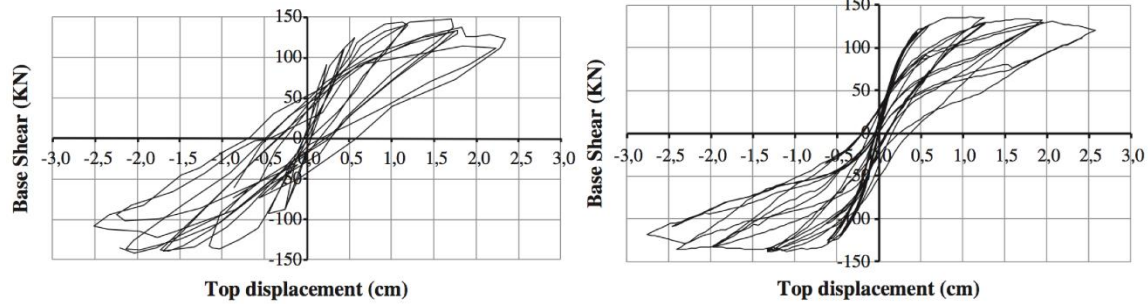


Figure 70.68 (Two-Storey building) Left: Experiment Right: Numerical (top floor response)

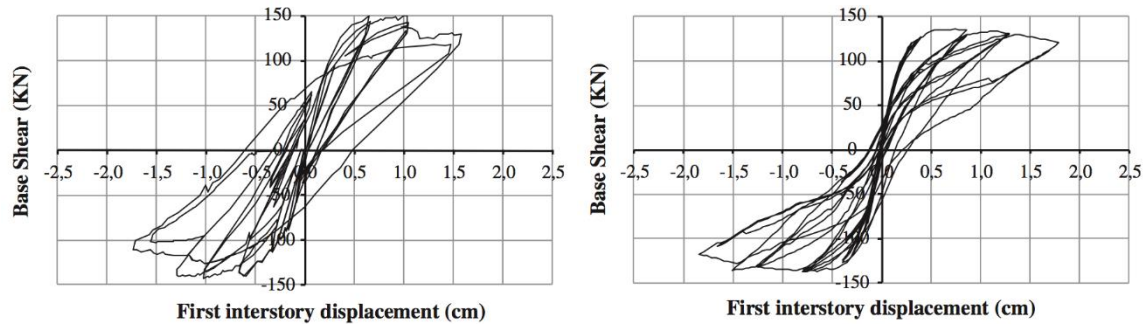


Figure 71.69 (Two-Storey building) Left: Experiment Right: Numerical (first floor response)

Laursen and Ingham (2004) predicted in-plane behavior of prestressed masonry walls through a flexural/rocking model, as well as a finite element model. The flexural/rocking model was based on the flexural theory (plane section analysis) where the wall displacement could be obtained by the integration of curvatures along the wall height. The rigid body rocking analogy was also used for large displacement predictions. The finite element model (FEM) used the force-displacement relationships that were obtained by flexural model to investigate the internal force flow within the toe crushing region. Figure 2.70 shows various definitions of wall deformations and forces used in applying the flexural theory. The force-displacement response based on the flexural model was able to predict the behavior at different stages of loading, including the Serviceability Limit State (SLS) and the Ultimate Limit State (ULS). Figure 2.71 shows the calculated base shear forces and corresponding lateral displacements at different points.

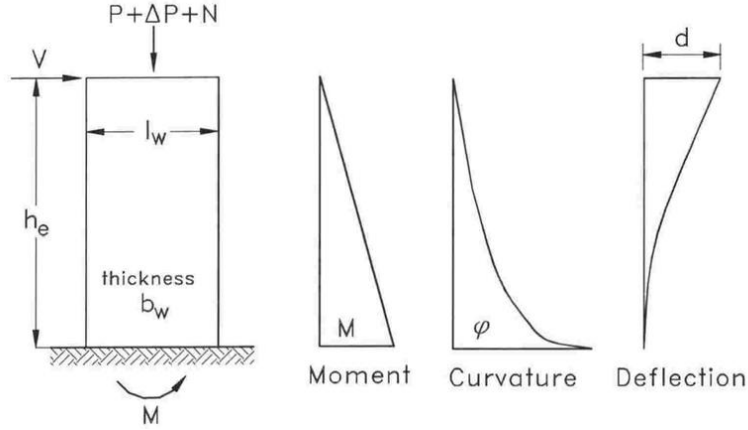


Figure 72.730 Flexural theory definitions

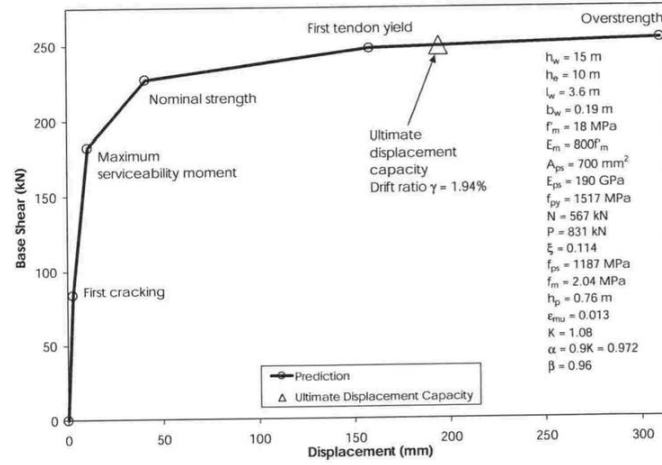


Figure 74.71 Calculated force deformation response based on flexural theory

First cracking moment M_{cr} and curvature ϕ_{cr} were computed based on the following expressions.

$$M_{cr} = \frac{f_m b_w l_w^2}{6} = \text{where } f_m = \frac{(P + N)}{l_w b_w} \quad (5)$$

$$V_{cr} = \frac{M_{cr}}{h_e} \quad (6)$$

Where l_w is the wall length, b_w is the wall thickness, h_e is the effective height of the wall, f_m is the average masonry stress, and V_{cr} is the applied load at the top of the wall that produces M_{cr} . The curvature at first tension cracking of the wall is:

$$\phi_{cr} = \frac{2f_m}{E_m l_w} = \frac{2(P + N)}{E_m l_w^2 b_w} \quad (7)$$

The lateral displacement of the top of the wall consisted of deformations due to flexure and shear as formulated below.

$$d_{cr} = d_{crfl} + d_{crsh} = \frac{2}{3} \frac{h_e^2 (P + N)}{E_m l_w^2 b_w} + \frac{2}{5} \frac{(1 + \nu)(P + N)}{E_m b_w} \quad (8)$$

E_m in the above expressions is the modulus of elasticity of masonry and ν is the Poisson's ratio, which is usually considered to be 0.2 for concrete materials. The maximum serviceability moment occurred when the stress in the compression toe region reached kf'_m , which was a fraction of the concrete compressive capacity. Coefficient k was defined in the New Zealand code (NZS 3101:1995) to be in the range of 0.45 and 0.55, depending on the load category. The following formula relates the maximum serviceability moment to the first cracking moment, which was calculated from the force equilibrium given below.

$$M_e = \left(3 - \frac{4f_m}{kf'_m}\right) M_{cr} = V_e h_e \quad (9)$$

The curvature corresponding to the maximum serviceability moment was computed by Eq. 10:

$$\phi_e = \frac{(kf'_m)^2}{2f_m E_m l_w} \quad (10)$$

The deflection of the top of the wall consisted of flexural and shear deformations and was computed by Equation 11 by assuming $k=0.55$.

$$d_e = d_{efl} + d_{esh} = (0.3 - 0.029 \frac{f_m}{f'_m}) \frac{f'_m h_e^2}{E_m l_w} + \frac{12(1 + \nu) h_e}{5 E_m l_w b_w} V_e \quad (11)$$

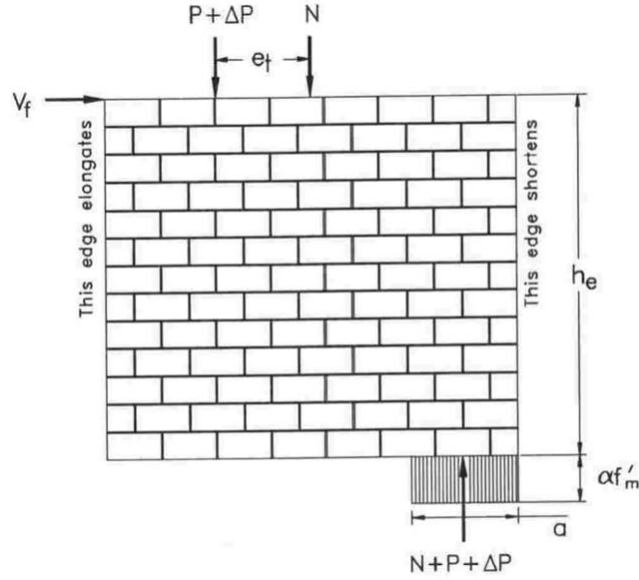


Figure 75.72 Force equilibrium in masonry wall

The nominal strength calculation was based on the equivalent stress block defined in NZS 4230:1990 standard for unconfined and confined concrete masonry. Table 2.19 gives these details.

Table 2.19 Equivalent stress block coefficients for confined and unconfined masonry

	α	Extreme fiber strain, ε_{mu}
Unconfined Concrete Masonry	0.85	0.0025
Confined Concrete Masonry	0.9	0.008

The nominal moment M_n and the lateral force V_f were obtained from the equilibrium of forces shown in Figure 2.72.

$$M_n = (P + \Delta P) \left(\frac{l_w}{2} + e_t - \frac{a}{2} \right) + N \left(\frac{l_w}{2} - \frac{a}{2} \right) = V_f h_e \quad (12)$$

Where a is the length of the equivalent stress block as shown below:

$$a = \frac{P + \Delta P + N}{\alpha f'_m b_w} \quad (13)$$

In equation 12, ΔP represents the increase in tendon force due to flexural deformations and e_t accounts for the tendon force eccentricity. The lateral displacement at the top of the wall consisted of flexural and shears deformations. The flexural deformation expressions were obtained

through integration and curve fitting. They are valid for axial load ratios of ξ_n varying between 0.05 and 0.025.

$$d_n = d_{nfl} + d_{nsh}$$

Unconfined:
$$d_{nfl} = (2.30\xi_n^2 - 1.38\xi_n + 0.856) \frac{f'_m h_e^2}{E_m l_w}$$

Confined:
$$d_{nfl} = (7.63\xi_n^2 - 5.40\xi_n + 1.69) \frac{f'_m h_e^2}{E_m l_w}$$

$$d_{nsh} = \frac{12(1+\nu)h_e}{5E_m l_w b_w} V_f \quad (14)$$

$$\xi_n = \frac{P + \Delta P + N}{f'_m l_w b_w}$$

An iterative process was recommended to find the total tendon force increase based on estimated wall end elongation u_e , and shortening u_s as shown in Figure 2.73.

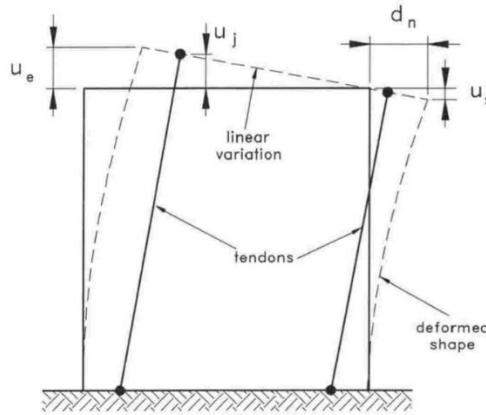


Figure 76.73 Wall deformation shape

The following equations were developed to calculate u_e and u_s for unconfined and confined masonry. The elongation is positive and shortening is negative. The tendon elongation u_j can be found from the geometry.

Unconfined:
$$u_e = (4.01\xi_n^2 - 2.37\xi_n + 0.835) \frac{f'_m h_e}{E_m}$$

$$u_s = (3.36\xi_n^2 - 2.12\xi_n + 0.073) \frac{f'_m h_e}{E_m} \quad (15)$$

Confined: $u_e = (22.5\xi_n^2 - 10.4\xi_n + 1.83) \frac{f'_m h_e}{E_m}$

$$u_s = (1.67\xi_n^2 - 1.67\xi_n + 0.142) \frac{f'_m h_e}{E_m}$$

The effective tendon force eccentricity relative to the wall centerline was formulated as follows; where, y_j is the horizontal location of the tendon j with respect to the centerline, which is considered positive on the tension side of the wall.

$$e_t = \frac{\sum (P_j + \Delta P_j) y_j}{(P_j + \Delta P_j)} \quad \text{Where} \quad \Delta P_j = \frac{u_j}{l_j} A_{psj} E_{ps} \quad (16)$$

The iterative process to find M_n and d_n begins by assuming $\Delta P = 0$ and continues with the use of Eqs. 12 to 16 to find $\Delta P = \sum \Delta P_j$. The calculations are then repeated with a new ΔP until the convergence of ξ_n is achieved. The yield moment M_y and displacement d_y for unbounded prestressed wall was formulated based on the rocking response of wall beyond the nominal strength. The rocking response of unbounded masonry wall was consistently observed during the experimental phase of research.

$$M_y = (N + P + \Delta P) \left(\frac{l_w}{2} - \frac{a_y}{2} \right) + M_{ty} = V_y h_e \quad (17)$$

$$d_y = d_n + d_{ty} \quad (18)$$

Where, M_{ty} is the resulting moment increase above M_n :

$$M_{ty} = \sum_{j=1}^n \Delta P_{tyj} \left(e_{tj} - \frac{a_y}{2} \right) = \sum_{j=1}^n \Delta P_{tyj} e_{tj} - \frac{a_y}{2} \Delta P_y \quad (19)$$

Where, ΔP_y is the total tendon force increase above that at M_n :

$$\Delta P_y = \sum \Delta P_{tyj} \quad (20)$$

Where, ΔP_{tyj} is the force increase in individual tendons:

$$\Delta P_{tyj} = \frac{\theta(e_{tj} - c)}{h_e} E_{ps} A_{psj} = (f_{py} - f_{ps}) A_{psj} \frac{e_{tj} - c}{e_{te} - c} \quad (21)$$

Where, e_{tj} is the location of the j 'th tendon with respect to the compression end of the wall,

and A_{psj} is the area of the j 'th tendon. The compression zone length at first tendon yield may be calculated as:

$$a_y = \frac{P + \Delta P_y + N}{\alpha f'_m b_w} \quad (22)$$

The wall displacement increase at the first tendon yield d_{ty} can be calculated based on the wall rotation θ and tendon strain increase $\Delta \epsilon_{py}$:

$$d_{ty} = \theta h_e = \frac{\Delta \epsilon_{py} h_e^2}{e_{te} - c} = \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{te} - c} \quad (23)$$

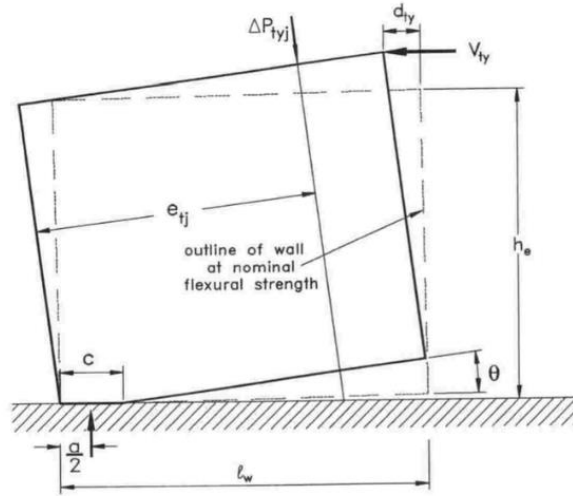


Figure 77.74 Force equilibrium and deformations in masonry wall

The maximum strength of unbounded prestressed wall was calculated based on the assumption that all the tendons have yielded. The flexural overstrength, M_o , can be calculated as follows:

$$M_o = (N + P_y) \left(\frac{l_w}{2} - \frac{a_o}{2} \right) = V_o h_e \quad (24)$$

Where, a_o , the length of the equivalent ultimate compression block, and P_y , the total tendon force when all the tendons are yielding are:

$$a_o = \frac{N + P_y}{\alpha f'_m b_w} \quad (25)$$

$$P_y = A_{ps}f_{py} \quad (26)$$

The resulting displacement is:

$$d_o = d_n + \frac{f_{py} - f_{ps}}{E_{ps}} \frac{h_e^2}{e_{tc} - a_o/\beta} \quad (27)$$

Where, e_{tc} is the distance from the compression end of the wall to the closest tendon, and f_{ps} is the stress in that tendon. The ultimate capacity of the wall is limited to the ultimate strain of tendons as well as the crushing strain of masonry. However, the crushing strain of masonry is usually the governing failure case because the ultimate strain of tendon is a high value (around 5%), which requires a very high lateral wall displacement.

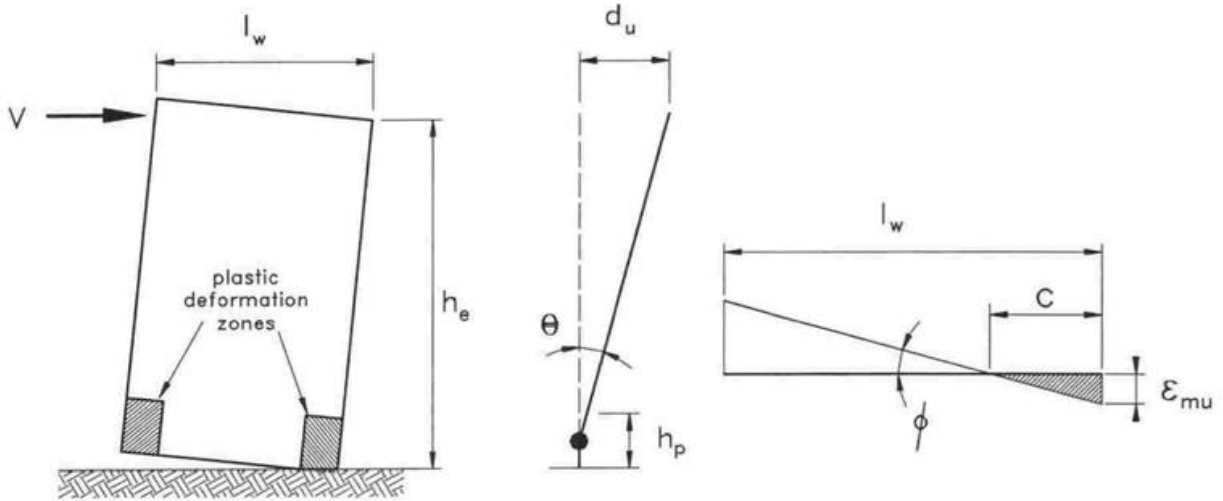


Figure 78.75 Plastic deformation and wall rotation

It was assumed that all the lateral deformation at the top of the wall is due to rotation, θ , of the plastic hinge and the relation between wall lateral displacement d_u and the masonry extreme fiber strain is ϵ_{mu} is formulated as follows:

$$d_u = \theta \left(h_e - \frac{h_p}{2} \right) \text{ and } \theta = \phi h_p = \frac{\epsilon_{mu}}{c} h_p$$

Thus:

$$d_u = \frac{h_p \left(h_e - \frac{h_p}{2} \right)}{c} \epsilon_{mu} \text{ where } c = \frac{a}{\beta} = \frac{P + \Delta P + N}{\alpha f'_m b \beta} \quad (28)$$

Where, $\alpha=0.85$ and $\beta=0.85$ for unconfined concrete masonry and $\alpha=0.9$ and $\beta=0.96$ for confined masonry. It is suggested to find the value of plastic hinge length as a function of effective length of the wall h_e . The wall strength V_u at d_u is calculated by interpolation between the first tendon yield and the overstrength limit state with respect to displacements.

$$V_u = V_y + \frac{V_o - V_y}{d_o - d_y}(d_u - d_y) \quad (29)$$

2.3 Conclusion from Literature Review

The following conclusions can be drawn from the literature review:

- Unreinforced masonry (URM) walls are vulnerable to seismic damage and there is need for the development of effective seismic retrofit methodologies.
- Some of the conventional methods of retrofitting URM walls consist of grout injection, and external and surface treatments.
- Surface-bonded FRP sheets provide strength against diagonal tension caused by shear, and hence can be used effectively for shear strengthening of URM walls.
- Surface-bonded FRP, when used for flexural capacity enhancement, require sufficient anchorage to wall foundation. This may be done either through FRP anchors or ductile steel sheet anchors.
- Among the more recent retrofit materials, polyurea may be a suitable material to control out-of-plane performance. However, it may not be suitable for sufficient increase in in-plane wall strength and ductility because of its low stiffness.
- There is limited previous research on the application of common construction materials like ordinary reinforcing bars and prestressing cables used as retrofit materials for URM walls.
- Post-tensioning proves to be a viable procedure for retrofitting masonry walls. It provides rocking and self-centering response.
- Unbonded post-tensioning tendons provide the best results in post-tensioning of URM walls.
- Long-term behaviour (shrinkage and creep) of URM walls should be accounted for when post-tensioning the masonry.

- Some of the common analytical models for masonry walls consist of truss model, flexural-rocking model, and discrete-element model.
- The use of conventional reinforcement and prestressing strands appear to be effective methods for increasing flexural wall capacity, with enhanced ductility. However, lack of shear capacity may need to be enhanced with the use of FRP sheets.

3 Preparation of Experimental Specimens

3.1 Introduction

The literature review presented in Chapter 2 indicates research needs for structurally sound, economically feasible and practical systems for retrofitting unreinforced masonry walls for improved seismic performance. The retrofit strategy should promote integrity of masonry blocks during earthquakes against in-plane and out-of-plane movements, provide sufficient strength, ductility and energy dissipation capacity. The strength requirement calls for sufficient increases in flexural and shear capacities. These requirements provide a challenge for retrofitting unreinforced elements, which lack the required tensile capacity for providing moment resistance and controlling shear-induced diagonal tension cracks. The former requires tension elements (reinforcement) in the vertical direction that form tensile components of internal moment resistance, while the latter requires reinforcement within the web for controlled diagonal tension cracking in two orthogonal directions. The conversion of an unreinforced masonry wall to a properly designed reinforced masonry shear wall, withstanding many cycles of inelastic deformation reversals, poses significant challenges, especially if the retrofit technique is to maintain practicality and simplicity required in construction.

3.2 Objectives and Scope of Wall Tests

The objective of experimental research was to develop seismic retrofit methodologies that provide sufficient reinforcement for flexure and shear, while promoting ductile response. The use of internally placed conventional deformed Grade 400 MPa reinforcement and high-strength post-tensioning strands as added reinforcement was considered as the retrofit strategy. The addition of vertical steel reinforcement was complemented with the addition of web reinforcement. The use of surface-bonded carbon fibre reinforced polymer (CFRP) sheets was considered as the web reinforcement to enhance diagonal tension capacity. In addition, the masonry cells were filled with concrete to increase diagonal compression capacity. Tests of half-scale masonry walls, retrofitted by placing either conventional vertical reinforcement or post-tensioning strands in the wall boundary elements, in combination with surface-bonded CFRP web reinforcement formed the experimental tasks.

3.3 Details of the Test Program

The experimental phase of research consists of two half-scale load bearing masonry walls retrofitted and tested under simulated seismic loading at the Structures Laboratory of the University of Ottawa. The walls were built and tested as companion walls to those tested in an earlier phase of the same overall research program at the University of Ottawa without seismic retrofitting by Taghdi et al. (2000a). This wall served as the reference wall, reflecting the as-built conditions for comparison. One of the two walls (Wall 1) that was tested in the current investigation was retrofitted by inserting internal reinforcement at the ends, generating boundary elements, while the other wall (Wall 2) was retrofitted by inserting and post-tensioning 7-wire strands in wall boundary elements. Both walls were reinforced against shear by applying surface-bonded CFRP sheets. The geometric details and properties of the wall specimens are presented in the following sub-sections.

3.3.1 Wall Details

Professional masons were hired to construct two half-scale masonry walls with the same size and similar properties as the companion retrofitted as-built URM walls tested by Taghdi et al (2000a). The use of a mason ensured proper simulation of construction practices in the construction industry. Both walls were built on reinforced concrete footings using concrete masonry blocks. The geometry of wall specimens is shown in Figure 3.2. The blocks consisted of standard 200 mm units with nominal dimensions of 190 mm x 90 mm x 90 mm, assembled with 10 mm joints. The walls were constructed using a type O mortar. The blocks were face-shell beaded using the same mortar. Further details of each wall with the details for seismic retrofitting are presented below.

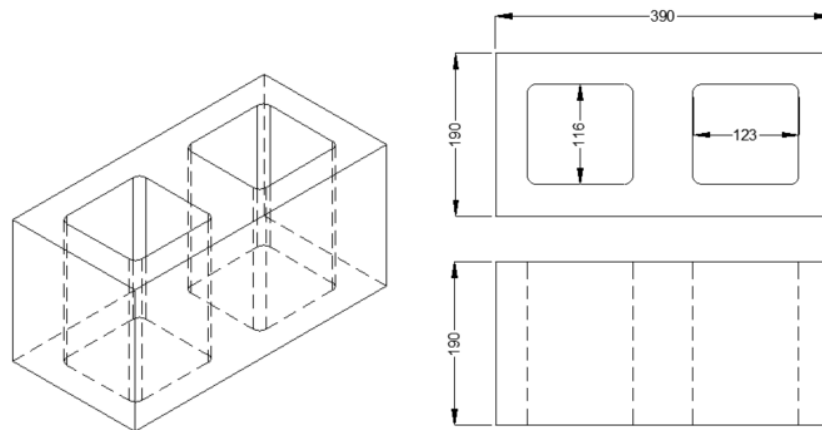


Figure 79.1 Dimensions of a masonry block

3.3.1.1 Wall 1

The retrofit methodology implemented for Wall 1 was based on controlling flexural deformations by longitudinal steel reinforcement and controlling shear deformations by CFRP sheets. The principle approach was to take advantage of the empty cells inside masonry blocks and create reinforced concrete columns on either side of the wall. The empty cells in each block of masonry lined up inside the wall creating hollow ducts, which were suitable for placing reinforcement and subsequent filling with concrete.

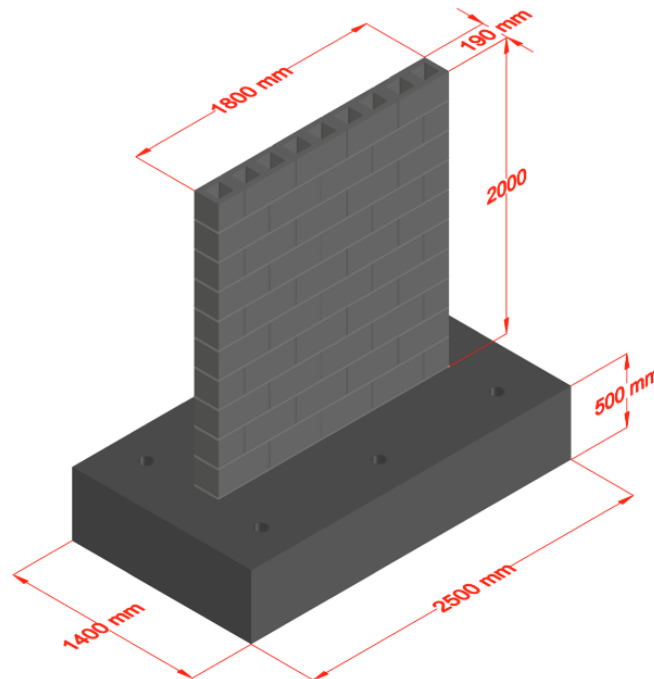


Figure 80.2 Masonry wall and footing dimensions

The following steps explain the sequence of the retrofit procedure employed with references made to appropriate sketches.

1. Two parallel vertical cuts were made at each wall end, resulting in four hollow vertical ducts along the wall height. These open spaces provide ideal locations to mount steel reinforcement inside the wall. The duct locations are shown in Figure 3.3.
2. Two holes were drilled in the foundation right underneath each wall duct for anchoring the vertical reinforcement into the foundation.
3. Steel reinforcing bars were placed inside the ducts at both ends of the wall, and the bars were inserted into the holes in the foundation. The holes were cleaned prior to placing the re-bars and filled with epoxy for proper anchorage. The embedment length of

reinforcement bars was 300 mm. Figure 3.4 shows the details of anchored steel reinforcement at one side of the wall.

4. The open side of the ducts on the front face of the wall were covered with sheets of plywood to allow for concrete placement in the ducts. The plywood was clamped on the wall to serve as formwork during concrete casting.
5. High-slump concrete was poured inside the ducts from top of the wall. The plywood sheets were then removed after the concrete gained its initial strength. Figure 3.5 illustrates the wall after the removal of the formwork.

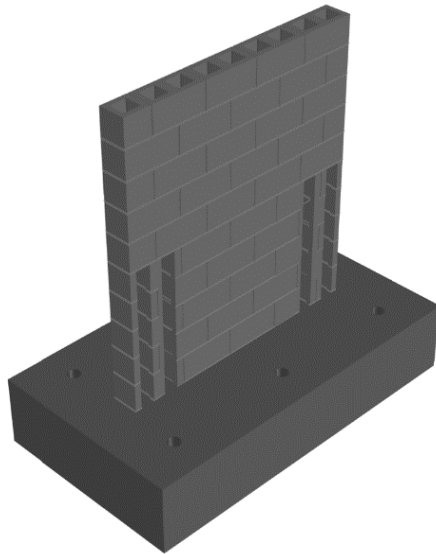


Figure 81.3 Two ducts being cut on either side of the wall

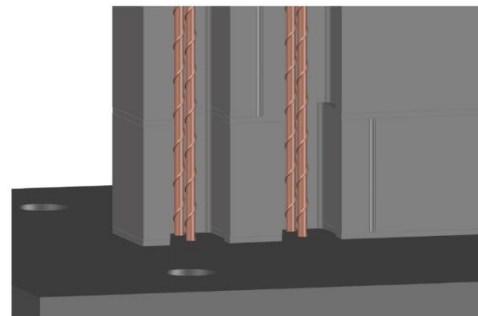


Figure 82.4 Longitudinal steel reinforcement

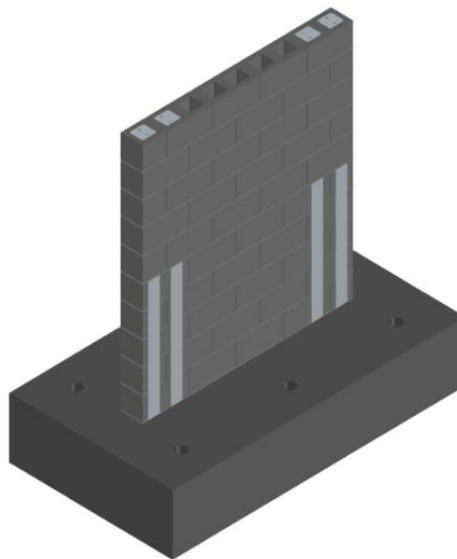


Figure 83.5 Concrete columns cast inside the wall

6. Two layers of FRP sheets were placed on the side that had been cut and filled with concrete using Epoxy substrate. The first layer was placed with fibers oriented in the horizontal direction, and the second layer was placed on top of the first layer with fibers oriented vertically. The primary purpose of the FRP sheets was to control diagonal tension caused by shear while also stabilizing the blocks in the out-of-plane direction. Figure 3.6 shows the FRP sheets attached on one side of the masonry wall specimen and Figure 3.7 shows the sides and plan view of the Wall 1.

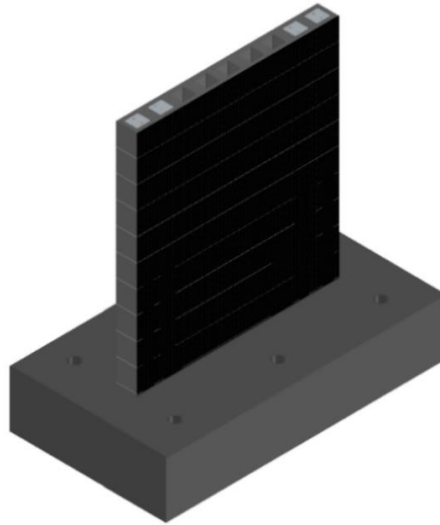


Figure 84.6 Two layers of FRP sheets attached horizontally and vertically on one side of the wall

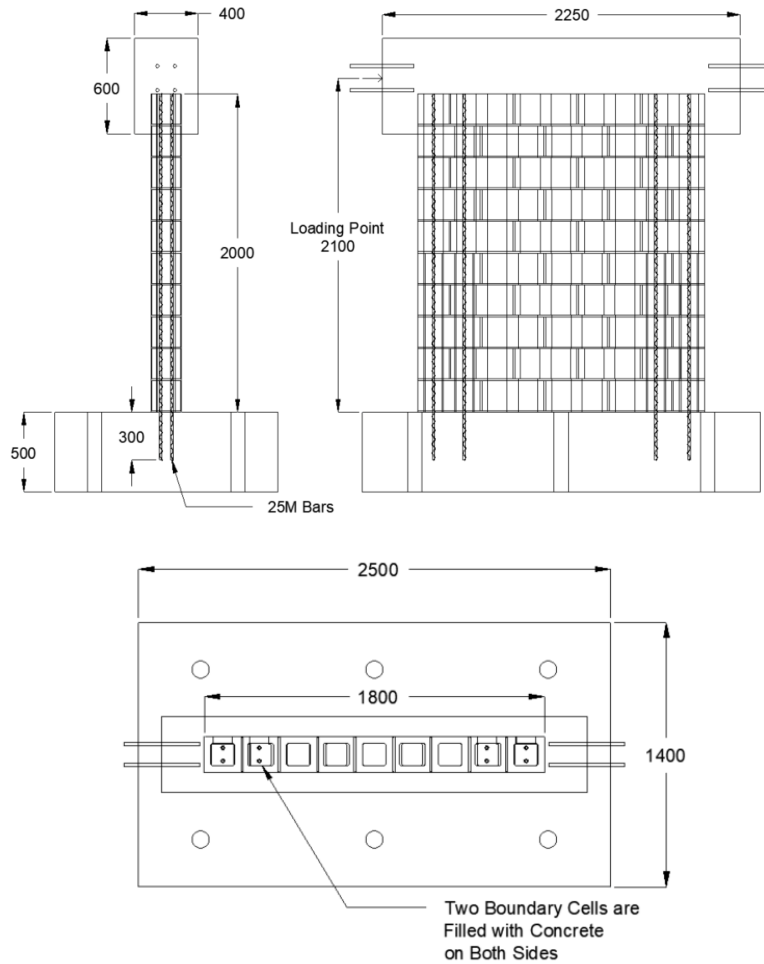


Figure 85.7 Sides and plan views of Wall 1, dimensions are in mm

3.3.1.2 Wall 2

The retrofit methodology for Wall 2 was based on increasing flexural strength while controlling flexural deformations by longitudinal post-tensioning strands and controlling diagonal tension by surface-bonded CFRP sheets. The idea of post-tensioned concrete columns was developed to generate a self-centering system for masonry walls. The FRP sheets placed on the wall surface controlled shear deformations and ensure the behaviour of the wall as a solid panel. The post-tensioned columns created at the ends controlled the overturning moment and toe-crushing. The retrofitting system was designed in such a way that the post-tensioning tendons resisted tension due to overturning, and the concrete cast around the tendons provided compression resistance against toe crushing. The tendons provided un-bonded post-tensioning. They were coated with grease, and covered by plastic sheathing, preventing bond between the steel and concrete. The following steps explain the retrofit process.

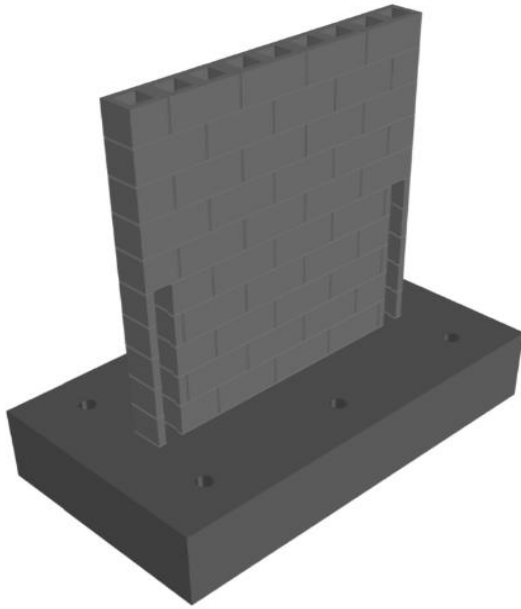


Figure 86.8 One-face shell cut on each side of the wall

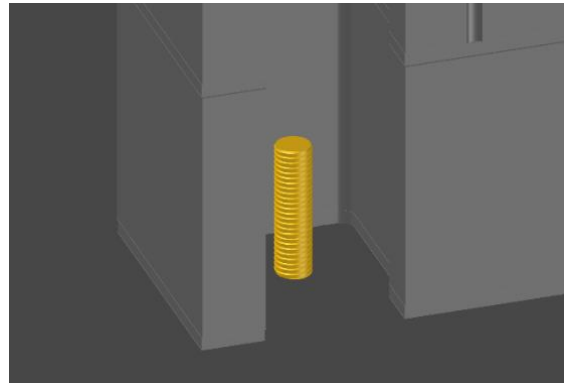


Figure 87.9 Threaded rod glued inside the drilled hole

1. Two strips of face shell were cut near the wall ends to create ducts similar to those explained in Step 1 of Section 3.3.1.1 for the previous retrofit method. The ducts were to be used for placing post-tensioning tendons. This is shown in Figure 3.8.
2. A hole was drilled in the foundation at the center of each duct. These holes were used to secure bolts, which were subsequently attached to post-tensioning anchors.
3. A threaded bar was anchored inside the drilled holes by epoxy glue as shown in Figure 3.9. The top (shorter) segment of the threaded bar projected out of the footing for assembling the prestressing anchorage system. The embedment length of threaded bars were 300 mm.
4. Prestressing anchorage cylinders, which were manufactured at the University of Ottawa Machine Shop, were used for post-tensioning. These anchors consisted of two cylindrical segments that were bolted together. The bottom segment had a threaded hole at the bottom that was attached to the installed threaded bar in the foundation. The

- top part had a conical opening for fitting the prestressing wedges. Figure 3.10 shows the components of a typical anchorage cylinder used to anchor the post-tensioning strands. Figure 3.11 illustrates the attachment of the cylinders to the anchor bolt.
5. Post-tensioning cables were placed vertically inside the available ducts and fixed to the prestressing anchors at the bottom. This is illustrated in Figure 3.12. The post-tensioning was applied by means of hydraulic jacks placed at the top of the wall.

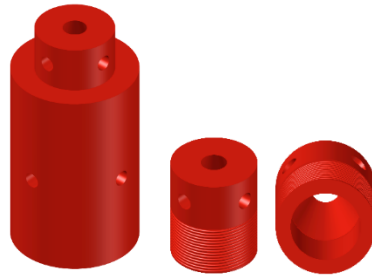


Figure 88.10 Prestressing anchorage cylinder

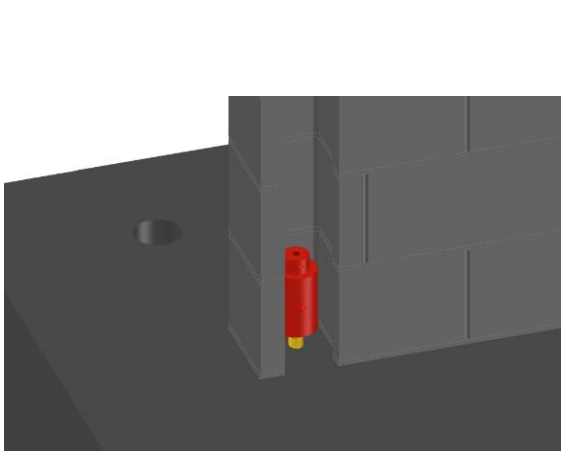


Figure 89.11 Prestressing anchor installed on the threaded rod

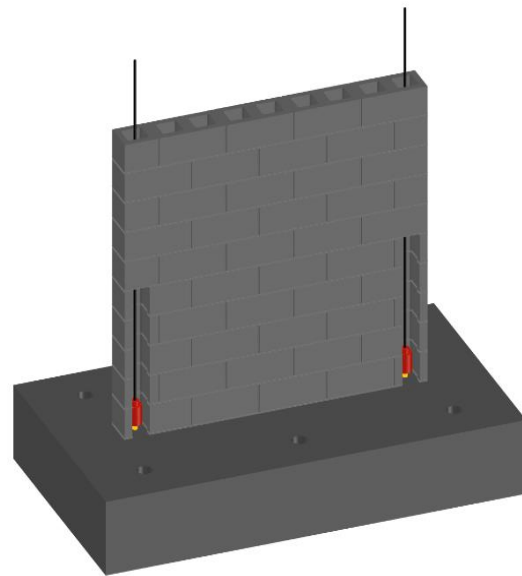


Figure 90.12 Post-tensioning cables fixed to the anchors

6. The open sides of the ducts were covered by plywood sheets that were clamped to the wall, acting as formwork for the placement of concrete inside the ducts.
7. High-slump concrete was poured inside the ducts from tip of the wall until it reached

- the top. The plywood sheets were removed after the concrete was set. By this approach, two concrete columns with a post-tensioning strand in each, were cast inside the masonry wall ends (Figure 3.13). The post-tensioning cables were longer than the concrete columns by one meter. The extension of cables passed through the loading beam during the experiment, as explained in the following steps. Based on the results of Wall 1, it was decided to fill all masonry cells of the wall 2 with concrete and place three vertical 10M bars inside the web of the wall, which is shown at Figure 3.18. These modifications are explained in Chapter 4.
8. The wall surface was then covered by two layers of FRP sheets, one layer in the vertical, and the other layer in the horizontal direction, as shown in Figure 3.14 to control diagonal tension caused by shear.

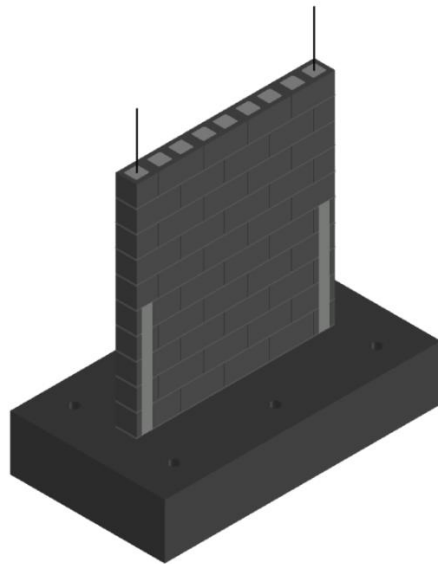


Figure 91.13 Two concrete columns cast inside the wall

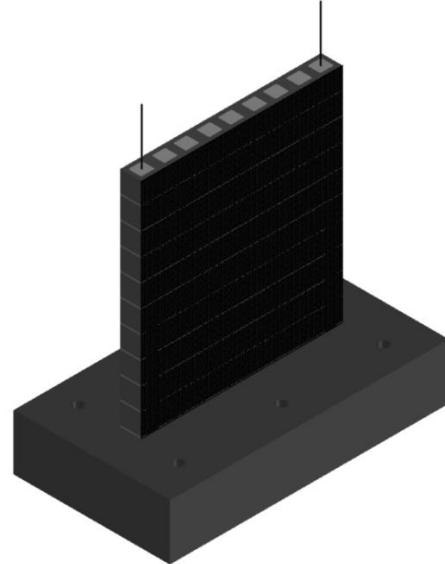


Figure 92.14 Surface-bonded FRP sheets on one side of the wall

9. The walls were tested by applying incrementally increasing deformation reversals. They were subjected to a constant axial load, which was applied through a precast reinforced concrete beam that was placed on top of the wall. The loading beam had a rectangular hollow section at the bottom to fit to the top of the wall. Figure 3.15 shows the loading beam from different angles. Bolts had been cast inside the loading beam at two ends, parallel to the longitudinal beam axis as shown in the figure. These bolts were used to connect the wall to the MTS actuator for applying the horizontal load. The extensions of the post-tensioning strands at the top of the wall passed through the holes

that had been drilled in the beam at corresponding locations.

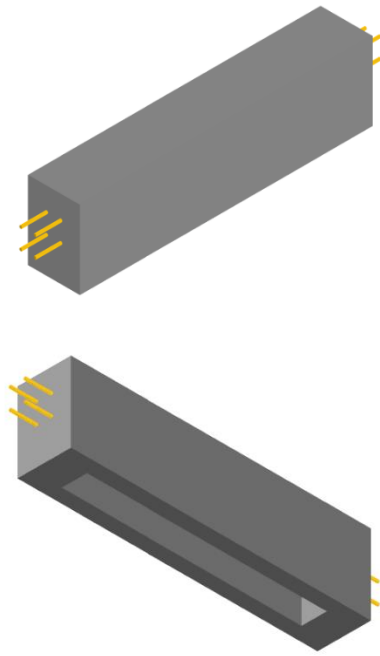


Figure 93.15 Loading beam

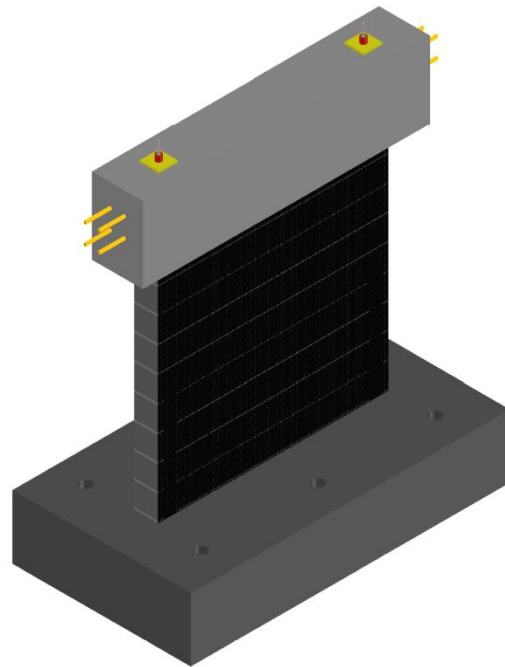


Figure 94.16 Loading beam in-place

10. The post-tensioning was done by means of a hydraulic jack, at the top of the beam. The post-tensioning anchors used on top of the wall are shown in Figure 3.17. The sides and plan view of Wall 2 are shown in Figure 3.18.

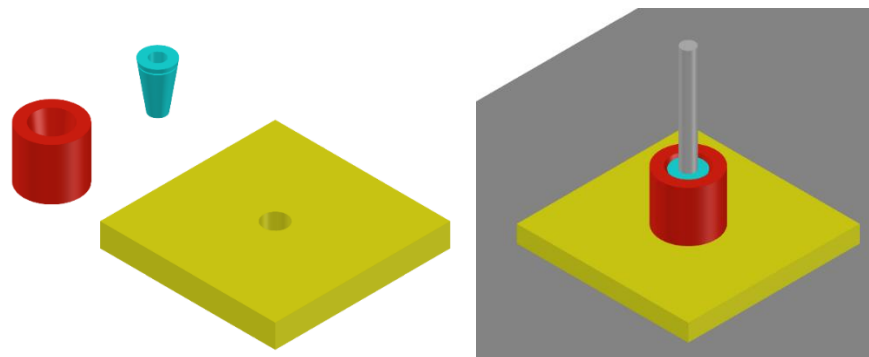


Figure 95.17 Prestressing anchorage system (Barrel and Wedges) on top of the loading beam

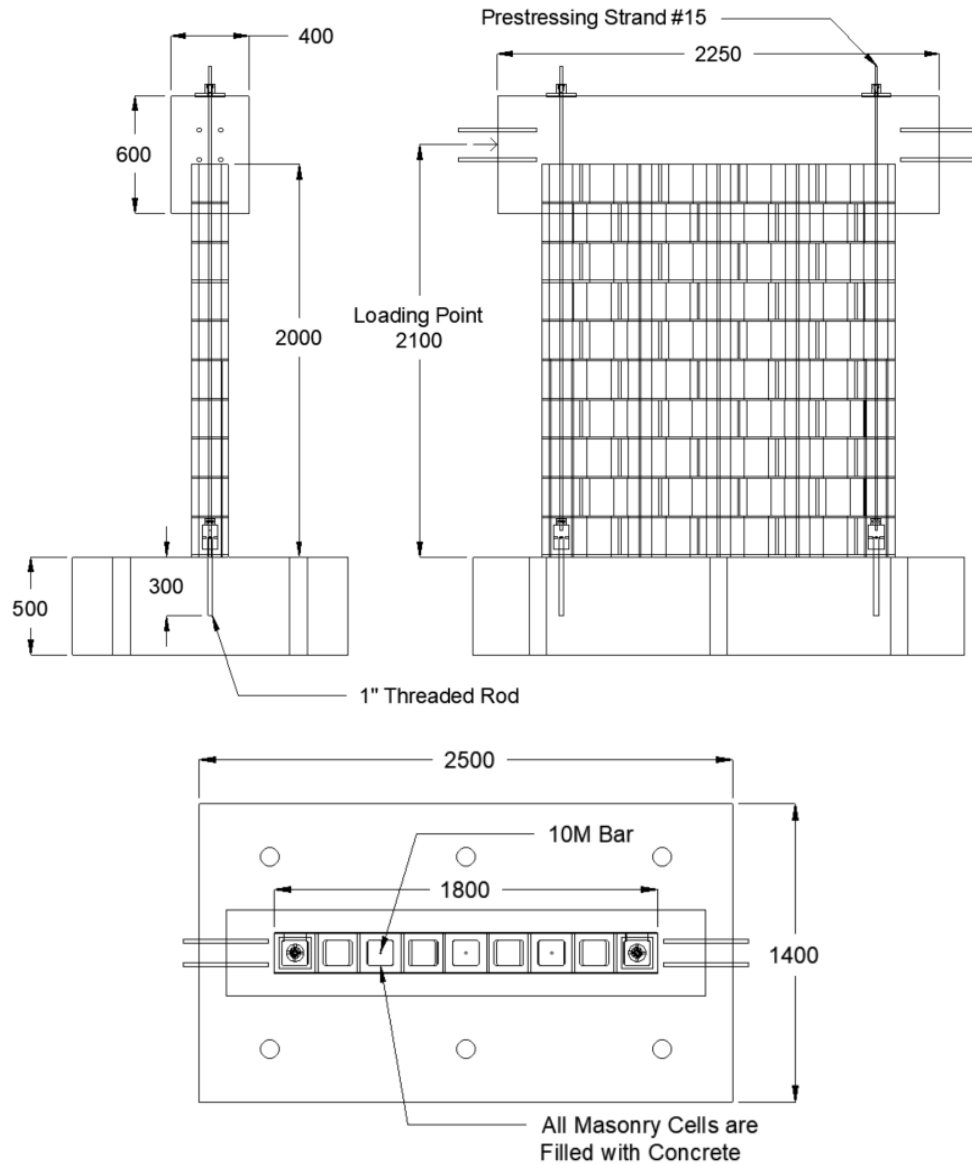


Figure 96.18 Sides and plan views of Wall 2, dimensions are in mm

3.3.2 Material Properties

The properties of different materials that were used in the experimental research are presented in this section. The materials consist of steel reinforcements, post-tensioning tendons, concrete blocks, different batches of concrete and mortar, and FRP sheets. The FRP material properties and bond properties used in the analytical investigation discussed subsequently were obtained from the manufacturer's data sheet and Shah Arifuzzaman's master thesis (2013).

3.3.2.1 Steel

Three types of reinforcing steel were used in the walls, consisting of 10M (with an area of 100 mm^2) and 15M (with an area of 200 mm^2) steel rebars and size 15 high strength prestressing strands (with an area of 140 mm^2). Three coupon tests were performed for each types of steel and the average stress-strain relationships were obtained. The stress-strain graphs for the rebars are shown in Figures 3.19 and 3.20. A typical coupon test, illustrating testing of the 7-wire strand is illustrated in Figure 3.21. The 7-wire strand coupon was secured between two end plates by means of two prestressing anchorage barrels and wedges. The stress-strain graph for the prestressing strands is depicted in Figure 3.22.

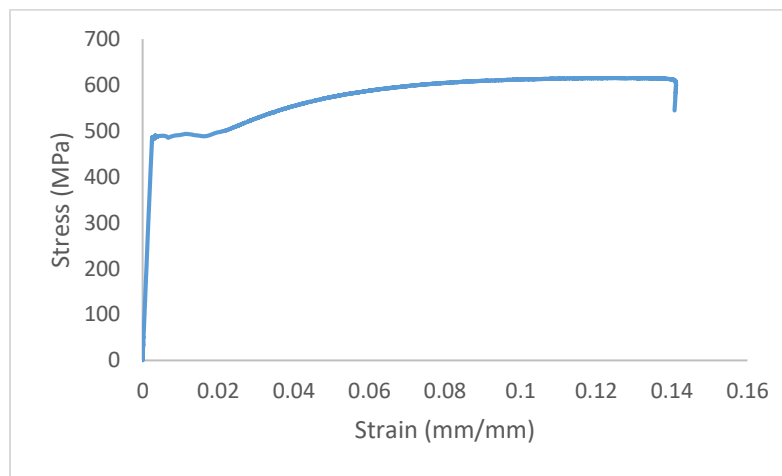


Figure 97.19 Stress-Strain of Steel Reinforcement Size 15M

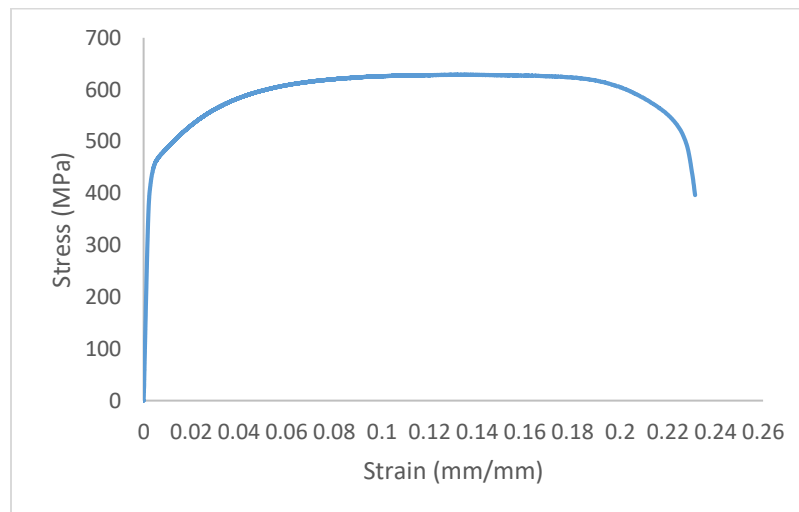


Figure 98.20 Stress-Strain of Steel Reinforcement Size 10M



Figure 99.21 Coupon test setup for prestressing strand #15

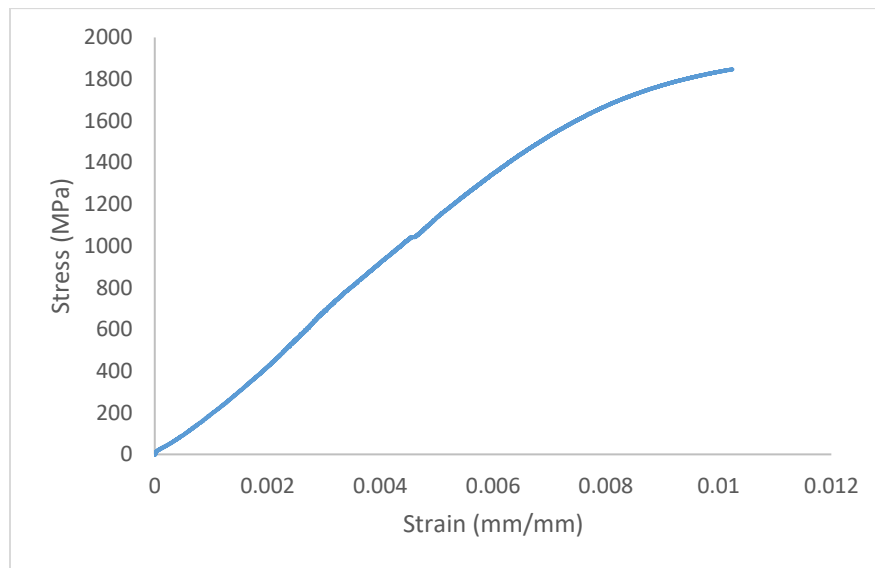


Figure 100.22 Stress-strain of prestressing tendon size #15

3.3.2.2 Concrete and Mortar

Different batches of concrete and mortar were used in the two masonry walls. Samples were taken from each batch and compression tests were performed. The concrete that was used in foundations was ordered from a ready-mix concrete company. The specified concrete strength for

concrete was 50 MPa. The average of 6 cylinders tested during the wall test period indicated a concrete strength of 62 MPa.

Two batches of concrete were mixed in the Structures Laboratory for casting concrete in the boundary element cells of masonry in Walls 1 and 2. The average strength of concrete for these two batches was 57 MPa. A separate batch of concrete was mixed later for filling the remaining central cells in Wall 2 for improved diagonal compression resistance. The average concrete compressive strength for this batch was 55 MPa on the day of wall test. Figure 3.23 shows a typical concrete cylinder failure. The mix design of concrete batches used to fill the masonry cells is presented in Table 3.1. Superplasticizer was used to increase concrete workability. Concrete had to have high slump to easily flow inside the masonry cells and fill all the gaps. The measured slump was 170 mm, which was suitable for filling the masonry cells within a 2.0 m height.

Table 20 Mix design for concrete in masonry cells

Total Volume (m^3)	Cement (kg)	10 mm Aggregate (kg)	Sand (kg)	Water (kg)	Super Plasticizer (kg)
0.23	69	270	138	36.8	0.8

The mortar that was made by masons during the construction of the walls was cast in six cubes. The average compressive capacity of these cubes was 7 MPa.



Figure 101.23 Failure of a concrete cylinder in compression

Four concrete blocks were cut in half and tested in compression to establish the compressive capacity of concrete blocks. This is illustrated in Figure 3.24. The average capacity

of masonry blocks was determined to be 14.74 MPa, which was calculated based on the ultimate load and the geometric dimensions of the block. Typical failure of concrete block is shown in Figure 3.24(b).

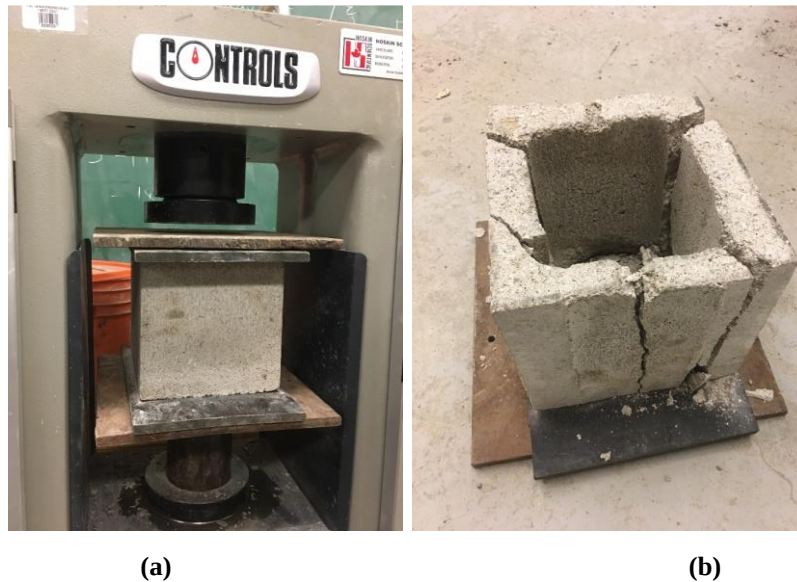


Figure 102.24 (a) Testing of half-block masonry in compression, (b) Failure of concrete masonry block in compression

3.3.3 Preparation of Wall Specimens

The masonry walls were constructed in the Structures Laboratory of the University of Ottawa. The following steps explain the construction process.

3.3.3.1 Footings

Two reinforced concrete footings were constructed as foundations for the masonry walls. The construction of footings involved the preparation of formwork, the assembly of steel reinforcement cage, and the casting of concrete. Figure 3.25 illustrates selected stages of construction.



Figure 103.25 Construction process of footings

Four plastic pipes were secured inside each formwork to provide holes inside the concrete through which the foundation is attached to the laboratory strong floor. Six threaded bars were cast vertically inside the concrete in two rows of three. These bolts were to be used for the application of the axial load by means of vertical prestressing tendons. Pre-bent 25 M bars were used as footing reinforcement. The footings were over-designed to ensure that they remain elastic and rigid during the tests. Hooks were also cast in concrete for subsequent handling purposes.

3.3.3.2 Walls

Professional masons were hired to construct two unreinforced masonry walls, which were built on top of their respective foundations. Figure 3.26 illustrates selected stages of construction.

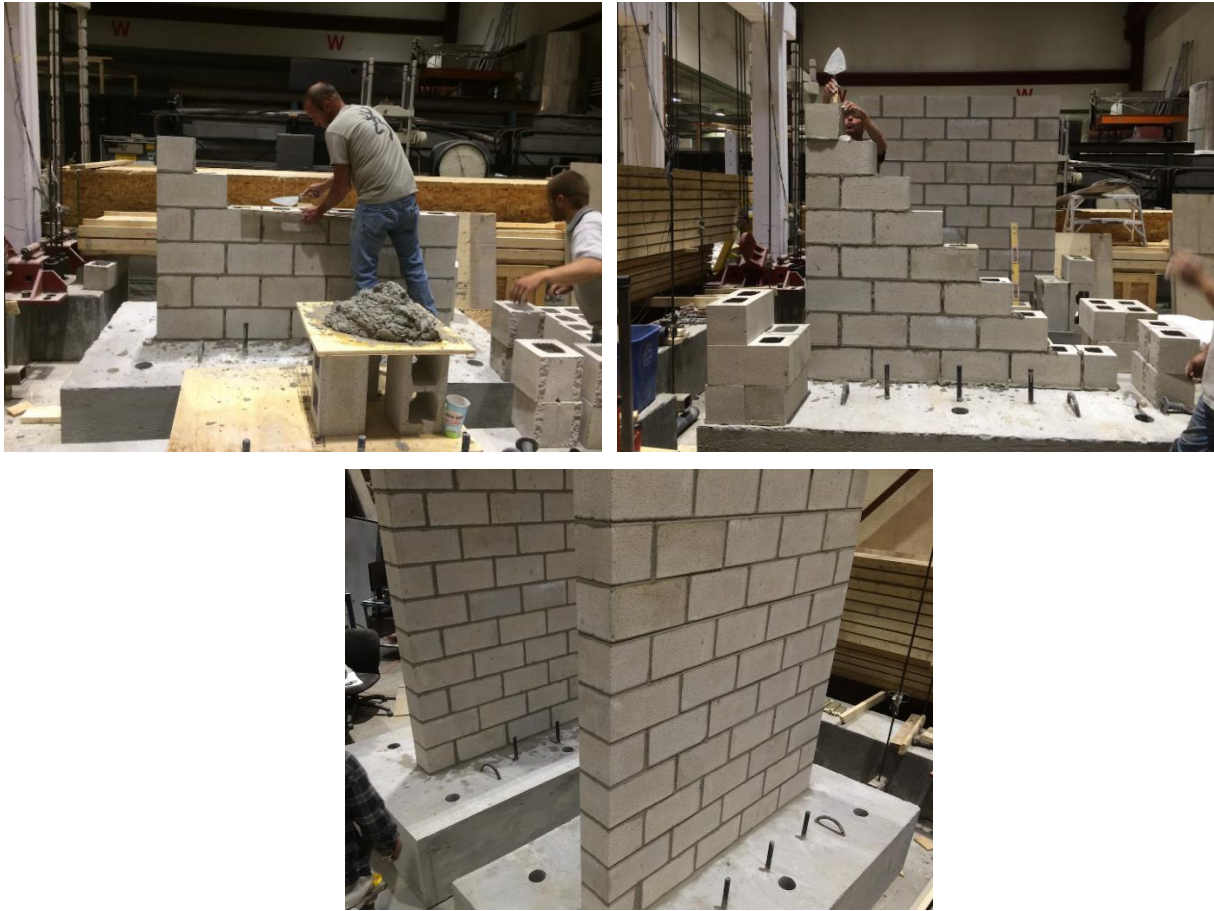


Figure 104.26 Masonry wall construction by professional masons

Regular concrete masonry units were used to build the walls similar to the companion as-built masonry wall that was tested by Taghdi et al. (2000a). Mortar type O was used to represent old masonry structures. Mortar type O has 1 unit of Portland cement, 2 units of hydrated lime, and 9 units of fine aggregate (sand) in damp, loose condition. Mortar cylinders were obtained to capture the strength of mortar on test dates. The walls were cured by spraying water for one week.

3.3.4 Retrofitting Phase

Two different retrofit strategies were applied on each wall as discussed in Sections 3.3.1.1 and 3.3.1.2. The following sections explain the sequence of construction for each wall.

3.3.4.1 Wall 1

Professional concrete cutters were hired to cut the face shell of masonry to create two concrete ducts on each side of the wall. The cutting process was very fast and easy because of the hollow nature of the units. This is shown in Figure 3.27. Two holes were drilled under each duct

for the placement of eight 15 M steel rebars, four at each end (Figure 3.28). SDS rotary hammer drill was used for this purpose. The hole size was established based on the bar size and the specifications of the epoxy that was used for anchorage. Figure 3.29 illustrates the wall prior to the placement of reinforcement bars. Sika Anchorfix 3 was used as a two-compound epoxy to anchor the steel bars inside the foundation (Figure 3.30). Wooden formwork was built with plywood and studs to cover the exposed parts of the ducts on the wall. The formwork was secured to its place by means of through bolts that were tightened on both sides of the wall with washers and nuts (Figure 3.31). A batch of self-leveling concrete was mixed. The concrete was cast inside the ducts from a concrete bucket that was lifted with crane. A wooden cone was placed on top of the wall to direct the concrete inside the ducts. The formwork was removed 24 hrs after casting (Figure 3.32).



Figure 105.27 Cutting of masonry walls to create open ducts

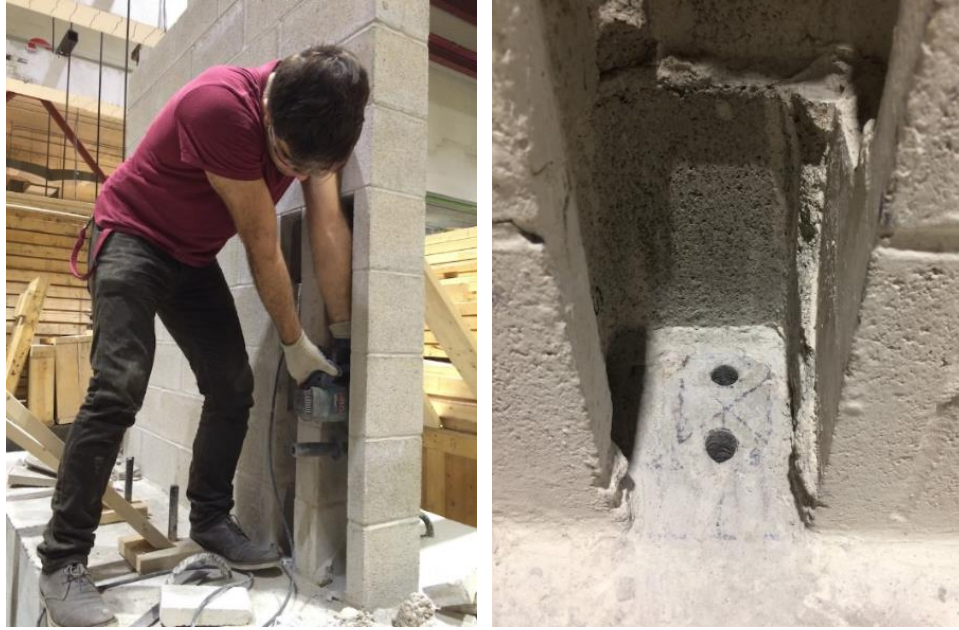
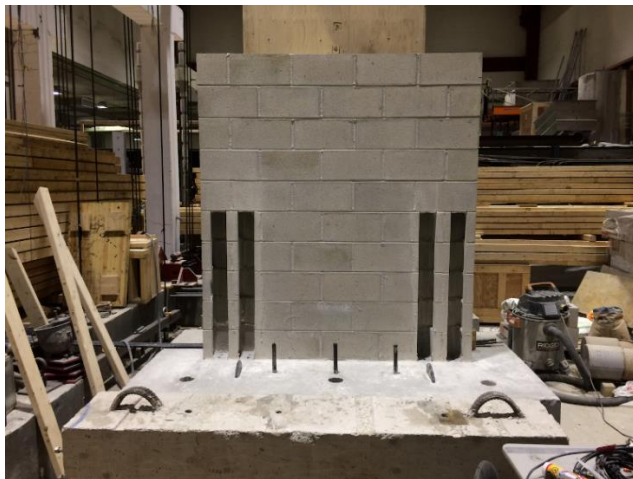


Figure 106.28 Drilling holes inside foundations by hammer drill



(a)



(b)

Figure 107.29 (a) Masonry wall after cuts, (b) strain gauges attached on reinforcement



Figure 108.30 Installation of steel reinforcements inside pre-drilled holes by epoxy glue



Figure 109.31 Installation of formworks on top of the open ducts by bolts

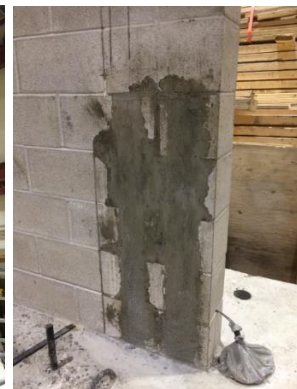


Figure 110.32 Left: Casting of concrete inside the masonry wall cells, Right: Masonry wall ducts after casting of concrete

3.3.4.2 Wall 2

The cutting process of masonry face shell was similar to that for Wall 1 but instead of two ducts on each side, one duct was opened. A 25 mm (1.0 in) diameter hole was drilled under each duct for the placement of threaded rods. Threaded rods were anchored inside the holes by means of Sika Anchor 3 adhesive as illustrated in Figure 3.33(a). Prestressing anchorage cylinders were installed on the threaded rods as depicted in Figure 3.33(b). The post-tensioning cables were temporarily secured inside the ducts by means of anchoring cylinders at the bottom and prestressing anchors at the top. This is shown in Figure 3.34. The cables were slightly tensioned to remain straight during concrete casting and the prestressing anchorage cylinders were covered by plastic sheets to prevent concrete penetration. Figure 3.35 illustrates the wall prior to filling the ducts with concrete. Strain gauges were attached to the threaded rods to measure post-tensioning

force in the cables after the removal of the jacks. Concrete was cast inside the ducts at wall ends similar to Wall 1 (Figure 3.36).



(a)



(b)

Figure 111.33 (a) Threaded rod installed in pre-drilled hole, (b) Anchorage cylinder on top of threaded rod



Figure 112.34 Post-tensioning cable secured at its place between to prestressing anchors



Figure 113.35 (a) Post-tensioning strand and anchor (covered by plastic) prior to concrete placement

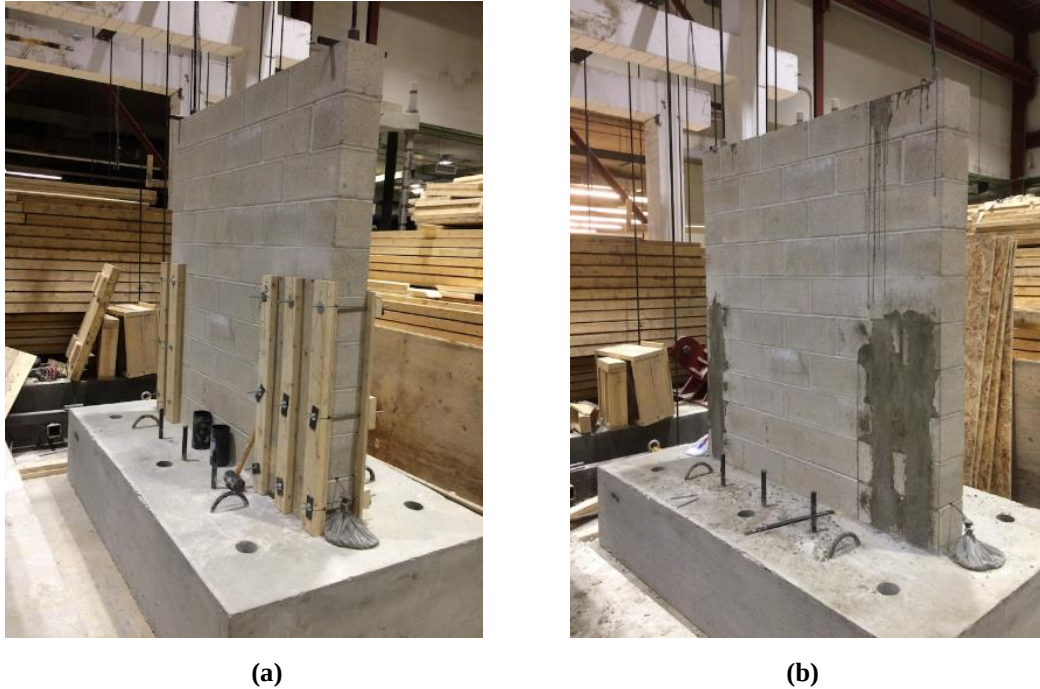


Figure 115.36 (a) formwork installed on top of ducts, (b) Masonry ducts after casting of concrete

3.3.4.3 FRP Placement

The placement of FRP sheets consisted of several steps. Surface preparation and FRP installation process depended on the type of FRP sheet and the epoxy resin used. The FRP sheets and the epoxy resin used in this research project were SikaWrap-230 C and Sikadur 330, respectively, both manufactured by Sika AG. The substrate surface had to be completely level with no irregularities or protrusions prior to the application of FRP. Any voids and gaps had to be filled with Sikadur 30, also from Sika AG. Extra concrete and mortar had to be removed by a grinder and the wall surface had to be cleaned by water. Figures 3.37 to 3.39 show the preparation of the wall surface for FRP application. This involved smoothening the surface, washing and drying the wall, filling the bed joints and deviations with Sikadur 30 (which is a two-component epoxy mortar) and grinding the surface again after curing of the epoxy mortar. The two component Sikadur 330 epoxy resin was applied on top of the wall surface at a rate of $0.7\text{--}1.2\text{ kg/m}^2$. The FRP strips (SikaWrap -230 C) that were cut to desired sizes were placed on top of the epoxy glue horizontally. All the irregularities and air pockets were removed by laminating roller. The second layer of epoxy glue was applied on top of the first later of FRP sheet at a rate of 0.5 kg/m^2 . Vertical layers of FRP strips were applied by repeating the same approach. A final seal coat of epoxy glue was applied at rate of 0.5 kg/m^2 over the final layer of fabric. The application of FRP

sheets is shown in Figures 3.40 and 3.41. Figure 3.42 shows both walls after the wall surface was made ready for FRP installation.



Figure 116.37 Masonry wall surface made level and clean by grinding and washing



Figure 117.38 Sikadur 30 epoxy mortar was used to fill irregularities and bed joints



Figure 118.39 (a) dried epoxy mortar was grinded to create smooth surface fore FRP attachment, (b) Sikadur 330 epoxy glue applied on top of the wall for FRP attachment



Figure 119.40 First layer of carbon FRP (SikaWrap-230 C) being attached on masonry wall

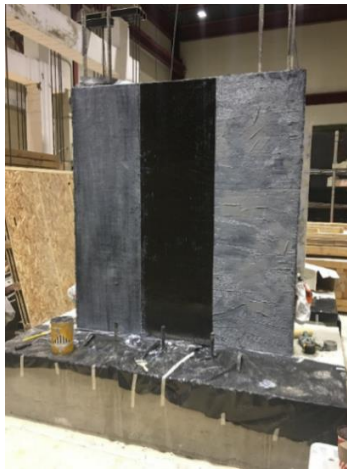


Figure 120.41 Installation of horizontal and vertical layers of FRP sheets



Figure 121.42 Retrofitted Walls

3.3.4.4 Shear Reinforcement Installation

Three 10M steel rebars were placed vertically inside Wall 2 and all the cells were filled with concrete. This was done to provide further diagonal tension capacity to the wall and prevent brittle shear failure. Wall 1 did not have enough shear capacity in the web, which affected the ductility and energy dissipation capacity of the wall. Figure 3.43 shows the location of 10M rebars before the installation. The bars were bent 180 degrees at both ends and were inserted from the top of the wall. Three strain gauges were attached on each rebar to measure strains during the test. Figure 3.44(a) shows the location of the centre rebars from the top of the wall. The strain gauges wires extended out of the wall to be connected to the data acquisition system. A batch of concrete was mixed in the Structures Laboratory and cast inside all the empty cells, including those that did

not contain reinforcement, with the intent of increasing diagonal compression capacity. This is shown in Figure 3.44(b).



Figure 122.43 Location of 10M steel reinforcements



(a)



(b)

Figure 123.44 Left: Placement of the rebars inside the wall from the top, Right: Top of the wall after cast of concrete

3.3.4.5 Installation of Reinforced Concrete Loading Beam

A reinforced concrete loading beam that was used earlier in the Structures Laboratory for testing similar walls in a companion test program was used to apply the horizontal load. The properties of this beam are explained by Arifuzzaman (2013) and are also briefly explained in Section 3.3.1.2. The installation of the loading beam is shown in Figure 3.45 and 3.46. The loading beam was lifted by the crane and placed on top of the wall. Extra spaces left between the masonry wall and the loading beam were filled with wood pieces and isolation foam (Figure 3.46(a)).

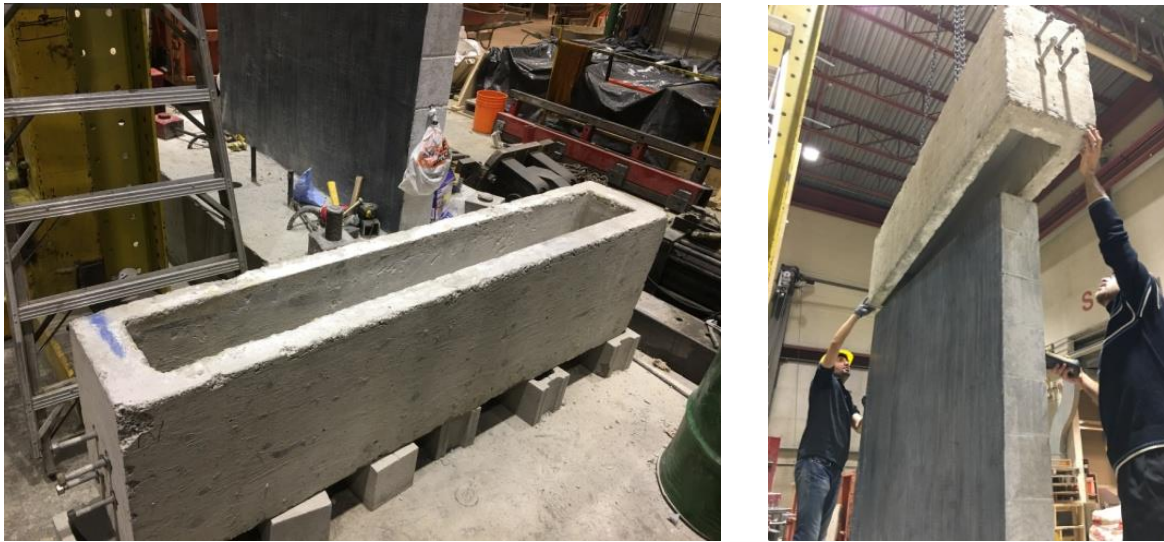
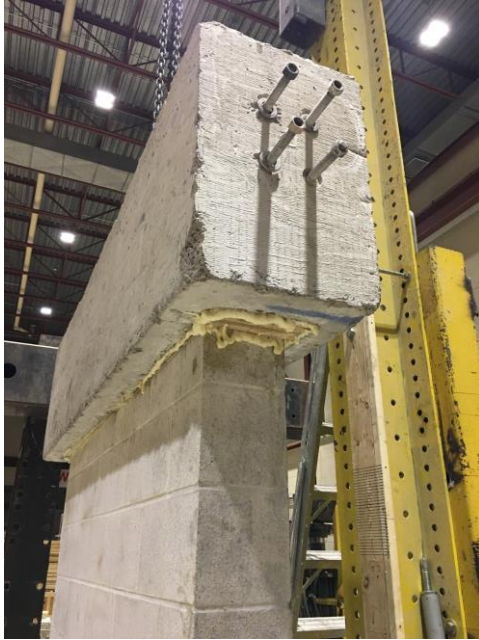
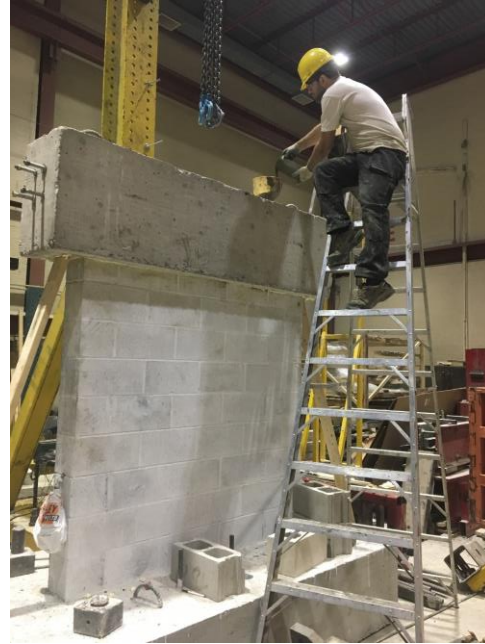


Figure 124.45 The reinforced concrete loading beam

A loose cement-based mortar was mixed and poured inside the loading beam through three holes on top of the loading beam (Figure 3.46(b)). The mortar filled all the extra spaces between the wall and the loading beam to create a strong connection.



(a)



(b)

Figure 125.46 The loading beam was secured at its place

3.3.4.6 Application of Post-tensioning

The required amount of post-tensioning in Wall 2 was investigated through sectional analysis of the wall. Moment-curvature diagrams of the wall were calculated with different levels of prestressing. The calculated values in the figure were obtained by an Excel based computer software. Figure 3.47 illustrates that the level of prestressing in tendons had a direct effect on in-plane stiffness of masonry walls. The level of prestressing in practical applications can be adjusted based on the required stiffness in the building. The prestressing level selected for the current project was equal to 80 kN of tension force per strand, though after the prestressing of all the strands the average prestressing force was measured to be 75 kN per tendon. The preliminary analysis of the wall indicated that this level of prestressing, with the size and the axial strand stiffness selected, provided sufficient wall lateral resistance and drift control without rupturing the steel and crushing the toe concrete under high levels of lateral deformations. The resulting prestressing corresponded to approximately 30% of the ultimate rupturing strength of the strands.

