

**TESTING AND MODELLING OF A REINFORCED CONCRETE SHEAR WALL
SUBJECTED TO COMBINED LATERAL AND AXIAL LOADING**

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Abstract

Reinforced Concrete (RC) slender shear walls are an integral structural system in modern mid-rise and high-rise structures. RC shear walls provide stability and stiffness to structures against lateral loads such as wind or seismic events. The capacity of shear walls to dissipate lateral excitations are affected by many factors, specific to this research however, the amount of axial loading being one. Axial loading in buildings is generated from gravity loading due to the live and dead loads of the structure. Higher axial loading requires higher lateral loads to displace the shear walls, however, during seismic events higher axial loads reduce the shear walls' abilities to dissipate energy. During these seismic events, the hysteretic response of shear walls with axial loading exhibits a pinching effect. Not only does this signify reduced energy dissipation, but this pinching effect is also representative of lower residual drifts and higher stored elastic potential energy. The experimental and numerical testing of a large-scale RC slender shear wall was conducted herein to investigate these behavioural characteristics. The results from testing using combined axial loading and reverse cyclic lateral displacements demonstrated that higher axial loads increased the lateral load required to displace the wall, reduced the residual displacements, increased the amount of stored elastic potential energy during the loading cycles, and confined most of the damage within the bottom half of the wall. The numerical analysis and assessment of the wall required multiple changes and an investigation of various constitutive models to achieve a satisfactory simulation.

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1 Introduction

1.1 Background

1.1.1 Reinforced Concrete

Understanding the causes of structural failures and how to mitigate them provides a pathway to progress and improve structural design. It is necessary to use the lessons learned from these failures to ensure they do not reoccur. One of the core values gained from these failures is the need to safeguard against loss of life. This has led to improvements in design standards and building codes to aid in mitigating these failures.

Outside of a laboratory setting, the ability to capture the behaviour of a slender Reinforced Concrete (RC) shear wall during a seismic event from an undamaged state to the point of collapse is impractical nor is it safe to do so. Understanding the response of shear walls subjected to seismic loading is crucial to observing their behaviour during seismic events. In previous experimental testing on shear walls, done by Alaa Abdulridha [1], Mohammed Zaidi [2], Mena Morcos [3], and Michael Soto [4], lateral loading was not combined with applied axial loading. The addition of applied axial loading provides a more realistic representation of the loading imposed on a shear wall during a seismic event. This exclusion has created a gap in understanding the response of shear walls during a seismic event. The addition of applied axial loading provides a more realistic loading condition that will provide a better comparison against shear walls that incorporate emerging structural materials and alternative reinforcement detailing.

The effects of seismic events on reinforced concrete structures can be devastating. Inadequate understanding of the behaviour of key structural components can result in more than irreparable damage. The possibility of structural collapse endangering the lives of the building occupants and those in the surrounding areas is not acceptable. Developing an understanding of the deficiencies that RC structural components exhibit during seismic events can allow for identifying areas of improvement, such as improved designs or increasing lateral resistance.

Figure 1.1 illustrates a step-by-step understanding of how RC buildings respond to seismic events. After low levels of lateral displacement, reinforced concrete will experience the onset of damage. In terms of a building, it will possess the ability to remain operational. At this stage,

cracks begin to propagate. When lateral displacements increase, there will be permanent cracking, possible delamination, but the building will remain repairable and stable. Eventually, the building will reach a stage where all structural integrity is lost and it is no longer repairable. It will be deemed life safe. Life safe is the point where the structure needs to be evacuated as it has reached the onset of collapse. Demolition of the structure is the only option at this stage.

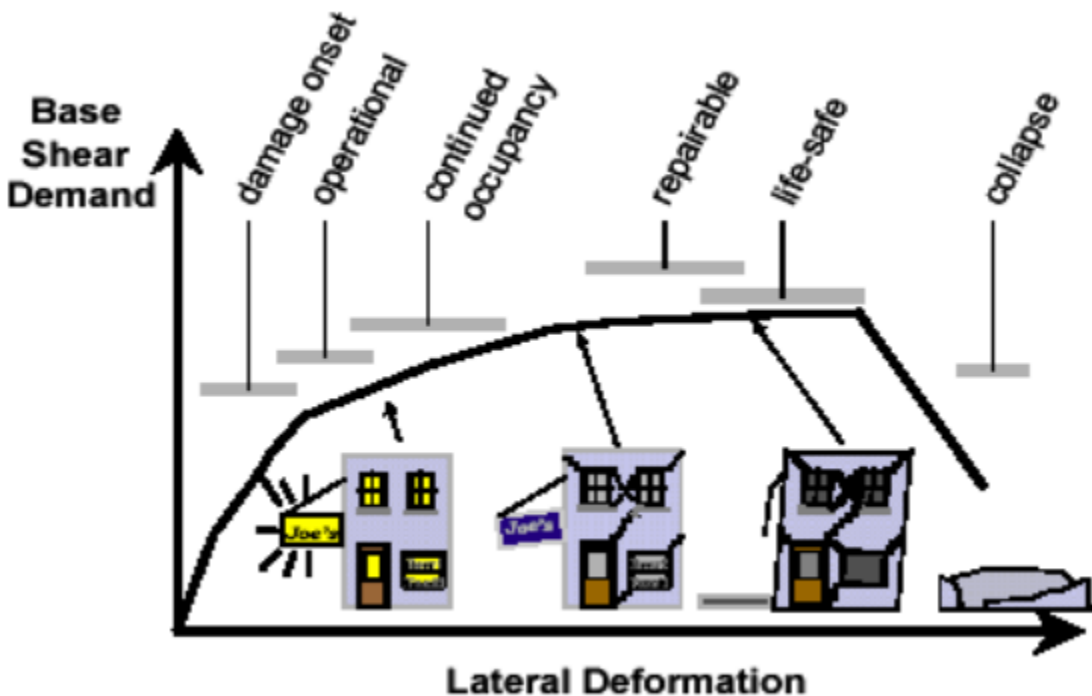


Figure 1.1: Level of Damage based on Shear compared to Lateral Deformation [5]

One of the fundamental concepts used in the analysis of structures is the tributary areas of beams and slabs and how they transmit their loading towards the vertical support members. In concrete buildings, the weight of the slab is transferred to the columns and walls. In some research, the effect of this weight is not captured. This added axial loading affects the stiffness and strength, and also impacts the transient and residual drifts. Though this is captured in a laboratory setting, in some situations, these failures can be catastrophic. Examples of catastrophic failures are building collapses due to crushing or buckling, which in a controlled environment does not occur as the testing is halted before collapse can occur. During seismic events however, this is not possible and collapses like the 2010 Chile earthquake, or the most recent 2023 Turkey-Syria earthquake. With higher axial loads present, failures can occur.

Structures in modern construction must conform to area specific seismic design standards and building design codes. These codes and standards provide a framework for designers to follow in order to create sufficient structural systems. Although structures, specifically buildings, can be made with a variety of materials such as timber and steel, reinforced concrete remains the most widely used construction material. Generally, reinforced concrete is used in the walls of the stairwells and elevator shafts in concrete structures, and many steel and timber buildings. In one way or another, reinforced concrete is routinely used as the seismic force resisting system of buildings. In addition, for mid-rise and high-rise buildings, reinforced concrete is attractive due to its cost and versatility.

Mid-rise and high-rise concrete buildings typically have a variety of concrete walls and columns. The main reason for this is that concrete lacks the ability to span long distances given its low tensile strength. For this reason, significant quantities of reinforcement would be required. Generally, RC buildings are used in the construction of condominiums as the walls provide natural dividers between the units. From an engineering point of view, more walls lead to higher lateral load resistance and increased stiffness. Thus, increasing the seismic performance of the building if designed properly.

Building codes provide a multitude of systems that are identified as acceptable Seismic Force Resisting Systems (SFRS). For mid-rise and high-rise structures, shear walls are a prevalent choice for the SFRS. Since the construction of these types of structures incorporate stairwells and elevator shafts, shear walls are a natural choice to form part of the SFRS. These walls serve as gravity loading support as well as lateral loading support. Lateral loading causes these walls to displace and oscillate laterally. Forms of lateral loading arise from wind and earthquakes. The walls must dissipate these oscillations and return to a state of equilibrium. Should there be inadequate reinforcement, concrete strength, or design, these walls can fail catastrophically.

1.2 Scope of Research

1.2.1 Reinforced Concrete Shear Walls

Shear walls are the primary lateral force resisting system in mid-rise and high-rise reinforced concrete buildings. Typically, during seismic events, depending on the magnitude, reinforced concrete will crack. Design standards and engineers will design RC to crack during

these events. The reason for this is the economic value in designing and building an RC structure to remain elastic is undesirable. The shear walls and columns would need to be so large that the building would be extremely costly and there would be minimal floor area. For this reason, design standards and engineers have minimum design requirements that allow RC to experience inelastic deformations during seismic events while still remaining serviceable.

1.2.2 Types of Shear Walls

Different types of reinforced concrete shear walls can be used in buildings. The two main types of shear walls seen in typical construction are slender shear walls and squat shear walls. The determining factor on whether a shear wall is slender or squat is the aspect ratio; the ratio of height to shear span length being less than, equal to, or larger than 2. In this thesis, the aspect ratio of the shear wall tested is 2.2, which classifies it as a slender shear wall.

1.2.2.1 Squat Shear Walls

The focus of this research is on slender shear walls, however a brief review of squat shear walls is necessary to understand the key differences. Squat shear walls are defined as shear walls with a height that is less than or equal to two times its length. The responses of these types of walls are dominated by shear-related mechanisms as illustrated in Figure 1.2. Squat walls generally experience higher axial and gravity loading than slender walls. Since shear generally governs, the crack patterns are typically inclined diagonal shear cracks.

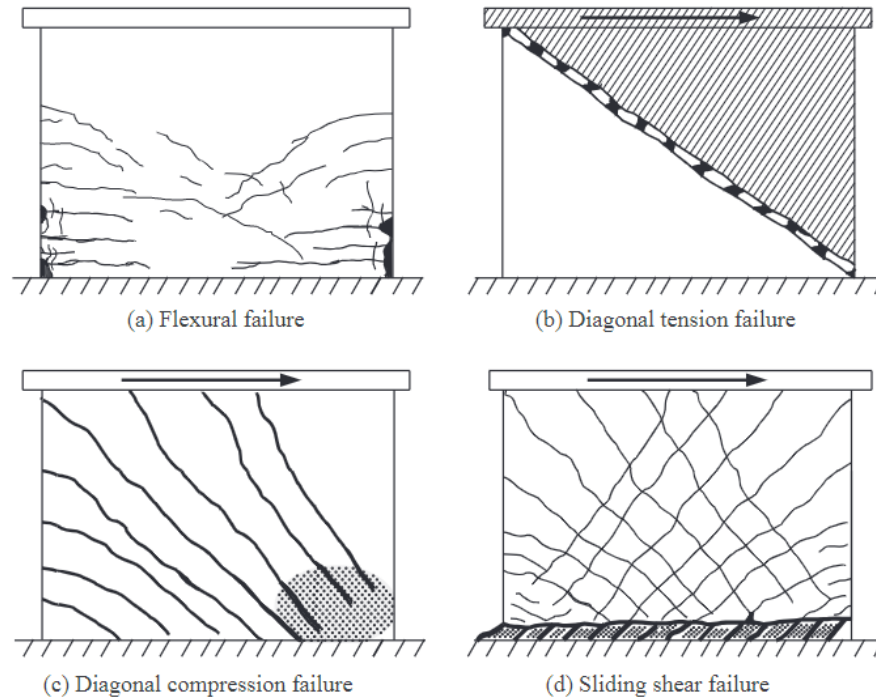


Figure 1.2: Squat Shear Wall Failure Modes [6]

1.2.2.2 Slender Shear Walls

Slender shear walls are defined as shear walls with a height that is greater than or equal to two times its width. This means that flexural-related mechanisms dominate the response. Slender shear walls can experience a variation of failures, as shown in Figure 1.3 below; generally, they can be classified as, sliding, flexure, compression, or tension failures. Typical slender shear wall crack patterns are inclined flexure-shear cracks which occur in the web and horizontal flexure-tension cracks which occur in the boundary regions.

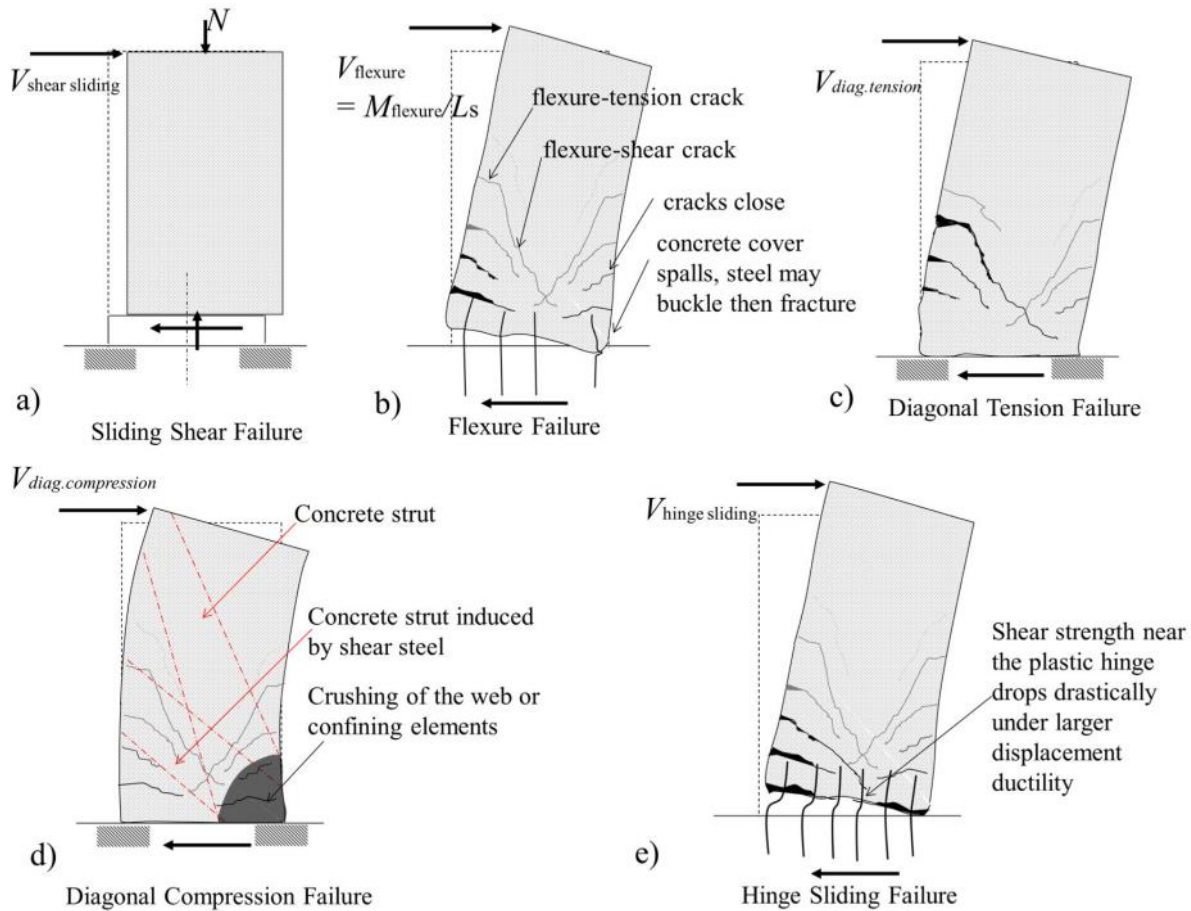


Figure 1.3: Slender Shear Wall Failure Modes [7]

1.3 Research Objectives

The main objective of this research is to experimentally test a reinforced concrete slender shear wall under combined reverse cyclic lateral displacements and axial loading. The purpose of this research is to monitor the response of the wall to combined loading, to assess the impact of the axial loading on the recentering of the wall and to develop a better understanding of how shear walls in mid-rise and high-rise buildings respond to seismic events. The anticipated result is the development of a test set-up which yields more realistic loading conditions generating data to give a better understanding of how axial loading impacts the transient and residual drift capacities compared to prior tests that did not explicitly include imposed axial loading. To achieve this objective, a large-scale reinforced concrete shear wall along with a specialized dual loading testing frame was constructed.

Secondary objectives of this study are:

- To provide test results from a wall that will serve as a control for future testing that will incorporate emerging materials such as SMA (Shape Memory Alloy) reinforcing bars and ECC (Engineered Cementitious Composite) in the plastic hinge region;
- To provide test data to corroborate numerical models developed through Formworks and analyzed with VecTor2 to predict the behaviour of RC slender shear walls subjected to combined lateral and axial loading. The corroborated numerical models will be used as the foundation for numerical models incorporating emerging materials noted above;
- To determine whether starter bars should be required in future tests with this set-up and loading condition.

1.4 Research Novelty

The specific RC shear wall presented herein will be used as a control wall for future experimental test programs that will be developed with emerging materials such as ECC and SMA. Though those walls will be novel based on the incorporation of the emerging materials, the novelty of this research is partly in the experimental setup and partly on the high level of axial loading imposed on the wall, which is at the high end for slender shear walls. The inclusion of axial loading is evident for different types of RC structural components; however, there is substantially less breadth on articles regarding shear walls. The novelty of this research is investigation of the effect of combined axial and lateral loading on a large-scale slender RC shear wall. Using axial loading on shear walls subjected to reverse cyclic loading is an understudied area and this research will aid in developing novel future work. In addition, the novelty of this research extends to the complementary numerical modelling. The modelling demonstrates how combined loading should be considered, the impact of explicitly modelling the $P-\Delta$ effect, and the importance of local effects at a crack given that the longitudinal reinforcement in the boundary elements fractured during testing. From a modelling perspective this includes a tension-stiffening model that considers and recalculates the fracture strain at a crack, the influence of strain gauges on the reinforcing bars, and a reinforcement hysteretic model that considers the accumulation of reinforcement strain at the crack.

1.5 Thesis Layout

The following thesis has been formulated into a sequence of seven (7) chapters. Chapter 1 introduces the research topic and concepts that have guided the study of RC shear walls subjected to combined lateral and axial loading. Chapter 2 encompasses an extensive literature review that focuses on RC slender shear walls subjected to combined loading and the different procedures considered by other researchers. Chapter 3 explains the construction of the shear wall, the test set up, and the instrumentation. This is followed by the testing program which was developed to test the wall. Chapter 4 illustrates the results of the experimental program. Chapter 5 provides a discussion of the experimental results. Chapter 6 provides complementary and preliminary nonlinear finite element modelling of the wall prior to and following the testing. Finally, Chapter 7 concludes the thesis and provides some insight on future work.

2 Literature Review – Testing and Modelling of Reinforced Concrete Shear Walls

2.1 Introduction

The main focus of this thesis is on reinforced concrete shear walls. The literature review, however, also includes shear wall research that involves novel and emerging materials. The purpose is that this research will serve as a control wall for a larger experimental program which will include novel and emerging materials. Reviewing experimental studies containing these materials will aid in identifying areas for future research on this topic.

