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Strength of Unbonded Post-Tensioned Walls

by

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16. Abstract

Post-tensioned masonry wall (PT-MW) is an ideal candidate for accelerating the construction of sound barriers in highways. PT-MWs have been in use for a while in buildings; however, there has been no rigorous single-study in the U. S. about in-plane strength of PT-MWs built out of concrete masonry units. This resulted in some contradictions between International Building Code (IBC 2010) and Masonry Standard Joint Committee (MSJC 2011). MSJC (2011) defines three types of PT-MWs: ordinary plain, intermediate, and special. However, the IBC (2010) combines the different types of PT-MWs into one type similar to that of unreinforced masonry (URM) walls. This represents a significant contradiction. Specially designed PT-MW would be designed according to MSJC (2011) for a seismic lateral force equal to one-third the required seismic force according to IBC (2010). Moreover, according to MSJC (2010), both intermediate and special walls have identical prescriptive bonded mild steel reinforcement and post-tensioning bars which is similar to that of specially reinforced masonry walls while the IBC (2010) does not have similar recommendations. Finally, ordinary post-tensioned walls do not have maximum spacing between tendons in both MSJC and IBC. Hence, there is a crucial gap in the current knowledge which requires an immediate investigation.

This project investigates the in-plane behavior of full-scale unbonded post-tensioned walls. All walls have the same total post-tensioning force and identical dimensions of 104 in. long, 96 in. high, and 8 in. wide. Spacing between tendons ranging from 24 in. to 96 in. were investigated. The walls were subjected to in-plane shear loads of increasing amplitude. Both flexural strength and shear strength were evaluated and compared to the strengths given by MSJC (2011).

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Strength of Unbonded Post-Tensioned Walls

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Executive Summary

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investigated. The walls were subjected to in-plane shear loads of increasing amplitude. Both flexural strength and shear strength were evaluated and compared to the strengths given by MSJC (2011).

1. INTRODUCTION

In 1953, Samuely examined post-tensioned brickwork piers in a school (Shrive 1988). Since then, the applications of post-tensioned masonry have increased and started to concentrate on the out-of-plane behavior (Schultz and Scolforo 1991, Laursen 2002, Lissel and Shrive 2003 and Bean 2007). The first experimental study on in this field was conducted by Page and Huizer (1988). Three post-tensioned masonry walls were tested under in-plane load and it was concluded that post-tensioning was an effective method to increase the shear strength of masonry walls.

Under in-plane loading of an unbounded post-tensioned masonry wall, wall cracks form at the wall-footing interface. By increasing the in-plane load, rocking of the wall occurs, which is characterized by rotation about the wall’s toe. This localizes the damage to the toe region. More importantly, the wall can return to its original vertical alignment if sufficient residual post-tensioning force remains in the tendons and the tendons do not develop significant inelastic strains (Wight and Ingham 2008).

The main drawback of unbonded post-tensioned systems is that the energy dissipation is comparatively low compared to conventional reinforced systems (ElGawady and Sha’lan 2011, ElGawady et al. 2010, Erkmen and Schultz 2009, Laursen 2002, Rosenboom 2002, and Wight 2006). To improve the behaviour of unbonded post-tensioned masonry walls, different methods have been tried, including incorporating supplemental mild steel or high strength concrete blocks. Rosenboom (2002) incorporated supplemental mild reinforcing steel between a wall and its footing. While this increased the strength, the displacement capacity slightly decreased.

Incorporating confinement plates at the toe region increases the masonry ultimate strain capacity and hence the displacement capacity of the wall (Rosenboom 2002). However, it does not increase the energy dissipation (Laursen 2002).

Ungrouted, partially grouted, and fully grouted post-tensioned masonry walls showed different behaviour, and failure mechanisms. Unlike fully grouted walls, partially grouted and ungrouted wall specimens showed a limited drift capacity and ductility and mainly failed in shear (Laursen 2002). Partially grouted walls do not allow a stable compression strut to be formed (Rosenboom 2002).

Experimental tests have also been carried out on improved post-tensioned masonry walls (Wight 2006, and Ewing 2008). Ewing (2008) concluded that it is possible to design perforated unbonded post-tensioned clay brick masonry walls to maintain all of the benefits of solid post-tensioned masonry walls.

2. Testing program

2.1. Parameters investigated

Tests were carried out on three 88 inch height and 103.6 inch width fully grouted masonry walls. A 7.625 X 7.625 X 15.625 inch masonry blocks were used in to build the walls. Two variables were investigated in an experimental study. The first parameter is the spacing between the post-tensioned bars. A 32 inch (fig.1) and 96 inch (fig.2) spacing were used consequentially with a constant axil net force on the walls. The second investigated parameter is the effect of the horizontal bond beam (shear reinforced beams). One wall with a two bond beams in the thirds of the height was tested to compare with same post-tension bars spacing (fig.3). Other parameters were studied in this project by using nonlinear finite element model solved by LS DYNA software.

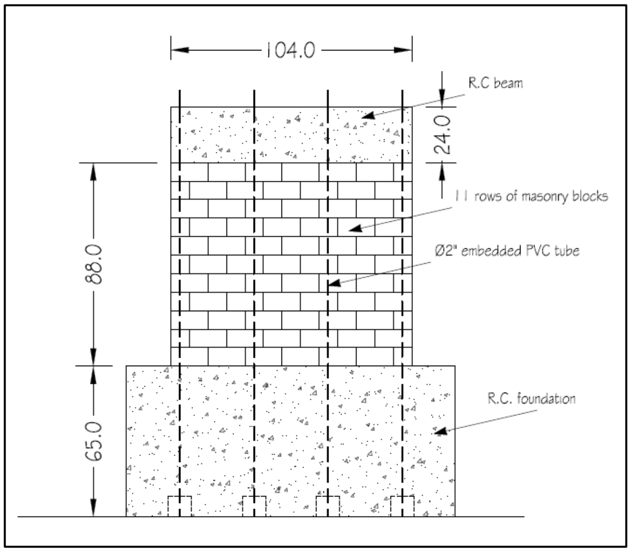


Figure (1): Post-tensioned wall with 32 inch spacing

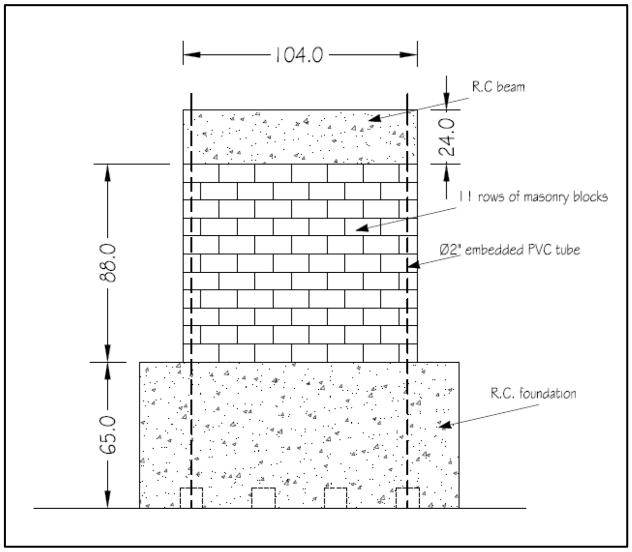


Figure (2): Post-tensioned wall with 96 inch spacing

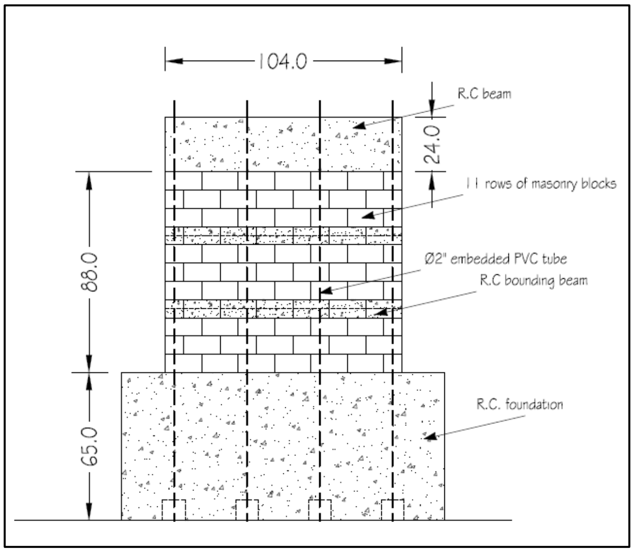


Figure (3):Post-tensioned wall with32 inch spacing and two bond beams

2.2. Walls construction

The tested walls consist of three major parts. The first part is the footing. The design of footing fig(4) considered the loads that come from the lateral load on the walls and the post tension load in the bar which connect the wall to the footing and the footing to the floor. For that, a PVC pipes were used to made the holes fig (6),(8),(9) and another technique was used to made the grove that contain the ending plate and nut of the post-tensioned bars fig(5).