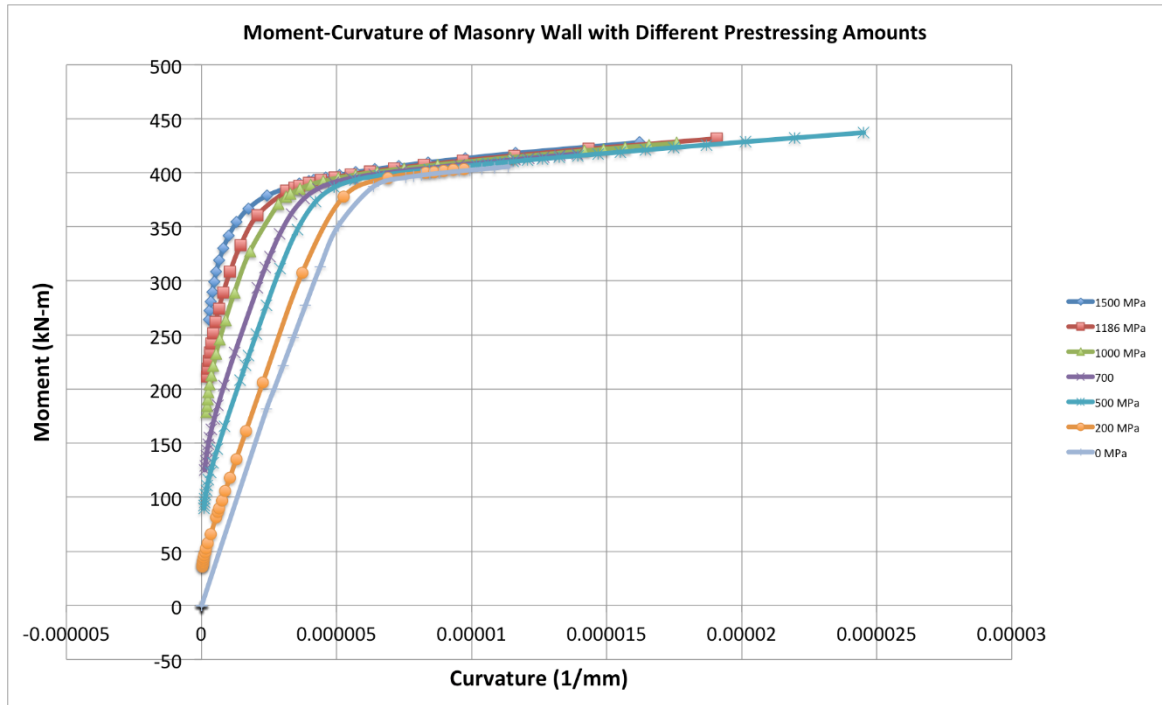


Figure 126.47 Moment-Curvature of masonry wall with different prestressing amount

The first step towards post-tensioning involved inserting the strands into the pre-drilled holes in the loading beam, one at each wall end. The top surface of the loading beam was not suitable for post-tensioning because of the pre-existing hooks that had been cast in the beam for handling purposes. Therefore, a steel box (HSS with end plates) filled with concrete for increased bearing capacity was used as a spacer. A smaller steel HSS section was placed on top of the steel box, through which the strand passed. This is illustrated in Figure 3.48. Although this assembly was initially used merely for construction convenience, the top HSS section served as a control mechanism to limit the strand force, by yielding prior to reaching the rupturing strength of strands while also preventing compression crushing of the wall toe region under a balancing compressive force. Therefore, these smaller HSS sections are referred to as “Yielding Agents,” serving as structural fuses. The details of their behaviour during testing and their role as illustrated in the finite element analysis discussed in Chapters 4 and 5, respectively. Strain gauges were installed on the strands in the HSS sections to measure the prestressing force as shown as the red mark in Figure 3.48. Figure 3.49 illustrates the post-tensioning operation.

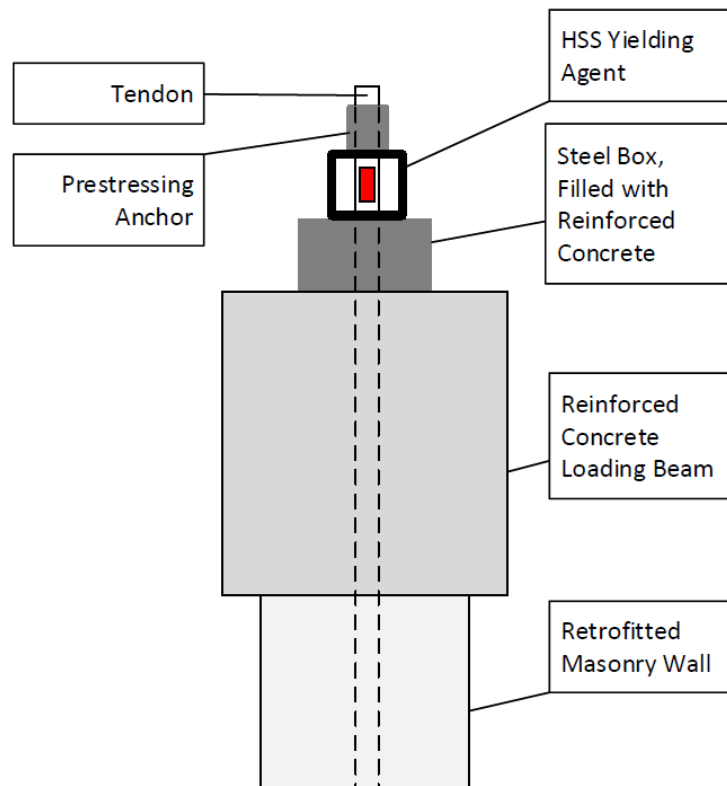


Figure 127.48 HSS-Tendon System (not scaled)

3.3.4.7 HSS Yielding Agent

Two HSS steel boxes were used during the prestressing process. These HSS boxes were installed under the prestressing anchors as shown at Figure 3.48. They were intended to provide an open space to protect the strain gauges that were installed on prestressing tendons. These boxes acted as yielding agents during the experiment that affected the overall structural behaviour of the wall. They had beneficiary roles. The post-tensioning force initially increased the stiffness of the wall. As lateral displacements increased the tension force in one of the tendons also increased. A balancing compressive force is developed in the toe region of the wall. It is conceivable that the increased tension and compression could lead to the rupturing of the tendon and the crushing of toe concrete. This did not happen during the test. Instead, it was observed that the increased tensile force in the tendon caused initially elastic bending, followed by the yielding of the HSS flange. This behaviour became evident when the hysteric relationship was inspected and the comparisons were made with the FEM analysis. Rupturing of steel and/or crushing of toe would cause a brittle behaviour with sudden drop of capacity, which is not desirable for a seismic resisting wall. These phenomena were observed in the analytical parametric investigation when HSS yielding devices

were not used. During the test, the HSS yielding agents had beneficiary roles by controlling the premature failure of the wall. This mechanism also increased the energy dissipation of the wall which is a desirable feature for an earthquake resisting system. Figure 3.49 (b) shows the location of one of the HSS sections during the post-tensioning process.

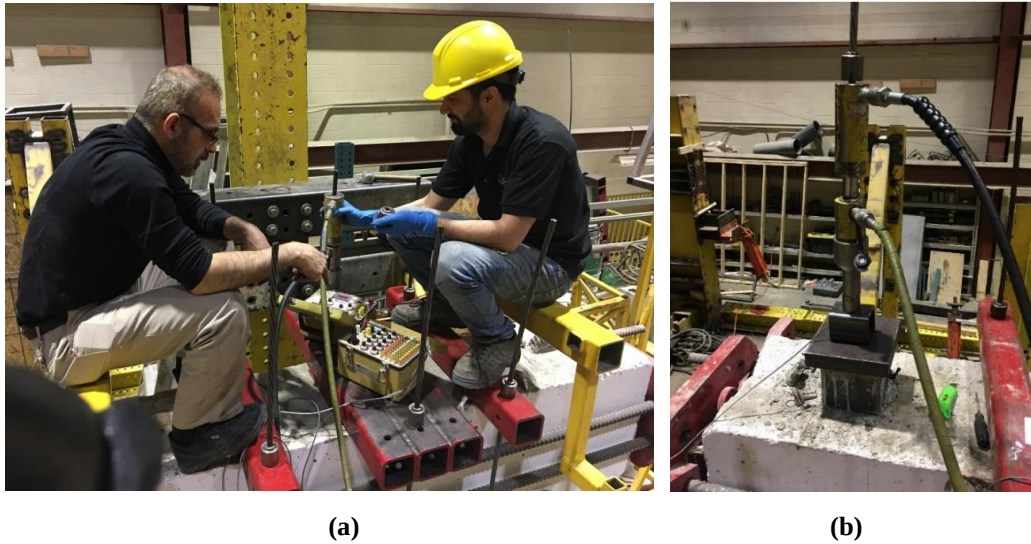


Figure 128.49 (a) Post-tensioning Process, (b) One of the tendons being prestressed to the desired level by two hydraulic jacks

3.3.5 Test Setup

Test setup consisted of a wall specimen with a foundation and a top loading beam, secured on the laboratory strong floor, and a mecano set made up of steel A-frames supporting a servo-controlled MTS hydraulic actuator in the horizontal direction. The wall was subjected to incrementally increasing horizontal displacements reversals with the MTS hydraulic actuator, which had a 1000 kN capacity. The gravity load on the wall was simulated by means of six prestressing tendons in two rows pulling the wall vertically, three tendons on either side of the wall. The wall footing was fixed to the laboratory strong floor by means of four 75 mm (3-in) steel bolts. An out-of-plane support system was designed and constructed to maintain wall response in the plane of the wall. The out-of-plane support system consisted of a steel frame that was bolted to a strong frame and four adjustable clamping wheels placed against the top loading beam that forced the wall to respond in its plane. The yellow steel components shown in Figure 3.50 illustrate the out-of-plane support system. Figure 3.51 shows the overall test setup.

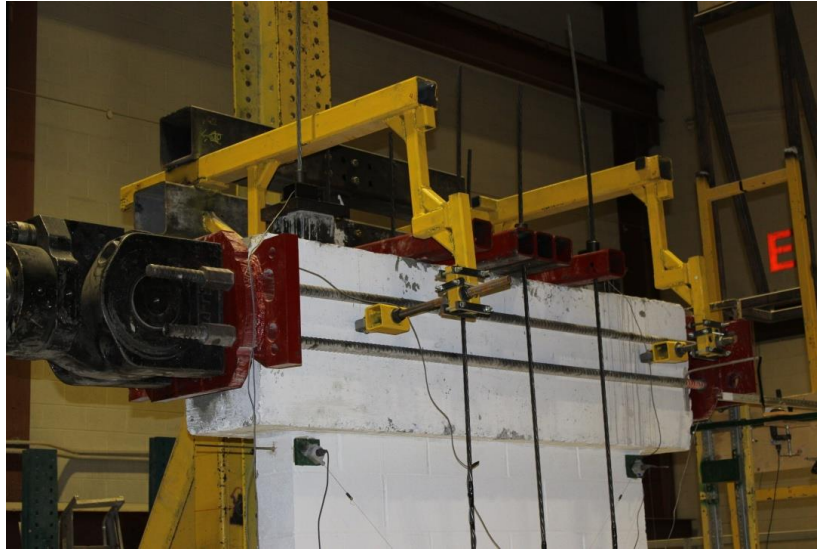


Figure 129.50 Out-of-Plane support (Yellow frame)



Figure 130.51 Experimental Test Setup

3.3.6 Instrumentation

Instrumentation used during the test consisted of cable transducers, LVDTs, actuator's load cell, and strain gauges. Figure 3.52 depicts the instrumentation arrangement for Wall 1. A cable transducer was used to measure the top displacement of the wall relative to the foundation. Other backup cable transducers were also used (not shown in the figure) to record backup readings. Two LVDTs measured the sliding of the wall at the toe region. These LVDTs were able to measure initial displacements accurately until the crushing of concrete occurred in the toe region. Three

LVDTs were used to measure horizontal and vertical movements of the footing (Figure 3.52). The actuator had a built-in load cell that measured the force in the actuator.

Strain gauges were installed on longitudinal steel re-bars and the prestressing tendons used for the application of gravity loads. Each strain gauge was labelled as indicated in Figure 3.52. The letter “F” in the label indicates strain gauges at the front (South side of the wall), and the Letter “B” indicates strain gauges at the back (North side of the wall).

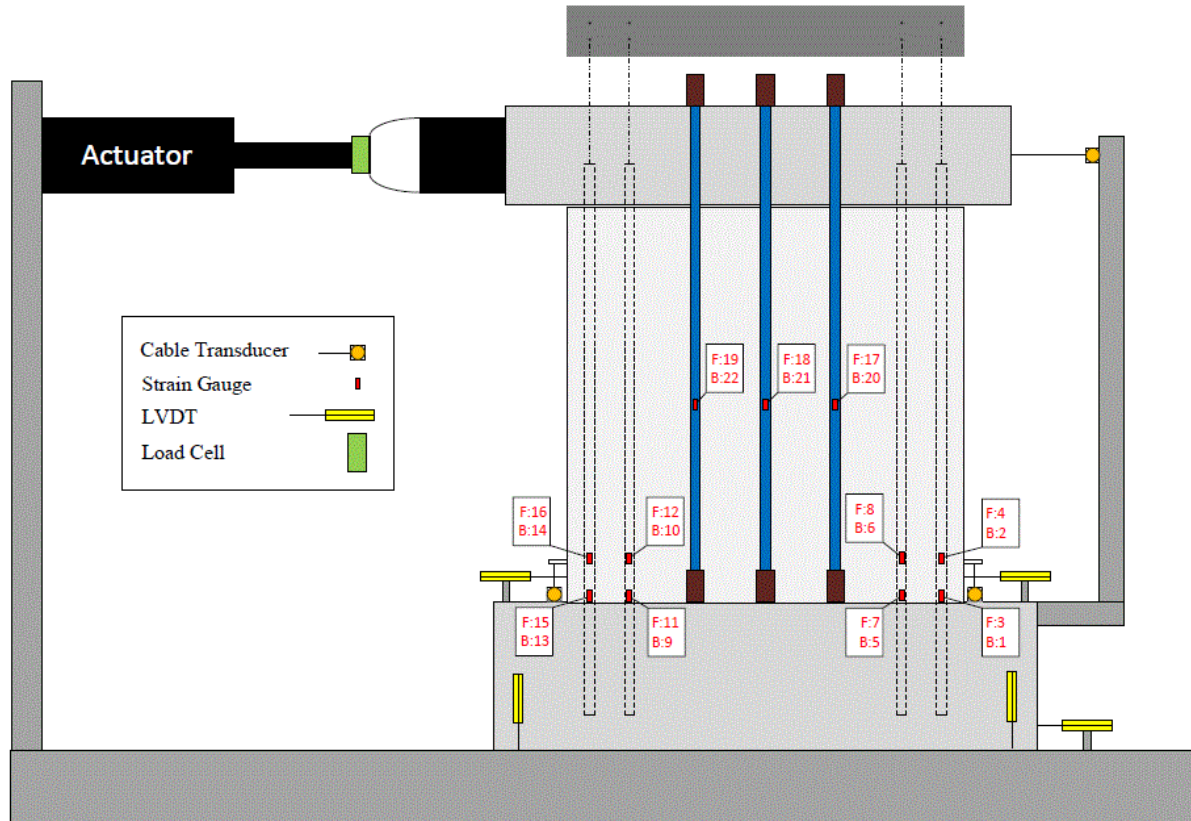


Figure 131.52 Instrumentation for Wall 1 (not scaled)

Similar instrumentation was used for testing Wall 2. The instrumentation for Wall 2 is shown in Figures 3.53, and 3.54. Strain gauges were located on the central longitudinal re-bars, tendons, and anchor bolts. The gauges placed on the 7-wire strands used for the wall retrofit, labelled as 1 and 2 were located at the top, whereas gauges 3, 4, 5, and 6 were located on the anchored 25 mm (1-inch) bolts at the bottom of the wall. The complete list of strain gauge results is presented in Appendix I. Some of the gauges were damaged and their graphs could not be presented.

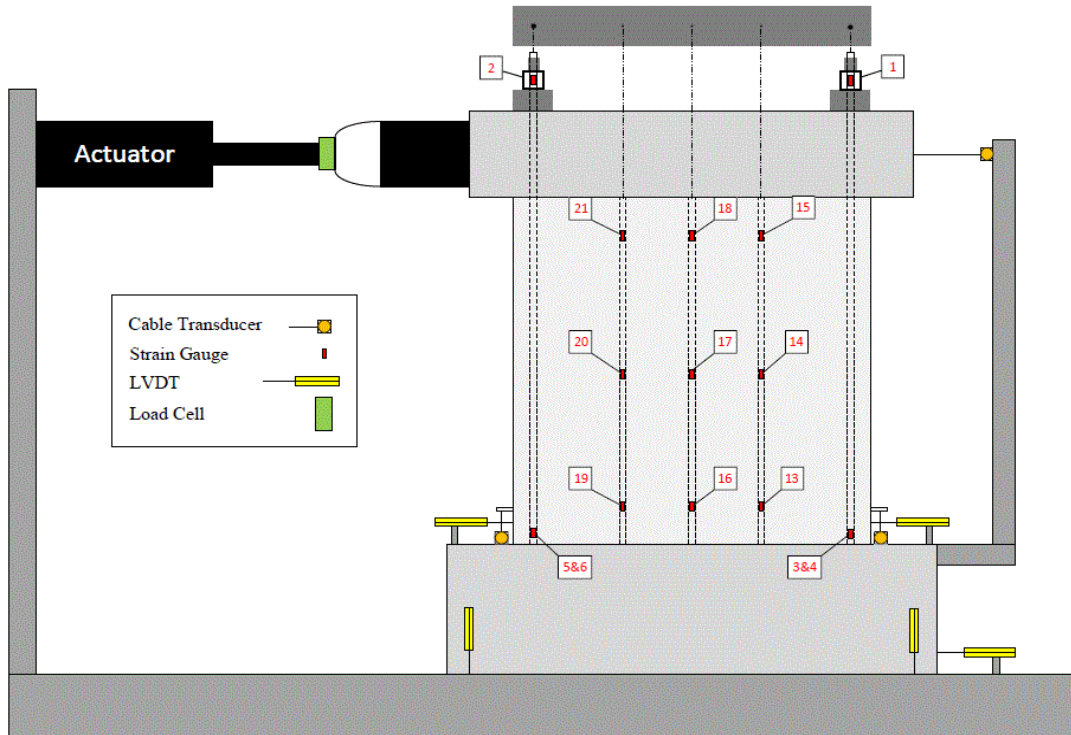


Figure 132.53 Instrumentation for Wall 2 (not scaled)

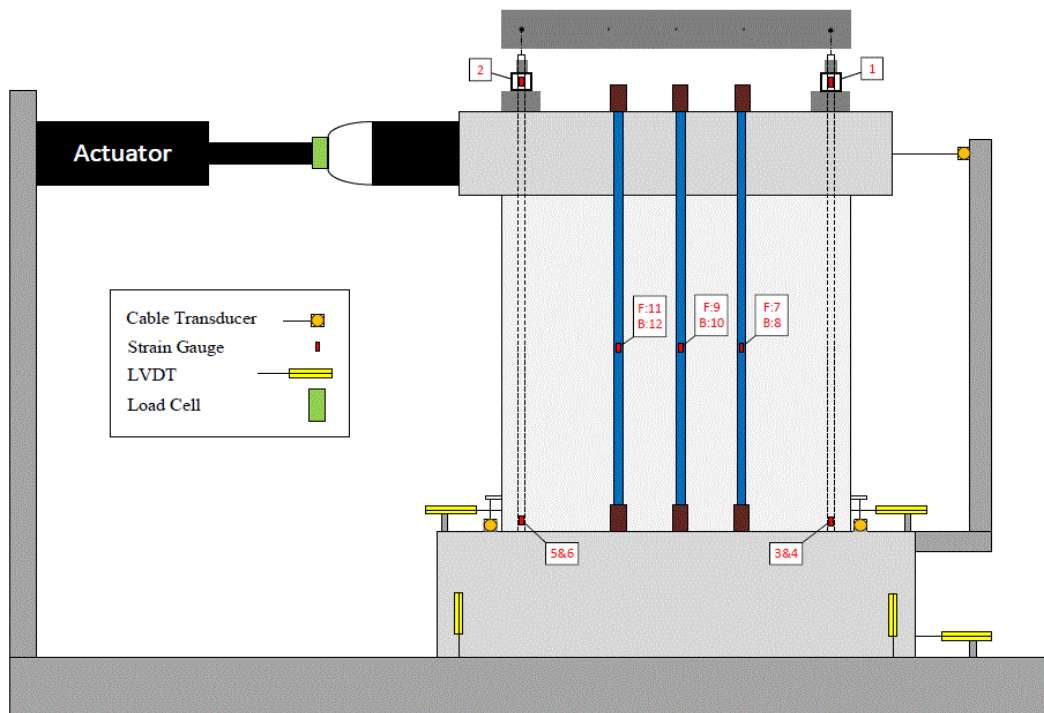


Figure 133.54 Strain Gauge numbering for tendons for Wall 2 (not scaled)

A close-up view of a wire gauge and an LVDT used at the base of the walls is shown in Figure 3.55(a). All the instruments were connected to a data acquisition system, which is shown

in Figure 3.55(b).

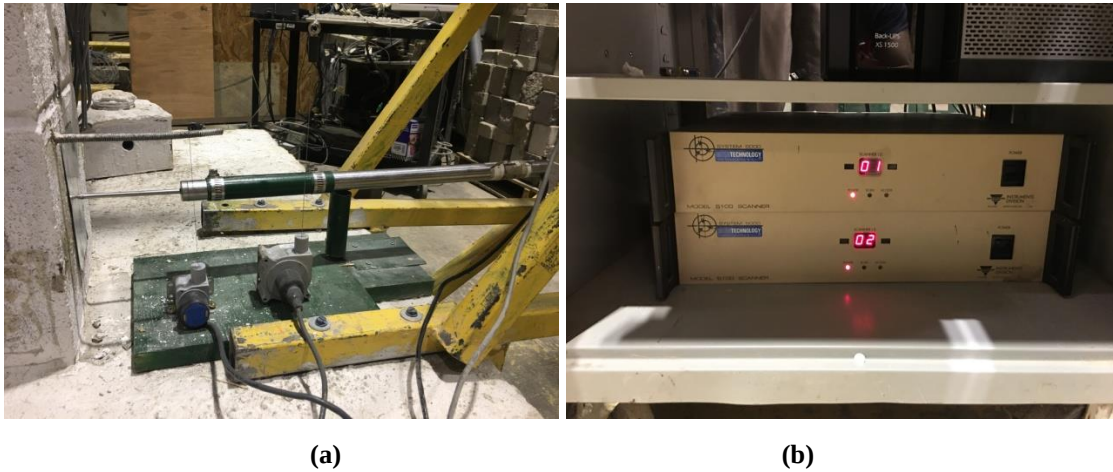


Figure 134.55 (a) Typical Cable Transducer and an LVDT; (b) Data acquisition system used during testing.

3.3.7 Loading

The walls were subjected to gravity loads first. They were retrofitted under gravity load to simulate actual construction practice prior to the lateral deformation reversals. The gravity load was applied by means of 6 size 15 7-wire high-strength prestressing strands (each with an area of 140 mm^2), with uniform spacing, three on each side. The total load applied was 100 kN, approximately 17 kN per tendon.

The lateral load reversals were applied in displacement-control mode as shown in Figure 3.56. The displacement reversals were incrementally increased until failure. The tests started with the application of 0.25% drift cycles. The deformation levels gradually increased in increments of 0.25% drift ratio. The walls were cycled three times at each deformation level.

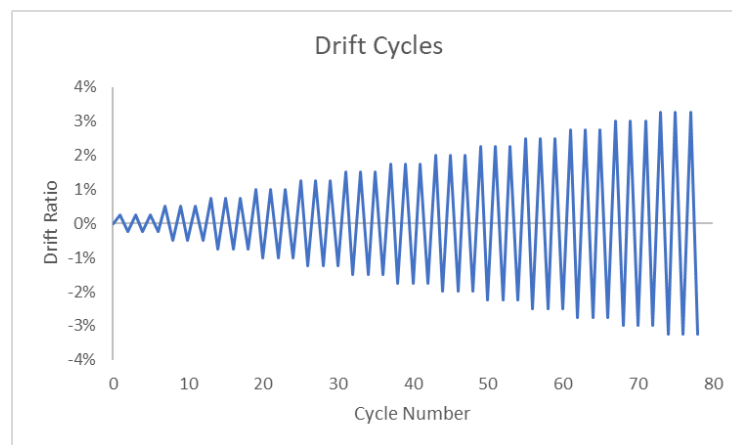


Figure 135.56 Displacement-control loading program

4 Experimental Observations

4.1 Introduction

Both retrofitted walls tested in the experimental program were subjected to reversed-cyclic loading until a drop in lateral load resistance in excess of 50% of the maximum load resistance was attained. Observations were made during the tests while the data was recorded continuously. Testing was stopped briefly at the end of each drift level for closer observations, marking of cracks, taking notes, and taking pictures. Push and pull cycles were repeated three times at each drift level as discussed in Section 3.3.7 and illustrated in Figure 3.56. Of significance were the load stages at which significant changes were observed in wall behaviour. The experimental observations and recorded data are presented in this Chapter.

4.2 Wall 1

Wall 1 was retrofitted with internally added re-bars and externally installed carbon FRP sheets. This retrofit strategy improved the seismic response of Wall 1 significantly relative to the as-built reference wall tested by Taghdi and Saatcioglu (2000). The wall failed in diagonal compression forces caused by high shear forces associated with increased flexural capacity. The presence of FRP sheets controlled diagonal tension cracking and hence diagonal tension capacity. Although the diagonal compression capacity was improved because of the added internal concrete in the toe regions, this could not be achieved in the central portion of the wall where empty masonry blocks started developing diagonal compression crushing. The shear capacity of the wall was high enough to permit the wall to attain its intended flexural force capacity although it was not high-enough to take advantage of the intended improvements in ductility capacity. The hysteretic force-deformation response recorded during the test is shown in Figure 4.1. The maximum load in the push direction was recorded to be 325 kN, which was by a factor of 5.0 higher than the unretrofitted reference wall capacity of 64.5 kN. During the first pull cycle towards 0.5% drift ratio, the wall was erroneously displaced to slightly higher than 1.0% lateral drift by the laboratory technician. Subsequent cycles were applied as per the loading regime until the 0.75% drift ratio was reached. The cycles at this deformation level resulted in a strength decay of more than 50 % with the maximum load resistance dropping down to 127 kN in the push direction. The wall experienced crushing of masonry blocks, which resulted in a tendency to twist the wall in the

out-of-plane direction. It was deemed unsafe to push the wall to 1.0% drift, though the behaviour during the pull cycles was more stable, but the strength decay during the pull exhibited similar trend. Therefore, it was decided to continue the test beyond the 50% strength loss stage by applying only the pull cycles until further reduction in load resistance was achieved. This behaviour explains the asymmetry in the hysteretic wall response shown in Figure 4.1. The stage-by-stage behaviour of the wall is discussed in subsequent sub-sections. The available strain gauge data recorded during the test are presented in Appendix I.

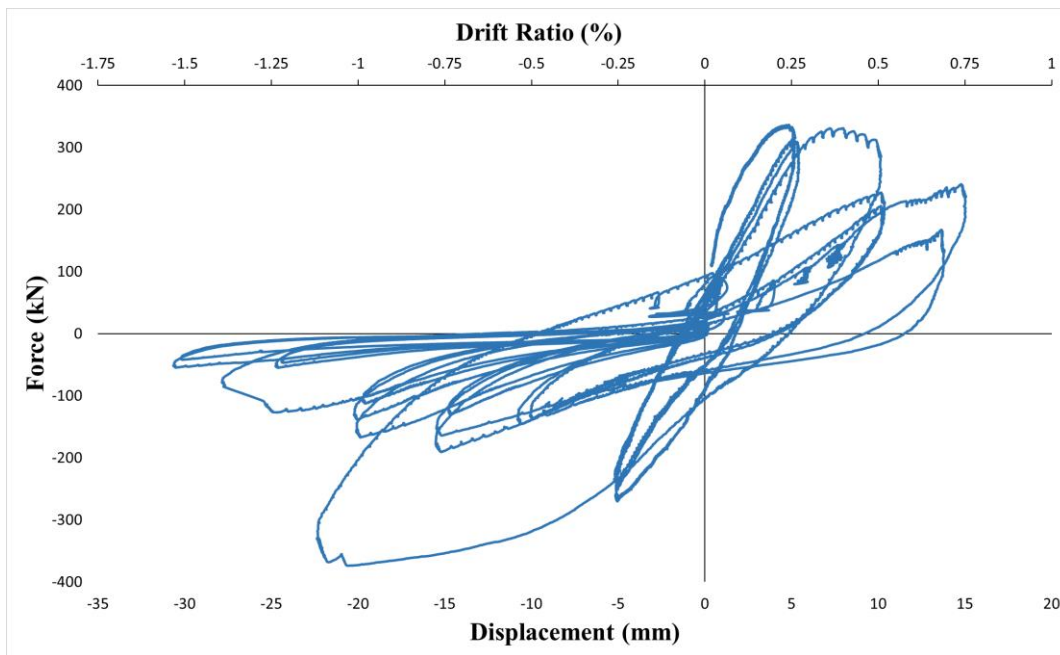


Figure 136.1 Wall 1 Force-deformation response

4.2.1 0.25% Drift Cycles

The test started with 0.25% drift cycles (equivalent to 5.17 mm horizontal displacement) in push direction, which resulted in 325 kN of load resistance. During the initial push (west to east) the cracks were directed from top west to bottom east corner of the wall as shown in Figure 4.2, where the FRP covered side of the wall was on the north side. The largest measured crack width was 0.8 mm. The test continued with the initial pull to -0.25% drift, which initiated a new diagonal crack between the top east corner and the bottom west corner. The largest new crack was 0.5 mm in width. This resulted in a load resistance of -265 kN. The first 0.25 % cycle initiated step-shaped criss-crossing of diagonal cracks along mortar joints. A crack was also observed along the wall-foundation interface at the bottom west corner.

The second push and pull at 0.25% drift resulted in 305 kN and -248 kN load resistances,

respectively. This indicates a slight loss in strength as compare to the first cycle. New horizontal and diagonal cracks were developed during the second cycle at block rows 3, 4, 5, and 7. The third cycle at 0.25% drift resulted in 299 kN and -237 kN load resistances, respectively. Some new cracks were observed and the previous cracks extended further. Figure 4.3 shows the wall at the end of the third pull at 0.25% drift level.

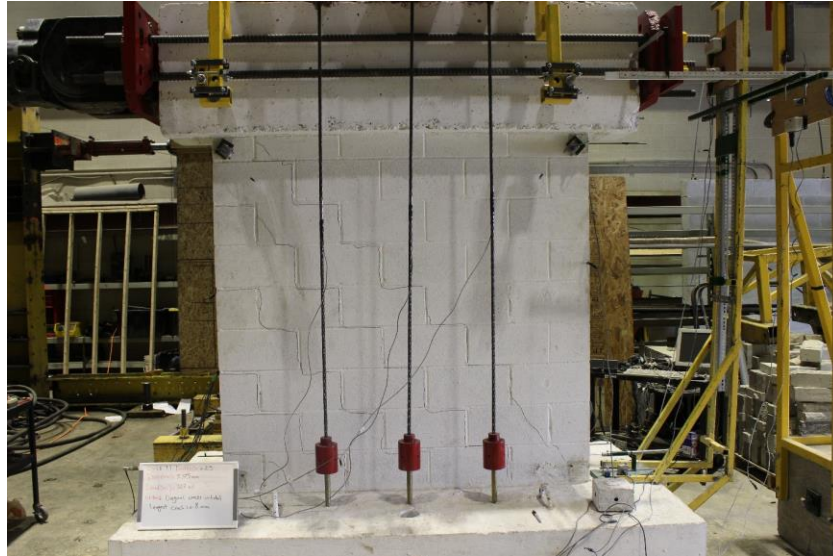


Figure 137.2 Wall 1 crack development at 0.25% drift, 1st push

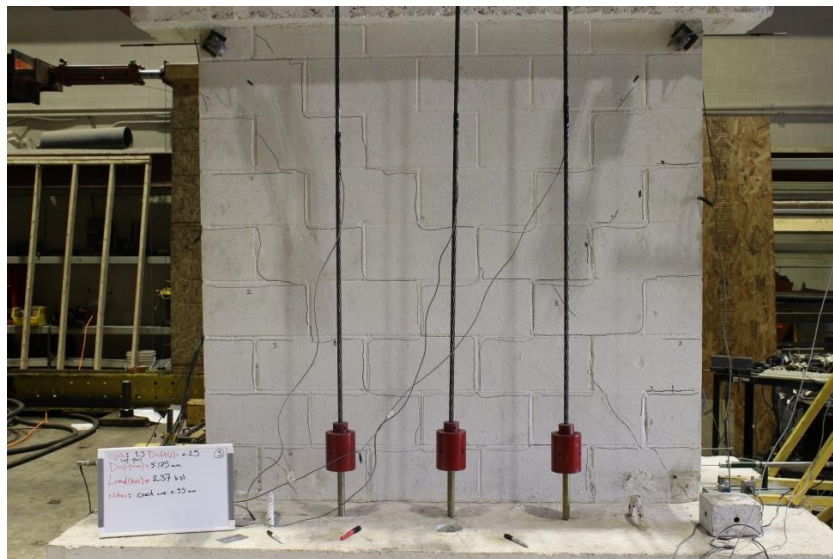


Figure 138.3 Wall 1 crack development at 0.25% drift, 3rd pull

4.2.2 0.5% Drift Cycles

During the forth cycle of loading, the wall was pushed to 0.5% drift for the first time (equivalent to 10.35 mm horizontal displacement). The load resistance was 289 kN. A large

diagonal crack formed between the top west and the bottom east corners. This is shown in Figure 4.4. The largest crack was measured to be 4 mm near the top left corner. The concrete blocks started moving slightly in the out-of-plane direction along the diagonal crack. This is illustrated in Figure 4.5. The out-of-plane movement of the blocks can be attributed to the slight twist that occurred in the wall during push, potentially because of the asymmetry created by having the FRP sheet on one side of the wall. The wall twist and the corresponding crack is illustrated in Figure 4.6. Widening of the interface crack along the wall base was observed in the west toe region, as shown in Figure 4.7. For the first pull at 0.5% drift level, the lab technician mistakenly pulled the wall to -22.0 mm instead of -10.35 mm, which corresponded to slightly higher than -1.0% drift ratio. New diagonal cracks were developed near the left (west) toe region. The out-of-plane movement of the blocks were not nearly as noticeable during the pulling stages of loading, as compare to those for pushing.

During the second push to 0.5% drift level the load resistance was reduced to 218 kN. The out-of-plane movement of blocks, crack widening, and FRP delamination were observed at this stage of loading. The maximum measured out-of-plane displacement of blocks was 5 mm. A major FRP bond failure occurred near the top west corner of the wall. The 4 mm maximum diagonal crack that had been recorded during the first push increased to 8 mm during the second push.

At the second pull load stage, the wall was displaced to -10.35 mm towards west as the original loading regime called for, as oppose to the mistakenly applied -22.0 mm first pull displacement. The concrete blocks that were displaced out-of-plane went back to their original straight position. The lateral load resistance was -135 kN. The third push and pull at 0.5% drift did not cause any major change in the damage level as externally observed. The lateral load resistance during the third push and pull cycles were 198 kN and -140 kN, respectively.

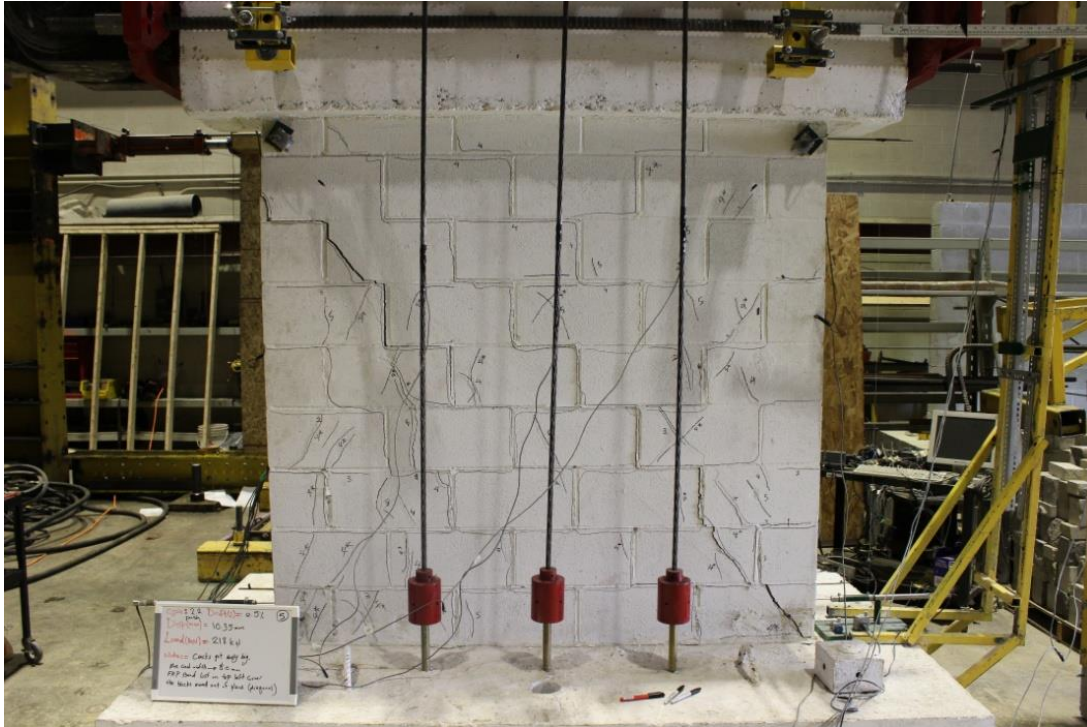


Figure 139.4 Wall 1 crack development at 0.5% drift, 2nd push



Figure 140.5 Out-of-plane movement of concrete blocks along the diagonal crack



Figure 141.6 Twist of the wall at 0.5% drift



Figure 142.7 Wall 1 uplift of wall at west toe, 0.5% drift push

4.2.3 0.75% Drift Ratio

As the seventh cycle, the wall was pushed to 0.75% drift level, which corresponded to 15.52 mm. The lateral load resistance at this level was 225 kN. The existing diagonal cracks became wider and a vertical crack started to form at the far west side of the wall as shown in Figure 4.8. The vertical crack formed along the line of the internal reinforced concrete boundary element. The wall twist continued to occur at this deformation level and the FRP bond separation region near the top west corner of the wall expanded. Upon reversing the loads towards the pull direction, the vertical crack that had initiated earlier became wider.

The wall twist and out-of-plane movement of the blocks during the second push towards 0.75% drift reached a level which was not safe to continue the test in the push direction. The lateral load resistance at this stage of loading was 160 kN. The twisting of the wall and the observed crack pattern can be seen in Figure 4.9. At this stage of loading the lateral load resistance had already dropped below the 50% level and the wall was considered to have exhausted its seismic load resistance capacity. The second pull towards -0.75% drift was accompanied by further FRP debonding as evidenced by the sound of FRP detachment. The FRP detachment was mainly observed in the lower of the wall. New vertical cracks formed near the far-east end of the wall. The load level dropped to -157 kN at this stage of loading. Beyond this stage of loading it was deemed unsafe to continue pushing the wall towards the third cycle of 0.75% drift. The wall was unloaded to zero displacement. However, it was decided to continue the test by applying the pull cycles only, which was the stable direction, although for all practical purposes the wall had exceeded its lateral deformation capacity. The wall was pulled back to -15.52 mm for the third time. The lateral load dropped to -127 kN and no change in the damage level was observed.

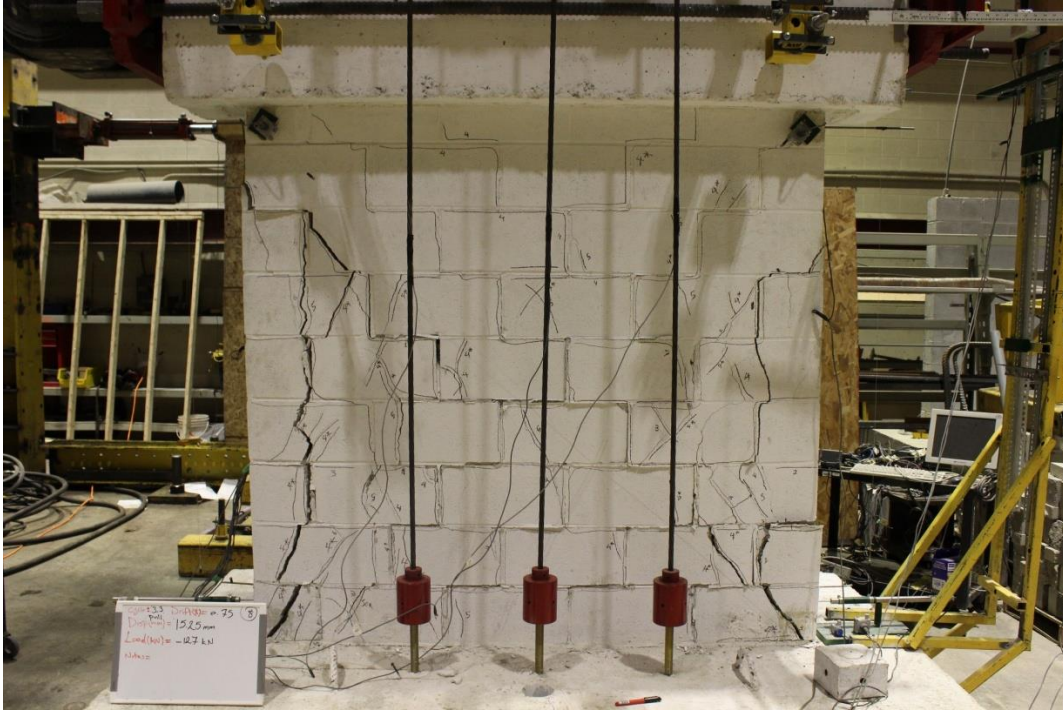


Figure 143.8 Crack development at 0.75% drift ratio

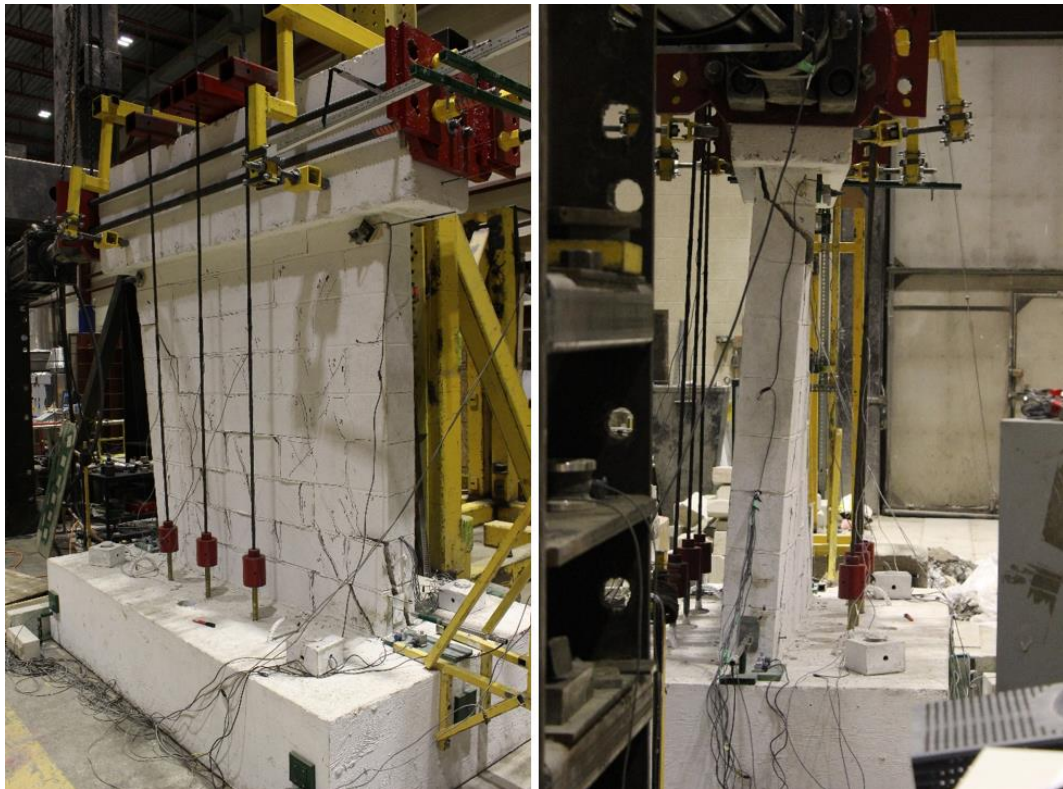


Figure 144.9 Twist of the wall at 0.75% drift

4.2.4 1.0% Drift Ratio

The tenth cycle of loading was applied by pulling the wall to -20.7 mm, which corresponded to -1.0% lateral drift. The load resistance at this stage was -164 kN. The out-of-plane movement of the blocks near the wall centre increased, and new FRP debonding was observed in the west and east toe regions. The load resistance dropped to -135 kN during the second cycle to -1.0%. Additional debonding of the FRP sheet was observed near the top west corner. The third cycle at this drift level was similar to the previous two cycles with a reduced lateral load resistance of -115 kN. Figure 4.10 illustrates the damage after the 1.0% drift cycles.

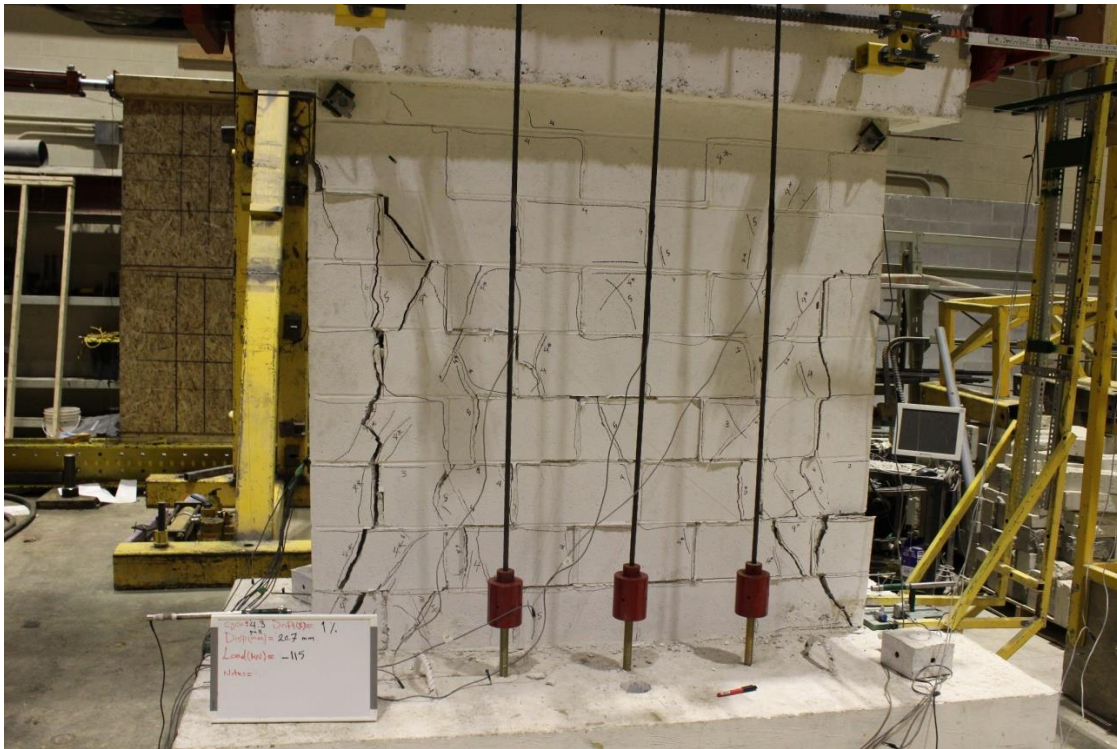


Figure 145.10 Crack development at 1.0% drift ratio

4.2.5 1.25% Drift Ratio

The test continued in the pull direction by displacing the wall to -25.87 mm, which corresponded to -1.25% drift ratio. The FRP debonding extended to the bottom centre and east toe regions. The sound of FRP detachment and rupturing was heard continuously during the application of this cycle. The load resistance dropped to -121 kN. The second and third pull cycles resulted in continuous deterioration in the lateral load resisting mechanism with loads dropping to -53 kN and -45 kN, respectively. Figures 4.11 and 4.12 depict the wall damage at the end of 1.25% drift cycles.

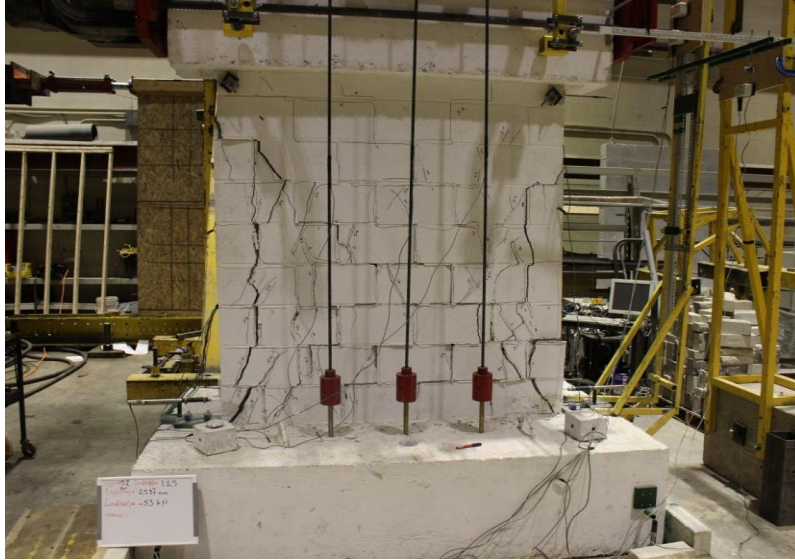


Figure 146.11 Damage of the wall at 1.25% drift ratio



Figure 147.12 Damage of the wall at 1.25% drift ratio

4.2.6 1.5% Drift Ratio

The wall was displaced to -31.02 mm, corresponding to -1.5% drift level, as the final stage of testing. The load resistance was -54 kN during the first pull. It was also pulled the second time towards the same displacement. The centre of the wall fully deteriorated at this final stage of loading, as can be seen in Figure 4.13. The actuator was pushed back to zero displacement to stop the test. At this stage the concrete blocks at the lower middle half of the wall collapsed as illustrated in Figure 4.14 shows the wall at the final stage of the experiment.

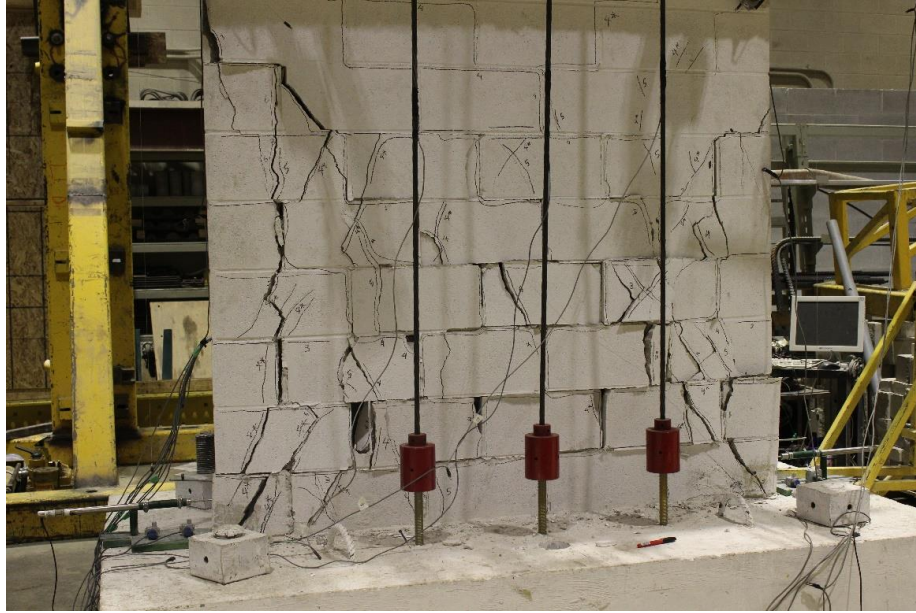


Figure 148.13 Damage of the wall at 1.5% drift ratio



Figure 149.14 Final stage of the experiment for Wall 1

4.2.7 Wall 1 Strain Gauge Data

Strain gauges were installed on steel reinforcements and tendons to study the behaviour of these elements throughout the test. Two strain gauges were installed on each of the eight 15M steel rebars provided, previously shown in Figure 3.52. The first set of strain gauges was placed at the wall-foundation interface level. The second set of gauges was placed 190 mm above the interface. Strain gauges were also installed on all the strands that were used for the application of gravity

load. These strain gauges were placed at the centre of each strand and provided information during the post-tensioning process. The post-tensioning load on the strands were checked against the strain readings taken by the strain gauges. These gauges were also used to monitor tension forces in strands during the experiment. The strain-drift graphs for Wall 1 are presented at Appendix II.

Typical strain recordings on internal re-bars are plotted in Figure 4.15. Some of the gauges on the internally placed re-bars were damaged during the construction process, and not all the gauges worked during the test. However, sufficient information was gathered to assess the performance of the bars. Accordingly, the bars exceeded their yield strains during the test, developing their yield capacities and the computed flexural capacity of the wall. The gauges at the wall-foundation interface recorded tensile strains of approximately 0.004 when 1.5% drift was applied in the pull direction, which was approximately twice the yield strain. Those that were placed 190 mm above the interface developed strains of up to 0.0025 to 0.003, indicating yield penetration up to approximately 300 mm above the interface during testing. The available strain gauge data recorded during the test are presented in Appendix I.

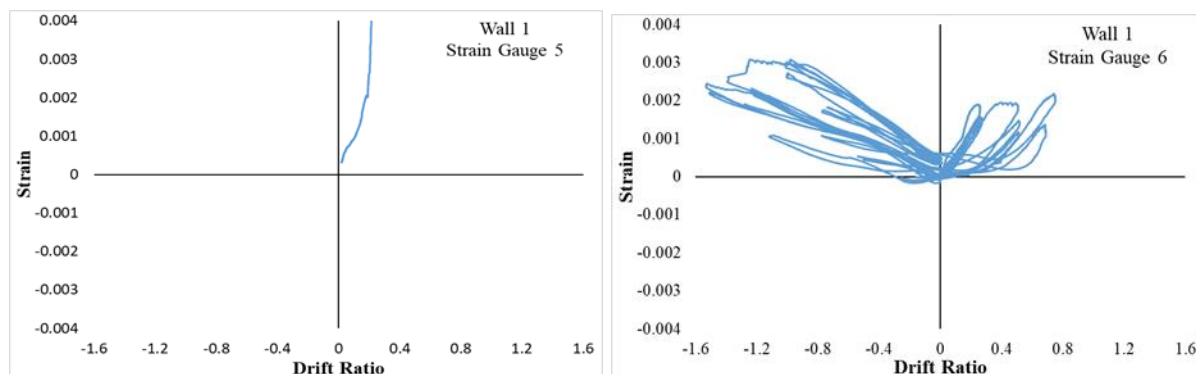


Figure 150.15 Strain-drift graphs for strain gauge 5 and 6 on rebar “c”

Six strain gauges were attached to the strands to measure tensile forces during post-tensioning and testing. Figure 3.52 shows the locations of these strain gauges with their assigned label. The strain-drift graphs of all strain gauges are depicted and presented at Appendix II. The strain of each strand was measured manually with a strain gauge box during the post-tensioning process (Yellow strain boxes in Figure 3.55(a) were used to read the strain). The strands were each post-tensioned to 16.7 kN of tension force, resulting in a total axial compressive force of 100 kN on the wall to simulate the gravity load. At the beginning of the test the strain values in the strain gauge box were initialized to monitor the change in strand strains associated with lateral deformation reversals. Figure 4.16 shows typical strain-drift relationships recorded by Strain Gauges 18 and 21 as

representative graphs. They both indicate an additional strain of up to 0.001 during lateral loading, which corresponds to a maximum force of 28 kN in each cable. As the far-end strands (near the east end) relative to the loading side increase in tension the near-end cables experience relaxation in the cables with reduced tension, and vice versa. The expectation is that, in spite of these variations in strains, the overall average gravity load was maintained during the test. The positive strain values indicate higher tension in the cables relative to the initial prestress level and the negative strain values indicate lower tension compare to the initial tension values. All six strain gauges placed on the tendons are showing positive strain values during the first push and pull cycles, but these values decreased as the wall started deteriorating at higher horizontal drift levels. Moreover, the tendons on the south side of the wall showed more tension loss at higher drifts in pull, which indicated out-of-plane rotation of the wall towards south. This is evident in the observed negative values of strains at the final pull cycles recorded by Stain Gauges 17, 18, and 19 on the south side of the wall. This phenomenon may be explained by the presence of FRP sheets on the north side of the wall, which limits the deterioration on the north face.

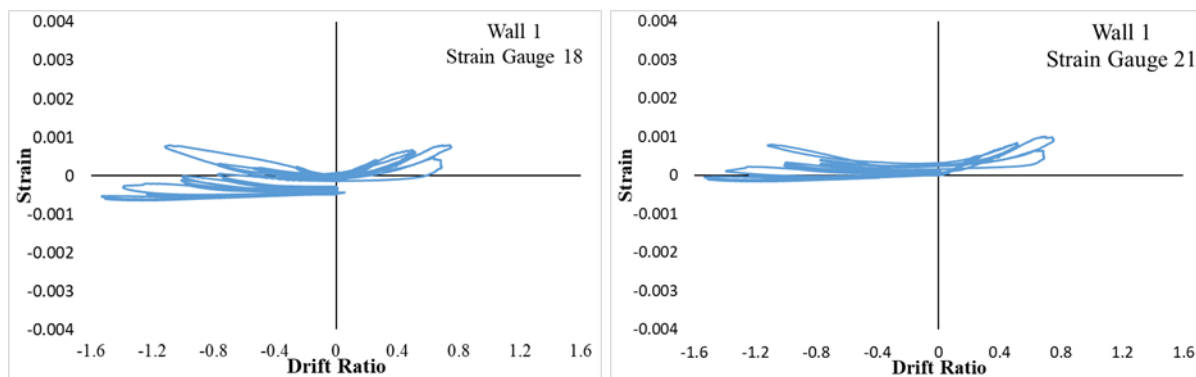


Figure 151.16 Strain-drift graphs for strain gauge 18 and 21

4.3 Wall 2

Wall 2 was retrofitted using prestressing strands as flexural reinforcement, which also clamped potential cracks and introduced a self-centering capability. This retrofit technique improved the seismic behaviour significantly. Figure 4.17 shows Wall 2 prior to testing. The lateral force-lateral drift hysteretic relationship recorded during the test is shown in Figure 4.18. The wall exhibited primarily flexural response and developed 6 times the lateral load capacity obtained from the unretrfoitted URM companion wall. In addition, it failed gradually as the concrete in the toe region crushed and the HSS yielding agents developed yielding, also exhibiting ductility. The wall

sustained many push and pull cycles that, starting with $\pm 0.25\%$ drift ratio and continued to $\pm 3.5\%$ drift ratio at which time the strength decay was as high as 70%. Of significance was the self centering capacity of the wall, because of the elastic restoring force capabilities of the prestressing strands. This can be seen in the hysteretic relationship by the unloading branches of hysteresis loops that tend to pass through the origin. The stage-by-stage response of the wall behaviour is provided in the following subsections.



Figure 152.17 Retrofitted masonry wall 2 test setup

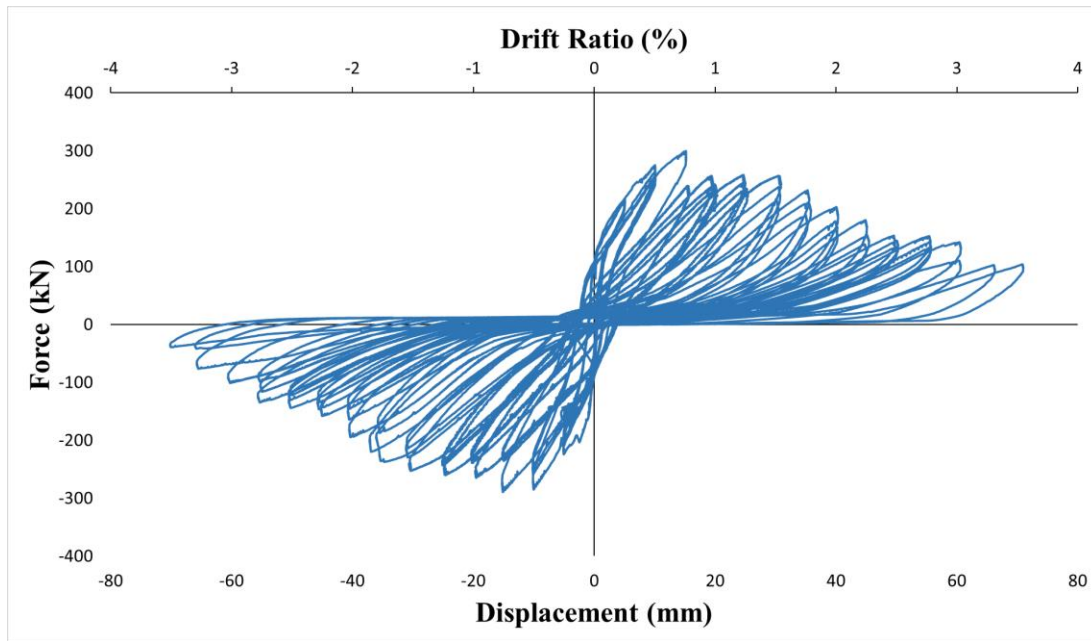


Figure 153.18 Wall 2 Force deformation response

4.3.1 0.25% Drift Ratio

The test started with pushing the wall towards east up to 0.25% lateral drift ratio, which was equivalent to 5.2 mm horizontal displacement. At this stage of loading the actuator force reached 215 kN. A horizontal crack was observed to have formed at the wall-foundation interface, creating a 4.0 mm crack opening at the west end heel region. This can be seen in Figure 4.19. The test continued by unloading the wall back to zero and continuing to reverse the deformation in the opposite direction to -0.25% drift ratio, corresponding to -5.2 mm displacement at -210 kN of load resistance. The pull at -0.25% drift initiated some crushing cracks in the west toe region of the wall. The interface crack propagated towards the centre of the wall. The wall was cycled for the second time between 5.17 mm and -5.17 mm deflections with corresponding force resistances of 211 kN and -200 kN, respectively.

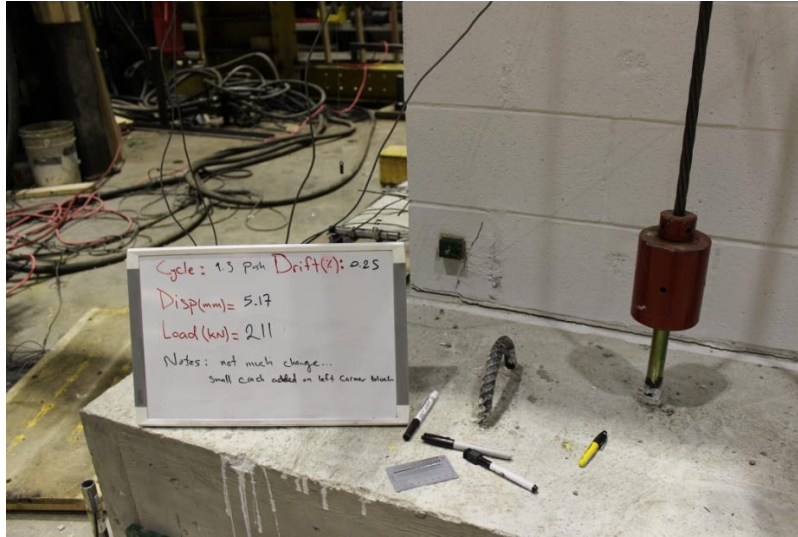


Figure 154.19 Uplifting of west side of the wall at 0.25% drift push

4.3.2 0.5% Drift Ratio

The next stage of loading was 0.5% drift ratio and corresponding horizontal displacement of 10.4 mm. The first push towards east resulted in a force resistance of 269 kN. The interface crack at this stage was measured to be 9 mm at the west end of the wall (Figure 4.20), which was accompanied by smaller cracks in the same region (Figure 4.21(a)). The wall was pulled back to the same drift level in the opposite direction, which resulted in -261 kN of load resistance. The previously formed cracks became wider and the maximum crack width at the interface was increased to 5 mm.

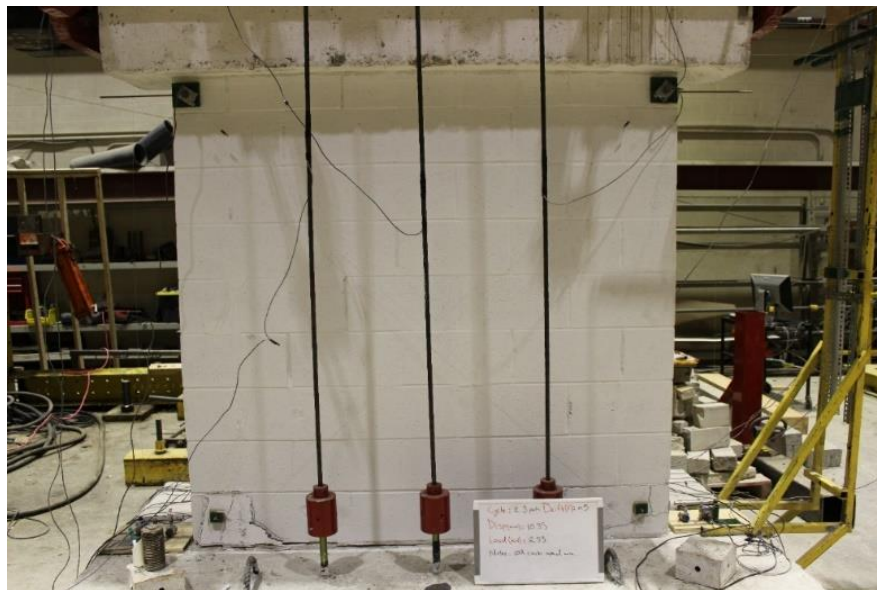


Figure 155.20 Wall 2 at 0.5% drift push

During the second cycle of 0.5% drift, the wall developed 260 kN and -246 kN force resistances when pushed and pulled, respectively. Some new cracks were observed and FRP debonding started taking place along the second and third rows of blocks on the east side. The cracks on the east side are shown in Figure 4.21(b). No significant change was observed during the third cycle of 0.5% drift level. The load resistance was 253 kN in the push direction and -238 kN when it was reversed.

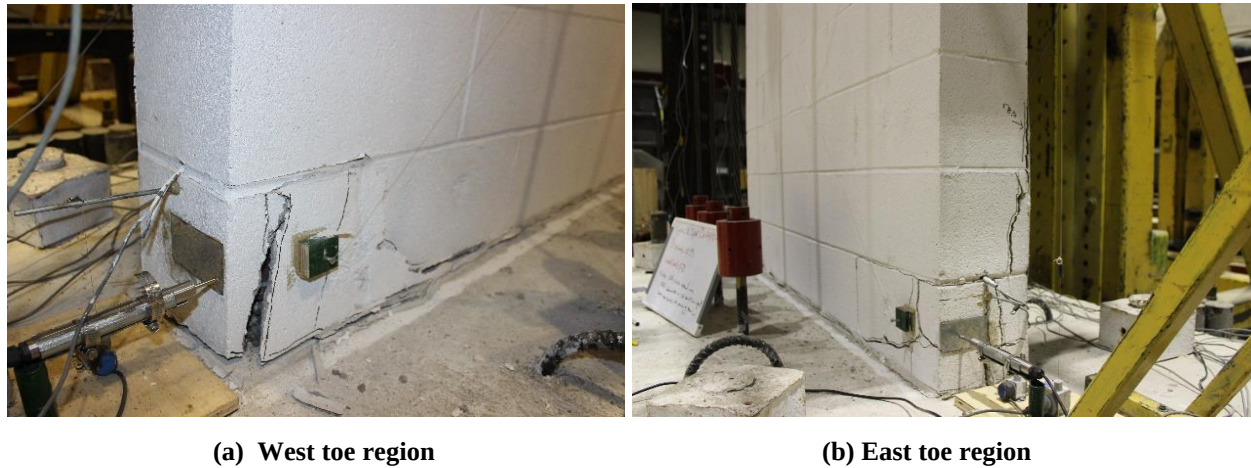


Figure 156.21 Toe regions at west and east side of the wall, 0.5% drift

4.3.3 0.75% Drift Ratio

The next load stage was three cycles at 0.75% drift ratio. The lateral load resistance was measured to be 295 kN at the first push cycle when the lateral displacement was 15.5 mm. This resulted in the further widening of the existing cracks on the west side. New cracks formed along the first and second rows of blocks. During the first pull towards west the wall started rotating clockwise about its east toe. This resulted in an out-of-plane displacement of 40 mm towards east. The first pull to -0.75% resulted in additional local FRP debonding and interface crack propagation on the east side.

During the second push to 0.75% drift, the actuator suddenly shut off due to an unknown reason and stopped working right after 15 mm of displacement. The displacement of the wall returned back to 2.5 mm by itself. This self-centering behaviour was attributed to the prestressing force in the internal strands. The wall was pushed towards east, but the actuator shut off again at the same displacement level. After a few trials, it was discovered that an actuator cable was caught under the foundation causing signal problems. The problem was fixed and the test continued. Figure 4.22 shows the incomplete cycles recorded. It is believed that these small displacement

cycles prior to significant damage to the wall did not have an effect on the test. The cycles in Figure 151 were not included in the hysteretic relationship shown in Figure 4.18. Indeed, the trend in response continued to be the same before and after the interruption.

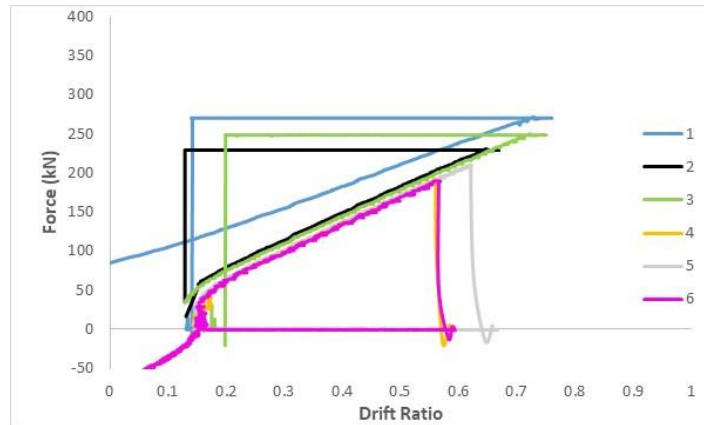


Figure 157.22 Force-drift response of the wall for six failing cycles

The load resistance was -226 kN during the second pull at -0.75% drift ratio. Additional local FRP debonding and toe concrete crushing was observed. The third cycle did not produce any noticeable change in wall behaviour with 235 kN and -223 kN load resistances in push and pull directions, respectively. Figure 4.23 shows the wall after the applications of 0.75 % drift cycles. Figure 4.24 illustrates wall damage in the toe regions.

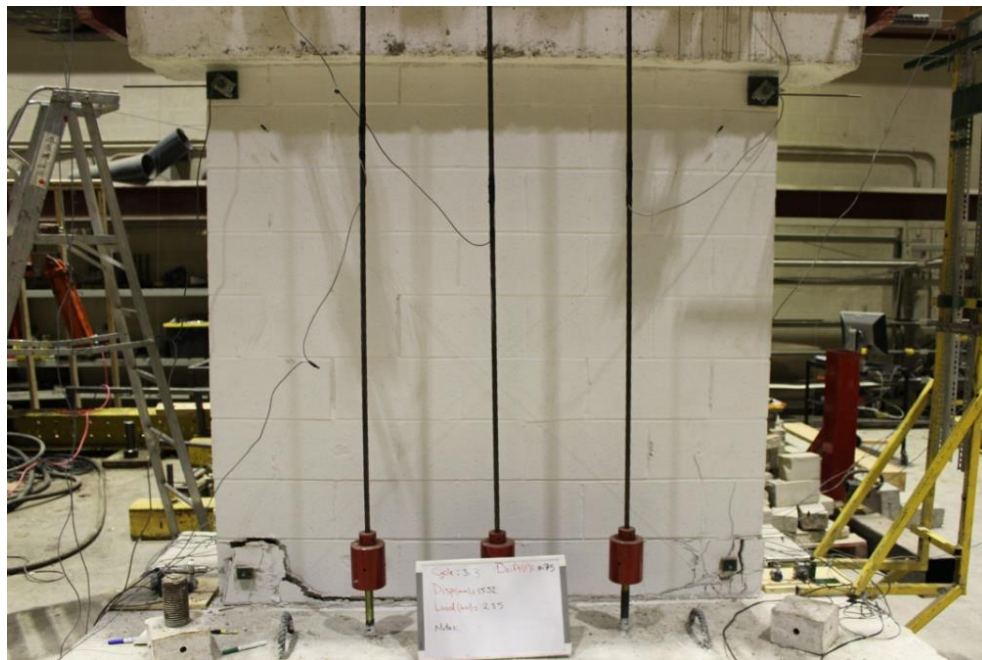


Figure 158.23 Wall 2 at 0.75% drift



(a) West toe region

(b) East toe region

Figure 159.24 Wall 2 concrete damage at the end of 0.75% drift cycles

4.3.4 1.0% Drift Ratio

The wall was displaced to 20.7 mm for 1.0% drift cycles. The load resistance was 249 kN in the east direction. This resulted in the propagation of the base interface crack, accompanied by increased out-of-plane rotation in the toe region. FRP debonding continued with some FRP rupturing. The subsequent two cycles did not cause further damage. Figures 4.25 and 4.26 show the damage observed at the end of 1.0 % drift cycles.

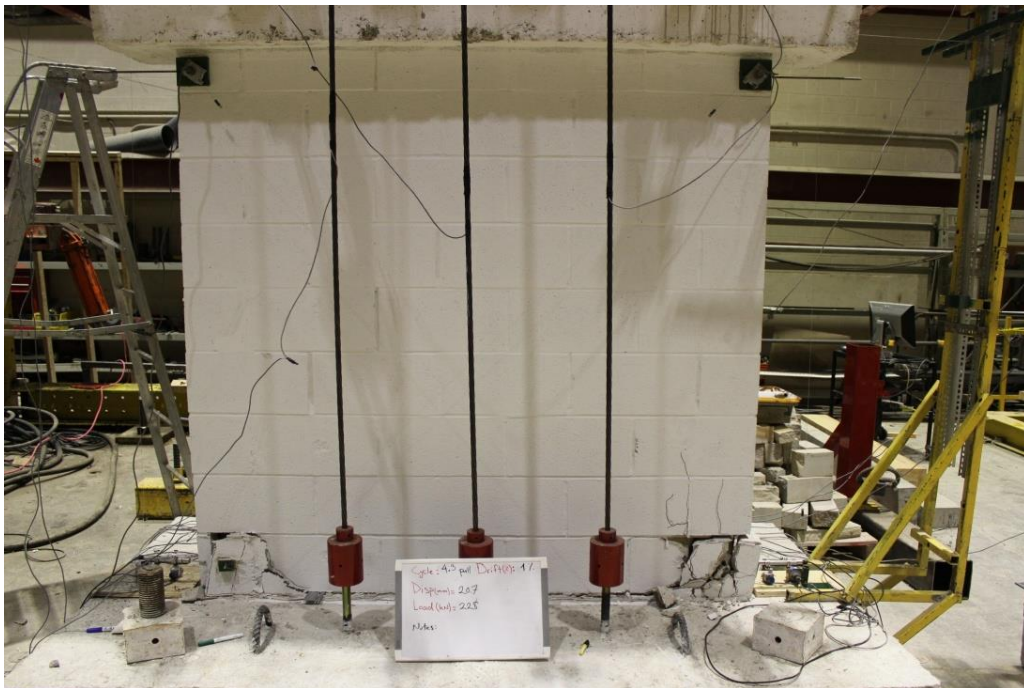


Figure 160.25 Wall 2 damage at 1.0% drift



(a) Rotation of east end towards north

(b) debonding of FRP

Figure 161.26 Wall 2 at the end of 1.0% cycles

4.3.5 1.0% to 3.25% Drift Ratio

The test continued by applying three cycles at 1.25% drift level. This drift level was equivalent to 25.9 mm horizontal displacement. The wall resistances at this stage were 255 kN and -256 kN, respectively. The previous cracks became wider with some additional FRP debonding. Figures 4.27 and 4.28 shows the damage observed at this stage of loading.

Further increase in the deformation level to 1.5% drift led to more deterioration of the wall. The actuator force indicated maximum resistances of 253 kN and -239 kN during push and pull, respectively. A masonry piece was detached and dropped from the second row near the east toe region, as illustrated in Figure 4.29, partially exposing the prestressing anchor and the post-tensioning strand.



Figure 162.27 Wall 2 at -1.25% drift push



Figure 163.28 Damage progress of wall 2 at toe regions, 1.25% drift



Figure 164.29 Wall 2 at 1.5% drift push

The test continued by applying 1.75% drift cycles. At this stage of loading the wall developed additional cracks near the second row of masonry blocks in the east toe region. Some new cracks were also noticed in the central base region. These new cracks can be seen in Figure 4.30. The maximum actuator load was 230 kN during the first push, which showed a significant reduction in load resistance relative to the first push at 1.5% drift ratio. The wall experienced additional FRP debonding, especially in the east toe region. The detachment of FRP is shown in Figure 4.31(a). An important phenomenon that initiated during earlier stages of loading was the yielding of the HSS yielding agents, developing permanent deformations. This is illustrated in Figure 4.31(b). The load on the west HSS was released when the wall was pulled to 1.75% drift towards west. The same happened when the wall was pushed towards east. The nonlinear deformation in HSS yielding agents indicated energy dissipation of the retrofit system, while also preventing tendon rupturing and increased concrete crushing due to flexural compression.

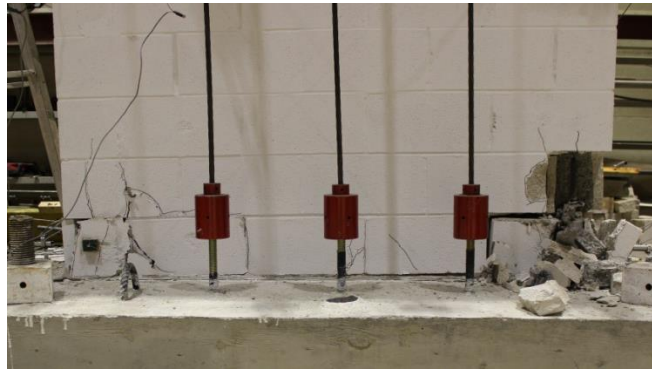


Figure 165.30 Wall 2 at -1.75% drift pull



Figure 166.31 (a) FRP delamination and rupture at 1.75 % drift push, (b) HSS yielding agent at 1.75% drift pull (right)

The deformation reversals were subsequently increased to 2.0% lateral drift with 200 kN and -192 kN force resistances in the east and west directions of loading, respectively. This indicated further strength decay relative to the previous cycles. The damage at this load stage is illustrated in Figures 4.32 and 4.33. Figure 4.34 shows the east end HSS yielding agent at the end of the first push cycle at 2% drift. At this stage the west HSS was engaged in tension while the east HSS was released due to reduced prestressing force.



Figure 167.32 Wall 2 at -2.0% drift pull



Figure 168.33 Toe crushing at west and east side, 2.0% drift ratio



Figure 169.34 HSS yielding agents at -2.0% drift push

The wall damage increased during 2.25% drift cycles. Some of the concrete block started spalling and separating from the wall. FRP debonding continued to expand, especially near the east and west ends. The load resistances were 177kN and -157 kN, respectively, showing further

decay. Figures 4.35 and 4.36 show the observed damage at this stage of loading. Further cycling at 2.5% and 2.75% lateral drift ratios caused significant strength loss with load resistances dropping to 150 kN/-141 kN and 151 kN/-132 kN, respectively. The damage state during these load stages are illustrated in Figures 4.37 and 4.38. The HSS yielding agents maintained their functions throughout these cycles, with permanent deformations at 2.75% drift (Figure 4.39) becoming higher than those at 2.0% drift (Figure 4.34).

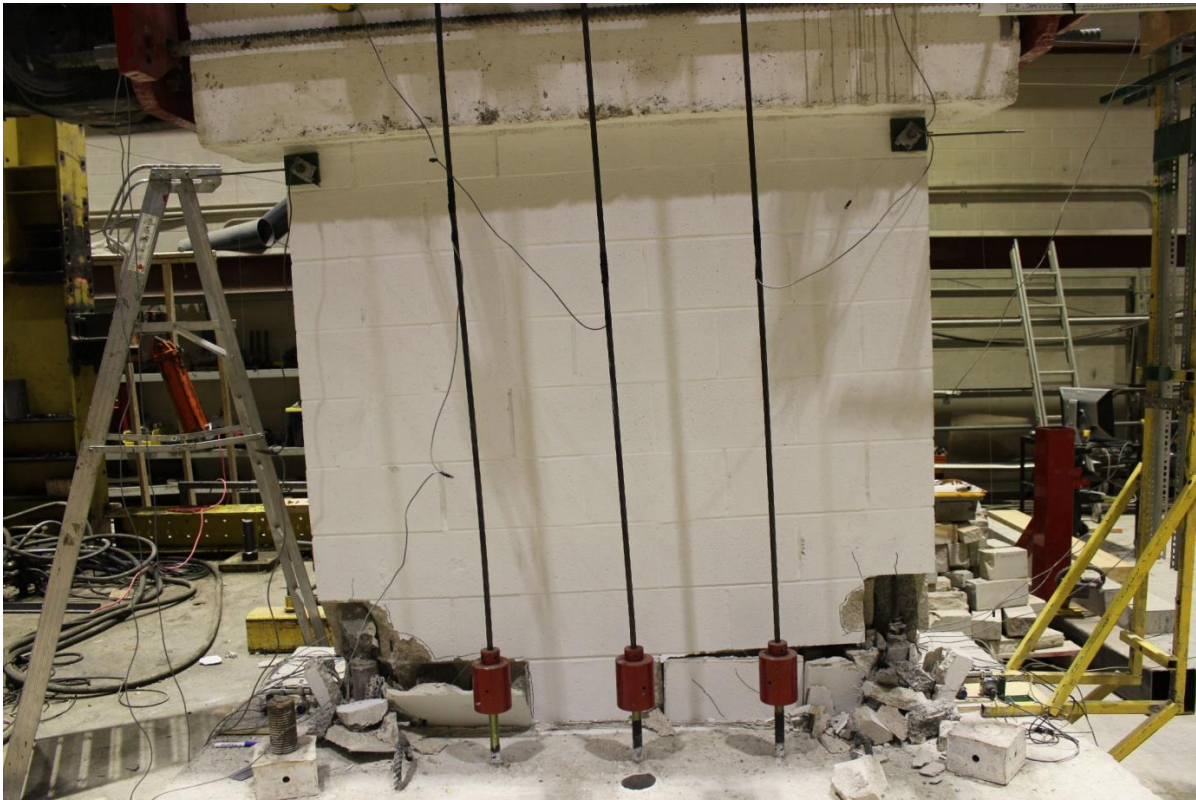


Figure 170.35 Wall 2 at 2.25% drift ratio



Figure 171.36 West side toe region of the wall at 2.25% drift



Figure 172.37 Wall 2 at -2.5% drift

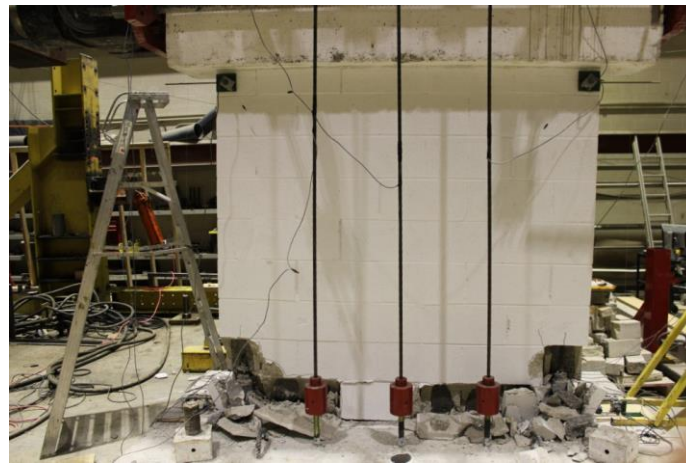


Figure 173.38 Wall 2 at 2.75% drift



Figure 174.39 HSS yielding agents at 2.75% drift pull

The wall went through two more cycles at 3.0% drift and an additional cycle at 3.25% drift ratio. The load resistance during the first push and pull cycle at 3.0% drift ratio was 140 kN and -100 kN, respectively. The wall could undergo more cycles, but it was decided to stop the

experiment since the capacity drop was in excess of 60%. Figure 4.40 shows the wall condition at 3.0% drift ratio.