This chapter will encompass four main sections of the literature review. The first section covers research conducted on reinforced concrete shear wall buildings following earthquakes. The second section will be on seismic testing and behaviour of shear walls without applied axial loading. The third section presents research pertaining to the behaviour of shear walls with combined loading, and the fourth section will be on shear walls with novel and emerging materials.

2.2 Reinforced Concrete Shear Walls in Buildings

2.2.1 Introduction

Understanding structural systems can help in identifying areas of weakness prior to the occurrence of seismic events. This knowledge can aid in reducing the economic damages as a result of these seismic events. As discussed throughout this thesis, shear wall systems provide added stiffness and resistance to lateral loading. Reinforced concrete structures are designed to experience plastic response rather than remain elastic. The reason for this is that the economic cost and member sizes would not be feasible for an elastic design. This section reviews systems subjected to seismic events and will highlight areas of concern and areas of confidence with these systems.

2.2.2 RC Buildings After Earthquakes

In 2010, an 8.8 magnitude earthquake occurred in Chile. In a case study by Wallace et al. (2012), it was noted that the majority of damage was experienced in newer buildings which were less stiff than older buildings. The observed damages included crushed concrete, spalled concrete, buckled vertical reinforcing bars, opening of 90-degree stirrups, and crack localization. In the majority of these cases, thin walls were an issue because the cover to wall thickness was

large, and when the cover concrete spalled off it greatly reduced the axial capacity of the wall [8].

Research by Massone et al. [9] provides a comparison of Chilean buildings and codes to US buildings and codes. Though the codes are not the focal point of this thesis, the findings found of Massone et al. provide important lessons. The stiff shear wall structures in Chile performed well with little to no damage during the 1985, 7.3 magnitude earthquake. The main reason for this performance is attributed to the buildings containing a large number of walls [9]. When walls are built thinner and taller, they experience higher axial loads. After the 2010, 8.8 magnitude earthquake, many of the newer and taller buildings in Chile experienced more severe damage than the older buildings. This is due to the thinner and taller walls of the newer buildings having a higher axial load ratio than the older buildings. This is something that requires further analysis, but due to the reduced width of walls, more floor area and weight needs to be supported by the reduced number of walls. The axial compressive capacity of shear walls is primarily a function of the compressive strength of the concrete and the cross-sectional area, assuming that slenderness is not an issue. If the cross-sectional area is reduced, the axial compressive capacity of the wall is reduced or the axial load ratio on the wall increases.

Lagos et al. [10] published a paper in 2021, 9 years after the above paper. It references the 2010 earthquake in Chile with some new findings. Between 1985 and 2010, 9970 residential buildings were erected in Maule, Chile. This area was affected by the 2010 earthquake. Out of these buildings, specifically buildings between 4 and 18 stories, 40 were severely damaged and needed to be demolished, and 4 collapsed [10]. The article does not state the type of SFRS used within these buildings. This is less than 1%. This paper provides an in-depth analysis on the need for lateral stiffness for seismic resilience. A solution to reduce lateral displacement demand is using walls with high lateral stiffness.

This research is supported with the findings in 2019 by Ozkul et al. [11]. This article reviewed two buildings that were damaged from the 7.2 magnitude Van 2011 earthquake in Turkey. The research on these two buildings was performed with SAP2000. Both buildings were constructed with poor material properties which resulted in low ductility and strength. A parametric study conducted on the two buildings considered the actual material properties, and

what is deemed good material properties. The model with the better material properties improved the buildings damage resistance in some areas but the better material properties did not resolve the inadequate shear wall design. A SAP model with properly designed shear walls provided significantly improved seismic performance of the building. Part of the study considered improving the design of the shear walls but not the columns; this alone improved the seismic performance of the building.

2.3 Early Shear Wall Research

Shear wall research has significantly enhanced within the last fifty years. This area of study has been continuously improved upon as will be discussed in the following sub sections. Early research was conducted by Oesterle et al. [12] on three RC wall configurations. A total of nine walls (rectangular, barbell, and flanged shapes) with aspect ratios of 2.4 were constructed and tested. The walls were loaded laterally at the mid height of the top beam and were not subjected to imposed axial loads. One of the key differences between the walls were the confinement within the boundary region. In some of the walls, the boundaries were unconfined, while in others they were confined. This study highlighted that confinement provided improvements to the displacement ductility but did not affect the rotational ductility. As well, inelastic performance was increased due to the confinement by delaying the reinforcing bar buckling within the boundary regions.

A second paper by Oesterle et al. [13] was published a few years later with seven more shear wall tests. The new walls were subjected to constant axial compressive loading during testing. Five different failure modes were observed between the seven new tests and the nine previous tests: bar fracture, bar buckling, instability of the compression zone, web crushing, and boundary crushing. These papers and another paper in 1980 by Oesterle, Fiorato, and Corley [14] on reinforcement details and design recommendations for earthquake resistant structural walls aided in creating the framework that the following papers used to research and test shear walls.

2.4 RC Shear Wall Research without Axial Loading

Research has been conducted on reinforced concrete shear walls without the inclusion of axial loading for a multitude of reasons. Many of which are due to: lack of resources, lack of available space, or lack of research on the topic. Though the basis of this research presented

herein is on combined loading, it is important to review the responses of shear walls without applied axial loading. In this section, the recentering characteristics, residual crack widths, and damage to the vertical reinforcement will be the specific focus.

There is a wide range of research that has been performed on the behaviour of slender reinforced concrete shear walls when subjected to seismic loading conditions. More focus is being placed on the behaviour of these types of walls when subjected to combined lateral and axial loading. Since the research presented in this thesis is a continuation of a larger ongoing research study, it is important to identify how this research has been conducted and to present the results that have been reported. Reviewing current studies that incorporate combined loading is important to establish a baseline and to determine any knowledge gaps. An extensive literature review of shear wall tests conducted by Blandón and Bonett [15] aids in providing a list of the slender and squat walls that have been researched in recent years.

The current research is part of a larger experimental program. It continues from the work of Abdulridha [1], Ziadi [2], Morcos [3], and Soto [4]. Each study included a control, steel reinforced wall, and a superelastic SMA reinforced wall. Abdulridha and Morcos built original walls, whereas Ziadi and Soto repaired, respectively, the walls. Abdulridha's novelty was the inclusion of superelastic SMA bars in the boundary regions of the wall. Morcos' novelty included a specialized coupling mechanism for the SMA bars to address issues in the coupling system noted in the study from Abdulridha. The original coupling mechanism imposed pre-damage to the SMA bars through the screw locks. In testing of the wall, one of the SMA bars fractured at the location of the coupler. The study by Morcos used a forging technique with screw couplers that eliminated damage to the SMA bars.

Both Abdulridha and Morcos used quasi static reverse cyclic loading to test their walls. The loading protocol remained the same for both of their sets of walls; however, the mode of failures were slightly different for the control walls. The walls tested by Abdulridha included added starter bars that extended from the foundation to 300mm into the wall over the web section. This resulted in a larger crack to form during testing just above the starter bars. The walls tested by Morcos did not include starter bars and the dominant crack surfaced at the base of the walls. Aside from this, both control walls experienced horizontal flexural cracking in the boundary

regions and inclined cracking in the web region. The walls experienced crushing of the concrete in the toe regions. The vertical reinforcement in the boundary regions experienced buckling and ruptured during testing. Buckling and rupturing of reinforcement occurred in the toe regions of the wall, and this can be attributed to loss of confinement and fatigue of the reinforcement. Similar behavioural characteristics are expected to arise when axial loading is applied; however, enhanced crack recovery is also expected. The control walls at the end of testing are shown in Figure 2.1.

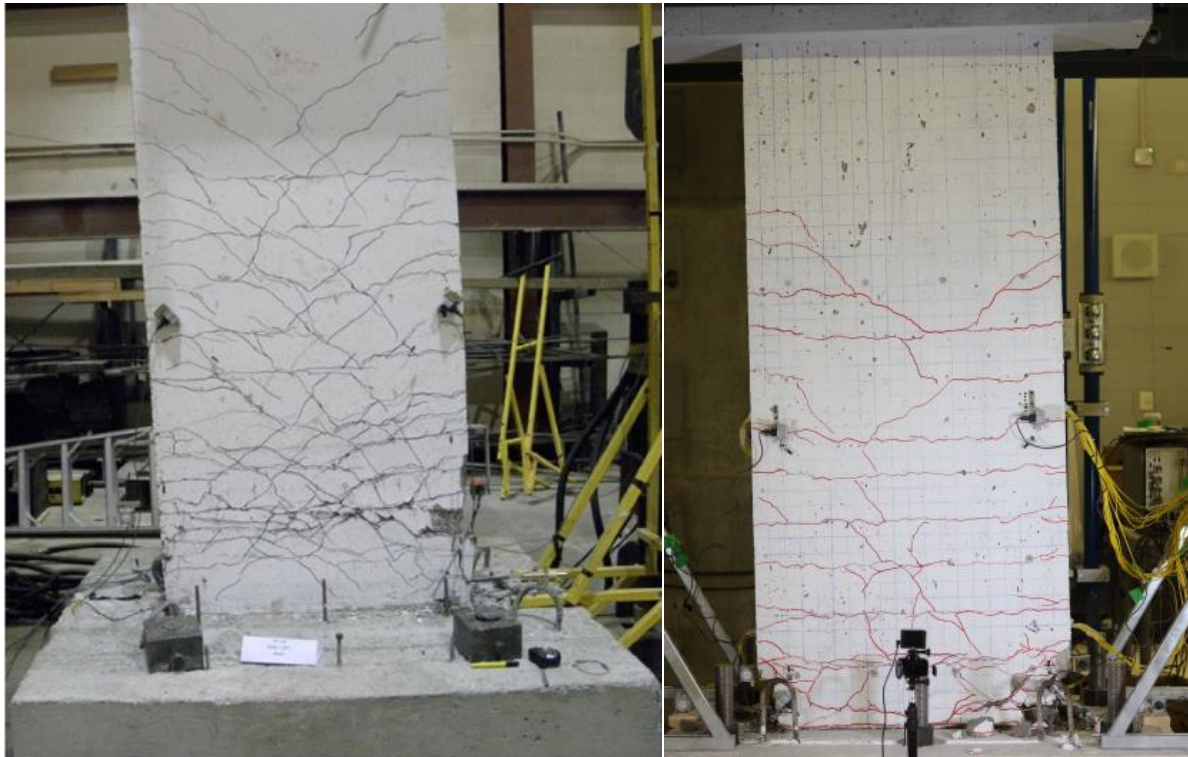


Figure 2.1: Reinforced Concrete Walls used by Abdulridha [1] (left) and Morcos [3] (right)

Similar research that predates the above was conducted in Iran by Tasnimi [16]. This research involved four identical, 1/8 scale shear walls with aspect ratios of 3. The shear walls were all tested using lateral load reversals and no applied axial loading. However, a difference to the research discussed above was the absence repetitions of cycles in the study by Tasnimi. The only difference between this set of walls was the rate of loading. The repetition of cycles allows for an investigation of cyclic strength degradation and increases crack development at each distinct displacement level. In this study, the cracking observed was horizontal in the boundary regions and diagonal in the web region of each of the walls. Research by Sosa et al. [17] on a

slender, barbell-shaped wall with an aspect ratio of 2.8, was also performed with no repetitions of cycles. Though the wall was barbell shaped, it experienced the same form of cracking: horizontal in the wider boundary region and diagonal through the web. Sosa et al. also used the results of this shear wall test to verify a numerical model developed with SeismoStruct.

2.5 RC Shear Wall Research with Axial Loading

In recent years an increased amount of research has focused on the effects of reinforced concrete shear walls when subjected to lateral and axial loading. As demonstrated in the above section, when axial load is not present, there are specific failure modes and patterns which would likely change with the presence of compressive axial loading. It is important to understand how and why this additional loading parameter affects a shear wall. The presence of axial load creates a more realistic loading case that shear wall in a building within a seismic region would experience.

Pushover and cyclic tests were conducted in research by Rama Rao et al. [18] in 2014 on three identical shear walls with aspect ratios of 1.92. These walls were laterally loaded with an actuator and had an 8.5-ton mass applied to the top of the wall to impose a 160 kN axial load as shown in Figure 2.2. One of the three walls was subjected to a monotonic pushover load, while the two other walls were cyclically tested. The walls subjected to the monotonic pushover experienced a higher lateral peak load and a larger displacement than the two cyclically tested walls. This is due to progressive damage and ductility of the walls. During cyclic loading, the walls were stressed and strained in two directions which can increase fatigue, damage, and stiffness degradation due to the loading repetitions. In pushover tests, the wall was loaded in one direction and, thus, not many secondary cracks formed and fatigue did not play a role. The presence of axial loading in this testing did cause a slight pinching to arise in the lateral load-displacement response. Buckling was also evident in reinforcing bars located in boundary elements as a result of the axial loading.

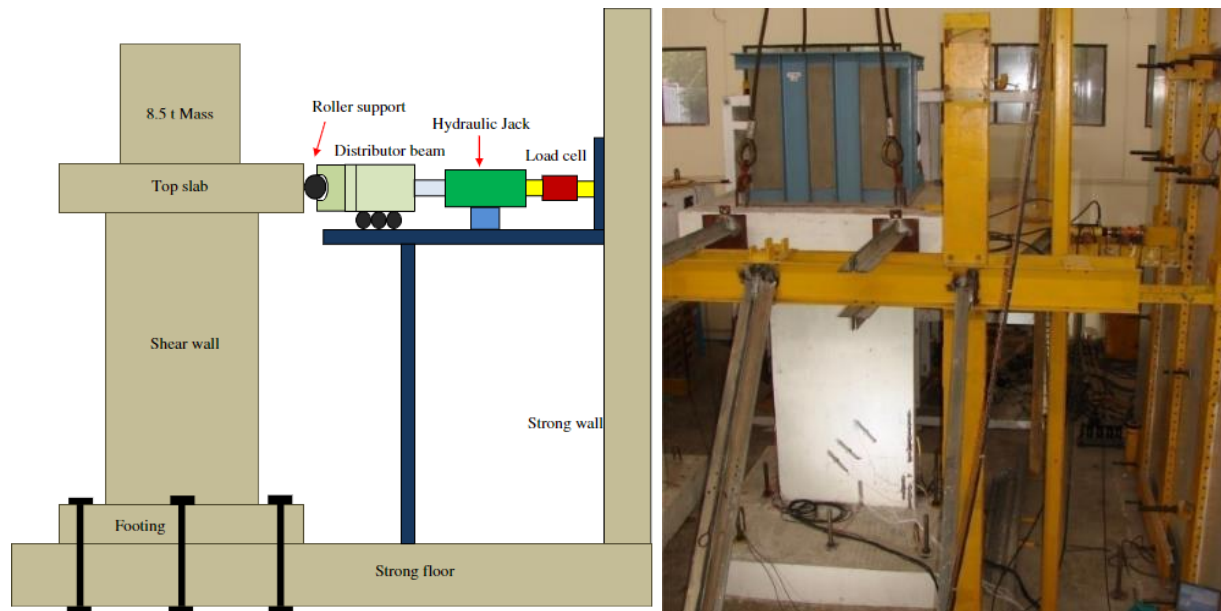


Figure 2.2: Test Set-up used by Rama Rao et al. [18]

Research by Tran and Wallace [19] on cyclic testing of five moderate aspect ratio (1.5 – 2.0), one half scale, RC structural walls was conducted in a similar method to the one presented in this thesis; however, post-tensioned rods were used to apply axial load rather than an hydraulic actuator. Four walls were subjected to 10% of the axial load capacity, and one was only subjected to 2.5%. An elevation view of the wall with reinforcement details is shown in Figure 2.3. The results of this research demonstrated horizontal cracks in the boundary regions, diagonal cracks in the web region, and crushing in the toe regions of all the walls (see Figure 2.4). Buckling and rupturing of the boundary principal reinforcement was also observed (see Figure 2.4). Not all the walls failed in the same manner; diagonal tension failure, sliding, and out-of-plane wall buckling were some of the observed failures. Consistent, however, was the boundary zones yielding at approximately 0.5% to 0.7% drift for all walls. The same can be noted for the loss of lateral load capacity at approximately 3.0% drift in all walls.

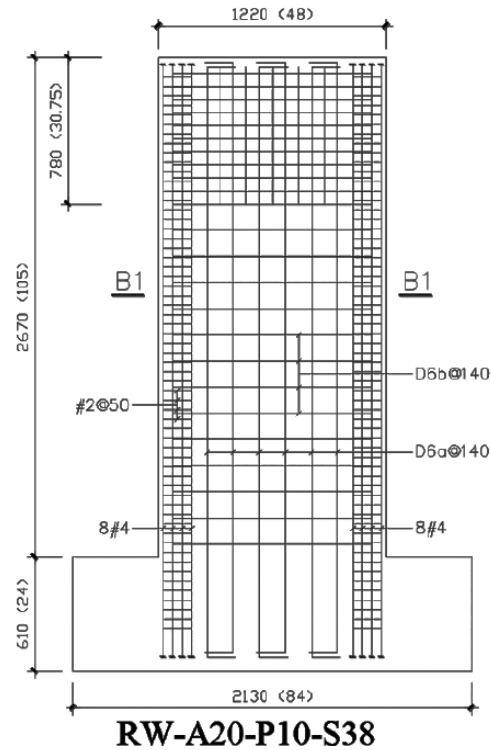


Figure 2.3: Reinforcement Layout of Wall RW-A20-P10-S38 from Tran and Wallace [19] (Units in mm)



Figure 2.4: Failure of Wall RW-A20-P10-S38 and boundary regions [19]

Research following the 2010/2011 earthquakes in Chile and New Zealand sparked serious interest in the effect of compressive forces on shear walls. This arose due to the observed damage in shear walls in the form of crushing of concrete and buckling of longitudinal

reinforcement. The research by Shegay et al. [20] consisted of four half-scale walls with axial load varying from 10%, 14%, and 20% and aspect ratios around 1.6. The changes among the walls included an increasing amount of transverse reinforcement. In all walls, loss of lateral load carrying capacity was deemed the primary failure model, and axial failure was considered a secondary failure mode as it occurred after loss of lateral load carrying capacity. Increased axial load resulted in increased concrete damage up the height and along the length of the wall. Behaviour of all walls was dominated by flexure. Stirrups had fractured during testing, and cross ties opened. Although these walls are considered squat walls, the failure modes and patterns are similar to slender walls with applied axial load.