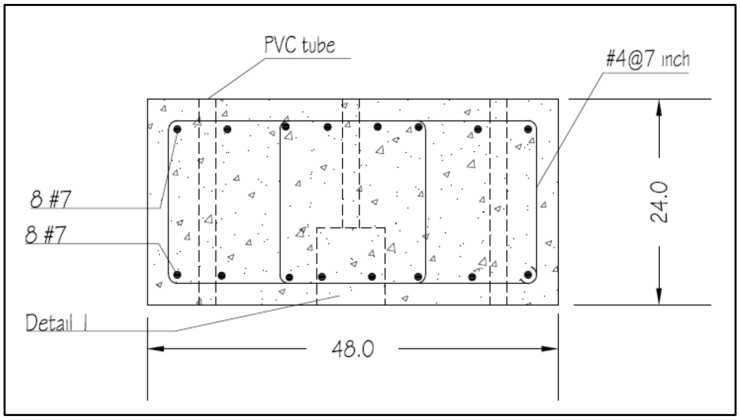


Figure (4): footing cross section

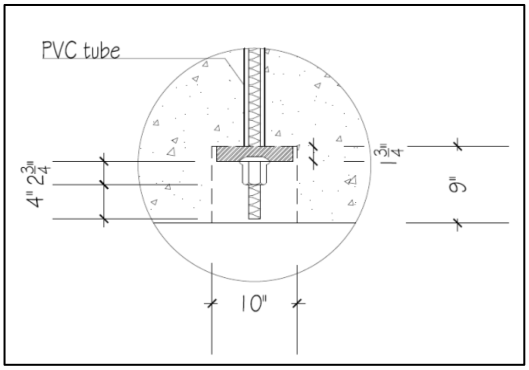


Figure (5): Detail 1

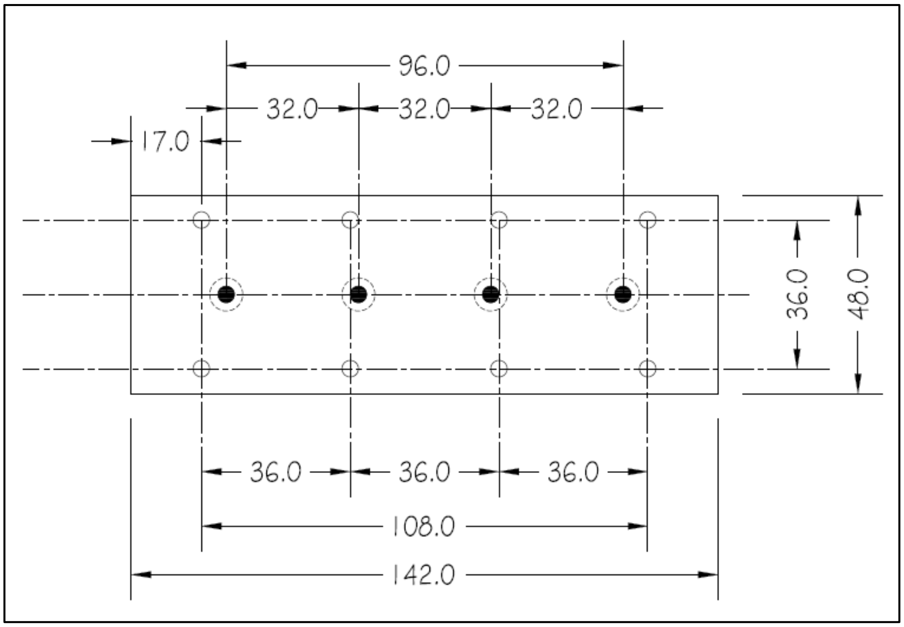


Figure (6): footing top view

To control the aspect ratio of the walls, a RC base was used under the footing fig (7). Its dimension is 41 height and 48X142 inch.

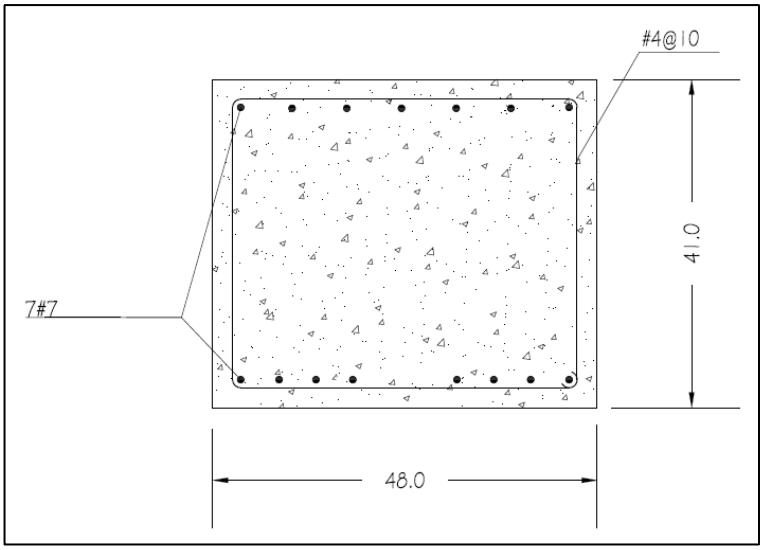


Figure (7): RC pedestal cross section

Figure (8): Reinforcements and holes in wall footing

Figure (9): Reinforced Concrete pedestal

The second part of the walls is the masonry block walls. It is fully grouted walls and they don't have any conventional reinforcements. A 2 inch PVC pipes were put in the walls to provide the unbounded length for the post-tensioned bars fig (10). Strain gages were attached to the steel reinforcement of the horizontal bond beam.

Figure (10): The masonry block wall with PVC pipe

The third part of the walls is the top beam. It is a RC beam that used to distribute the axil loads evenly and to transfer the cyclic lateral loads from the actuators to the wall. The design of the top beam took in the consideration that the top beam will be used with different post-tensioned bar spacing and different amount of load in each time fig (11). This beam contains a PVC pipes to create holes which match the holes in the walls and the foundations fig (12).

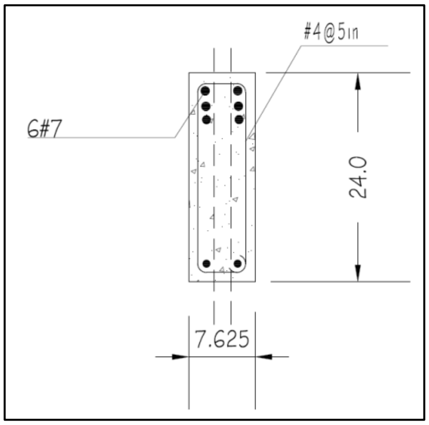


Fig (11): Details of RC top beam

Fig (12): The wall with top beam

More than one technique had been used to avoid to sliding between the top beam and the wall during applying the lateral load. Finally, using the 8x epoxy fig (13) was the best way to get a fully attached beam and wall. Before that, non-standard tests were done to find the adhesion for each type of epoxy fig (14)

Fig (13): LUCTITE 8x epoxy

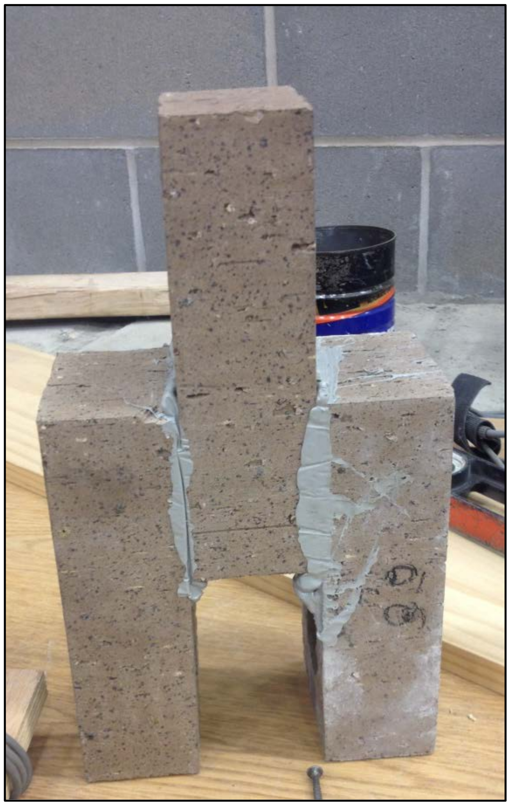


Fig (14): Testing the adhesion of the epoxy

Since the loads were applied as a push and pull to simulate the seismic loads. Special fixtures were needed to satisfy that. The load applying setup consists of two strong end plates connected by four high strength 1.5 inch diameter post-tensioned bars. An adapter was fixed at the head of the actuators to transfer the loads from the two actuators to the end plate and later to the wall through the top beam fig (15).

Fig (15): Top beam with the test setup

The top beam was attached to the actuators by using a special rigid adaptor. The reason behind that was keeping the wall and the test setup stable and having a one steady concentrated load comes from two actuators fig (16) (17).

Figure (16): Attaching the adaptor to the top beam

The next step is applying the post-tensioned forces to the walls. In this project, a unique technique which is never been used before were used. This technique is applying the post-tensioned forces with a different amount in each bar based on its location fig (18). This idea came from the fact that the strain in each bar will be different based on its location with respect to the N.A.

Using this technique has two advantages. The first one is avoiding the early yielding in the bars that have the farthest position from the N.A. the second advantage is increasing the resisting capacity of the walls.

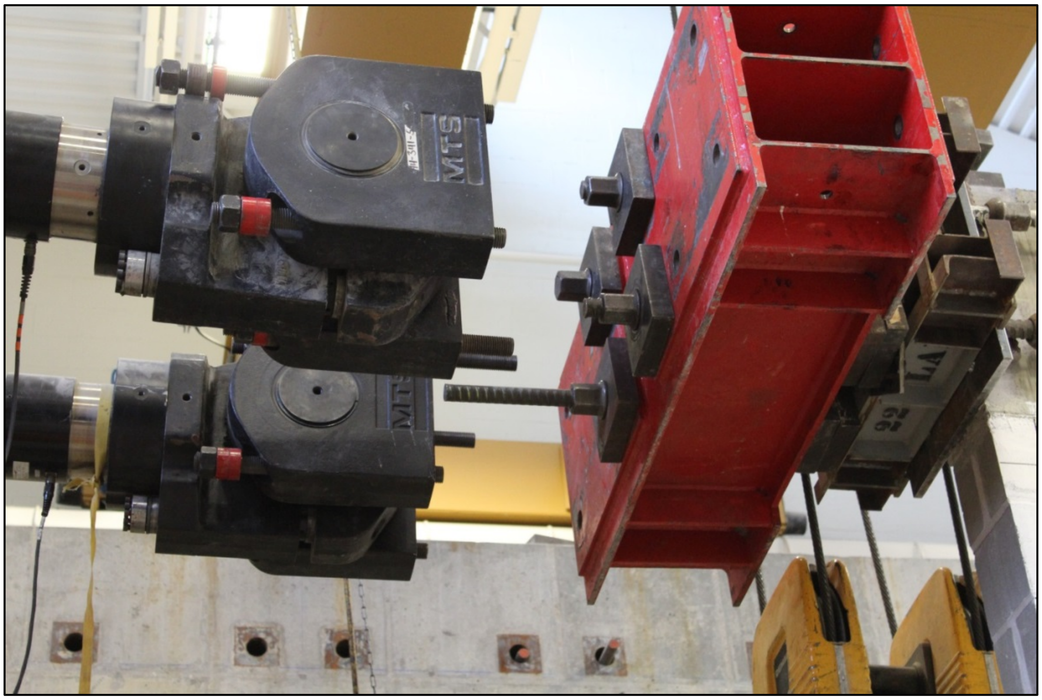


Figure (17): Attaching the adaptor to the top beam

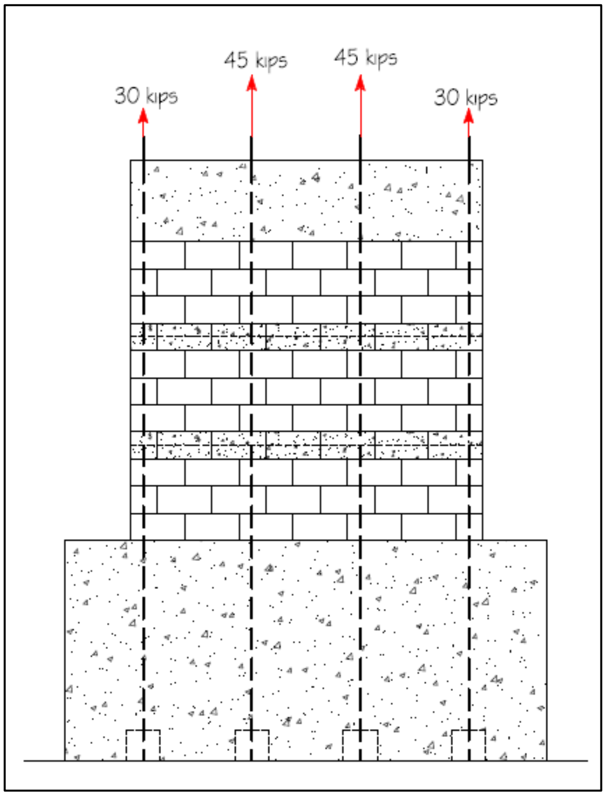


Figure (18): The variable amounts of post-tensioned loads within the wall

A very sensitive load cells were attached during applying the loads and during the tests to each post-tensioned bars. A hydraulic jack was used to apply the post-tensioned forces fig (19). These load cells recorded the forces in each bar during the tests and the can give indications about reaching the yield limit or not.