Figure 175.40 Wall 2 at 3.0% drift

At the end of the test, the test setup was disassembled and the masonry wall was removed from its foundation after the strands were cut by a grinder. The strands were in perfect condition with no sign of yielding. The deformed HSS yielding agents are shown in Figure 4.41(a), clearly demonstrating the permanent deformations they experienced, thereby controlling further increase in prestressing strands at higher deformation levels. Figure 4.41(b) shows the detached masonry wall before it was discarded. This figure also shows that the first row of blocks at locations of vertical reinforcement (the three 10M re-bars placed to further control diagonal cracking) remained attached to the wall.

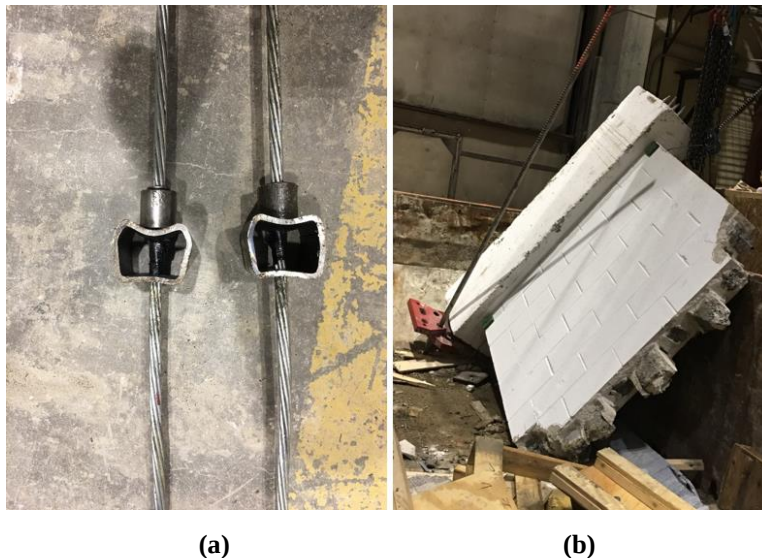


Figure 176.41 (a) HSS yielding agents and post-tensioning cables , (b) Wall 2 after being separated from the foundation

4.3.6 Strain Gauge Data

The strain gauge locations were discussed earlier in Section 3.3.6 under “instrumentation.” Figures 3.53 and 3.54 show the locations of strain gauges, with their assigned labels, for Wall 2. Strain Gauges 1 and 2 were installed on the internal post-tensioning tendons, but were damaged during the test. Strain Gauge 3, 4, 5, and 6 were installed on the threaded rods that were used for anchoring the post-tensioning tendons to the footing. Strain Gauges 7 to 12 were installed on tendons that were used to apply gravity loads, and Strain Gauges 13 to 21 were installed on the internally placed 10M shear reinforcements. The strain-drift graphs for the undamaged strain gauges are plotted and presented in Appendix I. General interpretations about these graphs are presented in the following paragraphs.

The strain gauges that were attached to the threaded rods were located in the toe-crushing region. Therefore, they were damaged when the concrete around them deteriorated. Strain Gauge 3 continued recording data up to 2.0% drift ratio cycles. Figure 4.42 shows the strain-drift graph of Strain Gauge 3. According to the figure, we can see that the dominant strain on the east treaded rod was negative when the wall was pushed due to the reduction in the initially imposed prestressing force. The reduction in the initial prestressing tension became more significant as the concrete in the region has crushed in flexural compression. The opposite occurred when the wall was pulled towards west and the prestressing force increased in the strand. The maximum increase recorded was equal to approximately 84 kN, corresponding to 0.3% strain increase, bringing the total tension force in the strand to about 160 kN (corresponding to 1140 MPa stress). The sudden changes shown in the strain-drift relationship of Figure 4.42 explain complete release of prestressing force when the strand is in compression under increasing lateral drift.

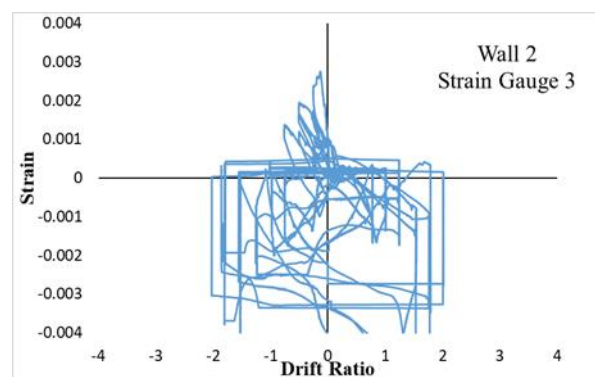


Figure 177.42 Strain gauge 3 on east side threaded rod

The strain gauges on the strands used for the application of gravity loads had similar behaviour. Figure 4.43 shows the strain-drift relationship recorded on south-east side of the wall. The positive strain values indicate increase in strand forces in tension when the wall was pulled and the negative strain values indicate reduction in initial prestressing force, dropping down to zero at higher drift values in the push direction. The reduction in prestressing force at different stages of loading can be explained by high lateral drift and associated bending of the wall, as well as the gradual damage observed in concrete, resulting in the downward settling of the wall and a partial release of the prestressing force.

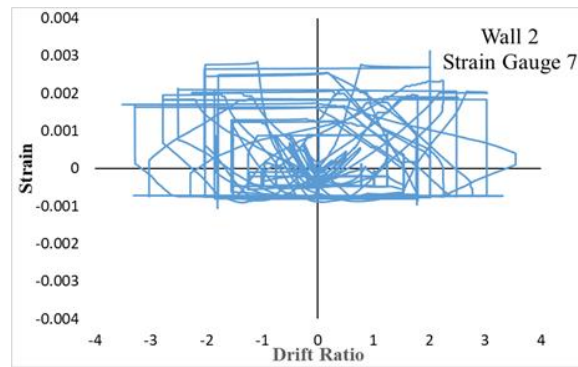


Figure 178.43 Strain-drift response of stain gauge 7 for gravity load tendon

The strain gauges that were attached to 10M rebars at the centre of the wall did not record any significant strain. This indicates that these reinforcing bars did not play an appreciable role in controlling web cracking. The externally placed FRP was sufficient to prevent diagonal tension failure when the internally placed concrete increased the diagonal compression capacity, eliminating shear failure, unlike the observations made in Wall 1 test, which provided the rationale for the placement of these three 10M bars. The strain-drift relationship in the central 10M rebar (Strain Gauge 18) is displayed in Figure 4.44. The other strain gauges on these rebars also exhibited similar behaviour showing virtually zero strains.

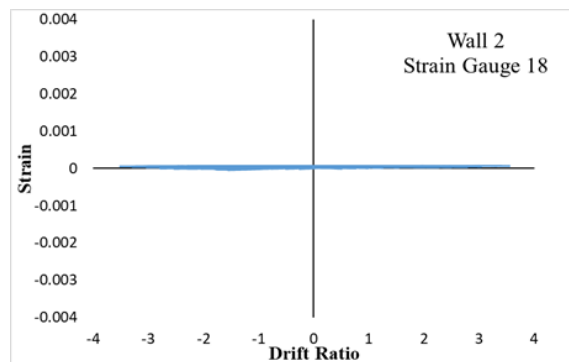


Figure 179.44 Strain-drift response of the strain gauge 18 for 10M steel reinforcement

5 Analytical Investigation

5.1 Introduction

The analytical research had two primary objectives; i) construction and validation of analytical models for the walls tested in the experimental phase of research, and ii) expanding the results obtained in the experimental phase by varying design parameters analytically. The first task involved the review of analysis techniques that are available in the literature and select the most suitable approach and the analytical tool. The second task was to generate analytical models that can be validated against test data. This was done separately for the two walls tested. Upon validation, the third task was to conduct an analytical parametric investigation by varying the design parameters analytically. The primary parameters of the proposed seismic retrofit techniques and their significance were investigated analytically for each wall to complement the experimental findings. This would also provide a better understanding of the effects of the parameters involved in the retrofit techniques. The analysis results would provide internal stresses in materials, highlighting the significance of the force component involved in the lateral force resistance mechanisms.

5.2 Analysis Options Available in the Literature

A literature review on previous analysis approaches was conducted and presented in Chapter 2. These approaches include non-linear finite element analysis, truss analysis, equivalent frame analysis, and flexural/rocking analysis. It was concluded that finite element models could produce reliable and accurate results for critical points on the force-deformation relationship, as well as the failure mechanisms observed in reinforced and unreinforced masonry walls. Alternative approaches, involving truss and equivalent frame models can be implemented with much less effort and can predict observed response with reasonable accuracy. However, the models used in these approaches are not as reliable and as expandable as the finite element models. It is not always possible to reproduce wall response with equivalent truss members when composite elements are involved as in the case of reinforced masonry and/or URM in combination with externally boded FRP or internally added reinforcement.

The analytical options indicated above can be implemented by using the general-purpose analysis software SAP2000. This software has become one of the industry-standard software. It is

used frequently for structural analysis. Upon considering the available approaches and analytical tools, it was decided to employ the finite element method of analysis with SAP2000 as the analysis tool.

5.3 SAP2000

SAP2000 is a fully integrated computer system for modeling, analysis, and design of structures. It was developed by Computers and Structures, Inc. (Computers and Structures 2016). At the heart of the software package is a powerful engine called SAPfire. The software is object-based, meaning that the user draws a structural object through graphical interface and then assigns the properties and loads to that object. There are four types of objects available in SAP2000, consisting of point objects, line objects, area objects and solid objects. Point objects consist of joint objects, which are automatically generated at the ends of each object, and grounded link/support objects that are useful for modelling special support conditions. There are four types of line objects including frame objects (used to model beams, columns, braces, and trusses), cable objects (used to model slender cables), tendon objects (used to model tendons within other objects) and connecting two-joint link/support objects (used to model isolators, dampers, and multi-linear springs). Area objects or shell elements include plate, membrane, and full-shell which are used to model walls, floors, and other thin-walled members. Solid objects are used to model three-dimensional solids.

The materials are used to define mechanical, thermal, and density properties for the objects. The material of an object can be defined as isotropic, orthotropic, or anisotropic, based on the element type. The axial stress-strain curve for each material is implementable to any material axis. These curves represent the behaviour in each of the three material axes for isotropic, orthotropic, and anisotropic materials. Different hysteretic models are available to define nonlinear behaviour and energy dissipation of materials under reversed cyclic loading. Some of the available hysteretic models are elastic, kinematic, degrading, Takeda, pivot, and concrete.

Nonlinear properties can be assigned to points along the clear length of the element in the form of user-defined hinges. Uncoupled hinges are available for moment, torsion, axial force, and shear. Coupled hinges are also definable, which yield according to the interaction between axial force and bi-axial bending moment at the hinge location. Plastic deformations along plastic hinges can be modeled by assigning length to discrete point hinges.

Shell elements are area objects with translational and rotational degrees of freedom that can support forces and moments. Shells are used to model membrane, and plate elements like floor systems, wall systems, bridge decks, and three-dimensional curved shells. Two available formulations for shells are homogenous and layered shells. Layered shells are suitable when the material nonlinearity has to be considered in the model. One of the most common applications for layered shell element is nonlinear shear wall modeling. A concrete shear wall with two vertical and two horizontal layers of rebar would be modeled by one layer of concrete shell and four layers of rebar shells embedded inside the concrete layer. Solid element is also available for modeling three-dimensional structural element. Solid element is a three-dimensional object with six quadrilateral faces and eight nodes located at the corners.

5.4 Masonry Wall Modelling with Shell Elements in SAP2000

Considering all the options available in SAP2000, it was decided to model the retrofitted masonry walls using shell elements. Shell objects are useful modeling tools that are often used to model reinforced concrete walls and floors. Reinforced concrete walls and floors usually consist of one type of concrete and steel. However, retrofitted masonry walls of the current investigation (Wall 1 and Wall 2) consist of wide range of materials and elements that should be represented in the structural model. The following sections explain the steps implemented in generating structural models for retrofitted masonry walls using SAP2000.

5.4.1 Meshing

A mesh was created according to the dimensions of the retrofitted masonry walls (Wall 1 and Wall2). The main idea was to divide the masonry walls into block areas. Figure 5.1 shows the initial meshing which formed the skeleton of the structural models for Walls 1 and 2.

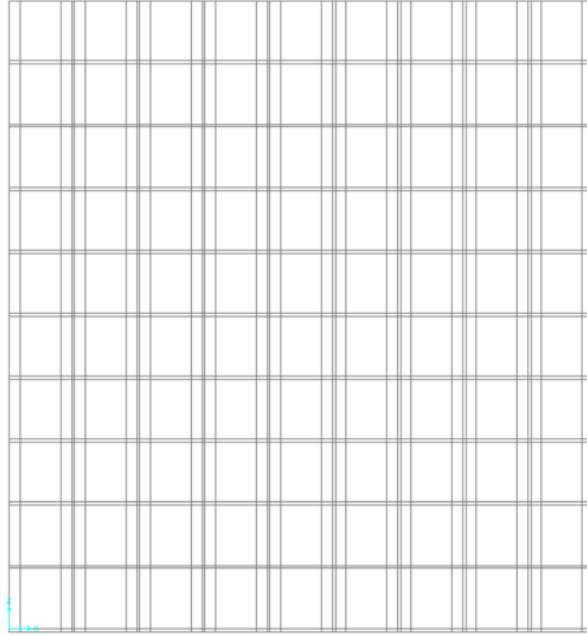


Figure 180.1 Initial meshing for wall 1 and wall 2

5.4.2 Boundary Conditions

Boundary conditions are identical in Wall 1 and Wall 2. At the base location all the nodes are restrained to move but free to rotate in local axes. All the nodes on top of the wall are restrained to move out-of-plane, but free to move and rotate in-plane.

5.4.3 Shells

The defined areas in meshing stage are filled with layered shells that consist of different material layers. The materials vary through the thickness of the wall. These composite volumes are different for every cell of the mesh which are modeled as nonlinear layered shell objects. Figure 5.2 shows 3D view of the model for Wall 1. The analytical model for Wall 1 was constructed from six different types of shells. Figure 5.3 and 5.4 show the front view and a close-up view of the upper left corner of the analytical model with colour coded shells. Each shell is modelled to represent a specific part of the wall.

Red shells represent boundary elements of Wall 1, filled with reinforced concrete. They consist of seven layers with four different materials. Figure 5.5 shows a screen-capture view of the software illustrating each layer with the assigned material type. The layers for the boundary elements include concrete, reinforcing steel, masonry and FRP materials. Each layer is assigned a name that represents the type of material used. These names consist of concrete, steelt, steelb,

masonryr, masonryb, FRPHT, and FRPVT. The properties of each material type are described in the subsequent subsections. They were specified based on the actual material stress-strain relationships obtained for the material used in the experimental program reported in Chapter 3.

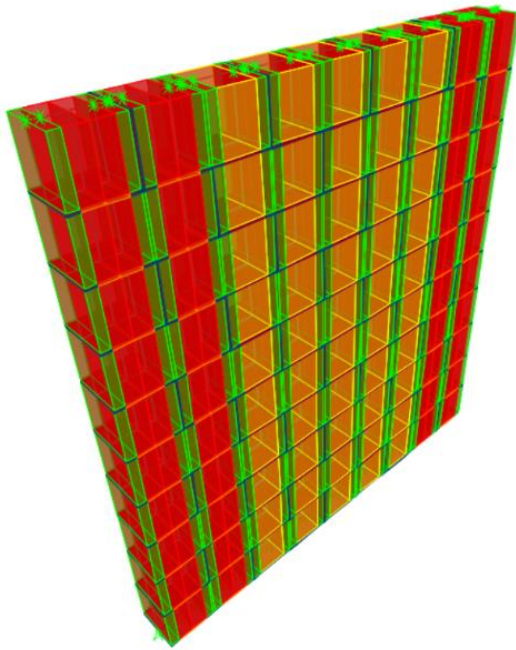


Figure 181.2 Three-dimensional view of the analytical model for Wall 1

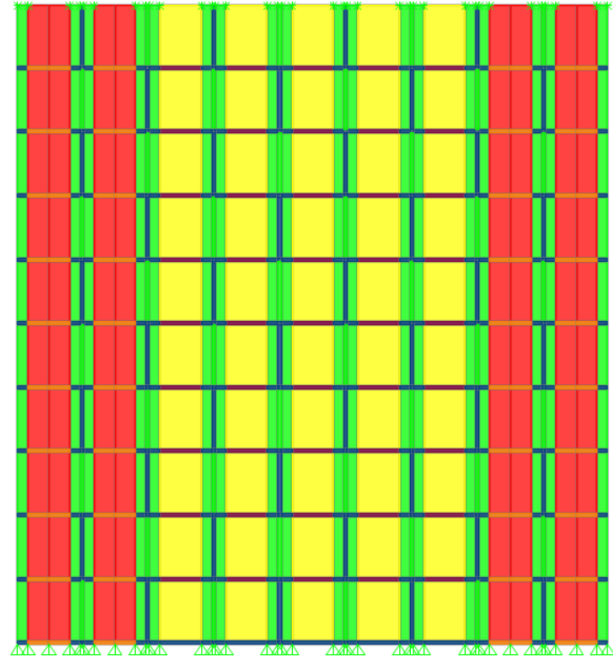


Figure 182.3 Front view of the analytical model for Wall 1

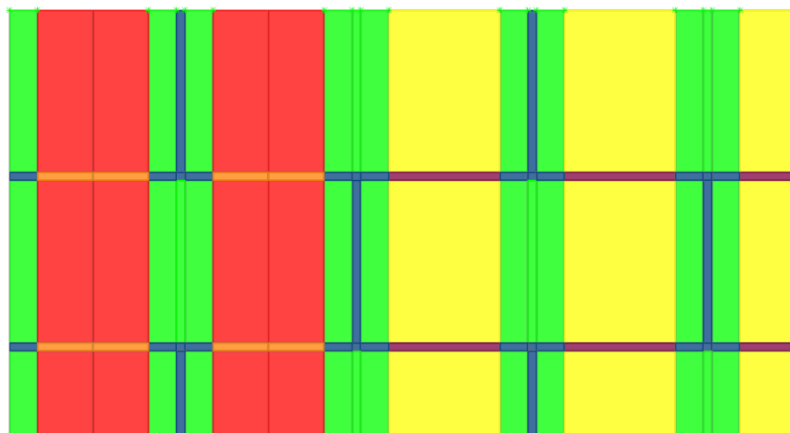


Figure 183.4 Close view to the top left corner of analytical model for Wall 1

The location of each layer relative to the wall centreline and the thickness of each layer are specified as per the wall geometry. Two layers of FRP with 1 mm thickness were considered on one side of the wall for all the shell sections. The horizontal FRP layer was named FRPHT with a material angle of zero degree, and the vertical FRP layer was labelled FRPVT with a material

angle of ninety degrees. Two layers of reinforcing steel with a 1.53 mm thickness were specified to represent two 15M rebars that were anchored inside the footing at each cell. In addition, two masonry layers are specified, representing the masonry block face shells, one on either side of the wall.

A total of six different color-coded shells are used to model Wall 1, with red, orange, green, blue, yellow, and purple shells. The red shell and its layers are described above. The explanation of other color-coded shells are provided in Appendix II.

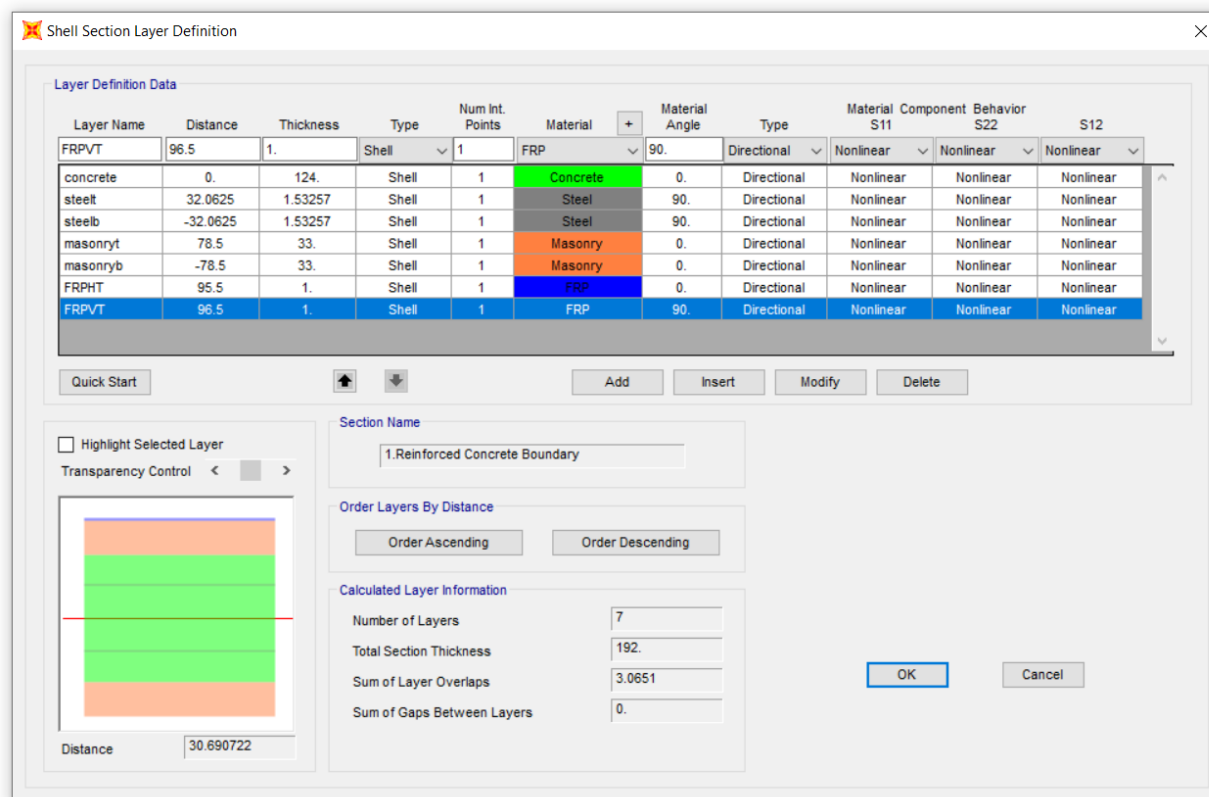


Figure 184.5 Red shell section for Wall 1

The analytical model for Wall 2 was constructed using eight different types of color-coded shells, consisting of red, orange, light green, light blue, green, blue, purple, and yellow. Figure 5.6 shows the front view of the model. A close-up view of the upper left corner is shown in Figure 5.7.

Red shell sections represent the boundary elements, as in the case of Wall 1 model. They are filled with concrete and prestressing steel consisting of 7-wire strands of size #15. A screen-capture view of the software, defining red shells, is shown in Figure 5.8. As indicated in this figure, the centre of this shell is filled with concrete and it has a strip of steel at the centre, representing the steel strand at each end, which was modeled as a steel layer of 1.06 mm thickness. The label

for the steel strand is named “Tendon” and its properties are defined in the next subsections, along with the properties of other materials. Similar to the Wall 1 model, two layers of masonry were placed one on either side of the concrete layer. In addition, two layers of FRP sheets with 1 mm thickness were placed horizontally and vertically on one side of the shell. The explanation of other color-coded shells are provided in Appendix II.

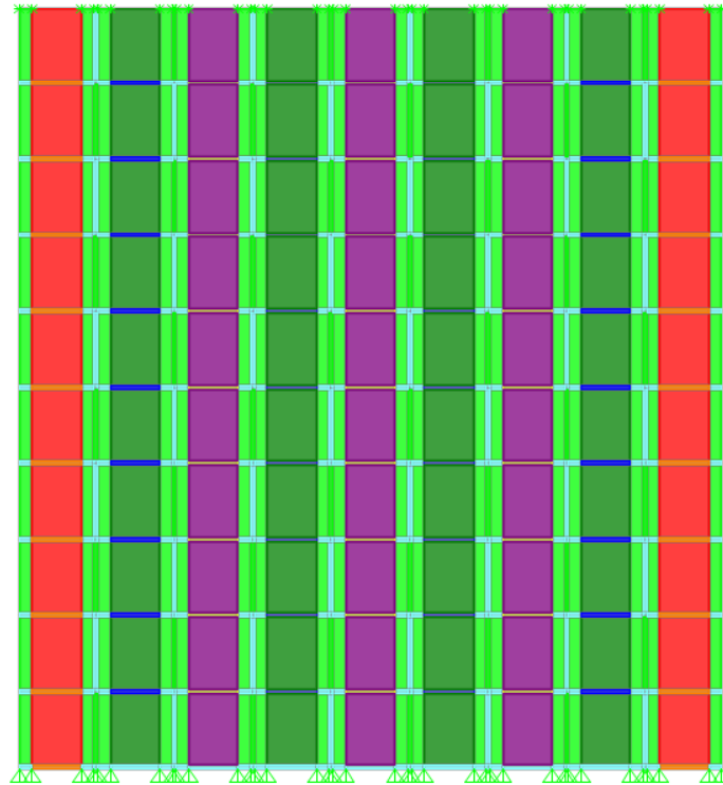


Figure 185.6 Front view of the analytical model for Wall 2

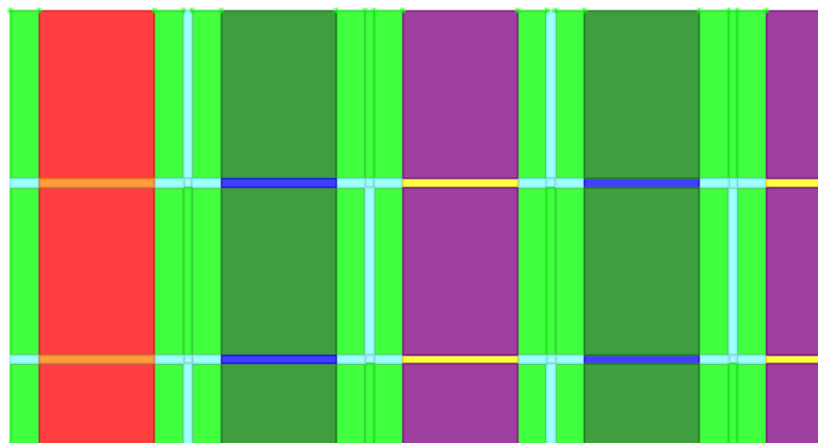


Figure 186.7 Close view to the top left corner of analytical model for Wall 2

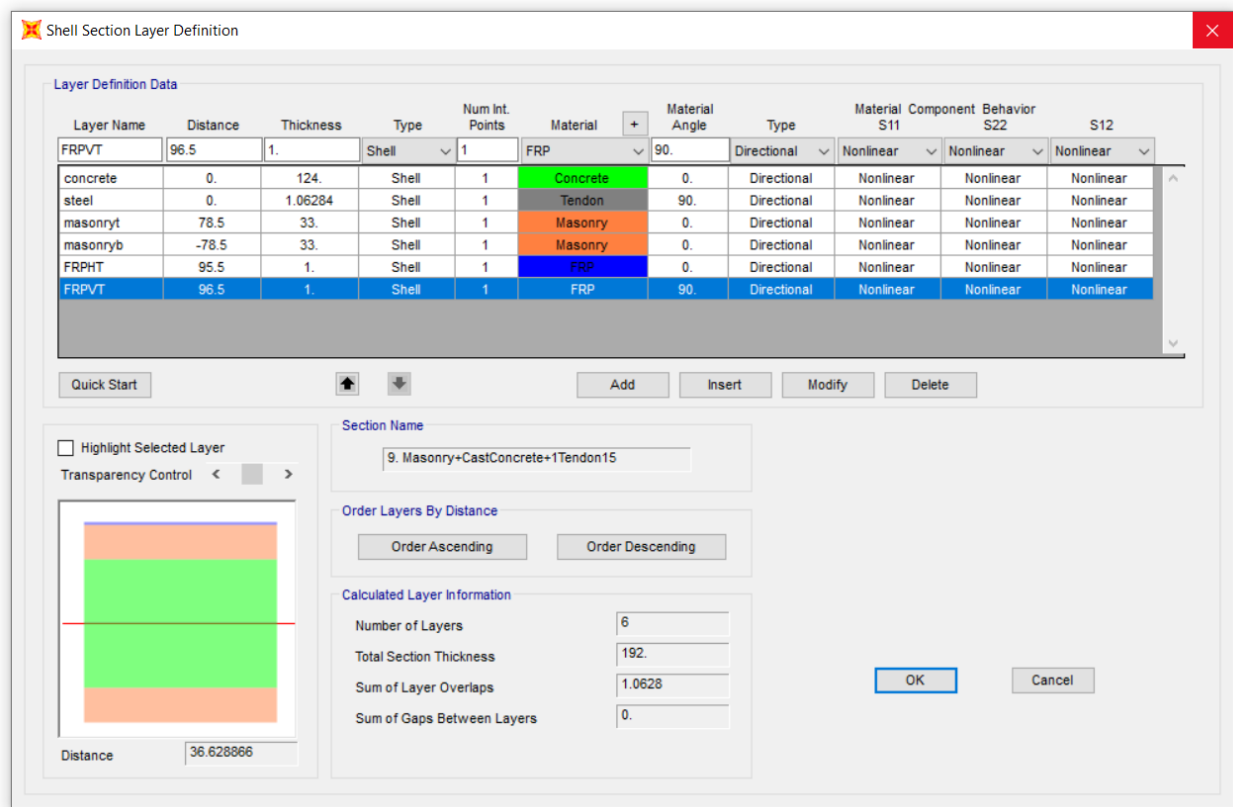


Figure 187.8 Red shell section for Wall 2

5.4.4 Materials

Material properties for each analytical model were defined according to the available material test results. The properties of concrete, masonry, and mortar used in the wall tests were established by performing material compression tests, as reported in Chapter 3. The properties of steel reinforcement were obtained through standard coupon tests as presented in Chapter 3. The 7-wire strands used in Wall 2 had a composite behaviour with the attached HSS yielding boxes, which was predicted according to the response of the strands that were tested in tension and the HSS box that was modeled by SAP2000 using the standard steel properties for the HSS. The FRP sheets were modeled according to the material behaviour that was reported by Arifuzzaman (2013). The following sections explain the details of the material stress-strain relationships as specified in SAP2000.

Concrete: The stress-strain relationship of concrete that was used inside the masonry cells in Walls 1 and 2 was obtained from standard cylinder tests of the actual concrete used. The concrete mix used resulted in an average compressive strength of 57 MPa. The concrete material model was

labelled as “Concrete,” and its stress-strain behaviour is specified as depicted at Figure 5.9. The concrete model selected in SAP2000 was of isotropic, directionally symmetric type. Concrete hysteresis type was selected for cyclic load analysis.

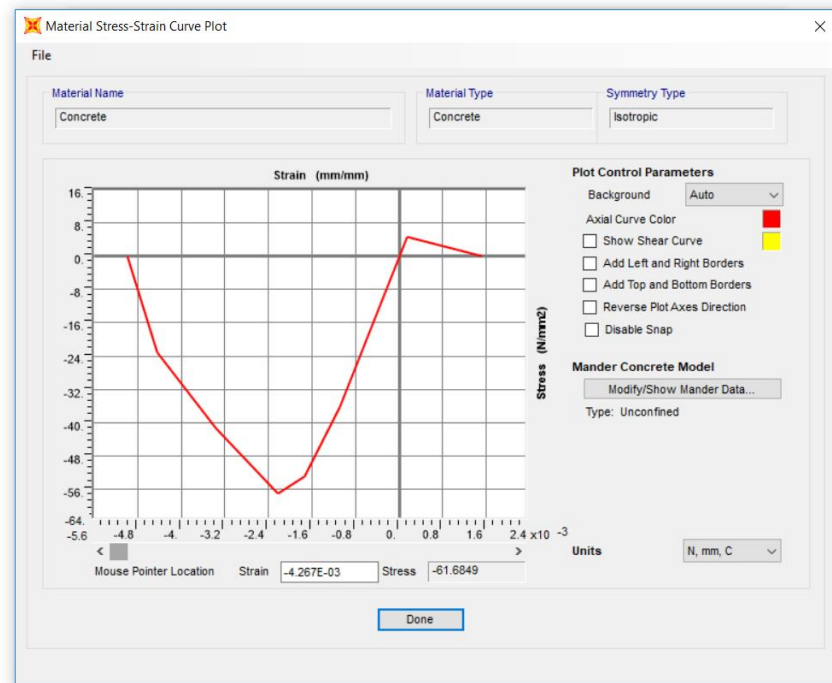


Figure 188.9 Concrete behaviour

Concrete Masonry: The concrete masonry blocks used in the experimental program were modeled on the basis of the compressive tests that were performed on masonry blocks. The average compressive capacity of the blocks was established to be 15 MPa. This material was labelled as “Masonry” in the analytical model with the stress-strain relationship specified as shown at Figure 5.10. Isotropic material symmetry type and concrete hysteresis type were selected in the analysis.

Mortar: The mortar that was used in the masonry joints of Wall 1 and 2 was modeled using the compressive strength of mortar, reported in Chapter 3. Figure 5.11 shows the stress-strain relationship used in the analytical model, which was named “Mortar,” with an average compressive strength of 7 MPa. Isotropic directional symmetry type and concrete hysteresis type was selected for analysis under reversed cyclic loading.

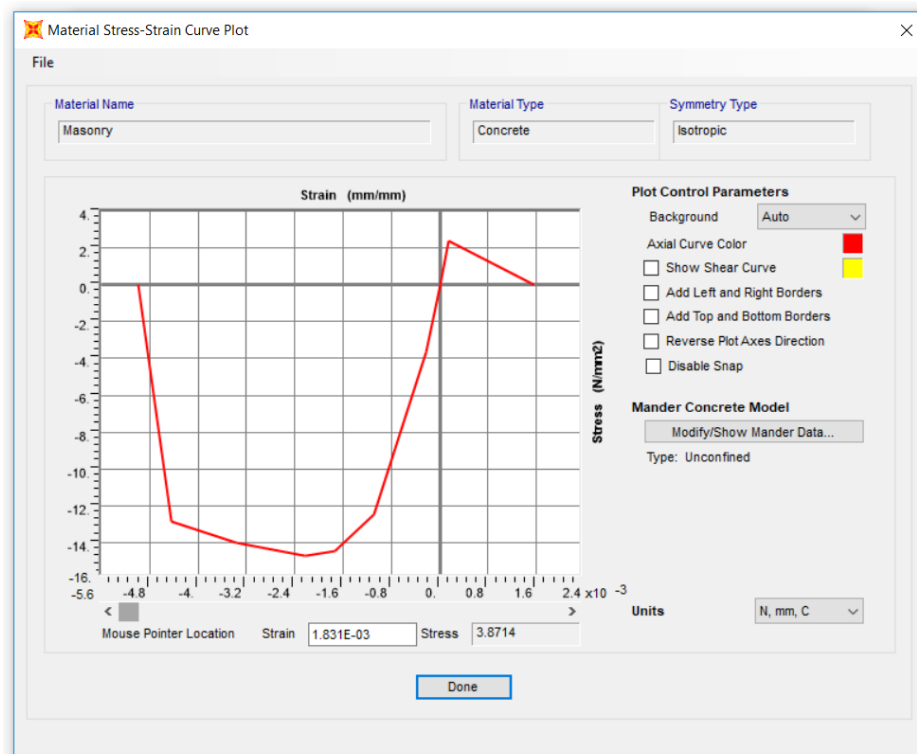


Figure 189.10 Masonry behaviour

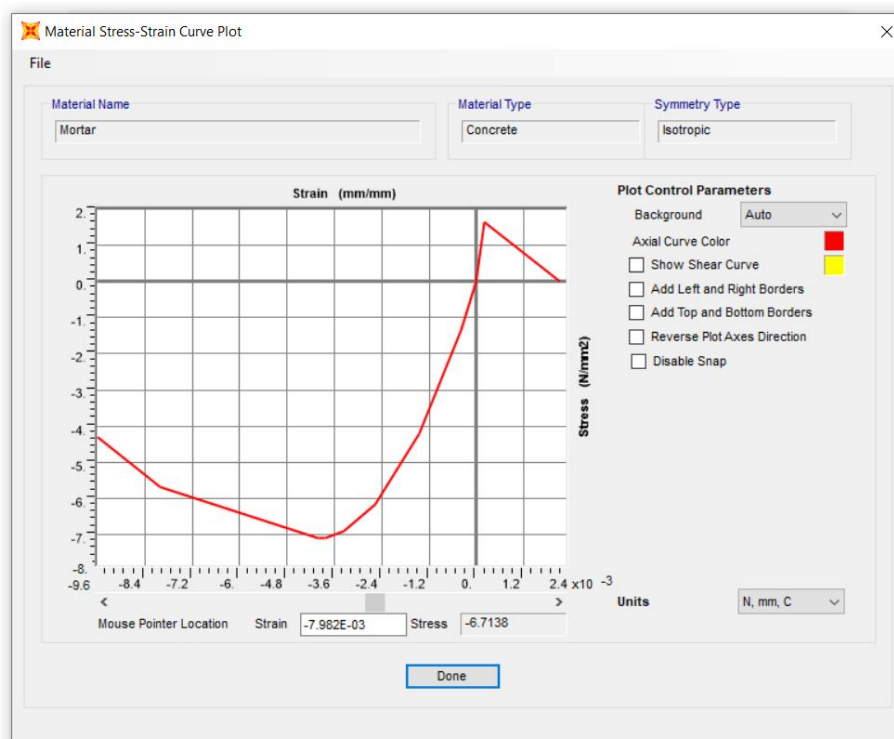


Figure 190.11 Mortar behaviour

Reinforcing Steel: Steel reinforcement used for retrofitting Wall 1 and Wall 2 was modeled to reflect the tension coupon tests conducted in the experimental investigation. The behaviour in compression was assumed to be the same as that obtained under tension. Figure 5.12 shows the stress-strain model used in wall analyses. The model was labelled as “Steel.” Isotropic directional symmetry type and kinematic hysteresis type was selected for the material.

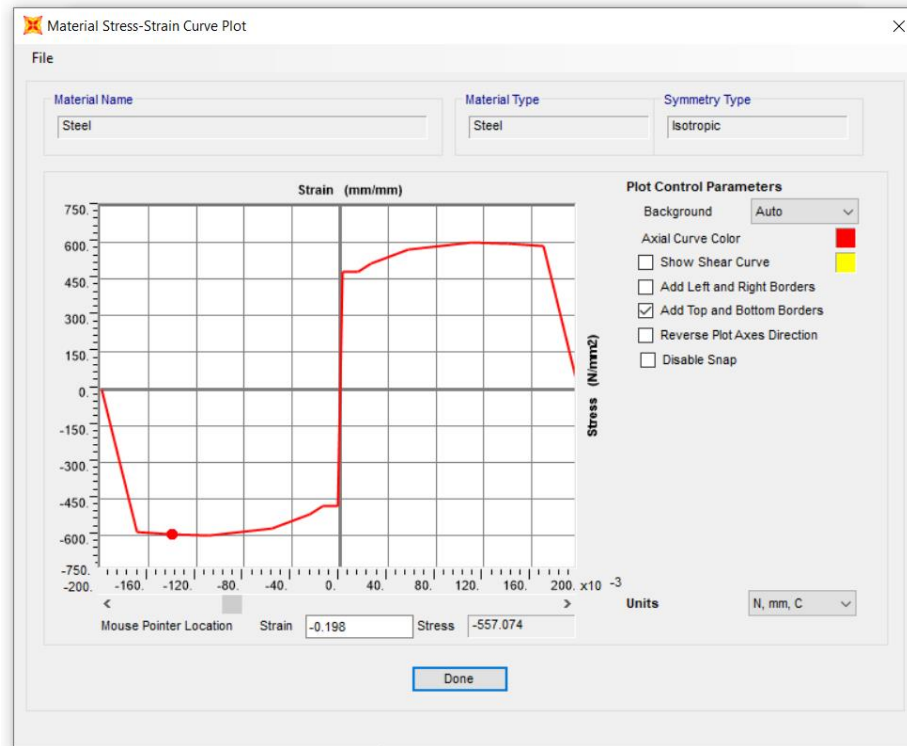


Figure 191.12 Reinforcement steel behaviour

Prestressing Tendons: The post-tensioning strands and HSS yielding agents behaved as a composite system during the Wall 2 test. The composite system resulted in the control of tensile forces in strands at higher lateral drift levels. The HSS yielding agent developed yielding prior to an excessive increase in strand tension force to prevent premature rupturing of the strands in tension or premature crushing of the toe concrete under flexural compression, while the strands remained elastic and provide self-centering capabilities to the wall. Therefore, the stress-strain relationship of the composite action was introduced to SAP2000 as the tendon-HSS composite material model. The yielding of HSS as a ductile element would also help dissipate energy.

The tendon model was specified after modelling the HSS with a tendon force, and conducting a FEM analysis of the composite system. The HSS box was modeled by Shell elements using SAP2000 as shown in Figure 5.13. A compressive point load was applied on top of the box at the

centre to simulate the loading scenario of the Tendon-HSS composite system. This analysis generated the force-deformation relationship for the HSS as shown in Figure 5.14. The composite material “Tendon” is defined, using the force-deformation response of HSS box and the stress-strain behaviour of the strand that was tested in tension, as reported in Chapter 3. Figure 5.15 shows the stress-strain behaviour of the Tendon-HSS composite material with a yield strength of 433 MPa, which is essentially because of the yielding of HSS. The yield capacity of 433 MPa is much less than the capacity of high-strength 7-wire strand used as prestressing tendon.

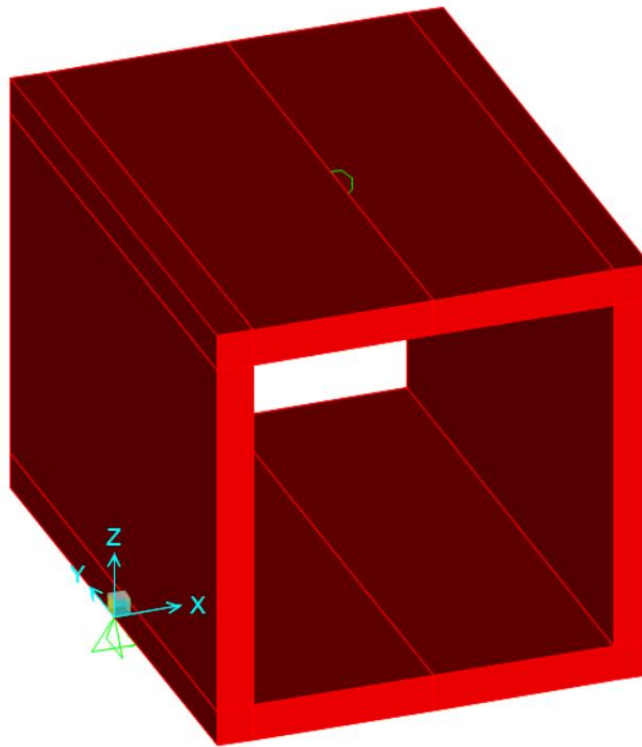


Figure 192.13 Shell model of HSS yielding box

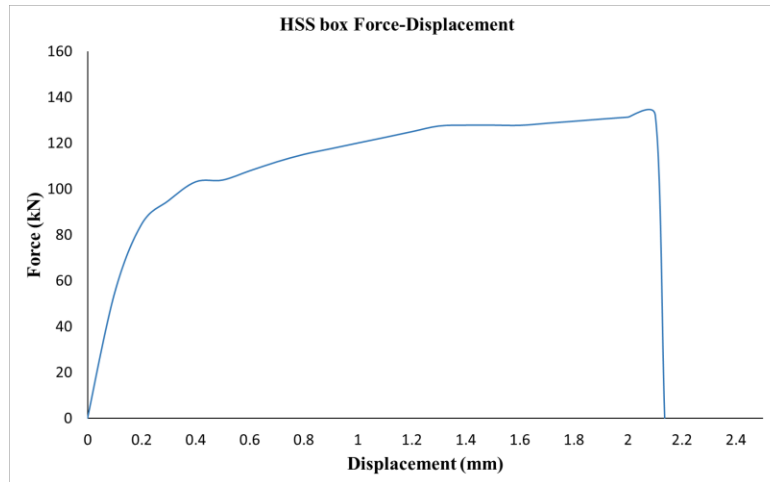


Figure 193.14 Force-deformation response of HSS box in compression

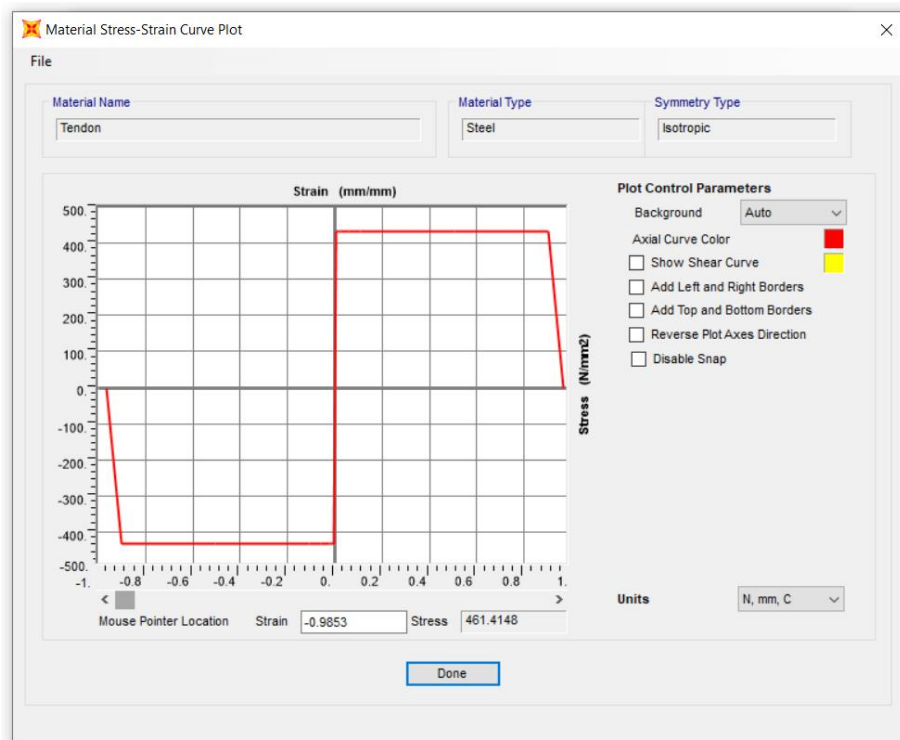


Figure 194.15 Tendon-HSS composite material behaviour

Fibre Reinforced Polymer (FRP) Sheets: The carbon FRP sheets used for retrofitting both Walls 1 and 2 were modeled using the FRP coupon tests conducted by Arifuzzaman (2013) as part of a companion test program at the University of Ottawa using the same material. The material showed linear elastic behaviour until rupturing in tension. Figure 5.16 shows the stress-strain response of the FRP material.

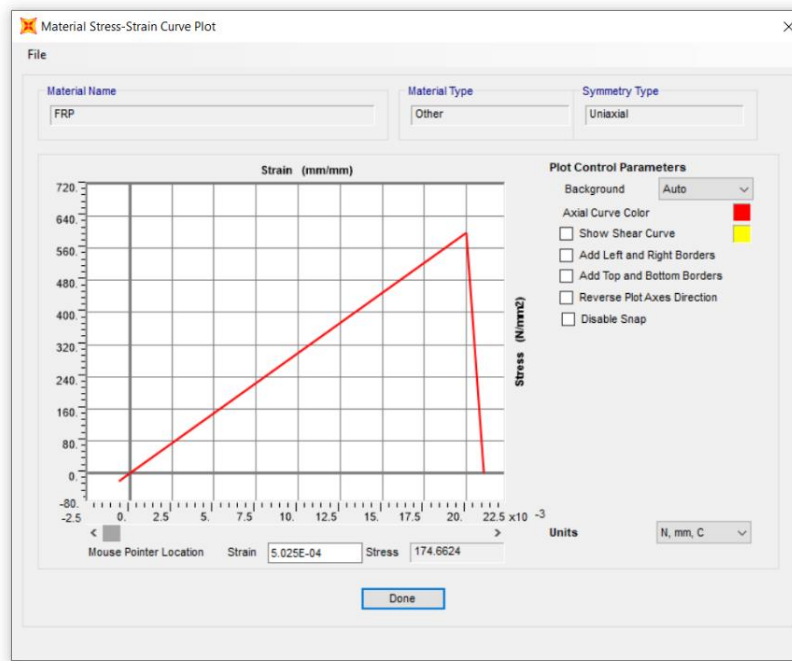


Figure 195.16 FRP behaviour

5.4.5 Loading

The analytical models were subjected to two types of loading functions to conduct the analyses: i) monotonic loading up to failure, and ii) reversed cyclic loading following the same loading regime as the tests. In all cases the same magnitude gravity load that was applied during the test program was also applied in the analyses. This corresponded to three equal concentrated loads along the top, resulting in a total vertical load of 100 kN. Pushover and cyclic loading functions that were created for the analyses are shown in Figure 5.18. Wall 1 was pushed 30 mm within 84 seconds, and Wall 2 was pushed 60 mm within 168 seconds for the monotonic (pushover) loadings. This loading rate was assessed to be slow enough to prevent strain rate effects on materials. Similarly, slow rate of loading was applied under reversed cyclic loading. Lateral and axial loads were applied over all the nodes on top of the wall. Figure 5.17 shows the approximate location of lateral loads in blue, and axial loads in orange.

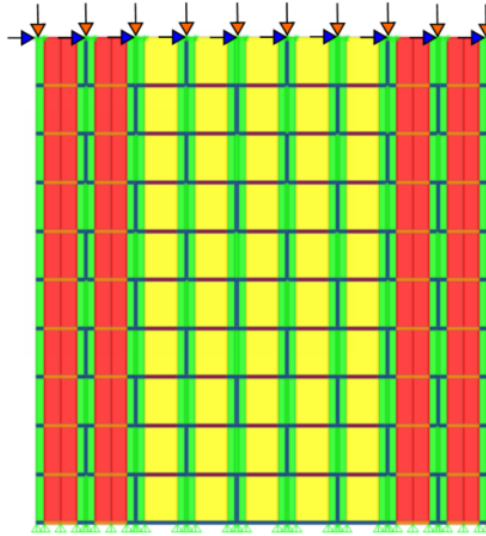


Figure 196.17 Application point of axial and lateral loads

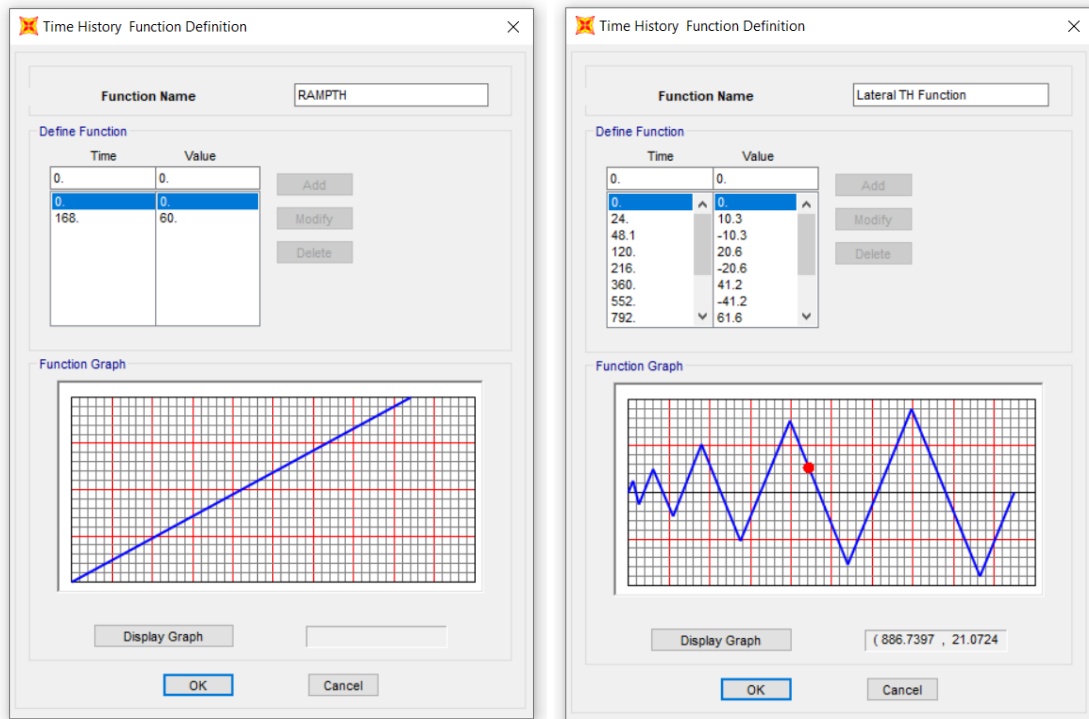


Figure 197.18 Pushover loading function for Wall 2 (Left), Cyclic loading function for Wall 2(Right)

5.5 Analysis of Walls

The analytical models developed for Wall 1 and Wall 2, presented in the preceding sections, were used to validate the models against experimental results. Companion models were also generated to undertake an analytical parametric investigation by varying some of the design

parameters. Four companion walls for Wall 1 and four companion walls for Wall 2 were generated to study the effects of filling masonry cells in wall web with concrete, changing the percentage steel or the level of prestressing, using FRP on both sides of the walls, and having different wall aspect ratio. The analytical results were presented in terms of wall force-drift response and stress contours for different materials in the wall models. The stress contours were captured at critical points on the force-drift relationships. Four critical points were selected for Walls 1 and 2, and two critical points were selected for the companion walls. The level of internal stress in each material was investigated at each drift level. Figure 5.19, obtained from SAP2000 manual, explains the nomenclature used in labelling the internal stresses.

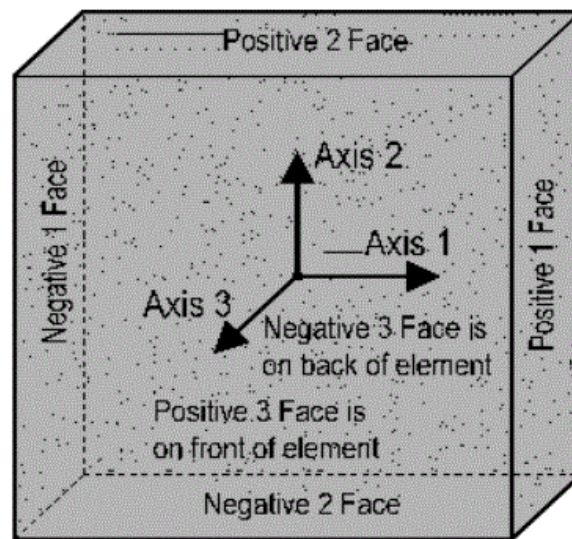


Figure 198.19 Internal stresses components, Sap2000 analysis reference manual

The stress components and internal forces referred to in this Chapter are expressed using the same nomenclature as that used in SAP200, as indicated below.

- **S11:** Direct stress (force per unit area) acting on the positive and negative 1 face in the 1-axis direction.
- **S22:** Direct stress (force per unit area) acting on the positive and negative 2 faces in the 2-axis direction.
- **SMax:** Maximum principal stress (force per unit area). Note that by definition principal stresses are oriented such that the associated shearing stress is zero.
- **SMin:** Minimum principal stress (force per unit area). Note that by definition

principal stresses are oriented such that the associated shearing stress is zero.

Material behaviour was studied by capturing the stress contours of S11, S22, SMax, SMin at different drift levels and by examining their variations. S22 in steel indicates vertical tension and compression in the longitudinal re-bars and tendons. The concrete, masonry, and mortar behaviours were examined by capturing the stress contours of SMax and SMin at different drift levels. These principal stresses indicate maximum compression and tension in concrete, masonry, and mortar.

Appendix III presents all the stress contours and internal forces that were captured in each analysis. Important stress contours that demonstrate structural behaviour are presented in this Chapter.

5.5.1 Validation of Wall 1 Model

The analytically generated pushover response curve for Wall 1 is compared with the experimentally recorded force-lateral drift hysteretic relationship in Figure 5.20. The analytical hysteretic relationship was also generated for the first cycle at each drift level because of the excessive time required to generate the entire hysteretic relationship. The comparison of experimental and analytical hysteretic relationships is shown in Figure 5.21. The initial stiffness of the analytical model agrees well with the experimental value. As was mentioned in Chapter 4, Wall 1 was erroneously pulled to 0.4% lateral drift when it was scheduled to be displaced to 0.2% drift ratio. This resulted in asymmetric behaviour of hysteretic response. When the same asymmetric loading history was applied to the analytical model, the hysteretic relationship showed fairly good match with that recorded experimentally. It was concluded from these comparisons that the analytical model for Wall 1 is representative of the experimentally recorded behaviour.

Stress contours for different materials that were used in the analytical model were captured at four different points on the pushover force-drift response. These critical points are shown in Figure 5.22. Point one is the point at which the initial elastic stiffness starts showing significant changes, indicating cracking and the onset of material nonlinearity in concrete. Point 2 is the maximum load capacity of the wall, where yielding of the wall as a whole is indicted. Point 3 and 4 indicate stress levels right after a significant drop in load resistance and a point during the residual load resistance region of the force-deformation response respectively.

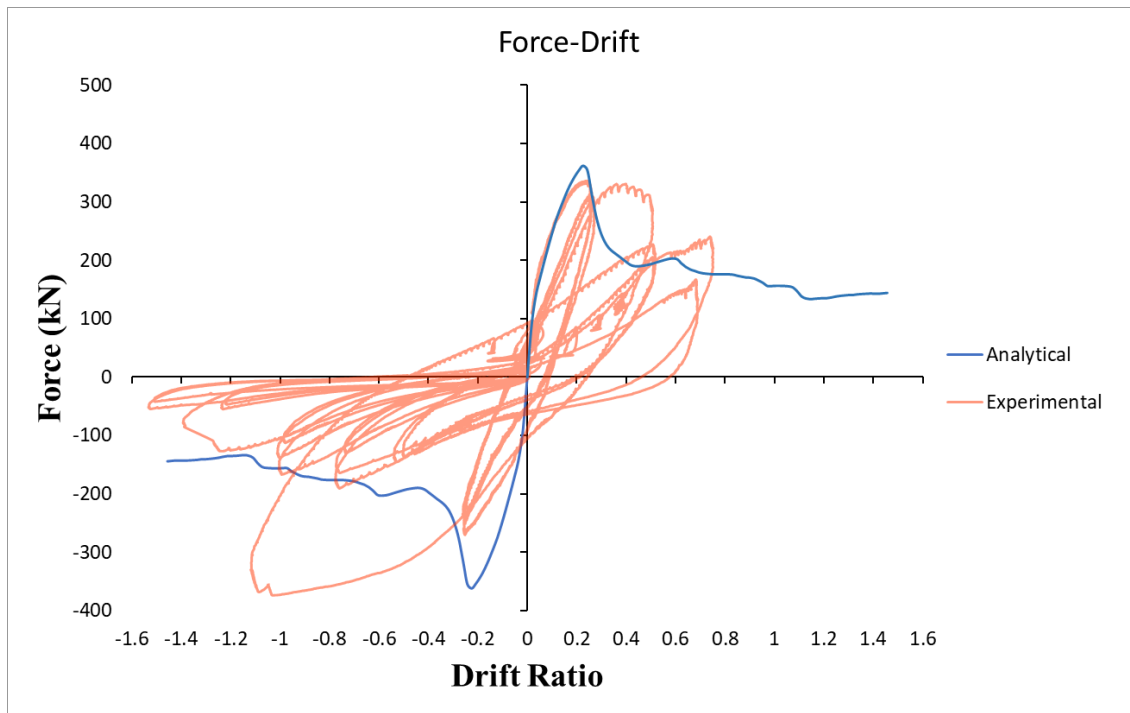


Figure 199.20 Analytical pushover vs. experimental cyclic response of Wall 1

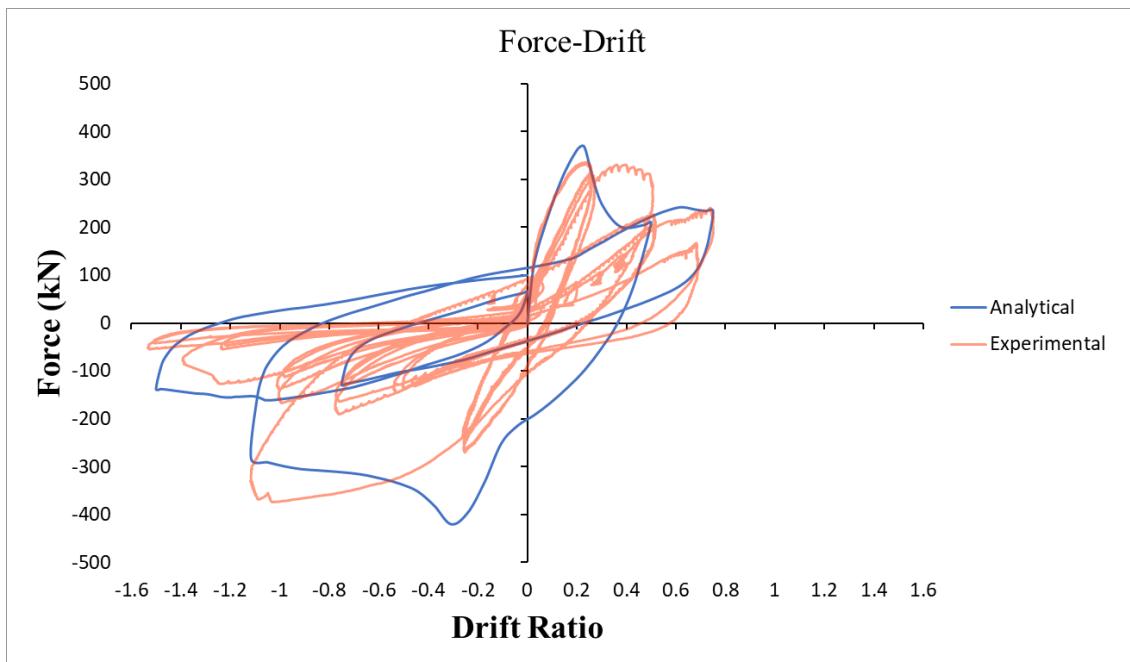


Figure 200.21 Analytical cyclic vs. experimental cyclic response of Wall 1

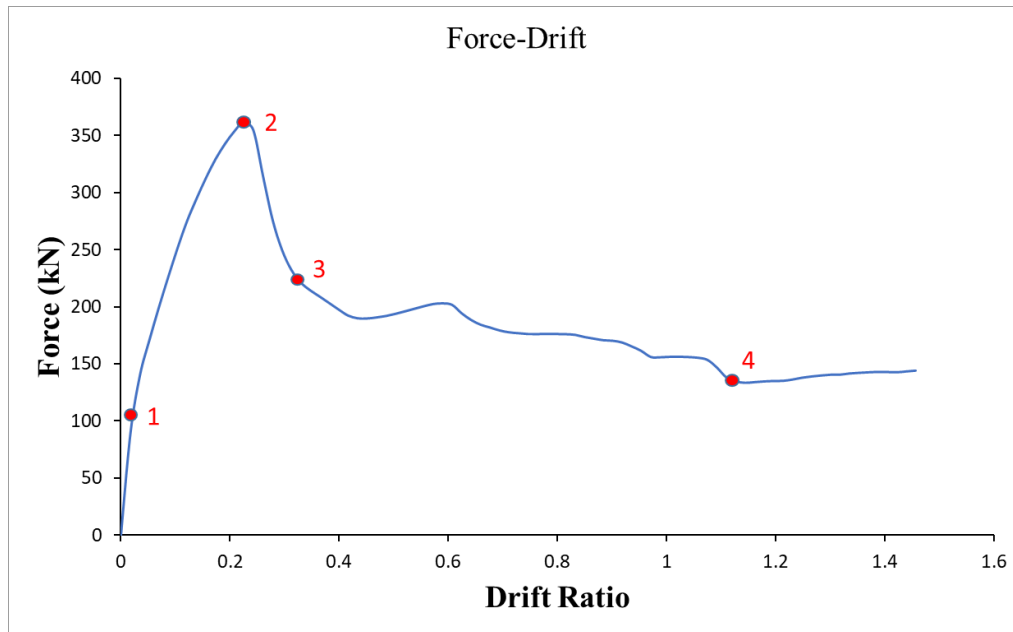
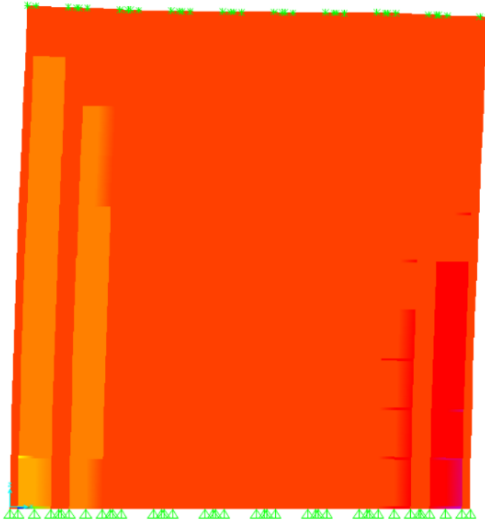
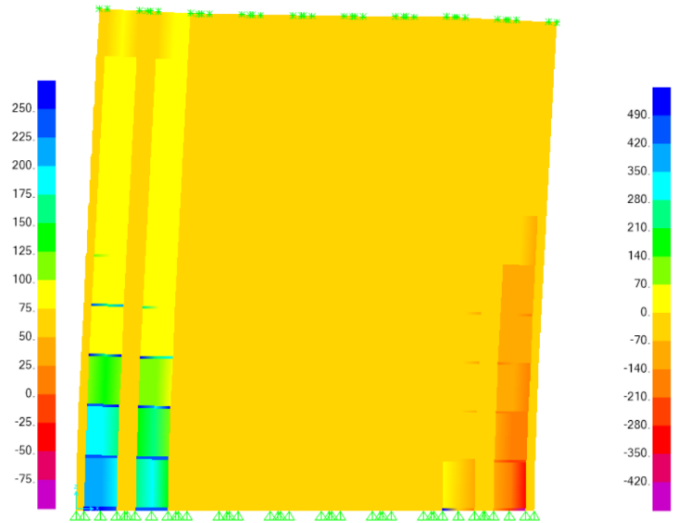


Figure 201.22 Pushover response of Wall 1

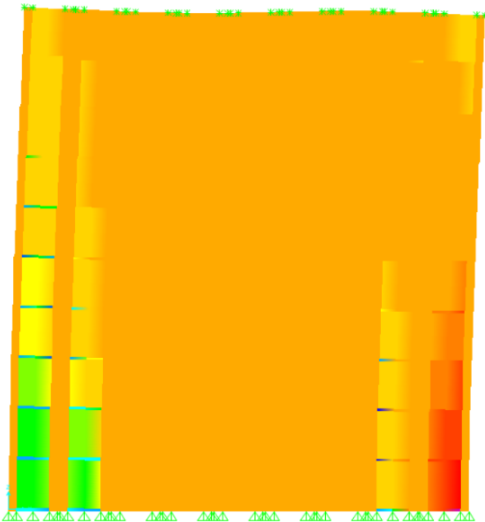
Figures 5.23 illustrate longitudinal stresses in reinforcing steels (S22) at four drift levels under push-over loading. Tensile stresses in the far-left reinforcement at Point 1 are in the range of 225-250 MPa, and compressive stress in the far-right reinforcement is in the range of -50 to -80 MPa. The stresses at point 1 indicates that the reinforcement remains elastic at this stage of loading, as expected. Changes observed at mortar joint locations indicate the onset of cracking in mortar joints. The reinforcement stresses increase at Point 2. Figure 5.23(b) indicates that the tensile stress (on the tension side) ranges between 350 and 500 MPa, and the compressive stress (on the compression side) ranges between -140 and -430 MPa. Highest stress values are recorded at mortar joint locations, which indicates yielding of steel at the joints. Tensile stress in steel reinforcement at the second cell from the wall tie indicates that the neutral axis is located near the centre of the second cell. The steel stress values at points 3 and 4 indicate strain hardening in steel. This is shown Figure 5.23(c) and 5.23(d). At this stage of loading, the neutral axis starts moving toward the right side (towards the compression side) of the wall, taking place between cells one and two. The reinforcement steel at the second cell location is completely under tension and the stress value is increasing as the wall is pushed from Point 3 to 4.



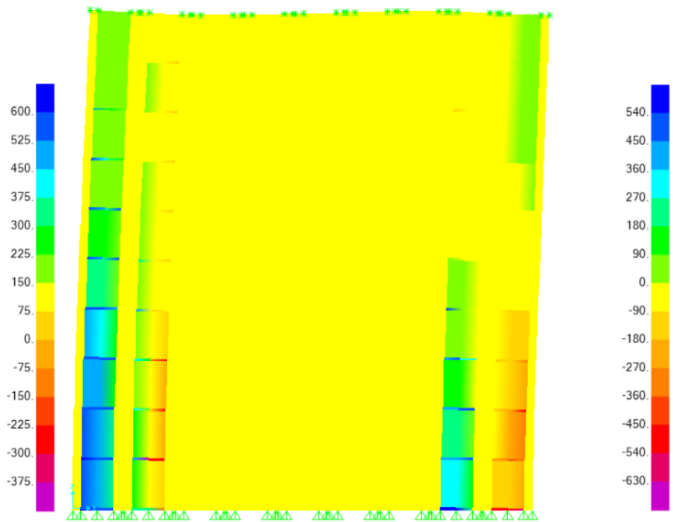
(a) At Point 1



(b) At Point 2



(c) At Point 3



(d) At Point 4

Figure 202.23 Steel stresses, S22 in Wall 1

Principle stresses, SMax (maximum tension) and SMin (maximum compression), are plotted for concrete, masonry, and mortar shells for the four stages of loading selected (Points 1 through 4). Figure 5.24 shows SMax stress contours of concrete for Wall 1. The maximum tensile stress at Point 1 is recorded at the heel region (left) of the wall, indicating 4.5 MPa stress, which is close to the cracking stress. The tensile stress at the heel is reduced to zero when Point 2 is reached, which indicates the cracking of concrete. At point 2, the maximum tensile stress has shifted up to row four from the heel of the wall, and some concrete shells started developing tension near the toe region, indicating the shifting of neutral axis towards the toe. The recorded SMax at

Points 2 and 3 for concrete are also presented in Figure 5.24(b) and 5.24(c). The cracking propagated further towards the toe region under increasing lateral drift at Points 3 and 4.

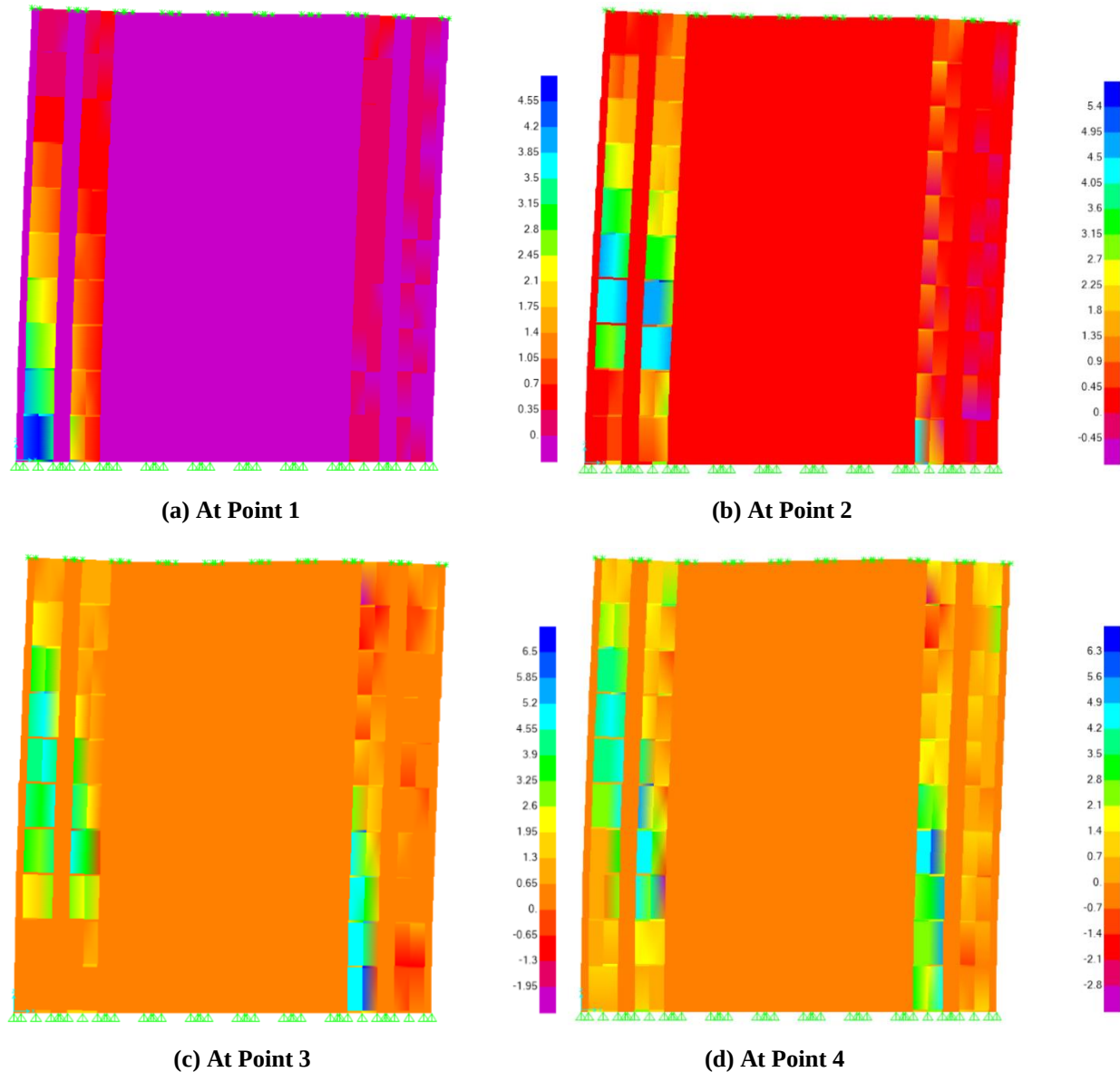


Figure 203.24 Concrete tensile stresses, SMax in Wall 1

The recorded concrete compressive stresses (SMin) are shown in Figure 5.25 for Wall 1. The stress contours indicate that maximum compressive stresses are generated in the toe region, as expected. The maximum compressive stress is increased from -5.5 MPa to -17 MPa as the wall is pushed from Point 1 to Point 2. As the lateral drift is increased towards Points 3 and 4, the maximum compressive stress remains at about the same level, maintaining a stress level of -16 MPa during load stages at Points 3 and 4. This indicates limited damage to concrete in the

compression zone of the wall toe, which is consistent with experimentally observed behaviour as this wall failed due to excessive diagonal compression in the web region.

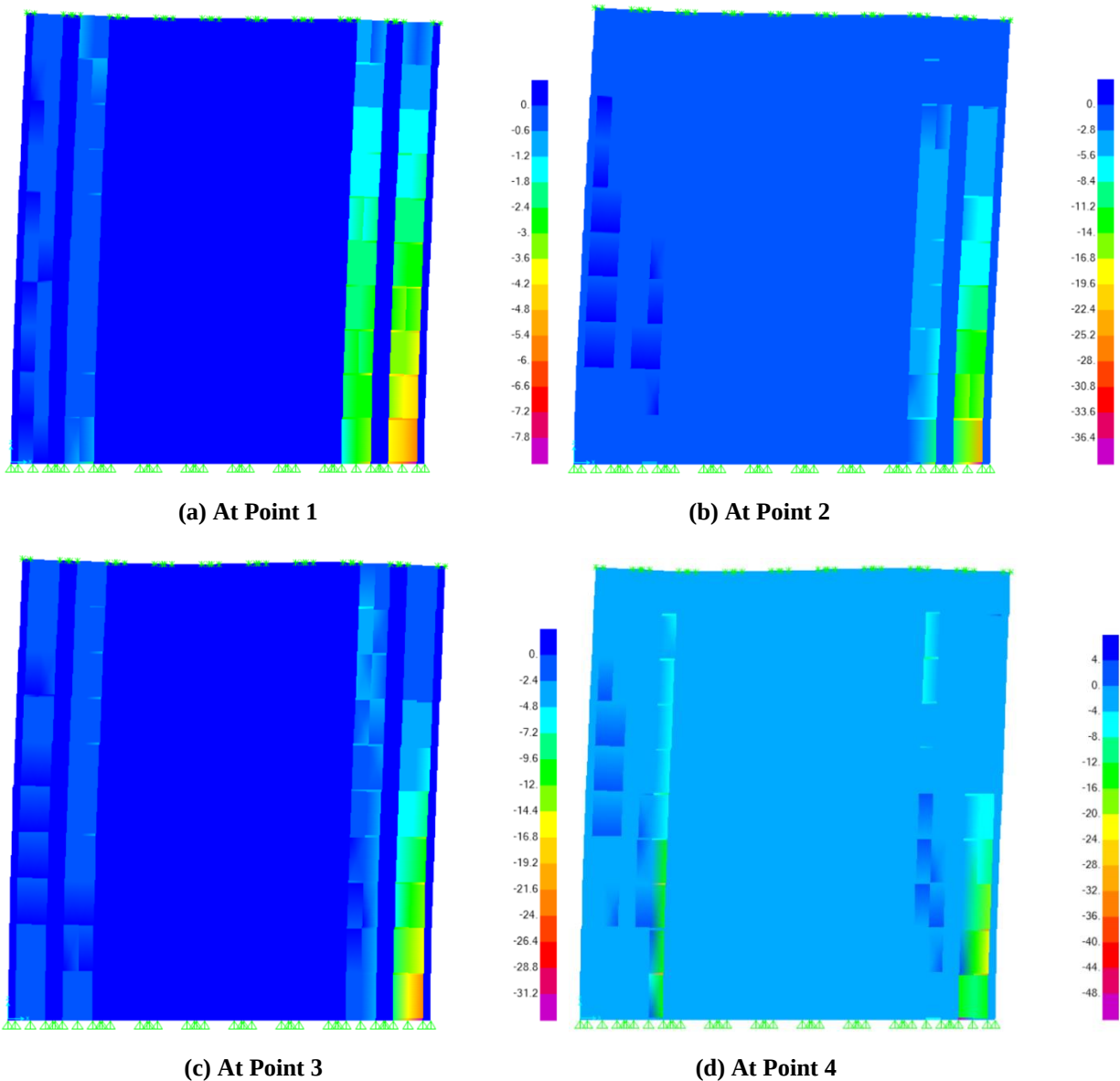


Figure 204.25 Concrete compressive stresses, SMin in Wall 1

The SMax, and SMin stress contours are also plotted for Masonry shells at the same four selected drift levels. Figure 5.26 shows the maximum tensile stresses (SMax) in masonry block shells. The maximum tensile stress of 2.0 MPa is recorded at Point 1 in the wall heel region, which decreases as the masonry concrete starts cracking during loading to Point 2. Higher stress levels are recorded in the empty cell regions within the wall web. This was attributed to the smaller cross-sectional area of masonry in these regions. The tensile stresses continued increasing in the central

portion of the wall at Points 3 and 4. High tensile stress (SMax) of 4 MPa of masonry (point 2) is taking place at the border of filled boundary columns and unfilled cells at the left and right sides of the web. This phenomenon is due to stiffness difference between boundary elements and centre of the web. This phenomenon is due to stiffness difference between boundary elements and centre of the wall that caused tensile cracks at left and right interfaces.

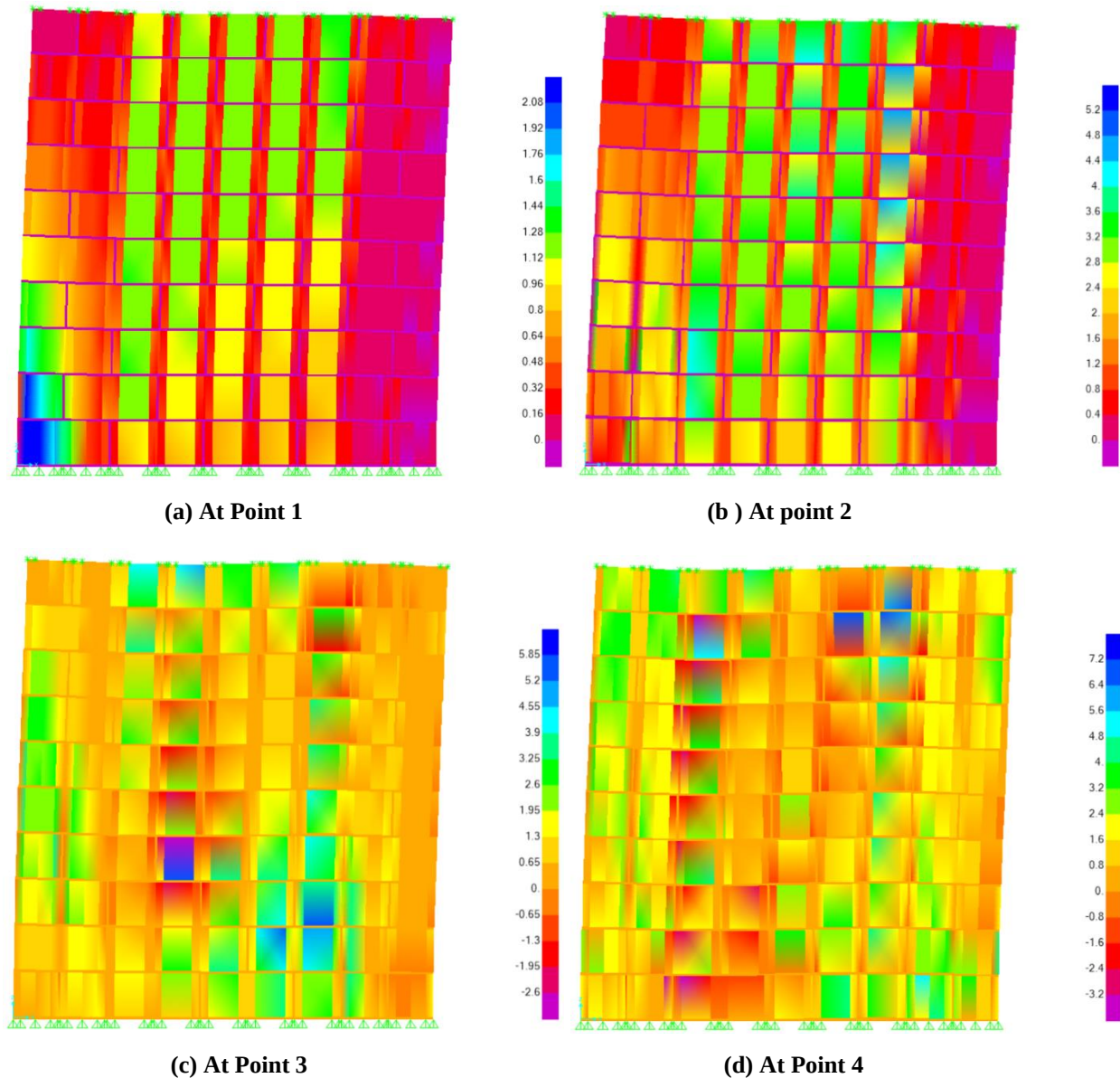


Figure 205.26 Masonry tensile stresses, SMax in Wall 1

Figure 5.27 illustrates the recorded maximum compressive stresses (SMin) in masonry shells. Compressive forces increase as the wall is pushed from Point 1 to Point 2. As is the case for maximum tensile stresses, maximum compressive stresses are observed to be higher in the hollow masonry unit regions of the wall web, ranging from -2.6 MPa to -5.85 MPa at point 2. The

stress values in the central and bottom rows of the wall (in the web) are reduced to zero at point 2, indicating the failure of masonry in these regions. These zero stress regions expand to higher levels of rows at loading Points 3 and 4. This indicates the failure of the web due to diagonal crushing which is consistent with the experimental observations.