Jiang, Wang, and Lu [21] studied seven walls with three different wall configurations (I, T, and rectangular sections), all with the same aspect ratio of 2.0. Axial load ratios varied from 0.1 – 0.3 of the axial capacity of the wall. Some differences between the walls also included different transverse reinforcement ratios. All walls were designed based on the Chinese seismic design code. The seven walls were tested under lateral cyclic loading and applied axial loading. This study observed that the damage sequence for all the walls were similar including cracked concrete, yielding of the longitudinal reinforcement, spalling of the concrete, reinforcement buckling, and finally reinforcement fracture. Again, horizontal cracks were observed in the boundary regions and diagonal cracks were observed in the web regions. Yielding occurred between 7.8mm – 12.5mm of lateral displacement; approximately 0.4% and 0.625% drift. No stirrups were fractured during testing. Increasing the axial load ratio, increased the damage progression and ultimately the shear effectiveness increased.

Wall shape can play a significant role in the response of a shear wall to seismic loading. Research by El-Aizy, Shedid, El-Dakhakhni, and Drysdale [22] studied this parameter on six half-scale shear walls with different wall shapes and aspect ratios of 2. The cross sectional shapes included rectangular I-shaped, and barbell shaped walls. The set had two walls with different reinforcement details. All walls were subjected to lateral and axial loading. Flanged walls demonstrated an enhanced seismic performance compared to the rectangular walls. Delayed concrete crushing at the toe of the wall and reduced crack localization were observed in the flanged walls. The larger confined area in flanged walls increased the compressive force

resistance. Increasing vertical reinforcement in the boundary region aided in initiating secondary cracks instead of larger localized cracking. All walls experienced crushing and spalling in the toe regions of the walls as well as boundary bar buckling prior to rupture.

Three identical walls were tested by Alarcon, Hube, and De La Llera [23], with details from 5 buildings damaged during the 2010 M8.8 earthquake in Chile. High axial loads resulted in brittle concrete crushing failure after cover spalling. This research aimed to reproduce brittle failure of walls by using a high ALR (Axial Load Ratio). The walls were rectangular in shape as they represented 32% of the walls in the five seriously damaged buildings. The walls had an aspect ratio of 2.3, had unconfined boundary regions and longitudinal reinforcement surrounded by 90-degree bends in the boundaries. The ALR ratios were 15%, 25%, and 35% respectively. The first cracking occurred at 0.5% drift and first yielding occurred at 0.6% drift. The reinforcing bars also yielded in compression which is likely due to the high compressive stresses. During testing, the walls with higher axial load experienced reinforcement yielding and failure at smaller lateral displacements. Out-of-plane buckling occurred in all three walls due to opening of the horizontal reinforcement; this was a result of the combination of low confinement and high axial load as shown in Figure 2.5 and Figure 2.6. Unlike the buildings that were being replicated, the vertical bars did not fracture, which was likely due to the test being terminated when significant stiffness loss was observed.

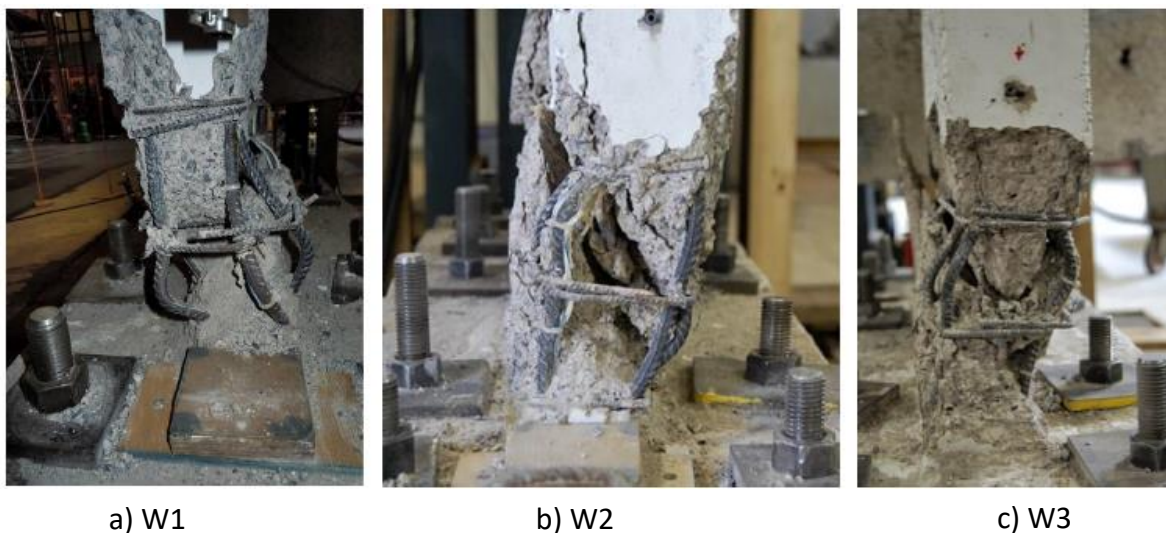


Figure 2.5: Failure due to crushing and buckling of: (a) W1; (b) W2; and (c) W3 [23]

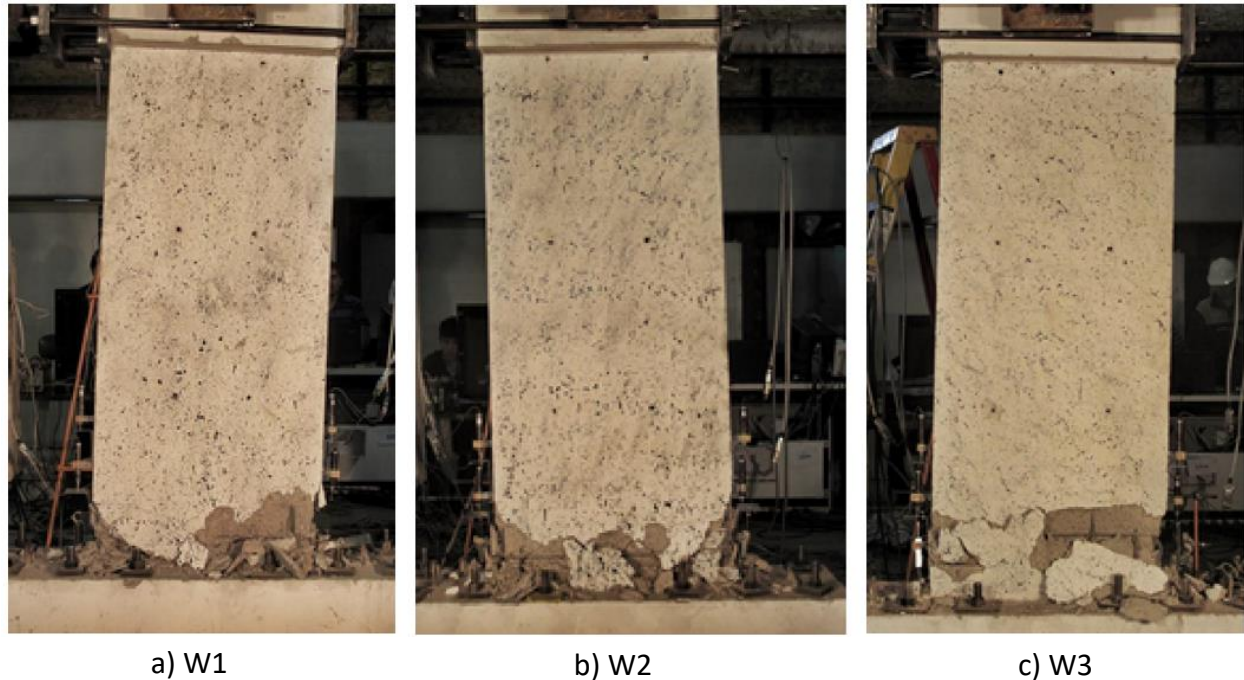


Figure 2.6: Damage at the Base of (a) W1, (b) W2, and (c) W3 [23]

Hube, Marihen, De La Llera, and Stojadinovic [24] studied six half-scale RC walls and subjected them to quasi static cyclic loading with constant axial loading of 15%. The test set-up used for these experiments is shown in Figure 2.7. Wall W1 from Alarcon, Hube, and De La Llera [23] was referenced as the control wall for these six walls. The walls tested were labeled W4-W9; W4 had a wall thickness of 75mm, whereas the rest of the walls were 100mm thick. W5 was only 1180mm in height, whereas the rest of them were 1600mm in height. Aside from W4 and W5, the only variations in the walls were the reinforcement detailing. Wall W8 has similarities to the wall tested in the research presented herein, refer to Figure 2.8. The results illustrated that in each wall the reinforcement buckled after compressive failure of the wall. The height-to-thickness ratio of the wall is important since walls which are thin and tall are more susceptible to lateral instabilities; this paper recommended the use of a more conservative ratio than is currently used in the Chilean code. In comparing Wall W8 to Wall W1, the addition of closed boundary stirrups in Wall 8 aided in increasing the ultimate displacement of the wall and reducing the probability of bar buckling. This also aided in confining the concrete cores in the boundary regions. As referenced in this paper by Hube et al., Kuang and Ho [25] also noted that closed stirrups in boundary regions provide enhanced seismic performance through improved ductility

and ability to dissipate energy. The use of cross ties in Wall W9, shown in Figure 2.9, was effective in improving energy dissipation, however, it did not restrain the buckling of the boundary bars as well as the full stirrups used in Wall W8. Possible reasons for the buckling in Wall W9 could be similar to the results from by Shegay et al. [20] where the cross ties in their walls opened during testing.



Figure 2.7: Test Set Up used by Hube et al. [24]

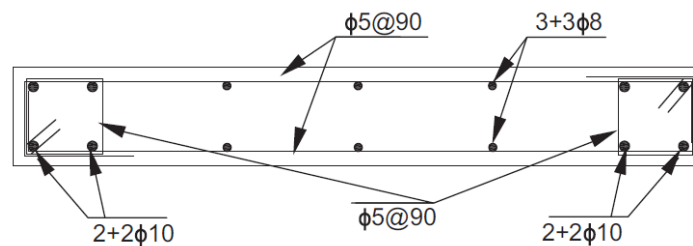


Figure 2.8: Wall W8 Cross Section [24]

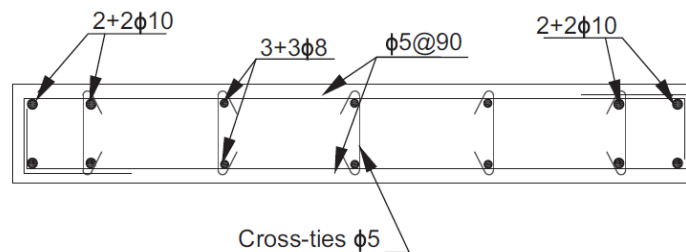


Figure 2.9: Wall W9 Cross Section [24]

Su and Wong [26] investigated three tall slender shear walls with aspect ratio of 4, where the effects of high axial load ratios was the focus. The first two walls consisted of the same amount of reinforcement, however, the third wall contained twice the amount of transverse reinforcement. The axial load ratios were 25% for Wall W1 and 50% for Walls W2 and W3. The loading for this test was not applied laterally, instead moments were applied, see Figure 2.10. The walls with 50% ALR yielded and failed at smaller displacements than the wall with 25% ALR. Essentially, Wall W1 exhibited ductile behaviour/failure, whereas W2 and W3 exhibited brittle behaviour/failure. An important detail observed was the flexural cracking that appeared in the wall with 25% ALR, however, cracks were not observed until the final load stage for the walls with 50% ALR. As stated in the paper, a limited degree of damage was observed until the final stage, which ultimately demonstrates the crack closing properties of high compressive loads. All walls failed due to out-of-plane buckling. Walls W2 and W3 also experienced a higher rate of strength loss compared to W1. Comparing Walls W2 and W3, W3 exhibited better flexural strength but failed at approximately the same displacement as Wall W2. Considering the transverse reinforcement was doubled, the results illustrate that the benefits were negligible. Another finding in this study is that the effectiveness of confinement is based on orientation and arrangement. As noted in some of the previous papers, establishing a boundary region can significantly enhance the confinement and strength of a shear wall.

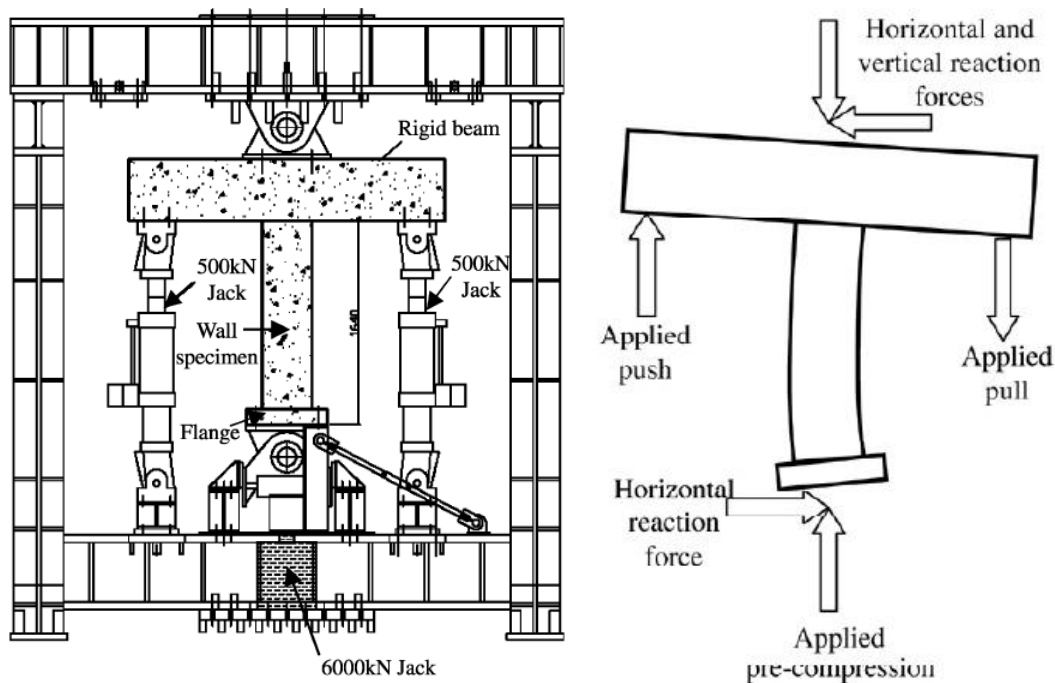


Figure 2.10: Testing Orientation used by Su and Wong [26]

At ETH Zurich, Dazio, Beyer, and Bachmann [27] tested six RC walls under quasi-static cyclic loading. The walls were modeled at half scale with aspect ratio of 2.0 and had different reinforcement configurations. Two cycles were used for each displacement level and each displacement was incremented in terms of the yield displacement. The loading protocol was slow to minimize dynamic effects. Failure in these tests was defined by a 20% drop in lateral load resistance. With the exception of the first wall that was tested, the other five walls exhibited concrete cracking before spalling and bar buckling before rupture. One of the walls tested had poor confinement and bar buckling occurred at the same instant as concrete spalling. The crack patterns in all walls included horizontal cracks in the boundary regions and inclined cracks in the web region (see Figure 2.11). Crushing and spalling was most prevalent in the toe regions of the walls; the areas of that experienced the highest compressive stresses during the loading cycles. Although it is not a direct focus of this research, the ductility of the reinforcement used in the first wall was considerably low and, this in turn, resulted in overall low wall ductility.

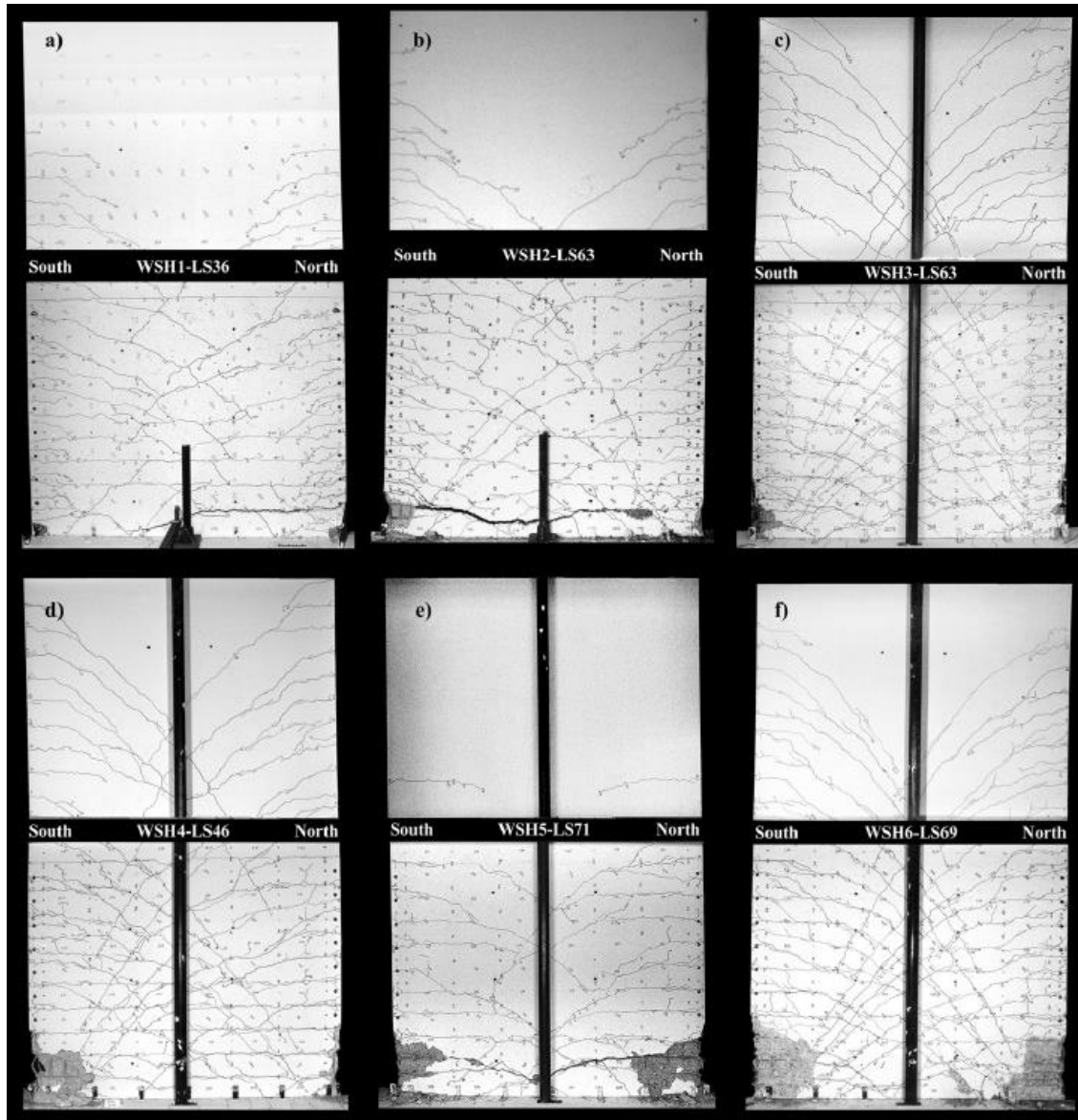


Figure 2.11: Crack Patterns of Walls after failure [27]

Aghniaey, Saatcioglu, and Aoude [28] investigated the effects of high strength steel (Grade 690 MPa) in shear walls with aspect ratios of 2.0 and 3.0. Both walls were constructed with a barbell cross section with heavily reinforced boundary regions (see Figure 2.12). The purpose for this research was to investigate the seismic response of shear walls with high strength steel. As noted in the paper, the Canadian CSA standards places restrictions on the use of high strength steel due to the reduction in ductility and the uncertainty of its yield point. In comparing the high strength wall to the wall with Grade 400 MPa steel, the high strength wall had a bar rupture at 1.8% drift, whereas the Grade 400 MPa wall experienced rupture at

approximately 2.5% drift. This illustrates that the high strength steel results in a less ductile response. This research highlighted that steel with poor ductile properties results in brittle failure. Much like concrete, the stiffer the steel, the more brittle it behaves. Thus, for shear walls, increasing stiffness will not result in increased ductility and may hinder the seismic performance of the wall. The damage of the wall with high strength steel is illustrated in Figure 2.12 and Figure 2.13.