Figure (19): Applying the post-tensioned forces by using hydrolic jack

The footing and the bedstil were conected together to the rigid floor by using eight 1.5 diameter post-tensioned high strength bars to insure a perfect fixation for the walls.in adition, a thick layer of high strenght hydroston were put between the footing and the bedstil to prevent any sliding or sliping fig(20).

Figure (20): Fixing the footing and the prdestel to the floor with layer of hydroston between them

3. Materials properties

Material properties are summarized in Table 1. Many samples were taken to be tested in 28 days and in the day of testing the walls to get the exact properities of the materials in the day of tasting fig (21),(22),(23). A Midwest brand mortar and high strength groute were used. An all thread 1inch diameter grade 150 DSI post-tensioned bars with its standard accesoroes were used in this project fig (24).

Table 1 : Matrerilas propereties

Items	**Tests**	**Results**
Mortar	Compressive strength f’c (28 days) ASTM C109 / C109M - 13	2820 (psi)
Grout	Compressive strength f’c (psi) (28 days) ASTM C1019 - 13	4240 (psi)
Masonry block prism	Compressive strength f’c (psi) (28 days) ASTM C1314 - 12	3580 (psi)
Footing and pedestal	Compressive strength f’c (28 days) ASTM C39 / C39M - 14a	6100 (psi)
Top beam	Compressive strength f’c (28 days) ASTM C39 / C39M - 14a	10200 (psi)
High strength post-tensioned bars	Tensile strength fy ASTM A325M	154000 (psi)

Figure (21): Mortar testing

Figure (22): Grout testing

Figure (23): Masonry block prism test

Figure (24): DWI post-tensioned bar system

4. Test details

The walls were tested in a hydraulically controlled MTS Testing system (rigid wall) of 400 kips capacity under controlled rates of ram frequency. Fig (25), (26)

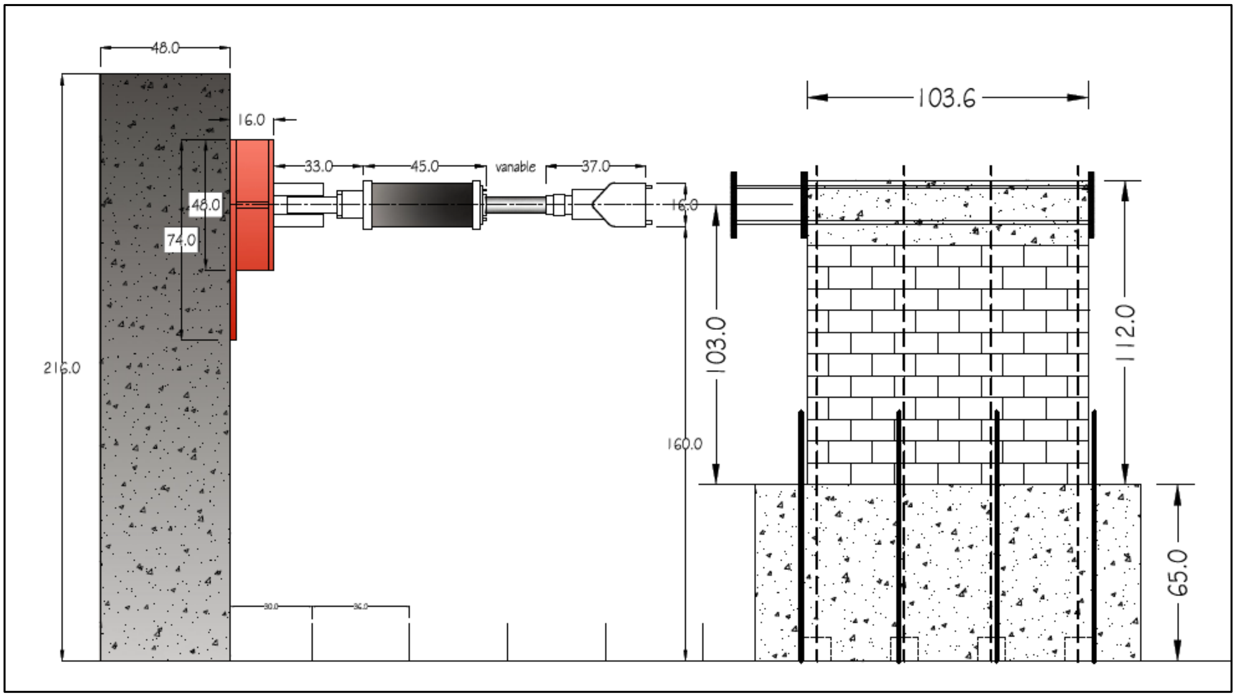


Figure (25): Test setup

Prior to testing, a wide measurement system was installed and hooked up the data accusation box to collect the data with 10 reading per second. The measuring system consists of LVDT's to measure the lateral and vertical displacements, strain, and drifting for the top beam, wall, footing and pedestal. The force in each post-tensioned bar was monitored.

Load was monitored by independent measurement across the hydraulic ram stroke was measured by an internal LVDT. However, stroke measurements were affected by machine flexibility; an accurate measurement for the stroke was measured by other external LVDT.

Any expected sliding or rocking of the footing and pedestal was monitored.

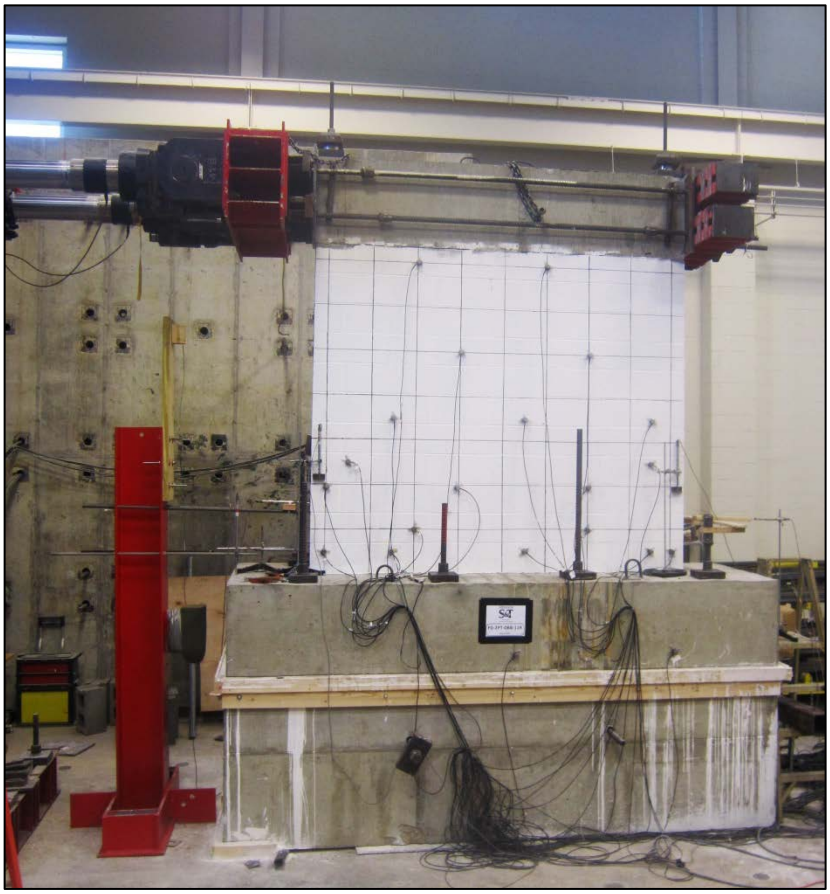


Figure (26): Specimen ready for testing

Regarding to the loading rate, a constant frequency was used to control the loading. Therefore; each single cycle has been finished within 50 seconds. This testing regime has been adopted by FEMA to give a better simulation of the seismic action fig (27).

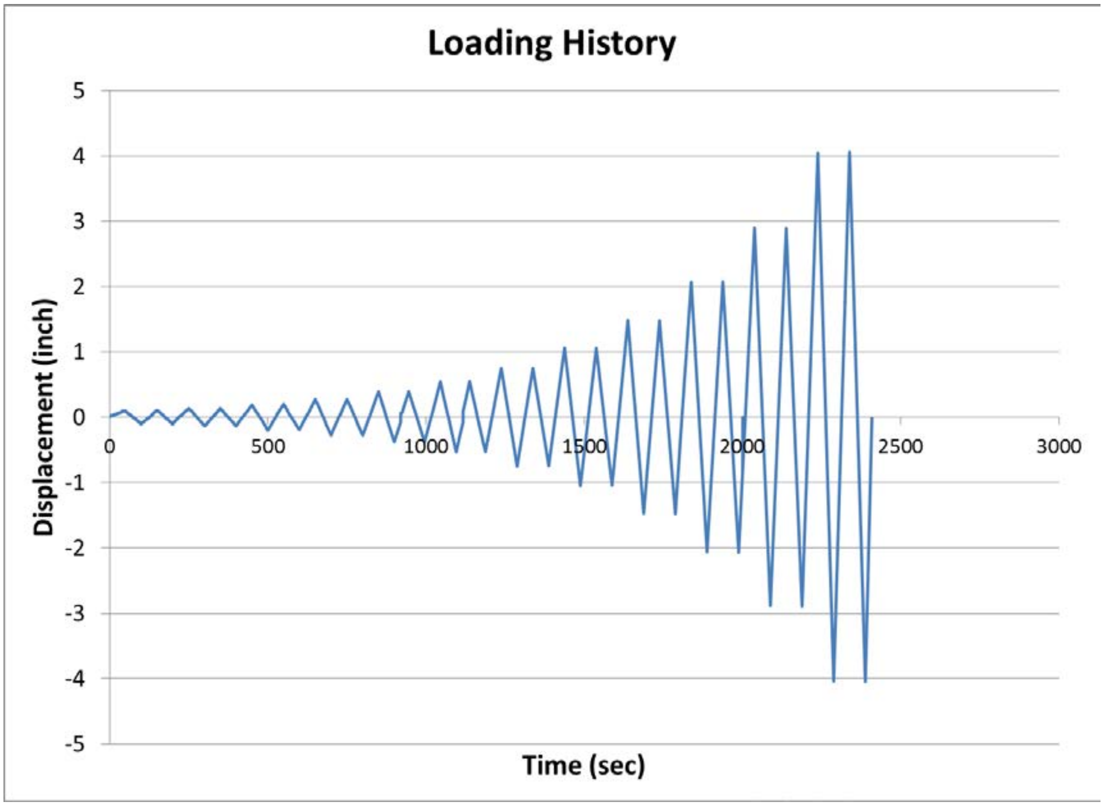


Figure (27): Loading history

5. Experimental Results

5.1. Wall 1

This wall contains four post-tensioned bar with 32 inch spacing between them. The force in each bar was different from the other ones fig (28). This wall contains shear reinforcement as a two horizontal bond beams. These beams were reinforced with 1#4 bar for each beam. The bars in each bond beam were hocked around the post-tensioned bars fig (29).