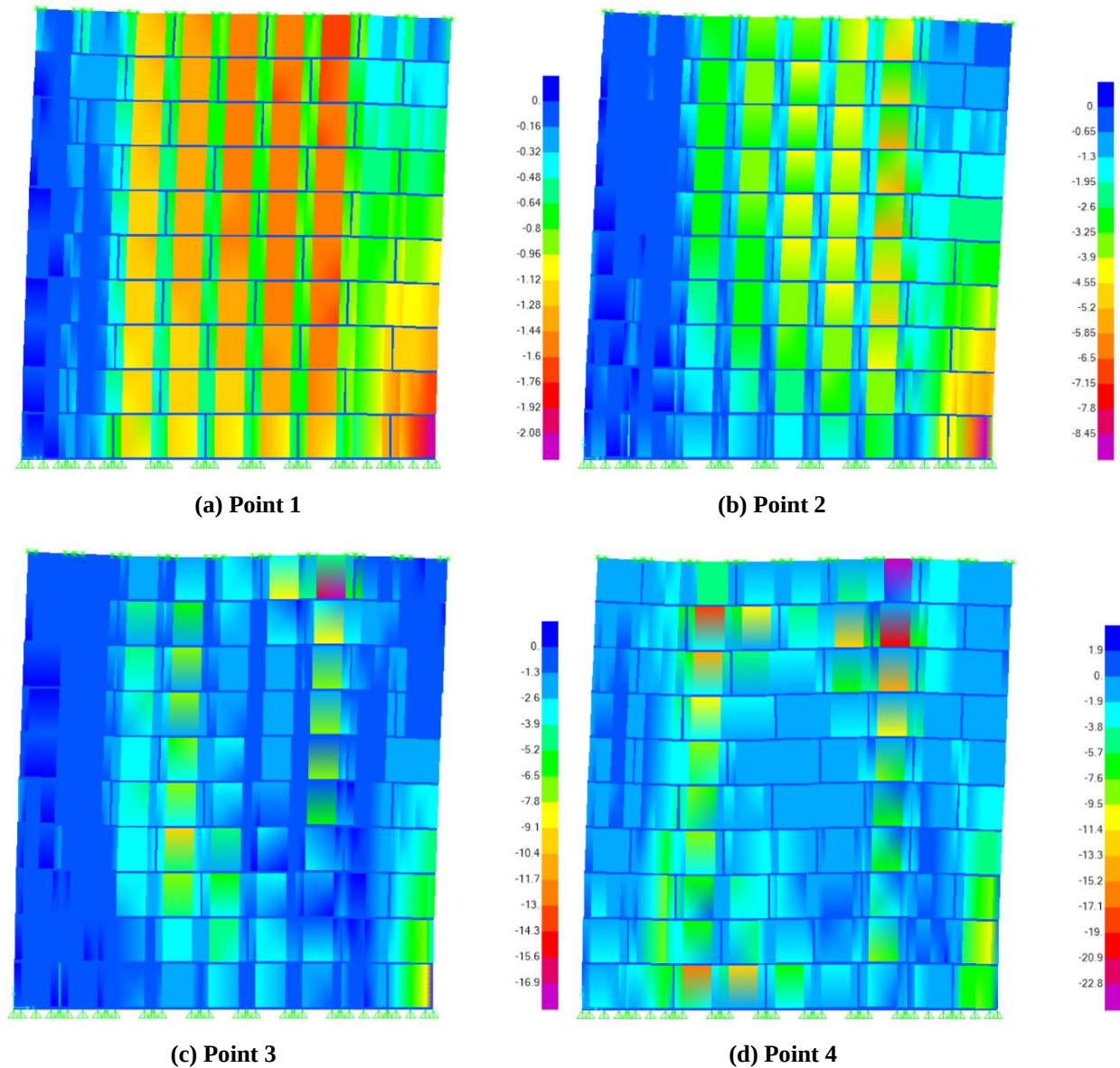
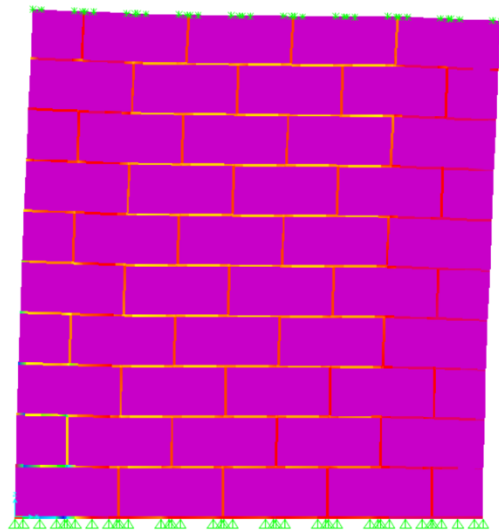


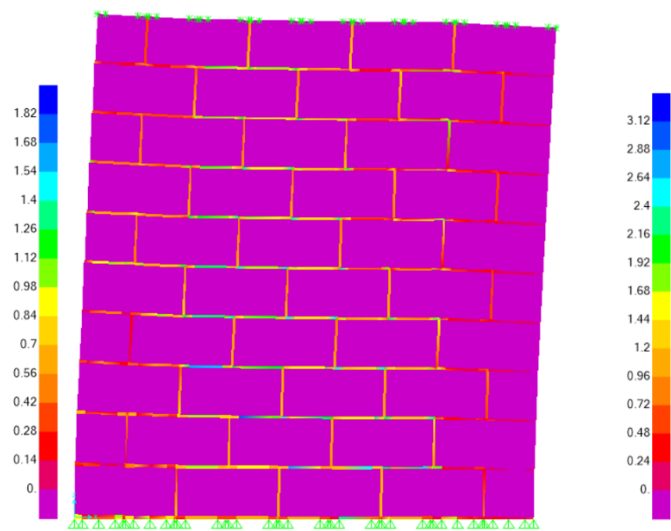
Figure 206.27 Masonry compressive stresses, SMin in Wall 1

Figure 5.28 shows maximum tensile stresses (SMax) in mortar joints. The SMax stress contour for mortar of Wall 1 at point 2 is clearly showing the highest values of tensile stress at the diagonal region of the wall. These stress values are ranging from 1.44 MPa to 2.4 MPa which justifies the development of diagonal step cracks at joints of the wall 1 during the experiment.

Maximum mortar tension of 1.85 MPa is recorded near the wall heel when the wall was pushed to Point 1. The mortar in the same location started cracking as the wall is pushed further to Point 2. Cracking is indicated by the reduction in mortar tensile stresses. The increase in tension near the heel region is consistent with the high tensile stresses observed in the reinforcing in the same location. The maximum tensile stress of 3.12 MPa at Point 2 is recorded in the wall web within the lower rows of masonry blocks. The high tensile stresses indicate tension failure of the mortar in the wall web. This phenomenon is consistent with the experimental observations. The central concrete blocks were observed to move relative to each other during the test at about the same load stage. As the lateral drift increased to Points 3 and 4, the SMax values in mortar joints gradually reduced to zero, indicating widespread cracking in the joints. The highest recorded tensile stress in mortar joints at Point 4 is located away from the central web region, which indicates the deterioration of the web.



(a) Point 1



(b) Point 2

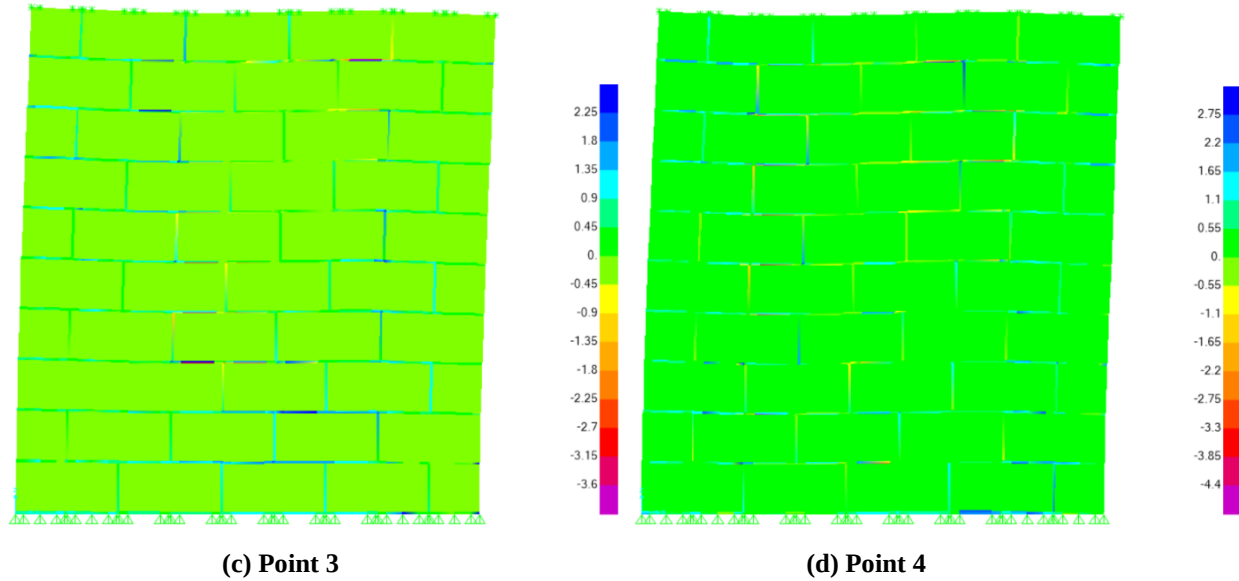
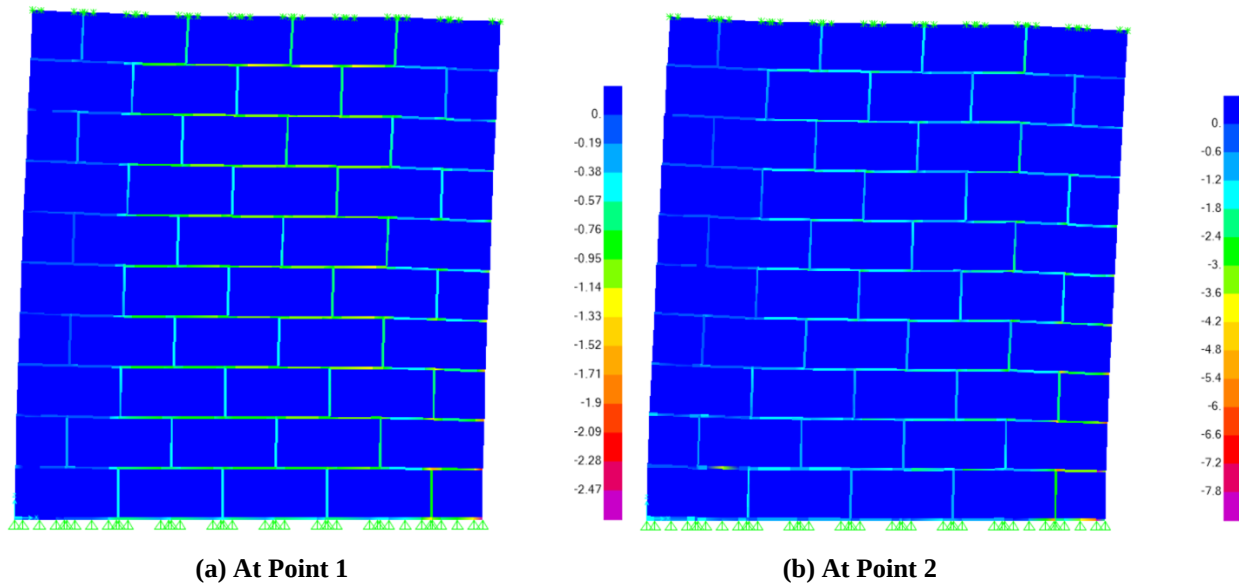


Figure 207.28 Mortar joint tensile stresses, SMax in Wall 1

Figure 5.29 shows maximum compressive stresses (SMin) in mortar joints of Wall 1. The maximum compressive stresses in joints increase from -0.6 MPa at Point 1 to -1.8 at Point 2. As the wall is pushed to Points 3 and 4, the maximum mortar compressive stresses become about -5.0 MPa and -8.0 MPa, respectively. Compressive failure of mortar is observed near the central web portion and the toe region. Once again this is consistent with the experimental observations.



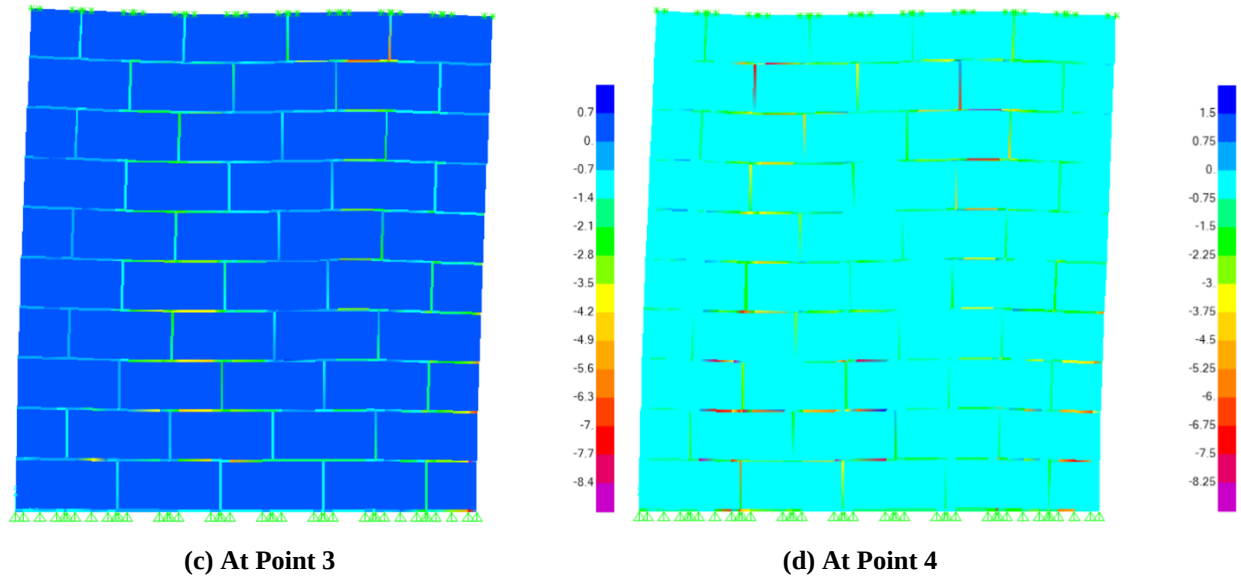


Figure 208.29 Mortar joint compressive stresses, SMin in Wall 1

The stress contours for horizontal and vertical FRP shells are captured as S11 and S22, respectively. The horizontal FRP shell is placed on the masonry, and the vertical FRP shell was placed on top of the horizontal FRP as the second layer. Horizontal and vertical directions refer to the directions of carbon fibres in the FRP. The FRP stress values indicate higher engagement of horizontal FRP compared to the second layer with fibers aligned in the vertical direction. Figure 5.30 shows stresses in the first layer of FRP sheet with fibres aligned in the horizontal direction. They show gradually increasing horizontal stresses with increasing lateral drift. The horizontal FRP showed highest stresses at vertical mortar joint locations when the wall was at Points 1 and 2. However, the maximum FRP stress at Points 3 and 4 are recorded in the central web region where the hollow blocks experienced damage.

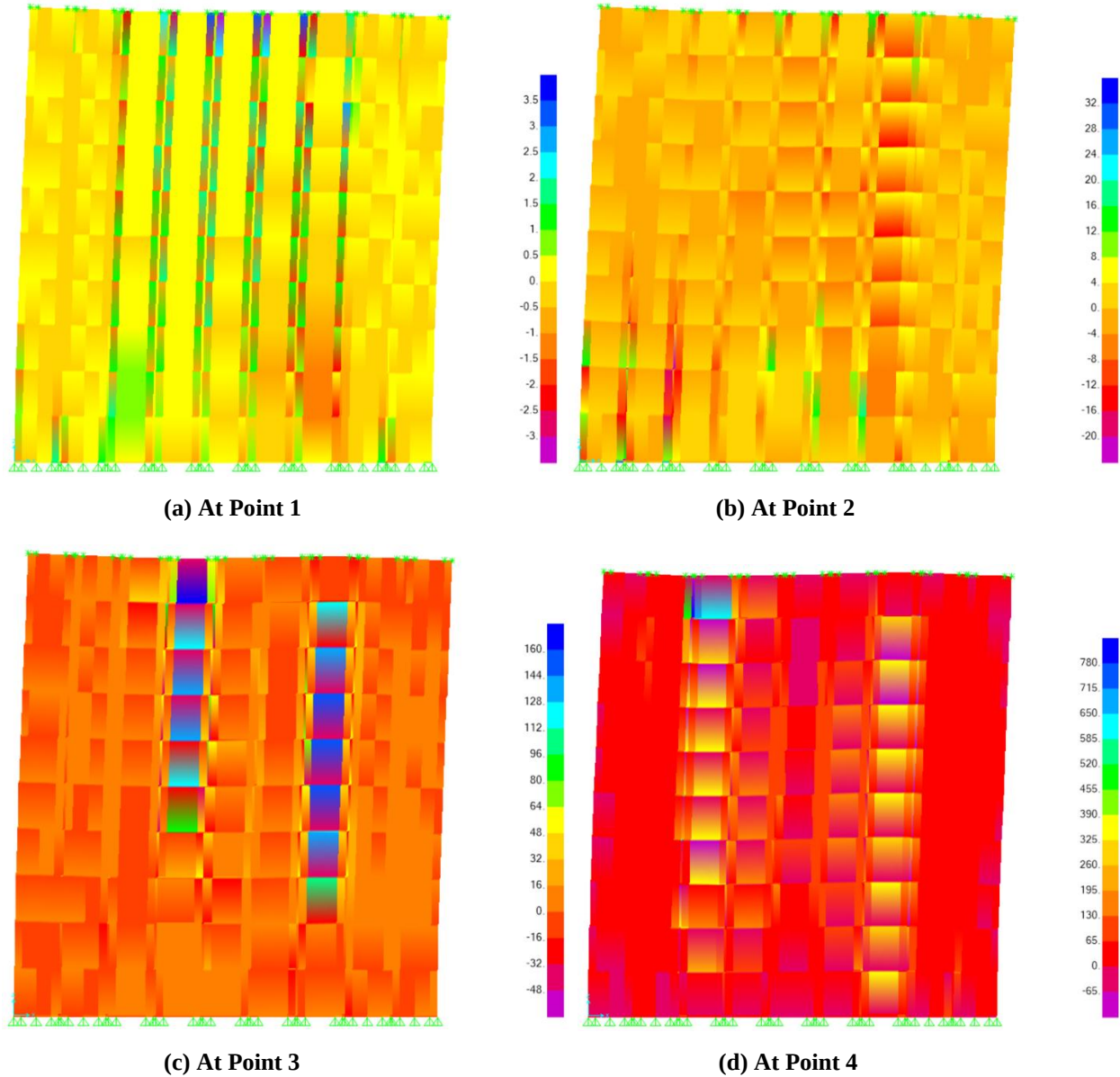


Figure 209.30 Horizontal FRP stress, S11 in Wall 1

Figure 5.31 illustrates maximum stresses recorded in the vertical FRP sheet (S22). Highest S22 stresses (336 MPa) are recorded at mortar joint locations near the left boundary element in the wall heel region. This is expected since this is the vertical tension region contributing to wall flexural resistance, even though the FRP fibres are not anchored to wall foundation. The recorded stress values for the vertical FRP at other locations are limited to 100 MPa. During Points 3 and 4 some regions show increased vertical FRP stress along the horizontal mortar joint, developing stresses in excess of 300 MPa.

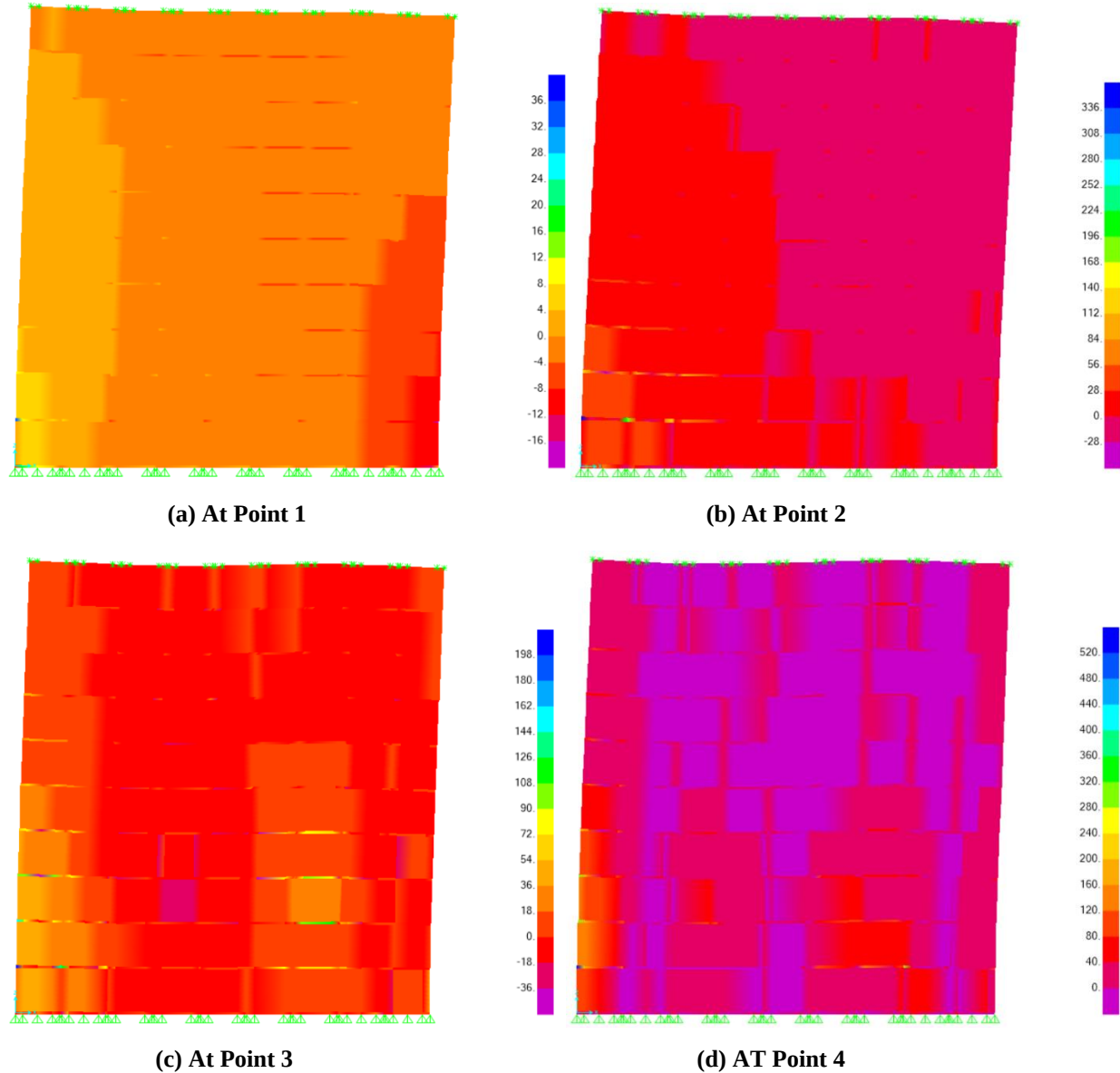


Figure 210.31 Vertical FRP stresses, S22 in Wall 1

5.5.2 Parametric Investigation of Wall 1

Following the validation of the analytical model for Wall 1, an analytical parametric investigation was carried out to investigate the significance of selected design parameters. These include; i) filling the wall web with concrete to enhance the web diagonal compression capacity, ii) increased percentage of reinforcement in the boundary elements, and iii) higher wall aspect ratio increasing flexural demands. The following subsections provide discussion of observed behaviour. The overall comparison of the effects of these design parameters, as well as the comparison with the unretrofitted reference wall are provided in the Chapter.

5.5.2.1 Wall 1 with Concrete Filled Web (Wall 1-a)

The experimental and analytical investigations of Wall 1 indicated lack of diagonal compression capacity in the central web portion of the wall. The wall was strengthened for flexure, by providing internal reinforcement and filling boundary masonry block cells with concrete. The diagonal tension capacity was also enhanced by providing carbon FRP sheets on one side of the wall. However, it was observed that the diagonal compression resistance provided by concrete blocks was not sufficient under reversed cyclic loading. This resulted in the crushing of masonry block shells, resulting in a rapid strength degradation after the initial peak load resistance at 0.2% lateral drift ratio. The lack of diagonal compression capacity was addressed in Wall 2 by filling the hollow cells of the block masonry with concrete in the central web region. The effect of this improvement on Wall 1 was investigated analytically. A new analytical model was developed to study the behaviour of Wall 1 with concrete filled cells in the web and was subjected to monotonically increasing lateral load (push-over load). This new wall model was identical as Wall 1-a. The pushover response of Wall 1-a is plotted in Figure 5.32. Considerable improvement in lateral load capacity and ductility was observed relative to Wall 1. The shear capacity of Wall 1-a increased by about 50%. The increased diagonal compression capacity of Wall 1-a prevented shear failure in the web and this allowed the internal steel reinforcement to reach its ultimate capacity, resulting in higher lateral force resistance and sustained ductility up to about three times that observed in Wall 1.

Stress contours for all the materials used in Wall 1-a are plotted in Appendix III at two load stages: Points 1 and 2. Point 1 represents the maximum load capacity of the wall and Point 2 represents a load stage during the strength decay region, approximately half way through complete collapse. The steel stresses (S22) indicate maximum tension and compression (450 MPa and -450 MPa) at mortar joints of toe regions where cracking of mortar occurs and high localised strains are induced. They also show that the neutral axis has shifted toward the toe region as strength decay takes place (Point 2). At a relatively high drift ratio, a substantial portion of the toe concrete continues resisting compression, unlike the companion Wall 1. This is attributed to the increased deformability associated with higher diagonal compression capacity of the web. Tensile stresses in concrete, masonry block and mortar all indicate cracking in the heel region, progressively increasing going from Point 1 to Point 2. Compressive stresses in these elements reach high values (-24 MPa) near the toe, as expected. Unlike Wall 1, Wall 1-a is not showing any major stress

concentration at the interface of web and boundary elements. The stress contour of mortar SMax for Wall 1-a at point 1 is not showing high stress concentration at diagonal regions. The recorded tensile stress values at joints are ranging from 0.46 MPa to 1.8 MPa and it is relatively constant all over the wall. This can be attributed to the improved compressive and tensile strength of the diagonal struts from concrete filled web at Wall 1-a. Maximum tensile stresses in horizontal and vertical FRP occur near the lower rows of masonry blocks and the heel region of the wall, respectively.

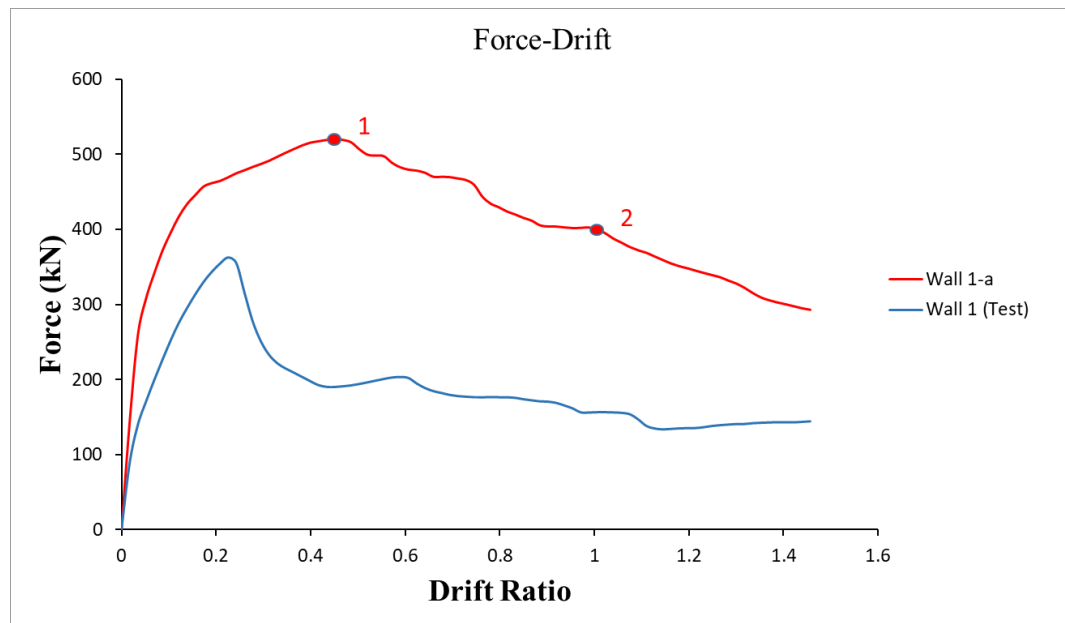


Figure 211.32 Push-over response of Wall 1-a and comparison with Wall 1 test

5.5.2.2 Wall 1 with Concrete Filled Web and Increased Boundary Element Reinforcement (Wall 1-b)

An analytical model was developed to investigate the effect of percentage of longitudinal reinforcements in the wall boundary element. The model was referred to as Wall 1-b. It was similar to Wall 1-a model (with concrete filled web) but with larger size bars (20M longitudinal bars) placed in both boundary elements (instead of 15M bars). Figure 5.33 shows the pushover response of Wall 1-b. The implementation of higher percentage of longitudinal reinforcements increased the in-plane force capacity of the wall by about 20%. This was the net result of increasing flexural reinforcement. However, this wall had a sharper post peak decay of strength compared to Wall 1-a. Higher lateral force capacity associated with increased flexural reinforcement increased shear force demand, resulting in somewhat earlier crushing of concrete in the compression toe region under combined flexural and diagonal compressive stresses.

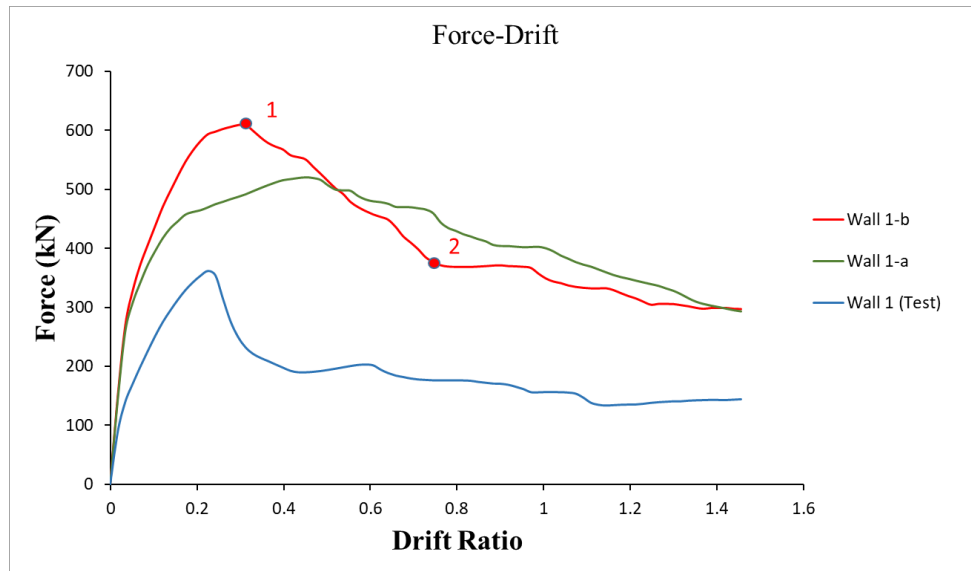


Figure 212.33 Push over response of Wall 1-b with Wall 1-a and Wall 1

Stress contours for different materials used in the wall are presented in Appendix III at two selected points, Point 1 representing the peak resistance and Point 2 representing a post-decay region. The stress contours for the 20M bars indicate that steel strain hardening was developed on the tension side at Point 1. Tensile stress of the left toe region at point 1 is ranging from 425 MPa to 560 MPa. It was further observed that the tension zone extended up to the second pair of 20M bars on the compression side (second row from the extreme compression fibre) at Point 2, indicating a significant shift in the neutral axis location as strength decay progressed towards Point 2. The stress contours in concrete indicates a variation in diagonal tensile stresses of 4 MPa to 7 MPa, exceeding diagonal cracking stresses in the web. Concrete compressive stresses reached up to about 40 MPa at point 1, decreasing towards Point 2. The stresses on FRP sheets were observed to be similar to those recorded for model Wall 1-a. The effect of concrete filled web at Wall 1 was studied by comparing the SMax values of Masonry shell between Wall 1 and Wall 1-a.

5.5.2.3 Wall 1 with Concrete Filled Web and Wall Aspect Ratio of 2.0 (Wall 1-c)

An analytical model was created to investigate the effect of wall aspect ratio on the proposed retrofitting technique for Wall 1. Model Wall 1-c was created to have a wall aspect ratio of 2.0 because of the increased wall height by a factor of 2.0. All other properties were identical to Wall 1-a. The force-drift response of Wall 1-c is presented in Figure 5.34 along with those for Wall 1 and Wall 1-a. The lateral load resistance was recorded to be 250 kN, which is about one half the load capacity of Wall 1-a. This was expected because of the increased lever arm.

Three points were selected on the force-deformation relationship to assess the behaviour of different materials used in the wall at different stages of loading. Point 1 represents the point at which initial stiffness of the wall starts changing and deviating from the initial elastic stiffness. Point 2 represents the peak load resistance and Point 3 represents a post-peak load stage at 1% lateral drift. Figure 5.35 shows steel stresses (S_{22}) in the longitudinal reinforcement. At Point 1, the tension steel at the base develops about 210 MPa. Stresses increase as Point 2 is approached, and the tension steel at Point 2 develops about 480 MPa, indicating yielding. During the decay segment, stresses in steel start decreasing as the load resistance starts dropping. Tensile stresses in concrete, masonry and mortar indicate cracking at Point 1 with propagation of cracks towards the wall toe as Point 2 is approached. These stress contours are shown in Appendix III. Figure 5.36 illustrates the variation of maximum compressive stress of concrete in Wall 1-c, indicating stresses in excess of 30 MPa near the wall toe at Point 2, with a reduction in maximum compressive stress during the strength decay region at Point 3.

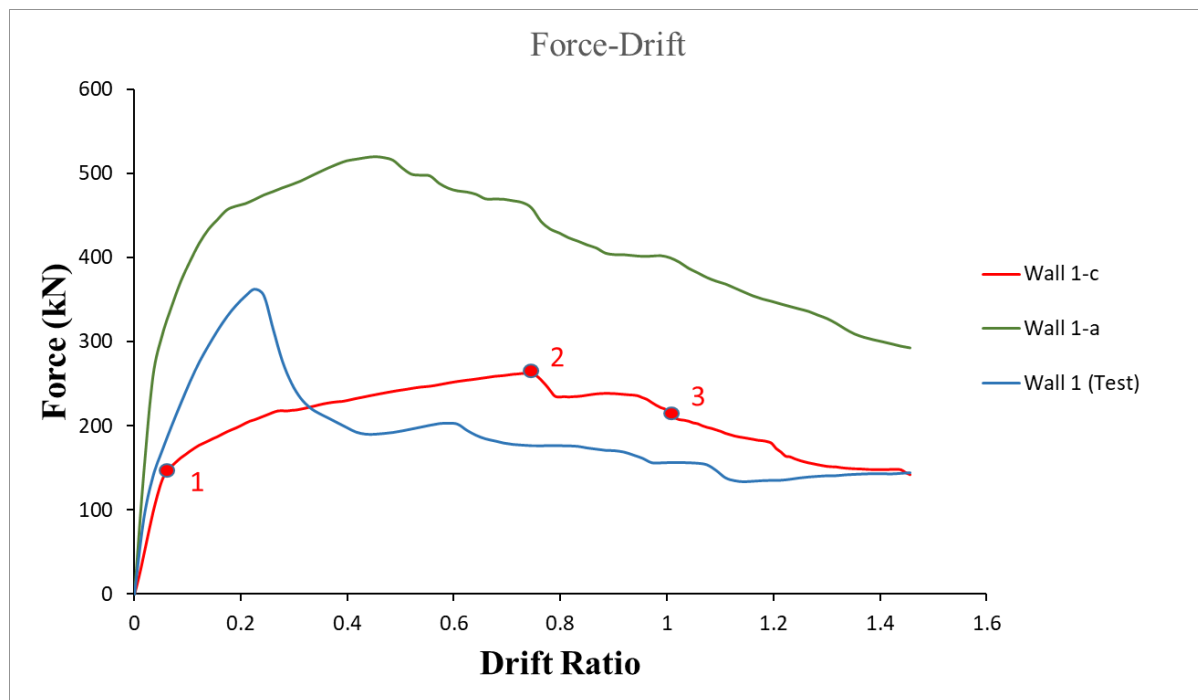


Figure 213.34 Comparison of Wall 1-c with Wall 1-a and Wall 1

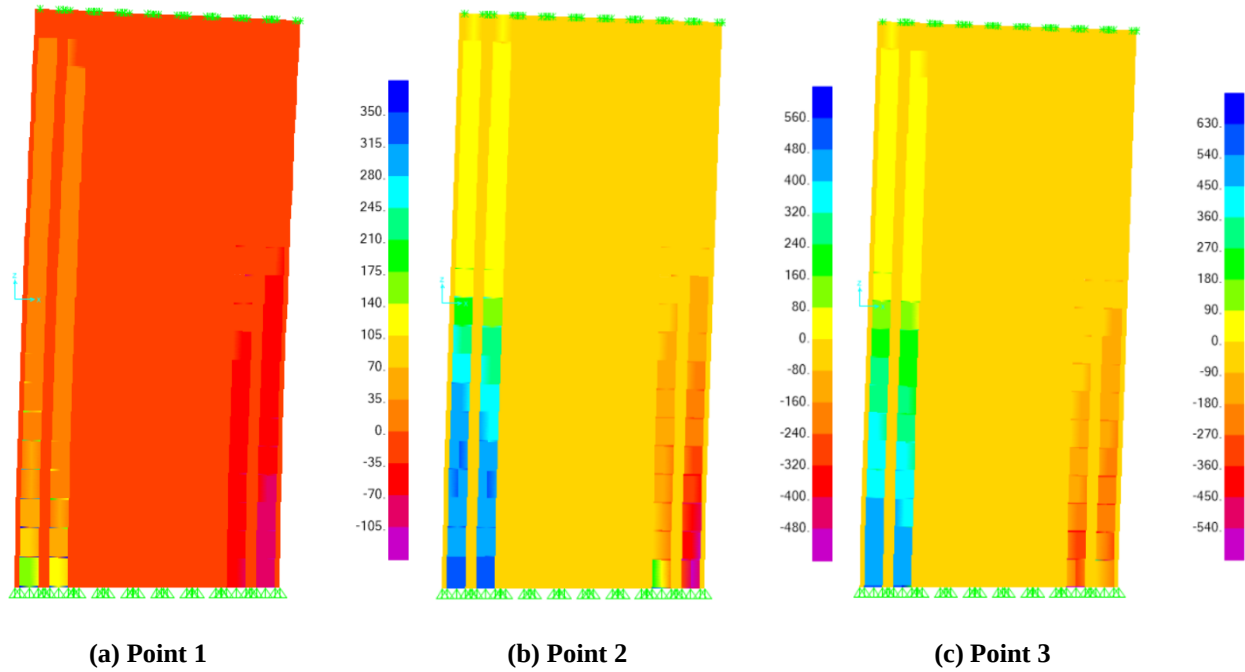


Figure 214.35 Steel stresses (S22) in Wall 1-c

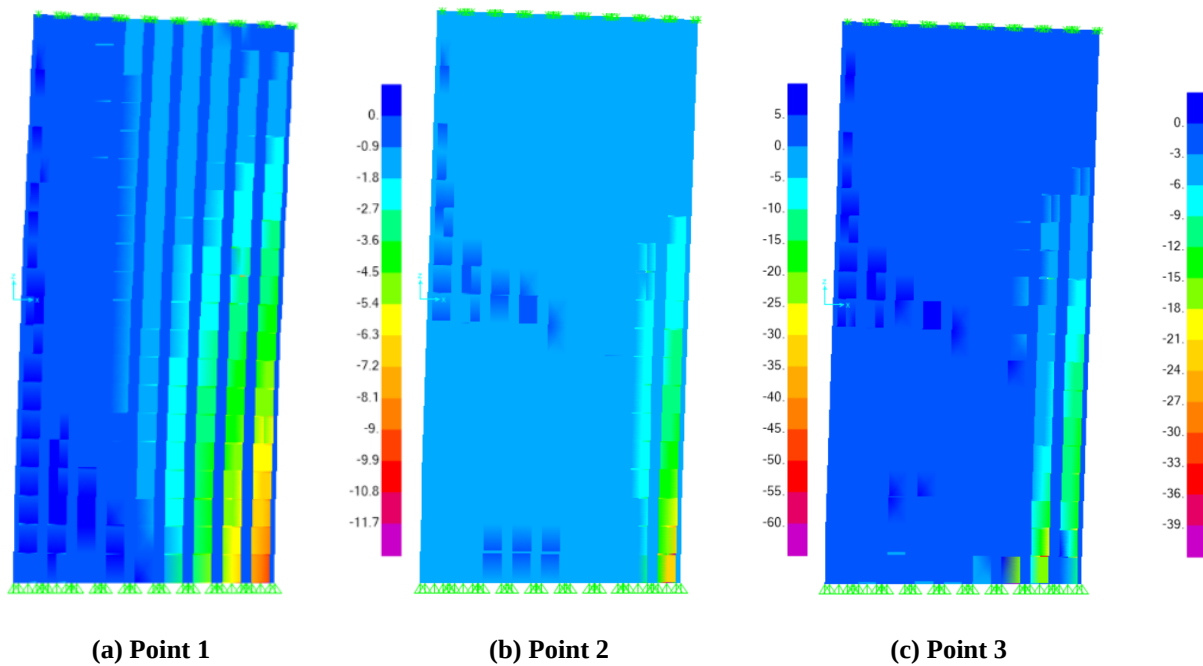


Figure 215.36 Maximum compressive stresses in concrete (SMin) of Wall 1-c

The S11 and S22 stress contours captured for horizontal and vertical FRP sheets, shown in Appendix III indicate maximum stresses ranging below 40 MPa in the horizontal direction and about 300 MPa in the vertical direction. These stress values are considered to be low relative to

the FRP strength, indicating that the behaviour was primarily governed by the yielding of steel and crushing of toe concrete.

5.5.3 Validation of Wall 2 Model

Wall 2 models were developed with due consideration given to the prestressing tendons. Three approaches were investigated to model the Tendon-HSS composite system used in Wall 2. The first approach involved the use of a built-in feature of SAP2000 for prestressing tendons. Two tendons were modeled, one on each side of the Wall 2, similar to the test specimen. A post-tensioning force of 75 kN was applied to each tendon. Figure 5.37 illustrates the force-drift primary curve for Wall 2 with the tendon elements. The figure also includes experimentally recorded hysteretic relationship. The initial stiffness of the wall was predicted well by the model, but the wall showed failure at 0.8% drift ratio. Wide range of modelling parameters were examined to resolve this issue, but none was successful. Consequently, it was decided to use shell elements to model the tendons, just like it was done for the other materials in the wall. This formed the second approach. The tendons were modeled similar to the steel reinforcement. Two point-loads (75 kN) were applied on top of the wall at the post-tensioning locations to introduce the effect of initial prestressing. Figure 5.38 shows the force-drift response of Wall 2 obtained using the second modelling approach. The initial stiffness of the wall was captured, while the stiffness of the model did not match the experimental results at higher displacements. The peak force resistance could be predicted reasonably well. However, the post-peak response of the model showed reduced load capacity as compared to the experimental results.

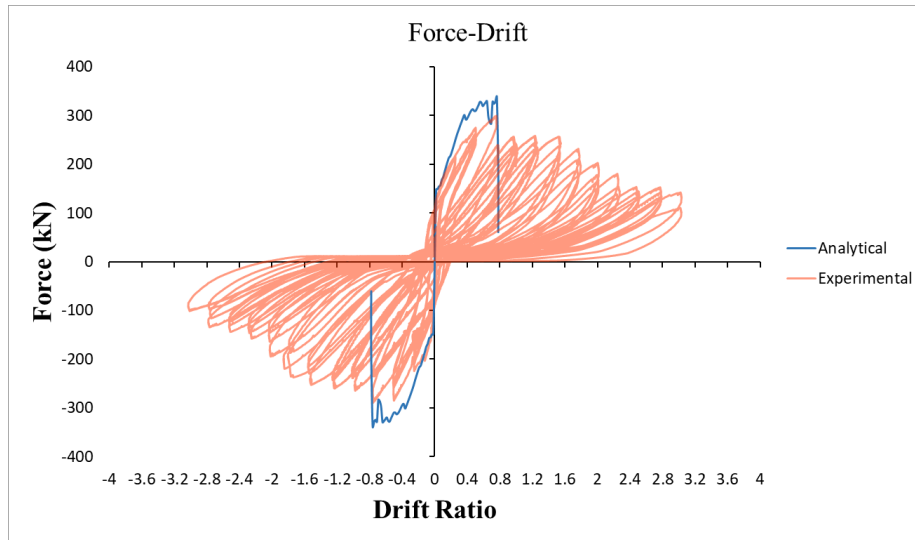


Figure 216.37 Force-drift response of Wall 2 modeled by tendon elements

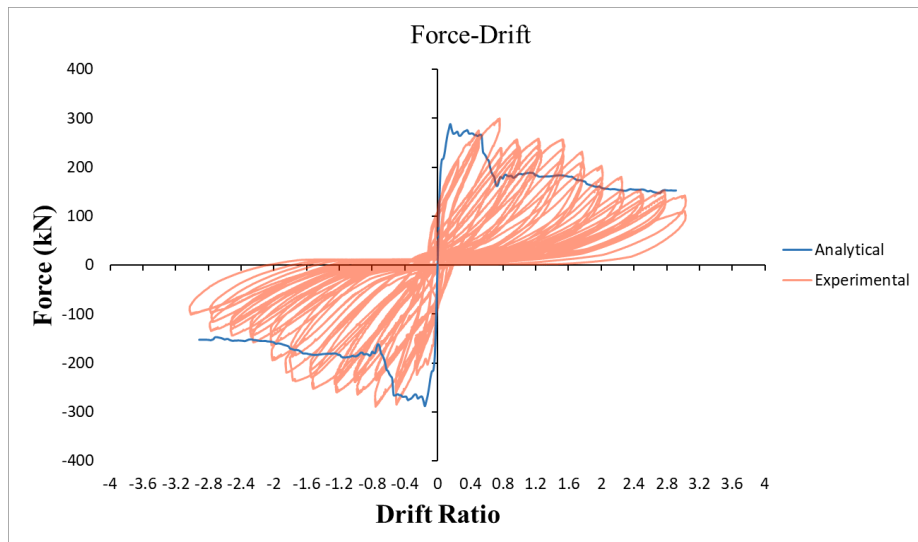


Figure 217.38 Force-drift response of Wall 2 modeled by tendon shells and axial point loads

It was experimentally observed that the tensile forces in tendons were changing when the wall was pushed during the test. When the wall was loaded from left to right, the tensile force in the left tendon increased with lateral displacement, while the tensile force in the right tendon decreased. This was expected as the tendon on the compression side would shorten and loose some of its initial prestressing. As the compression masonry was damaged, the compression side would shorten further. It was further observed that the steel HSS section used to prestress the strands started experiencing bending and inelastic deformations on the tension side. The tendon on the tension side (near the wall heel) was being stretched during loading, resulting in an increase in the initial prestress, while also losing its prestress because of the elastic and inelastic deformations of

the HSS section to which it was attached. The HSS section functioned as a “yielding agent,” dissipating energy and limiting the force in the tendon, thereby controlling excessive tension in the strand and protecting it against rupturing during higher levels of drift cycles. Indeed, the use of prestressing strands as diagonal bracing elements in an accompanying research program at the University of Ottawa indicated premature rupturing of the strands. Hence, it was decided to model the tendons along with the attached HSS units as a composite material. This required determination of the stress-strain relationship of the tendon-HSS composite element. The stress-strain relationship presented in Section 5.3.3 provides further information on the material model used to introduce the effects of internally placed tendons as retrofit methodology. This approach was used to validate the analytical model.

The force-drift response of the Wall 2 with composite tendon-HSS shell elements, without the axial point load, is shown at Figure 5.39. The initial stiffness provided by the model matches well the experimental behaviour, with some deviation during the second half of the elastic load stage. The peak lateral force capacity of the wall was captured well. The post-peak response of the model also provided good agreement with experimental results, considering the complications involved in the load resisting mechanism of a wall that consisted of concrete filled masonry blocks, mortar joints and prestressing tendons with steel HSS attachments. Consequently, it was decided to use this modeling approach for further investigation of Wall 2 to carry out a parametric investigation.

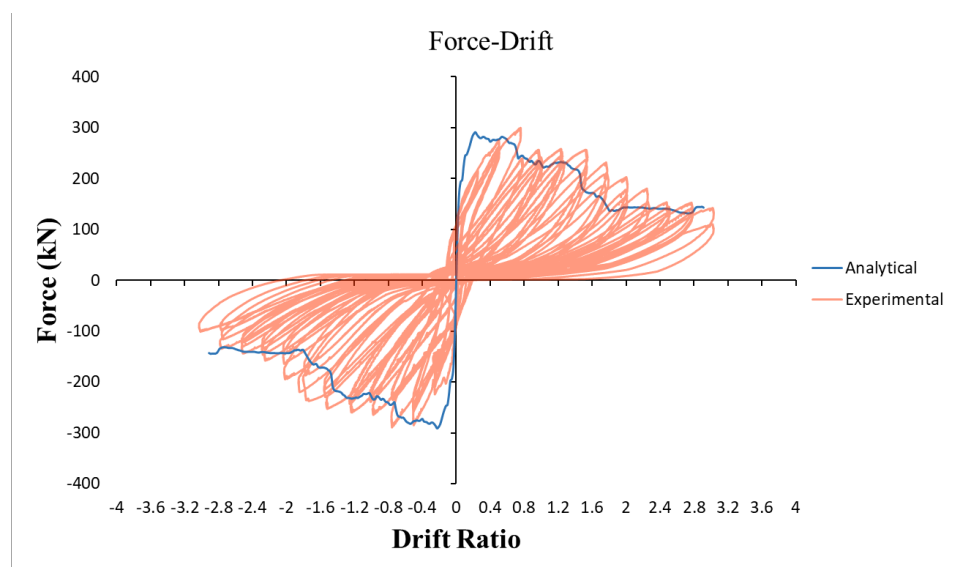


Figure 218.39 Force-drift response of Wall 2, modeled by tendon shells

The cyclic response of the analytical model for Wall 2 was also reproduced using SAP2000 for the first cycle of each of the initial three drift levels (corresponding to the 9-cycle range of cyclic loading). The excessive computational effort involved in reproducing the entire loading history was prohibitive. The cyclic analytical response of Wall 2, shown in Figure 5.40, indicates reasonably good match with test data, validating the analytical model.

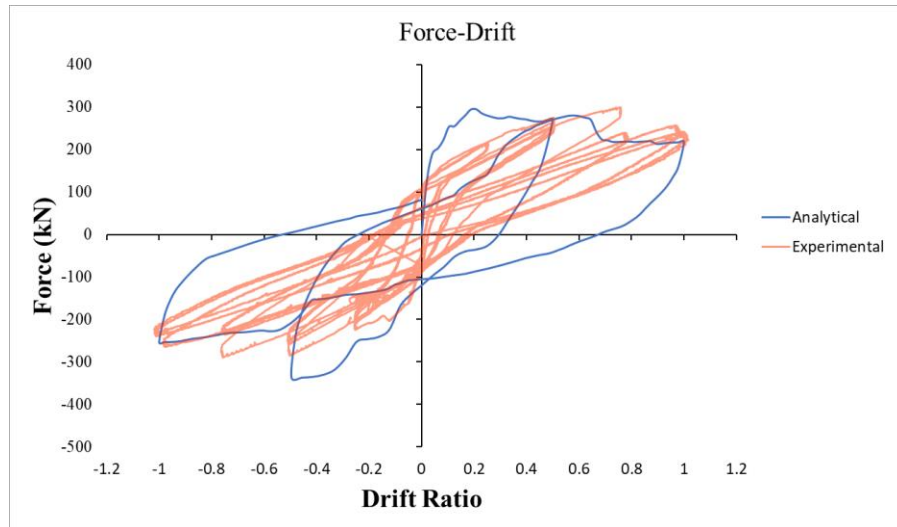


Figure 219.40 Cyclic response of the analytical model vs. experimental results

Stress contours for different materials that were used in the analytical model were captured at four different points on the pushover force-drift response of Wall 2. The selected four critical points are shown at Figure 5.41. Point 1 is the point at which the initial stiffness of the wall changes. Point 2 corresponds to the maximum load capacity of the wall. Point 3 and 4 indicate stress levels after the peak load at 250 kN and 150 kN load capacities, respectively.

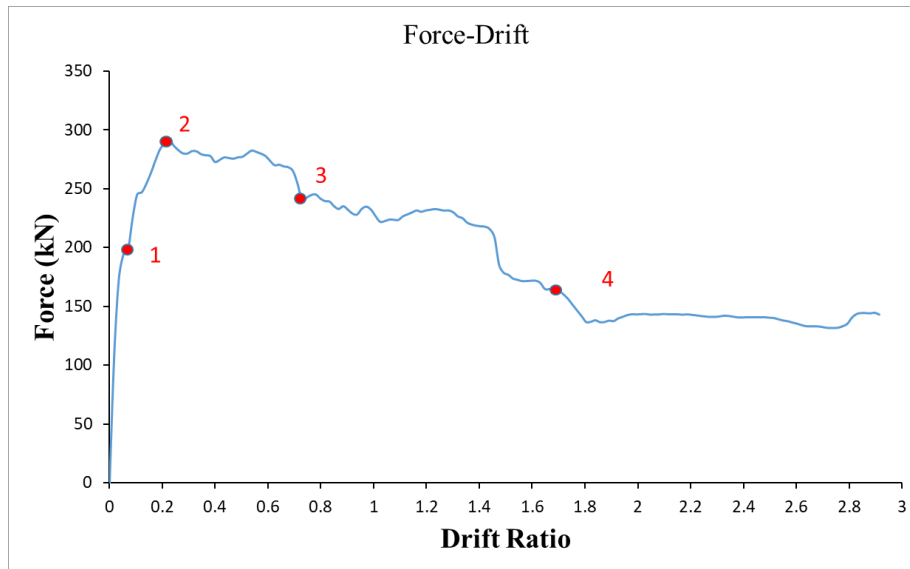
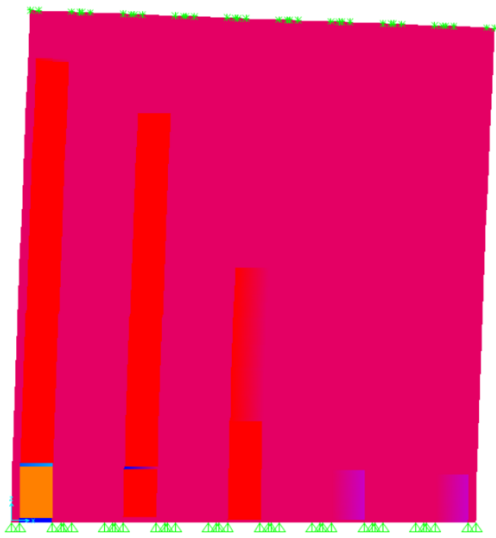
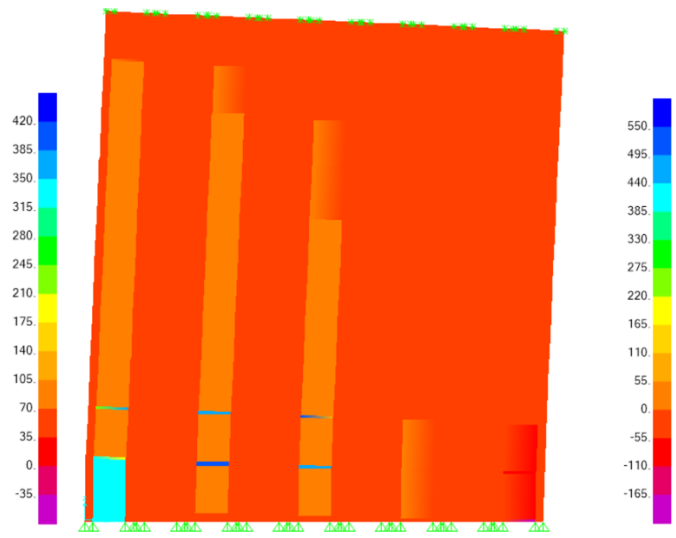


Figure 220.41 Pushover response of Wall 2

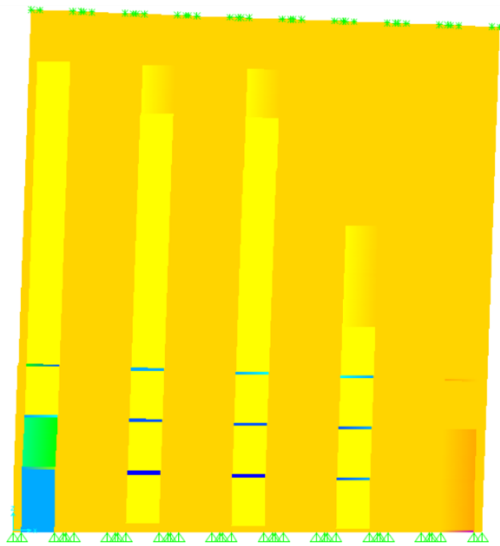
The tendon stresses obtained at the four control points are plotted in Figure 5.42. The figure also shows stresses in the 10M vertical bars placed in the web at three locations without anchorage to foundation for increased shear resistance. The tendon stresses increased going from Point 1 to Point 2, reaching the yield strength of the composite tendon-HSS shell at 430 MPa. This level of tensile stress in the heel region remained constant at Points 3 and 4 as stresses propagate towards higher locations along wall height. The stresses generated in the 10M re-bars were limited to about 100 MPa, except for a few locations at mortar joints where they developed yielding. Stress concentrations could be seen at mortar joint locations between masonry blocks. Maximum tensile and compressive stresses in concrete are plotted in Figure 5.43. Concrete cracking initiated in the wall heel region at Point 1, and it propagated towards the wall toe as the peak resistance was attained at Point 2. This also indicated shifting of the neutral axis. Maximum compressive stresses in concrete are shown in Figure 5.44. The highest compressive stress was recorded in the wall toe region. The maximum recorded compressive stress was 15 MPa, and 42 MPa at Points 1 and 2, respectively. Concrete compressive stresses were reduced at Points 3 and 4, which indicated crushing of concrete in these post-peak regions.



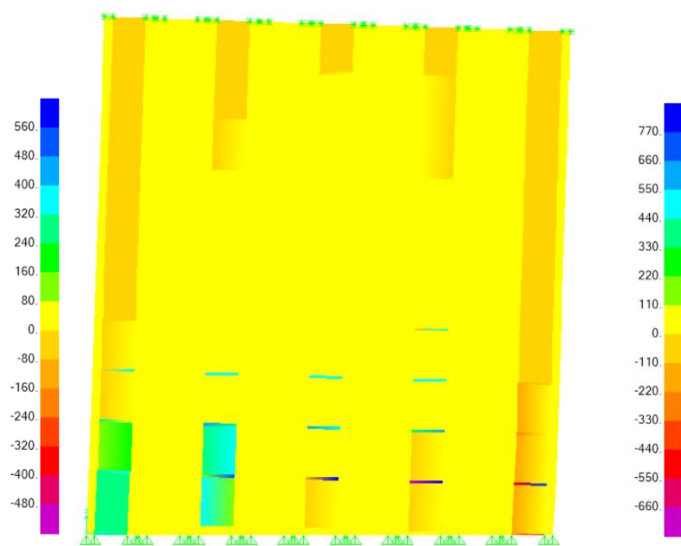
(a) Point 1



(b) Point 2

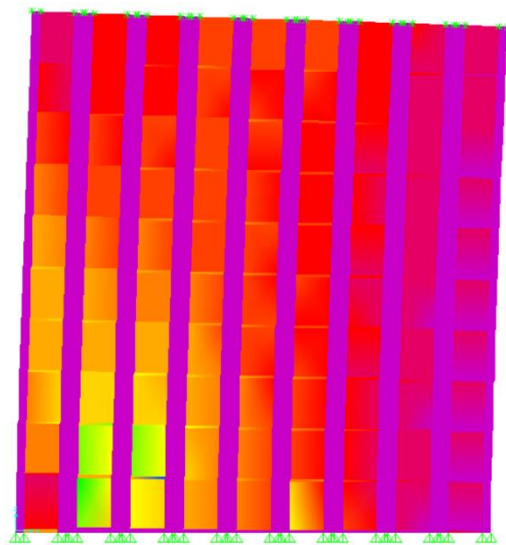


(c) Point 3

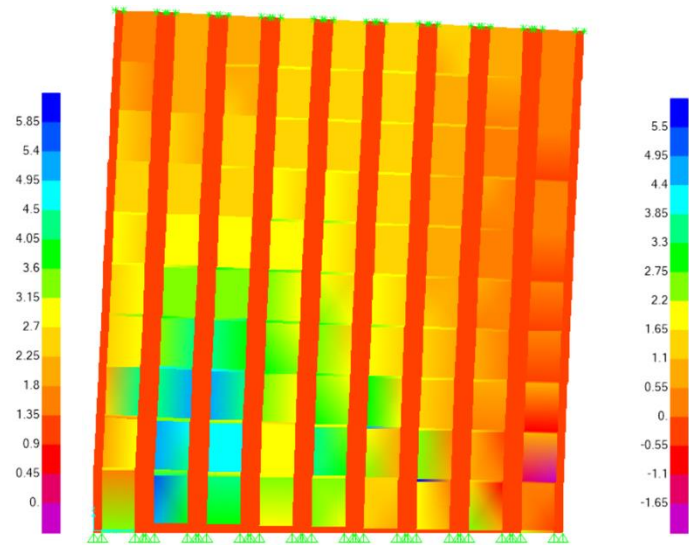


(d) Point 4

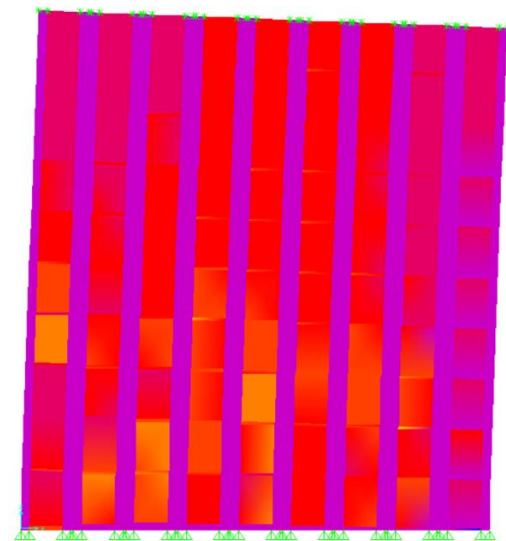
Figure 221.42 Tendon stresses, S22 in Wall 2



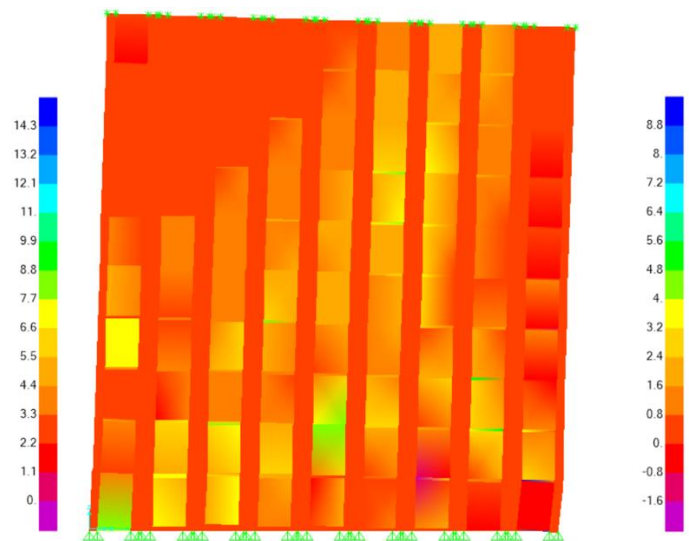
(a) Point 1



(b) Point 2



(c) Point 3



(d) Point 4

Figure 222.43 Concrete tensile stresses, SMax in Wall 2

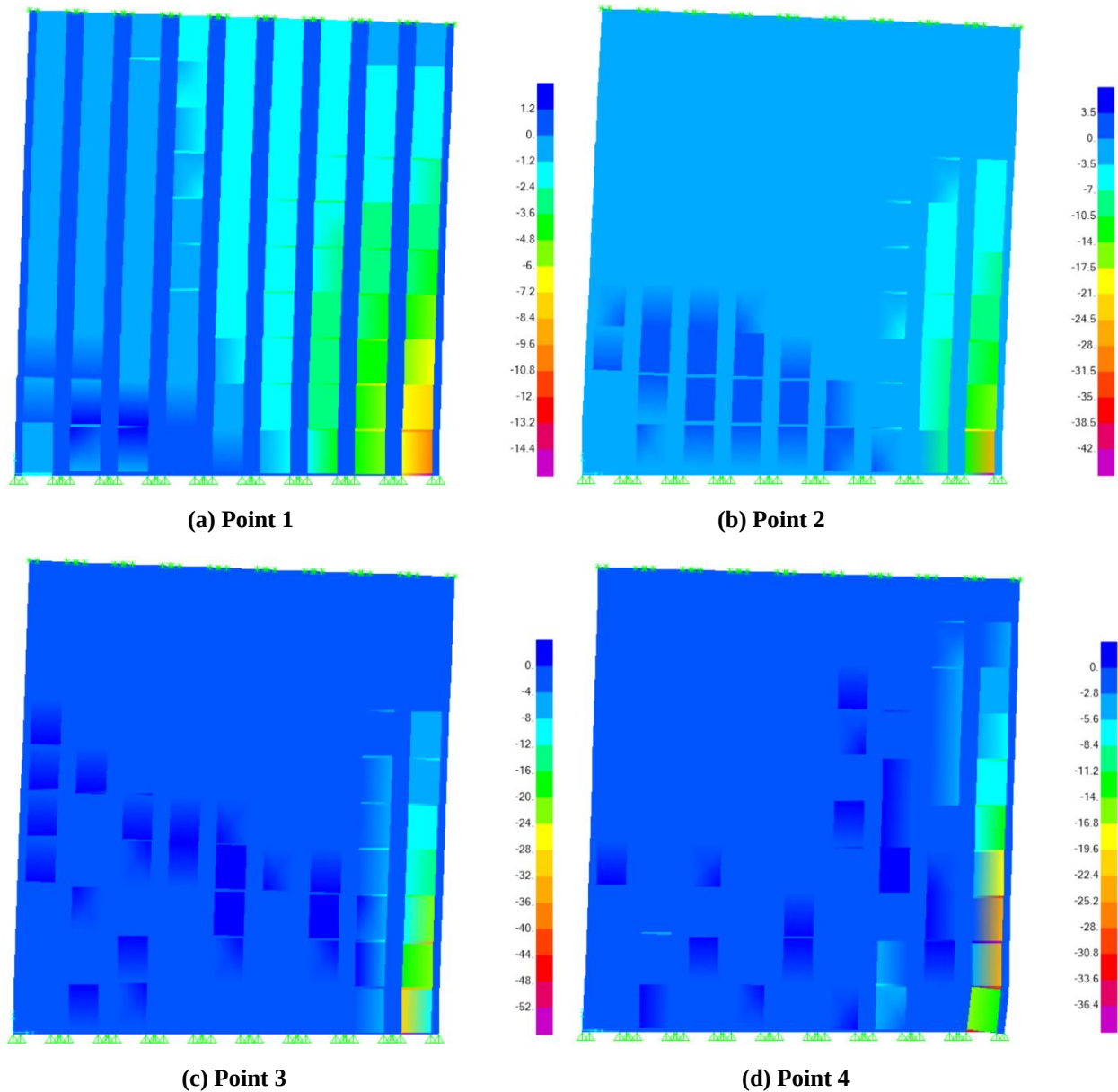


Figure 223.44 Concrete compressive stresses, SMin in Wall 2

Masonry stresses were also recorded at the same four points. They are shown in Figures 5.45 and 5.46 for maximum tensile and compressive stresses, respectively. As was in the case of concrete, masonry shells showed cracking at Point 1 and attained their maximum compression in the wall toe region. At Point 2, as the wall was loaded beyond its peak load resistance, the maximum tensile stresses shifted from the heel to the toe of the wall, signifying shifting of the neutral axis. The maximum compression in mortar was observed in the wall toe region, as expected. The maximum compressive stress in the wall toe region was 4.5 MPa, and 12 MPa at

Points 1 and 2, respectively. The toe region started crushing at Point 2 and the compressive stress values decline to 6 MPa at Point 4.

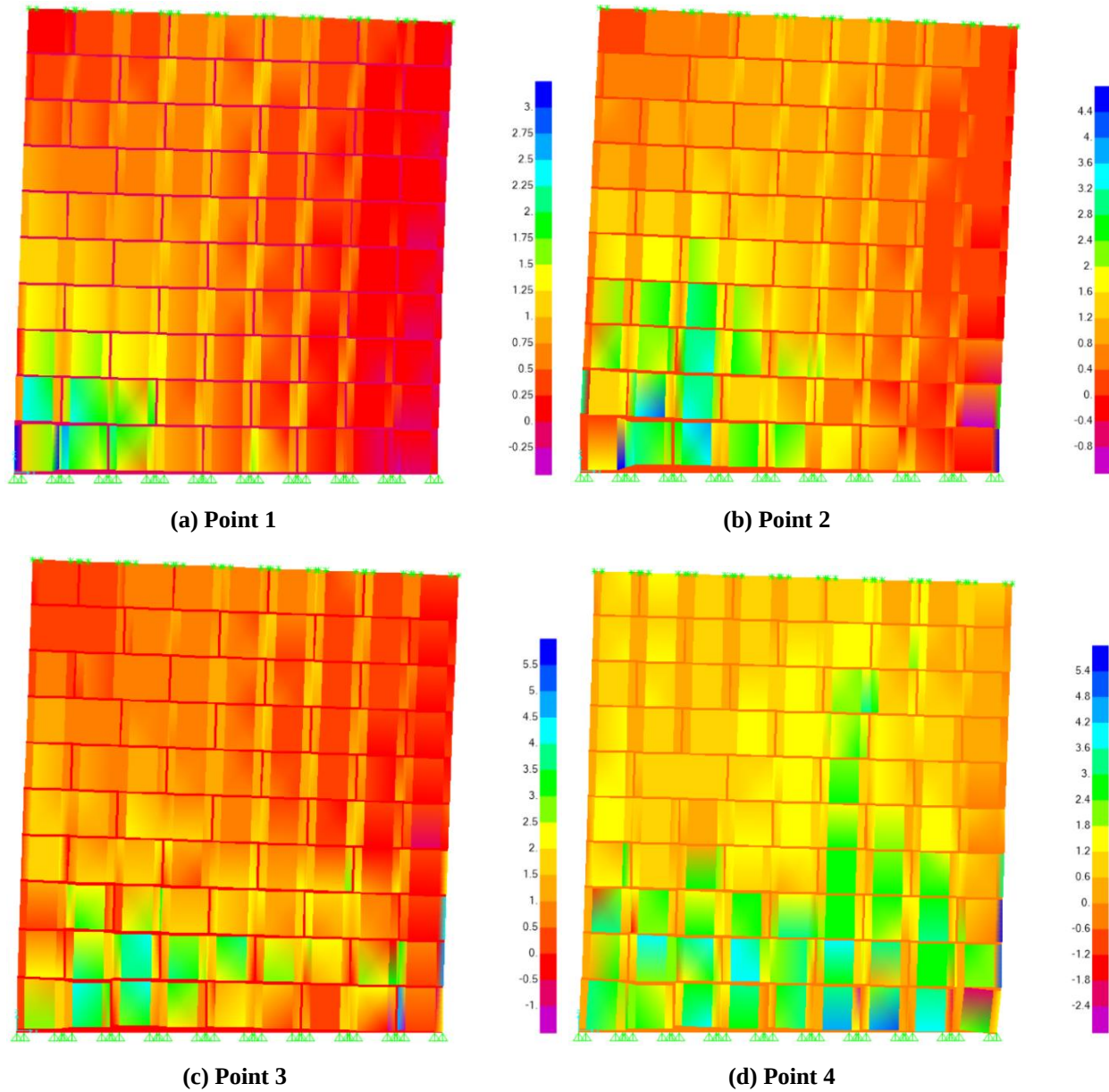


Figure 224.45 Masonry tensile stresses, SMax in Wall 2

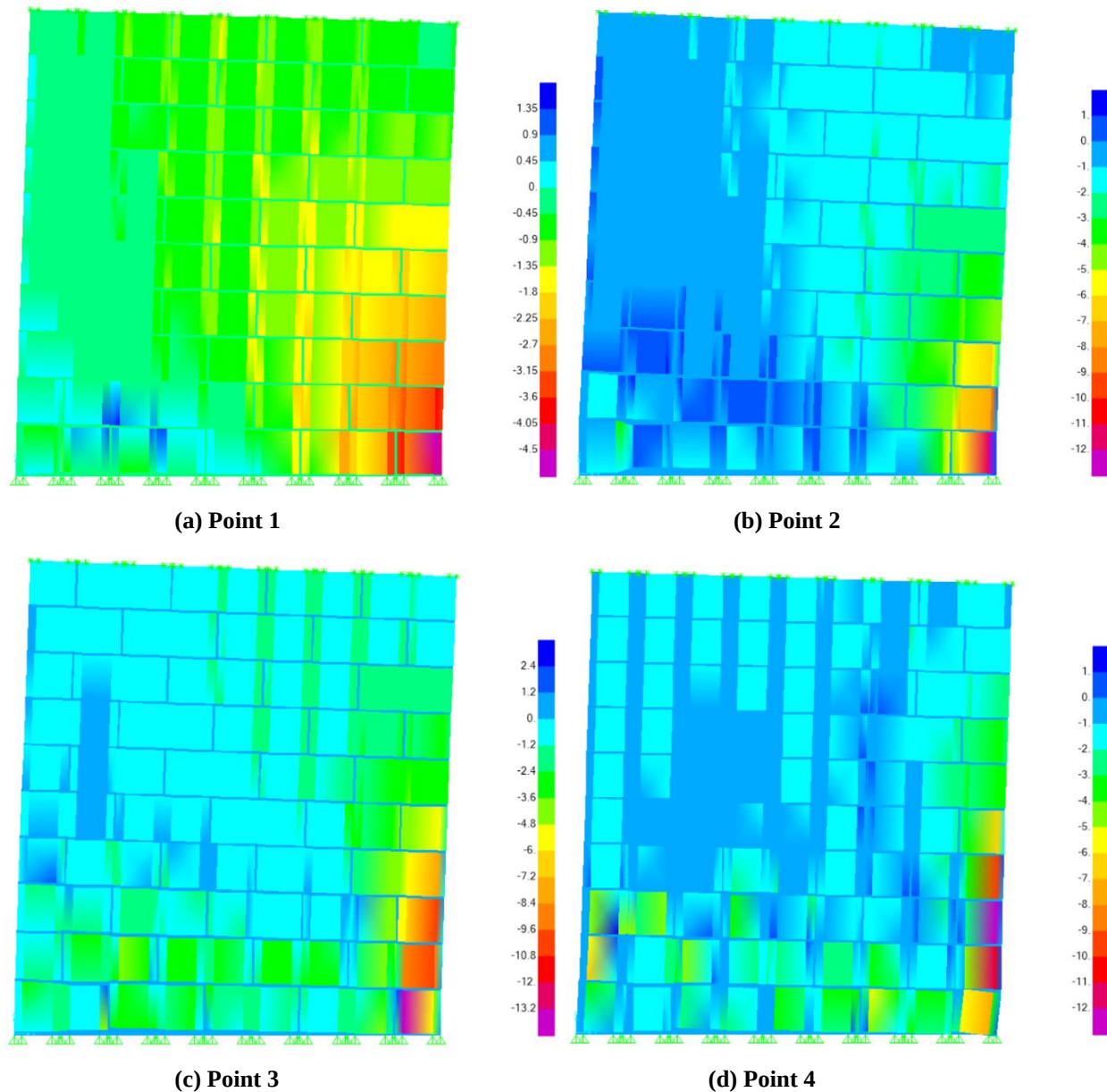


Figure 225.46 Masonry compressive stresses, SMin in Wall 2

Maximum tensile and compressive stresses in mortar shells are illustrated in Figures 5.47 and 5.48, respectively at the same four points on the force-deformation relationship. The highest tensile stress of 1.6 MPa was recorded in the wall heel region at Point 1, resulting in crack initiation in the mortar joint. As the lateral deflections increased, the cracking stresses propagated towards the toe. At Point 4, when significant strength decay was experienced, tensile stresses dropped to zero in cracked mortar joints. The highest compressive stresses were recorded in the toe region of the wall. The maximum compressive stress 6 MPa was recorded in mortar shells when the wall

reached its peak load resistance at Point 2. Subsequently these stresses decayed to zero at Point 3 when mortar in the joint crushed.

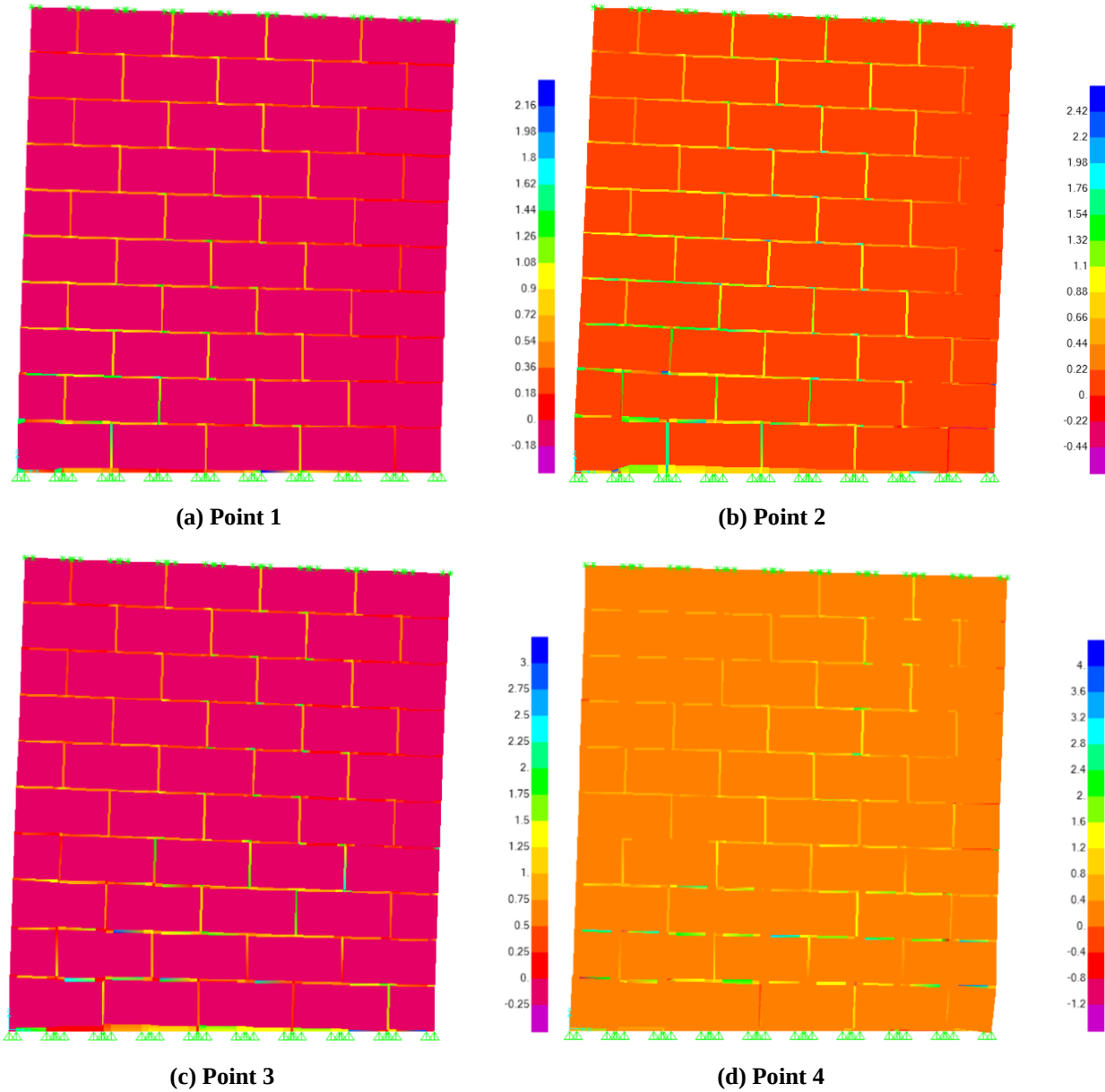


Figure 226.47 Mortar joint tensile stresses, SMax in Wall2

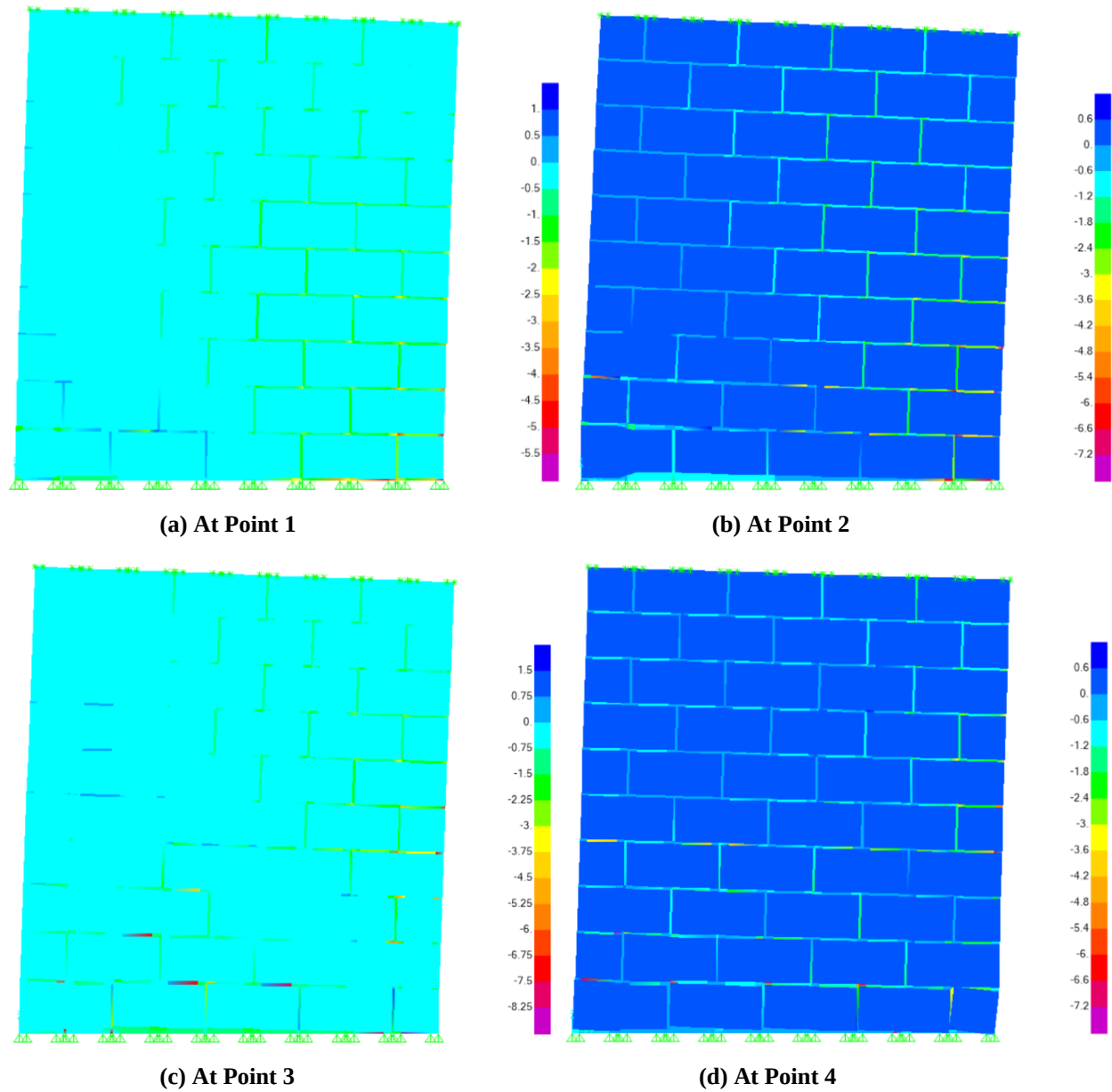


Figure 227.48 Mortar joint compressive stresses, SMin in Wall 2

The stresses in horizontal and vertical FRP sheets were also captured for Wall 2. Figures 5.49 and 5.50 show maximum horizontal and vertical FRP stresses recorded at the previously selected four points on the force-deformation relationship. The highest tensile stress in horizontal FRP took place across the vertical mortar joints, initially above the wall heel region (at Points 1 and 2) and subsequently spreading over the entire wall surface. The horizontal FRP stresses increased from 28 MPa at Point 1 to 176 MPa at Point 2. During the post-peak region, the stresses further increased to 275 MPa and 420 MPa at Points 3 and 4, respectively.