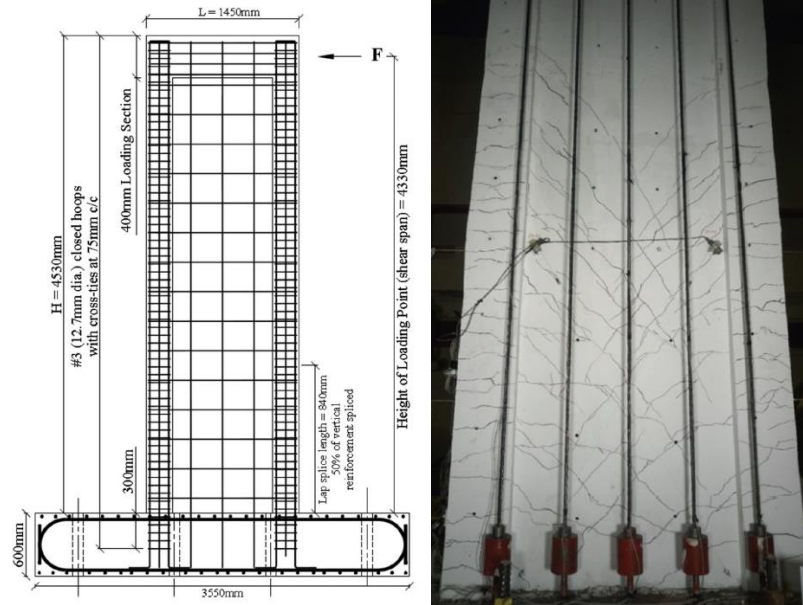


Figure 2.12: Wall Dimensions (left) and Crack Pattern of wall after 0.8% Drift (right) [28]



Figure 2.13: Boundary Regions of Figure 2.12 after completion of testing [28]

From the above literature review, it is apparent that when shear walls are subjected to combined lateral and axial loading, they generally react in a flexure dominant pattern with some

shear failures arising. This is also dependent on the aspect ratio and axial load ratio of the wall. In terms of specific damage, concrete spalls off in the boundary regions above the toes of the walls which leads to a reduction of confinement. Concrete will crush at the toes of the walls and due to the compressive loading, the boundary longitudinal steel buckles. Buckling occurs after the onset of yielding to the steel on the compression side of the wall, specifically when the steel is in a strain hardening phase; it also is affected by the reduction of confinement due to cover spalling. Repeated cycles will cause the steel to fatigue and eventual rupture under tension stress. When the steel in the boundary regions ruptures, the overall wall stiffness and strength significantly reduce. As additional reinforcement ruptures, the overall stiffness and strength continually reduces. The inclusion of axial load does enhance crack closing ability. Understanding the behaviour of RC shear walls can lead to methodologies to increase the ductility and recovery from seismic forces. Adding emerging materials into the plastic hinge region of shear walls can lead to enhanced ductility of shear walls and potentially increase the transient and residual drift capacities without a significant loss in strength and stiffness.

2.6 Shear Walls with Emerging Materials

This section discusses research on novel and emerging materials with a view to determine if it is possible to implement such materials into construction practices. Previous research by Abdulridha [1], Ziadi [2], Morcos [3], and Soto [4] involved shear walls with superelastic shape memory alloy. Specifically, they used a nickel titanium alloy (Nitinol) with excellent recovery properties. Unlike shape memory SMA, superelastic SMA can return to its original shape without heat. The observed results from Abdulridha and Morcos indicated that the presence of SMA provided improved recentering properties; this was illustrated through the inelastic displacement recovery. Abdulridha noted that the control steel-reinforced wall exhibited approximately 24% crack width recovery, whereas the SMA-reinforced wall experienced 88% crack recovery. A potential issue with SMA bars is their smooth surface, which resulted in wider cracks forming along the length of the bar in the boundary regions.

2.6.1 ECC and SMA

Another emerging material used in the above noted experimental research was Engineered Cementitious Composites (ECC). This material was implemented by Soto in the repair

methodology of the original walls tested by Morcos. ECC is constituted from small aggregates (sand), cement, super plasticizer, and Polyvinyl Alcohol (PVA) fibres. More details of this material are available elsewhere [4]. ECC is a type of concrete with superior tensile and flexural capacity in comparison to normal strength concrete. ECC relies on fibers to bridge concrete cracking, which improves the tensile strength and ductility of the material. Therefore, this material will experience micro cracking that is controlled by the presence of the fibers. This property allows ECC to experience more deformation than normal concrete without significantly large cracks. However, this property is not always achieved as a large crack can form and reduce this bridging effect. In terms of shear walls, this material is expected to increase ductility and strength of the wall given that ECC is a more ductile concrete and can retain some strength at the cracks due to fiber bridging.

Zheng, Liang, and Deng [29] from Xi'an University of Architecture and Technology in China report on 6 rectangular shear walls with aspect ratio of 2.1. All six walls were subjected to combined lateral and axial loading. This aspect ratio of these walls is considered intermediate and they are influenced by flexural and shear behaviour. These walls were constructed with HPFRCC (High-Performance Fiber-Reinforced Cement Composites) in the plastic hinge or inelastic deformation critical region. Based on the mix design, the HPFRCC contained PVA fibres and can ultimately be classified as a form of ECC. The ratio of fly ash to cement for this mix proportions was 1:1. All walls were tested with varying axial loads ranging from 880 – 1320 kN, and the heights of the HPFRCC varied from 400 – 800mm. In some of the walls, different length steel sleeves were added around the boundary region steel to prevent buckling, however, due to their smooth surface, the HPFRCC did not bond to them. All the walls in this experiment experienced similar crack patterns, with horizontal cracks in the boundary regions and diagonal cracks in the web regions. The overall results from this research illustrated that the addition of HPFRCC increased the ductile performance of the walls when compared to RC walls with normal concrete.

Almedia, Steinmetz, Rigot, and Cock [30] investigated shape memory alloy bars for their ability to recover residual displacements and damage in shear walls. Two shear walls at 2:3 scale were tested. The walls were 2000mm tall, 200mm thick, and 1200mm long. The comparison of the two walls was based on the performance of the control normal concrete RC wall to a RC wall

with SMA bars in the boundary regions. A constant axial load of 7.3% was applied consistently throughout both wall tests. In comparing the cyclic responses of the two walls, the steel-reinforced RC wall exhibited a stiffer initial response as the peak load during testing was 10% higher than the wall reinforced with SMA. Noted during the tests, the RC wall failed prematurely and without the expected ductile response desired. Aside from the premature failure, the comparison does demonstrate that the walls which included the SMA bars provided superior energy dissipation and ductility in comparison to the walls with only deformed bars. The residual displacement recovery observed was much better from the SMA wall as expected. An important feature of the SMA bars to note is that the ability to provide recovery is only activated after the material reaches its yield point. Thus, it would only be activated when the concrete cracks. This material would only be implemented in high seismicity zones where concrete cracking and reinforcement yielding are expected. In areas of low seismicity, the cost of SMA and the lack of need for it would outweigh its usefulness. This paper provides a benchmark that should be used in future shear wall tests with combined loading. Hoult and Almedia [31] followed up on these tests with further analysis of the SMA wall in more detail. This specific research provided new experimental results on the crack distribution, longitudinal strains, and plastic hinge lengths.

Similar research was performed by Kian and Cruz-Noguez [32] on four shear walls (one control wall) with aspect ratio of 2. The materials used in their study were SMA, Fiber Reinforced Cementitious Composites (FRCC) and Glass Fiber Reinforced Polymer (GFRP) bars. The FRCC used were ECC and SFRC. These walls were tested with lateral reverse cyclic loading and without applied axial loading. Similar findings of improved self-centering and residual drift recovery were reported for the ECC walls and parallels the research above.

Hoult and Almeida [33] reviewed a variety of different shear wall tests on rectangular walls. For conventional RC walls, they gathered and plotted 76 different results with varying axial loads, wall widths, and reinforcement ratios to determine the average drift which corresponds to an allowable residual drift of 0.5%. Essentially, on average 1.5% is the maximum drift that corresponds to a residual drift of 0.5%. According to allowable limits, after a seismic event, if the building returns to no more than 0.5% drift, then it is deemed operational. Other findings from were walls with larger amounts of reinforcement had lower inelastic tensile strains compared to

lightly reinforced walls. As noted, axial load also provided a recentering effect. This paper referred to several other studies on shear wall tests which incorporated SMA. This study reconfirmed that the inclusion of SMA aids in returning walls that have experienced more than 1.5% drift back to a residual drift within the 0.5% limit. This article provided an extensive review of finite element modeling of SMA reinforced walls, which is useful for the experimental program presented in this thesis. In a similar approach, Abraik and Youssef [34] investigated steel- and SMA-reinforced shear wall and the varying level of fragility that each type of material experiences during testing.

2.7 Other Research

This section reviews research that is more related towards shear walls in construction and improving their ductility, stiffness, and lateral load resistance. With the addition of cross braces or steel sections in the boundary regions, the performance of shear walls can be enhanced. Although implementing some of these concepts in construction may not always be possible or feasible, it is an area of shear wall research that should be reviewed to better understand how to improve the seismic performance.

Wang and Guo [35] from Hunan University in China investigated $\frac{1}{2}$ scale shear walls subjected to different axial load ratios. The walls incorporated X-shaped steel plate bracing (SRCXSW), see Figure 2.14, and one wall had a cast-in-place section at the base of the wall which was implemented with an enhanced damping concrete in the plastic hinge, see Figure 2.15. The intent was to improve the connection performance of the assembly joints. The walls had an aspect ratio of 1.625. Higher ALR lead to concrete spalling and vertical crack propagation at smaller displacements. Higher ALR also lead to larger axial deformations and compressive stress before lateral loading. Smaller inclined cracks of shorter length were observed with larger ALR. Ultimately, larger ALR resulted in better shear characteristics. The X-braces were effective at overcoming the concrete crushing, and thus improved the load-bearing performance.

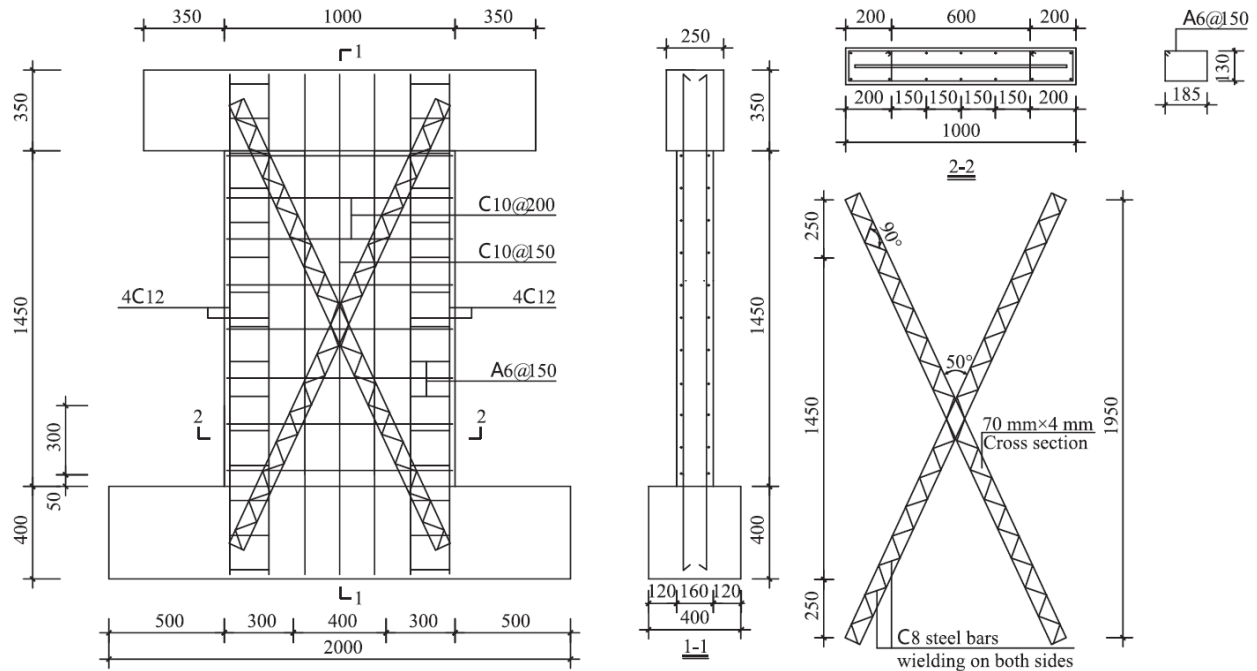


Figure 2.14: Wall used by Wang and Guo [35] (Units in mm)

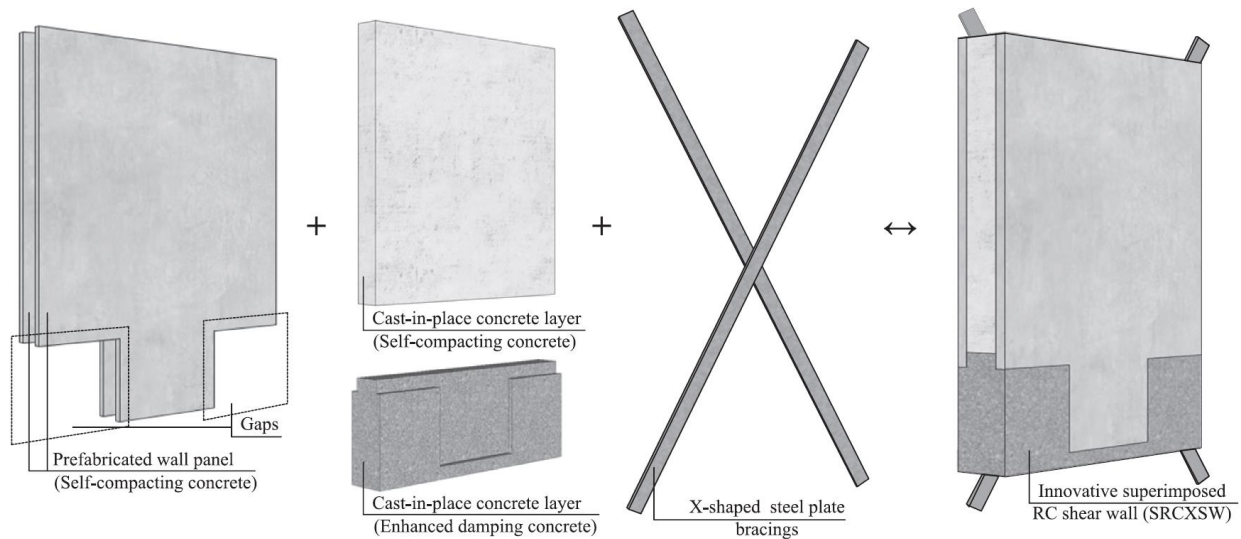


Figure 2.15: Casting Process used by Wang and Guo [35]

Massone, Sayre, and Wallace [36] studied three slender reinforced concrete walls with the inclusion of structural steel in the boundary regions. The aspect ratio for these walls was 4.0 as they were intended to represent 4 stories of a building. Axial loading on the first two walls (SRCW1 and SRCW2) was 10% and was increased to 18% for the third wall (SRCW3), which was a replica of the first wall. The transverse reinforcement spacing in SRCW2 was doubled in

comparison to SRCW1. These walls contained W6 steel beams in their boundary regions; walls SRCW1 and SRCW2 had W6x16's and wall two had W6x9's. Each W section was welded to a plate that was anchored into the base foundation of the walls. Holes were drilled into the W sections to allow for transverse reinforcement to be passed through. Consistent with other research, SRCW1 experienced an earlier onset of buckling compared to SRCW2 due to larger spacing in the transverse reinforcement. Slip of the steel sections was observed, and vertical cracks or splitting occurred at the locations of the webs of the steel sections. The slip of the steel sections was observed in all three walls and this reduced their overall stiffness.

3 Experimental Program

3.1 Introduction

An experimental program was conducted to capture the behaviour of a large-scale reinforced concrete shear wall subjected to combined lateral and axial loading. This study is the continuation of previous research Abdulridha [1], Zaidi [2], Morcos [3], and Soto [4]. This chapter contains the design of the shear wall, the materials and their properties, the construction process, the test setup, and the testing program used for the experimental testing and evaluation. The wall was tested using quasi-static reverse cyclic loading and continuous axial loading.

3.2 Design of Shear Wall

The shear wall built in this study, named SFW (Shear Wall 5th Edition), has been modified from previous studies by Abdulridha [1] and Morcos [3]. The wall was designed according to the CSA A23.3-04 standard [37] and verified to be compliant with the CSA A23.3-14 standard [38]. This wall was designed previously as a ductile slender shear wall, which remained the same for this experimental program. The wall was geometrically the same as the previous studies, having a width was 150mm, length of 1000mm, and the height of 2200mm, as shown in Figure 3.1. This resulted in the same aspect ratio of 2.2. The foundation also remained the same at 1600mm long, 1000mm wide, and 500mm deep with six anchorage points. There were three anchorage locations on either side of the wall with a center to center spacing of 600mm.

The lifting points were also modified to be closer to the edges of the foundation to avoid crane chains contacting the cap beam. Due to the modified test set up, with the inclusion of axial loading, the cap beam dimensions were changed to account for the larger actuators. The original dimensions used by Morcos [3] were 1600mm long by 400mm wide by 400mm deep. The new dimensions were 1200mm long by 520mm wide by 590mm deep (see Figure 3.1). These changes were incorporated to ensure the actuators would be at half of the 500mm stroke when in contact with the wall at the start of testing. The increased widths also allowed for maximum contact area with the larger actuator heads.