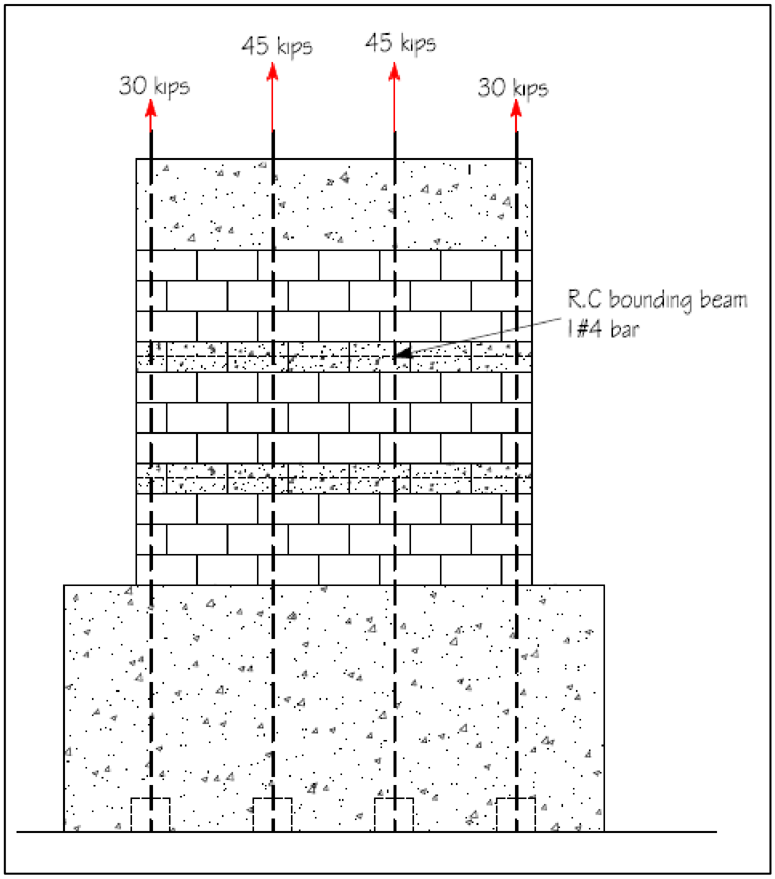


Figure (28): Details of wall 1

Figure (29): Hocking the bond beam reinforcement to the post-tensioned bars.

A very sensitive strain gages were attached to the reinforcements of the bond beams. They were attached close to the post tensioned bars.