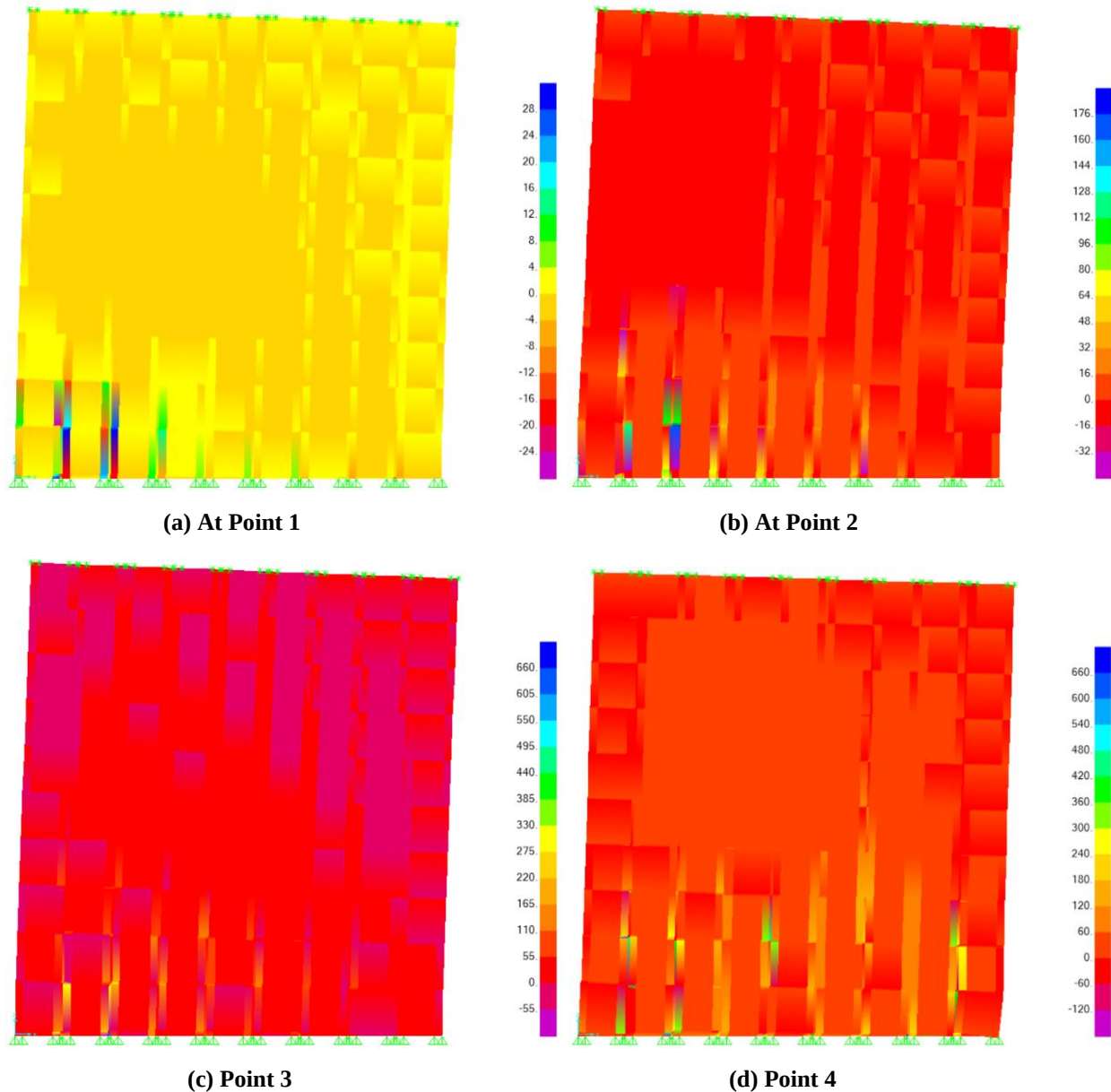


Figure 228.49 Horizontal FRP stresses, S_{11} in Wall 1

Vertical FRP was strained mostly in the tension boundary element region. At Point 1, the vertical FRP developed maximum tensile stress of 156 MPa across the mortar joint between the first and second rows of masonry blocks. At Point 2, when the wall reached its peak load resistance, the vertical FRP stress in the first three rows of masonry blocks reached 350 MPa. Subsequently the vertical FRP stress dropped to about 200 MPa during the strength decay region. The FRP in both directions were able to control diagonal tension. The two 10M internal vertical reinforcement also helped control diagonal tension. Web concrete provided resistance to diagonal compression. The wall was able to develop its full flexural capacity. These observations were consistent with

experimental observations.

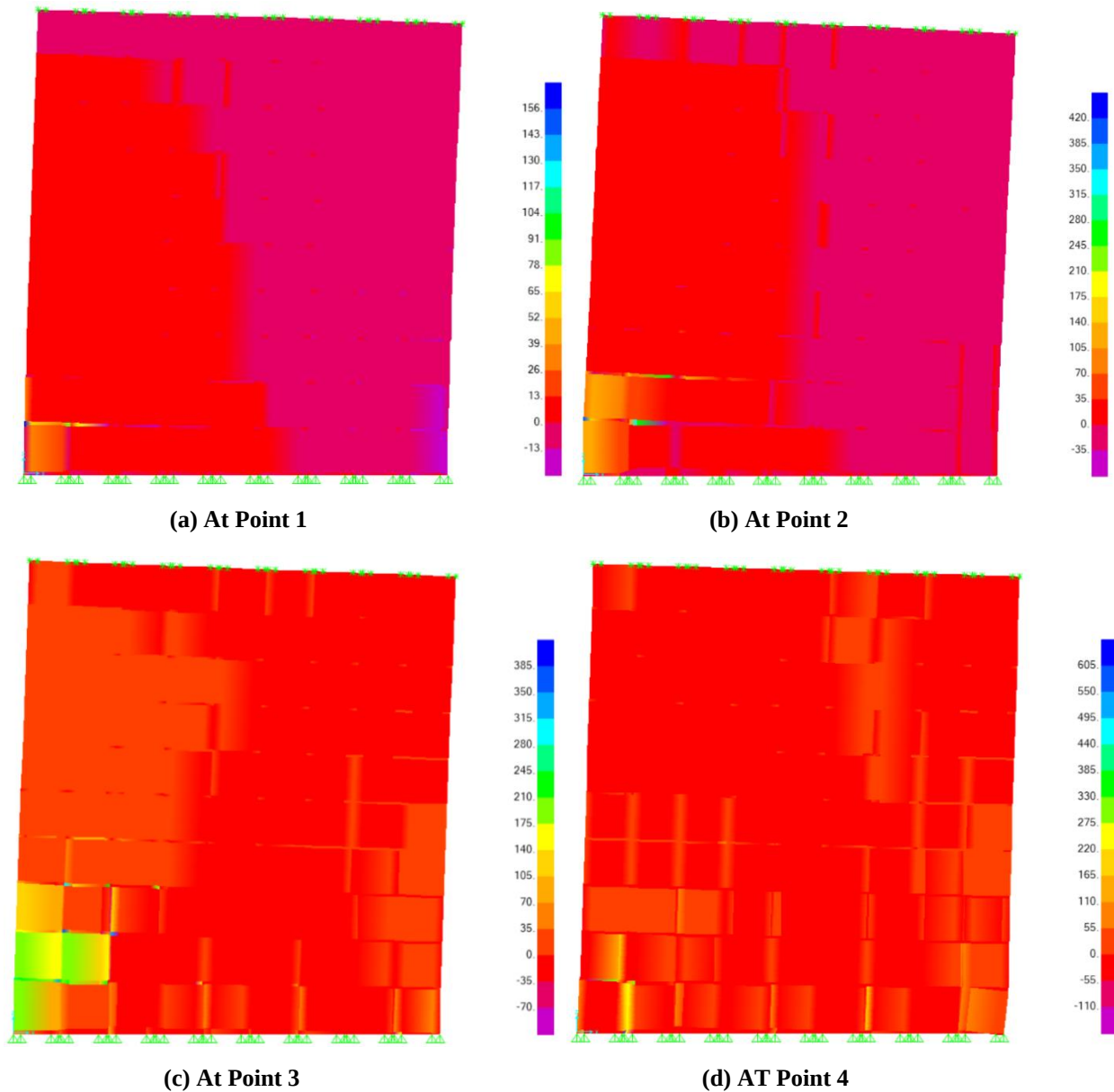


Figure 229.50 Vertical FRP stresses, S_{22} in Wall 2

5.5.4 Parametric Investigation of Wall 2

An analytical parametric investigation was carried out using the analytical model for Wall 2 that had been validated against the test data. The parameters considered included; i) the effect of HSS yielding agent, ii) area of prestressing tendons, and iii) wall aspect ratio. The following subsections provide discussions of observed behaviour. The overall comparison of the effects of

these design parameters, as well as the comparison with unretrofitted reference wall are provided in Chapter 6.

5.5.4.1 Wall 2 without the HSS Yielding Agents (Wall 2-a)

The presence of HSS yielding agents in Wall 2 had two major effects; limiting the force in the prestressing tendon to prevent steel rupturing, and to dissipate energy by yielding of the steel flanges of the HSS. Limiting tensile force in the tendon would limit flexural compression in the boundary element, thereby delaying the crushing of concrete. Consequently, HSS-Tendon composite system improved the ductility of the retrofitted wall. An analytical model (Wall 2-a) was developed to study the behaviour of the retrofitted masonry wall with regular 7-wire strands in boundary elements, without the HSS yielding agents. Figure 5.51 presents the force-drift response of Wall 2-a under pushover loading. Also behaviour of Wall 2 is indicated in Figure 5.51. When the HSS sections were excluded in the analysis, the wall resisted a higher lateral force, but showed a sharp drop in load capacity at about 0.6% drift. Subsequently the tendon ruptured shortly after 2% drift, as opposed to Wall 2, which sustained lateral load resistance, albeit after substantial strength decay.

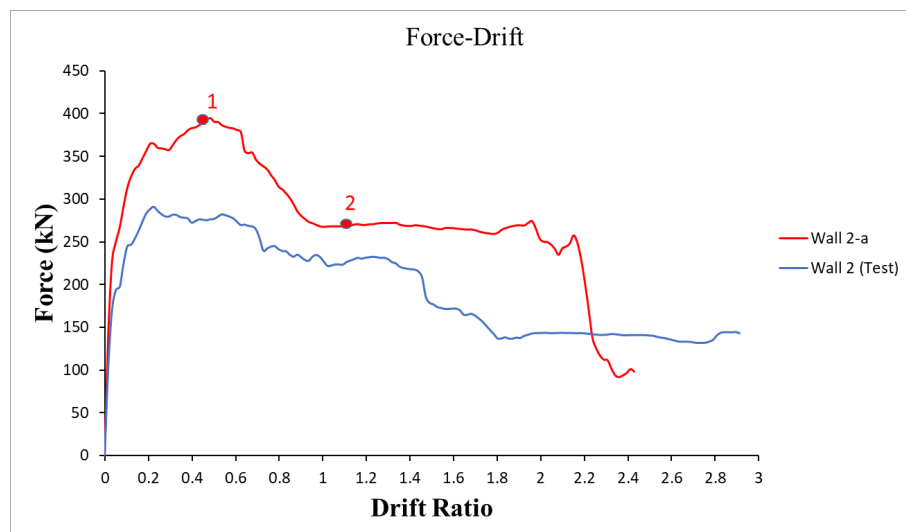


Figure 230.51 Pushover response of Wall 2-a and comparison with Wall 2

Stress contours for each material, recorded at two load stages (Points 1 and 2 in Figure 5.51), are presented in Appendix III. The maximum tensile stress of steel at Point 1 was about 1320 MPa, which increased to 1760 MPa at Point 2. The ultimate capacity of high-strength tendons was 1860 MPa. This was attained in Wall 2-a at about 2.2% drift ratio and led to failure. Maximum tensile and compressive stresses in Concrete, Masonry, and Mortar shells were observed to be

similar to those recorded in Wall 2, except for tensile stresses in Wall 2-a, which were approximately 55% higher than those measured in Wall 2. However, maximum compressive stresses were similar to those for Wall 2, but extended over a larger area to equilibrate increased tensile forces in the tendons of Wall 2-a. The recorded stresses in the FRP were about the same as those measured in Wall 2.

5.5.4.2 Wall 2 with Tendon Area Reduced to Half (Wall 2-b)

An analytical model was created for Wall 2 with the tendon area reduced to one-half of that used in Wall 2. This model was labelled as Wall 2-b and it was used to investigate the significance of the tendon size on response. Figure 5.52 shows the force-drift relationship of Wall 2-b along with that of Wall 2. The two relationships look similar. The lateral load capacity of Wall 2-b with reduced tendon area is about 40 kN less than that for Wall 2. However, as the drift level increased the force resistance for the two walls approached each other. Two reference points were selected on the force-deformation relationship shown in Figure 5.52, one at the peak load (Point 1) and the other shortly after the onset of strength decay (at Point 2) to present the stress counters recorded during the analysis. Recorded stresses in each material is displayed in Appendix III.

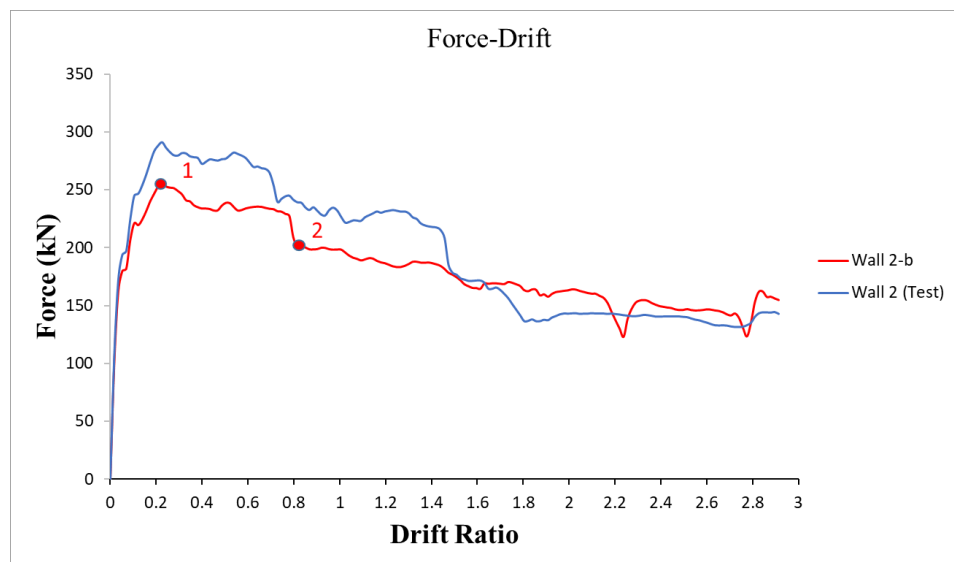


Figure 231.52 Pushover response of Wall 2-b and comparison with Wall 2

Tensile stresses in steel tendons and rebars (S22) at Point 1 was observed to be the same as those for Wall 2. This was expected because the same stress-strain relationship was used in the analysis for the tendon-HSS composite behaviour, though the force in each element would be different. The maximum tension in the heel region reached the tendon-HSS composite yield

capacity (450 MPa) when the peak resistance was attained at Point 2. The maximum tensile and compressive stress contours (SMax and SMin) recorded in Concrete, Masonry, and Mortar shells indicated a significant shift in the neutral axis towards the wall toe at Point 1. The failure of both Walls 2b and 2 were due to the compression crushing of concrete in the toe region. Concrete crushing initiated at Point 1, followed by masonry crushing at Point 2. The principal stresses in the web, as well as the horizontal and vertical FRP stresses were similar in both Walls 2-a and Wall 2.

5.5.4.3 Wall 2 with Aspect Ratio of Two (Wall 2-c)

The effect of wall aspect ratio was investigated by increasing the wall height by a factor of 2.0. This promoted flexural behaviour in the wall by limiting applied shear through reduced shear span. The analytical model with a wall aspect ratio of 2.0 was labelled as Wall 2-c. Increased aspect ratio generated larger deformations in tendons when the wall was loaded. This resulted in increased tensile stresses in tendons. It was decided not to prestress the tendons to take advantage of the higher tensile capacity available in the tendons. Therefore, yield capacity of 973 MPa was specified in the analytical model for the equivalent HSS-Tendon composite element.

The lateral force-drift relationship for the wall is shown at Figure 5.53. Three points were selected on the relationship to examine the material behaviour at these load stages. Point 1 is the point at which the initial wall stiffness showed a marked change. Point 2 corresponds to the maximum lateral load capacity of the wall, whereas Point 3 represents post-peak behaviour at about 1.2% lateral drift ratio. All the recorded stress counters for the wall are shown in Appendix III.

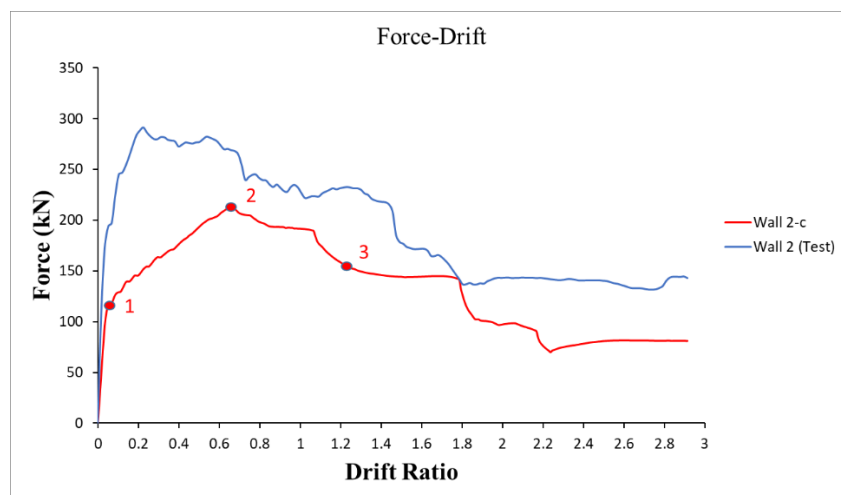


Figure 232.53 Pushover response of Wall 2-c and comparison with Wall 2

Tensile stresses (S22) in steel indicated that the tendon-HSS combination yielded on the tension side (wall heel) at Point 2. As post-peak deformations increased, yielding propagated towards the mid-wall height covering this region when Point 3 was reached. In reality the yield stress recorded refers to the yielding of the HSS yield agent, rather than the tendon attached to it. The central 10M web reinforcement also developed yielding at Point 2, first the bar on the tension side, followed by the second bar towards the compression toe of the wall. Tensile stresses in steel are illustrated in Figure 5.54.

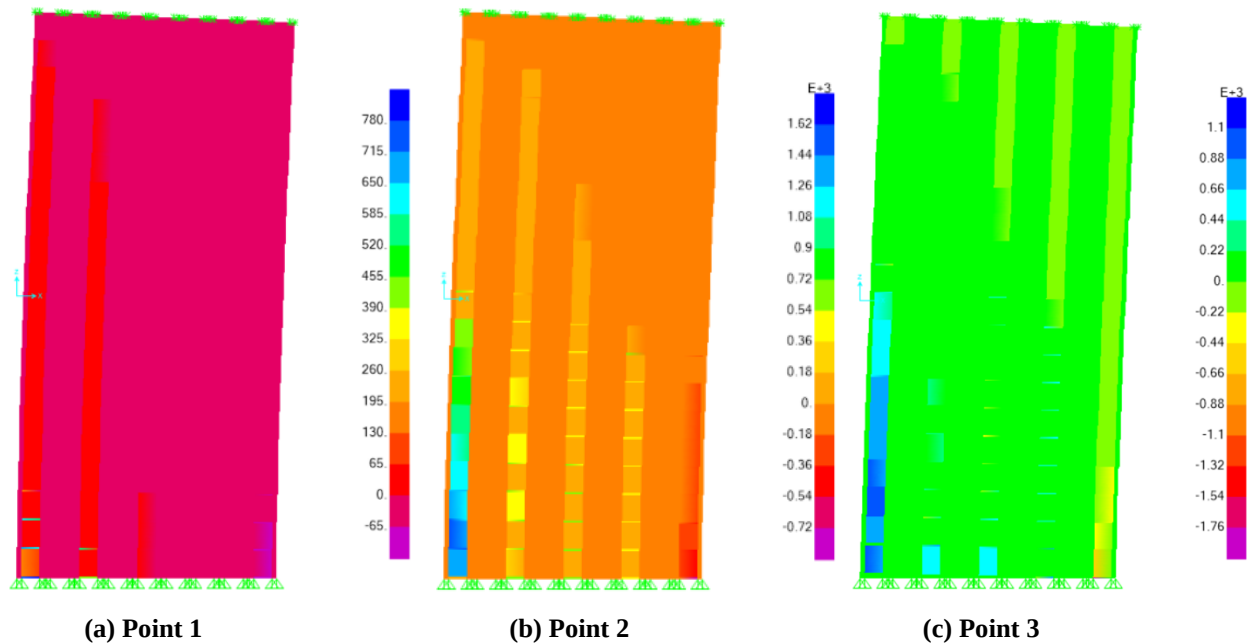


Figure 233.54 Maximum tensile stresses in steel shells, S22 (Wall 2-c)

Tensile cracking of concrete, masonry, and mortar occurred in the wall heel region at Point 1, as evidenced by SMax stresses recorded. Concrete and masonry in the toe region developed compression crushing at Point 3. Compressive stresses recorded at the three loading stages selected are shown in Figure 5.55 and 5.56. Principal stresses recorded on FRP (S11 and S22) indicated stress contours of the selected points were captured for horizontal and vertical FRP sheets. Similar to Wall 1-c, failure of the wall took place due to yielding of tension steel and crushing of concrete and the FRP sheets did not reach their ultimate tensile strength. Highest stresses were recorded at Point 3 (during the post-peak strength decay region). The majority of horizontal stress readings were within 85 MPa, though at few vertical mortar joint locations they reached up to 340 MPa. Similarly, vertical FRP stresses were relatively low, developing up to about 140 MPa tensile stresses in the heel region, but extended almost the entire height of the wall, with few locations

along the second mortar joint level reaching 420 MPa.

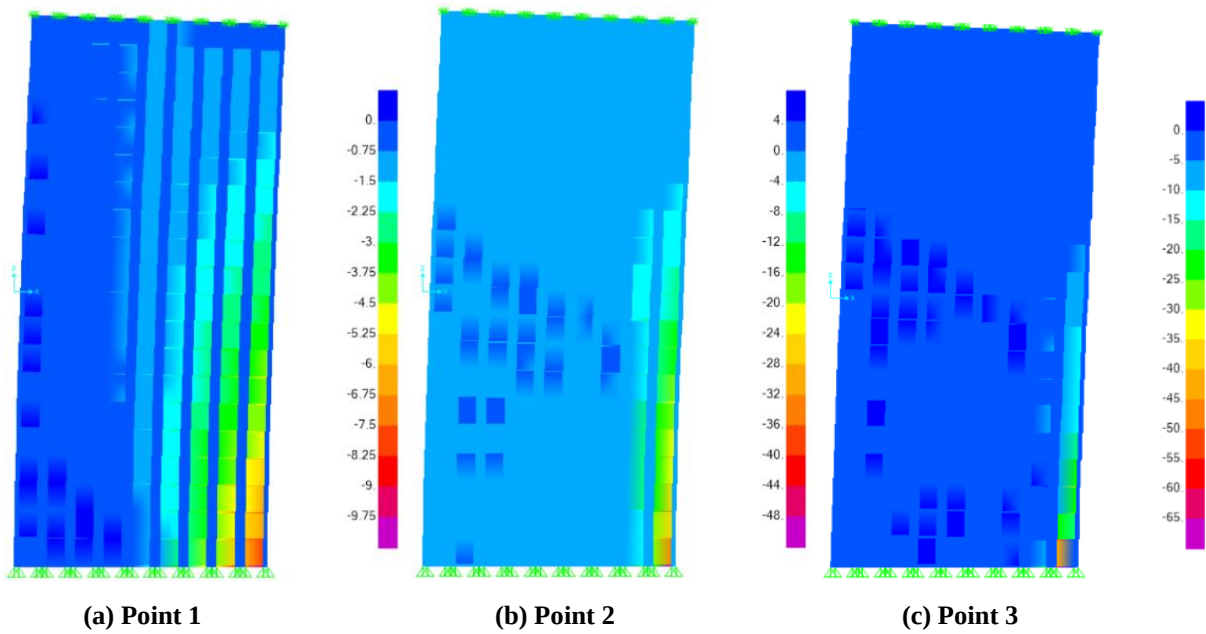


Figure 234.55 Maximum compressive stresses in concrete, SMin (Wall 2-c)

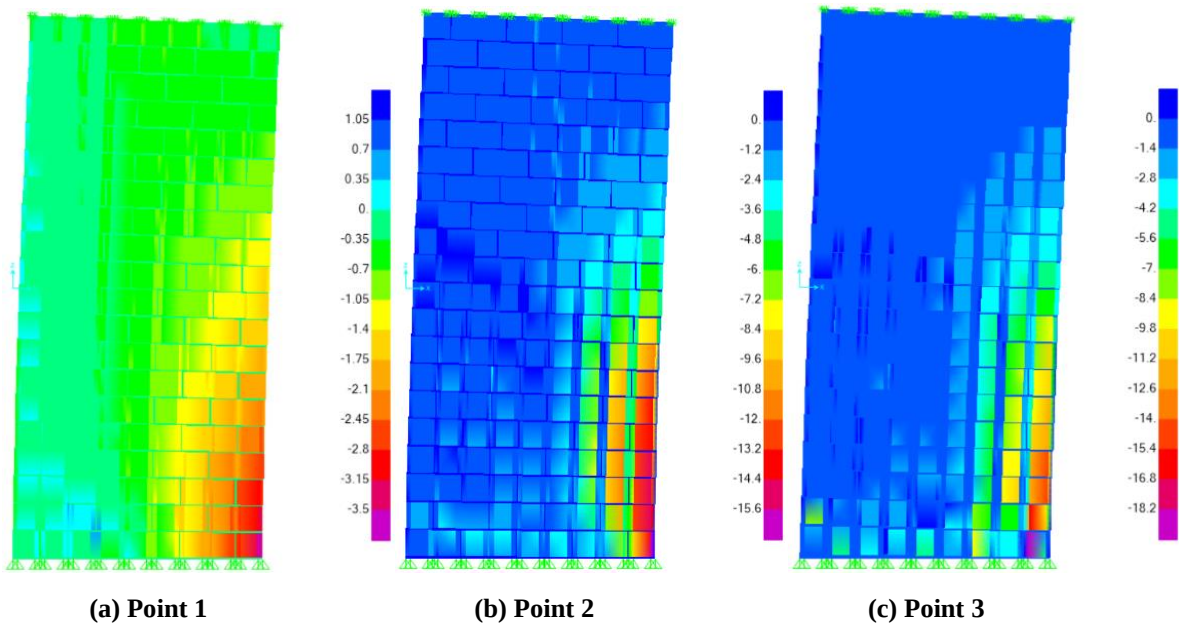


Figure 235.56 Maximum compressive stresses in masonry, SMin (Wall 2-c)

6 Comparison and Discussion

6.1 Introduction

The effectiveness of the two retrofit systems developed in the current investigation is assessed in this Chapter with design recommendations made for use in practice. The experimental results for retrofitted walls (Wall 1 and Wall 2) are compared with those of the un-retrofitted reference wall tested earlier by Taghdi et al. (2000). The results of the analytical parametric investigation conducted in Chapter 5 are also compared in this Chapter to assess the significance of each parameter. The comparisons of the experimental and analytical results provided sufficient insight into the effectiveness of the retrofit schemes and the significance of each design parameter, which led to the development of a design procedure.

6.2 Comparison of Experimental Results

Taghdi, Bruneau, and Saatcioglu (2000a) tested four concrete block masonry walls to develop a new seismic retrofit technique for low-rise masonry walls using steel strips. The reference as-built unreinforced masonry (URM) wall used in the current investigation was one of these walls. It was companion to the two retrofitted walls tested in the current investigation, with the same geometry and material properties, subjected to similar loading history. Figure 6.1(a) shows the geometric properties of the control wall. It was built using standard concrete blocks of 200 mm by 400 mm nominal size. The mortar joints were approximately 10 mm in thickness. The wall consisted of nine courses, resulting in a nominal height of 1800 mm. Type O mortar was used to represent an older masonry wall. Average compressive strength of the ungrouted and grouted masonry prisms were 12.5 MPa and 8.1 MPa, respectively. The wall was tested using three 1000 kN capacity MTS actuators. Two of the actuators were placed vertically to apply a total of 100 kN axial load, simulating gravity loads. The third actuator was placed horizontally to apply incrementally increasing lateral displacement reversals. Figure 6.1(b) shows the test setup.

The wall performed in a combination of rocking and sliding modes. The sliding occurred during the push cycles after the development of 64.5 kN of lateral force capacity. The rigid-body rocking occurred, with a small amount of sliding, during the pull cycles after developing an ultimate load capacity of -58.5 kN. The wall developed large lateral displacements due to sliding and rocking with little strength decay and energy dissipation. Figure 6.2 shows the cracking and

sliding experienced between the second and the third courses, while fully grouted base course remained intact. It was clear that lack of tension reinforcement limited flexural capacity and promoted rocking action, while lack of shear capacity promoted sliding shear. Figure 6.3 shows the force-drift response of the un-retrofitted as-built wall.

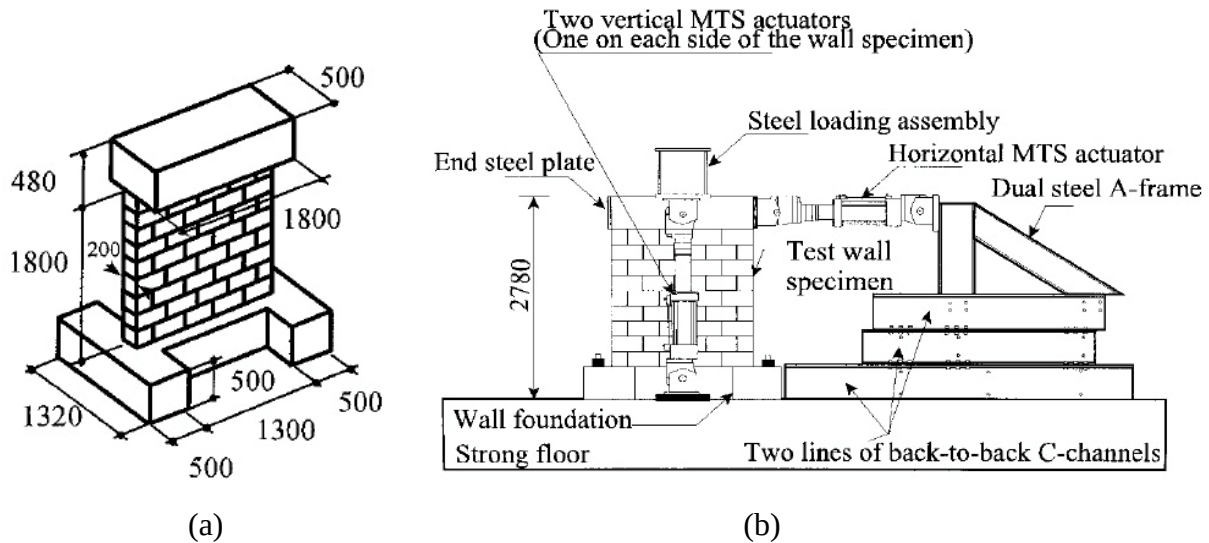


Figure 236.1 (a) Reference URM wall (b) Test setup used in testing the reference wall

(Taghdi et al. 2000a)

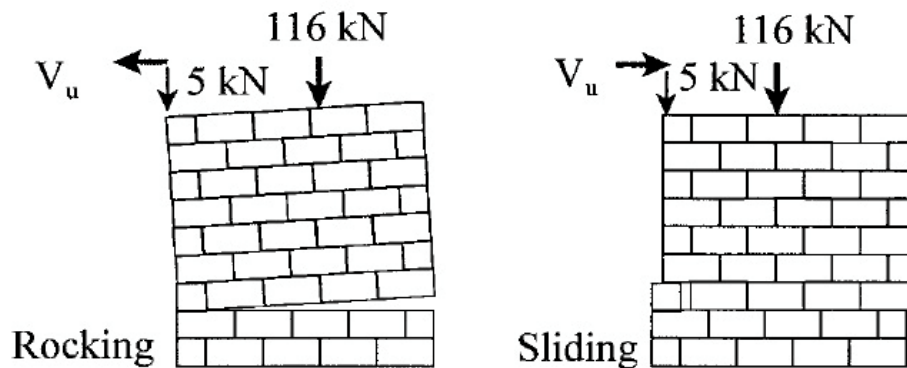


Figure 237.2 Behaviour of reference wall (Taghdi et al. 2000a)

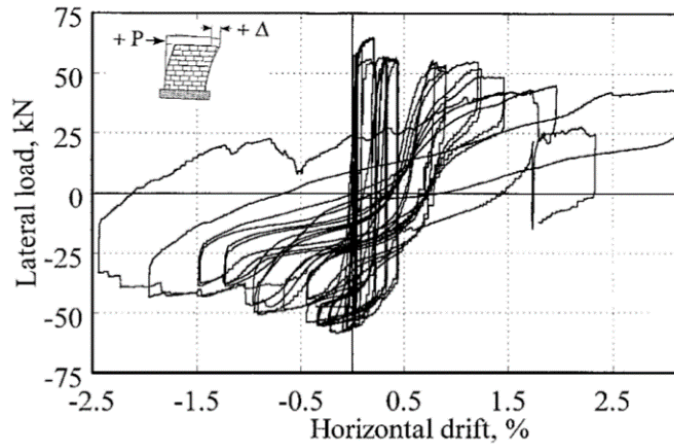


Figure 238.3 Load-drift response of the unreinforced masonry wall, tested by Taghdi et al. 2000a

The retrofitted walls were designed to introduce significant flexural capacity by providing vertical reinforcement, either in the form of re-bars placed at the ends of the walls to create boundary elements, or as prestressing strands with self-centering capabilities. CFRP sheets were surface bonded to control potential diagonal tension cracks while maintaining the integrity of the wall to develop the intended flexural capacity without rocking or sliding, and without the anticipated shear distress due to the low aspect ratio of the walls. The CFRP sheets were applied only on one side to simulate the practical requirements of the walls in practice, for which easy access to walls is usually available only on one side. The details of the retrofitted walls are covered in detail in Chapter 3 with test results reported in Chapter 4.

Figure 6.4 shows hysteretic lateral force-lateral drift relationship for Wall 1 with internally placed reinforcement. The implemented retrofit technique improved the in-plane lateral load capacity in excess of five fold. The mode of failure has changed from sliding and rocking to flexure and shear. The wall completed three cycles at 0.25% drift level, at which the internally placed reinforcement approached yielding. The flexural yielding took place during the first cycle at 0.5% drift ratio. The wall was erroneously subjected to higher drift ratio during the initial pull cycle, as explained in Chapter 4, but also developed its flexural capacity in the pull direction, as expected. While the strengthening objective of the retrofit technique was achieved, the wall started experiencing diagonal crushing due to shear during the second cycle at 0.5% drift. This behaviour limited wall ductility. It became clear that the URM wall, with empty cells in the web did not have the diagonal compression capacity associated with flexural strength, in spite of the enhanced diagonal tension capacity. This shortcoming of the retrofit technique was addressed in Wall 2 by

filling the hollow central blocks with concrete.

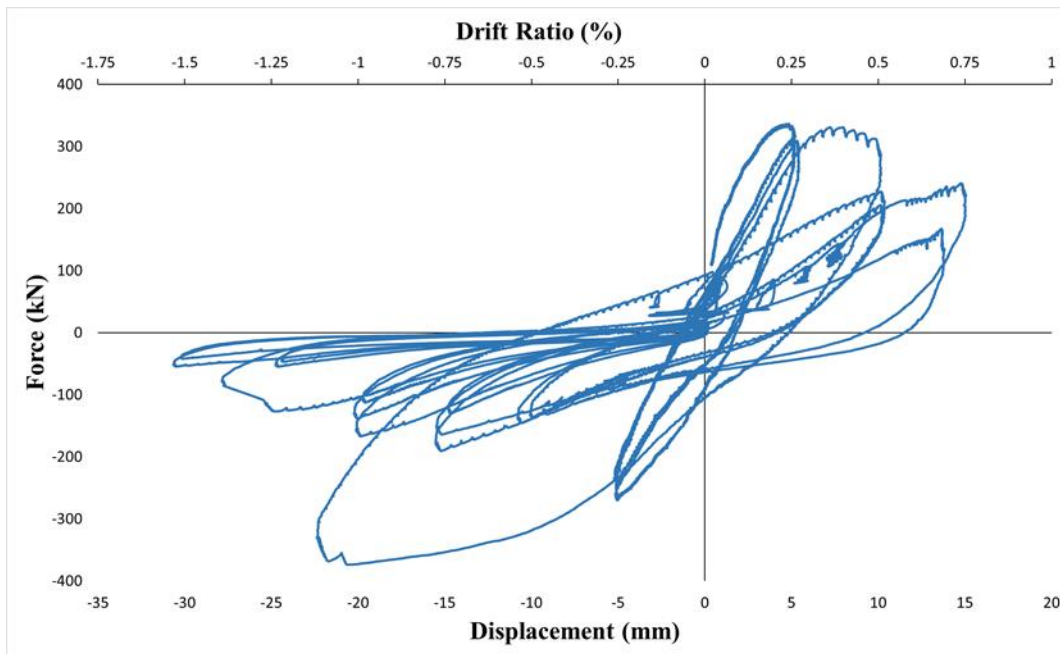


Figure 239.4 Wall 1 force-drift response

Figure 6.5 shows the hysteretic relationship of Wall 2, with end prestressing strands. Similar to Wall 1, in-plane load capacity increased by 4.7 times, relative to the reference wall. Moreover, considerable improvement in displacement ductility was observed. The wall reached its full lateral load capacity at 0.75% lateral drift and experienced gradual strength decay as the compression concrete at the toe started crushing gradually. However, the wall maintained 80% of its capacity until after 1.5% drift cycles were applied.

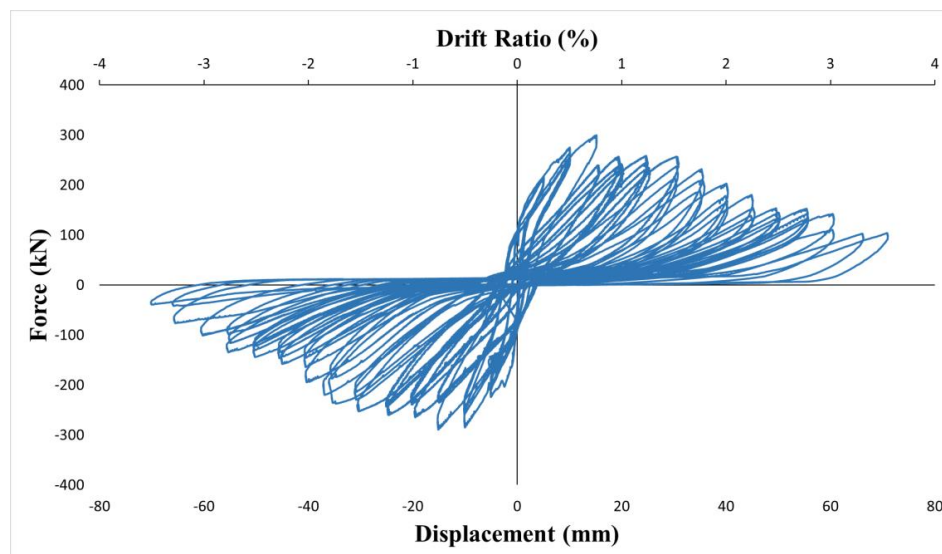


Figure 240.5 Wall 2 force-drift response

Closer look at wall behaviour revealed that the prestressing strands remained elastic, although the wall resistance levelled off at about 0.75% lateral drift. It was further observed that the initial loading exhibited softer response near the peak, in spite of the elastic behaviour of the strands. This is attributed to the behaviour of the HSS sections attached to the strands at the top of the wall, which served as yielding devices, controlling maximum force in the strands while also elastically deflecting prior to yielding. This behaviour helped the strands remain elastic, and provide a self-centering mechanism. Indeed, the hysteresis loops shown in Figure 6.5 indicate that the unloading branches pass through the origin because of the pulling action of the strands. The softening of the HSS yielding devices also limited flexural compression, as well as diagonal compression, thereby preventing concrete from crushing. These observations are verified through the finite element analysis conducted in Chapter 5 and provide insight into the behaviour of the retrofitted wall, which prove invaluable in developing a design procedure.

6.3 Comparison of Analytical Results

The experimental research provided proof of concept but was limited to two walls with specific retrofit schemes. Therefore, an analytical parametric study was conducted and reported in Chapter 5 to expand the experimental observations in an effort to develop a generalized retrofit design procedure. Upon developing analytical models and validating them against the test data, the effects of design variables were investigated.

6.3.1 Wall 1 Parameters and Comparisons

The following design parameters were investigated for Wall 1 in Chapter 5, and their affects are compared in Figure 6.6.

- The effect of filling web masonry blocks with concrete.
- The effect of increased boundary element reinforcement.
- The effect of increased wall aspect ratio.

Wall 1 achieved the objective of attaining flexural strength enhancement by a factor in excess of 5.0. This is a significant step forward for the strength enhancement of the wall, which would otherwise suffer from low seismic force resistance triggered by rocking and/or sliding behaviour as observed in the unretrofitted reference wall. The flexural resistance was in line with

the percentage of boundary element reinforcement provided as tension ties, well anchored into the foundation. However, the wall experienced shear failure shortly after developing its peak flexural resistance due to the crushing of masonry units in the web region under high levels of diagonal compression struts. The wall web was clearly deficient in resisting diagonal compression because of the empty cells, further damaged by diagonal tension cracks that were controlled by the CFRP sheets only on one side of the wall. Crisscrossing of diagonal tension cracks reduced compression capacity of thin masonry wall cells, resulting in a rapid decay of strength.

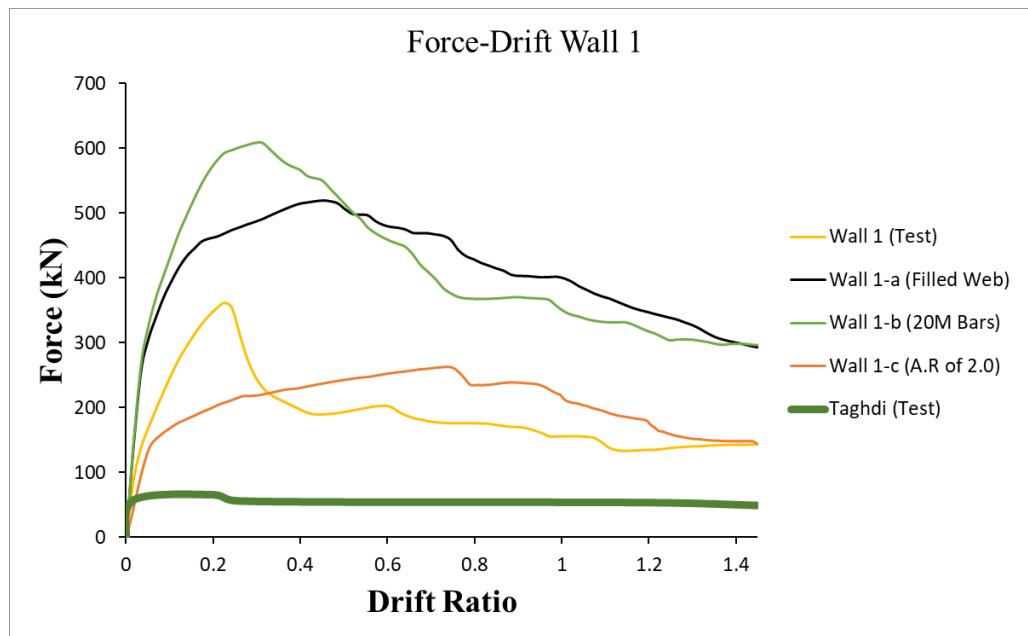


Figure 241.6 Comparisons of analyses results for Wall 1

The effectiveness of web filled with concrete was investigated analytically and discussed in Chapter 5. The comparisons of analyses results shown in Figure 6.6 indicates a substantial improvement in wall behaviour because of the wall web strengthening by internally placed concrete. The initial wall rigidity was increased, as expected, and the wall deformability improved from 0.25% lateral drift to 0.5% drift, followed by a more gradual strength decay. The attempts to further improve the wall behaviour by increasing the percentage of boundary element reinforcement generated a higher lateral force capacity, as expected and as discussed in Chapter 5, but did not improve wall deformability. The wall experienced higher shear distress under increased flexural reversals associated with increased flexural reinforcement and flexural capacity, hence the crushing of concrete.

The comparison of the previous three cases indicates that in low-rise walls, while providing

internal flexural reinforcement results in a very significant enhancement in seismic force resistance relative to the unretrofitted URM wall, the ductility (deformability) can be improved in a limited way until the crushing of concrete occurs. The resulting improvement in wall displacement ductility ratio is approximately equal to 2.0. This level of ductility corresponds to “moderate ductility” in the National Building Code of Canada (NBCC 2015) with a ductility-related force modification factor (R_d) of 2.0.

The analysis was extended to cover a wall with a higher aspect ratio (aspect ratio of 2.0). The wall was retrofitted with internally placed boundary element reinforcement, concrete filled masonry cells, and surface-bonded CFRP sheets on one side. The increase in the aspect ratio promoted flexural response with reduced flexural and shear demands because of the increased level arm. The force-displacement response shown in Figure 6.6 indicates ductile flexure dominant response with little strength loss up to 1.0% drift. This is a significant improvement in wall behaviour.

6.3.2 Wall 2 Parameters and Comparisons

The following design parameters were investigated for Wall 2 in Chapter 5, and their affects are compared in Figure 6.7.

- The effect of prestressing without the yielding agents.
- The effect of reduced area of prestressing strands.
- The effect of increased wall aspect ratio.

Wall 2, with prestressed strands and concrete filled masonry units performed well during the test and developed strength and ductility enhancements. The lateral force capacity was governed by the yielding of the HSS yielding agents, while the prestressing strands remained elastic, acting as self-centering restoring elements. The role of the HSS yielding agents was important in controlling the wall behaviour. Bending and subsequent yielding of the flanges of the HSS sections limited the tension force in the prestressing strands. This in turned, prevented yielding of the strands and promoted their elastic self centering capabilities. It also limited compression forces in concrete struts, both in flexural and diagonal struts, preventing early crushing of concrete. The combined reduction in tension ties and compression struts enhanced the ductility of the wall.

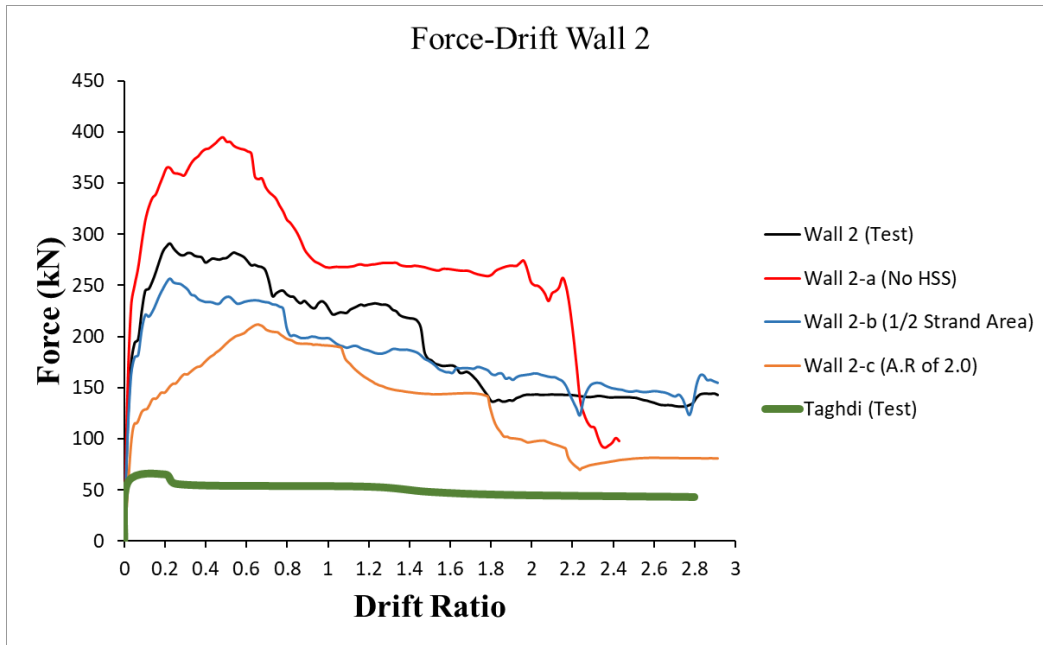


Figure 242.7 Comparisons of analyses results for Wall 2

The consequence of prestressing the wall without using the HSS yielding agents was investigated analytically in Chapter 5. The elimination of the yielding agents showed higher lateral force capacity, as expected, because the prestressing strands were able to develop higher forces. Figure 6.7 shows force-drift responses of Wall 2 with and without the HSS yielding agents. The wall developed about 100 kN extra lateral load capacity without the HSS agents. The strands were able to yield and develop inelastic behaviour. The increase in prestressing forces led to corresponding increases in flexural and shear concrete compressive forces. This resulted in the crushing of concrete, which caused a sudden strength loss in the wall, starting at 0.5% lateral drift. The effect of yielding agent clearly indicated that the design of this retrofit scheme requires optimization of HSS yielding agents relative to the prestressing forces expected during seismic response. The analysis of Wall 2-b with reduced area of prestressing steel indicated reduced lateral force capacity because of the reduced internal force couple for flexure. The reduction in flexural resistance was accompanied by a reduction in shear forces and deformable post-yield response. This analysis emphasized once again the importance of reducing shear forces for improved wall deformability. The low rise walls investigated in the current research project can attract high seismic shear forces if their flexural capacity is increased. But their response is followed by a subsequent shear failure. This behaviour changed when a companion wall with a higher aspect ratio of 2.0 was analyzed. Wall 2-c had twice the wall height, but it was retrofitted using the same

method. This wall was not post-tensioned in order to provide more displacement capacity prior to steel rupture. The wall developed a lateral load capacity of about 200 kN (3.5 times the capacity of the unretrofitted wall) at a lateral drift of 0.7%. It developed a lateral drift ratio of 1.8% prior to a strength decay of more than 20% of the peak load resistance primarily because of the dominance of flexural behaviour.

6.3.3 Comparisons of All the Analytical Scenarios Considered

Force-drift responses of the analytical models for all the walls with selected design parameters are compared in Figure 6.8. All the retrofitted masonry walls demonstrate significant increases in seismic force capacities relative to the unretrofitted wall tested by Taghdi et al. (2000). The level of increase in seismic force resistance varied between 3.5 to 5.5 times, depending on the design parameters. The increase in wall flexural capacity attained by the use of internal reinforcement, including post-tensioning, increased seismic shear demands. Because of the low aspect ratio of the walls, higher shear forces resulted in reduced deformability. The higher the force capacity was, the lower the deformability was. Filling of the web masonry cells with concrete, in addition to providing reinforced or prestressed concrete boundary elements improved the wall seismic performance.

6.4 Recommended Design Procedures for the Seismic Retrofit Methodologies Developed

Existing URM low rise walls do not have the required seismic capacity in medium and high seismic regions in Canada. This includes mostly populated Eastern and Western Canada. An effective way to mitigate seismic risk in these regions is to assess the vulnerability of URM walls and retrofit them to increase their seismic resistance. Seismic resistance of URM walls are limited by shear or rocking modes of failure because of the lack of internal reinforcement. Lack of vertical reinforcement, well anchored into the wall foundation thereby clamping the wall to its support is the major missing element for flexural response. Therefore, adding internal vertical reinforcement was considered as an effective way of providing flexural resistance. This was done either by adding deformed reinforcement near the ends of the wall to generate wall boundary elements, or by post-tensioning the walls by means of 7-wire strands, also placed at the ends of the wall where they would be the most effective. In addition, surface-bonded CFRP sheets were applied for improved shear resistance. The design parameters considered experimentally and analytically as discussed

in the preceding sections provide design information that can be used along with well accepted design provisions outlined in the current National Building Code of Canada and the clauses of CSA Standard S304 for Design of Masonry Structures and CSA S806 for design of FRP reinforced concrete structures.

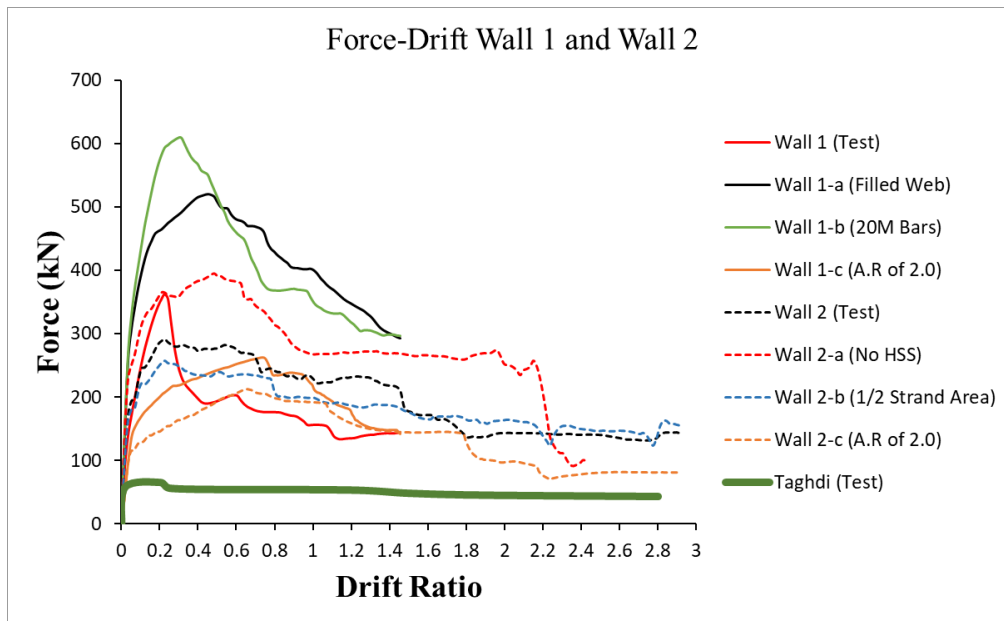


Figure 243.8 Comparison of all the scenarios for Wall 1 and Wall 2

The following steps are recommended for conceptual design of the proposed seismic retrofit techniques for URM walls based on limited research conducted in the current investigation:

1. Determine seismic force demands on the wall using the seismic hazard for the region and the seismic design provisions of NBCC based on $R_d = R_o = 1.0$ (elastic force demand).
2. Establish the seismic deficiency in flexure, and compute the force couple that can be generated in the wall to provide the required flexural resistance.
3. Decide on the type of internal reinforcement to be placed in the wall; i) non-prestressed deformed reinforcement with concrete filled masonry cells within the entire wall length, or ii) post-tensioning strands to be implemented by following the construction procedure outlined in Chapter 3, also with concrete filled masonry cells within the entire wall length.
4. If non-prestressed deformed re-bars are to be used for seismic retrofitting, determine the area of reinforcement required to provide tension tie forces in boundary elements to be

anchored into the wall foundation (or the structural element below) as flexural reinforcement. Otherwise go to step 8 to design for post-tensioning strands.

5. Compute seismic shear demand associated with probable moment resistance (computed using $1.25 f_y$ for reinforcement) for use in shear design.
6. Design for surface-bonded CFRP sheets as shear reinforcement (for diagonal tension capacity) as per CSA S806 (2012). Check for maximum diagonal compression capacity in the web based on masonry filled with concrete. If the diagonal compression capacity is not exceeded, continue with the design. Otherwise, use another retrofit technique that can increase the shear capacity further.
7. If elastic design is to be performed, the design is complete, move to the implementation phase. If moderately ductile behaviour is required, reduce the seismic force demand determined in Step 1 by $R_d = 2.0$ and $R_o = 1.5$ as per the NBCC requirements for moderately ductile masonry shear walls. Ensure that the seismic shear force demand established in Steps 5 is less than the seismic shear capacity computed in Step 6 by at least a factor of 2.0. Note that this factor of safety of 2.0 for increase in shear capacity relative to shear demand is recommended based on limited research conducted in the current investigation by analyzing walls with an aspect ratio of 2.0. If demonstrated through analyses, a lower ratio may be used for shear.
8. If prestressed strands are to be used for retrofitting, compute the area of strands and the level of prestressing that provides the required tension tie established in Step 2.
9. Compute seismic design shear forces associated with moment resistance increased by a factor of 1.25 to safeguard against brittle shear failure. If the strands provide higher tension tie forces than that corresponding to strand yield strength, and self-centering capability of

the strands is required in design, then a yielding device consisting of HSS sections similar to the ones used in the experimental phase can be implemented to control the strand forces and keep them in the elastic range. The latter approach eliminates the possibility of strand rupturing while promoting ductile response.

10. Implement Steps 6 and 7 to safeguard against brittle shear failure.

7 Summary and Conclusions

7.1 Introduction

Combined experimental and analytical research was conducted to develop seismic retrofit techniques for URM walls. The retrofit techniques involved providing internal reinforcement to improve flexural capacity through either the use of non-prestressed deformed reinforcement or prestressed 7-wire strands. It also included the use of externally applied surface-bonded CFRP sheets for enhanced shear capacity. Analytical research was conducted to expand the findings of the experimental research, by performing non-linear FEM analyses to assess the significance of design parameters. The research findings resulted in a conceptual design methodology for retrofitting seismically deficient URM walls with internal steel reinforcement and external CFRP sheets. The conclusions are presented in the following sections.

7.2 Experimental Conclusions

The following conclusions can be drawn from the experimental phase of research:

- The use of deformed reinforcement in an existing URM wall, by cutting the exterior shells of masonry units and inserting re-bars that are well anchored to the structural element below (ex: foundation) and by filling the masonry units with concrete, results in reinforced concrete boundary elements at the ends of the wall. The resulting improvement in flexural capacity is equal to that of a reinforced masonry shear wall and can be substantially higher than the resistance attained by a URM wall experiencing rocking and sliding modes of behaviour under seismic forces. The wall tested in the current investigation exhibited a strength increase of 5.5 times that of the unretrofitted control wall. The flexural strength enhancement can be established through sectional analysis and using the requirements of CSA Standard A23.3 (2014).
- Surface bonded CFRP sheets with fibres oriented in horizontal and vertical directions provide resistance to diagonal tension generated by lateral shear force reversals. The thickness of CFRP composite layers can be designed using the requirements of CSA S806 (2002).
- URM walls, retrofitted by internally placed flexural reinforcement develop higher

flexural resistance, which in turn increases seismic shear force demands. While the surface-bonded CFRP sheets can control inclined shear cracks and enhance the diagonal tension capacity, thin walls of masonry units in the wall web are not enough to resist shear-induced diagonal compression stresses, resulting in the crushing of the web, followed by a premature failure of the wall, resulting in brittle response. Filling the cells of URM walls with concrete improved resistance to diagonal compression, increasing wall ductility.

- Post-tensioning of existing URM walls by removing the exterior shells of masonry units at wall ends and providing 7-wire strands that are well anchored into the structural element below (ex: foundation) and post-tensioning them against concrete filled boundary elements improves wall flexural capacity substantially. The post-tensioned wall that was tested in the experimental program developed 3.5 times the strength of the unretrofitted control wall. Furthermore, the same post-tensioned wall with concrete filled masonry cells developed a lateral drift of 1.5% prior to losing more than 20% of its maximum strength. The wall exhibited gradual strength decay and significantly improved inelastic deformability.
- If post-tensioning cables of retrofitted URM walls remain elastic during response, they exhibit self-centering capabilities with elastic strands pulling the wall back to its neutral position during cyclic response. This is evident in the wall tested, which showed hysteresis loops with unloading branches passing through the origin. The stress level in post-tensioning cables can be controlled by using steel HSS sections at the ends of the cables that can be designed to yield, dissipate energy and limit the force in the strands under high levels of inelastic deformations, preventing the rupturing of the cables and mitigating the risk of crushing concrete struts in low-rise walls associated with increased shear demands.
- The implementation of the retrofit techniques developed, and the associated construction procedures (explained in Chapter 3) are feasible as demonstrated experimentally. The seismic retrofit strategy presented and validated in the experimental program can be used to improve strength and inelastic deformability of otherwise seismically deficient, brittle URM walls to conform to the requirements of

modern seismic resistant design practices.

7.3 Analytical Conclusions

The following conclusions can be drawn from the analytical phase of the research:

- The FEM models developed for the retrofitted masonry walls tested in the experimental phase of research, consisting of nonlinear layered shell objects in Software SAP2000, produce reasonably good agreement with experimentally recorded force-displacement relationships. The analytical parametric investigation conducted using the above models generated significant design information that enabled the development of a conceptual retrofit design strategy for internally reinforced URM walls.
- Low-rise URM walls retrofitted with internally placed non-prestressed re-bars or prestressed cables may develop sufficiently high flexural capacities and increased shear demands such that diagonal compression crushing may be triggered, especially if diagonal tension is controlled by externally bonded CFRP reinforcement. This brittle failure mode can be delayed or prevented if the masonry cells are filled with concrete as part of the retrofit strategy, improving flexural and diagonal concrete strut capacities. The analytical models with and without the web-filled concrete show a significant difference in post-peak behavior of low-rise walls, with concrete filled walls showing 0.8% drift ratio at 20% strength decay level in comparison with the wall with empty masonry cells in the web, which develops 0.25% drift at the same level of strength decay.
- An increase or decrease in the percentage of steel in wall boundary elements translates into equivalent strength enhancements in walls. Low-rise walls with high shear demands may have to have a limit on the amount of flexural reinforcement to prevent

premature shear failure.

- Post-tensioning cables with and without the HSS yielding devices develop controlled or uncontrolled tension forces in the cables. The level of control has to be computed for the specific wall with a specific shear demand so that cable rupturing and/or compression crushing of concrete can be prevented to avoid brittle failure.
- Increased wall height while keeping the wall length constant, (increased aspect ratio) results in reduced force demands because of the increased lever arm. These walls exhibit ductile response when retrofitted with either one or the other retrofit methods presented.
- The conceptual framework for a retrofit design strategy developed, including the use of the seismic provisions of NBCC and the relevant CSA material standards, can be employed to design URM walls using the seismic retrofit techniques developed in the current investigation.
- The retrofit strategy introduced can be used to upgrade URM walls from being vulnerable brittle elements to reasonably deformable moderately ductile reinforced masonry shear walls.

7.4 Contributions to the Field of Earthquake Engineering

The research project reported in the current dissertation resulted in original contributions to the field of earthquake engineering in the area of seismic retrofit of URM walls. Two new approaches were developed, one consisting of the use of internally placed non-prestressed deformed reinforcement, and the other consisting of internal post-tensioning, where both approaches involved protection against shear by surface-bonded CFRP sheets and concrete filled masonry cells. Proof of concept was achieved through tests of large-scale specimens under inelastic deformation reversals. The retrofit techniques also involved unique

construction procedures for their implementation in practice. In addition, the experimental phase resulted in the generation of invaluable test data on performance of retrofitted URM walls for use by other researchers and practicing engineers.

The analytical phase of the research expanded the design data generated in the experimental program. It involved the construction of analytical models suitable for nonlinear seismic analysis of URM walls, incorporating masonry, mortar, concrete, mild steel, prestressing steel and CFRP material modelling. The analytical model developed may provide guidance to researchers and practicing engineers who may be involved in analyses of retrofitted masonry walls. A large number of parametric analyses were conducted to generate useful design information. The significance of design parameters in terms of failure modes and associated flexural and shear capacities were assessed as they relate to overall strength and ductility. The observations made from this combined experimental and analytical investigations having led to the development of a conceptual framework for retrofit design may be used in practice for seismic risk reduction.

In summary, the research work reported in the dissertation has numerous original contributions with potentially significant impact on seismic risk reduction of otherwise seismically vulnerable URM buildings and building components.

7.5 Recommendations for Future Research

The current research project involves development of two new seismic retrofit methods for low-rise masonry walls. The concepts developed in the thesis can be expanded by further research. The following topics are recommended to be pursued in future research projects:

- Tests of URM walls with different wall aspect ratios retrofitted with internally placed reinforcement/prestressing cables. The aspect ratios can be higher and lower than 1.0 to promote flexure-dominant and shear dominant behaviour, respectively.
- Tests of walls with different percentages of vertical boundary element reinforcement and levels of prestressing to investigate different flexure-to-shear capacity ratios.
- Tests of masonry walls with internal reinforcement and surface-bonded CFRP sheets applied on both sides of the walls, while also varying the amount of fibres for shear stress control.

- Tests of masonry walls retrofitted with internal post-tensioning and different size HSS yielding devices to attain different levels of prestressing force control and achievement of self-centering capabilities.
- Analyses of walls to validate analytical models for the above proposed cases while expanding analytical parametric investigations to consider different levels of existing gravity loads.
- Selection of a prototype masonry building and dynamic inelastic analyses of the building with and without the proposed retrofit techniques to assess their effectiveness in real size buildings under eastern and western Canadian seismic hazards.
- Experimental research to investigate out-of-plane behaviour of masonry walls with and without retrofits under earthquake loading.

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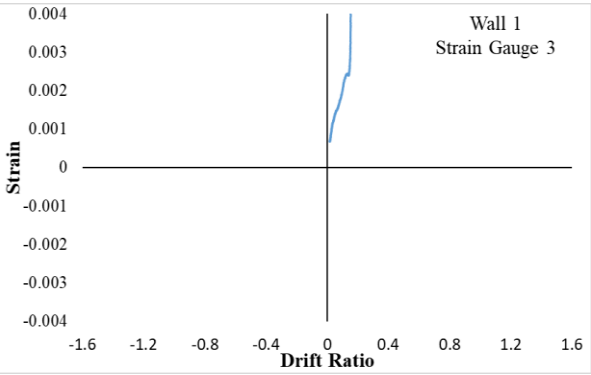
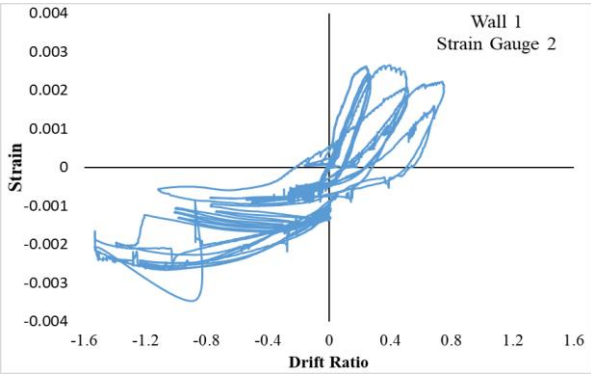
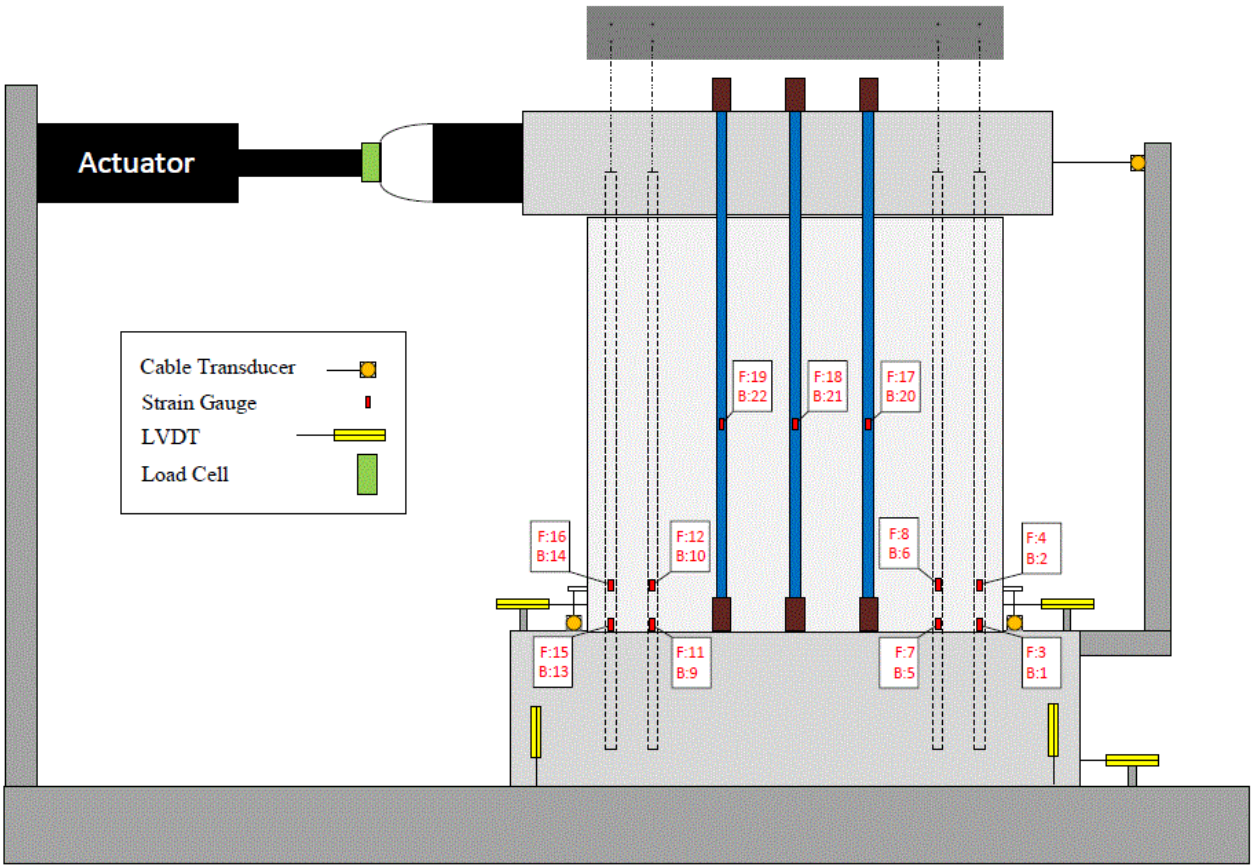
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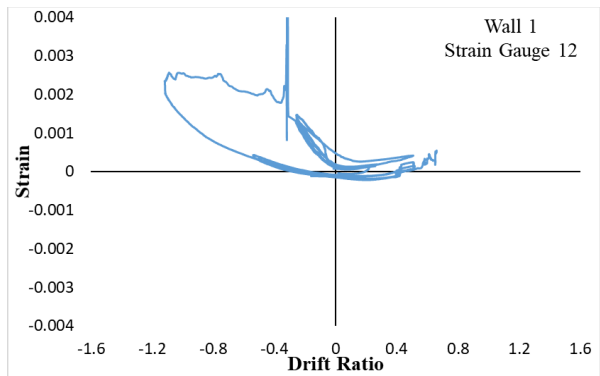
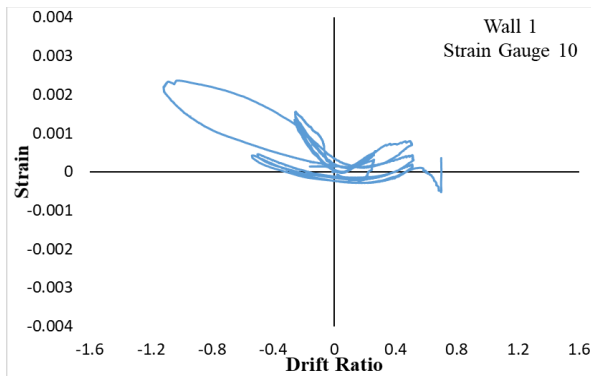
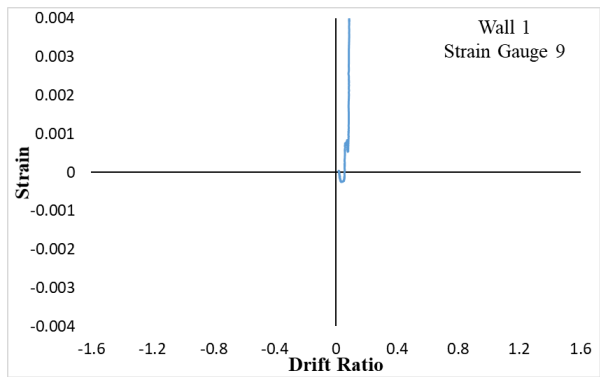
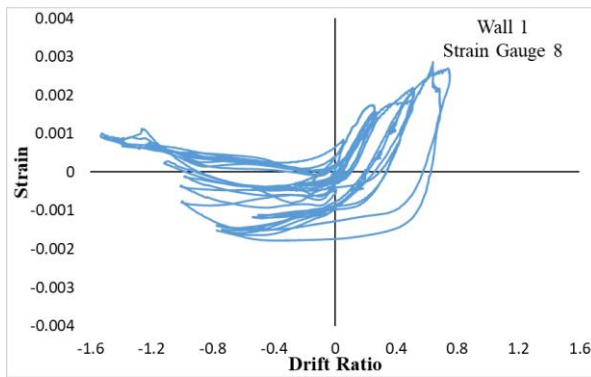
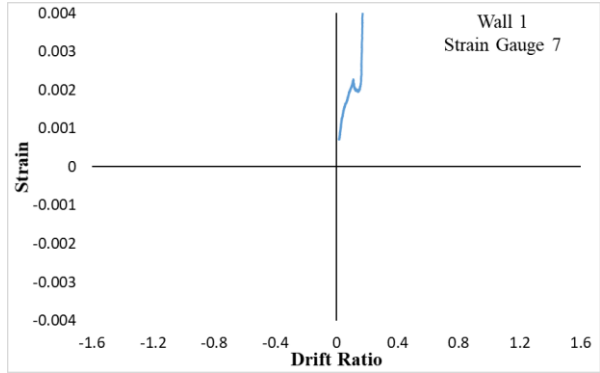
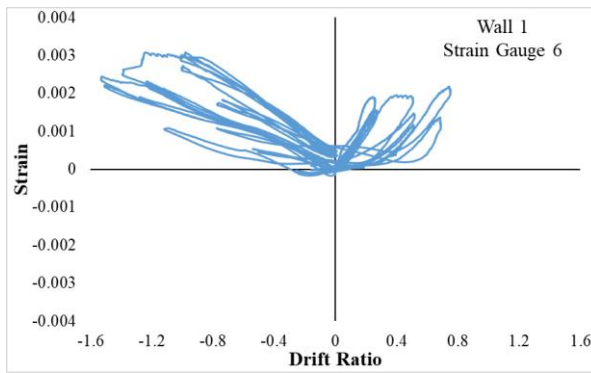
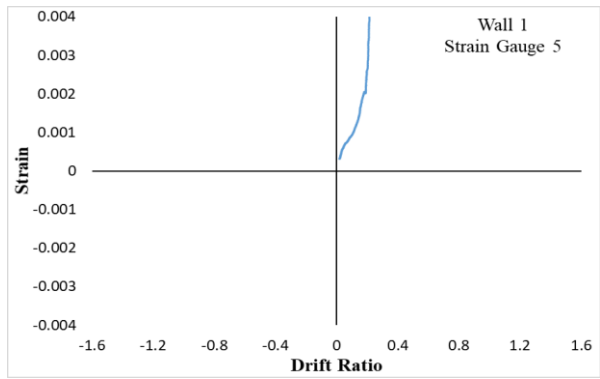
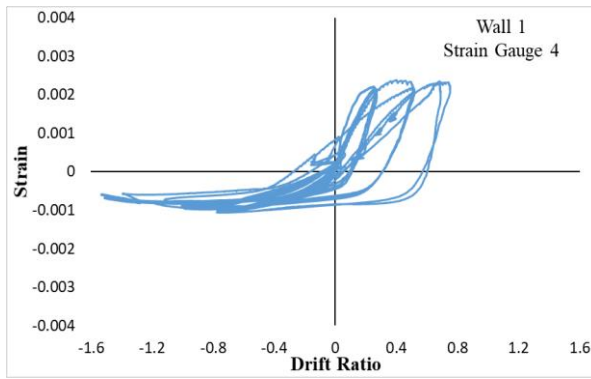
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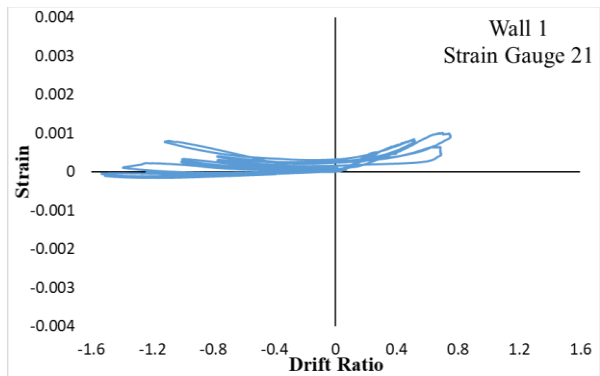
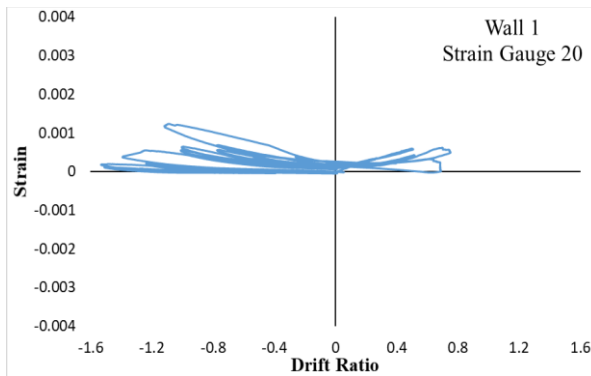
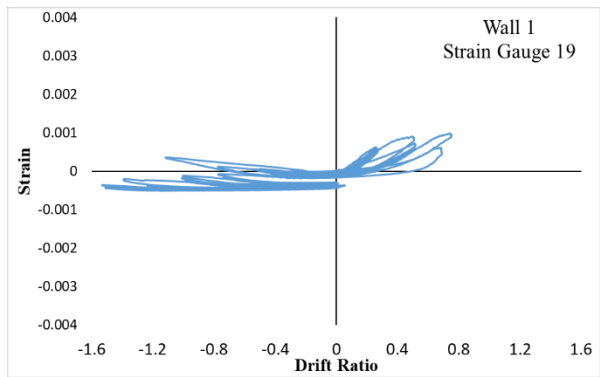
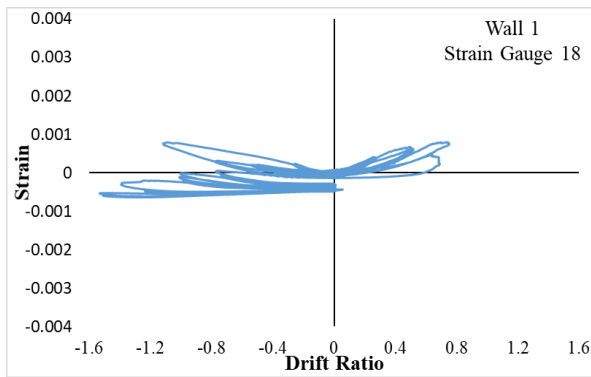
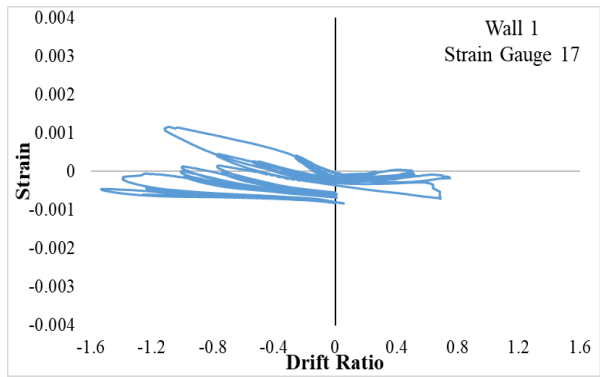
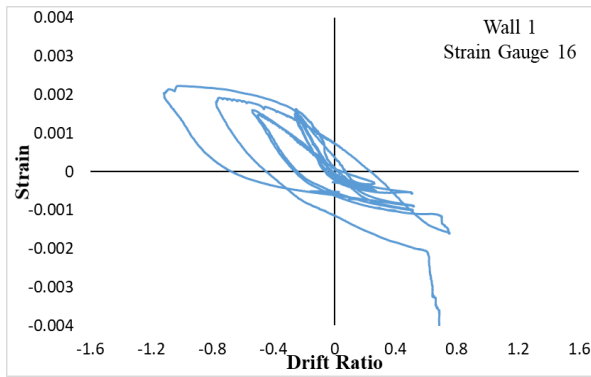
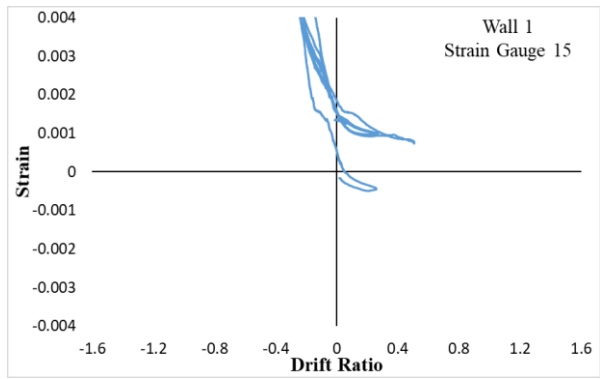
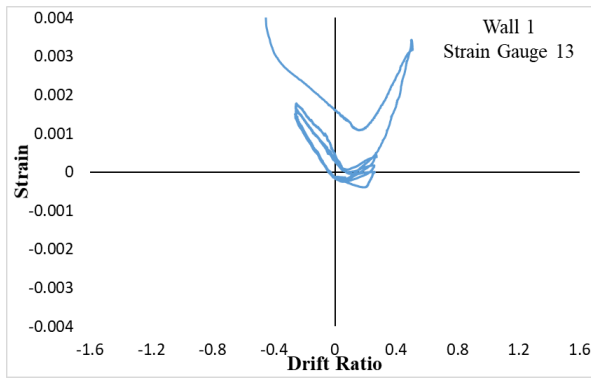
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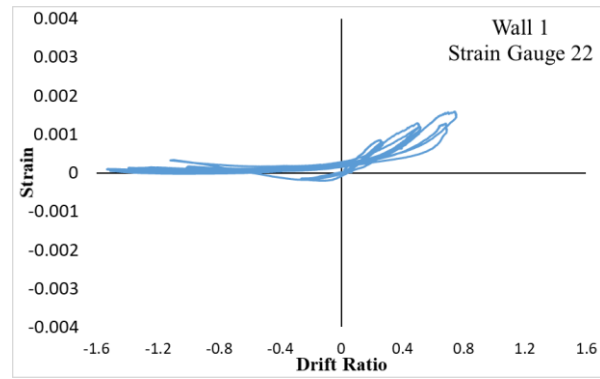
Appendix I. Strain Gauge Data for Wall 1 and Wall 2