The reinforcement within the cap beam was modified to accommodate these dimensions. The clear concrete cover remained at 50mm. Instead of eight pairs of U-Shaped stirrups along

the length of the cap beam, only six pairs of U-Shaped stirrups were used in the current wall. Along the length of the beam, instead of ten longitudinal bars, twelve longitudinal bars were used. These changes were necessary to facilitate proper reinforcing details for the cap beam and to allow for adequate reinforcement in the cap beam to maintain its stiffness during testing.

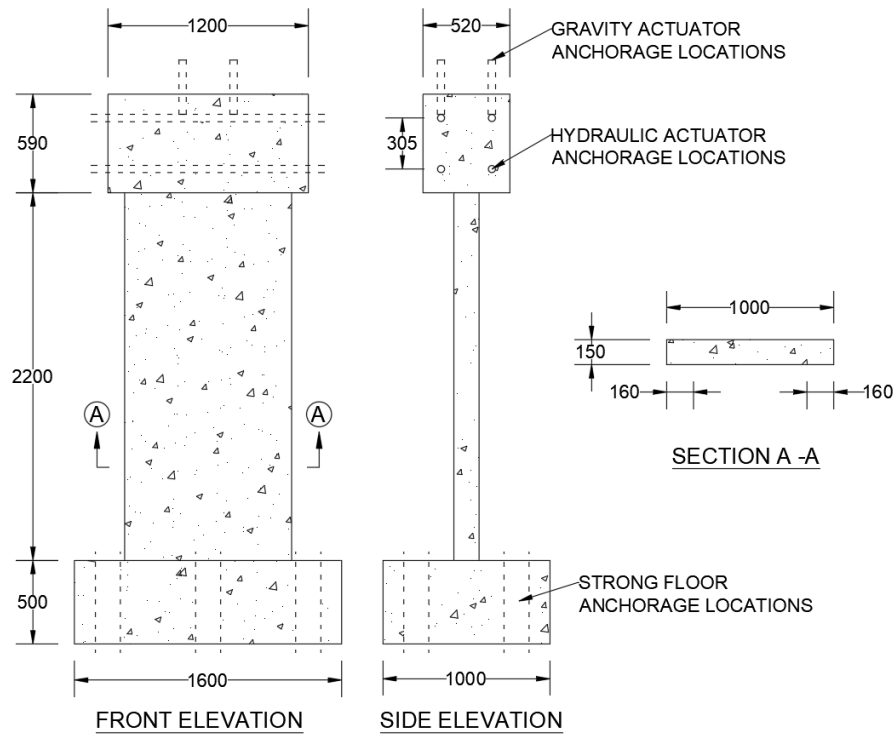


Figure 3.1: Shear Wall Dimensions and Cross Section (all dimensions in millimeters)

3.2.1 Reinforcement Details

The wall was reinforced with conventional deformed steel. In the foundation of the wall, a top and bottom cage of U-shaped 15M bars were used. A total of six PVC pipes were cast into the foundation for the anchorage bolts. The cap beam reinforcement consisted of 10M bars, twelve in the longitudinal direction, secured to two U-shaped bars creating a stirrup. There were six total stirrups or twelve U-shaped bars in the cap beam. Four PVC pipes were cast into the cap beam and along the length to allow for a location for threaded rods to pass through and secure the horizontal actuator. The clear cover on all sides for both the foundation and cap beam was 50mm (see Figure 3.2).

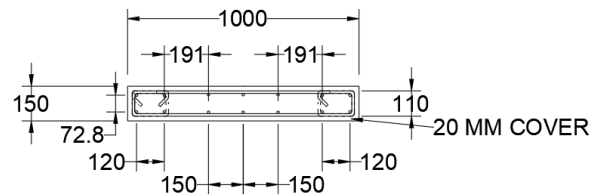
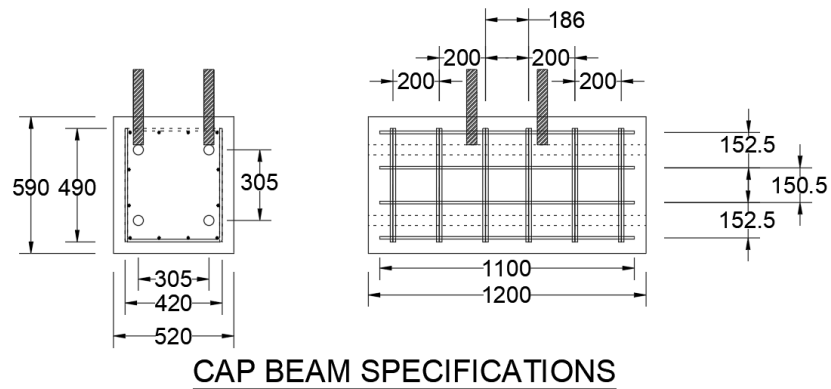
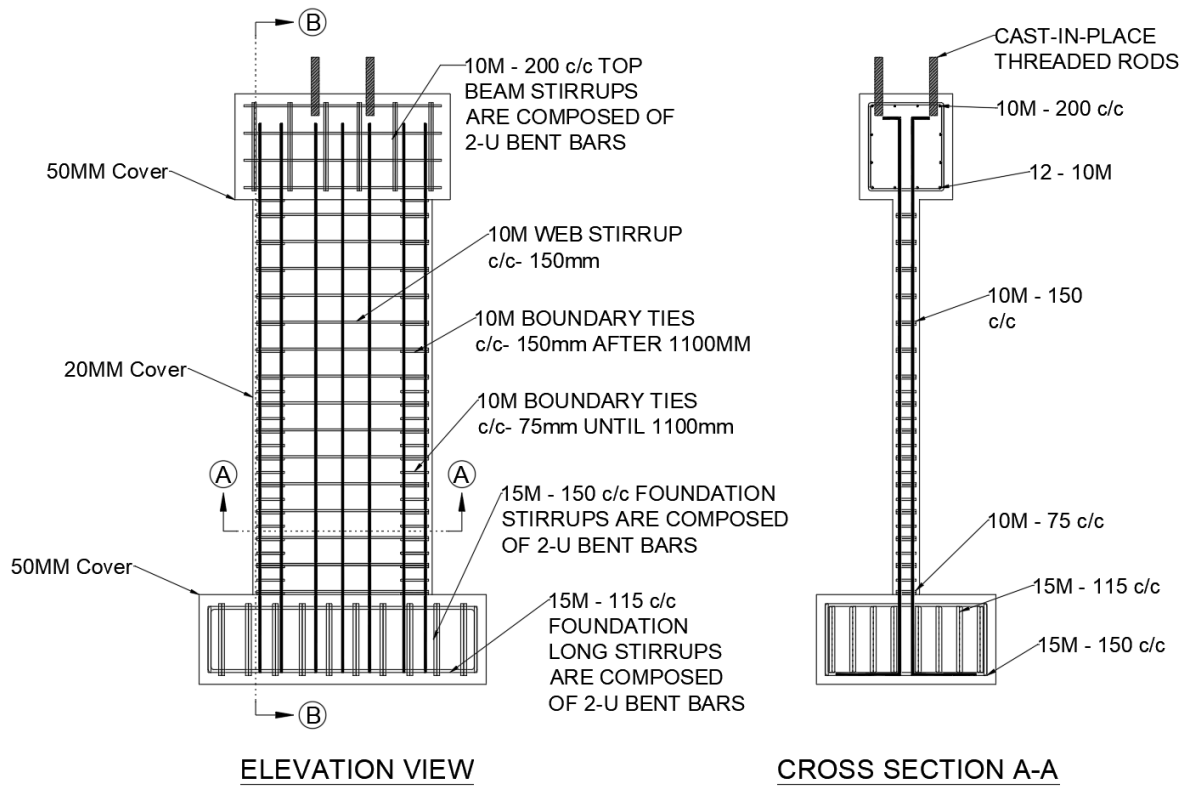


Figure 3.2: Reinforcement Details for Shear Wall SFW

In the wall, the concrete cover was 20mm on all sides. The wall was reinforced with 10M deformed bars. The wall was separated into three sections, including two boundary regions and one web region. In the longitudinal direction fourteen 10M hooked bars (90 degrees) were present, six in the web of the wall, and four in each boundary region of the wall. This resulted in seven longitudinal bars on the front face of the wall and seven longitudinal bars on the back face, with two in each boundary and three in the web. In the boundary regions, closed ties were placed at 75mm center to center to a height of 1100mm from the base of the wall to ensure adequate anti-buckling ties within the plastic hinge region. Above 1100mm in height, the spacing increased to 150mm center to center. Large stirrups were used to encase the longitudinal reinforcement in the boundary regions and the web at a spacing of 150mm center to center throughout the entire wall. Using Clause 21.5.2.1.2 of CSA A23.3-14, the plastic hinge length was estimated to have a height of 720mm (Equation 3.1). The reinforcement details for the plastic hinge were extended to approximately 1100mm to ensure there were adequate ties placed in the boundary regions in the event the plastic hinge was higher than 720mm. In addition, the intent was also to have similar reinforcement details as the walls previously tested by Abdulridha [1] and Morcos [3].

Equation 3.1

$$(0.5 * l_w) + (0.1 * h_w)$$

$$(0.5 * l_w) + (0.1 * h_w) = 0.5 * 1000 + 0.1 * 2200 = 720mm$$

Note that there are no lap splices at the base of the wall which is common among buildings within low seismic regions in Canada. However, in higher seismic zones, lap slices are normally not permitted while the plastic hinge region of shear walls and the reinforcement would extend the entire length from the foundation to the first floor.

3.3 Material Properties

The following sub-sections discuss the properties of the materials used in the construction of the shear wall. Deformed reinforcing steel and normal concrete were the main materials used to construct the wall. In addition, this section will also identify the type of testing used to quantify the properties of the materials.

3.3.1 Reinforcing Steel

The wall was reinforced with deformed steel bars and they were provided by Halton Rebar Inc. The steel was classified as black rebar Grade 400W and it came in two size specifications, 10M and 15M. The steel was subject to uniaxial tensile testing according to the ASTM A370 standard [39]. Three coupons of similar length for both diameters of steel were tested to determine the mechanical properties. The steel was not subject to cyclic tension testing.

The uniaxial tension tests were performed on three coupons of each steel size used. For the 10M bars, a length of approximately 475mm was used, and for the 15M bars, a length of 600mm was used. A 50mm extensometer was placed at the length of the bar to capture the change in length and strain during rupture (see Figure 3.3).

The clamped lengths of the 10M coupons were approximately 315mm and the clamped lengths of the 15M coupons were about 440mm. The speed of the testing was dependent on the gauge length. From the start of testing to yielding, a speed of 1.5% of the gauge length per minute was used, and after reaching the yield plateau, the speed was increased to 5% of the gauge length per minute until rupture occurred. This is shown in Table 3.1.

Table 3.1: Uniaxial Coupon Test Specifications

Bar ID	Length (mm)	Clamped Length (mm)	Speed (mm/min)	
			1.5% Length	5% Length
10.1	477	320	4.8	16
10.2	477	315	4.73	15.75
10.3	478	315	4.73	15.75
15.1	604	440	6.6	22
15.2	600	436	6.54	21.8
15.3	602	440	6.6	22



Figure 3.3: Uniaxial Tension Test of Reinforcement

3.3.2 Concrete

The concrete was provided by CeMix. Unlike other companies, CeMix mixes batches onsite using an automated system which allows for multiple batches to be created by one truck. The wall was cast in two stages, first the foundation was poured and then the wall and cap beam were poured in tandem. To ensure steps were taken for an adequate concrete mix, the first batch from the mixing truck was poured into a wheelbarrow and was then tested for slump. In both instances, the slumps from the first batches were inadequate and the concrete in the wheelbarrows were discarded. Prior to pouring both the foundation and cap beam/wall combination, the second batches were again poured into a wheelbarrow and had their slumps measured. The adequate mixes were then poured into the wall components. This ensured a proper slump and flowability were achieved as generally the first mixes appeared to have too many coarse aggregates.

Prior to pouring, a location in the high bay lab was selected and checked to ensure it was level to cast cylinders. The level location was next to the location of the wall and was adequate to facilitate properly leveled cylinders. During the casting, 4 in. x 8 in. cylinders were cast to be tested for compressive strength. Cylinders were cast for the foundation and cap beam/wall mixes. These cylinders were placed in a water bath for 28 days following removal of formwork and were tested at various times throughout the hardening process.

3.4 Wall Construction

The wall was constructed in stages and is described in the following paragraphs.

The construction phase began with the building and designing of formwork. The foundation and wall formwork were repaired and reused from the construction of the walls by Soto [4]. Due to the change in design of the test set up, the cap beam dimensions were changed and, therefore, the previous formwork was not used. The addition of extra support to the existing wall formwork was used to ensure adequate coverage under the cap beam and to support the formwork.

The construction began with setting up the formwork for the foundation and marking the locations where PVC pipe needed to be placed. This PVC pipe in the foundation of the wall provided the 6 locations where the anchor rods would secure the wall to the strong floor. Blocks of wood, the same diameter as the PVC pipe, were cut and fastened into the bottom formwork of the foundation. This was followed by assembling the bottom portion of the foundation rebar cage (see Figure 3.4). Due to the shape of the bars used in the foundation, the top cage had to be placed after the wall cage was erected.



Figure 3.4: Bottom Foundation Reinforcement Cage

Prior to building the reinforcement cage for the wall, some of the ribs on the bars were grinded and sanded to create smooth, flat contact surfaces for the strain gauges. The strain gauges were wired and soldered at the same time as the grinding process. To place the strain gauges, Kapton tape was used to keep the strain gauges secured to the sanded surface of the

reinforcement. A few zip ties were used to secure the lead wires of the gauges to the reinforcement above/ or below the sanded area and to prevent them from moving while the installation of the strain gauges was carried out. To ensure that the strain gauges were adequately attached to the bars, the following process was completed. First, a small piece of black electrical tape was placed down on the sanded area of the reinforcement to ensure that the metal ends of the strain gauge did not ground out on the reinforcing bar. Then the Kapton tape and strain gauge were placed with the exposed wires on the electrical tape as the gauge was on the bare steel. Next, the tape was lifted along with the gauge and a thin layer of super glue was placed on the steel to allow for proper bonding of the strain gauge. This also allowed for the removal of the Kapton tape so that the gauge was directly on the bare steel. Next, a rubberized coating was applied to the gauge to cover it, and a thin layer of Teflon tape was placed over the gauge. A generous layer of epoxy was then placed on top of the entire gauge to ensure maximum protection during the casting process.

After the strain gauges were placed at the pre-determined bar locations, the wall cage was assembled. Prior to building the cage, all the longitudinal bars were marked to identify the locations of the stirrups. In the plastic hinge region, the markings were at every 75mm and above this region they were marked at every 150mm. The large web stirrups were also marked with the locations of the longitudinal bars to ensure they were placed properly. Due to the size and amount of steel, the wall cage was built horizontally to allow the stirrups to be placed with ease, as shown in Figure 3.5.

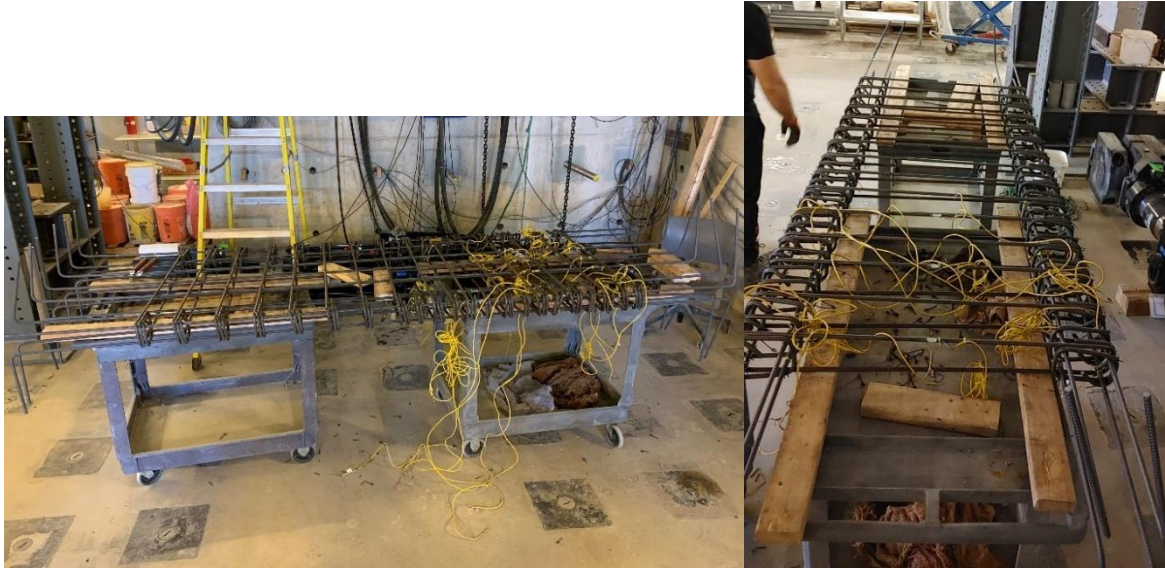


Figure 3.5: Constructing Wall Reinforcement Cage

The wall cage was then placed on top of the bottom layer of the foundation reinforcement, followed by placing the top layer of the foundation cage into the foundation. At the interface of the wall cage and foundation cage, 4 in x 4 in timber was used to secure the base of the wall cage from moving. In addition, several 2 in x 4 in timber were cut to stabilize the wall cage while the foundation cage was tied together. The locations of the PVC pipe were capped with 2 in x 2 in timber that had wooden blocks fastened to them and spaced to match the locations of the perforations in the strong floor. This ensured that the rods used to secure the wall foundation to the floor were aligned. The foundation was then casted, as illustrated in Figure 3.6.

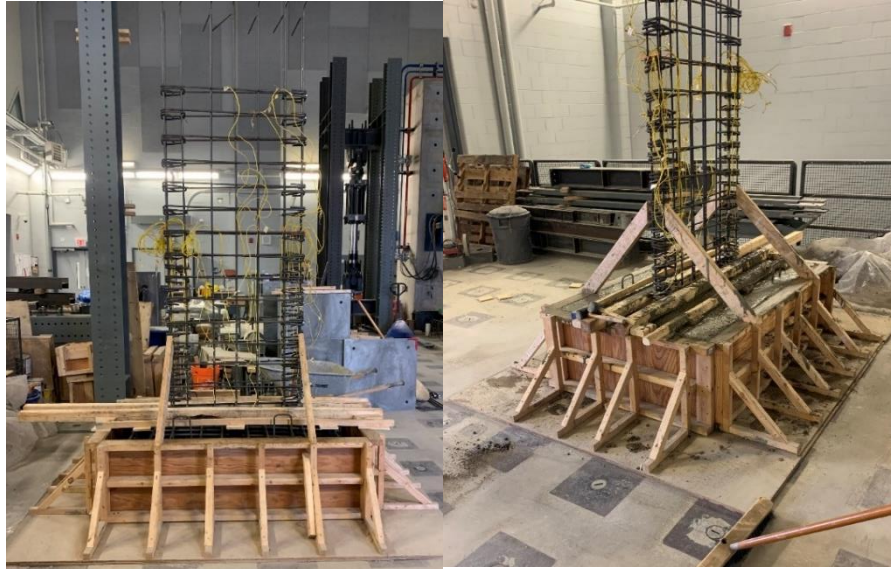


Figure 3.6: Foundation Formwork Bracing (before pouring left, after pouring right)

The foundation was cast first so that the wall formwork could be erected directly on top of it. The wall formwork consisted of two bulkheads placed at the ends of the wall and two panels placed on the front and back sides of the wall. Figure 3.7 provides an elevation view of the wall formwork.