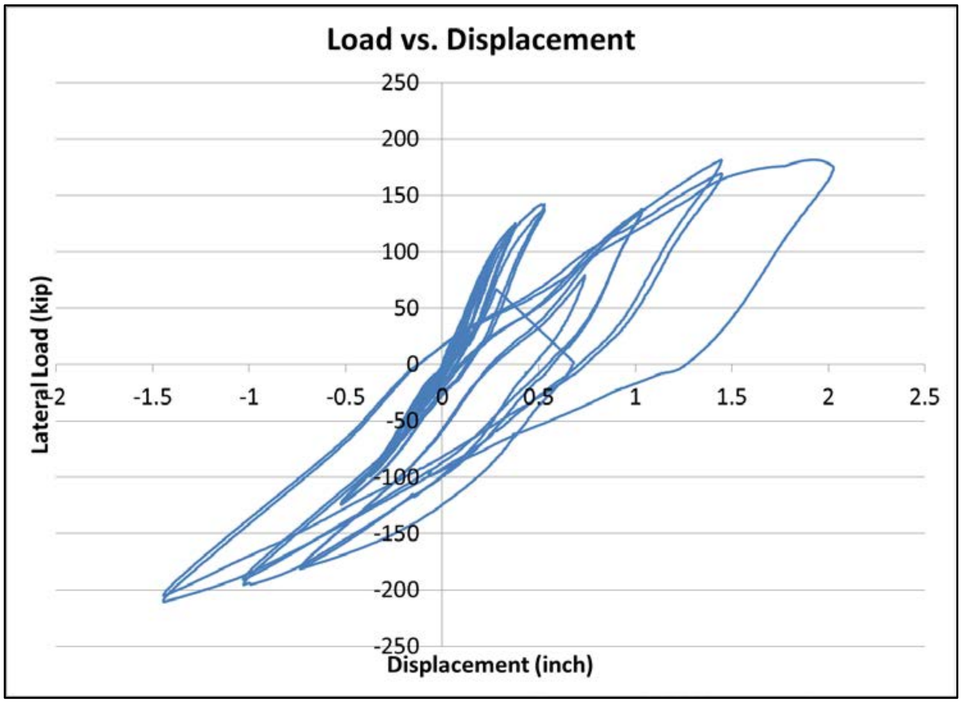


Figure (30): Lateral load vs. Displacement curve for wall 1

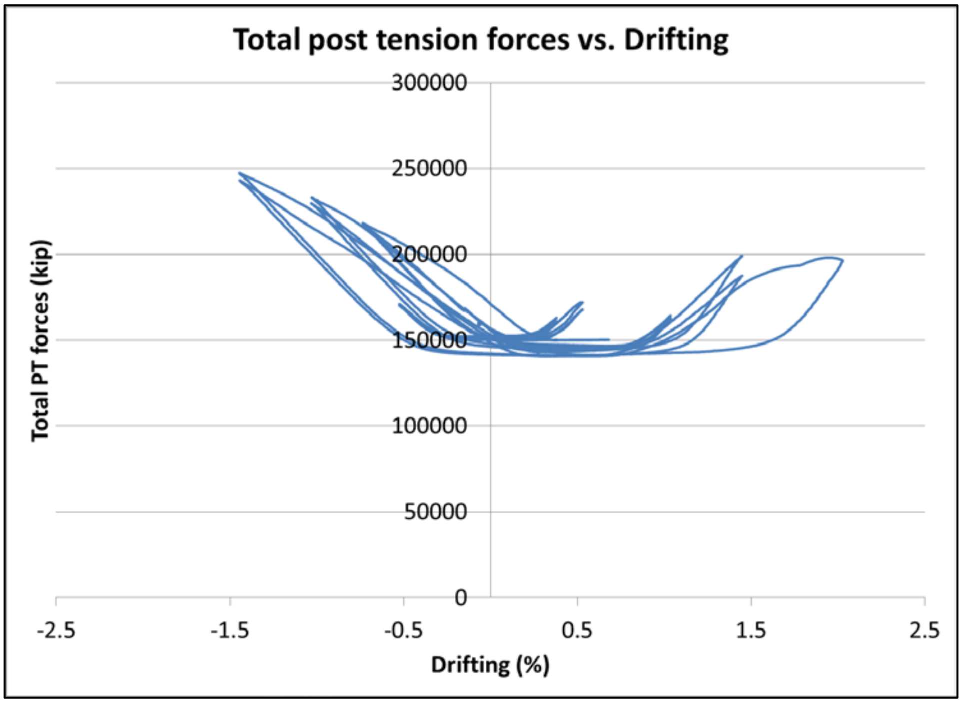


Figure (31): Total post tension forces vs. drifting curve for wall 2

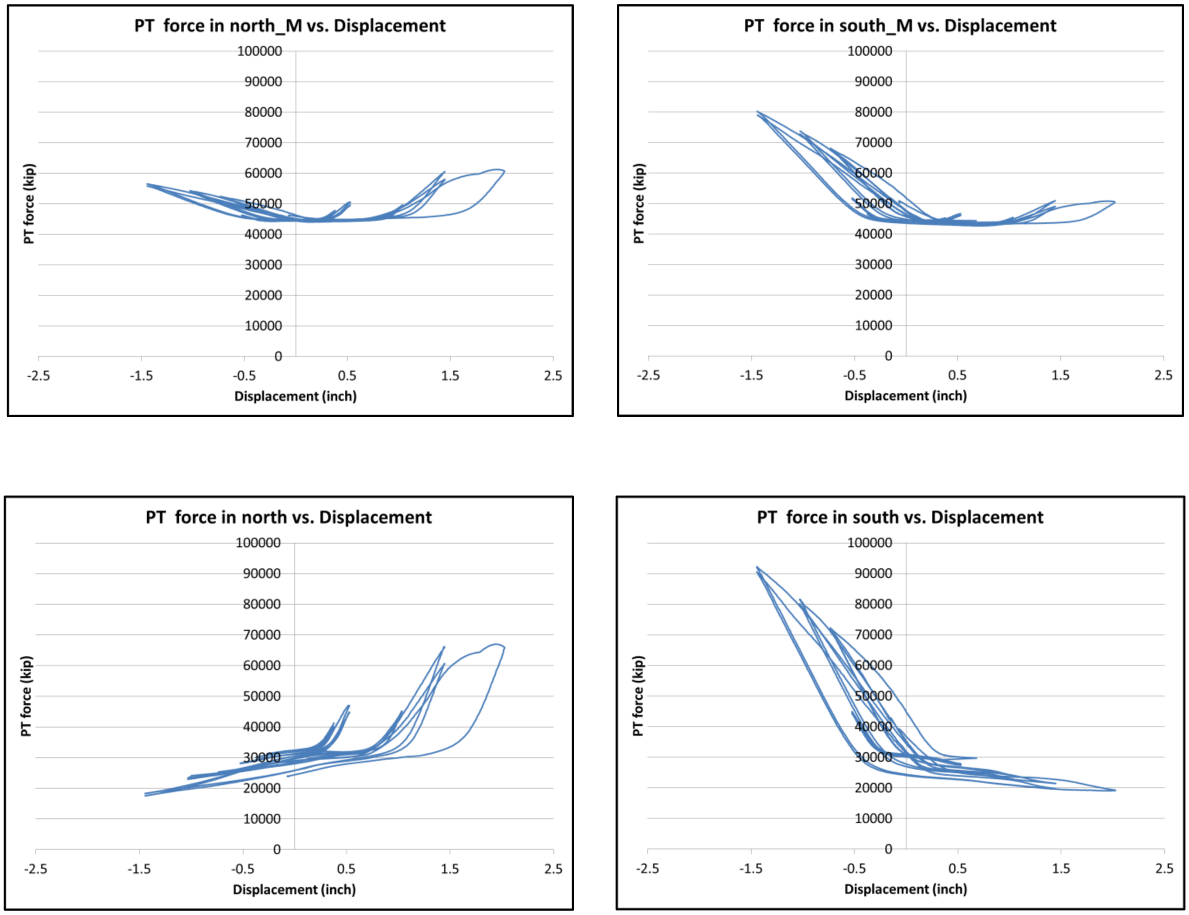


Figure (32): Post-tension force in each bar vs. Displacement curve for wall 1

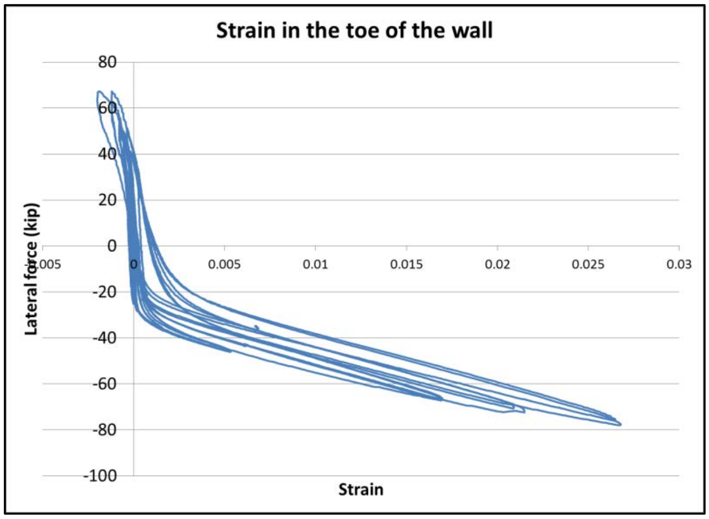
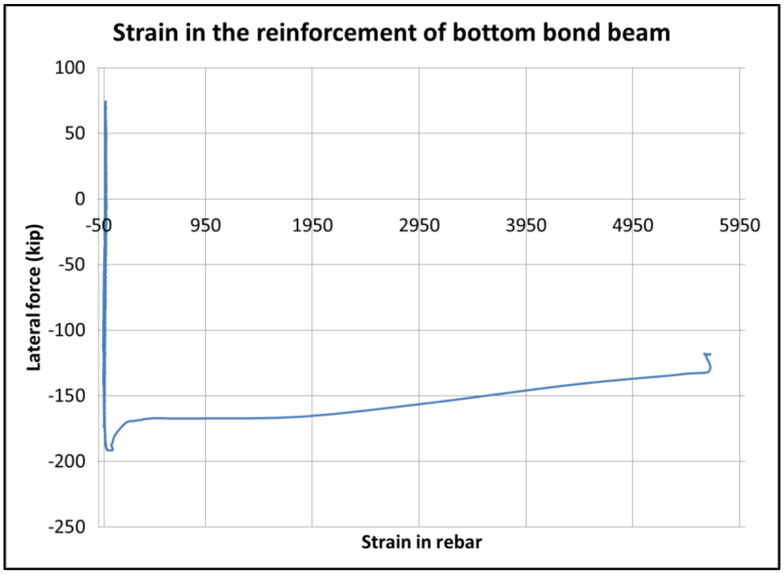


Figure (33): Strain in the north toe of wall 1



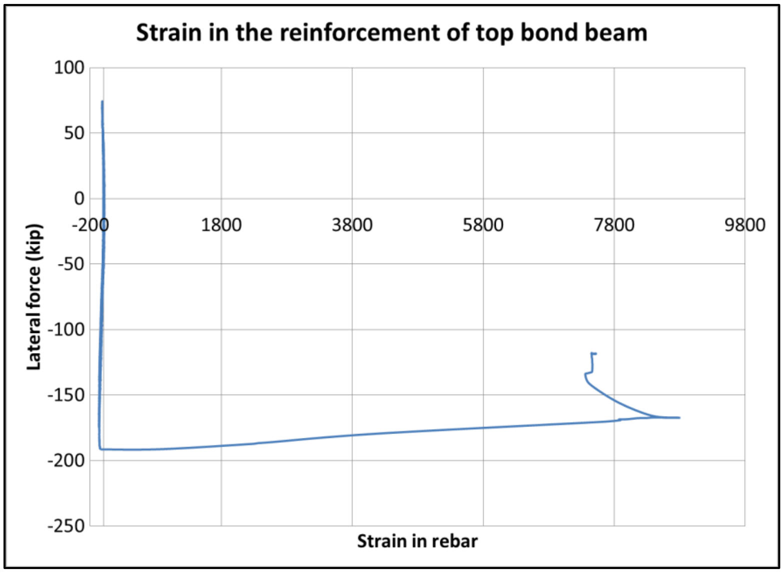


Figure (34): Strain in the reinforcement of the upper and lower bond beams

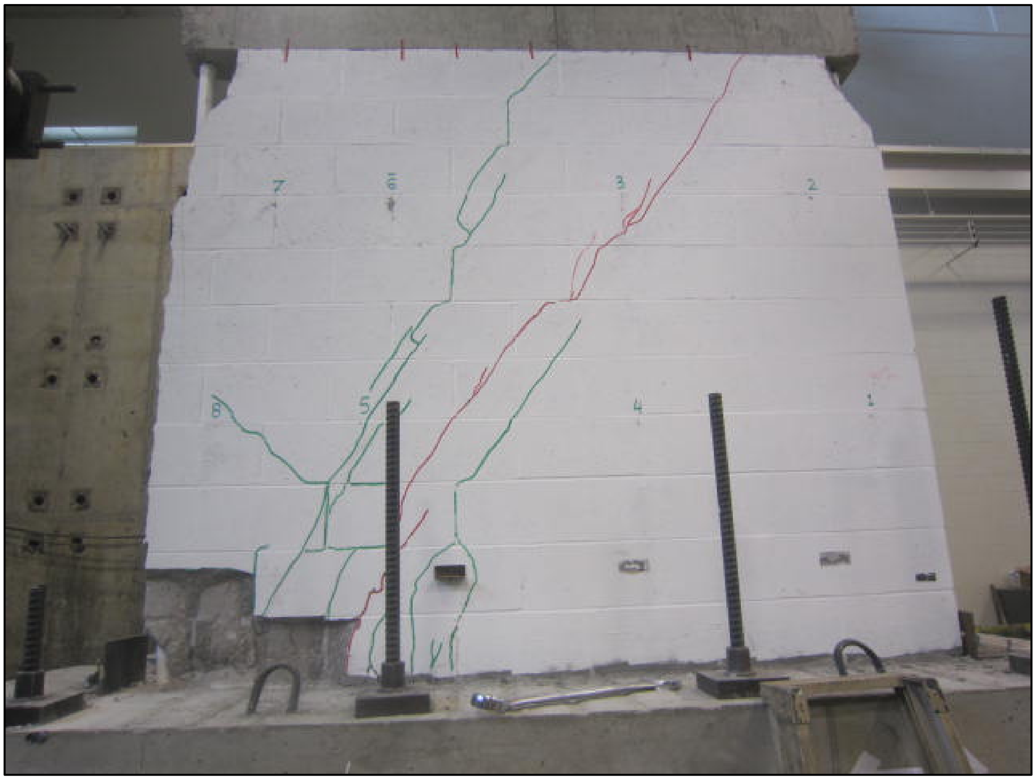


Figure (35): The general mode of failure for wall 1

Figure (36): The toe failure for wall 1

Failure mechanism:

The small spacing with the horizontal bond beams made the wall very strong and very stiff. As results, the shear failure was dominant fig (35). However, the post-tensioned bars were in the elastic range fig (32) but the shear reinforcements have yielded fig (34). The toe crashing was very clear fig (35). The strain in the toe was very high comparing to the maximum strain in concrete fig (33).

5.2. Wall 2

This wall contains four post-tensioned bar with 32 inch spacing between them. The force in each bar was different from the other ones fig (32). This wall did not contain any shear reinforcements or bond beams.