STRAIN GAUGES OF WALL 1

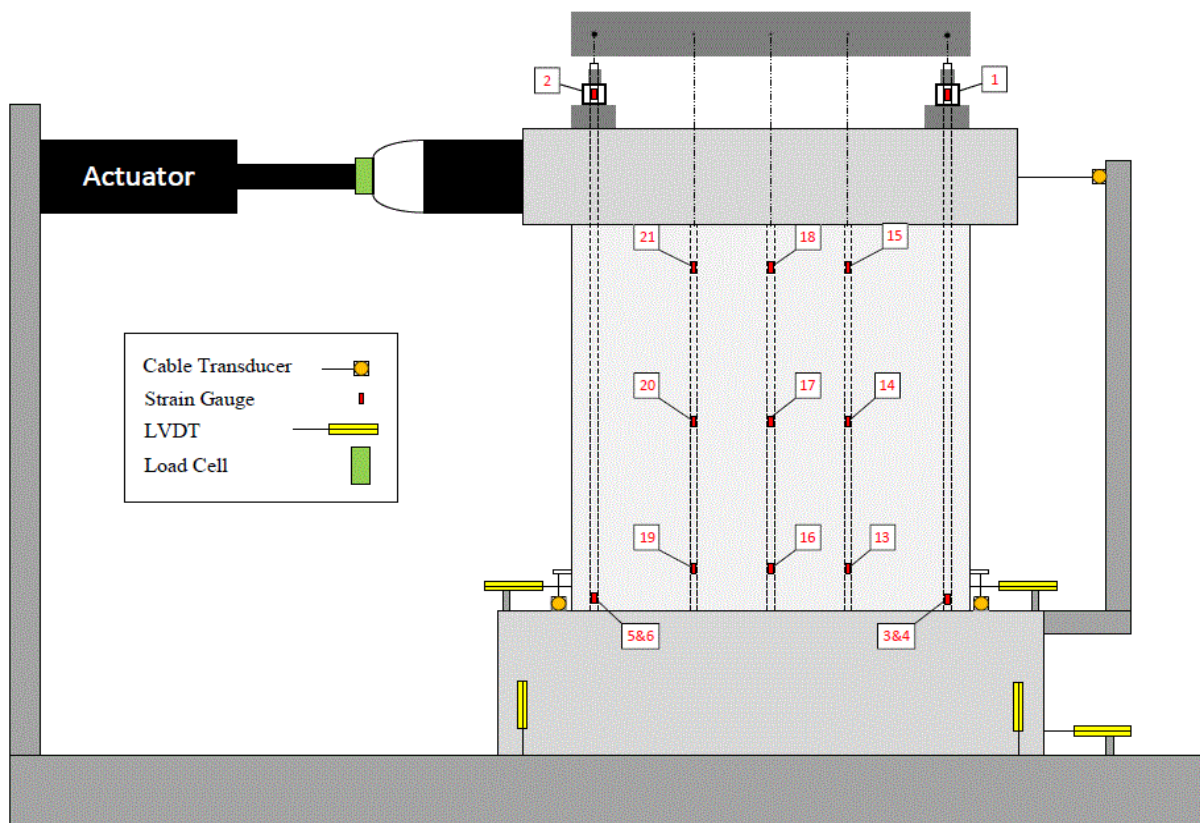




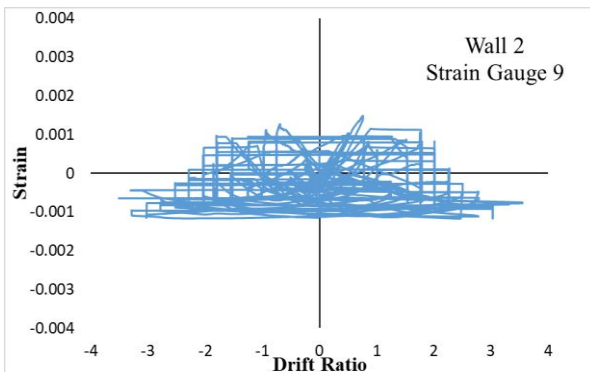
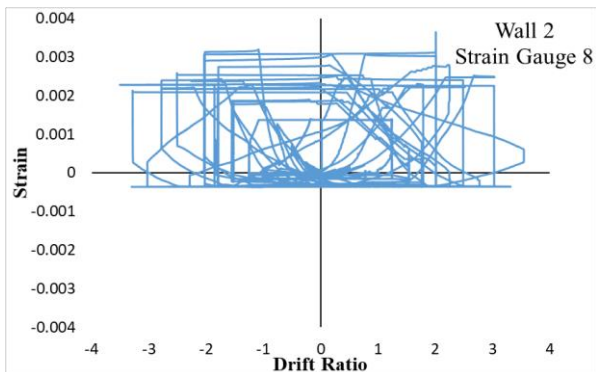
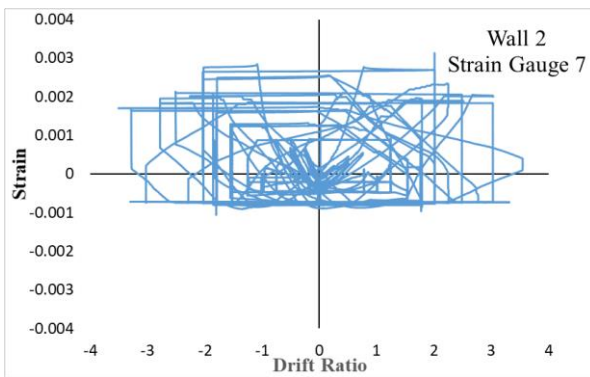
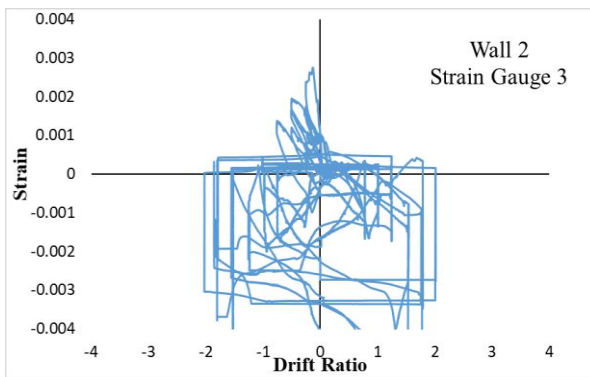
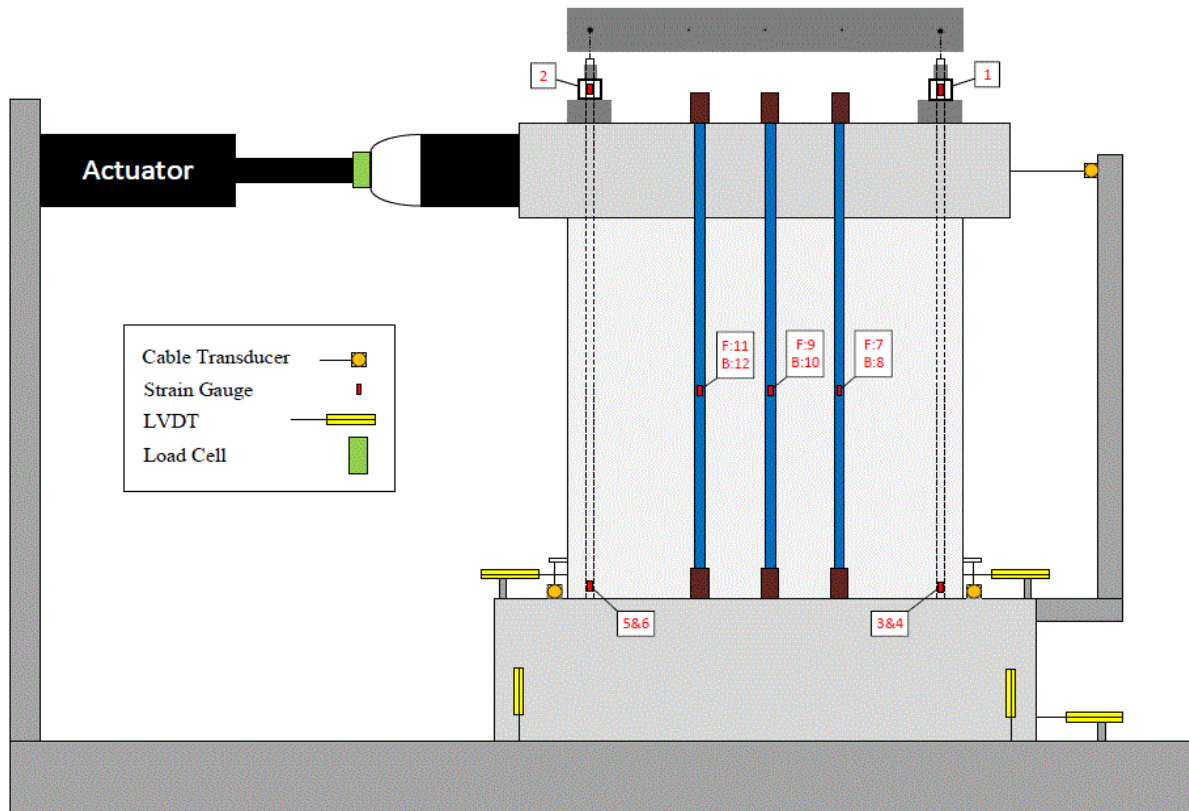


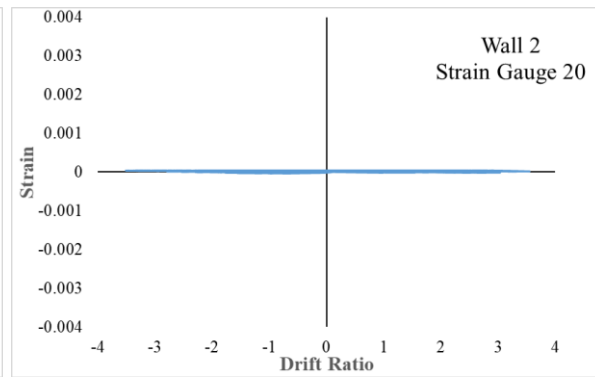
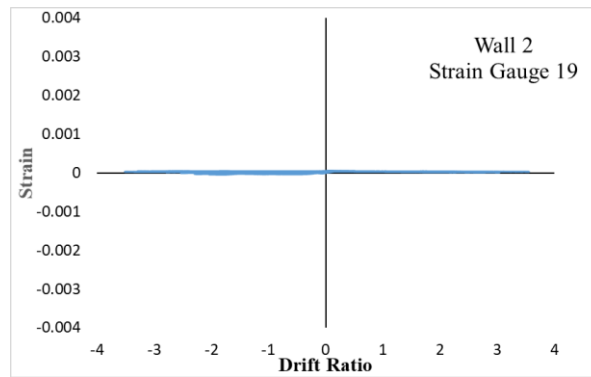
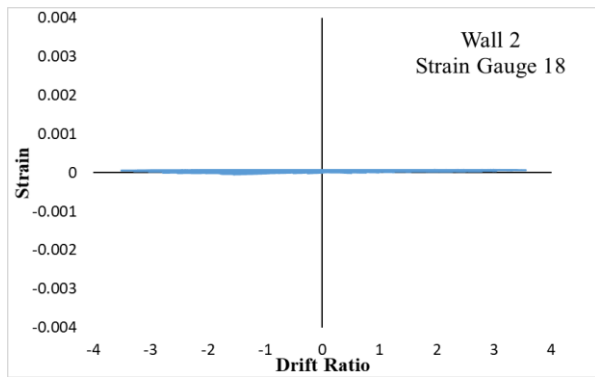
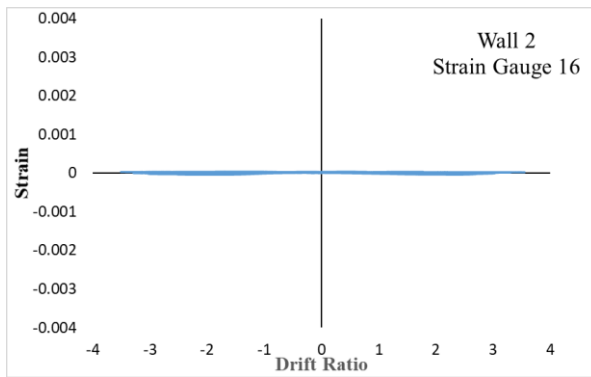
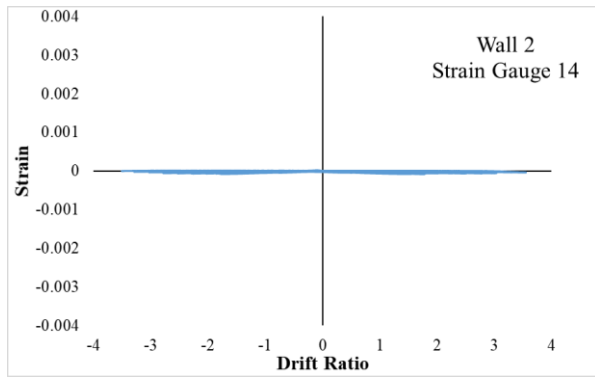
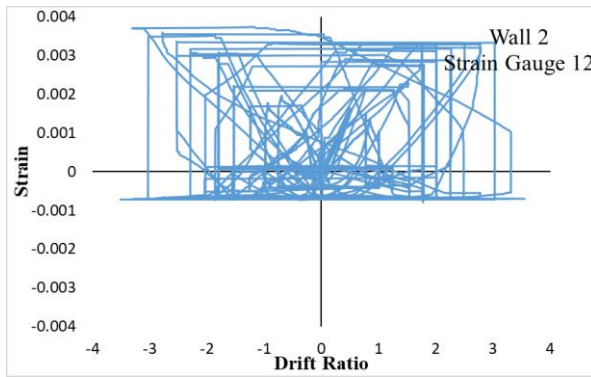
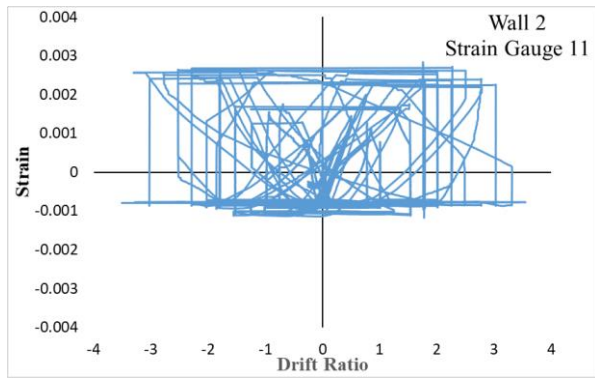
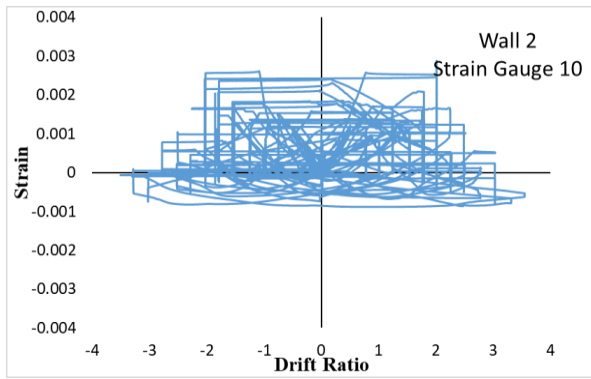


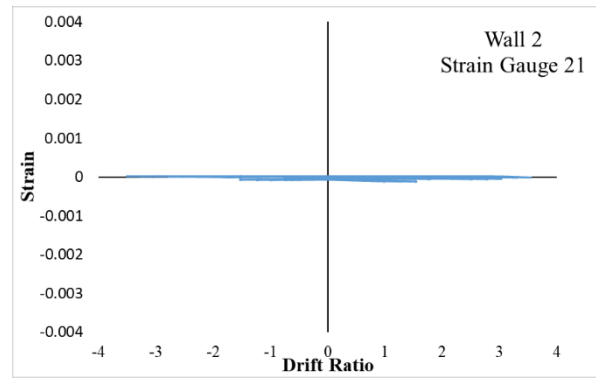
Strain Gauges of Wall 2



Strain Gauge numbering for tendons for Wall 2







Appendix II. Layers of Shells for Wall 1 and Wall 2

Layered Shells of Wall 1

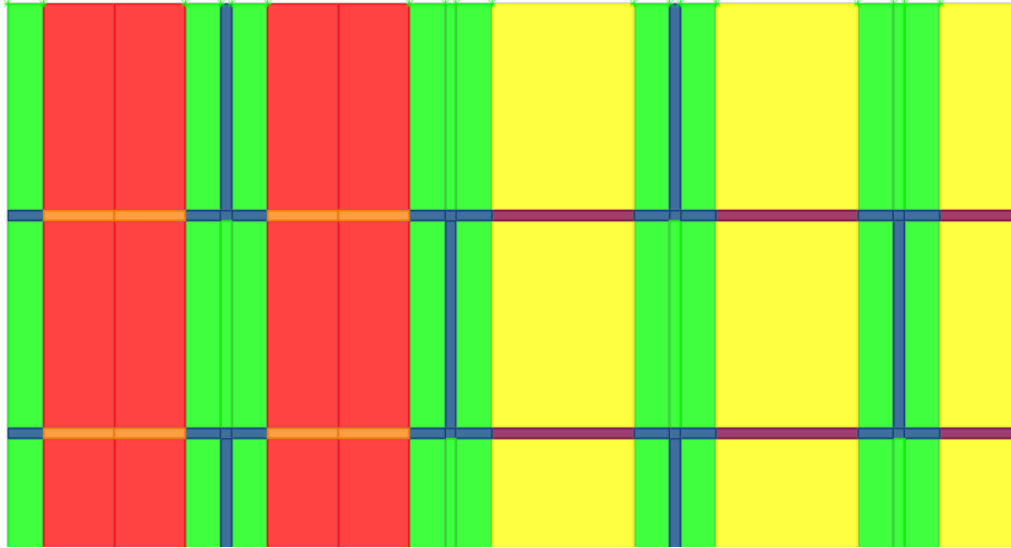


Figure A244.1 Close view to the top left corner of analytical model for wall 1

Red

Red shell section at Wall 1 is consisting of the seven layers with four different materials, which are shown at Figure A2.1. Red shell sections are representing the boundary cells of the Wall 1, that were filled with reinforced concrete. As it is shown at the left side of the Figure A2.1, the layer names are: concrete, steelt, steelb, masonryt, masonryb, FRPHT, and FRPVT. The distance of layers relative to the centre line and the thickness of layers are all selected according to the geometrical properties of the tested walls at the red shell locations.

Four types of materials (concrete, steel, masonry, and FRP) that were used inside shell sections are all defined according to the actual material behaviours that were explained in Chapter 3. The defined material properties for software are explained in section 5.3.3.

Two layers of FRP with 1 mm thickness are considered on one side of the wall for all the shell sections of wall 1 and wall 2. The horizontal FRP layer is named FRPHT with material angle of zero degree, and the vertical FRP layer is named FRPVT with material angle of ninety degree.

Two layers of steel with 1.53 mm thickness are representing two 15M rebars that were anchored inside the footing at each cell.

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Behavior S22	S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steelb	32.0625	1.53257	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
steelb	-32.0625	1.53257	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryb	-78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Figure A245.2 Red shell section for wall 1

Orange

Orange shell sections at Wall 1 are representing the joint areas at boundary cells. Orange shell is similar to red shell, except the masonry layers that are replaced with mortar layers. Figure A2.2 shows the properties of orange shell.

Green

Green shell sections are representing the part of the wall that is just made of masonry concrete blocks and FRP. As it is shown in Figure A2.3, green shell is consisting of one layer of masonry and two layers of FRP.

Blue

Blue shell is similar to green shell, but it is consisting of one layer of mortar and two layers of FRP. These shells are representing the joint locations between the green shell sections. Figure A2.4 shows the properties of blue shell section.

Yellow

Yellow shells are representing the part of the wall 1 that is not filled with any material and it is consisting of hollow masonry block. As it is shown at Figure A2.5, this shell has two layers of FRP and two layers of masonry with a hollow space in between.

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steel	32.0625	0.999998	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
steelb	-32.0625	0.999998	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
mortar	78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
mortarb	-78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 4. Mortar+CasterConcrete+Rebar

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 7
Total Section Thickness: 192.
Sum of Layer Overlaps: 2.
Sum of Gaps Between Layers: 0.

OK Cancel

Figure A246.3 Orange shell section for wall 1

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
masonry	0.	190.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 2. Masonry Concrete

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 3
Total Section Thickness: 192.
Sum of Layer Overlaps: 0.
Sum of Gaps Between Layers: 0.

OK Cancel

Figure A247.4 Green shell section for wall 1

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Behavior S22	S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
mortart	0.	190.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

☐ Highlight Selected Layer
Transparency Control < [] >

Section Name
5. Mortar

Order Layers By Distance
Order Ascending Order Descending

Calculated Layer Information

Number of Layers 3
Total Section Thickness 192.
Sum of Layer Overlaps 0.
Sum of Gaps Between Layers 0.

OK Cancel

Distance -84.509278

Figure A248.5 Blue shell sections for wall 1

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Behavior S22	S12
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryb	-78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

☐ Highlight Selected Layer
Transparency Control < [] >

Section Name
3. Hollow Masonry Concrete

Order Layers By Distance
Order Ascending Order Descending

Calculated Layer Information

Number of Layers 4
Total Section Thickness 192.
Sum of Layer Overlaps 0.
Sum of Gaps Between Layers 124.

OK Cancel

Distance

Figure A249.6 Yellow shell sections for wall 1

Purple

Purple shells are representing the mortar space between the yellow shells. This shell is similar to yellow shell, but it has two layers of mortar rather than masonry. As it is shown in Figure A2.6, purple shells are consisting of two layers of FRP and two layers of mortar with a hollow space in between.

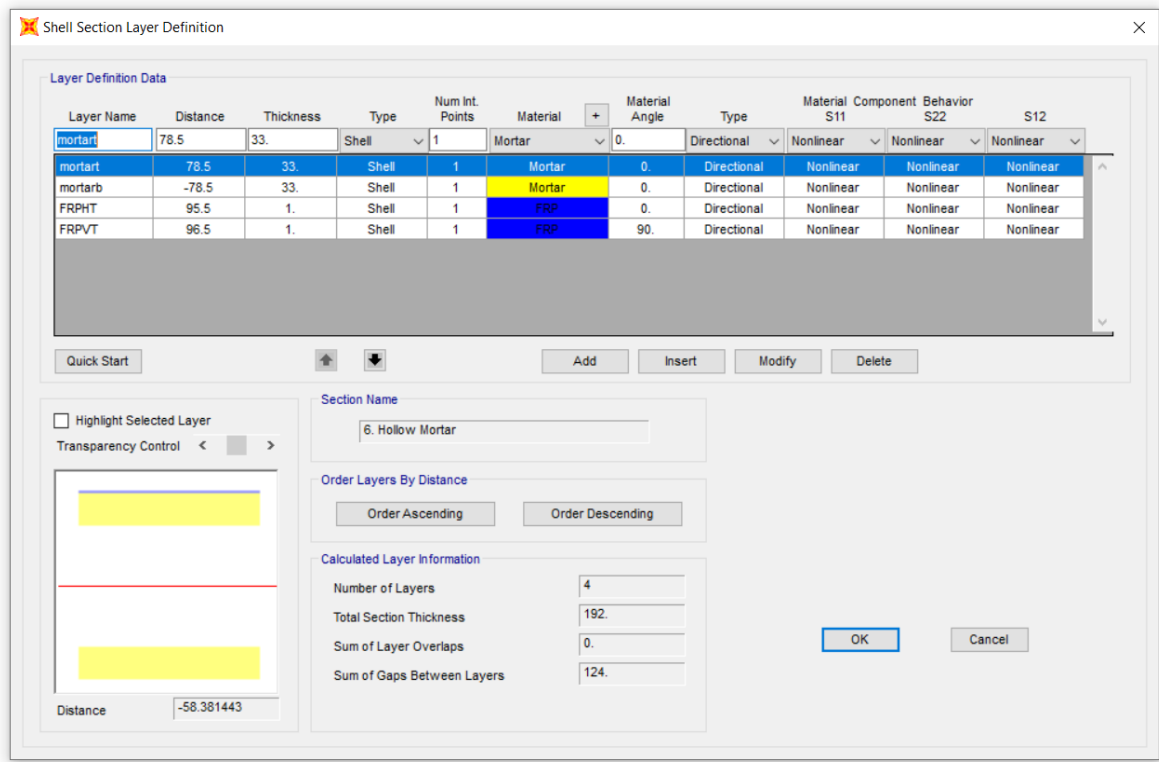


Figure A250.7 Purple shell section for wall 1

Layered Shells of Wall 2

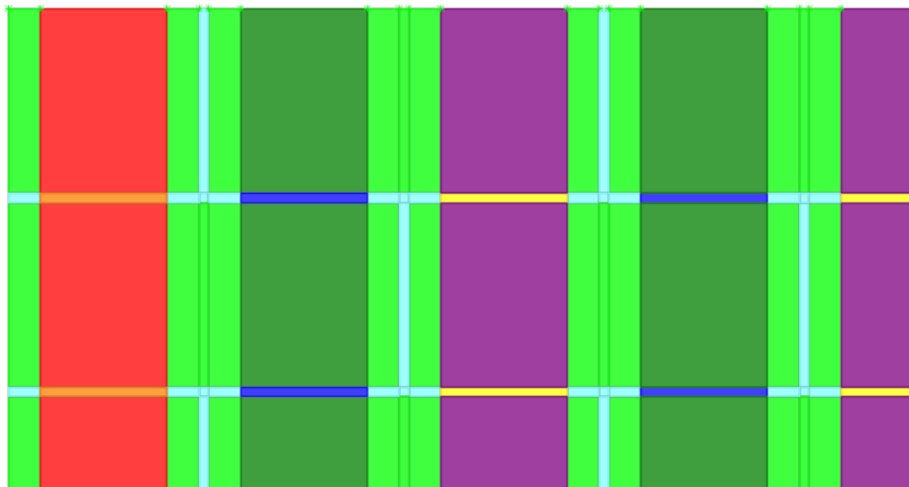


Figure A251.8 Close view to the top left corner of analytical model for wall 2

Red

Red shell sections are representing the boundary cells of the wall 2 that are filled with concrete and prestressed with #15 strand. As it is shown in Figure A2.9, the centre of this shell is filled with concrete and it has a strip of steel at the centre. The steel layer is representing the tendon which is modeled as a layer with 1.06 mm thickness. The material of tendon is named “Tendon” and it is illustrated in section 5.3.3.5.

Similar to wall 1, two layers of masonry were placed at the top and bottom of concrete layer. Finally, two layers of FRP sheets with 1 mm thickness were placed horizontally and vertically on one side of the shell.

Orange, Light Green, and Light Blue

Orange shell is similar to red shell and it represents the space between red shells that are covered with mortar. Figure A2.10 shows different layers of orange shell. All the layers are identical to red shell, except the two masonry layers that are replaced with mortar. Light green and light blue shells in wall 2 are identical to green and blue shells in wall 1. Figure A2.11 and A1.12 demonstrate these shells respectively.

Green

Green shells are representing the masonry cells that are filled with concrete, but with no steel reinforcement. This shell is consisting of five layers, as it is shown at Figure A2.13. A layer of concrete is placed between two layers of masonry and two layers of horizontal and vertical FRP sheets are located on one side of the shell.

Blue

Blue shells are located between green layers and represent the joint space between concrete blocks. Blue shell is similar to green shell, but it has two layers of mortar instead of masonry. Layers and properties of blue shell is shown at Figure A2.14.

Purples

Purple shell is similar to green shell, but it has one 10M reinforcement at the centre. The steel reinforcement is modeled by a layer of steel with 0.74 mm thickness. Figure A2.15 depicted the properties of purple shell.

Yellow

Yellow shell is similar to purple shell, expect two layers of mortar that were considered instead of masonry. Yellow shells are placed between purple shells at the joint locations.

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Behavior S22	S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steel	0.	1.06284	Shell	1	Tendon	90.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryb	-78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 9. Masonry+CastConcrete+1Tendon15

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 6
Total Section Thickness: 192.
Sum of Layer Overlaps: 1.0628
Sum of Gaps Between Layers: 0.

Distance: 36.628866

OK Cancel

Figure A252.9 Red shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Behavior S22	S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steel	0.	1.06284	Shell	1	Tendon	90.	Directional	Nonlinear	Nonlinear	Nonlinear
mortar	78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
mortarb	-78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 11. Mortar+CastConcrete+1Tendon15

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 6
Total Section Thickness: 192.
Sum of Layer Overlaps: 1.0628
Sum of Gaps Between Layers: 0.

Distance: 116.2

OK Cancel

Figure A253.10 Orange shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryrt	0.	190.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 2. Masonry Concrete

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 3

Total Section Thickness: 192.

Sum of Layer Overlaps: 0.

Sum of Gaps Between Layers: 0.

OK Cancel

Distance: 79.383505

Figure A254.11 Light green shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
mortar	0.	190.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 5. Mortar

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 3

Total Section Thickness: 192.

Sum of Layer Overlaps: 0.

Sum of Gaps Between Layers: 0.

OK Cancel

Distance: -107.07423

Figure A255.12 Light blue shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryb	-78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 8. Masonry+CastConcrete

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 5

Total Section Thickness: 192.

Sum of Layer Overlaps: 0.

Sum of Gaps Between Layers: 0.

OK Cancel

Distance: -102.32371

Figure A256.13 Green shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material S11	Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
mortart	78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
mortarb	-78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

Section Name: 13. Mortar+CastConcrete

Order Layers By Distance: Order Ascending, Order Descending

Calculated Layer Information:

Number of Layers: 5

Total Section Thickness: 192.

Sum of Layer Overlaps: 0.

Sum of Gaps Between Layers: 0.

OK Cancel

Distance:

Figure A257.14 Blue shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Material Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steel	0.	0.7407	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryt	78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
masonryb	-78.5	33.	Shell	1	Masonry	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

☐ Highlight Selected Layer

Transparency Control < >

Section Name

10. Masonry+CastConcrete+1Rebar10

Order Layers By Distance

Order Ascending Order Descending

Calculated Layer Information

Number of Layers 6

Total Section Thickness 192.

Sum of Layer Overlaps 0.7407

Sum of Gaps Between Layers 0.

OK Cancel

Distance 116.2

Figure A258.15 Purple shell section for wall 2

Shell Section Layer Definition

Layer Definition Data

Layer Name	Distance	Thickness	Type	Num Int. Points	Material	Material Angle	Type	Material Component S11	Material Component S22	Behavior S12
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear
concrete	0.	124.	Shell	1	Concrete	0.	Directional	Nonlinear	Nonlinear	Nonlinear
steel	0.	0.7407	Shell	1	Steel	90.	Directional	Nonlinear	Nonlinear	Nonlinear
mortar	78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
mortarb	-78.5	33.	Shell	1	Mortar	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPHT	95.5	1.	Shell	1	FRP	0.	Directional	Nonlinear	Nonlinear	Nonlinear
FRPVT	96.5	1.	Shell	1	FRP	90.	Directional	Nonlinear	Nonlinear	Nonlinear

Quick Start

☐ Highlight Selected Layer

Transparency Control < >

Section Name

12. Mortar+CastConcrete+1Rebar10

Order Layers By Distance

Order Ascending Order Descending

Calculated Layer Information

Number of Layers 6

Total Section Thickness 192.

Sum of Layer Overlaps 0.7407

Sum of Gaps Between Layers 0.

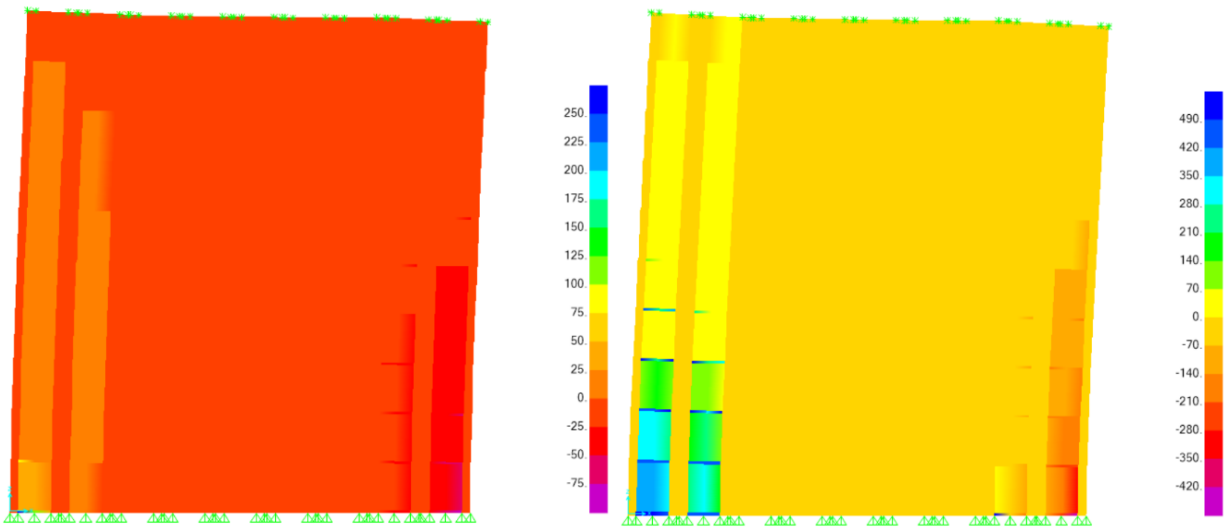
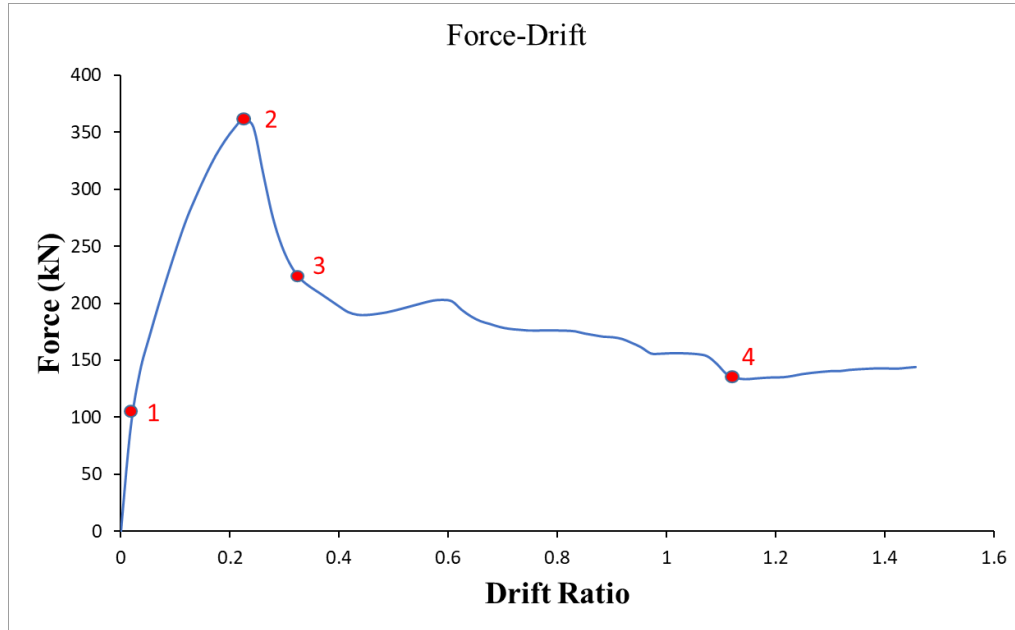
OK Cancel

Distance -80.946392

Figure A259.16 Yellow shell section for wall 2

Appendix III. Stress Contours for Analytical Models

Wall 1



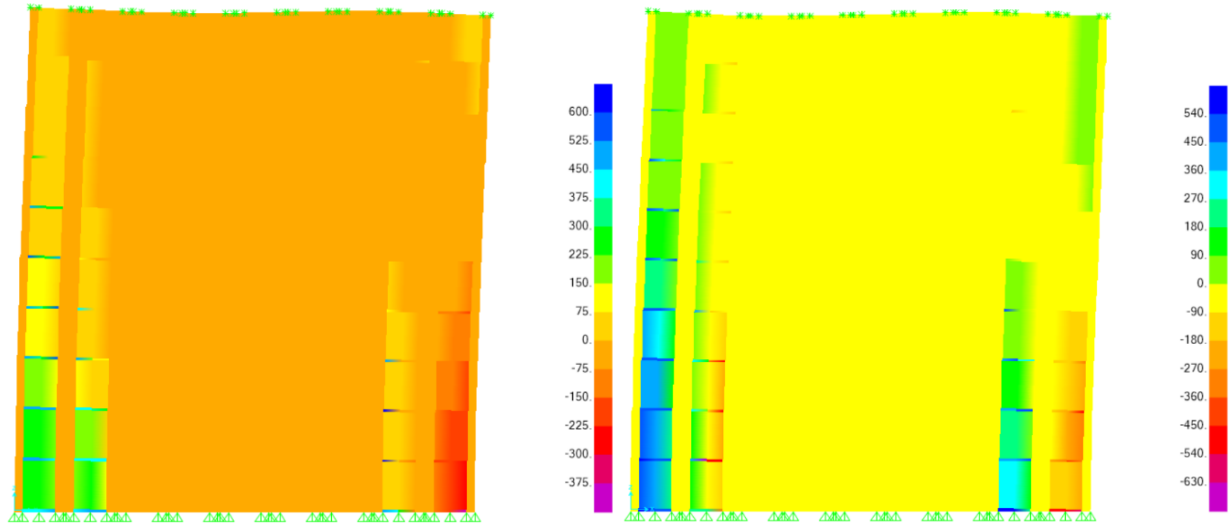


Figure A261.2 Point 3 (Left), and Point 4 (Right), Steel, S22

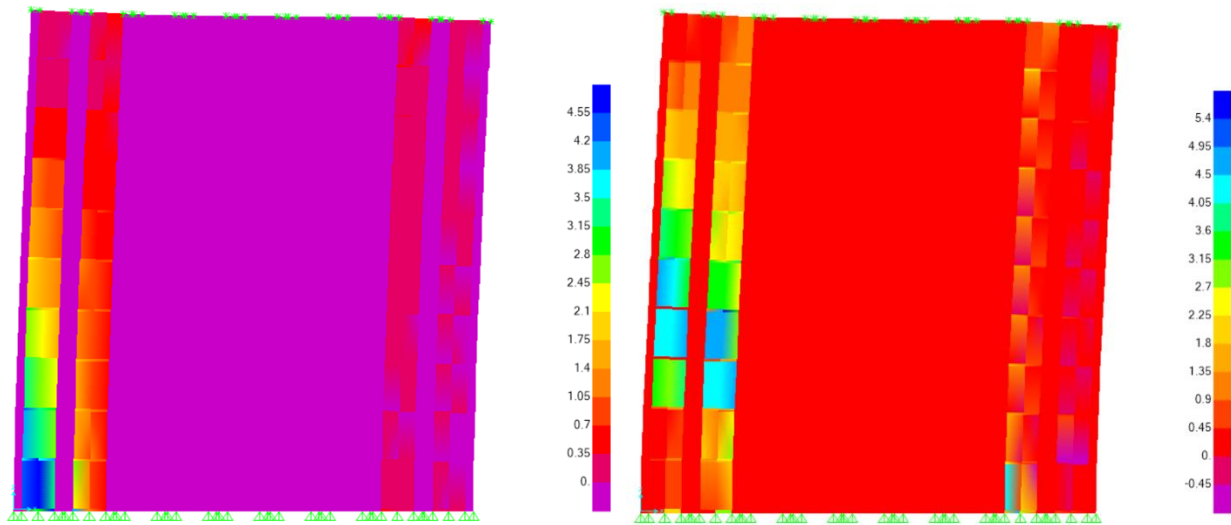


Figure A262.3 Point 1 (Left), and Point 2 (Right), Concrete, SMax

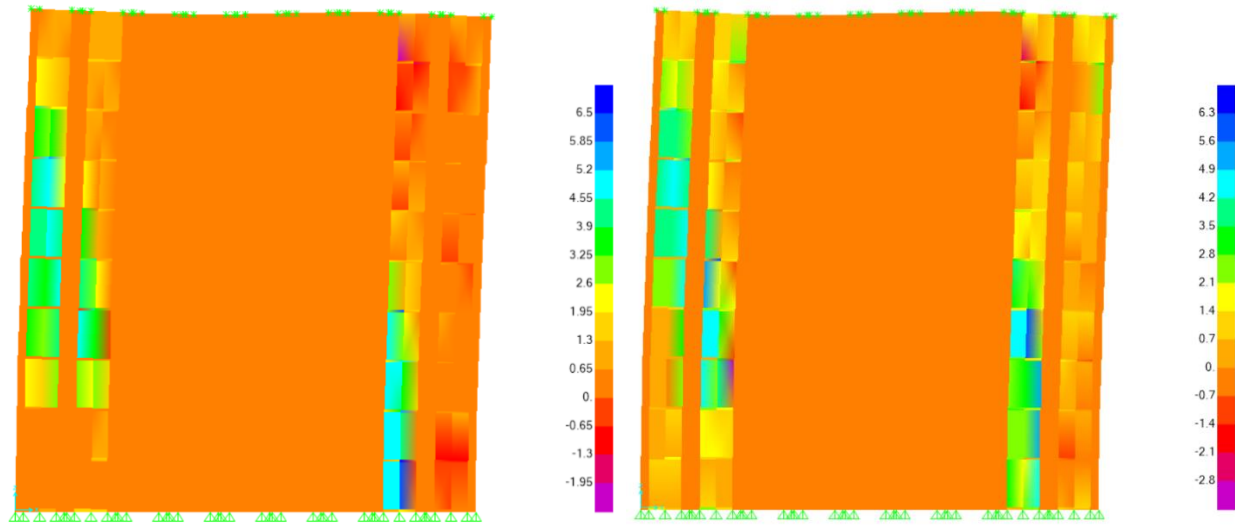


Figure A263.4 Point 3 (Left), and Point 4 (Right), Concrete, SMax

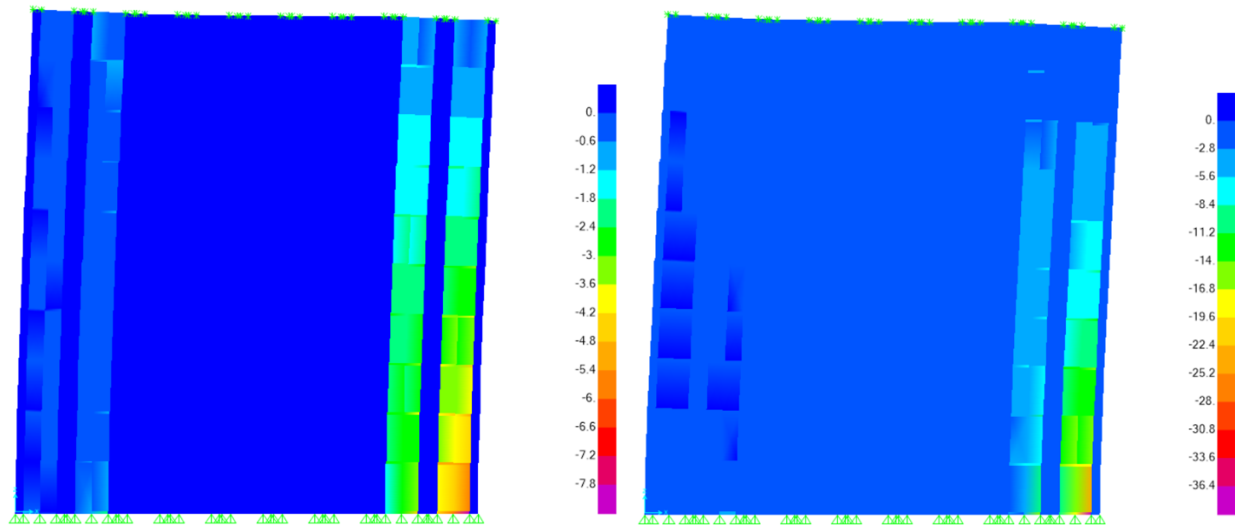


Figure A264.5 Point 1 (Left), and Point 2 (Right), Concrete, SMin

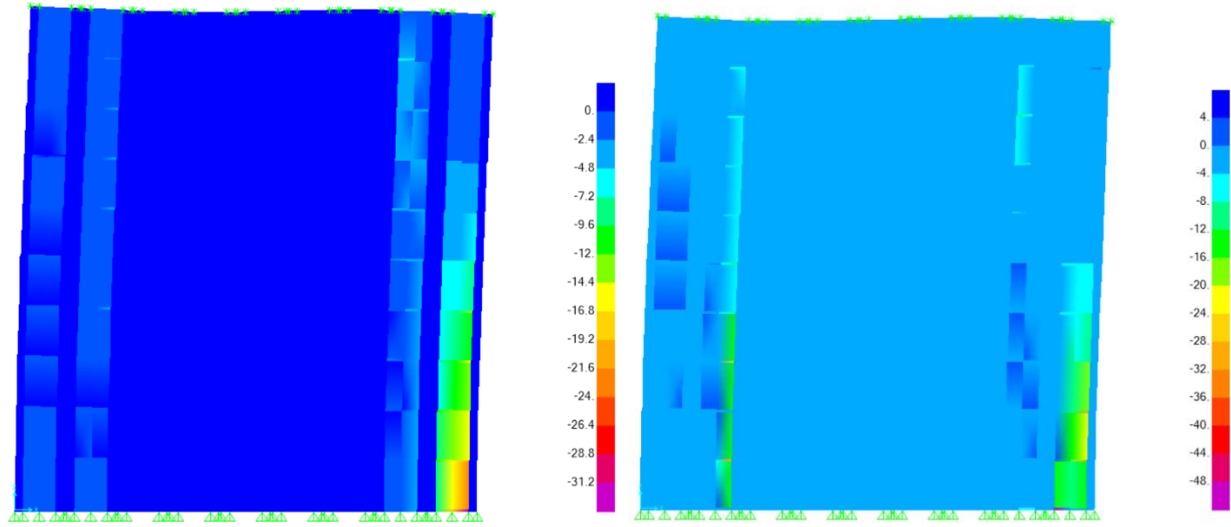


Figure A265.6 Point 3 (Left), Point 4 (Right), Concrete, SMin

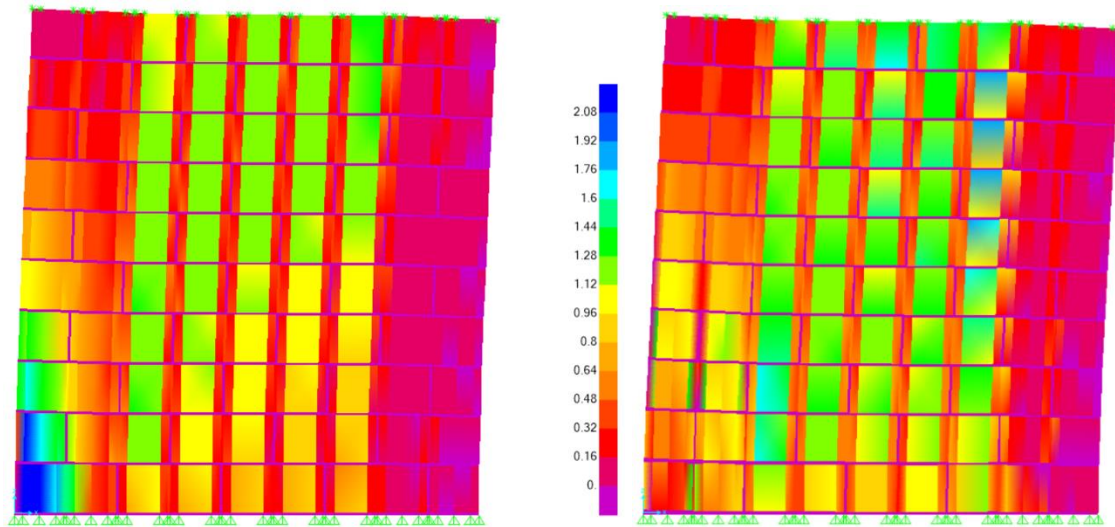


Figure A266.7 Point 1 (Left), Point 2 (Right), Masonry, SMax

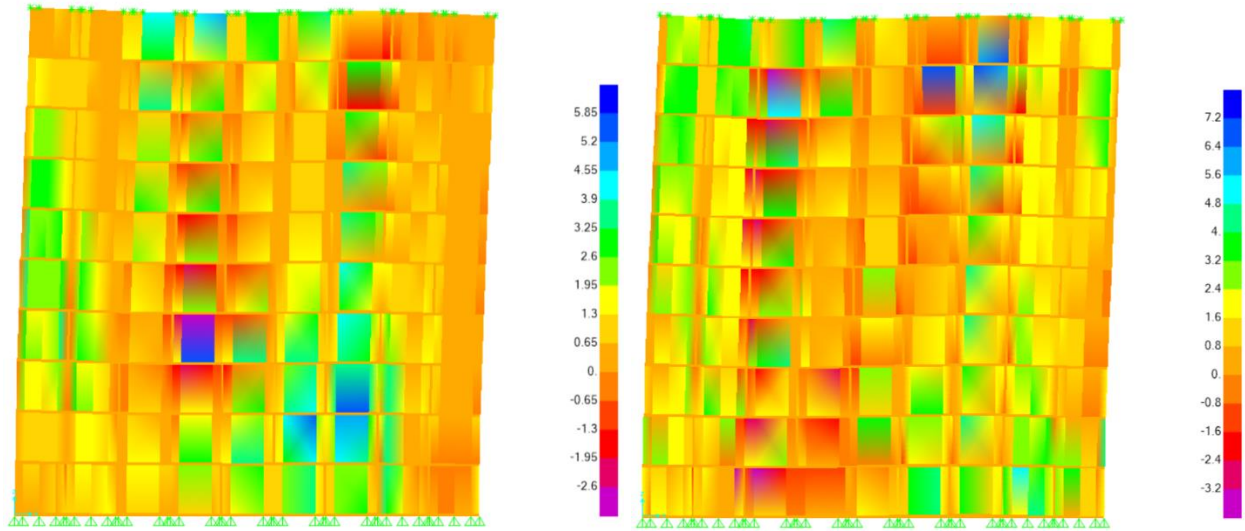


Figure A267.8 Point 3 (Left), Point 4 (Right), Masonry, SMax

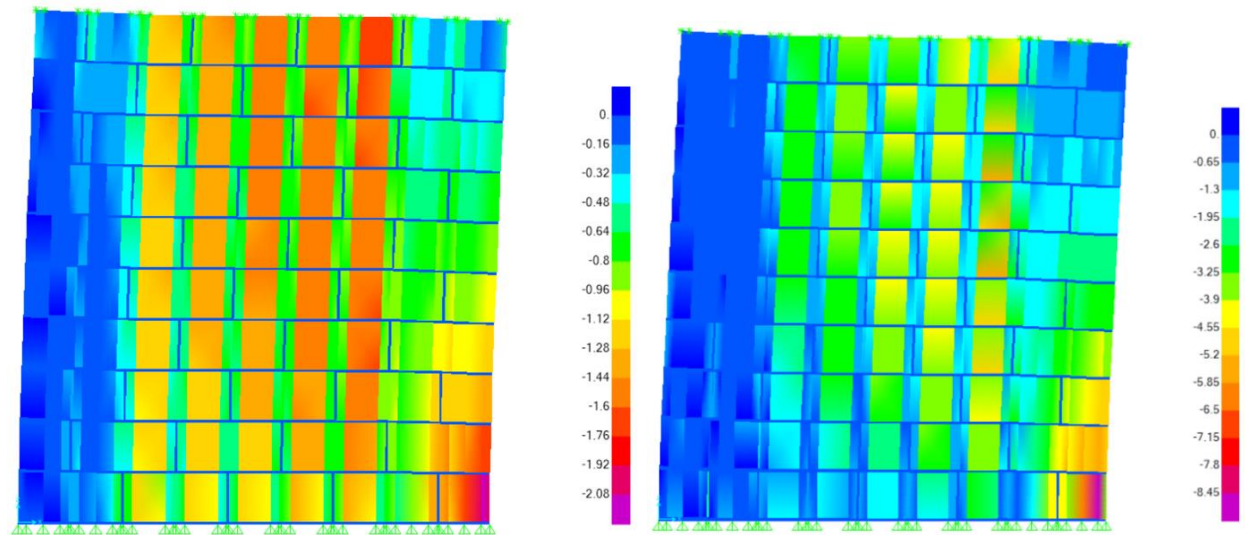


Figure A268.9 Point 1 (Left), Point 2 (Right), Masonry, SMin

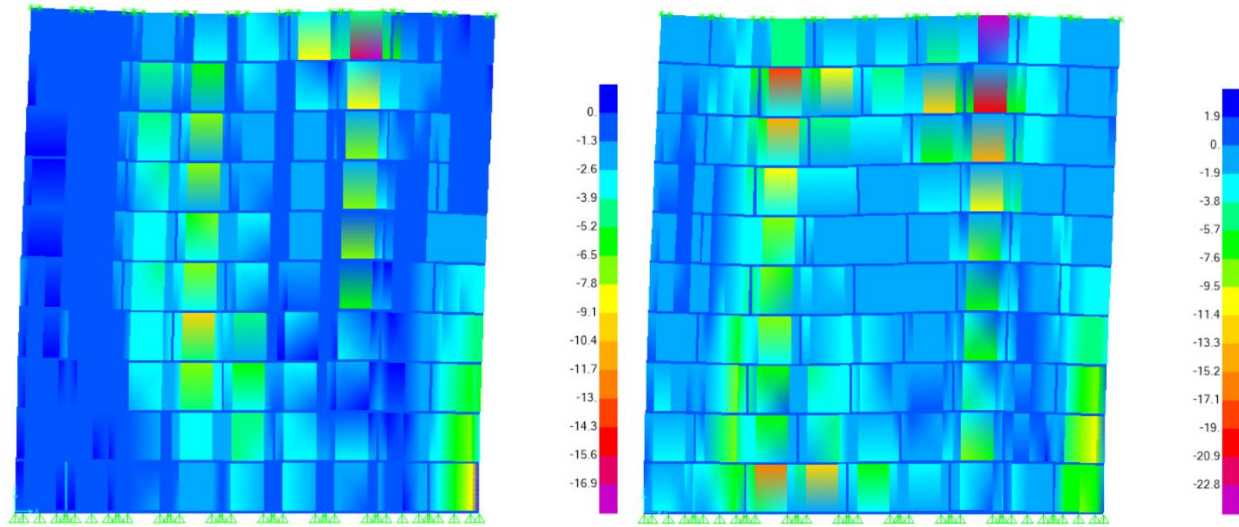


Figure A269.10 Point 3 (Left), Point 4 (Right), Masonry, SMin

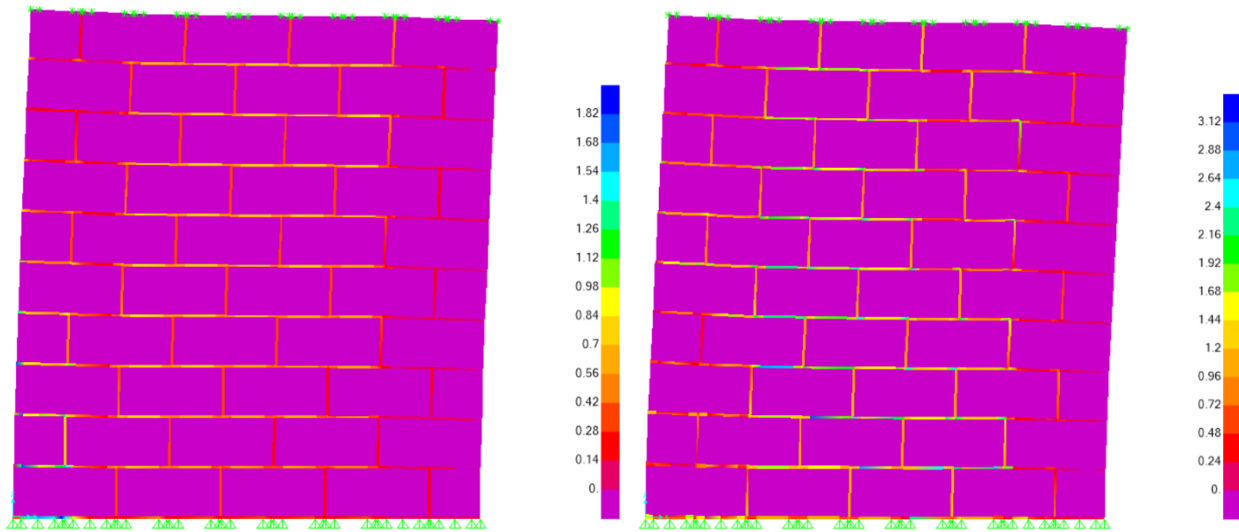


Figure A270.11 Point 1 (Left), Point 2 (Right), Mortar, SMax

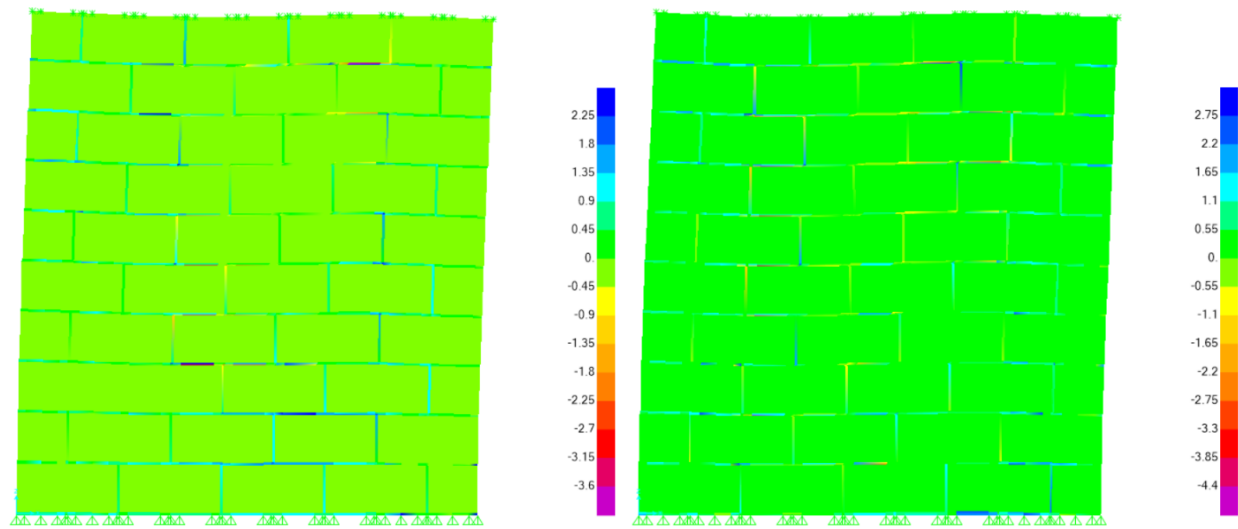


Figure A271.12 Point 3 (Left), Point 4 (Right), Mortar, SMax

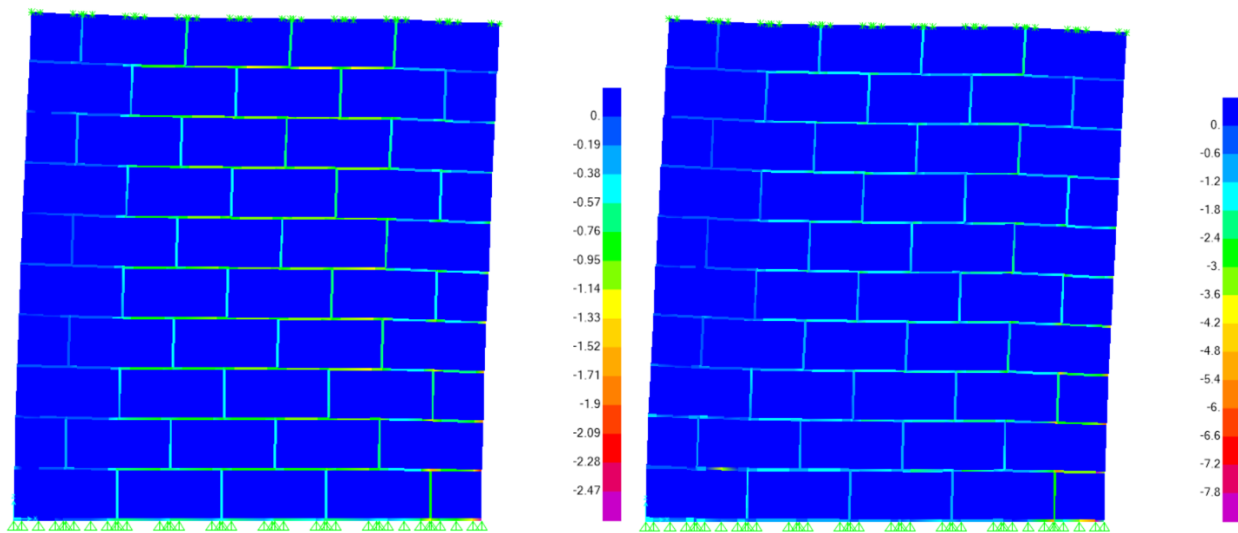


Figure A272.13 Point 1 (Left), Point 2 (Right), Mortar, SMin

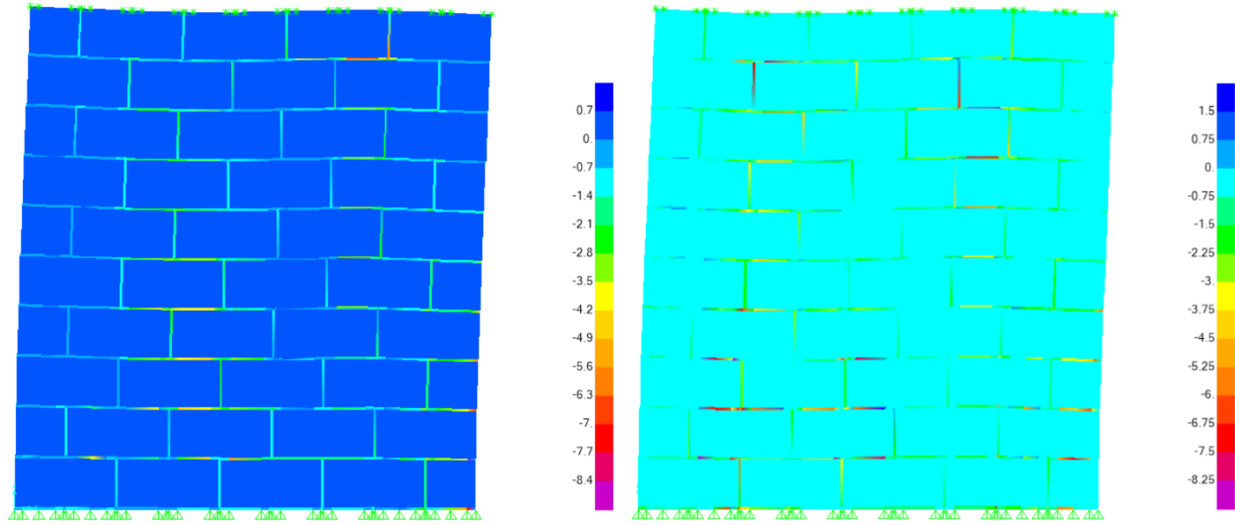


Figure A273.14 Point 3 (Left), Point 4 (Right), Mortar, SMin

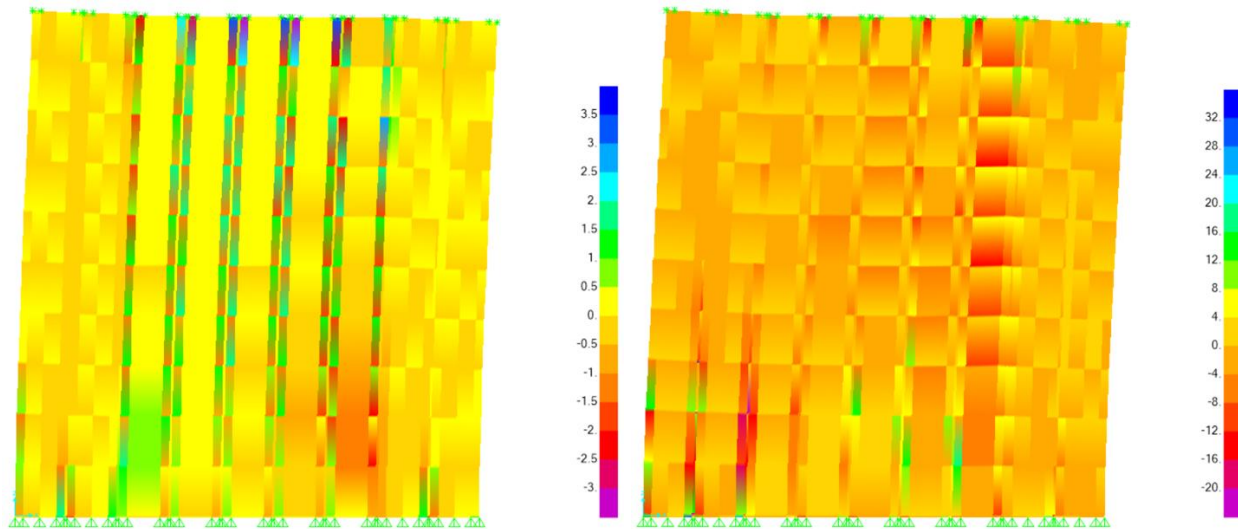


Figure A274.15 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

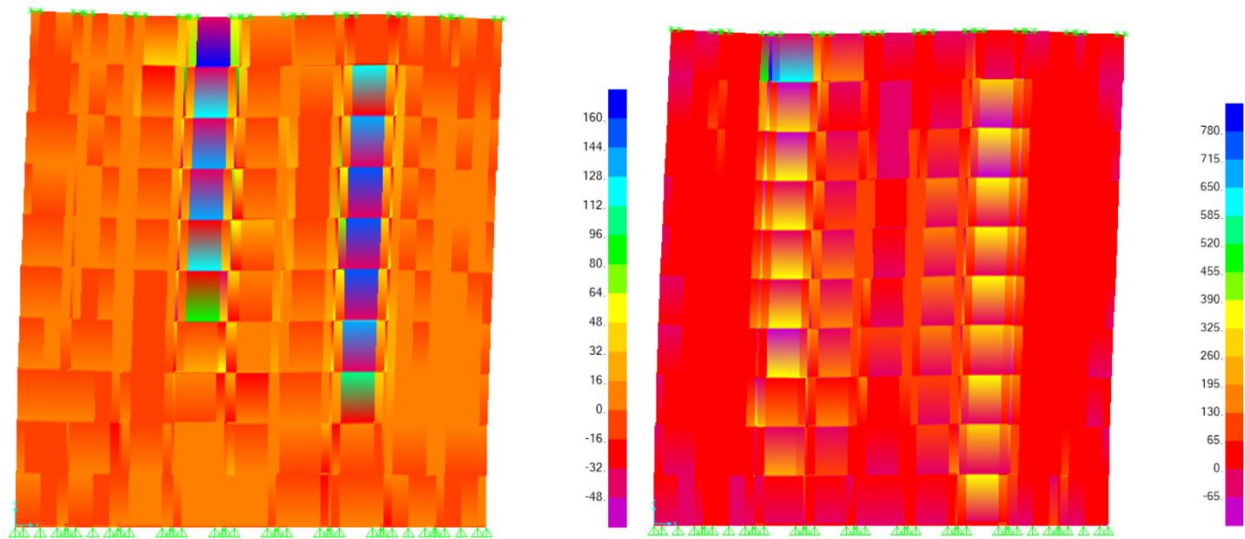


Figure A275.16 Point 3 (Left), Point 4 (Right), Horizontal FRP, S11

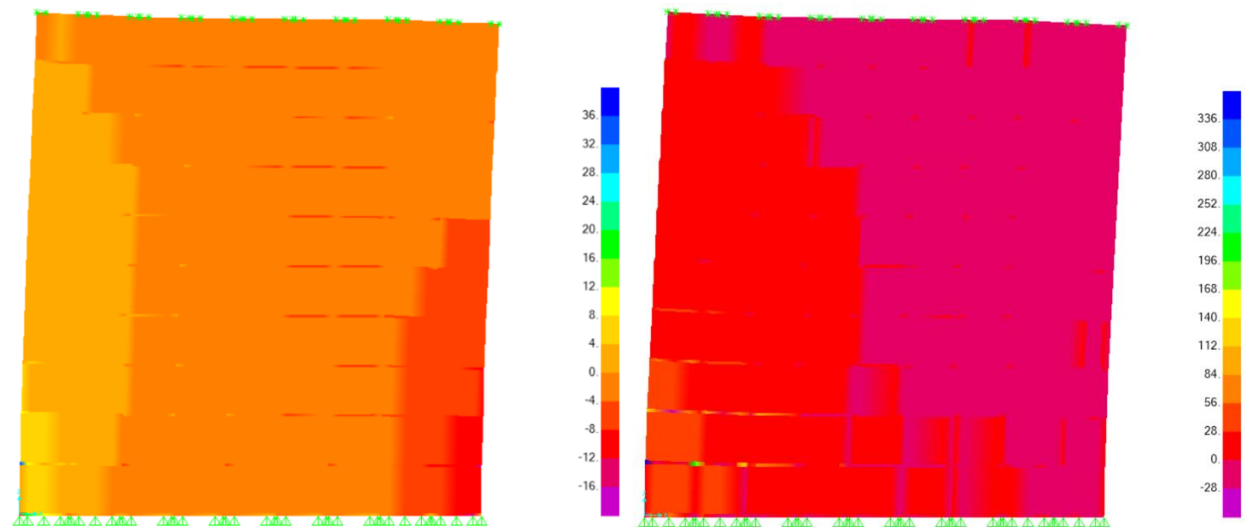


Figure A276.17 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

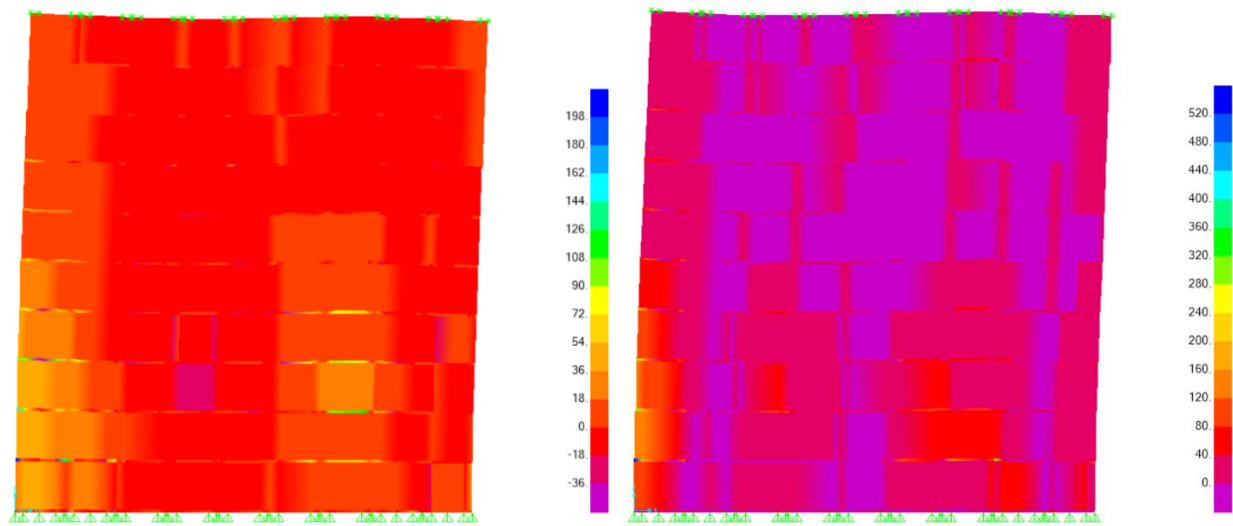
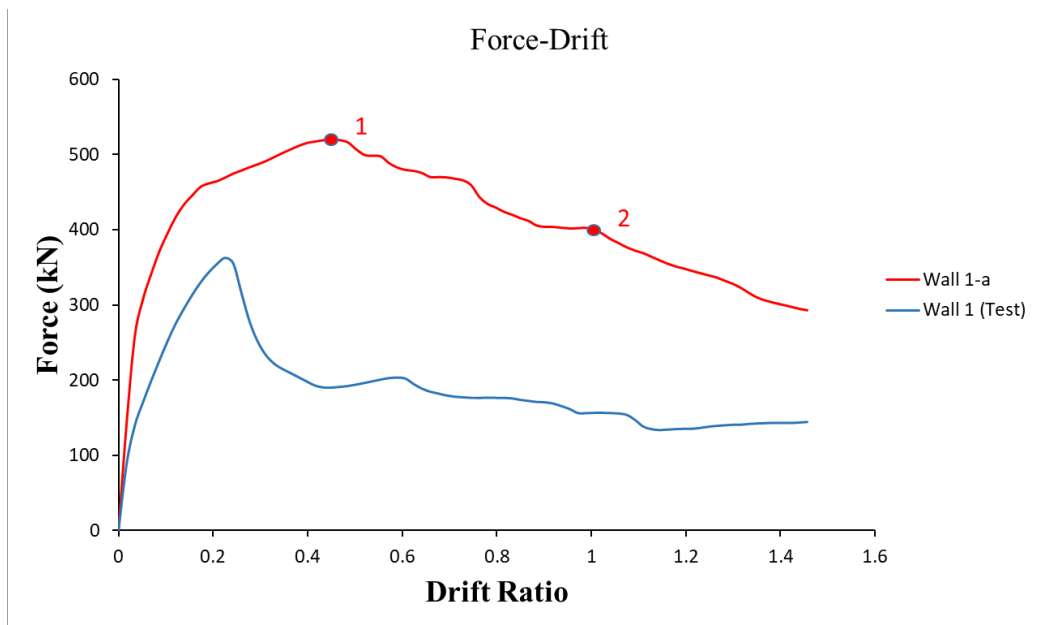


Figure A277.18 Point 3 (Left), Point 4 (Right), Vertical FRP, S22

Wall 1 with Concrete Filled Web (Wall 1-a)