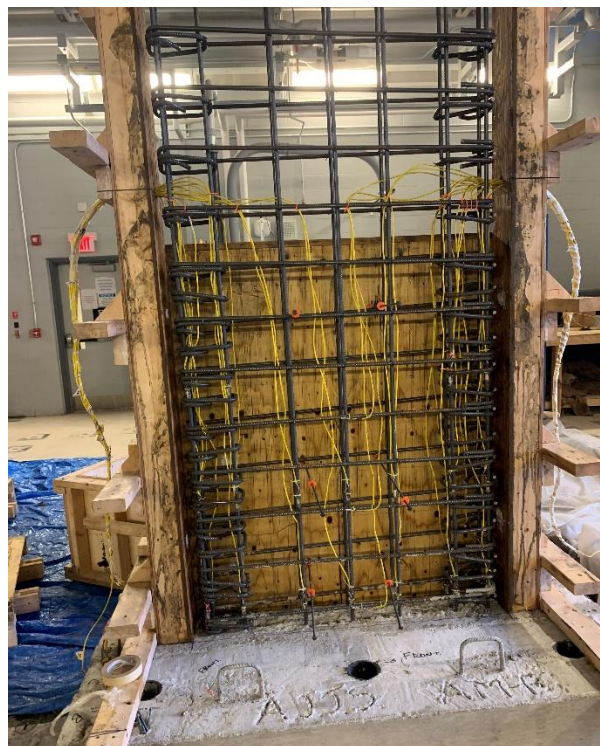


Figure 3.7: Wall Prior to Placing all Formwork

The existing formwork was designed for a smaller cap beam, therefore, additional 2 in x 4 in timber was attached to support the change in width of the new cap beam. The cap beam was redesigned, and new formwork had to be constructed. In the new formwork, four holes were drilled at either end to allow for PVC pipe to be put in place for the threaded rods which would be secured to the horizontal actuator (see Figure 3.8).



Figure 3.8: Cap Beam Formwork and Cage

The wall dimensions were marked at the interface of the wall and foundation to provide a placement guide for the placement of the formwork. This ensured the wall was in the middle of the foundation. Thereafter, 2 in x 4 in timber was cleated to the exact width of the foundation and was placed just behind the bulkhead formwork. This ensured there was no movement of the bulkheads. To minimize movement of the wall panels, 2 in x 4 in timber were fastened to the cleated timber, effectively making a frame which was fixed in place. This frame at the base of the formwork was intended to prevent the base of the wall from opening during casting. The cap beam reinforcement cage was built outside of the formwork and then placed on top of the wall formwork. The cap beam formwork was then mounted, and the PVC pipe was placed through the formwork and secured with tape at either end to restrict movement during casting. An extra

piece of plywood was cut to the width of the cap beam, with holes cut out of it for the cast-in-place threaded rods. After the concrete was poured, this extra piece of plywood was fastened into place and 4 threaded rods were driven into the wet concrete. These rods were used to secure the actuator which would impose the axial load on the wall. Figure 3.9 illustrates the wall before and after casting.



Figure 3.9: Wall and Cap Beam Formwork before (left) and after (right) Casting

The wall and cap beam were poured together, exactly one month after the foundation was poured. A concrete bucket was used to cast these sections. Multiple layers were poured and vibrated to ensure adequate consolidation and distribution within the highly reinforced regions of the wall. After several days, the wall formwork was removed from the wall and cap beam. After casting, there was some honeycombing at the base of the wall, the deepest portions being 11mm into the wall. Self-Consolidating Concrete (SCC) was used to fill the voids. After the SCC had hardened, the wall was covered in wet burlap and wrapped in plastic (see Figure 3.10). Wetting of the burlap was performed every few days. The cylinders were also placed in a water bath.



Figure 3.10: Wetting Process for Wall and Cylinders (bottom left in black bin)

The wetting process took place for 28 days after the formwork was removed and filling of the voids. After 28 days, the burlap was removed from the wall and the cylinders were removed from the water bath. Once the wetting had been completed, any concrete protrusions were chipped and sanded to make an adequate surface to paint. The wall was then coated in a few layers of white acrylic paint. The acrylic paint was watered down to create thin layers to allow for cracks to appear. The smooth wall surface allowed a grid to be imprinted using a blue chalk line on the painted surface of the wall. The spacing for this grid matched the sizing of the finite elements used in the development of the numerical model in FormWorks and analyzed through VecTor2. The majority of the grid was 50mm x 50mm; however, in the boundary sections of the wall, the grid width was either 40mm or 60mm due to the longitudinal bar spacing.

3.5 Test Setup

Three major components constituted the test set up for this shear wall: the instrumentation consisting of different potentiometers, the test frame equipped with two hydraulic actuators; and the data acquisition units.

3.5.1 Instrumentation

The internal instrumentation for the wall consisted of 33 strain gauges, and the external instrumentation was made up of a total of 15 cable potentiometers and two linear potentiometers. There were five of each type of cable potentiometer, CP-300, CP-600, and CP-1250, and two LP-100. The majority of the CPs were mounted to a rigid steel frame that was fixed to the foundation block. This ensured that the readings were relative to the wall and were not influenced by rigid body movements of the wall, such as base sliding or base overturning.

3.5.1.1 Cable and Linear Potentiometers

The CPs were numbered from 1 to 15, with each location having a specific purpose. The locations of the CPS are shown in Figure 3.11. CP 1 was placed at the midpoint of the cap beam to capture the lateral displacement of the wall due to the lateral load from the actuator. CP1 was used as the control measurement in determining when a loading cycle was reached before unloading and loading in the other direction. CP 2 was placed just below the cap beam and underneath the actuator to capture the lateral displacement of the top of the wall. CP 3 was placed at the midpoint of the wall to record the lateral displacement at that location. CP 4 was placed at the base of the wall to capture sliding of the wall relative to the foundation. Together, these CPs can be used to develop a lateral displacement profile along the height of the wall from the foundation level to the midpoint of the cap beam and applied load.

CP 5 and CP 6 were used to capture the elongation or slip of the wall in the boundary regions, just above the base of the wall. CP 7 and CP 8 were used to measure the vertical displacement from the bottom of the cap beam and the top of the foundation. CP 9 through CP 12 were used to create an approximate 800mm by 800mm square in the plastic hinge region. Two CPs measured the vertical displacements and the other two the diagonal displacements. This was used to calculate the shear strain in the plastic hinge region, a method used in previous theses. Due to the placement of the stirrups, the vertical spacing was 850mm.

CP 13 was mounted to the exterior columns of the test frame to just below the top of the foundation to record sliding of the wall relative to the strong floor. CP 14 was attached to the exterior columns of the test frame, in line with the top set of threaded rods, to measure the total displacement, including the effects of rigid body motions of the entire wall. CP 15 was attached on the back face of the wall at the same height from the foundation as CP 3. This was used to gauge out-of-plane displacements of the wall during testing. LP 1 and LP 2 were used to measure uplift of the foundation from the strong floor at either end of the foundation.

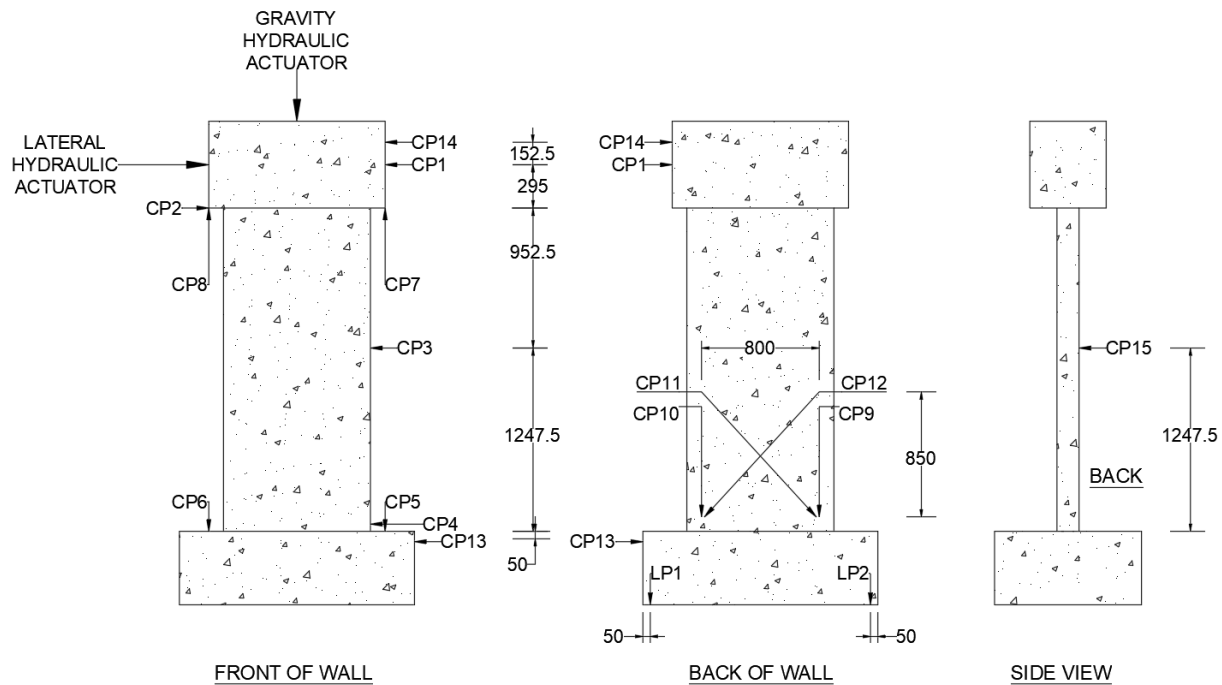


Figure 3.11: Potentiometer Locations

3.5.1.2 Strain Gauges

The internal instrumentation consisted of 33 strain gauges all located within the plastic hinge region, as shown in Figure 3.12. The bulk of the strain gauges were placed on the longitudinal reinforcement in the boundary regions of the wall; three gauges per bar, for a total of 24. They were placed at 50mm above the base, 400mm above the base, and 750mm above the base. Only the back face of the wall had strain gauges on the reinforcement within the web region, and they were located at 50mm above the base (SG7 – SG9) and 400mm above the base (SG14 – SG16) for a total of six gauges (see Figure 3.12). The final three strain gauges were located

on the long stirrups in the middle of the wall. The first, third, and fifth stirrups from the base of the wall had strain gauges.

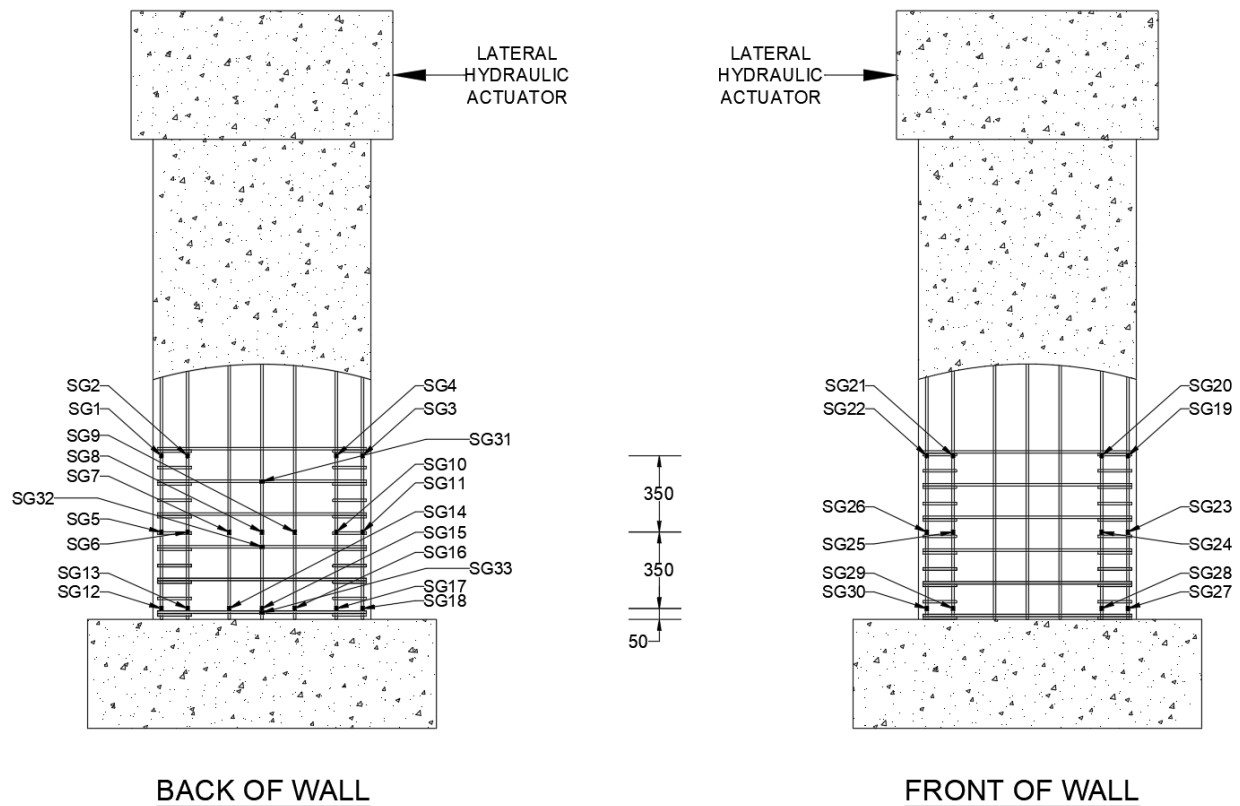


Figure 3.12: Strain Gauge Map

3.5.2 Test Frame

The testing frame that was designed and built for this experimental testing was developed from the frame used by Morcos [3] and Soto [4]. The main portion of the frame that was previously designed had four, six meter tall W310x107 steel columns equipped with a bottom plate to allow them to be bolted to the strong floor. The holes in the strong floor were also spaced at 600mm center to center. From the center line of the foundation of the wall, the columns were located at the corners of the foundation. In the short direction, the columns were 1200mm apart, center to center. In the long direction, the columns were 3000mm apart, center to center. These columns were located around the foundation of the wall and connected together by two short direction beams and two long direction beams. In the short direction of the wall, the short beam was placed attached at 3350mm from the laboratory floor, which is above the cap beam to not

interfere with the threaded rods. Prior to attaching the short beams, a loading beam was fastened to the strong wall to which the actuator was connected (see Figure 3.13).

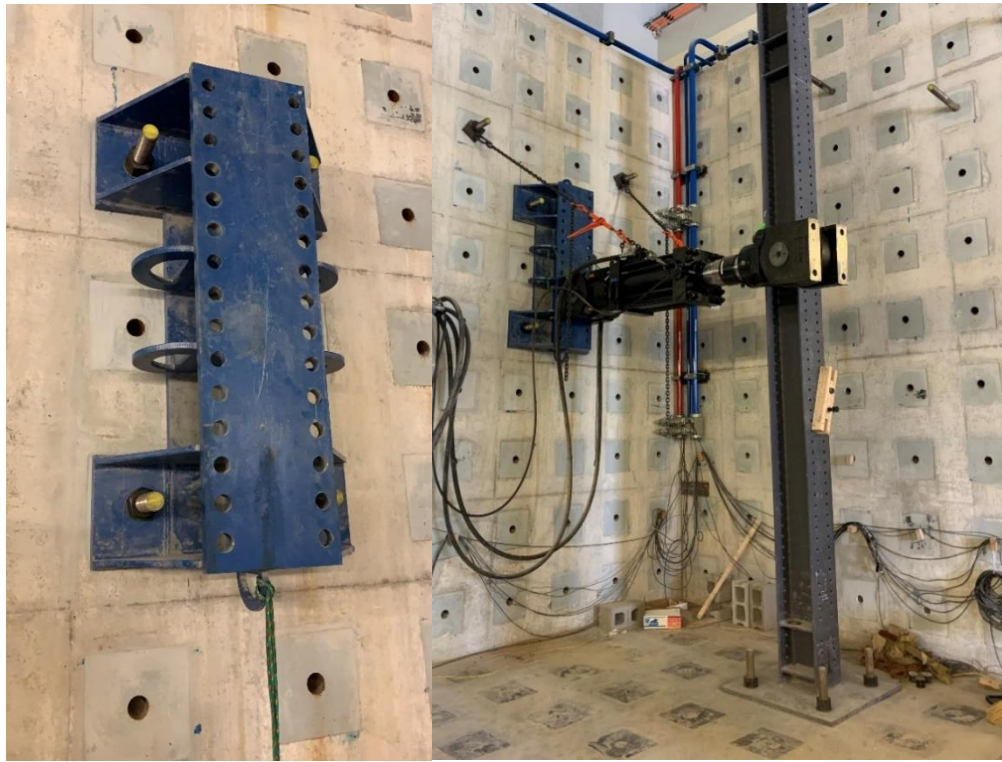


Figure 3.13: Loading Beam (blue) and Lateral Actuator (right)

The actuator that was used to apply the lateral load was then secured to the loading beam on the strong wall. Two lifting hooks were attached above the loading beam to provide locations to connect chains and turnbuckles. These chains were then connected to lifting points on the actuator to restrain it from moving prior to testing (refer to Figure 3.13). The height of the head of the actuator was later adjusted using the turnbuckles too. After this was completed, on the side with the actuator, the short beam was attached 150mm higher to provide clearance above the actuator. In the long direction, two long beams were located at 3000mm above the laboratory floor, which coincided with the center of the cap beam. Modified C150x19 C-channels with casters were fastened to the long beams to act as rollers to resist out-of-plane displacements of the cap beam (see Figure 3.14).



Figure 3.14: Side View of Wall to Observe Casters

Column extensions were bolted to the existing corner columns to erect the vertical actuator that was used to impose the axial loading (see Figure 3.15 and Figure 3.16).