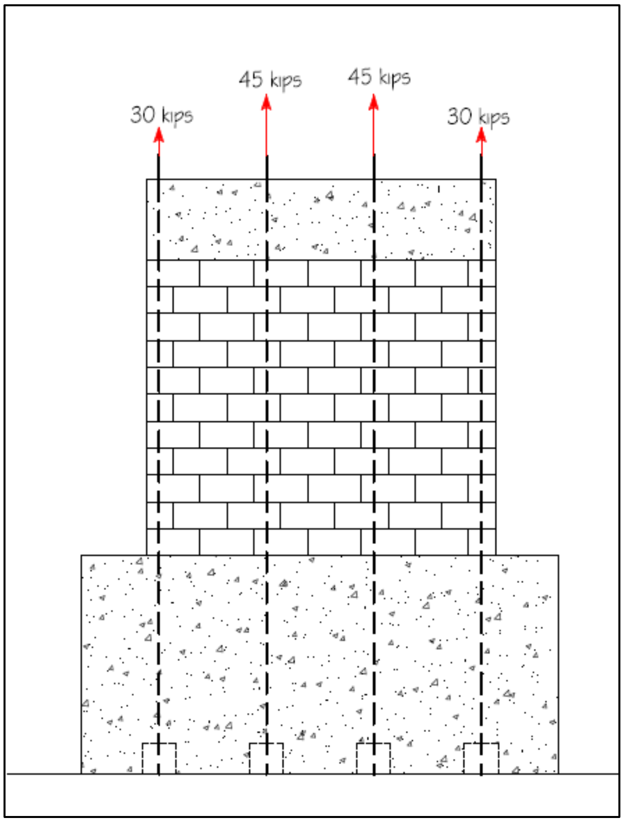


Figure (37): The details of wall 2

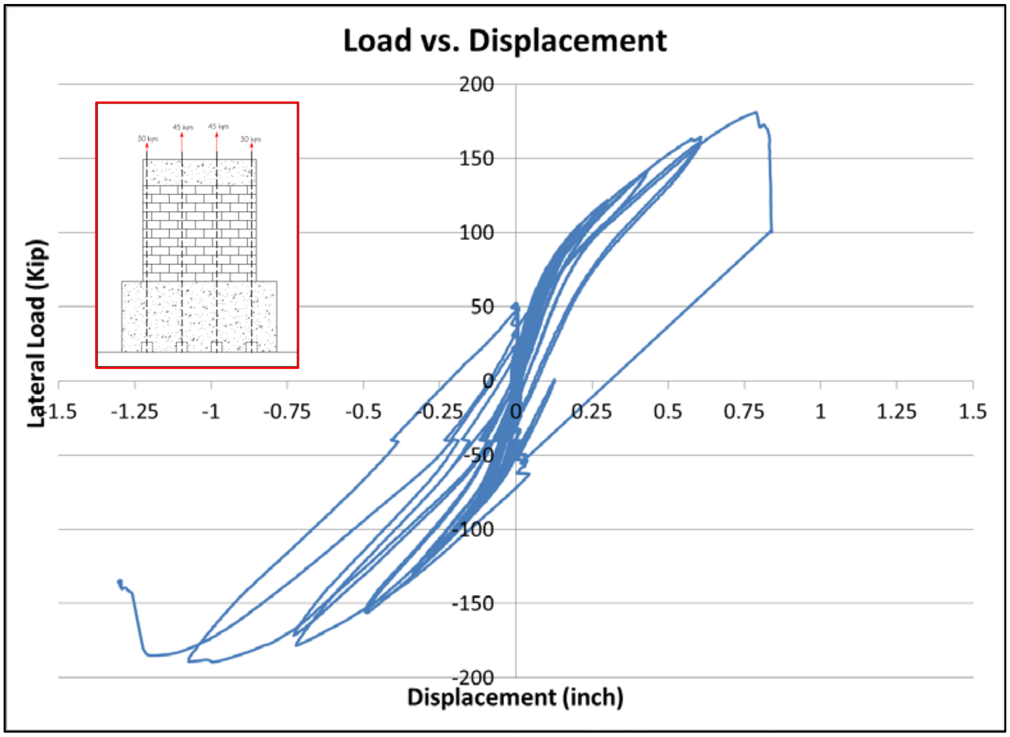


Figure (38): lateral load vs. Displacement curve for wall 2

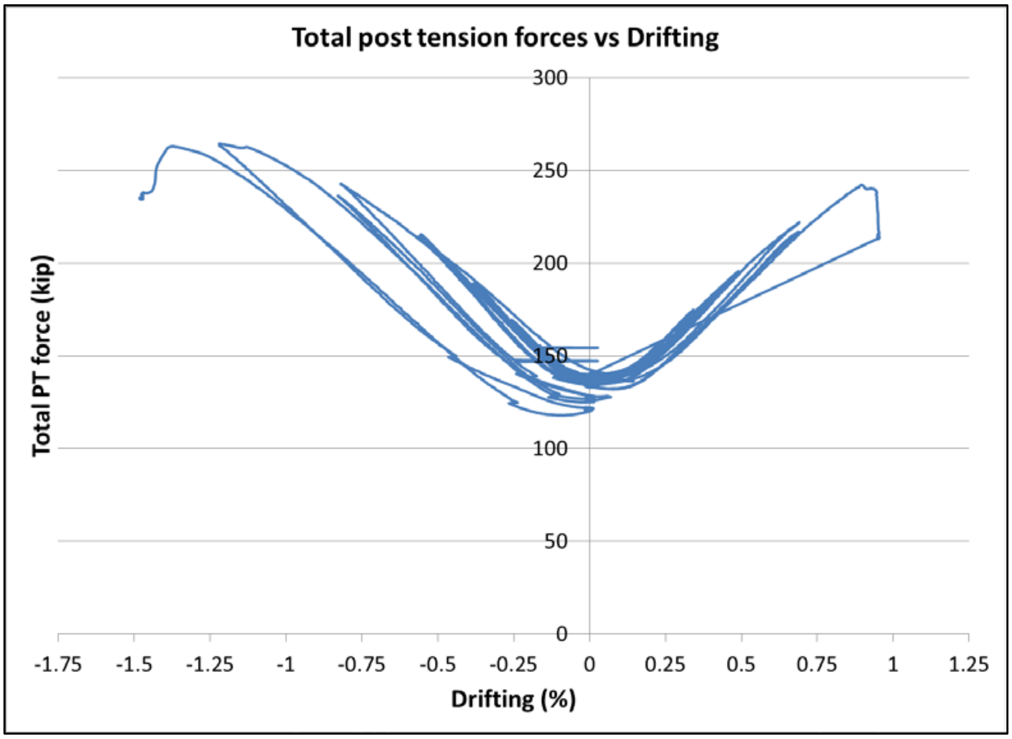


Figure (39): total post tension forces vs. drifting curve for wall 2

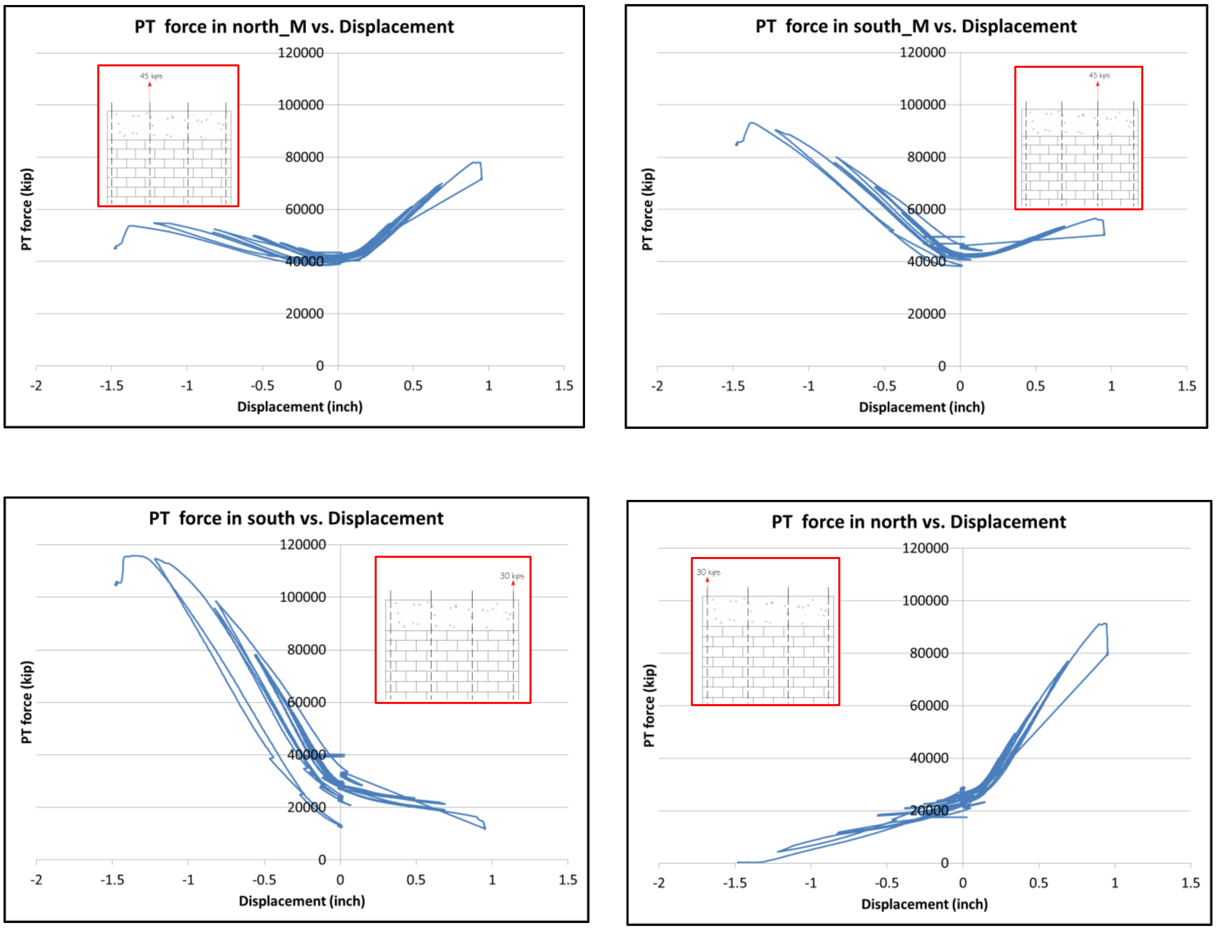


Figure (40): Post-tension force in each bar vs. Displacement curve for wall 2

Figure (41): The general failure of wall 2

Figure (42): The toe failure for wall 2

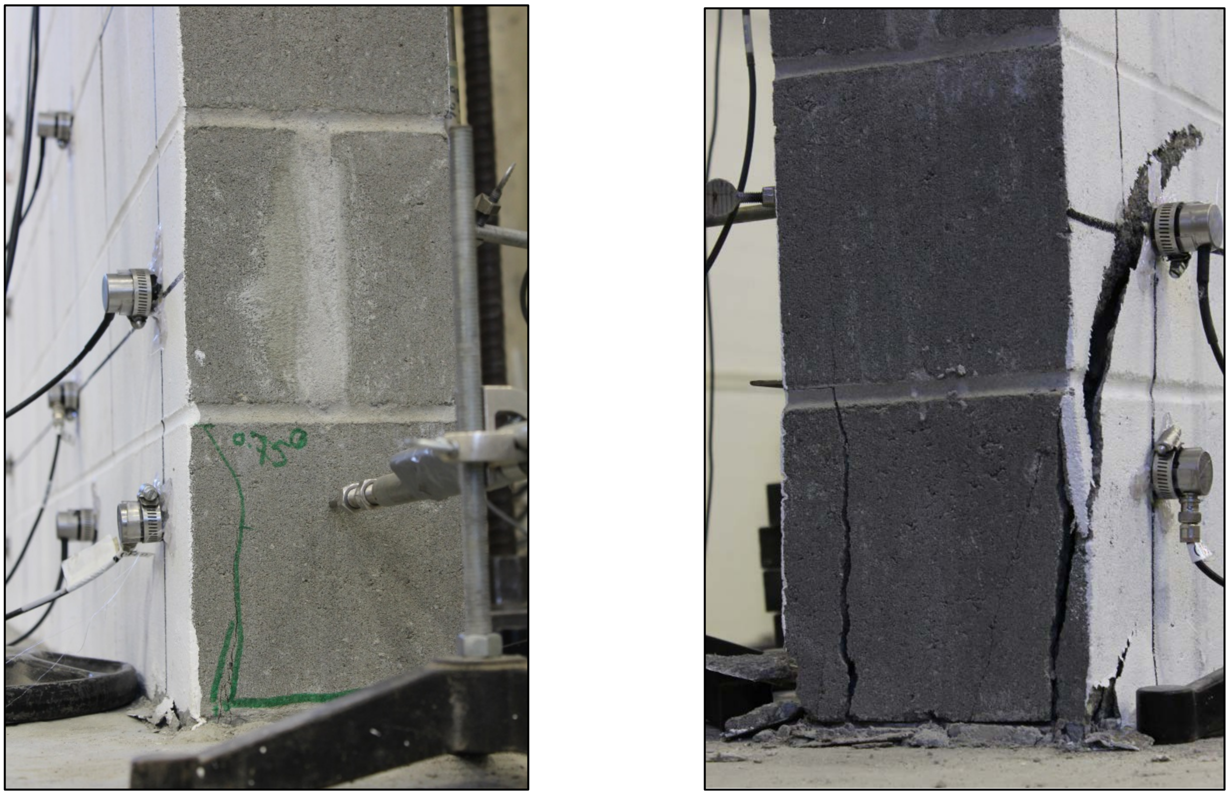


Figure (43): The first crack in the toe at 0.75 inch displacement

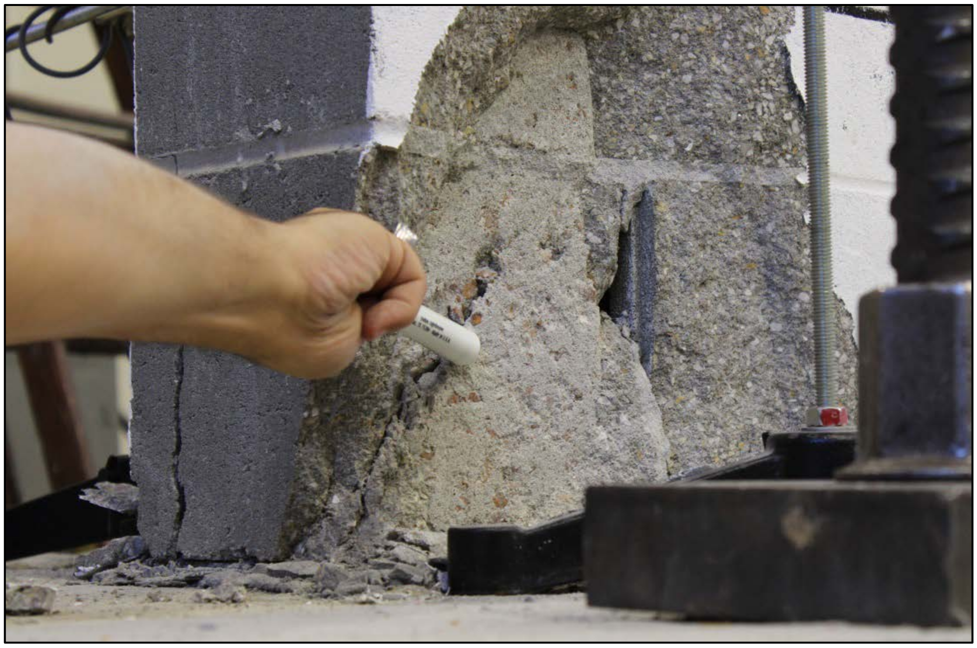


Figure (44): Developed toe crack

Failure mechanism:

The small spacing made the wall very strong and very stiff. As results, the shear failure was dominant fig (41). However, the post-tensioned bars were in the elastic range fig (40). The toe crashing was very clear fig (35). The strain in the toe was very high comparing to the maximum strain in concrete fig (43), (44), and (45). There is no that much impact of removing the bond beams in wall 2.

5.3. Wall 3

This wall contains two post-tensioned bar with 96 inch spacing between them. Each bar has the same amount of force fig (45). This wall did not contain any shear reinforcements or bond beams.. At the end of the day, this wall has the same amount of axil post-tensioned force. However, this force had been applied by two post-tensioned bars rather than four.