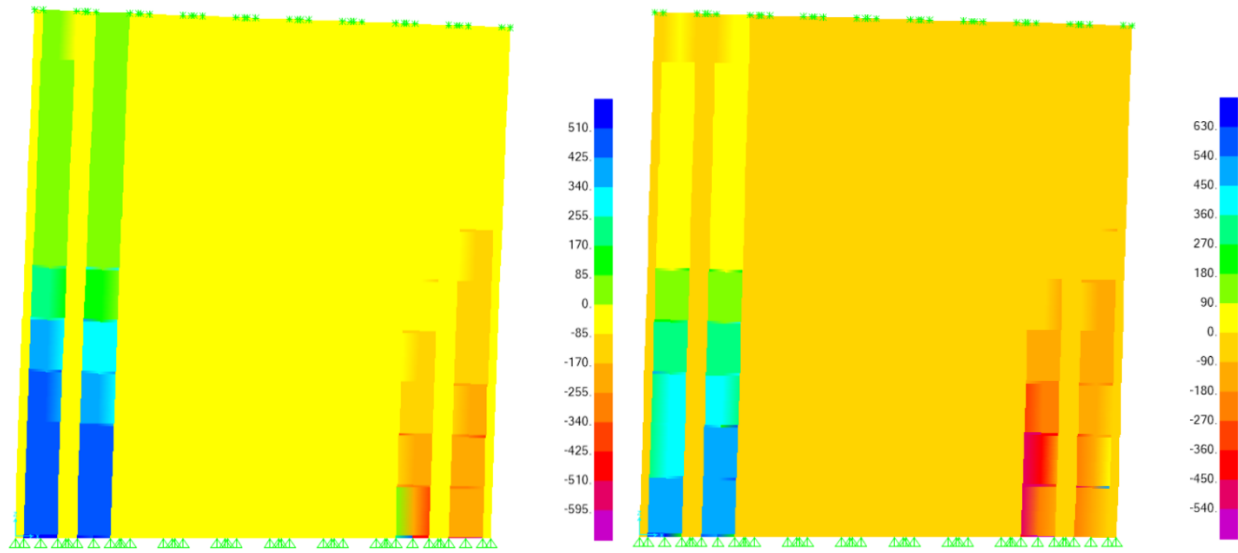


Figure A278.19 Point 1 (Left), and Point 2 (Right), Steel, S22

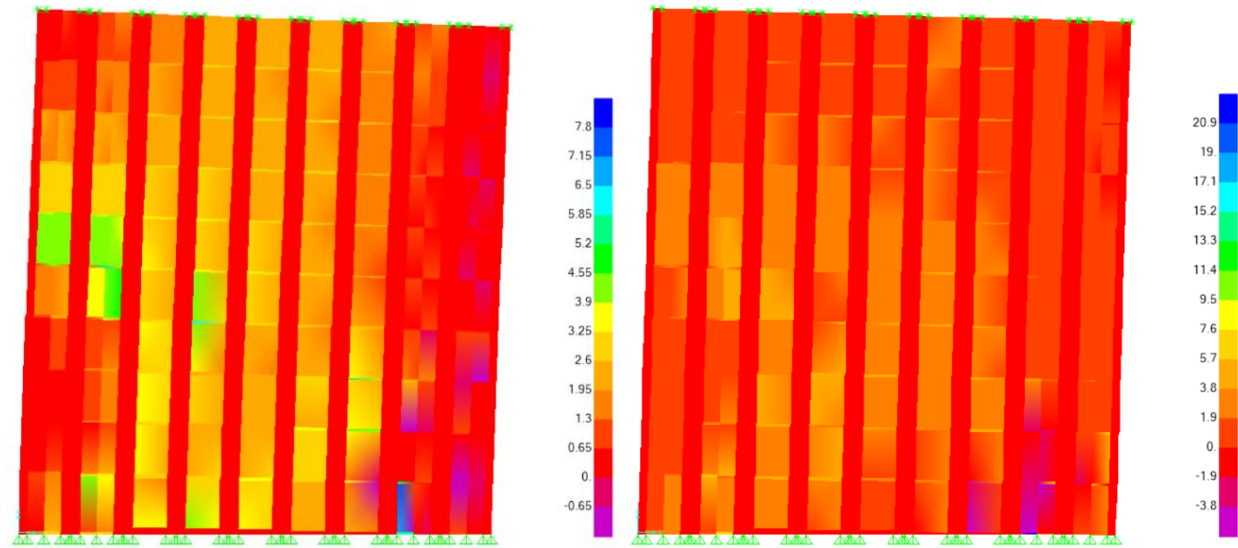


Figure A279.20 Point 1 (Left), and Point 2 (Right), Concrete, SMax

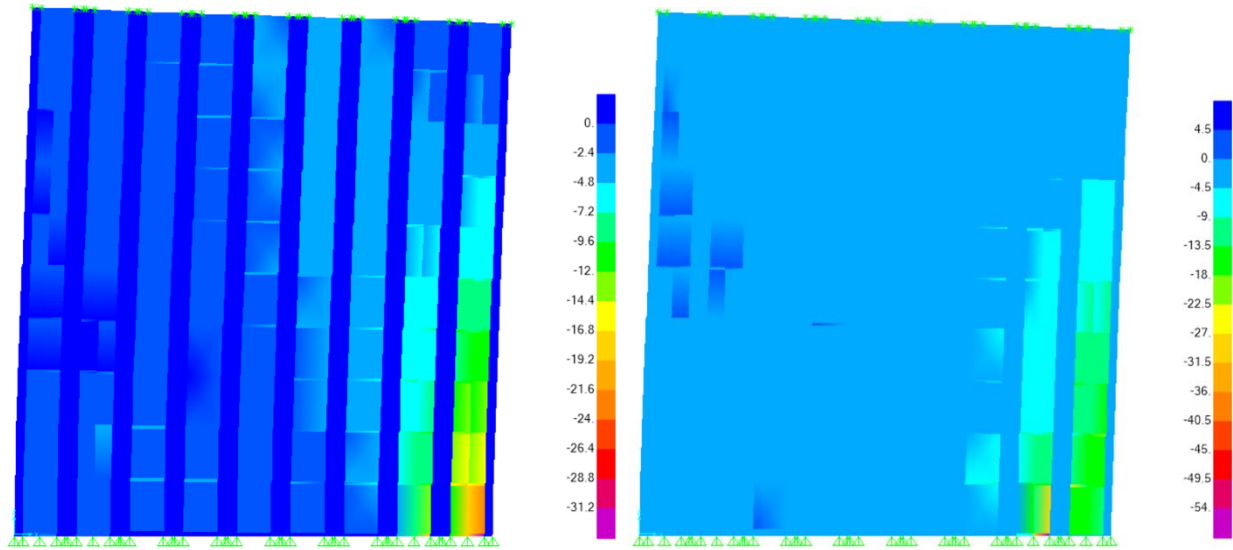


Figure A280.21 Point 1 (Left), and Point 2 (Right), Concrete, SMin

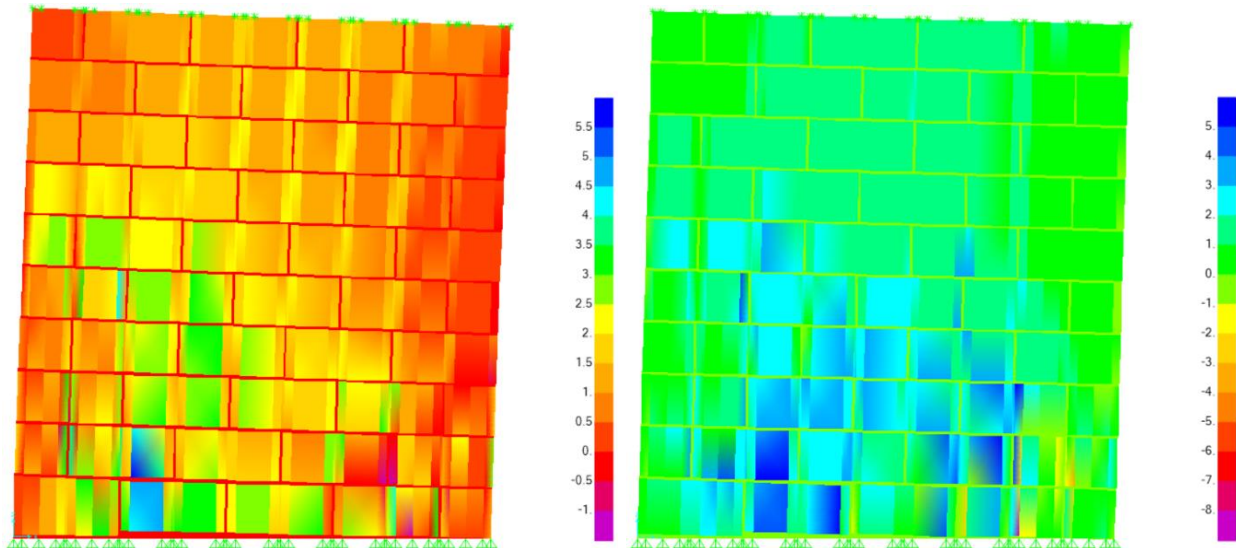


Figure A281.22 Point 1 (Left), Point 2 (Right), Masonry, SMax

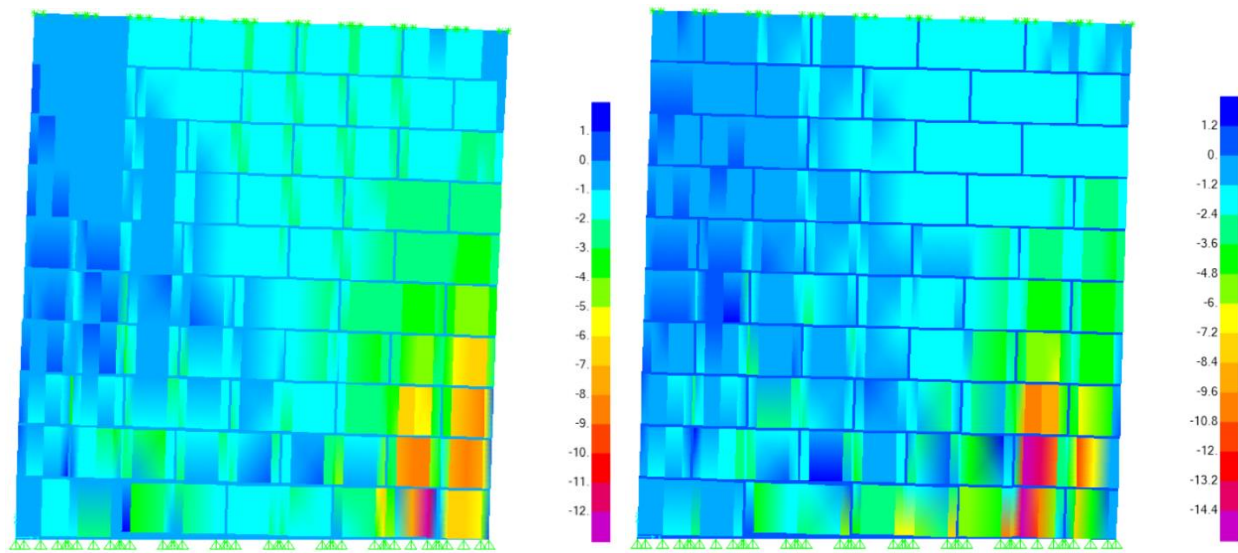


Figure A282.23 Point 1 (Left), Point 2 (Right), Masonry, SMin

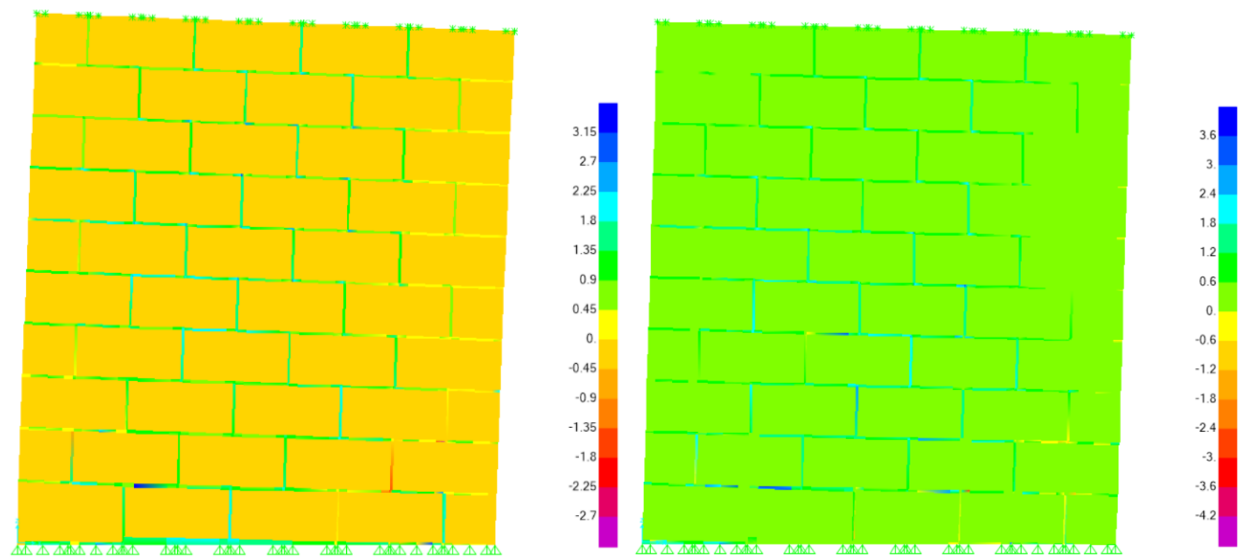


Figure A283.24 Point 1 (Left), Point 2 (Right), Mortar, SMax

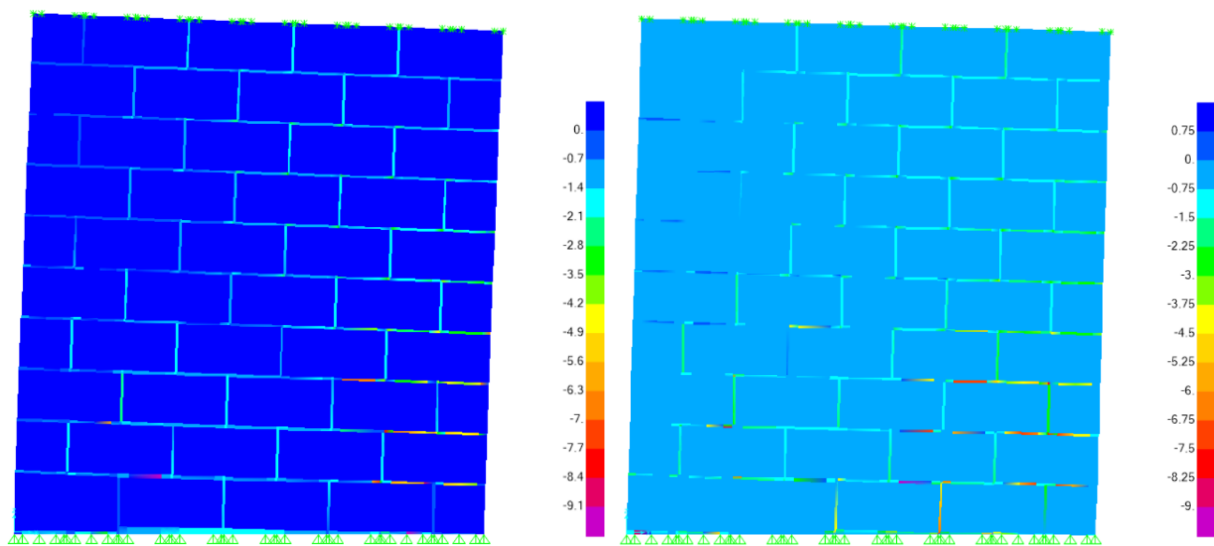


Figure A284.25 Point 1 (Left), Point 2 (Right), Mortar, SMin

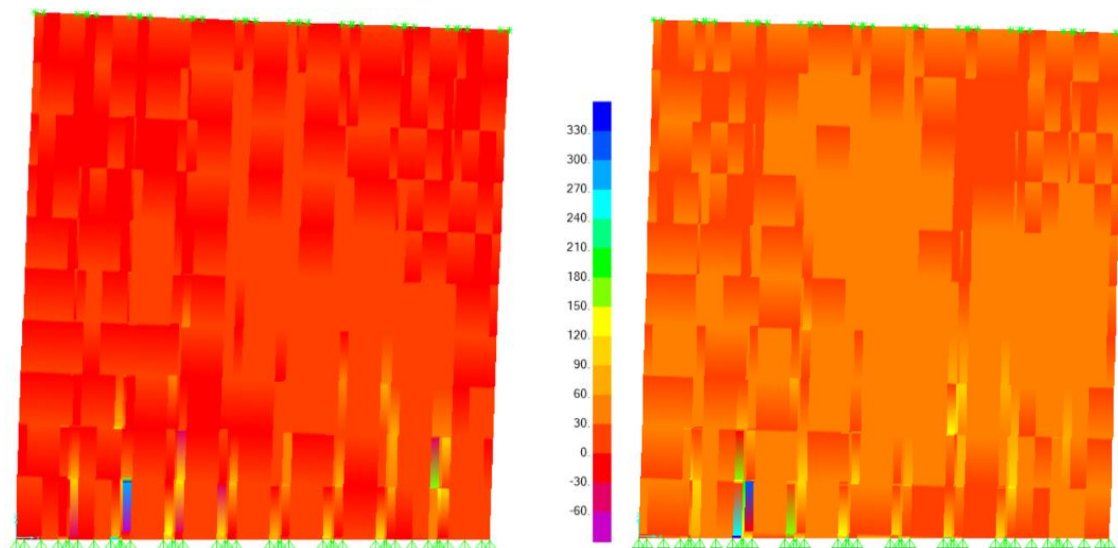


Figure A285.26 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

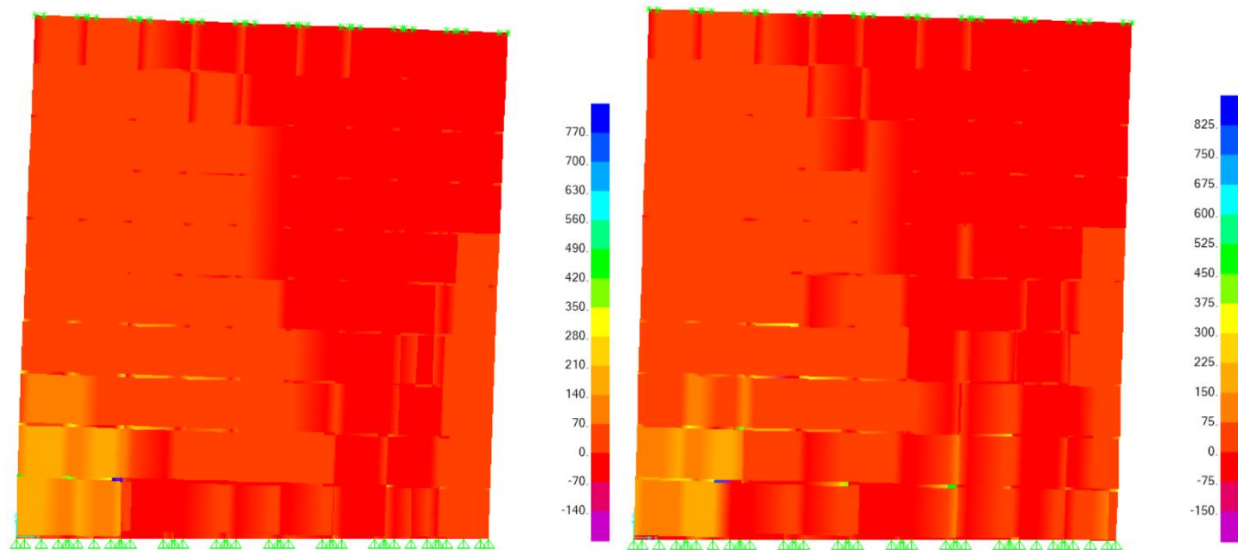
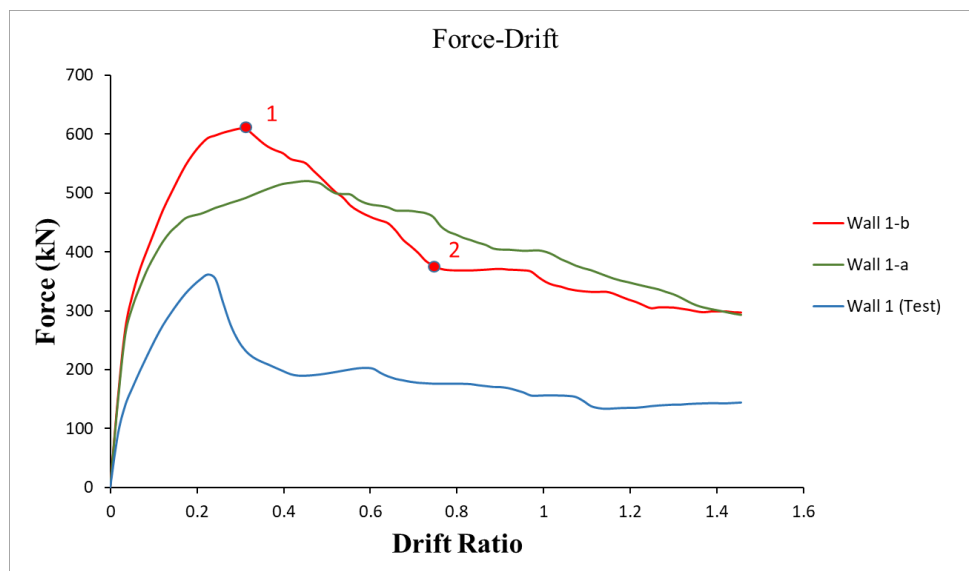


Figure A286.27 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

Wall 1 with Concrete Filled Web and Increased Boundary Element Reinforcement (Wall 1-

b)



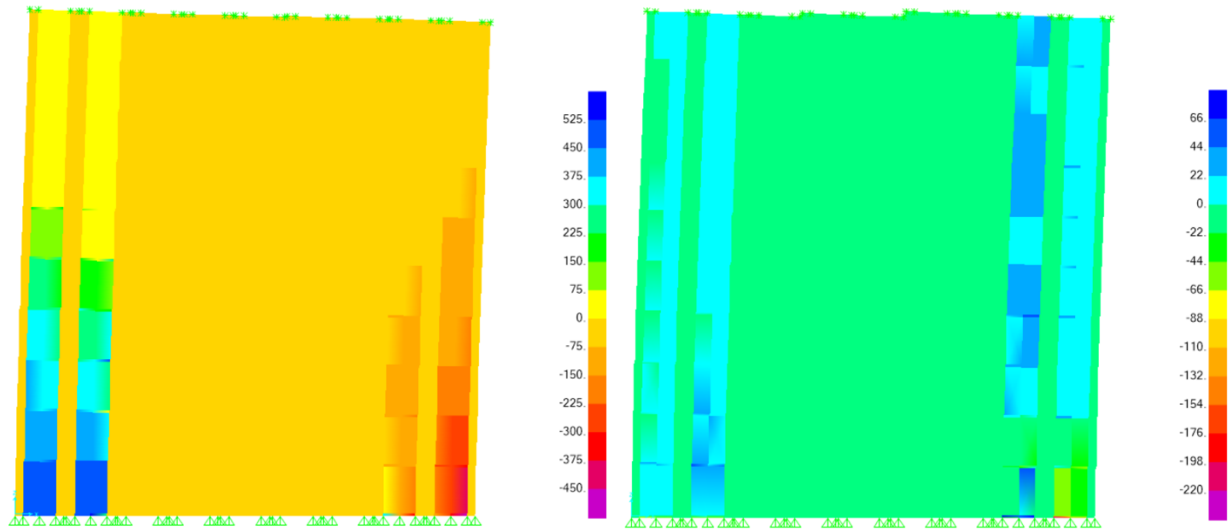


Figure A287.28 Point 1 (Left), and Point 2 (Right), Steel, S22

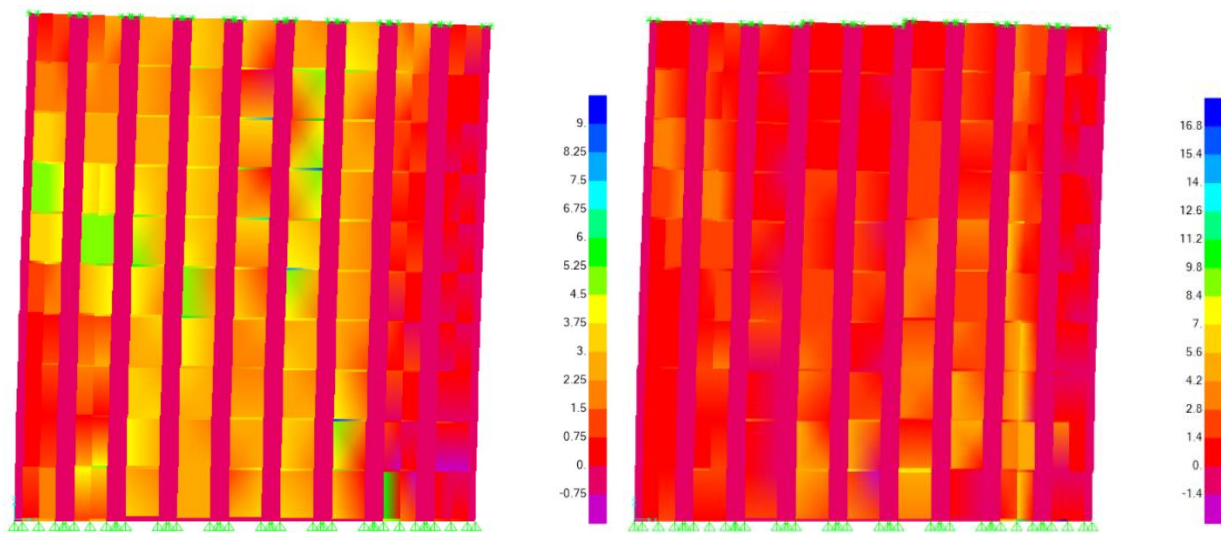


Figure A288.29 Point 1 (Left), and Point 2 (Right), Concrete, SMax

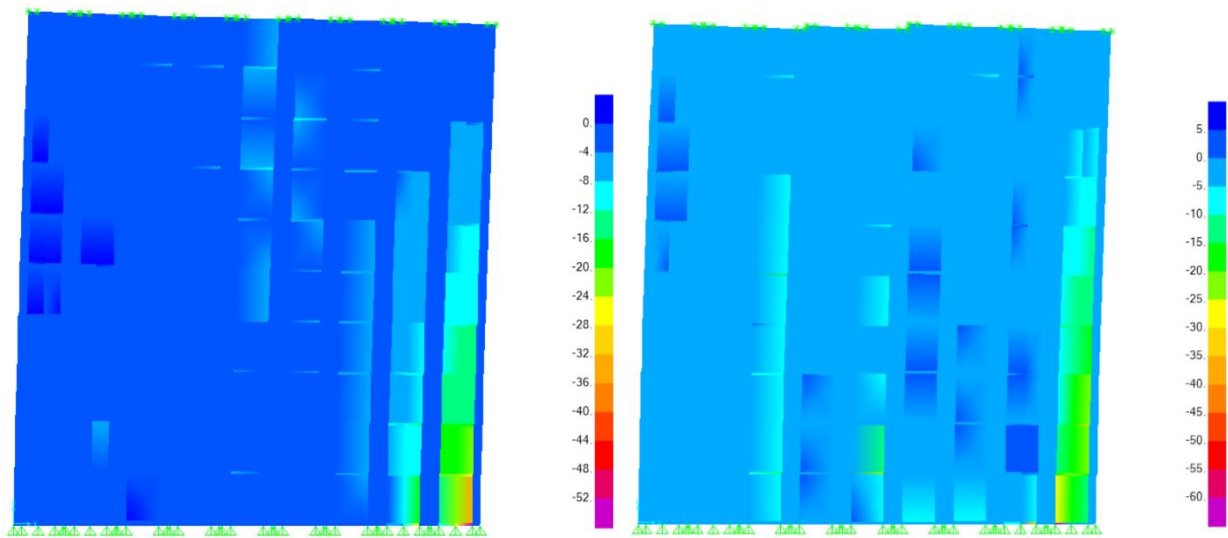


Figure A289.30 Point 1 (Left), and Point 2 (Right), Concrete, SMin

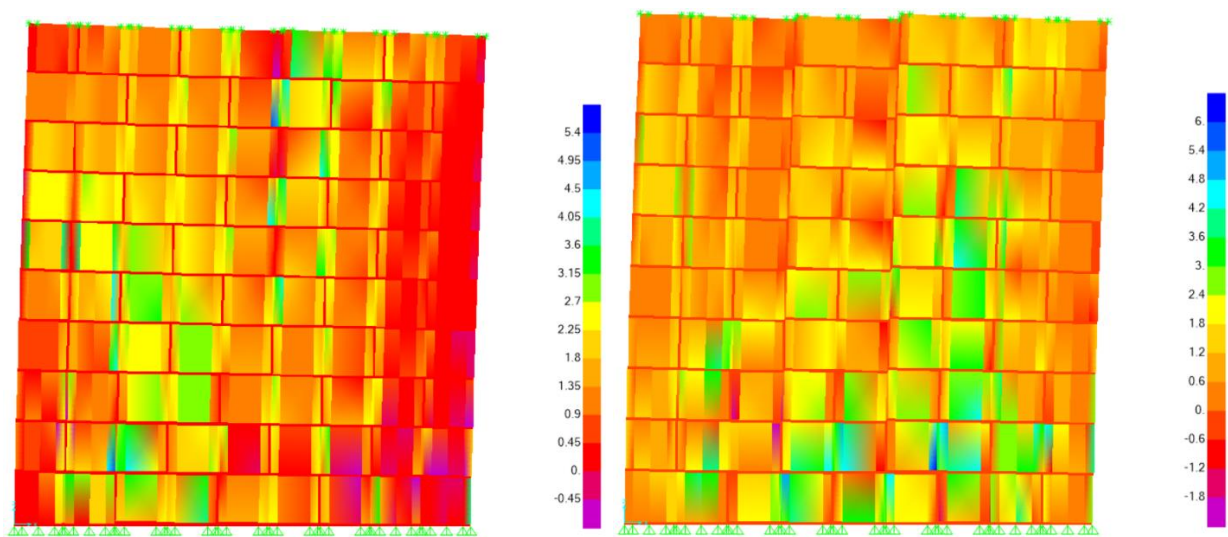


Figure A290.31 Point 1 (Left), Point 2 (Right), Masonry, SMax

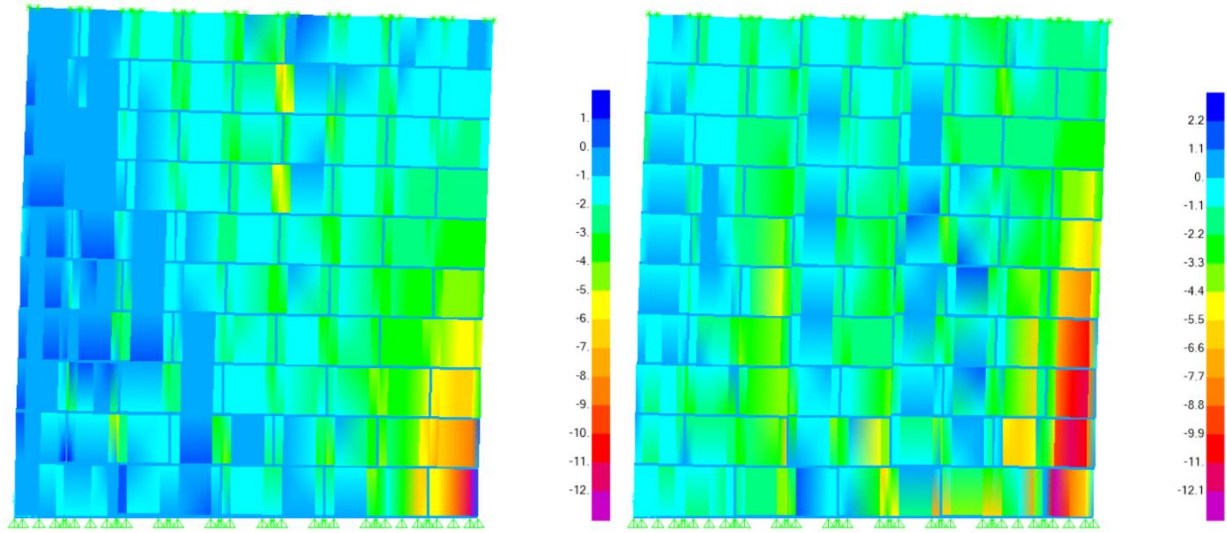


Figure A291.32 Point 1 (Left), Point 2 (Right), Masonry, SMin

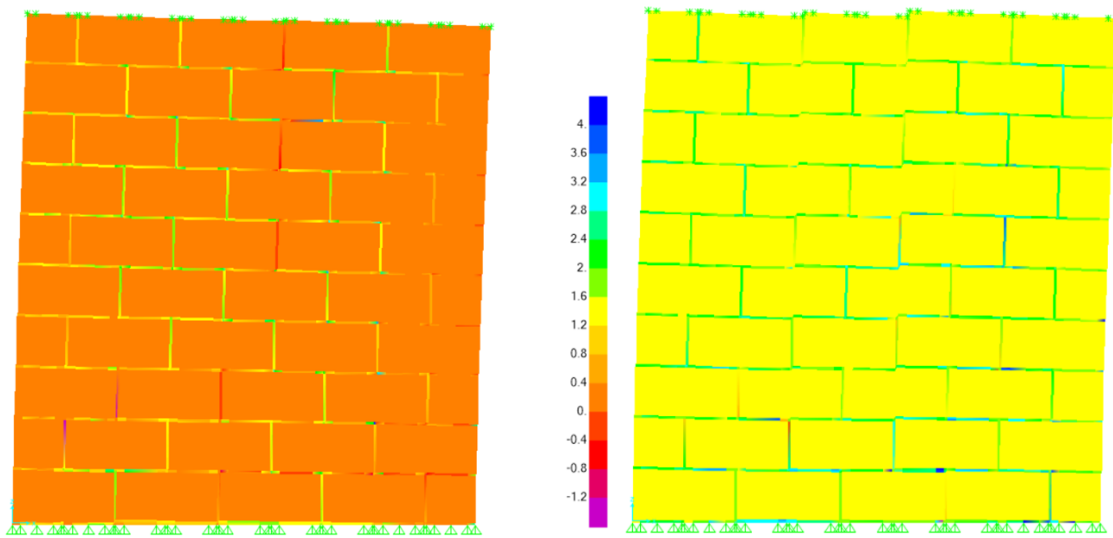


Figure A292.33 Point 1 (Left), Point 2 (Right), Mortar, SMax

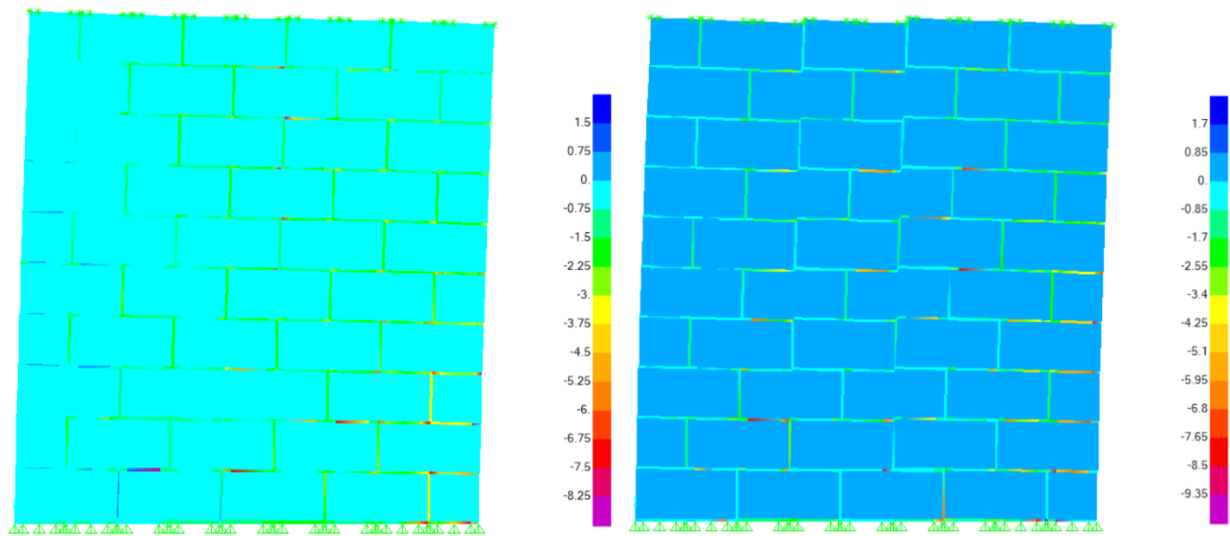


Figure A293.34 Point 1 (Left), Point 2 (Right), Mortar, SMin

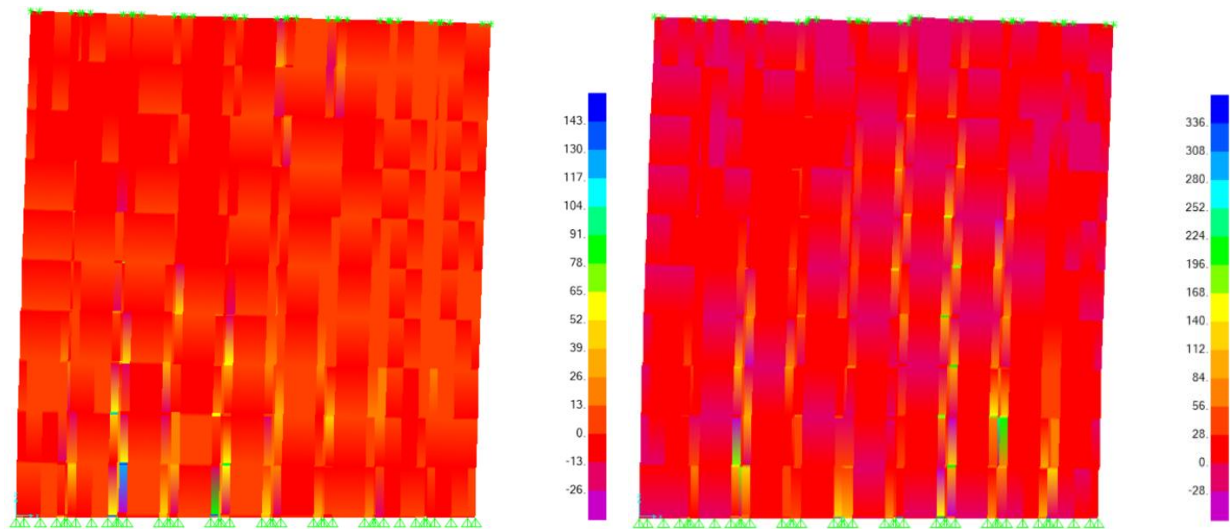


Figure A294.35 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

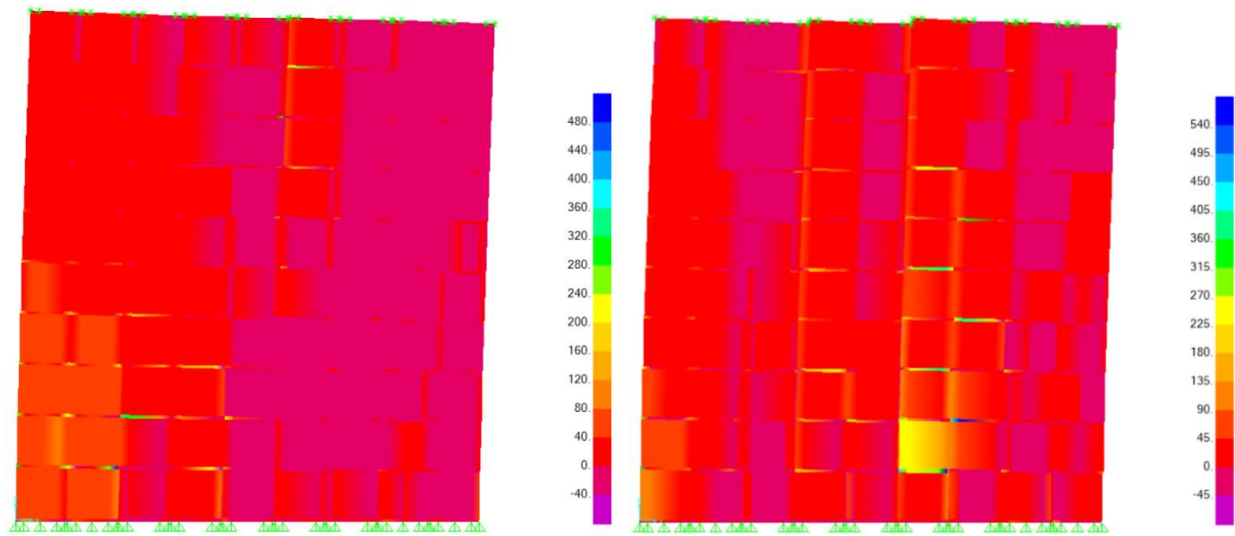
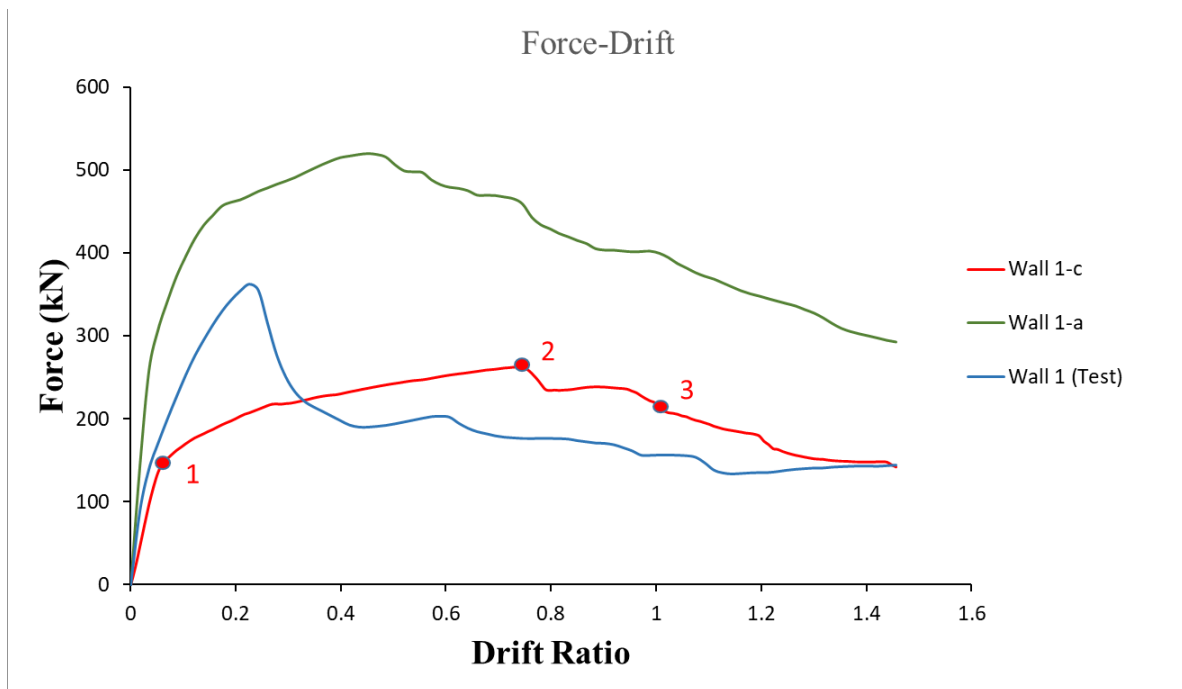


Figure A295.36 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

Wall 1 with Concrete Filled Web and Wall Aspect Ratio of 2.0 (Wall 1-c)



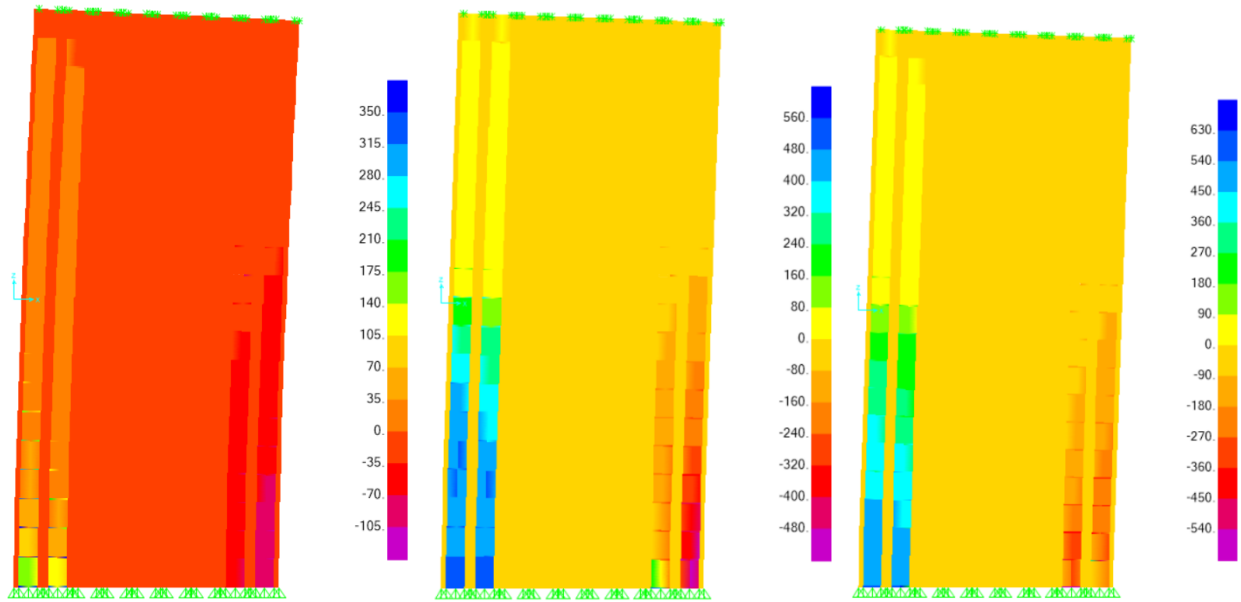


Figure A296.37 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Steel, S22

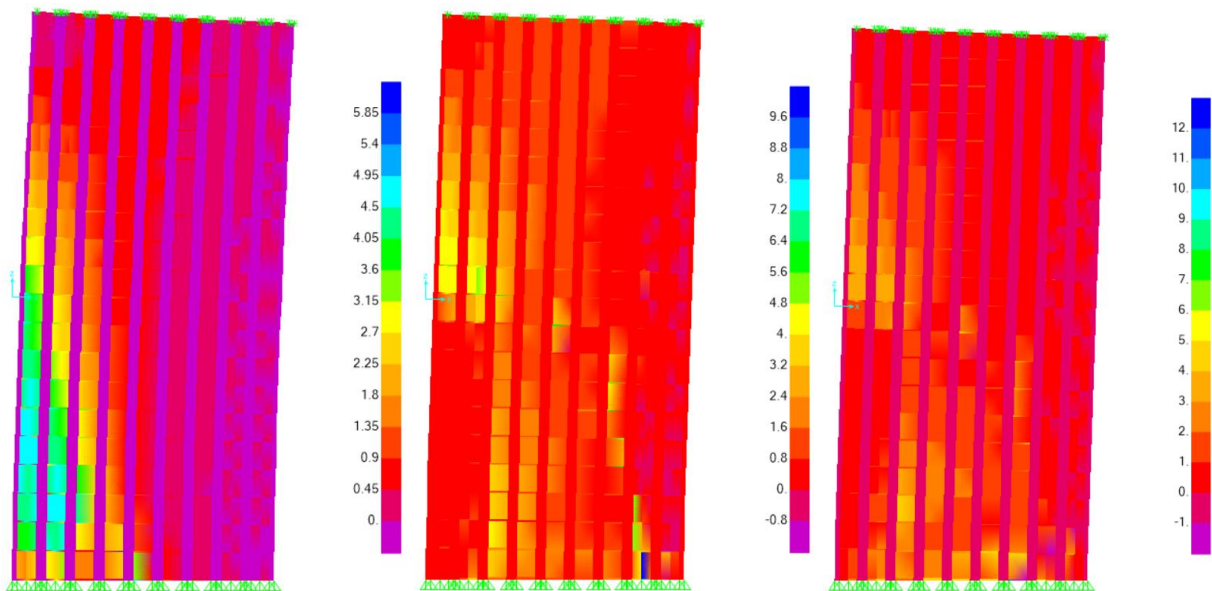


Figure A297.38 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Concrete, SMax

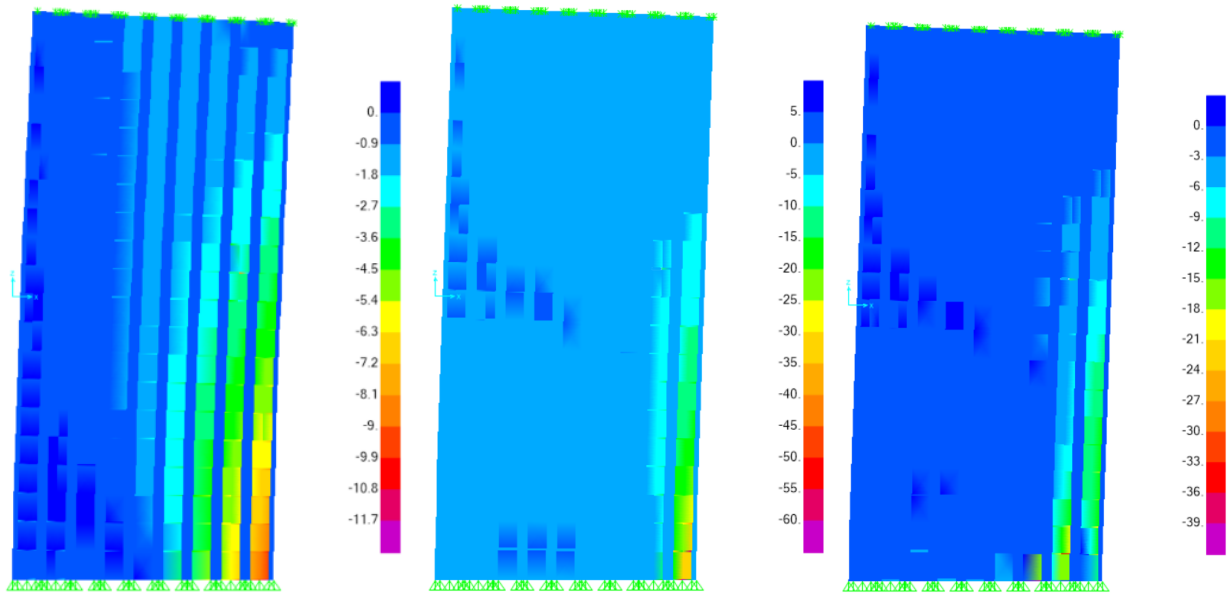


Figure A298.39 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Concrete, SMin

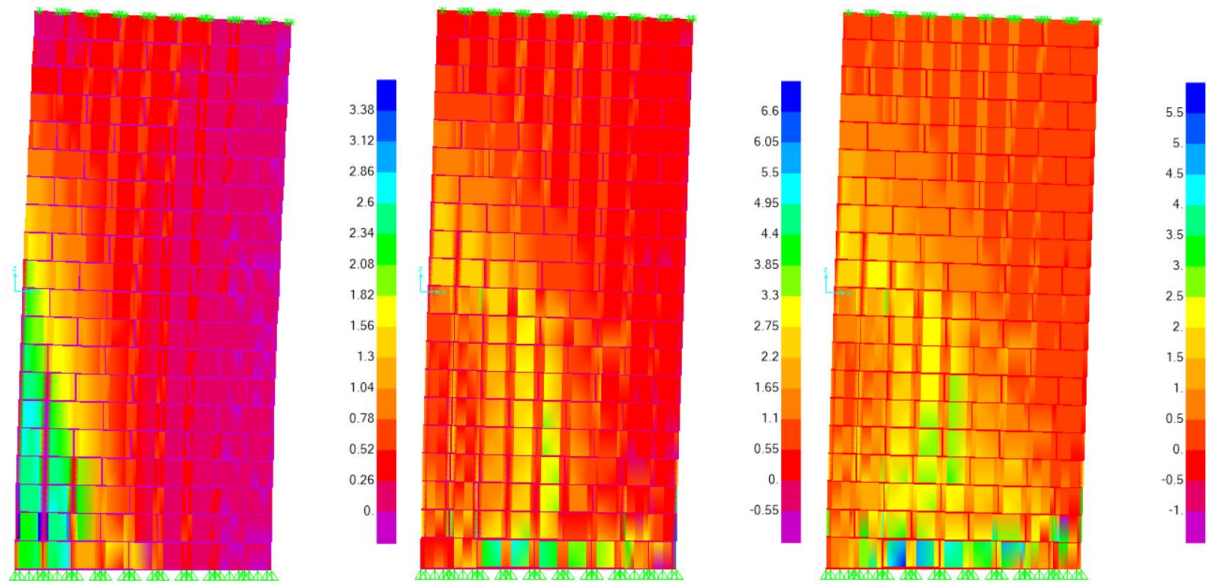


Figure A299.40 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Masonry, SMax

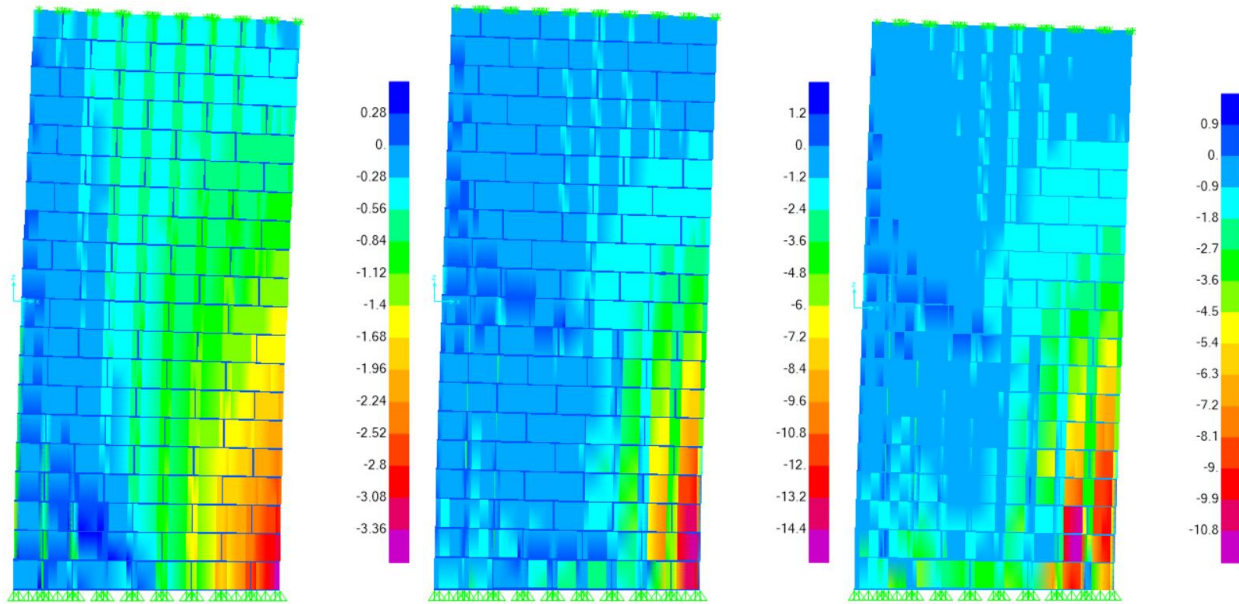


Figure A300.41 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Masonry, SMin

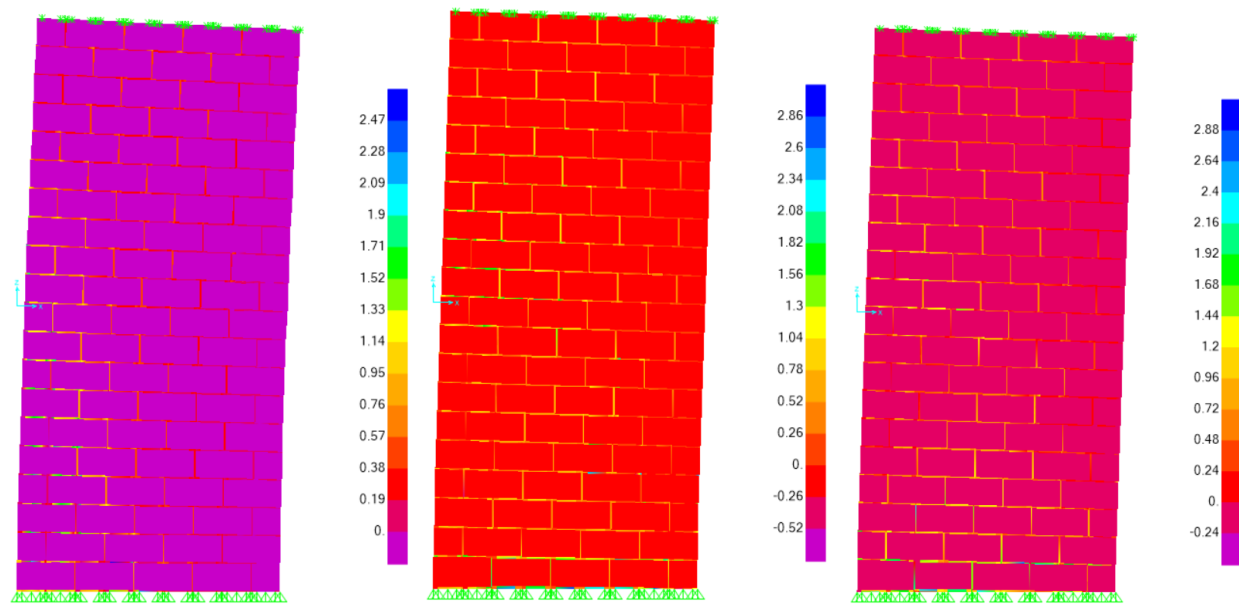


Figure A301.42 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Mortar, SMax

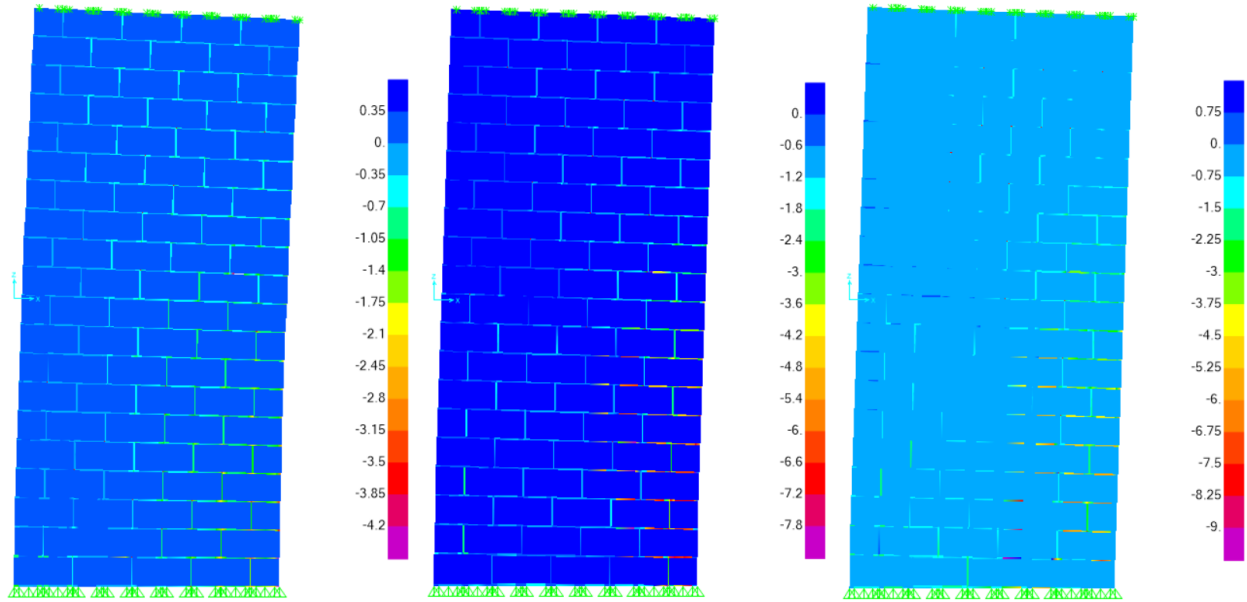


Figure A302.43 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Mortar, SMin

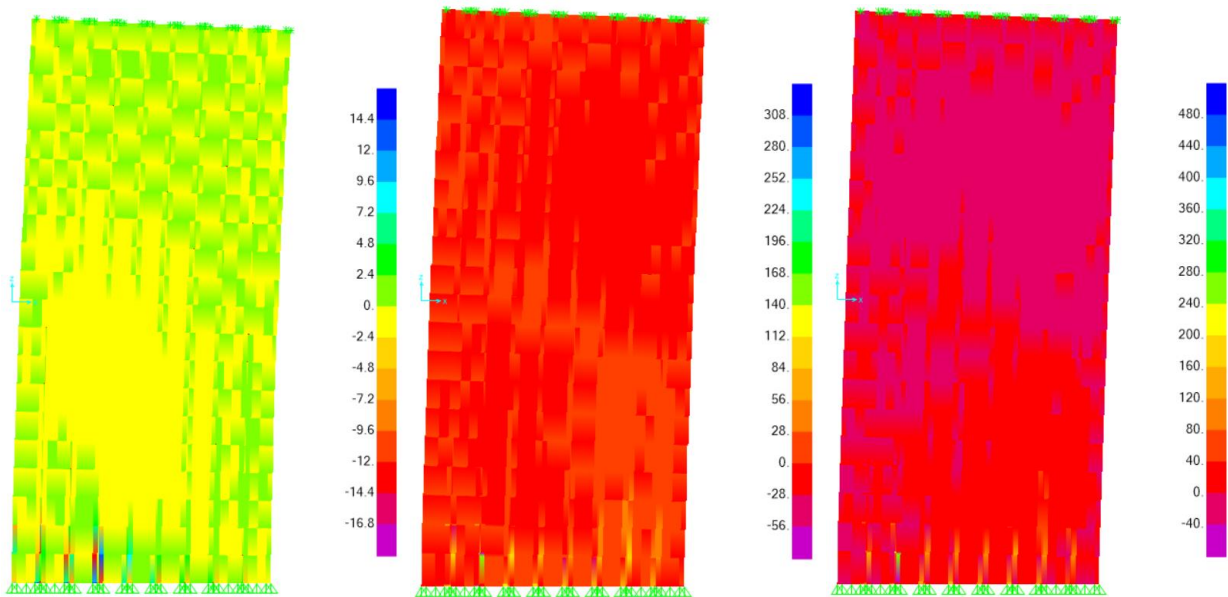


Figure A303.44 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Horizontal FRP, S11

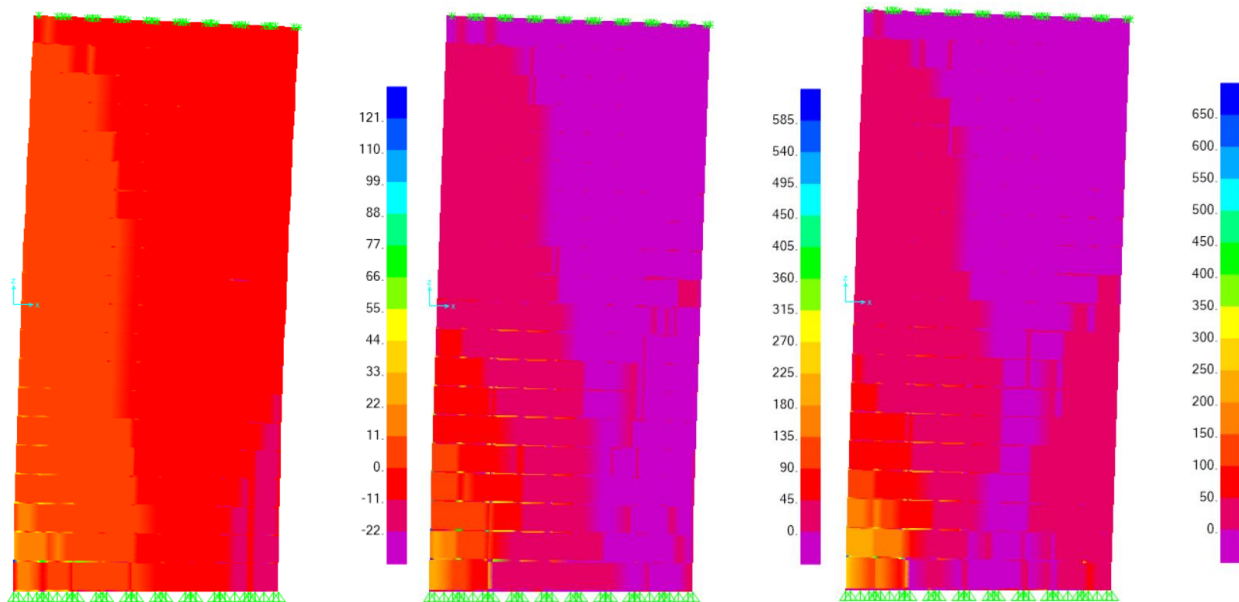
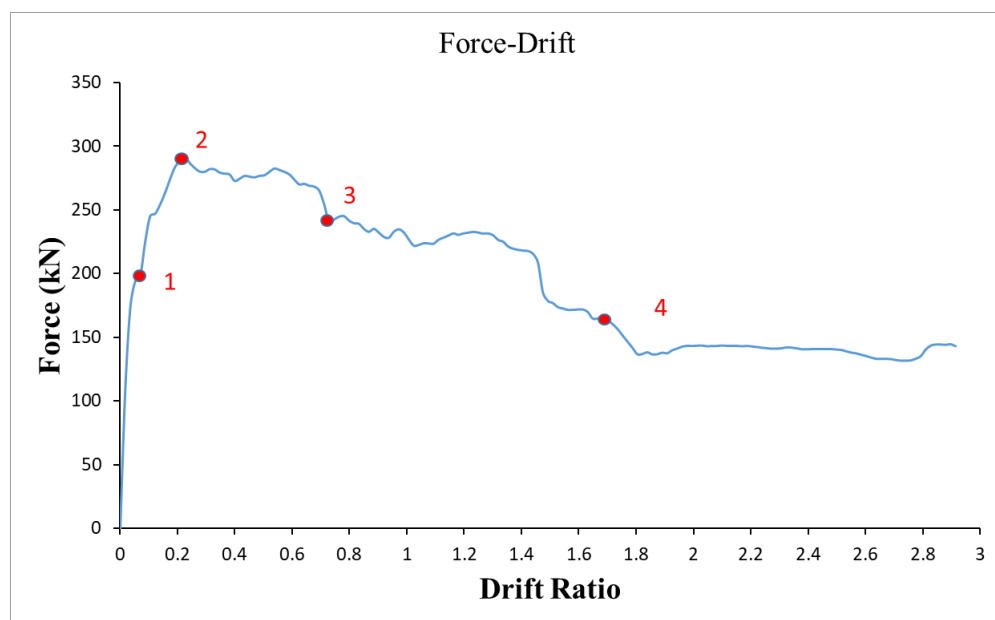


Figure A304.45 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Vertical FRP, S22