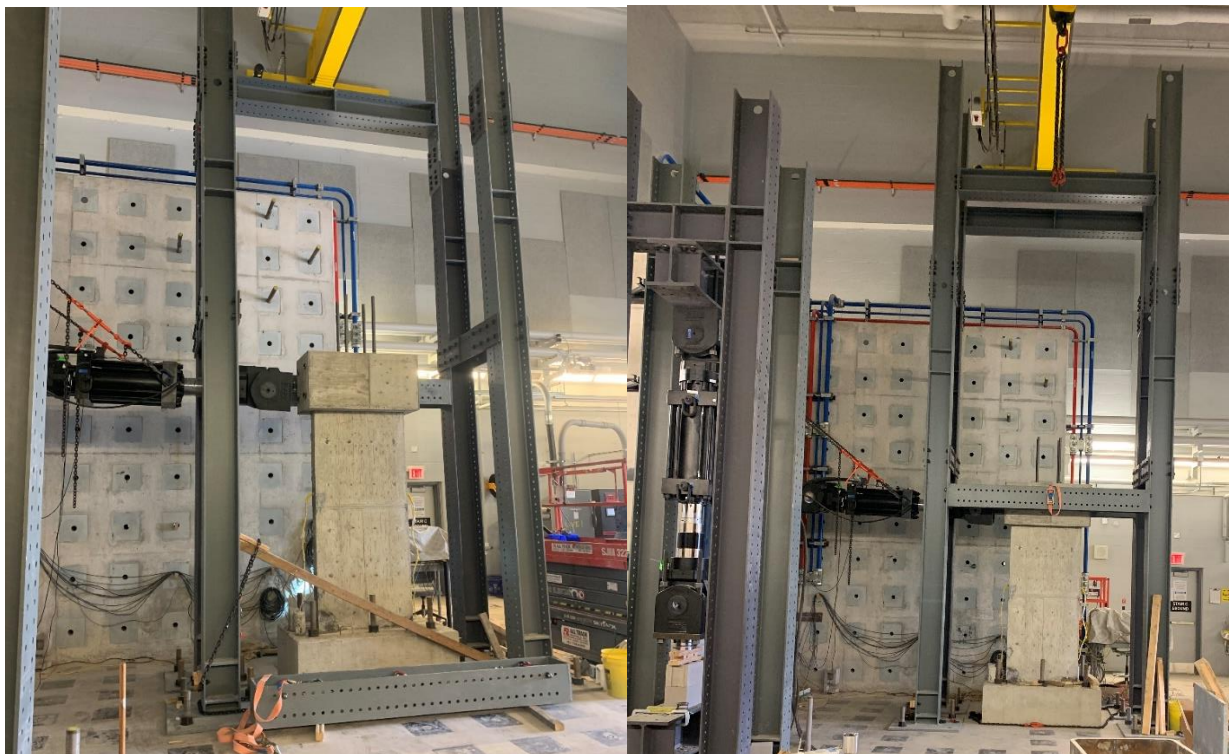


Figure 3.15: Attaching Long Direction Beams (left, back beams) (right, all long beams done)

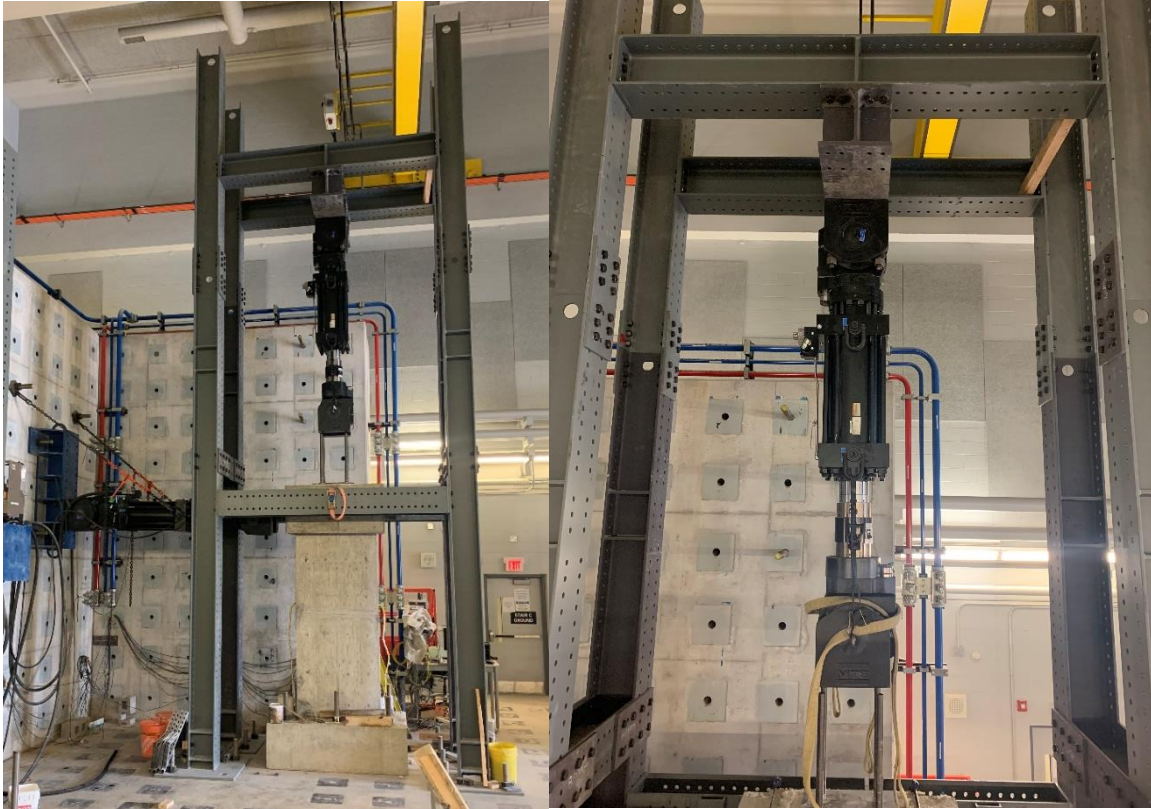


Figure 3.16: Test Setup Prior to Lowering Actuator (left, entire set up) (right, axial actuator)

After the two-meter long column extensions were attached to the top of each column, the remainder of the test set up was completed. Long beams were bolted first at a height of approximately 7200mm above the floor level; these beams were the same size as the lower beams but were not equipped with rollers. After securing the two long-direction beams, the vertical actuator was attached to a short W360x216 beam and secured underneath the two long-direction beams (refer to the image on the right of Figure 3.16). Once the W360x216 cross member was attached to the middle of the long beams, the three beams and actuator were lowered 250mm. This ensured that the actuator could be connected without contacting the threaded rods protruding out of the cap beam. In addition, this ensured that the actuator would be at half stroke or extended out 250mm of 500mm stroke capacity prior to testing (see Figure 3.17). The short direction cross beams were then placed above the long direction cross beams at approximately 7200mm above the floor. Further information and figures on the test set up can be found in Appendix H.



Figure 3.17: Actuator at Lowered Location

3.5.2.1 Actuators

Two Series 201.70 Hydraulic Actuators were used to complete the test set up, one horizontal to apply the lateral load, and one vertical to apply the axial load. These actuators have maximum tensile capacities of 965kN and compression capacities of 1460kN. In the following section, the anchorage system for both actuators will be discussed, and here how they were secured to the wall will be presented. Both actuators applied the loading through steel bearing plates (10mm thick, 520mm x 520mm) for adequate load dispersion. The vertical actuator was anchored to four cast-in-place threaded rods and bolted to ensure there was no uplift during testing and to maintain a constant axial load. The lateral actuator head was bearing against a steel plate, and on the side opposite of the actuator another steel plate was used to disperse the load from the nuts and washers attached to the threaded rods during pull cycles. A layer of

mortar was placed under the plate to ensure that there is a level surface for the plate to rest on and to ensure the actuator applied the load on a level and surface.

Both actuators were very tight due to factory torque specifications and had to be loosened such that they could rotate according to the rotation of the wall. This was completed using an Allen key and a spanner wrench. The super nut of the actuator was loosened which allowed the spherical washers to be loosened. The super nut was then retightened. This ensured that there would be no backlash in the actuator head but would allow it to rotate freely during testing

3.5.3 Data Acquisition

The data acquisition for the shear wall was collected using four data acquisition units (DAQs) and two different computer software (refer to Figure 3.18 for the data acquisition flowchart). Computer program Catman controlled and collected all the data from the DAQs. The strain gauges, cable potentiometers, linear potentiometers, and the actuators connected to the DAQs.

MTS Multipurpose Elite was the software used to control and run the actuators. Catman also collected the displacement and load of the actuators, however the actual test program was designed and executed using Multipurpose Elite. The load cell of the actuators was connected to one of the DAQs to measure their force output as well as the displacement readings.

Due to the amount of instrumentation attached to the DAQs and only one computer to collect all the data, the strain gauges and linear potentiometers were set to a refresh frequency of 1 Hz to ensure the computer did not malfunction during testing.

The instrumentation included 33 strain gauges, 15 cable potentiometers, and 2 linear potentiometers. The two actuators occupied 4 ports in the DAQs. In total, out of 56 available ports between the four DAQs, 54 were used.

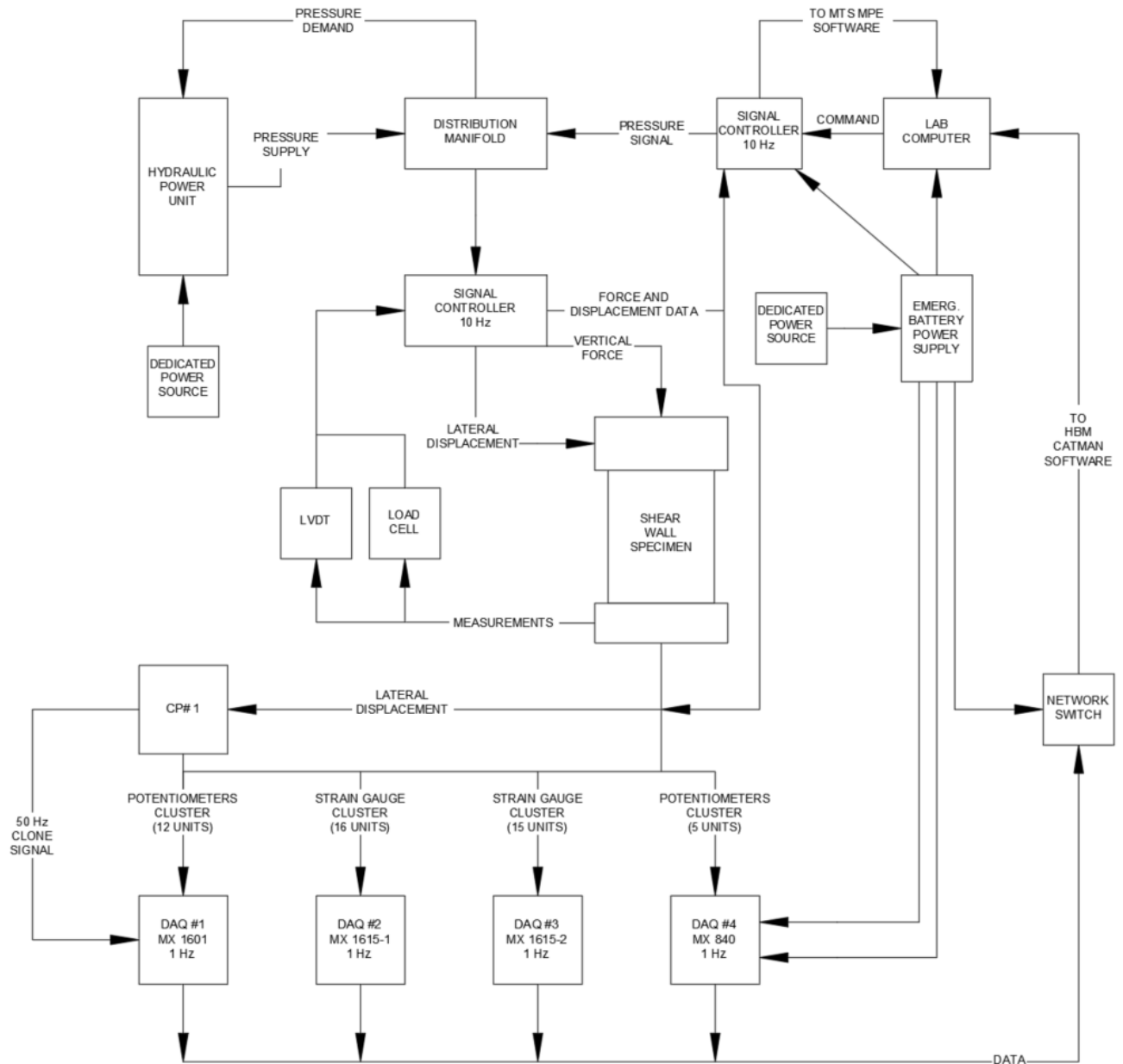


Figure 3.18: Data Acquisition Flowchart

3.6 Test Program

The purpose of this experimental program was to construct and test a slender reinforced concrete shear wall to serve as a control wall for a larger experimental program. This experiment is also a continuation of the previous shear wall research, with the novelty of adding an applied axial load in addition to reverse cyclic lateral loading. The reverse cyclic quasi-static displacements simulated seismic loading, and the applied axial loading simulated the gravity load of slabs on shear walls in reinforced concrete structures.

3.6.1 Loading Protocol

The loading protocol used for this testing was altered slightly from previous work by Morcos [3] and Soto [4]. Due to changes in the dimensions for the cap beam of the shear wall, the displacements were approximately 1.04% larger (Table 3.2). The displacement against number of cycles is shown in Figure 3.19.

To keep as much congruency as possible, the drifts and number of repetitions used in the original loading protocol were kept the same. The original loading protocol was a displacement-controlled protocol modified from ATC-24 [40]. ATC-24 pertains to steel structures but was modified for these tests due to the relevance of the protocol. For this test, the loading protocol was force-controlled rather than displacement-controlled. The displacement of the cap beam was monitored using CP1, in doing so, when the desired displacement was achieved, the load was paused and reversed. From 0.05% drift to 1.0% drift, the displacements were repeated 3 times in the positive and negative directions. The loading rate and unloading duration were relative to the desired displacement. The purpose for changes in loading rate was to ensure quasi-static lateral loading.

Table 3.2: Testing Displacement Sequence

No.	Drift (%)	Displacement (mm)	Cycles	Loading Rate (mm/min)	Unloading Duration (min)
1	0.05	1.25	3	1.25	0.4
2	0.1	2.50	3	2.5	0.8
3	0.2	5.00	3	3	1.6
4	0.3	7.50	3	3	2.4
5	0.4	10.00	3 (yield @ 0.37%)	3	3.2
6	0.5	12.50	3	3	4
7	1.0	25.00	3	6	4
8	1.5	37.50	2	6	6
9	2.0	50.00	2	6	8
10	2.5	62.50	2	15	4
11	3.0	75.00	2	15	4.8
12	3.5	87.50	2	15	5.6
13	4.0	100.00	2	15	6.4
14	4.5	112.50	2	15	7.2
15	5.0	125.00	2	15	8

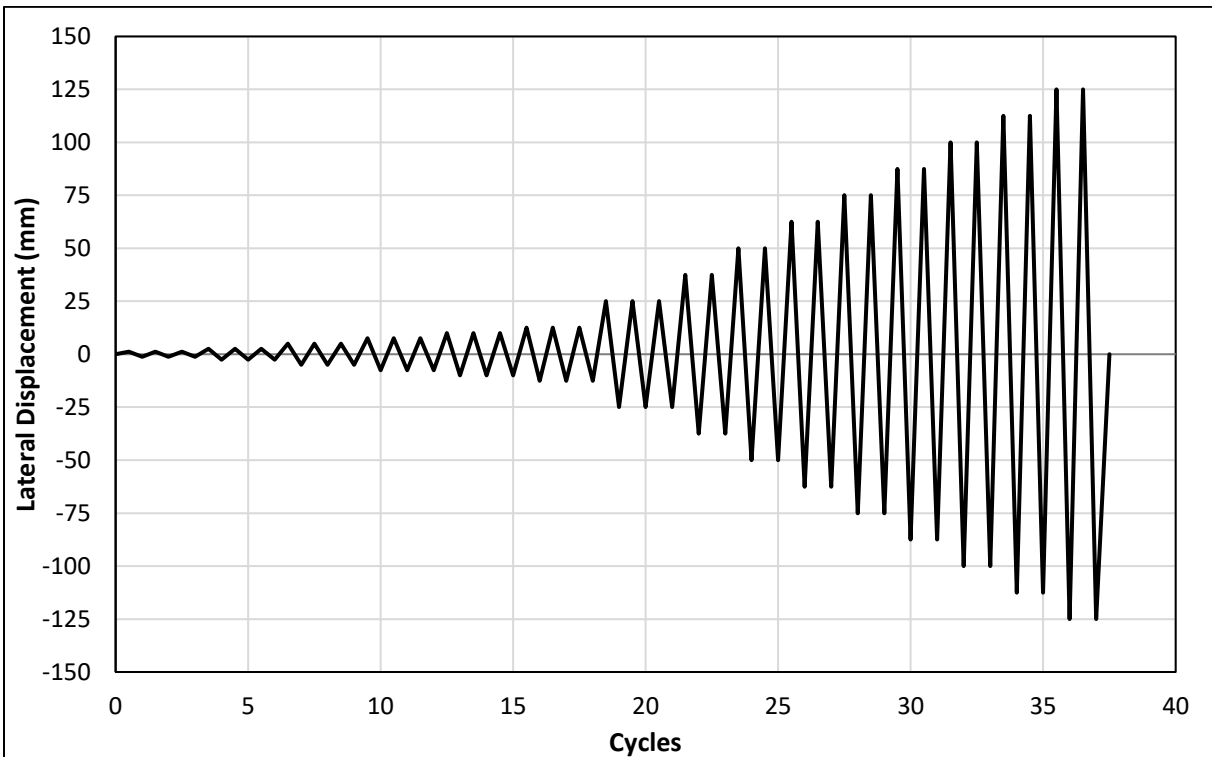


Figure 3.19: Reverse Cyclic Displacement Protocol (ATC-24 [40])

4 Experimental Results of Shear Wall

4.1 Introduction

This chapter presents the experimental results that was recorded during testing of the steel-reinforced concrete shear wall SFW. This wall was subjected to quasi-static load reversals and a constant applied axial load. This chapter will include the results from cylinder compression tests for the concrete and reinforcement tension tests for the two different bar sizes. The data captured from the instrumentation attached to the wall also be presented, including: the lateral load-displacement response, the strains in the reinforcement from the recorded by strain gauges, the wall displacements obtained from the linear and cable potentiometers, and the data recorded from the actuators. The posterior face of the wall was photographed to analyze crack patterns for comparison to the numerical models and for failure identification.

4.2 Concrete Properties

The wall tested in this experiment was cast in two pours. The foundation was poured initially, with the wall foundation interface roughened for adequate bond. The second pour was for the wall and cap beam. The desired concrete strength for both pours was 30 MPa after 28 days. The cylinder size used was four inches in diameter by eight inches in height. Cylinders were tested at 7 days, 28 days, and day of testing. Cylinders were tested the day before testing to ensure the numerical model could be analyzed and the axial load could be calculated. Note that the axial load was based on 10% of the cross-sectional strength of the wall, $f'_c A_g$, where f'_c is the compressive strength of the concrete and A_g is the gross sectional area. Due to delays in the experimental program, the wall was tested 160 days after casting. The compressive strength of the concrete is provided in Table 4.1. The concrete strength is based on the average of 4 cylinder tests.