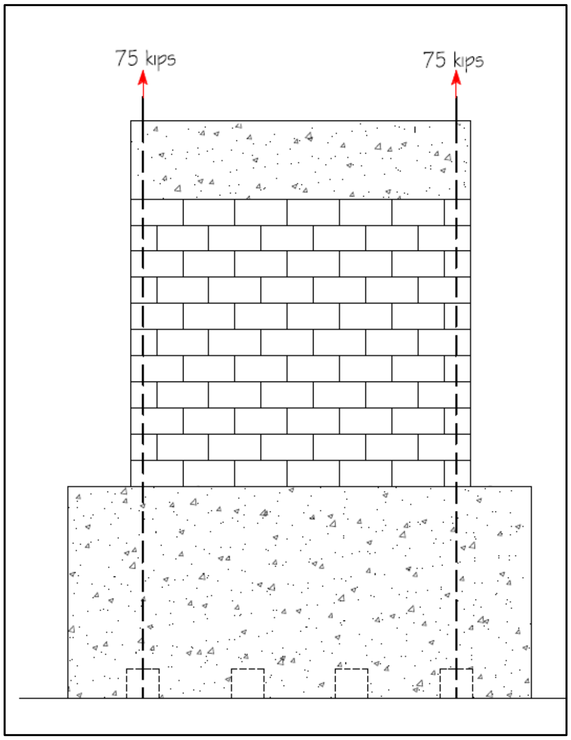


Figure (45): The details of wall 3

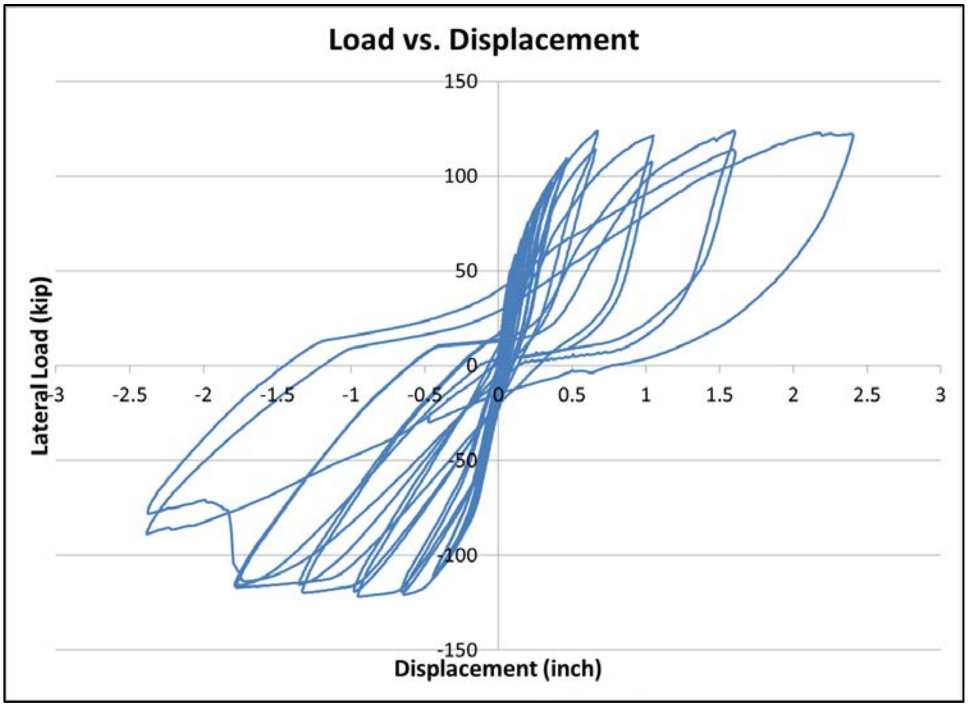


Figure (46): lateral load vs. Displacement curve for wall 2

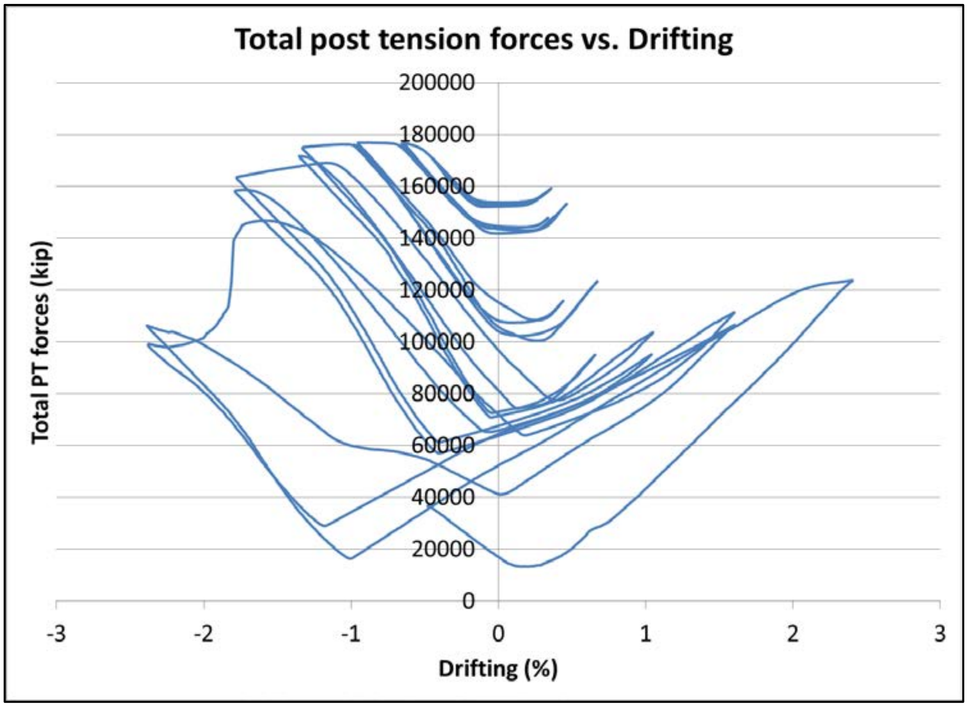


Figure (47): total post tension forces vs. drifting curve for wall 2

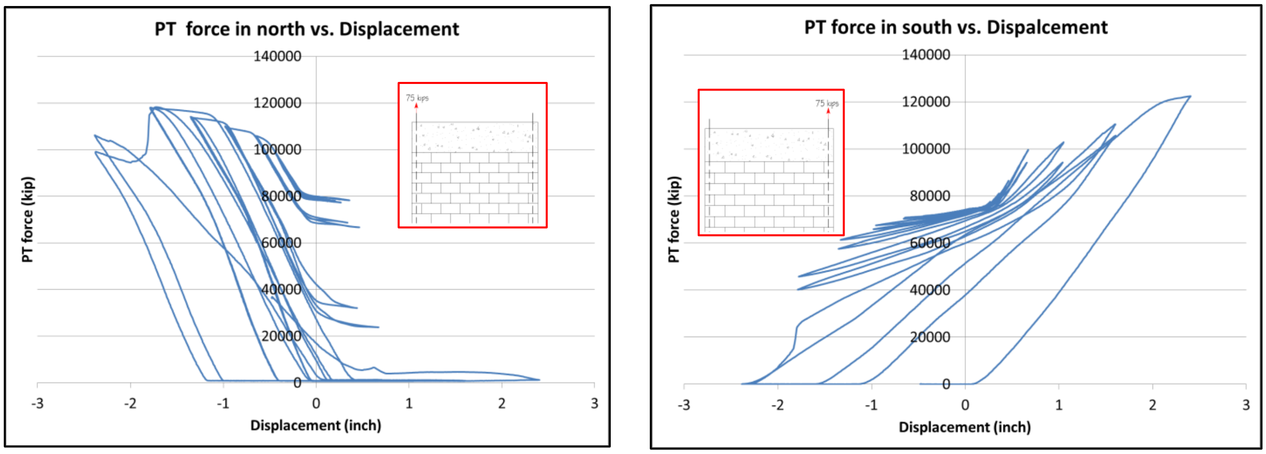


Figure (48): Post-tension force in each bar vs. Displacement curve for wall 3

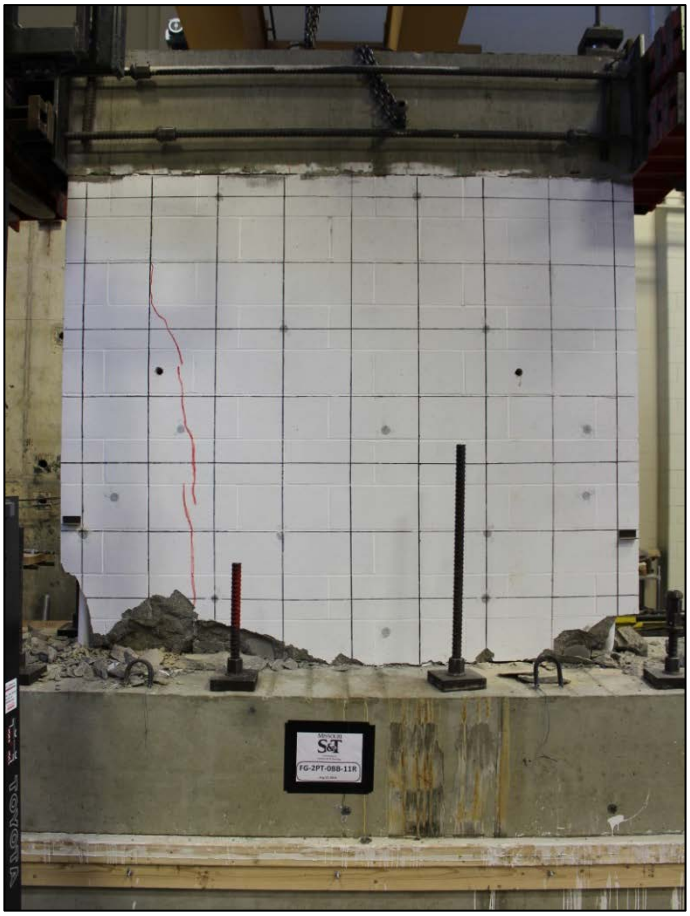


Figure (49): The general mode of failure for wall 3

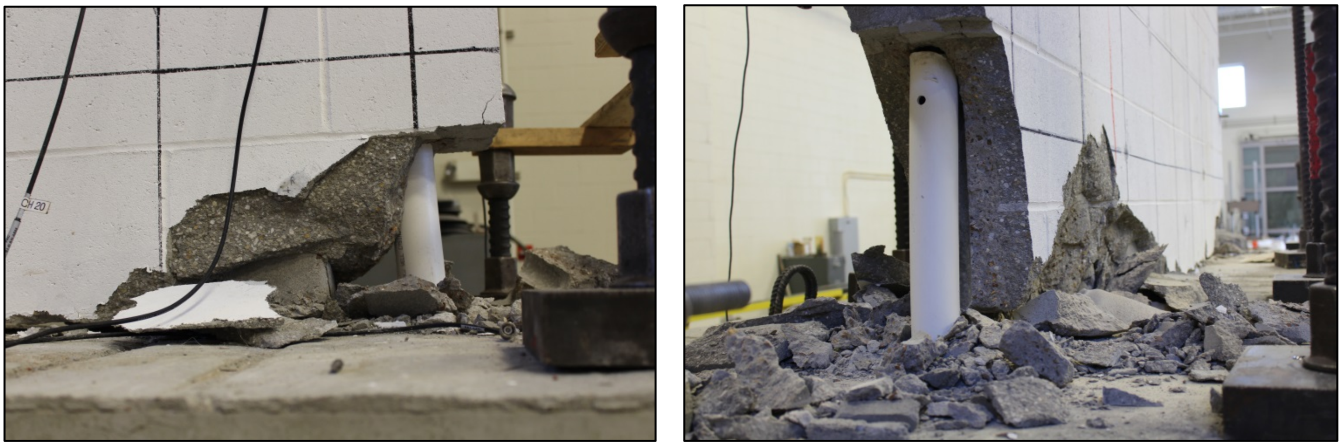


Figure (50): the toe failure in wall 3

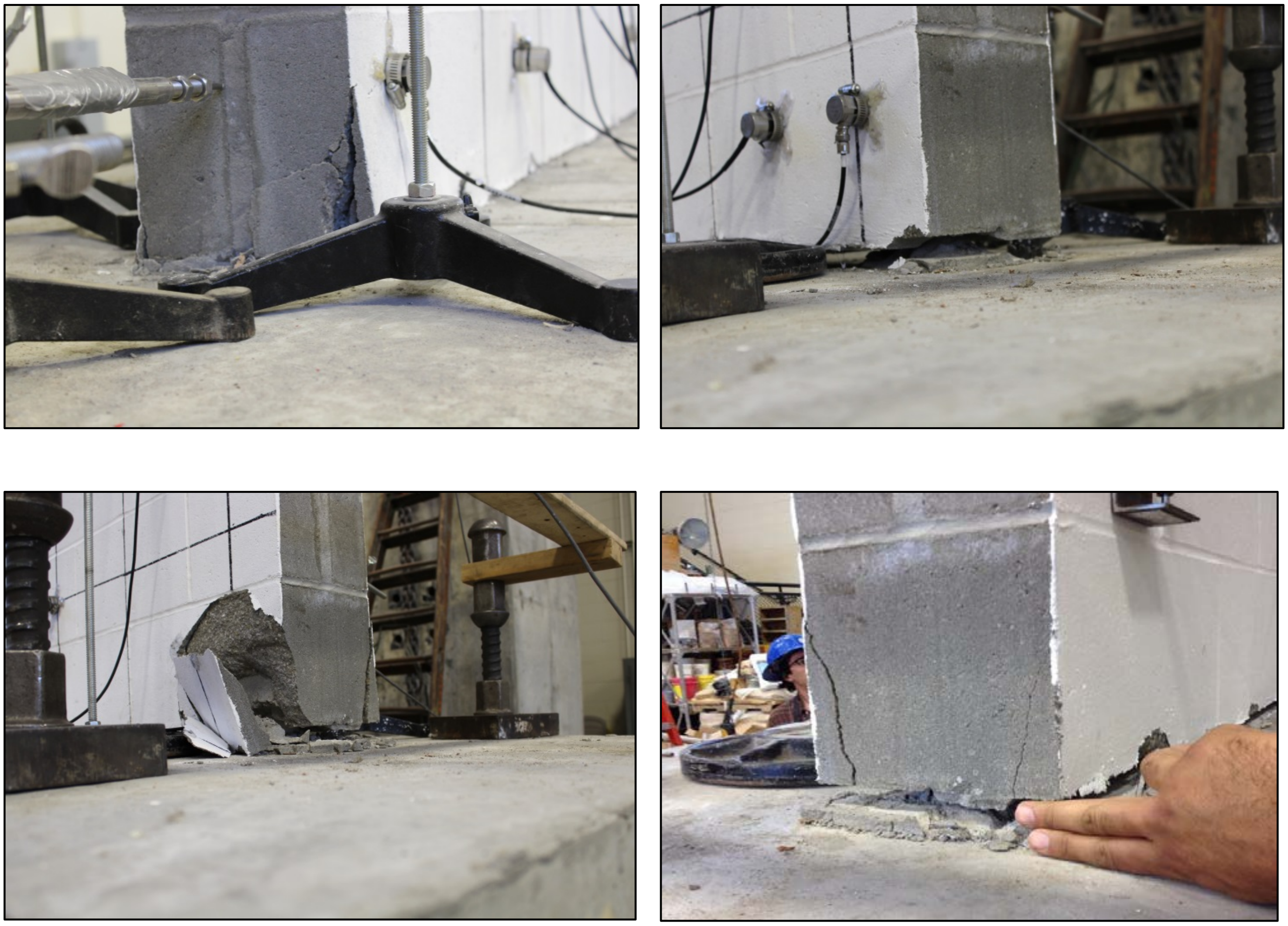


Figure (51): Cracks developing in the toe of wall 3

Figure (52): Moving the wall after testing

Even after failure, the wall still acts as a one piece fig (52). Therefore; it is very easy to repair the toe only by replacing the broken or crashed blocks to give back the original strength to the wall.

Failure mechanism:

The large spacing without the horizontal gave the wall a significant amount of drifting. As results, the shear failure was not dominant fig (49). However, the post-tensioned bars have yielded fig (48). The toe crashing was very clear fig (50). The strain in the toe was very high comparing to the maximum strain in concrete. Due to the big difference in forces between the two ends of the wall, a vertical crack developed close to the edge of the wall fig (49). This fact was stated before by using finite element model with large spacing between the post- tensioned bars.

6. CONCLUSIONS AND RECOMMENDATIONS

- Based on the tested walls, it can be stated that fully grouted post-tensioned masonry walls has a great in plane capacity to resist the shear forces.
- The bond beams does not have a significant impact on the shear resistance of the in-plane shear strength of PT-MWs.
- PT-MWs act as a rigid plate and can resist a significant amount of forces after developing cracks in their toes especially if the post-tensioned bars did not yielded.
- Spacing between bars have significant effects on wall cracking and strength

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