Wall 2



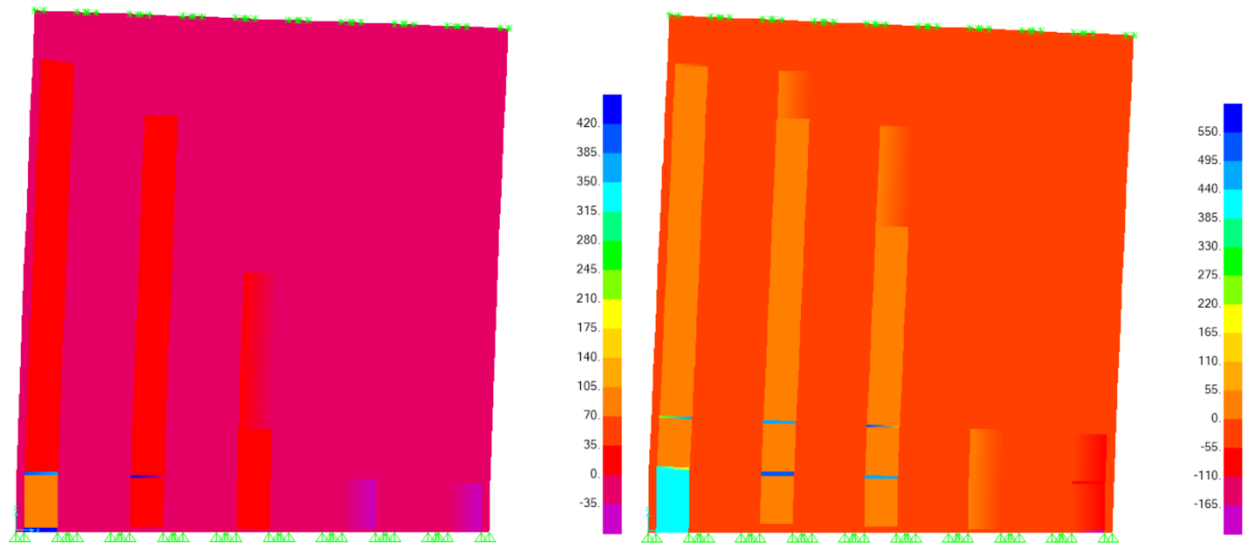


Figure A305.46 Point 1 (Left), and Point 2 (Right), Steel, S22

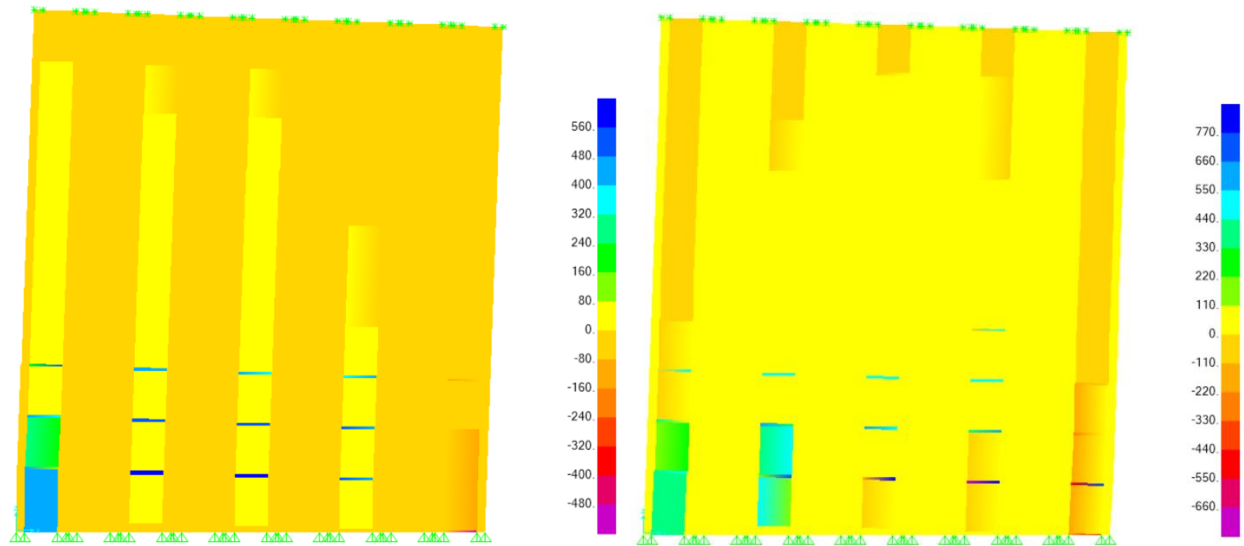


Figure A306.47 Point 3 (Left), and Point 4 (Right), Steel, S22

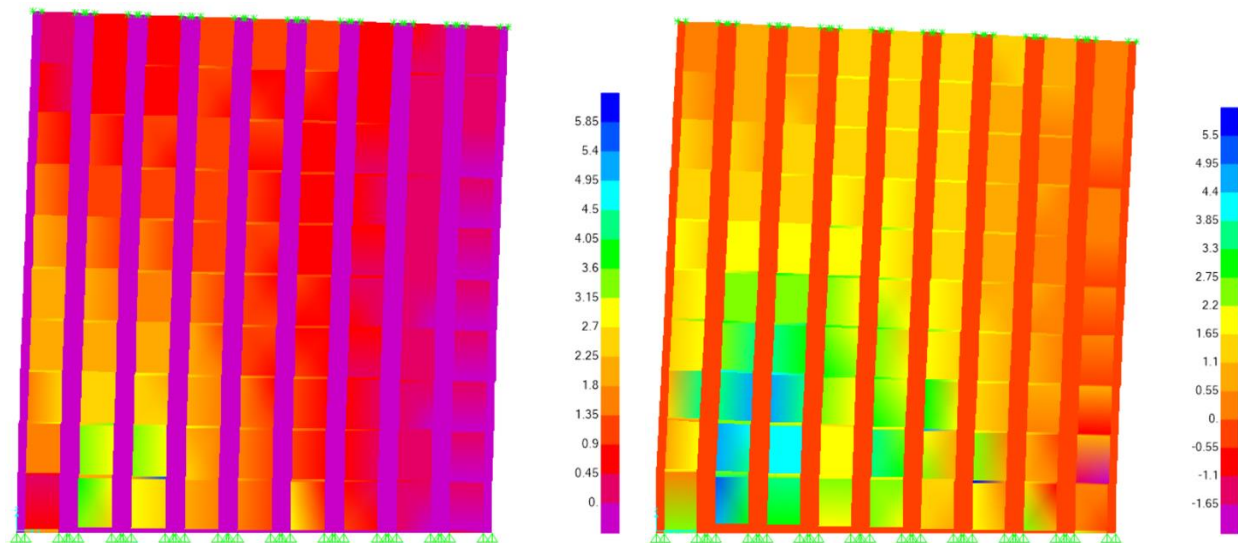


Figure A307.48 Point 1 (Left), and Point 2 (Right), Concrete, SMax

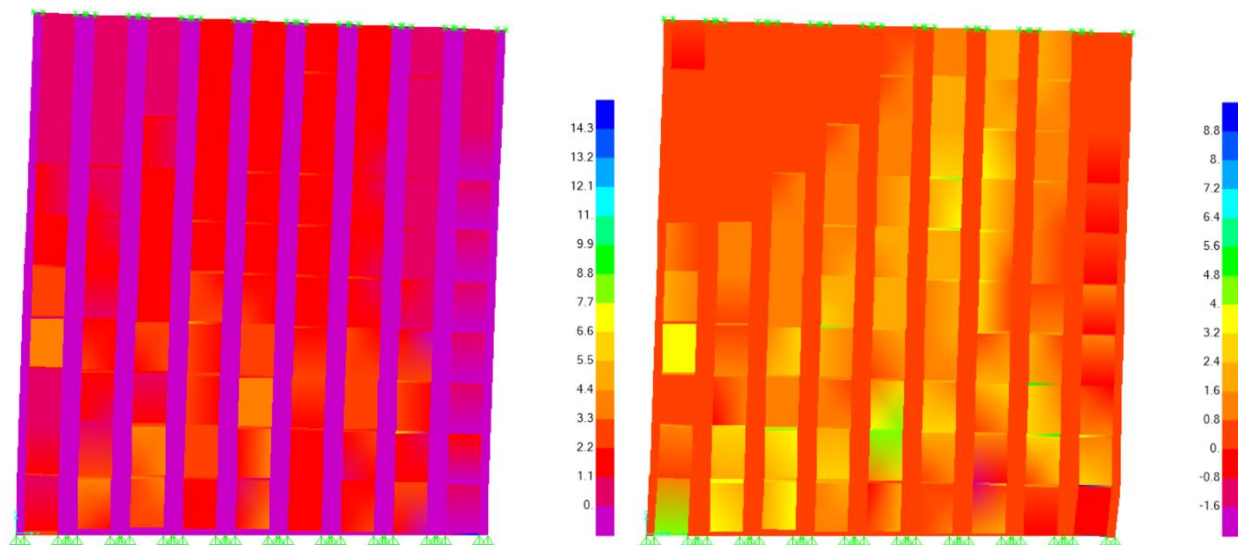


Figure A308.49 Point 3 (Left), and Point 4 (Right), Concrete, SMax

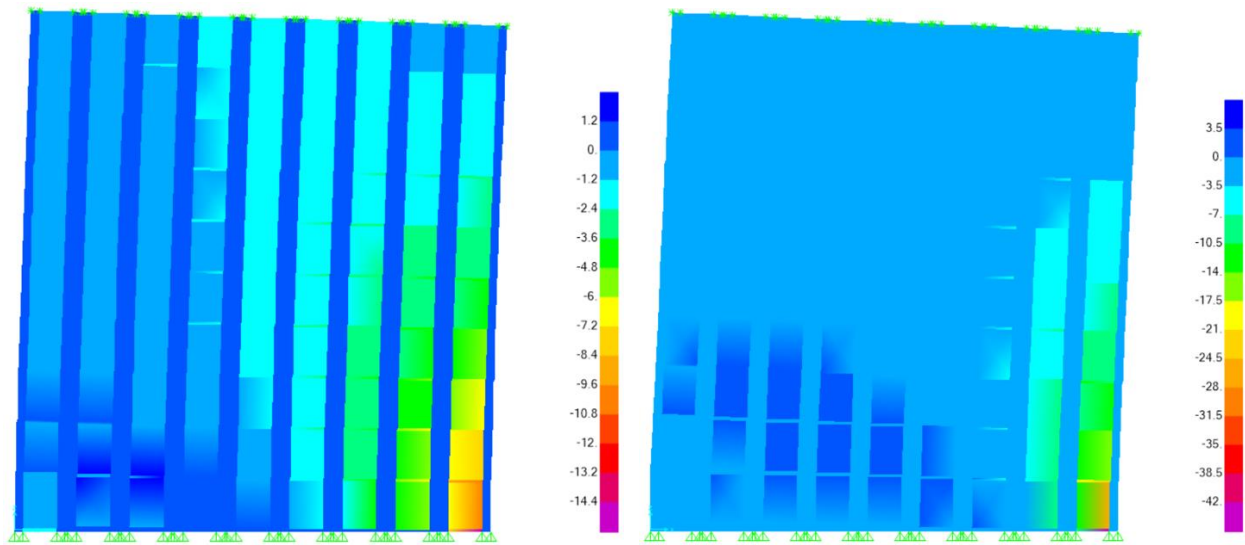


Figure A309.50 Point 1 (Left), and Point 2 (Right), Concrete, SMin

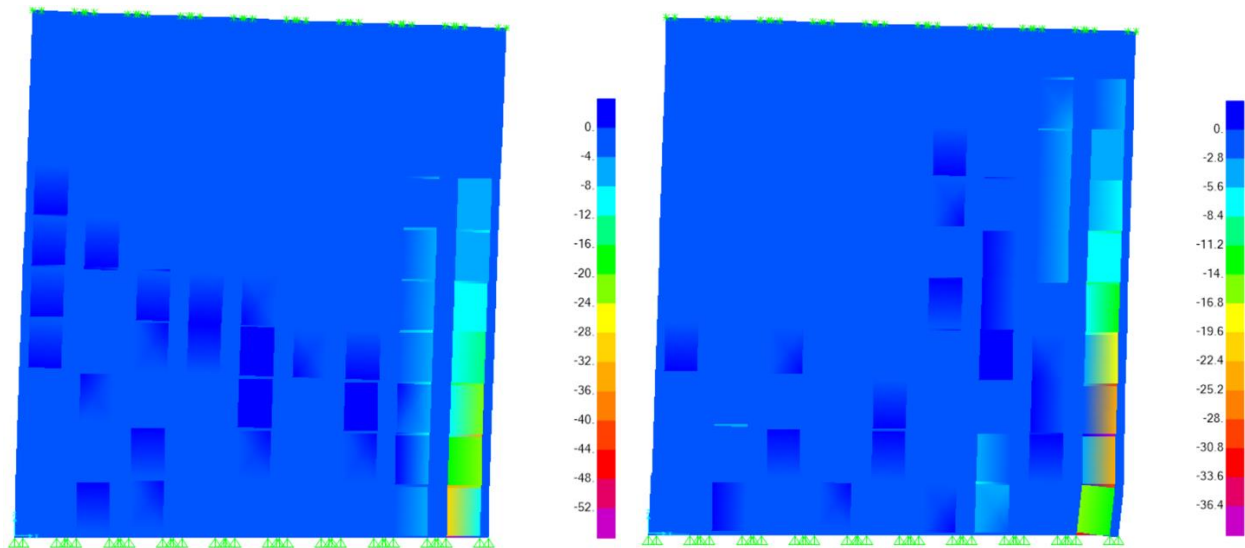


Figure A310.51 Point 3 (Left), Point 4 (Right), Concrete, SMin

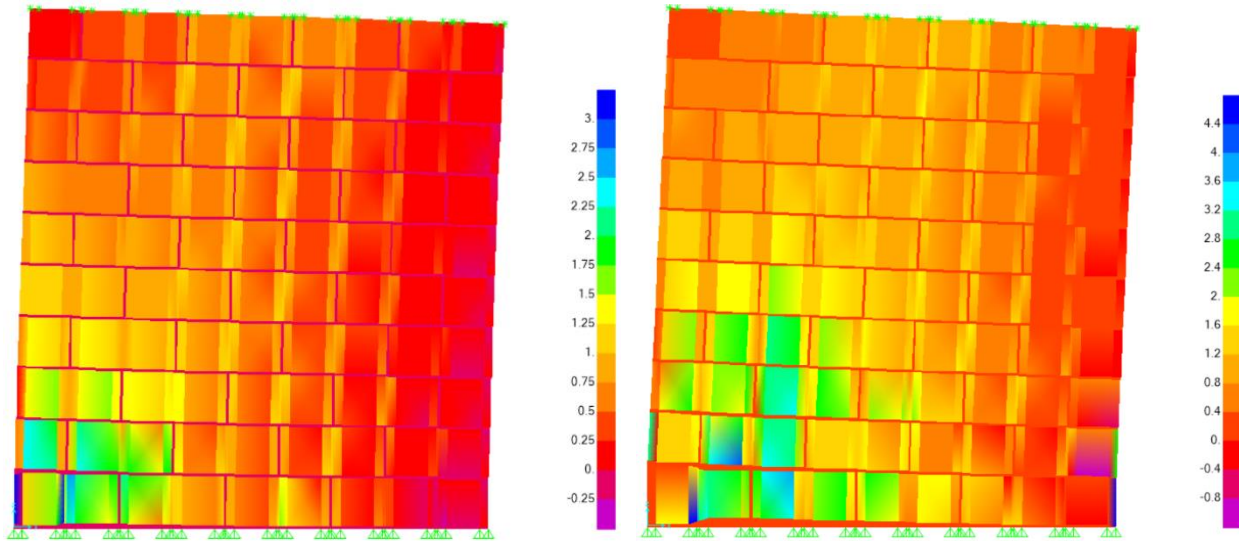


Figure A311.52 Point 1 (Left), Point 2 (Right), Masonry, SMax

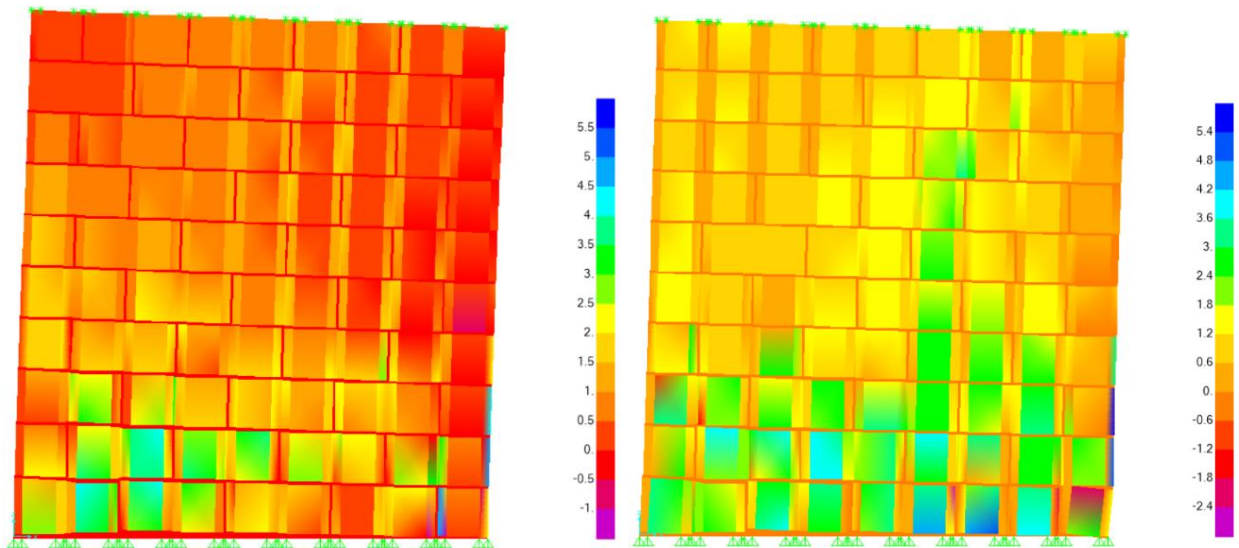


Figure A312.53 Point 3 (Left), Point 4 (Right), Masonry, SMax

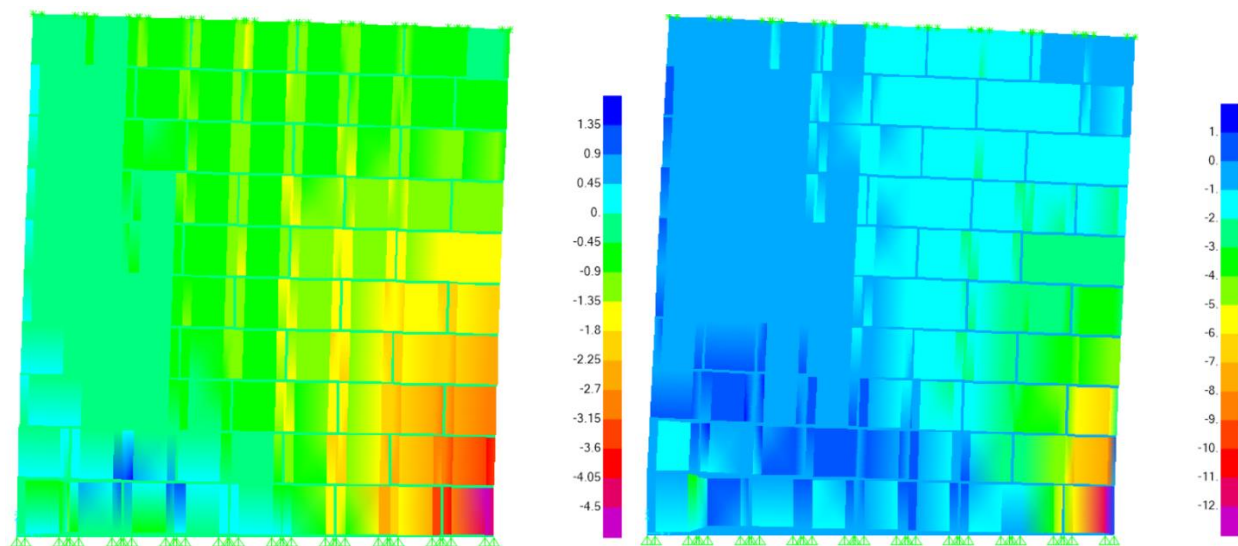


Figure A313.54 Point 1 (Left), Point 2 (Right), Masonry, SMin

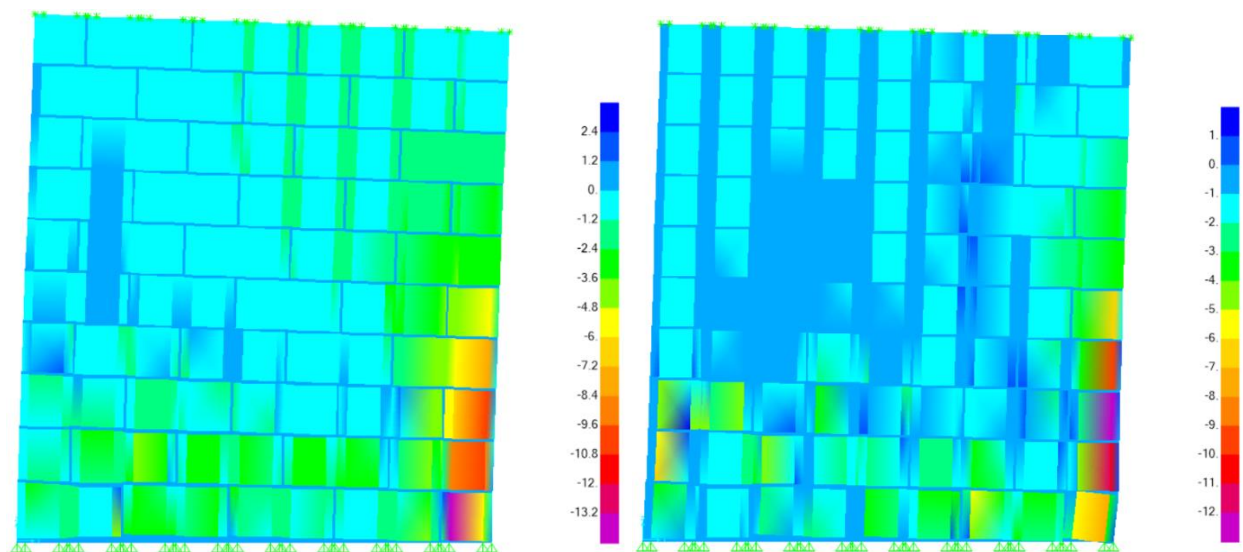


Figure A314.55 Point 3 (Left), Point 4 (Right), Masonry, SMin

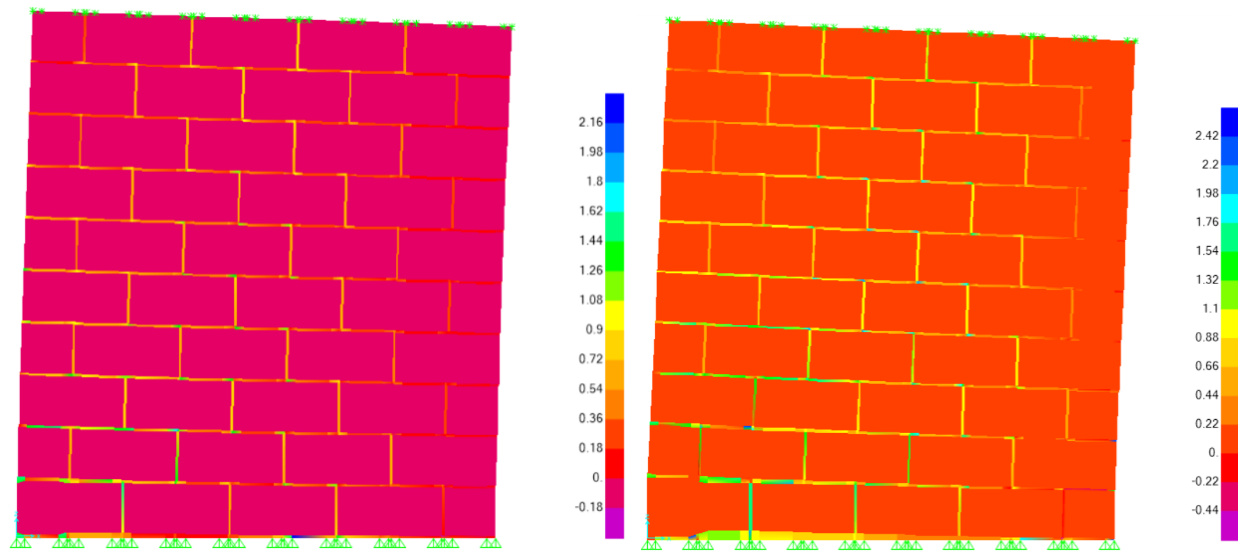


Figure A315.56 Point 1 (Left), Point 2 (Right), Mortar, SMax

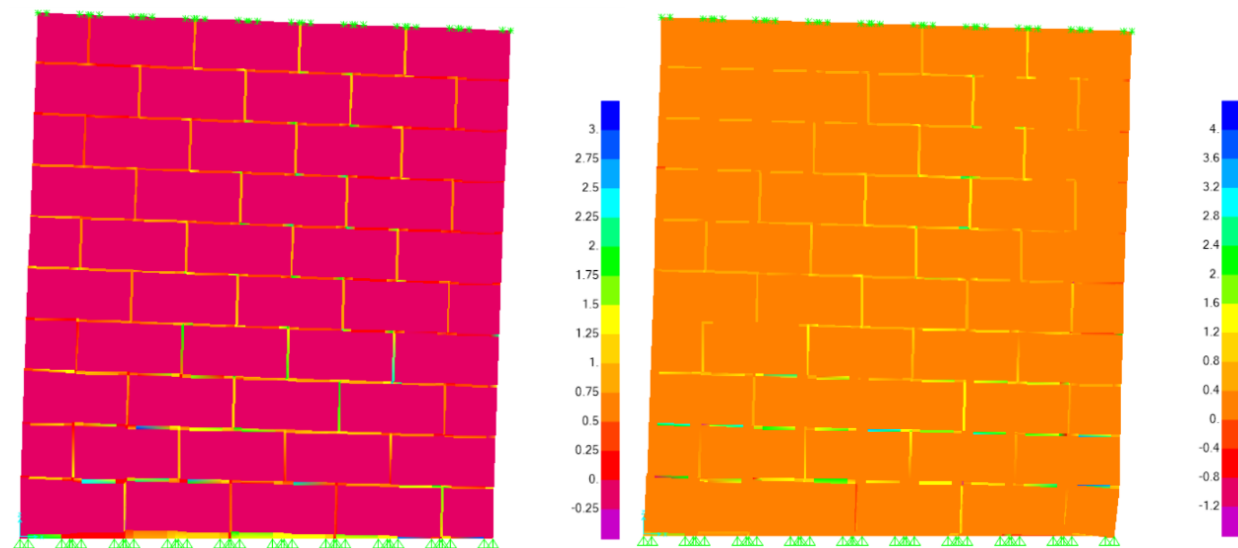


Figure A316.57 Point 3 (Left), Point 4 (Right), Mortar, SMax

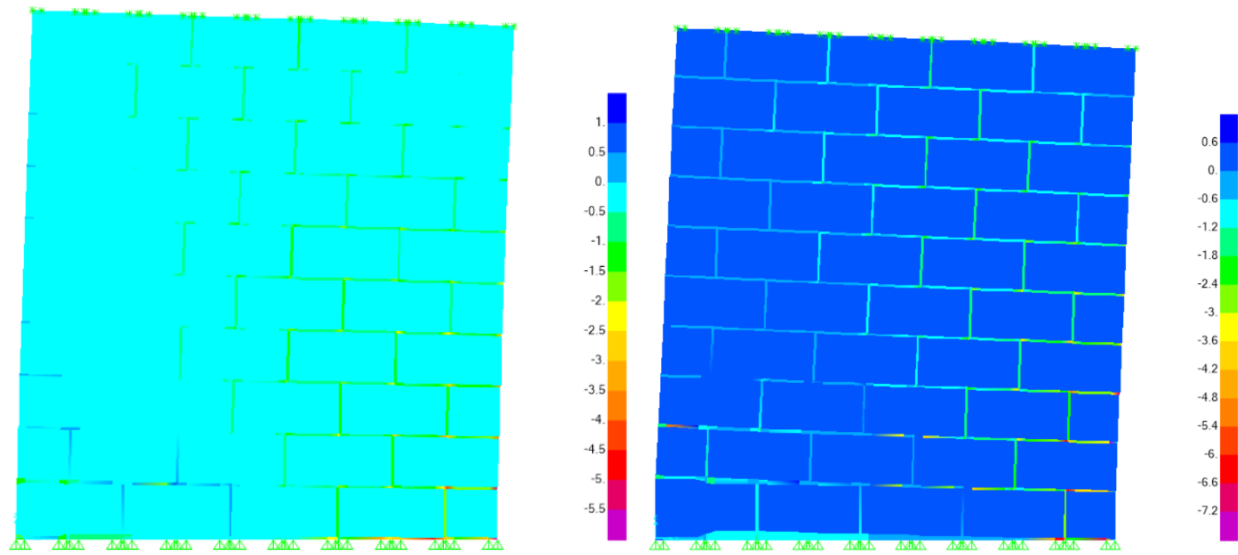


Figure A317.58 Point 1 (Left), Point 2 (Right), Mortar, SMin

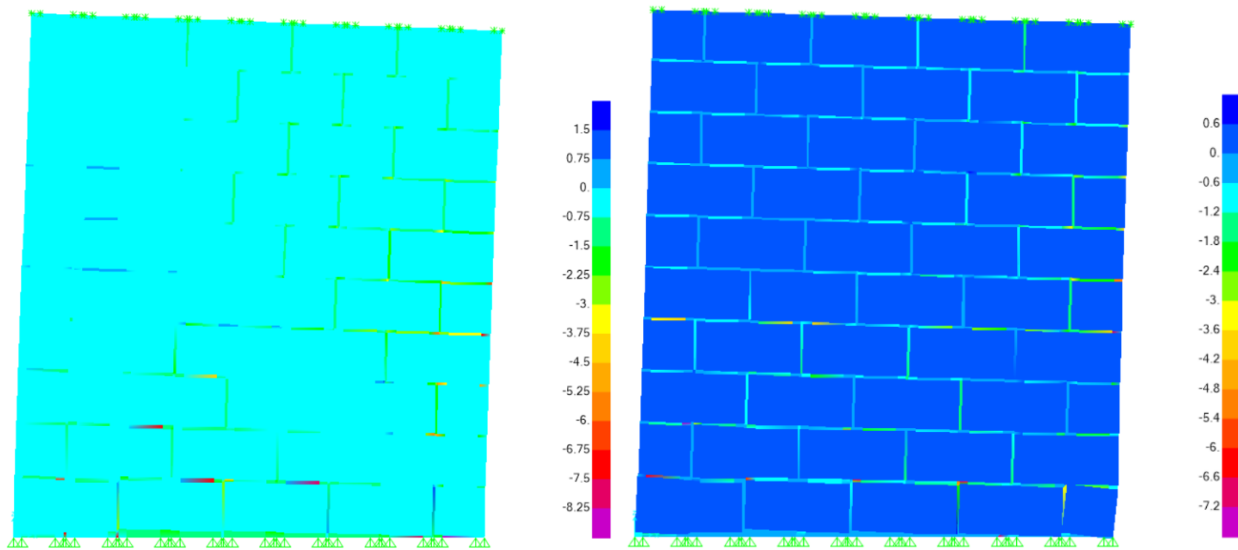


Figure A318.59 Point 3 (Left), Point 4 (Right), Mortar, SMin

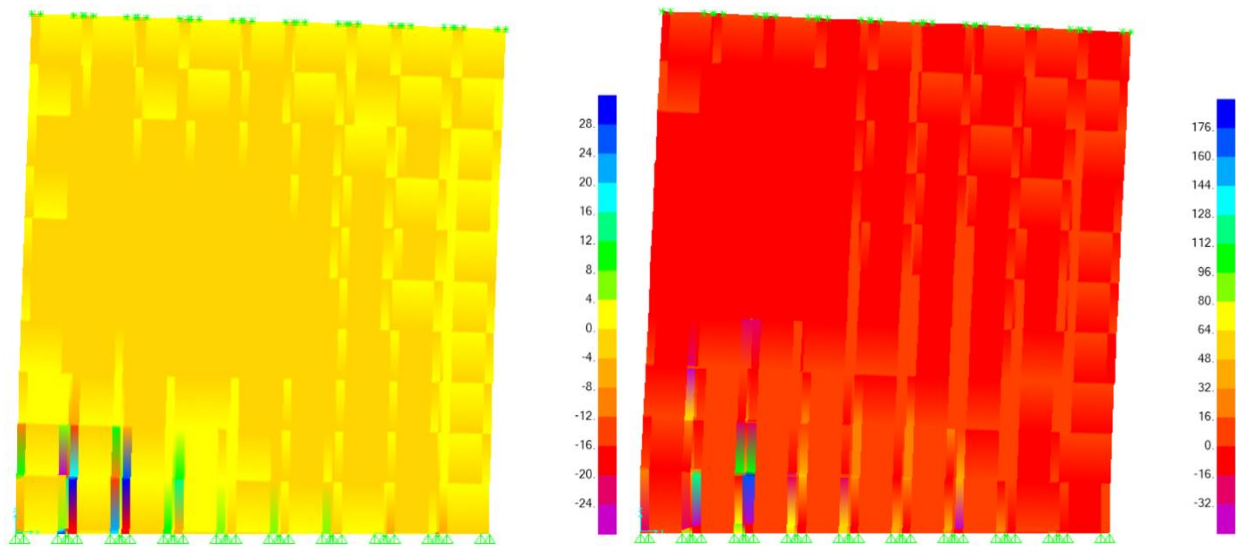


Figure A319.60 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

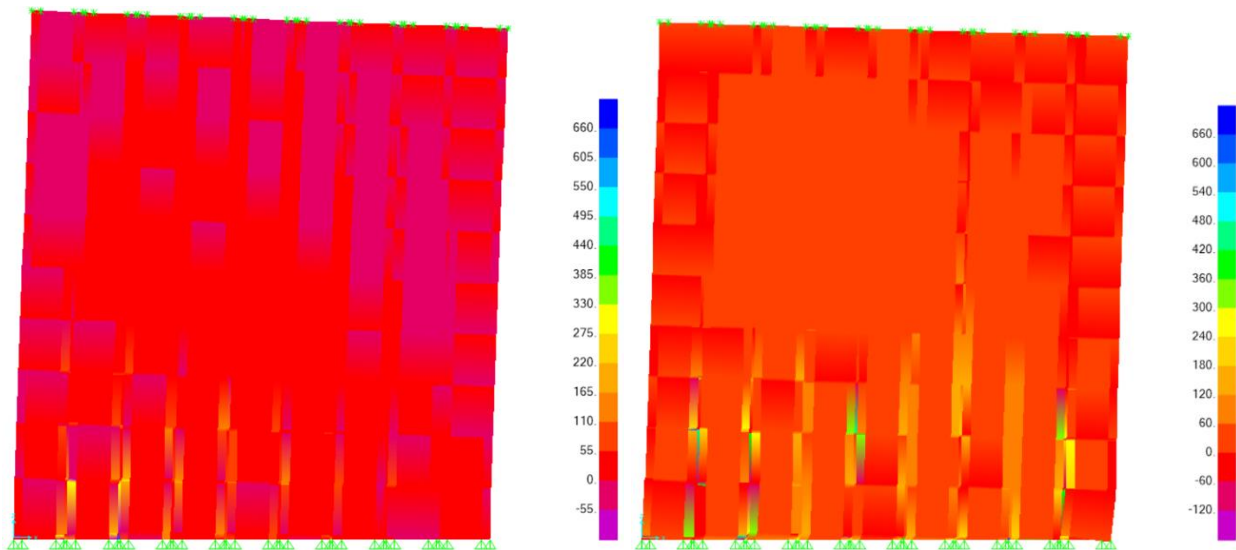


Figure A320.61 Point 3 (Left), Point 4 (Right), Horizontal FRP, S11

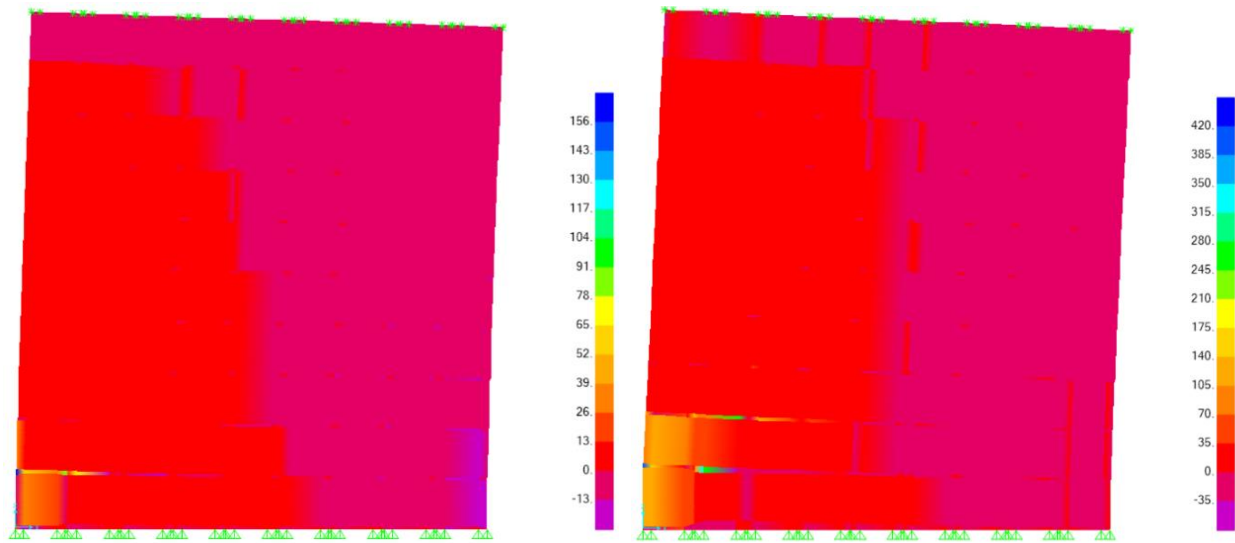


Figure A321.62 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

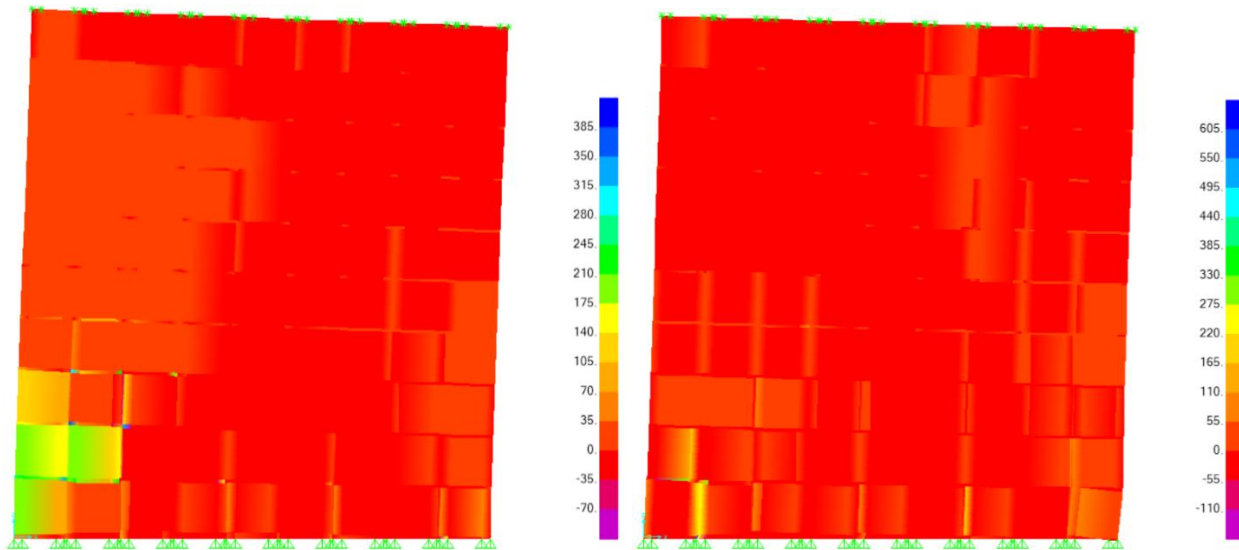


Figure A322.63 Point 3 (Left), Point 4 (Right), Vertical FRP, S22

Wall 2 without the HSS Yielding Agents (Wall 2-a)

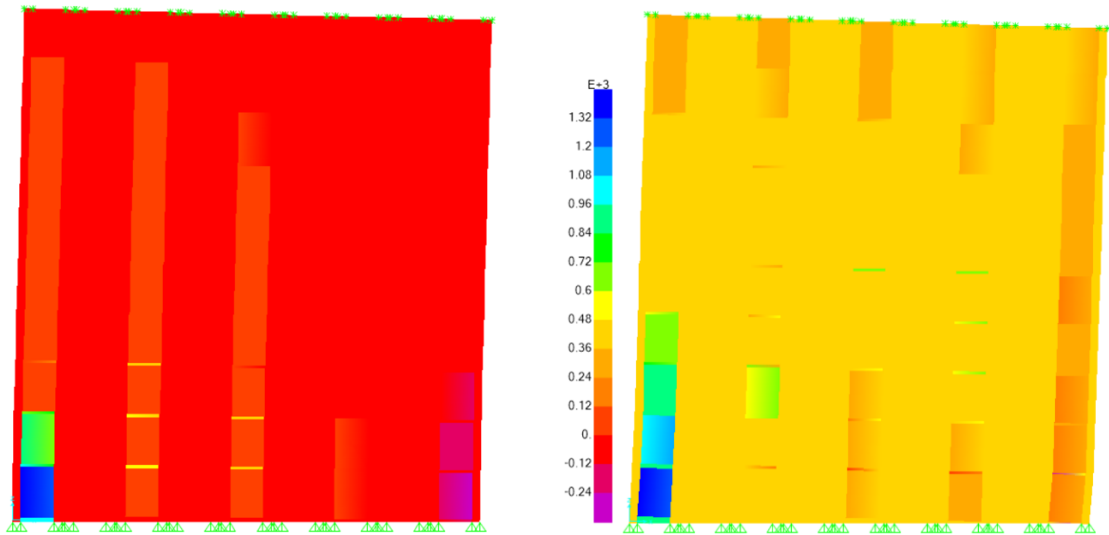
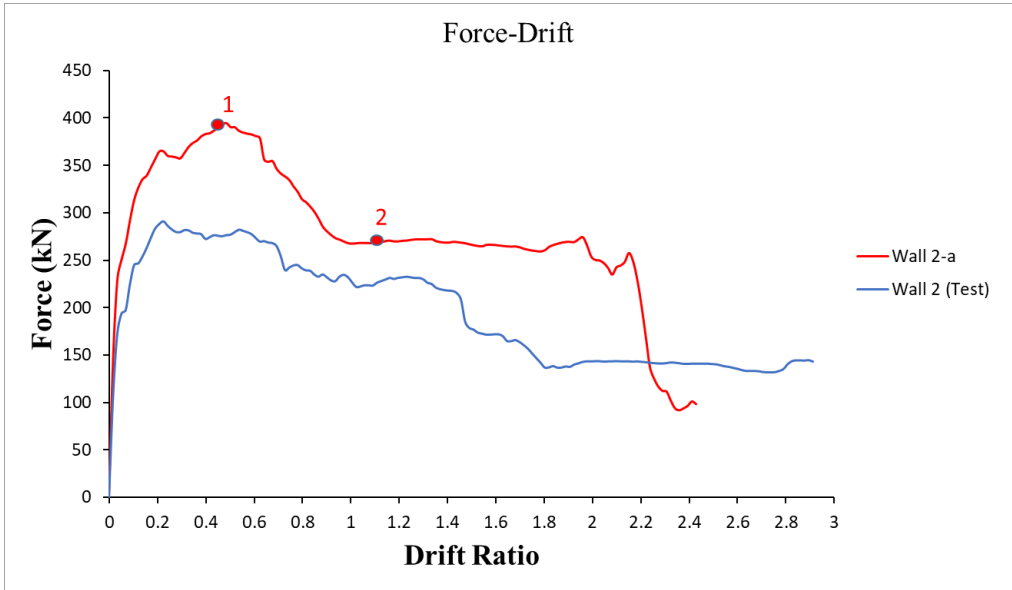


Figure A323.64 Point 1 (Left), and Point 2 (Right), Steel, S22

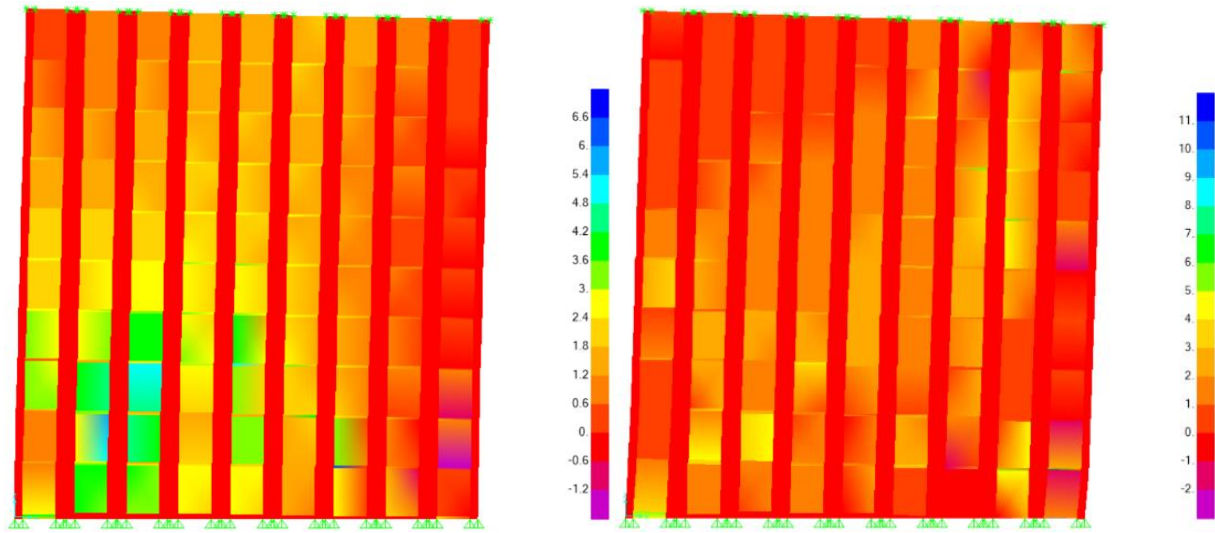


Figure A324.65 Point 1 (Left), and Point 2 (Right), Concrete, SMax

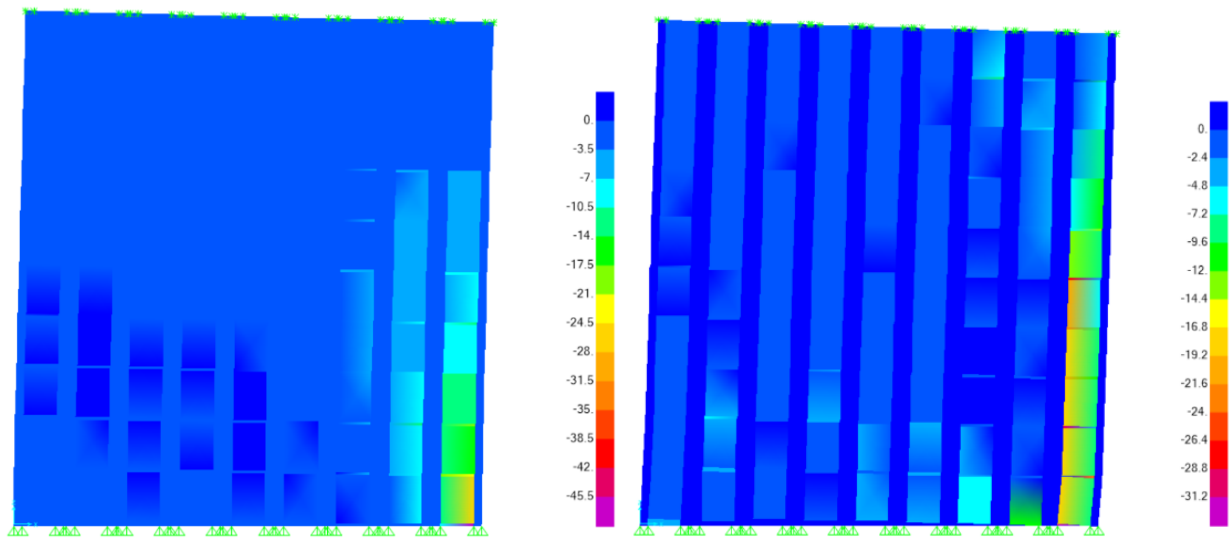


Figure A325.66 Point 1 (Left), and Point 2 (Right), Concrete, SMin

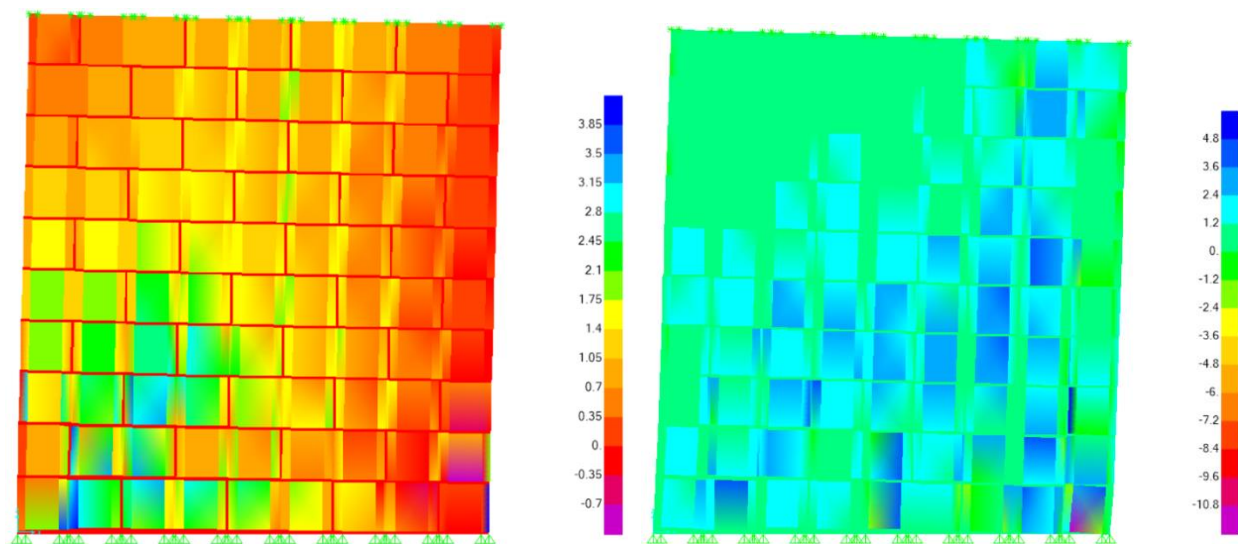


Figure A326.67 Point 1 (Left), Point 2 (Right), Masonry, SMax

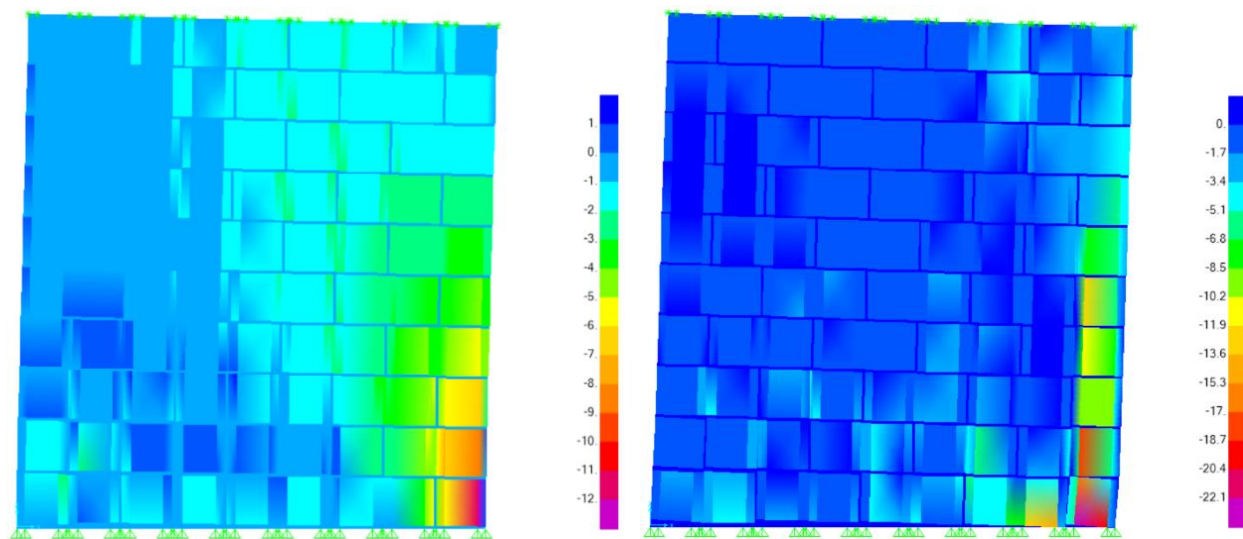


Figure A327.68 Point 1 (Left), Point 2 (Right), Masonry, SMin

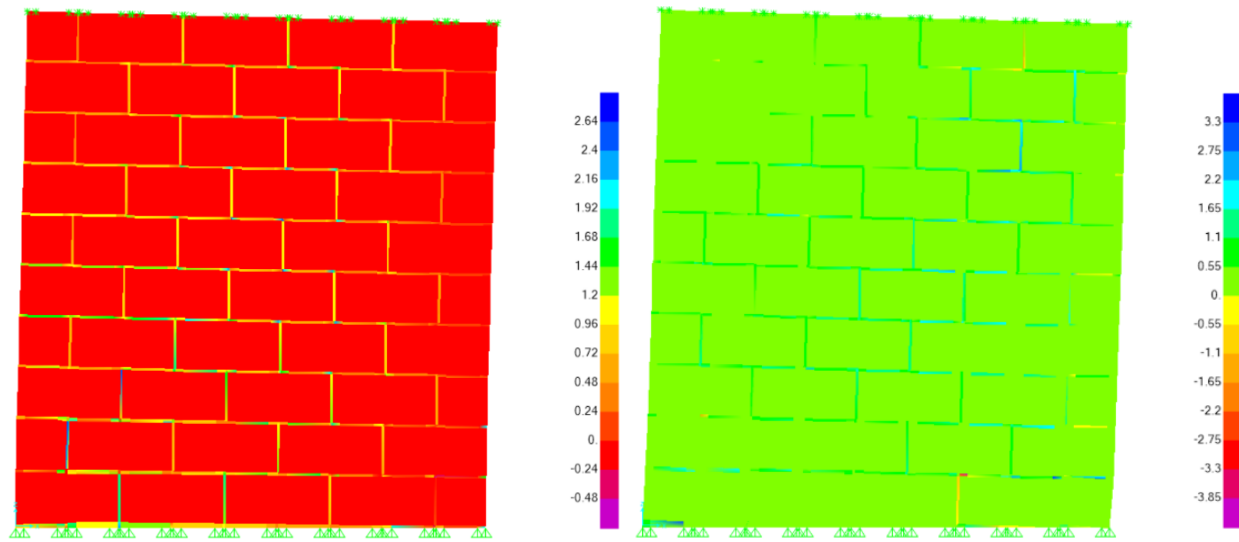


Figure A328.69 Point 1 (Left), Point 2 (Right), Mortar, SMax

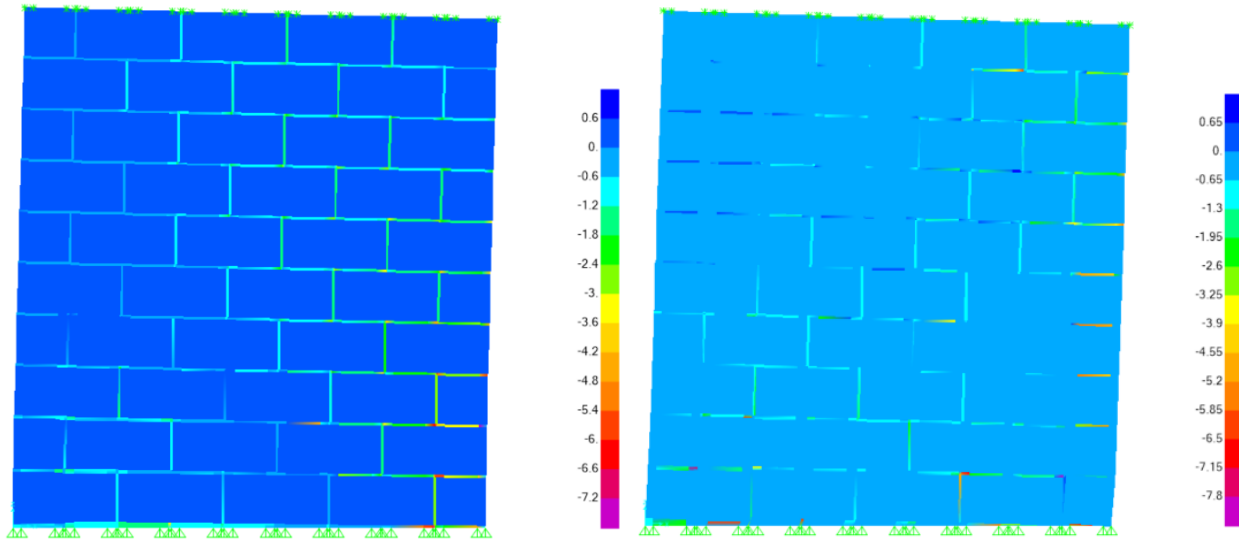


Figure A329.70 Point 1 (Left), Point 2 (Right), Mortar, SMin

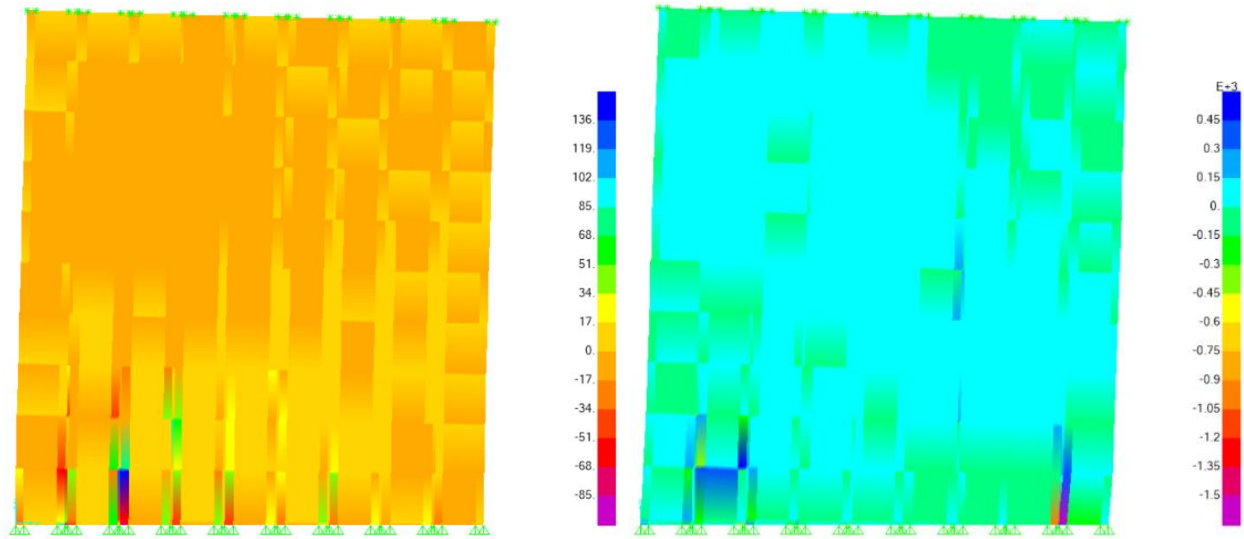


Figure A330.71 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

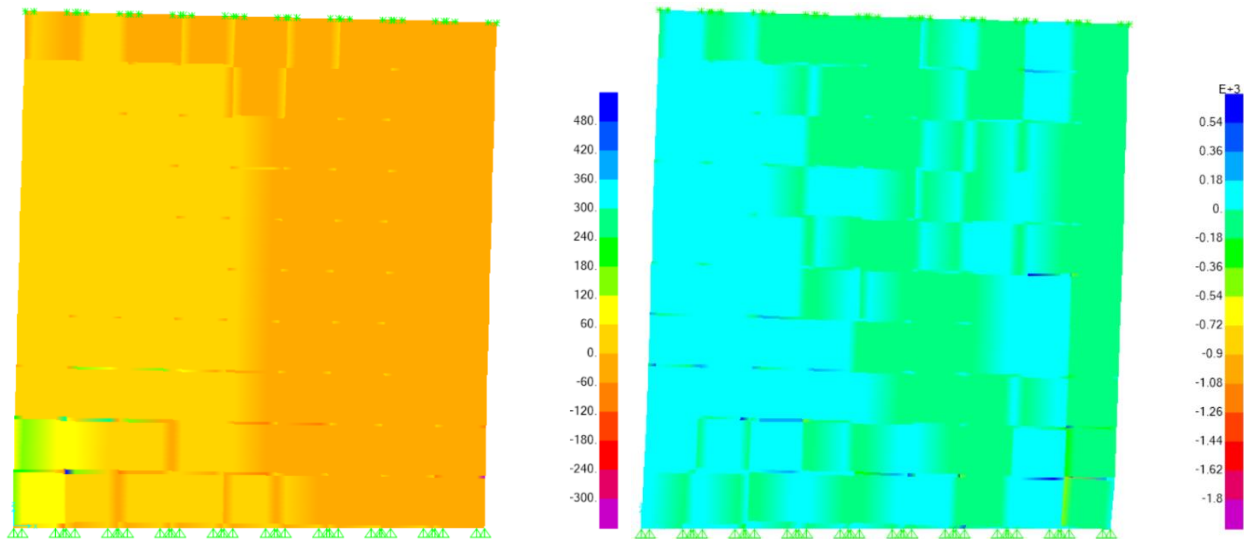


Figure A331.72 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

Wall 2 with Tendon Area Reduced to Half (Wall 2-b)

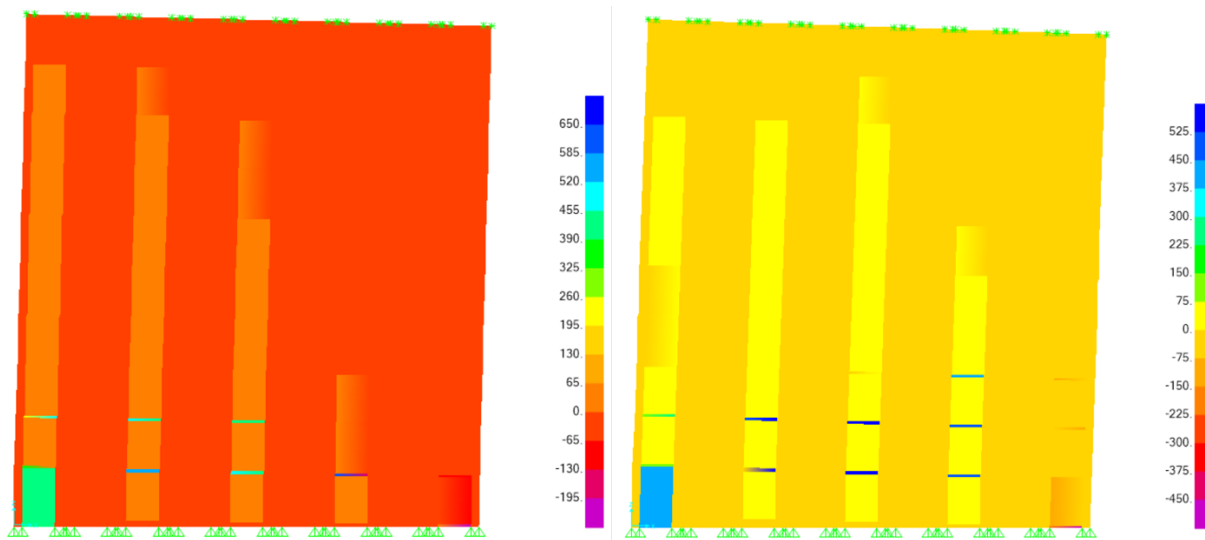
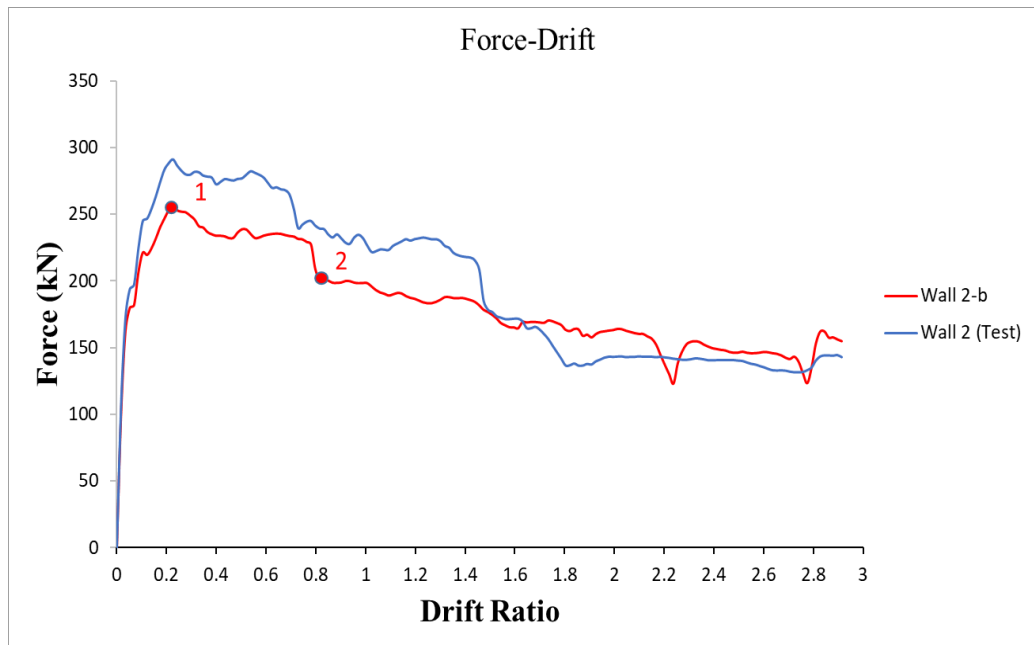


Figure A332.73 Point 1 (Left), and Point 2 (Right), Steel, S22

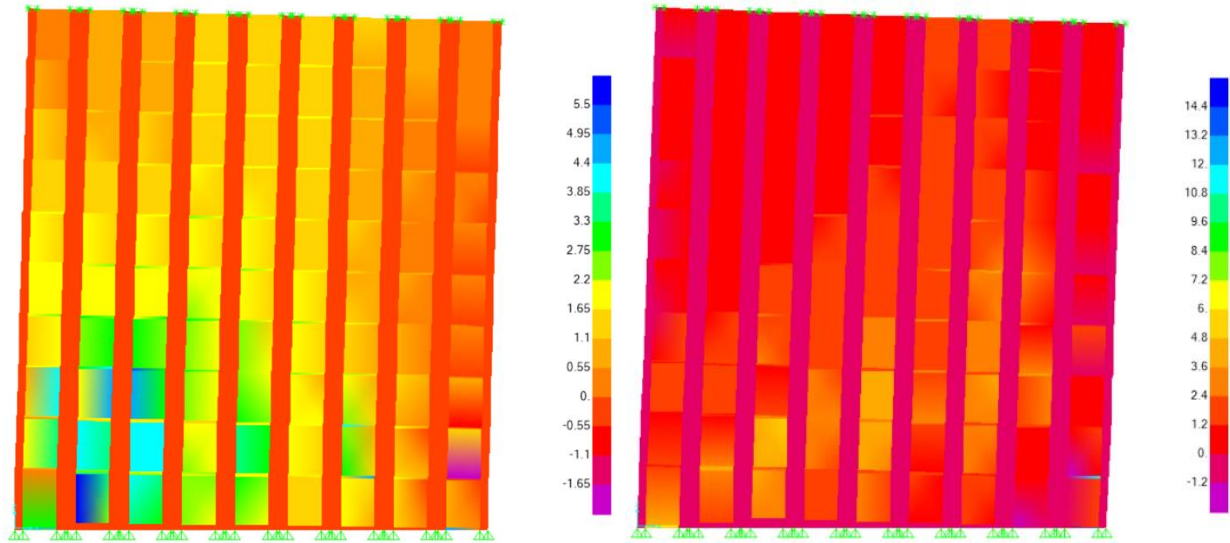


Figure A333.74 Point 1 (Left), and Point 2 (Right), Concrete, SMax

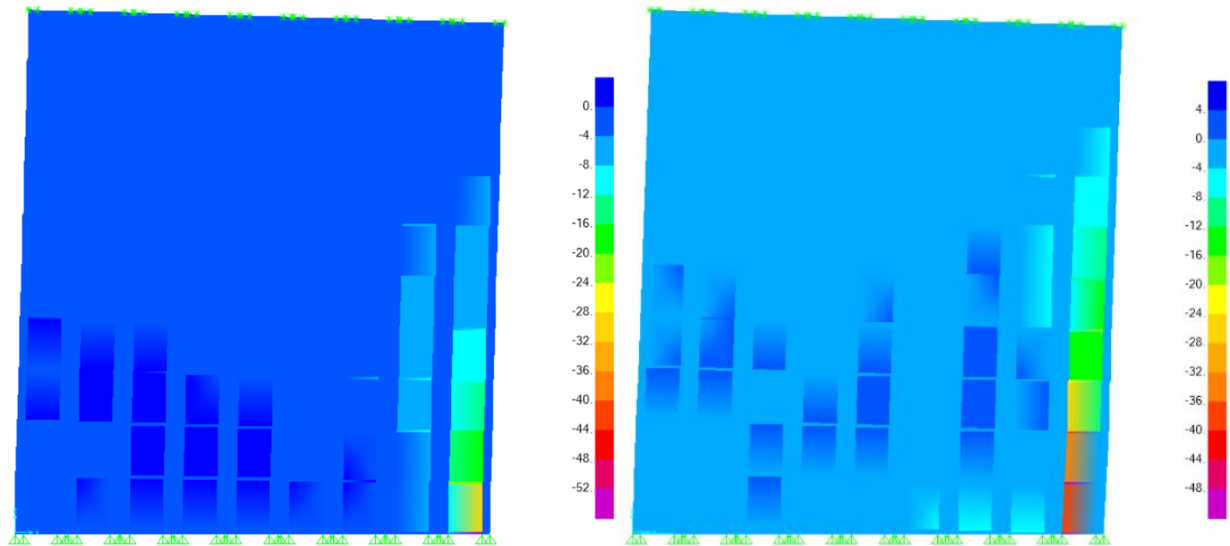


Figure A334.75 Point 1 (Left), and Point 2 (Right), Concrete, SMin

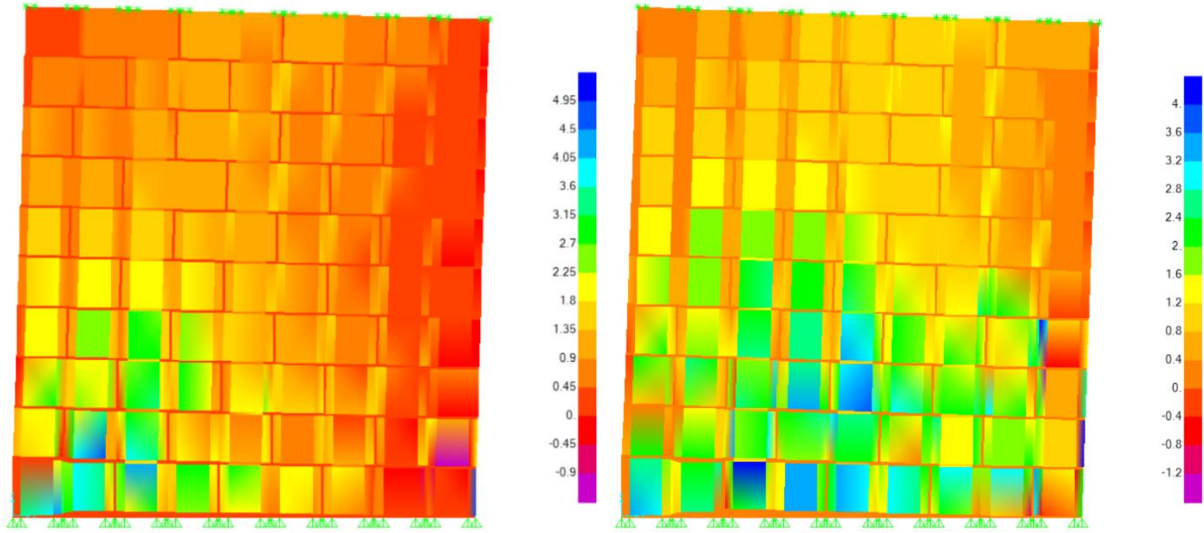


Figure A335.76 Point 1 (Left), Point 2 (Right), Masonry, SMax

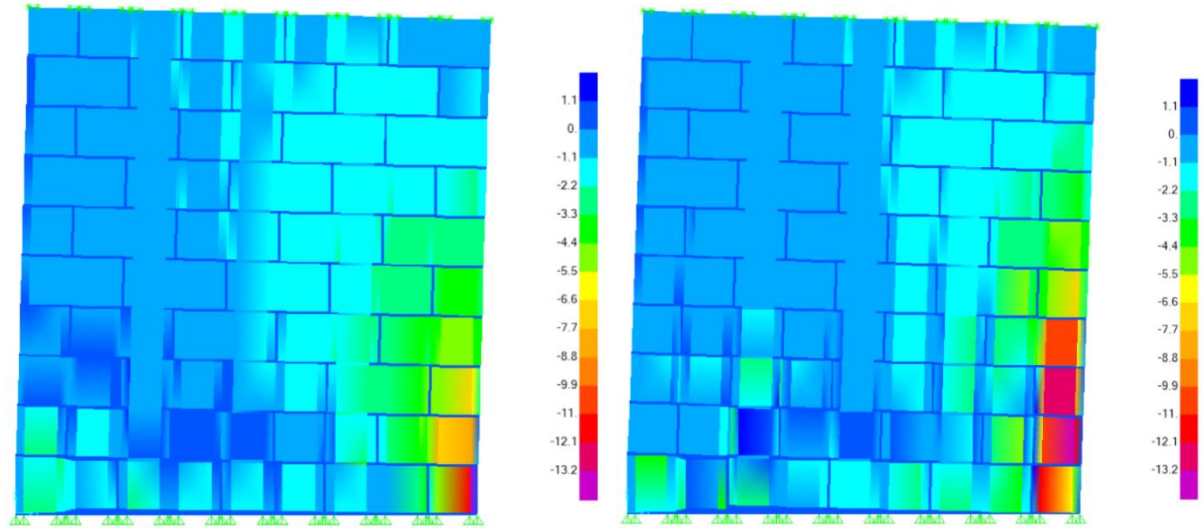


Figure A336.77 Point 1 (Left), Point 2 (Right), Masonry, SMin

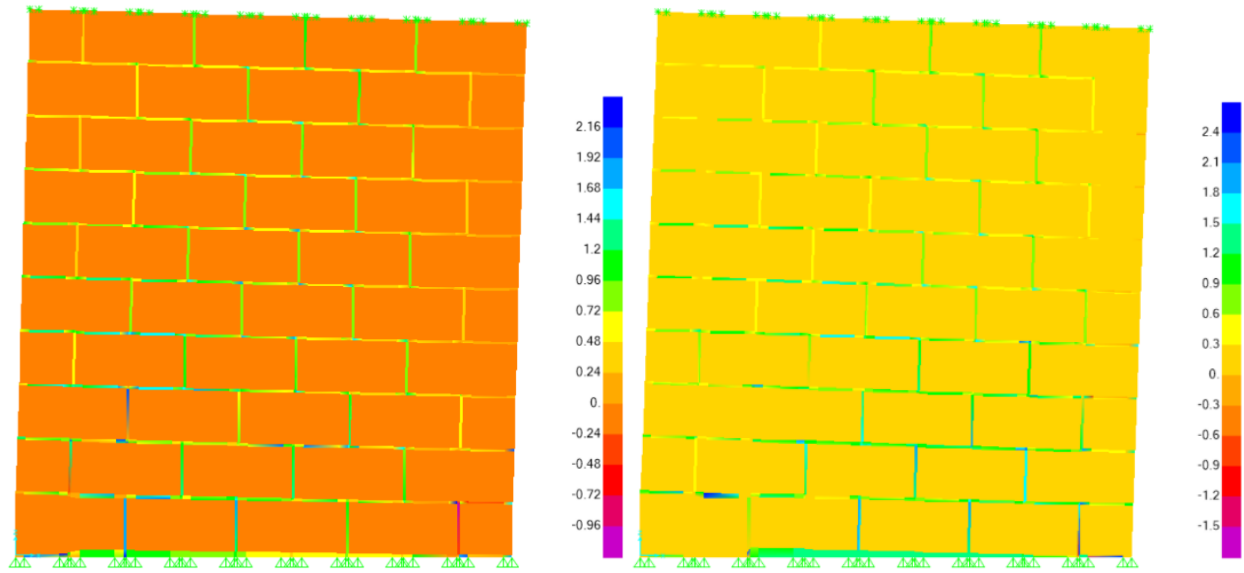


Figure A337.78 Point 1 (Left), Point 2 (Right), Mortar, SMax

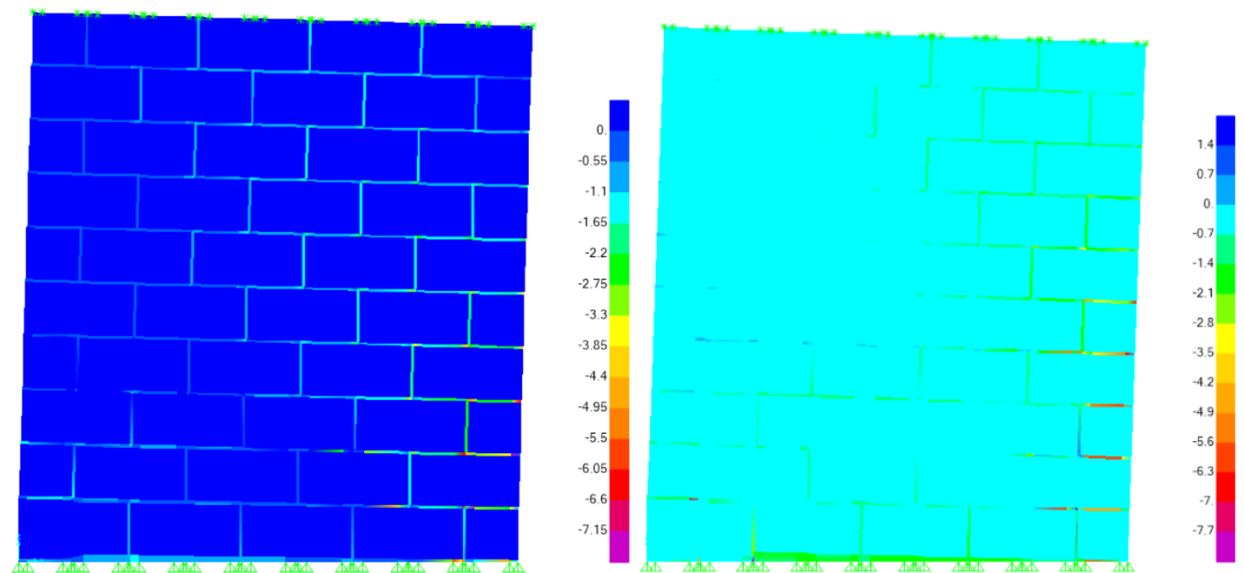


Figure A338.79 Point 1 (Left), Point 2 (Right), Mortar, SMin

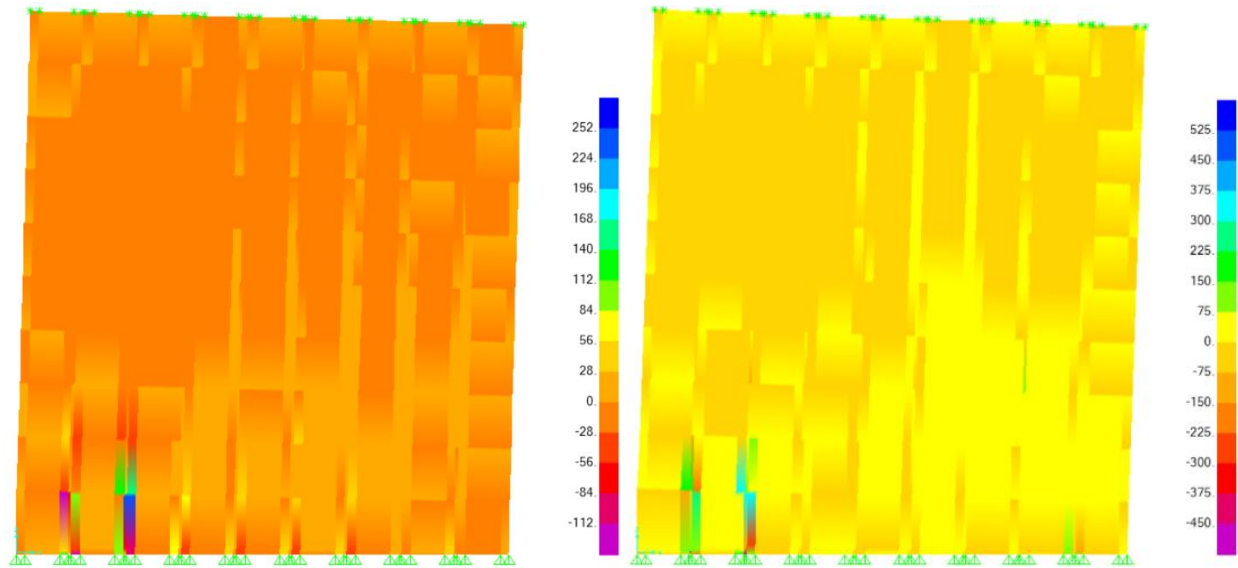


Figure A339.80 Point 1 (Left), Point 2 (Right), Horizontal FRP, S11

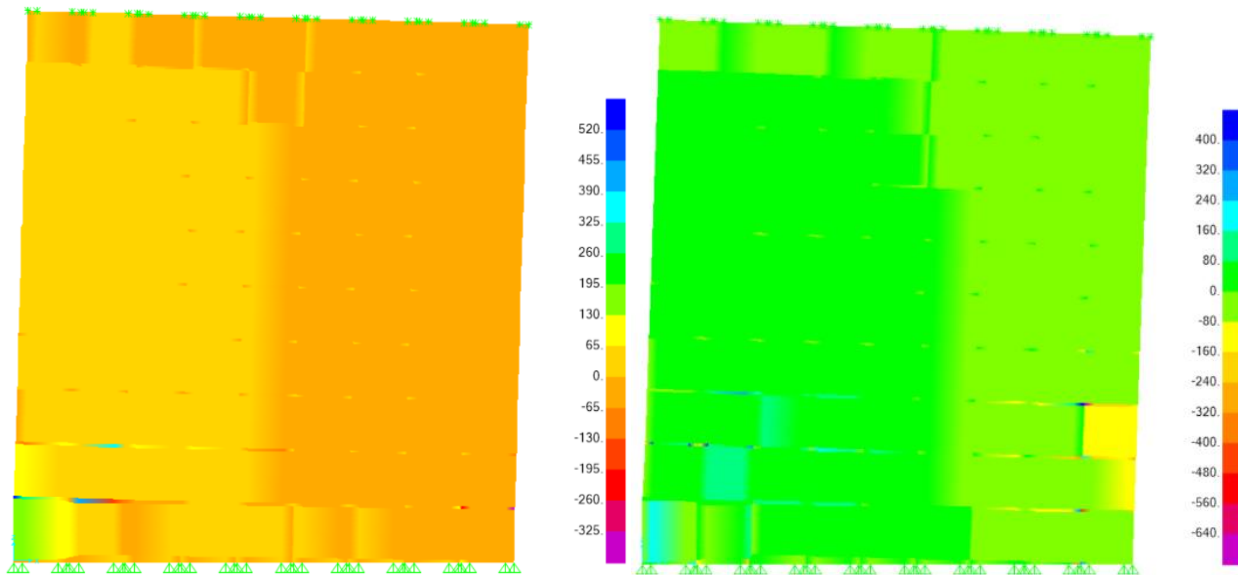


Figure A340.81 Point 1 (Left), Point 2 (Right), Vertical FRP, S22

Wall 2 with Aspect Ratio of Two (Wall 2-c)

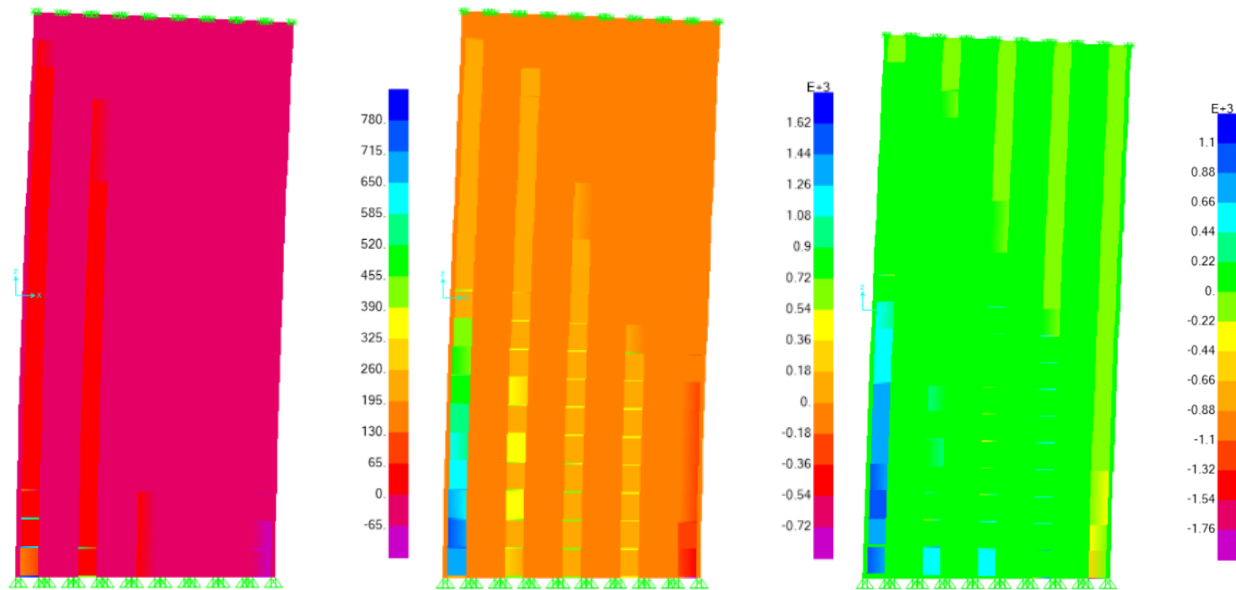
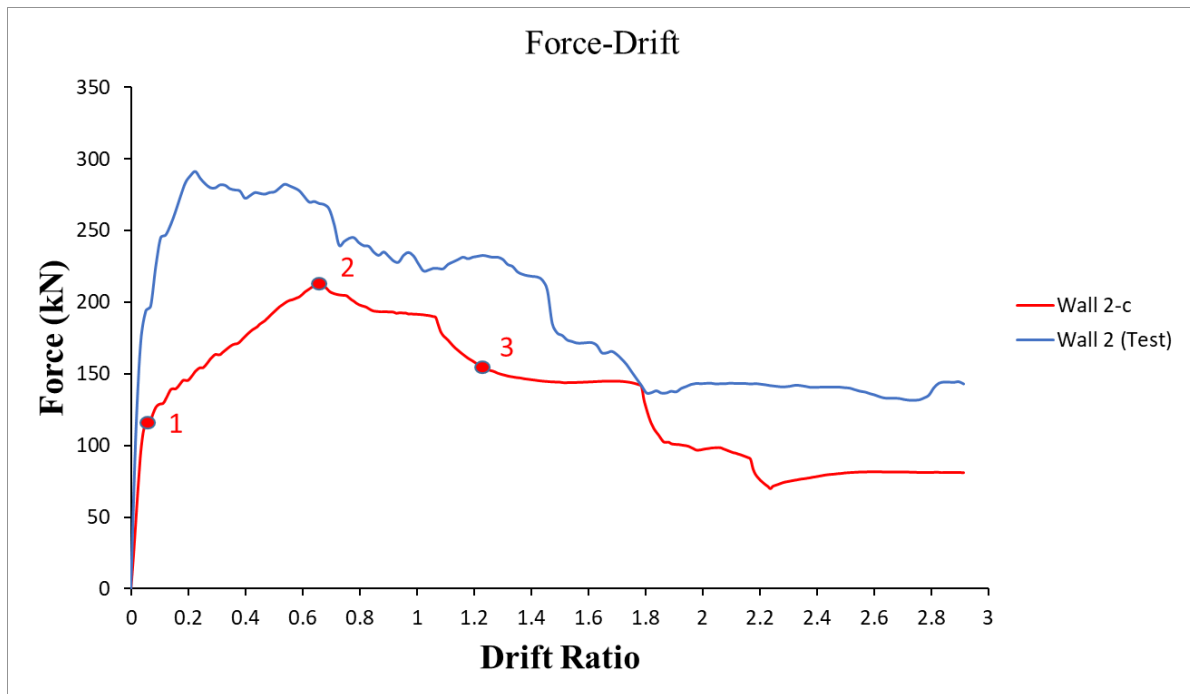


Figure A341.82 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Steel, S22

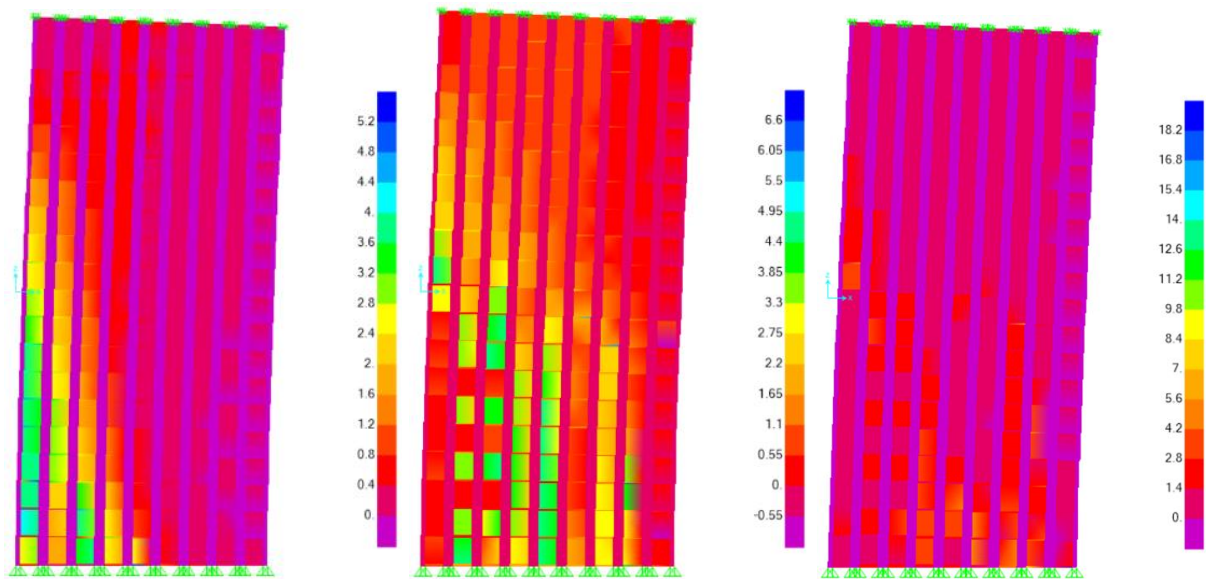


Figure A342.83 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Concrete, SMax

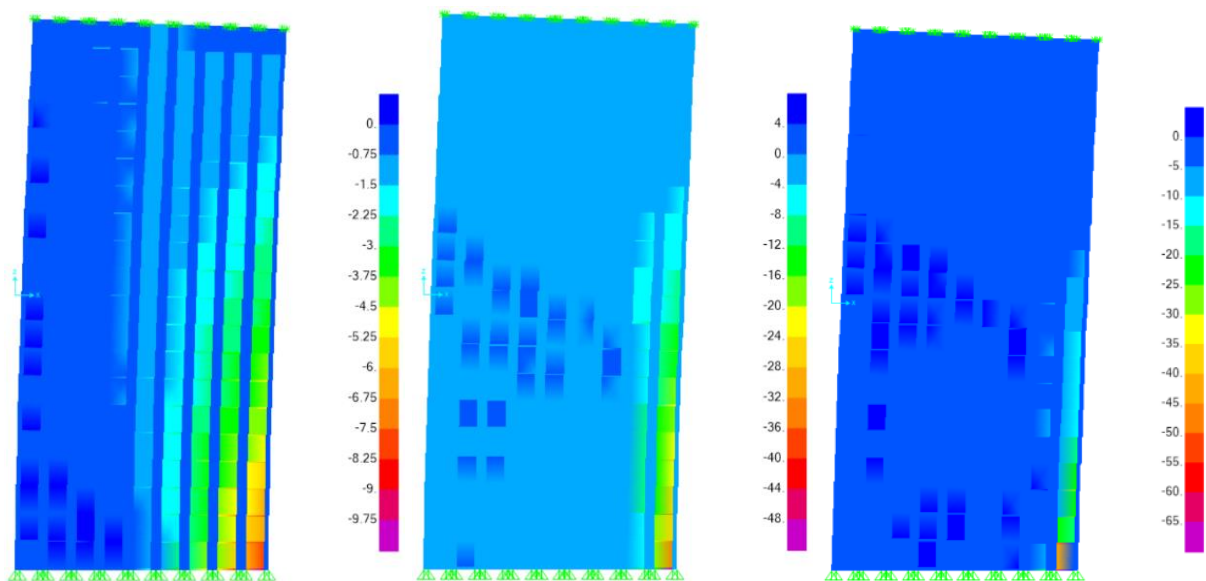


Figure A343.84 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Concrete, SMin

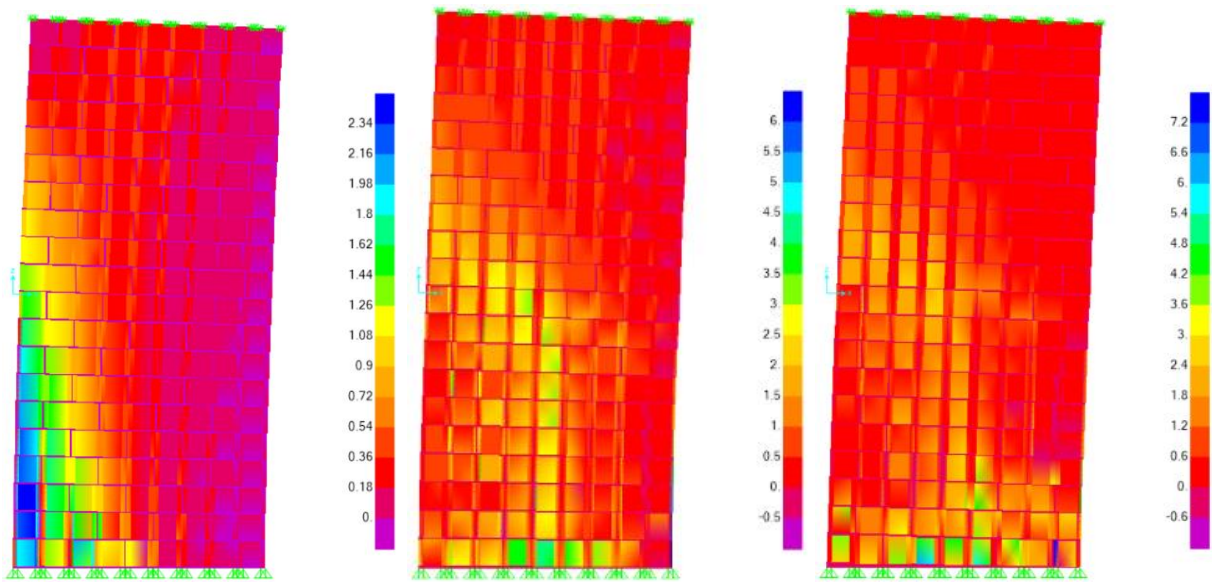


Figure A344.85 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Masonry, SMax

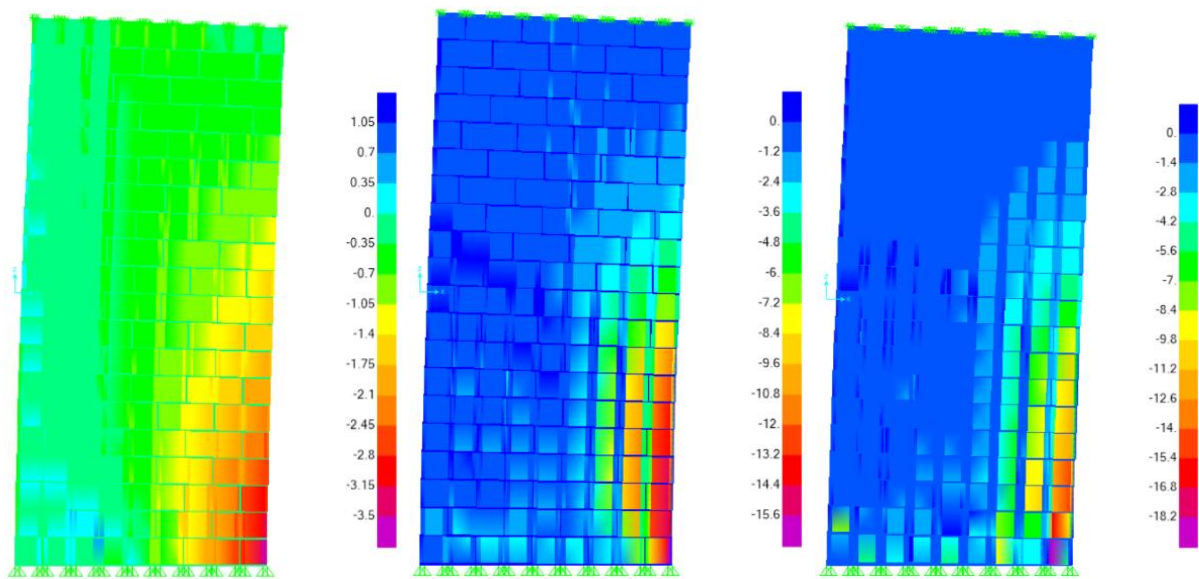


Figure A345.86 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Masonry, SMin

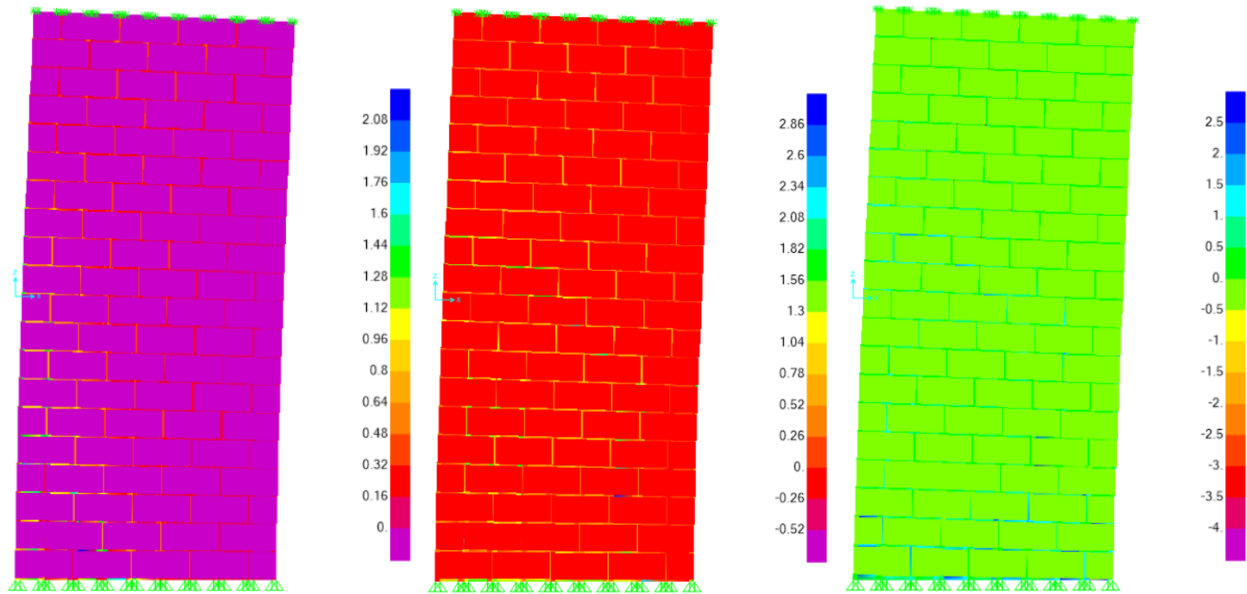


Figure A346.87 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Mortar, SMax

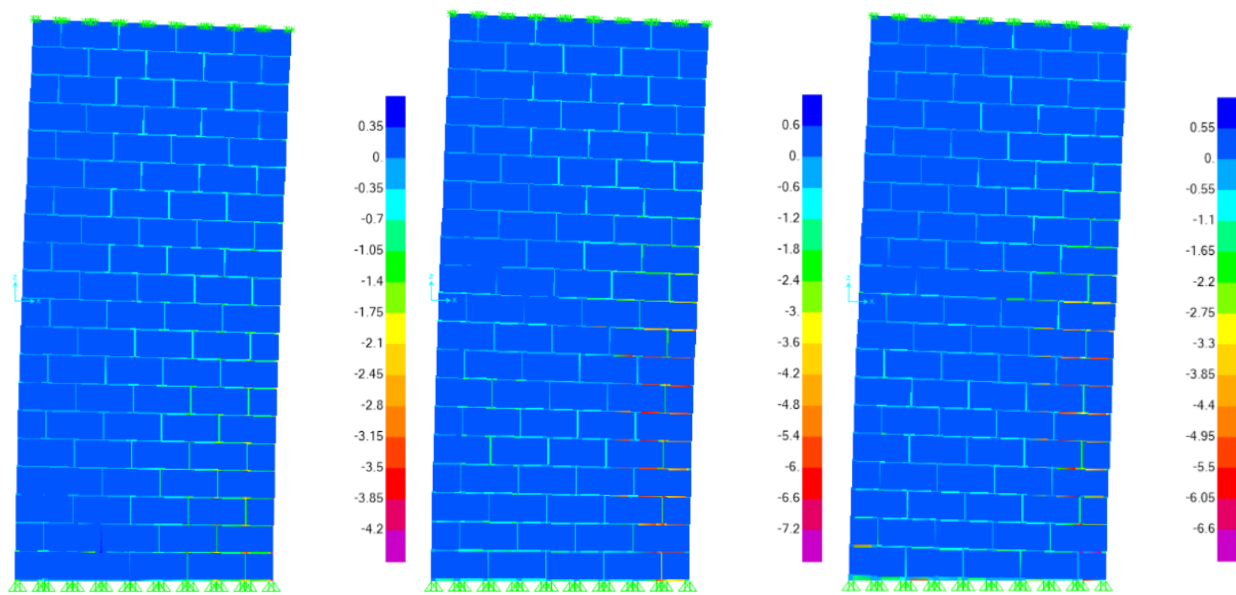


Figure A347.88 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Mortar, SMin

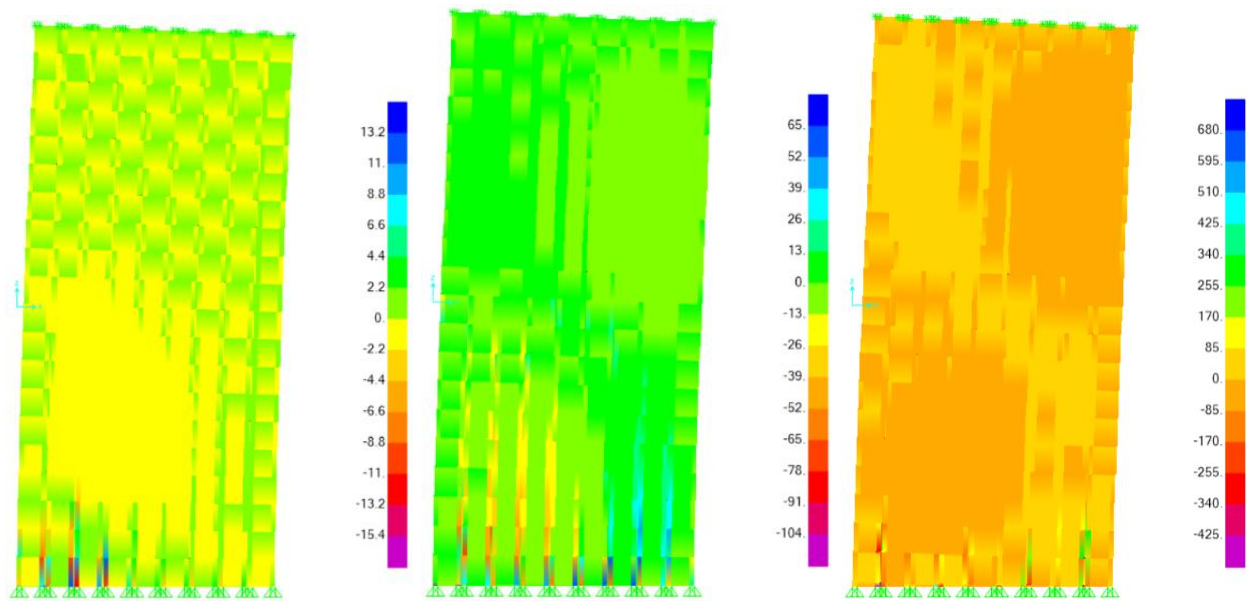


Figure A348.89 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Horizontal FRP, S11

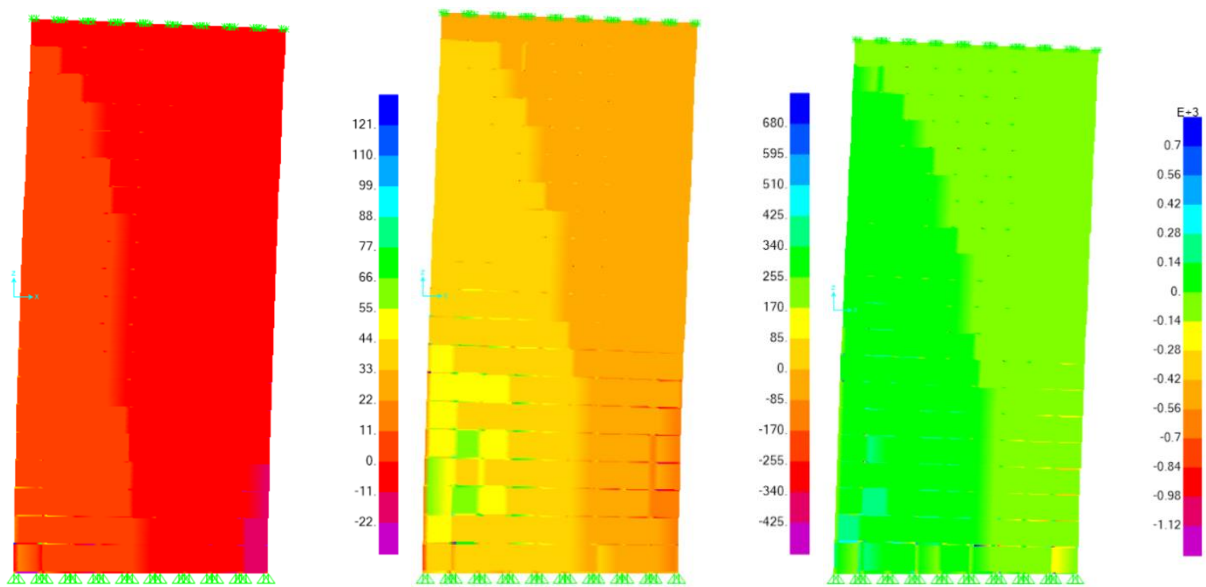


Figure A349.90 Point 1 (Left), Point 2 (Centre), and Point 3 (Right), Vertical FRP, S22