Table 4.1: Dates of Casting and Concrete Compressive Strengths

Location	Date of Casting	Date of Testing	f'_c [MPa]	
			28 Days	Day of Testing
Foundation	March 7, 2022	September 14, 2022	43.2	43.2 (191 Days)
Wall/Cap beam	April 7, 2022	September 14, 2023	29.4	38.3 (160 Days)

There were several repair locations in the wall. After casting the wall had some light honeycombing due to improper consolidation around the toes at the base of the wall as well as along the base on the front face of the wall. These locations are depicted in Figure 4.1, Figure 4.2, and Figure 4.3. Along the back of the wall there was no honeycombing along the base. None of the locations required chipping and recasting as the deepest voids were under 10mm in depth. To rectify these imperfections, an SCC mix was prepared and used to fill the voids. The wall was then covered in wet burlap and plastic. It was routinely wet for 28 days after the repair. The repair took place on April 13th, 2022. The SCC was not tested as the volumetric amount used was deemed not sufficient to have a significant impact on the wall's behaviour.



Figure 4.1: Locations of Poor Consolidation at the Toe on the Actuator Side



Figure 4.2: Locations of Poor Consolidation at the Toe Opposite the Actuator Side



Figure 4.3: Locations of Poor Consolidation along the Base of the Front Face of the Wall

4.3 Reinforcement Properties

Three, 10M and three, 15M steel coupons were subjected to uniaxial tension tests to rupture to determine the mechanical properties of the reinforcement. These coupon tests took place on May 20th, 2022.

The results obtained from the tension tests were averaged to determine the properties of the 10M and 15M reinforcing bars. The yield strength (f_y) was determined by the location of the yield plateau. The ultimate strength (f_u) was determined as the maximum force output of the actuator divided by the original area of the bar. The modulus of elasticity (E) was calculated from the slope of the elastic region prior to yielding. The strain data was recorded by an external extensometer. For the 10M bars, the modulus of elasticity (E) was determined to be 204 GPa. The yield strength (f_y) and yield strain were calculated to be 440MPa and 0.23%, respectively. The ultimate strength (f_u) and corresponding strain were determined to be 597MPa and 13.88%, respectively. No cyclic tests were conducted on the steel reinforcement. Two of the three bars ruptured within the extensometer for both sets of bars. When bars rupture outside of the extensometer, a sudden drop occurs. This is evident by the rapid decline at maximum by bar 10.1 in Figure 4.4. The results for the uniaxial tension tests are listed in Table 4.2.

Table 4.2: Uniaxial Tension Test Results

Type of Rebar	Bar Diameter (mm)	Yield Strength (MPa)	Yield Strain (%)	Modulus of Elasticity (MPa)	Ultimate Strength (MPa)	Ultimate Strain (%)
10 M	11.3	440.0	0.23	204077.5	597.0	13.88
15M	16.0	437.9	0.18	194830.0	619.4	13.23

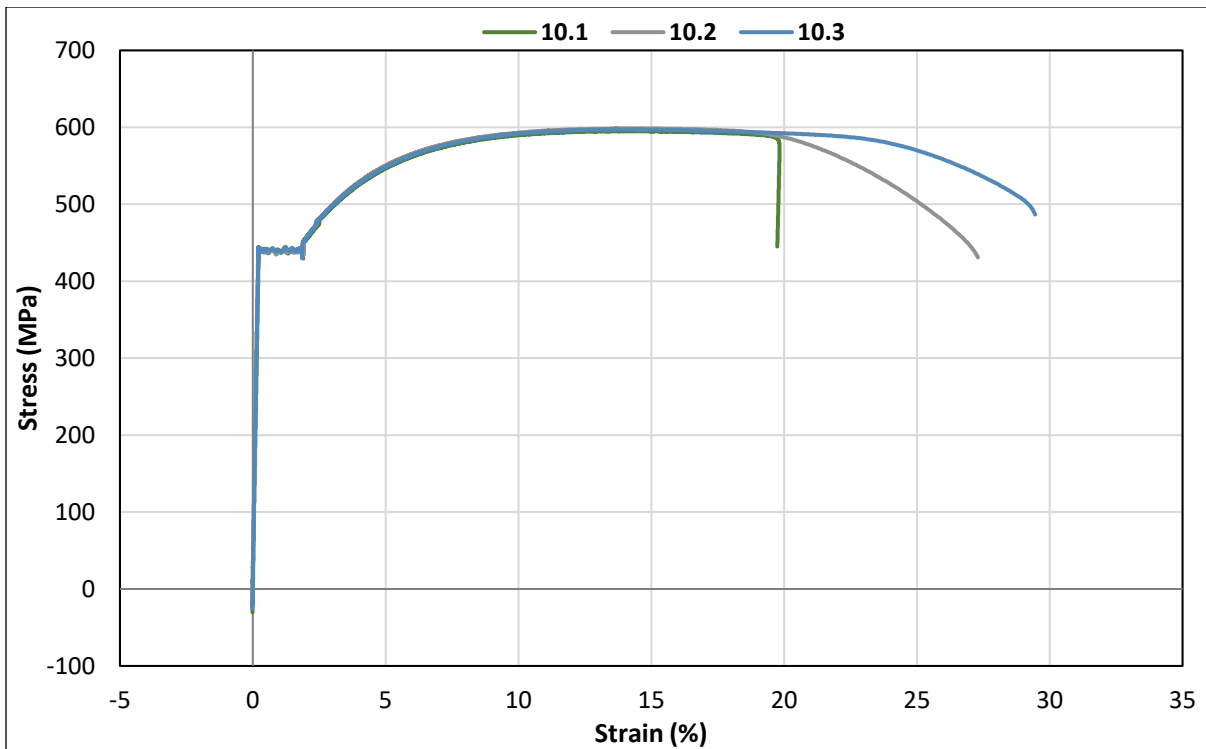


Figure 4.4: Uniaxial Tension Test Results for 10M Bars

4.4 Shear Wall Testing

4.4.1 General Response

This section focuses on the major points identified on the load-displacement response of the shear wall. Section 4.4.2 Cracking Sequence provides a cycle-by-cycle analysis of the cracking during testing. The wall experienced cracking at different drifts, and they grew to different lengths and widths. Only the most important cracks will be presented within this section, while the detailed cracking will be resented in Section 4.4.2

The complete lateral load-displacement hysteretic response of the wall was recorded using a cable potentiometer located at the mid-height of the cap beam, CP1, and the load recorded from the actuator. CP1 was secured to the instrumentation frame, which was anchored onto the foundation block to ensure rigid body motions of the overall wall would not affect the results. The displacement readings recorded by CP1 were monitored during testing to determine when to pause loading, unload, and reload. Using the data recorded from CP1 and from the force output of the actuator, the performance points (yield displacement, peak load, and ultimate displacement) were established. The performance points were derived from the averages

between the positive and negative cycles of loading. The test commenced with the application of axial loading. This resulted in the localization of cracks within the bottom half of the wall and is consistent with research reported in the literature. It was hypothesized that the damage of the concrete would be localized within the bottom half of the wall.

The first two displacements of 1.25mm (0.05% drift) and 2.5mm (0.10% drift) did not cause cracking, as illustrated in Figure 4.5. [Note that Push refers to the positive direction of loading and pull the negative direction of loading.] Cracks began to develop during the third displacement of 5mm (0.20% drift). On both sides of the wall, four flexural cracks developed below a height of 650mm above the base. These cracks were all horizontal across the boundary region of the wall (refer to Figure 4.6). Cracks also developed at the construction joint between the wall and the foundation. After unloading from the third cycle of 5mm (0.20% drift) all cracks closed, and this can be attributed to the applied axial loading and the reinforcement responding in the elastic range.

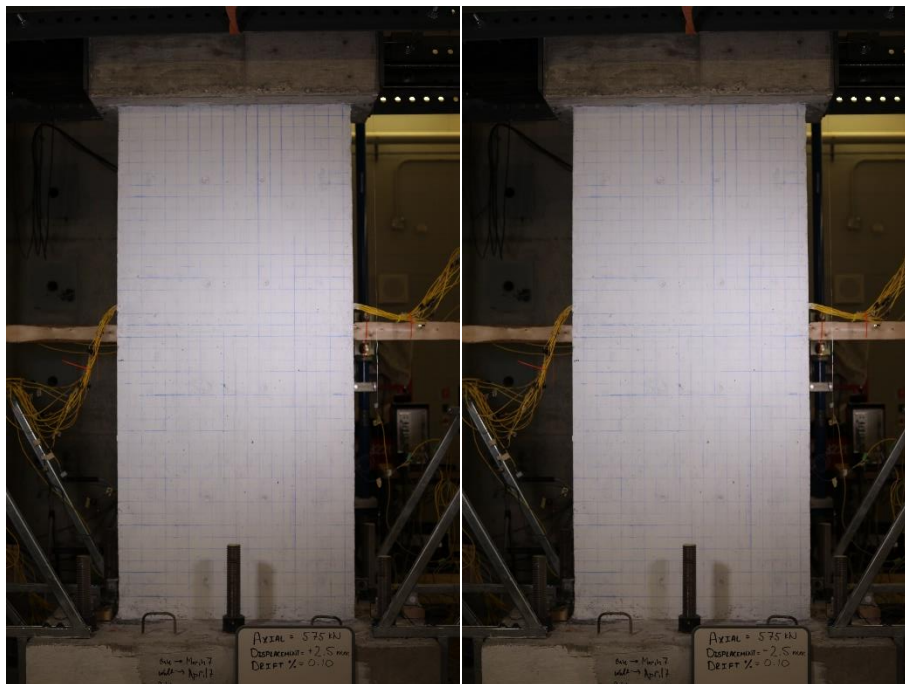


Figure 4.5: Wall at 0.1% Drift (Push on the Left, Pull on the Right)



Figure 4.6: Wall at 0.2% Drift (Push on the Left, Pull on the Right)

The cracks were able to close during unloading until after the displacement cycles of 25mm (1.0% drift). During the fourth displacement of 7.5mm (0.30% drift), more flexural cracks appeared on the tension faces of both the push and pull cycles (refer to Figure 4.7). These cracks appeared at a maximum height of 850mm above the base and the lower cracks became diagonal in the web section of the wall. These were identified as shear cracks. The flexure-shear cracks only occurred in one crack on either side of the wall at a height of 300mm above the base. Some spalling surfaced at the toe of the wall in the SCC surface repair region on the push side. The fifth displacement of 10mm (0.4% drift) saw the development of flexure-shear cracking up to a height of 850mm above the base of the wall (refer to Figure 4.8). Cracks were observed and measured at heights of half of the height of the wall. Yielding was expected to occur during the fifth displacement cycle. This cycle saw the largest increase in crack lengthening. During the sixth displacement of 12.5mm (0.5% drift), more cracks appeared on the tension face of the pull cycles, however no new cracks formed on the tension face of the push cycles (see Figure 4.9). The existing cracks continued to widen and lengthen diagonally through the web. Spalling continued to develop at the toe of the wall in the SCC repair region on the push side.



Figure 4.7: Wall at 0.3% Drift (Push on the Left, Pull on the Right)

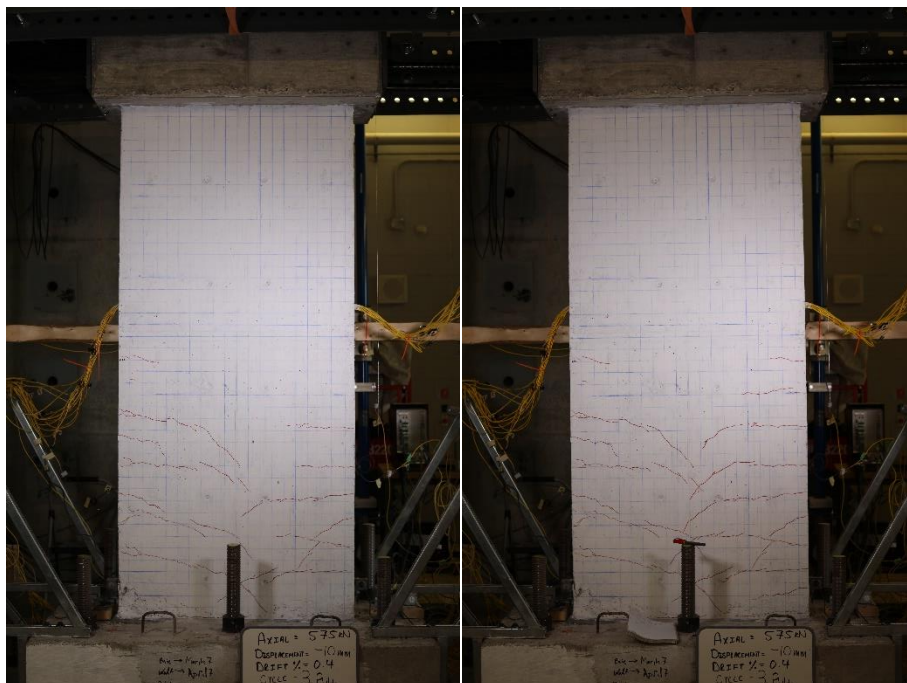


Figure 4.8: Wall at 0.4% Drift (Push on the Left, Pull on the Right)

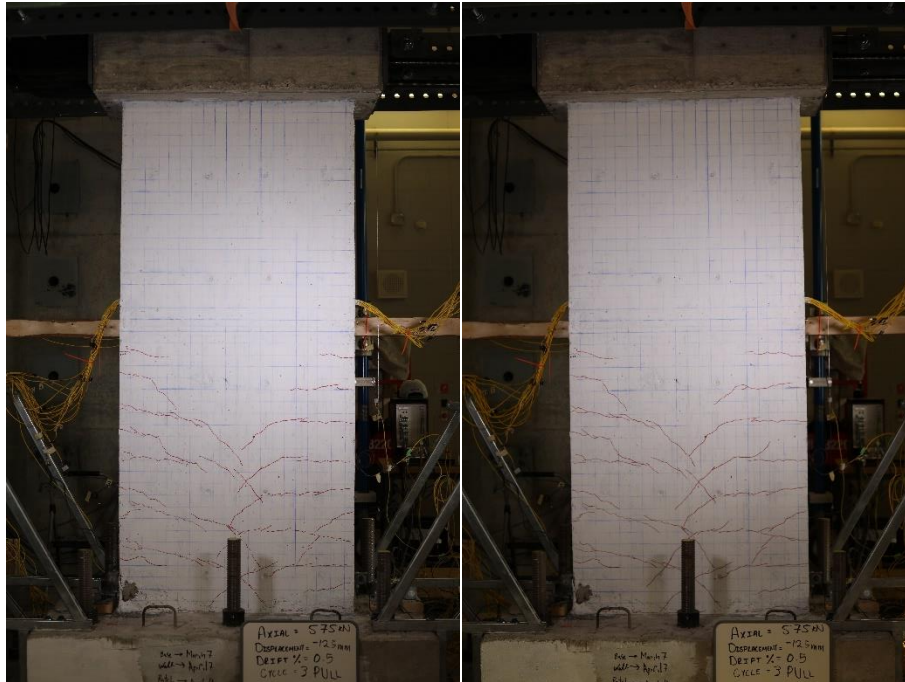


Figure 4.9: Wall at 0.5% Drift (Push on the Left, Pull on the Right)

The seventh displacement of 25mm (1.0% drift) saw the development of the last new cracks on the wall (see Figure 4.10). The cracks on both sides of the wall were all located within 1250mm above the base; only on the tension face of the pull side of the wall was there a crack at 1400mm above the base. Also, during the seventh displacement cycles, the cracks began to merge in the web. The extent of the damage at this point was maintained within the bottom half of the wall, however the cracks at 1250mm on both sides developed diagonally into the web from the boundary regions. The cracks along the base began to widen significantly compared to previous cycles. The residual drift at this point in the test was noticeable; the cracks on the tension side of the push cycle did not fully recover and close, however the same was not true for the pull side. On the pull side, the cracks located within 200mm above the base only recovered slightly and remained open. The spalled section at the toe of the push side during the pull cycles began to develop as some concrete had spalled off. On the toe of the pull side for the pull cycles, the concrete began to spall off up to 100mm above the base.

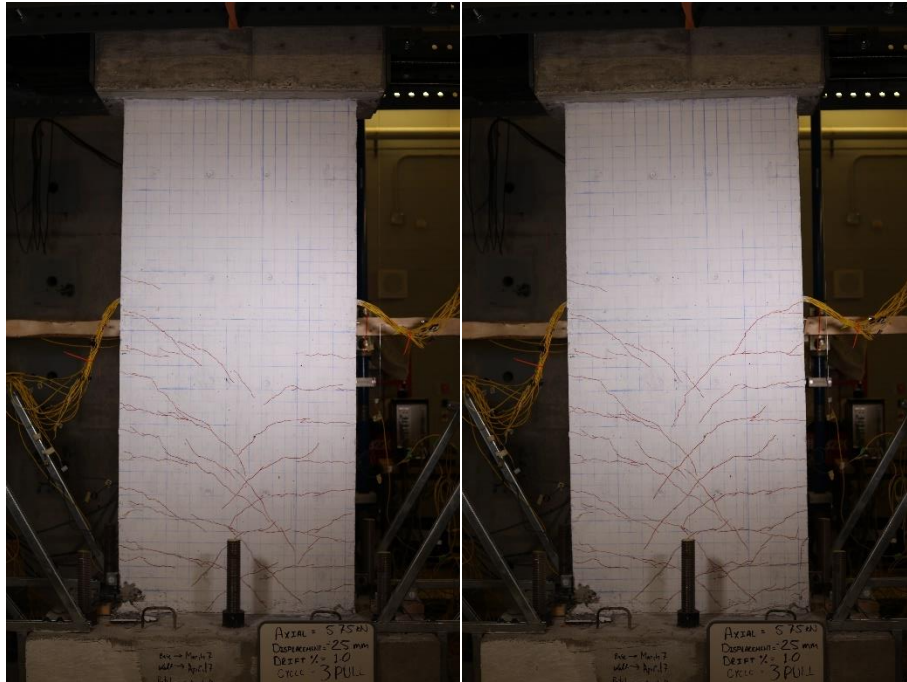


Figure 4.10: Wall at 1.0% Drift (Push on the Left, Pull on the Right)

The eighth displacement of 37.5mm (1.5% drift) lead to widening of the cracks; however, the further crack length development was not apparent (see Figure 4.11). On both compression sides of the push and pull cycles the concrete began to crush, and spalling began to occur on the compression side of the push cycles. Large pieces of concrete spalled off, but no steel was exposed at this point of the test. Crack recovery was evident for the cracks located above 450mm from the base. All cracks below 450mm had unrecoverable widths. An important note, however, is that all cracks appeared to have an even distribution of widths with no cracks opening significantly more than the others.

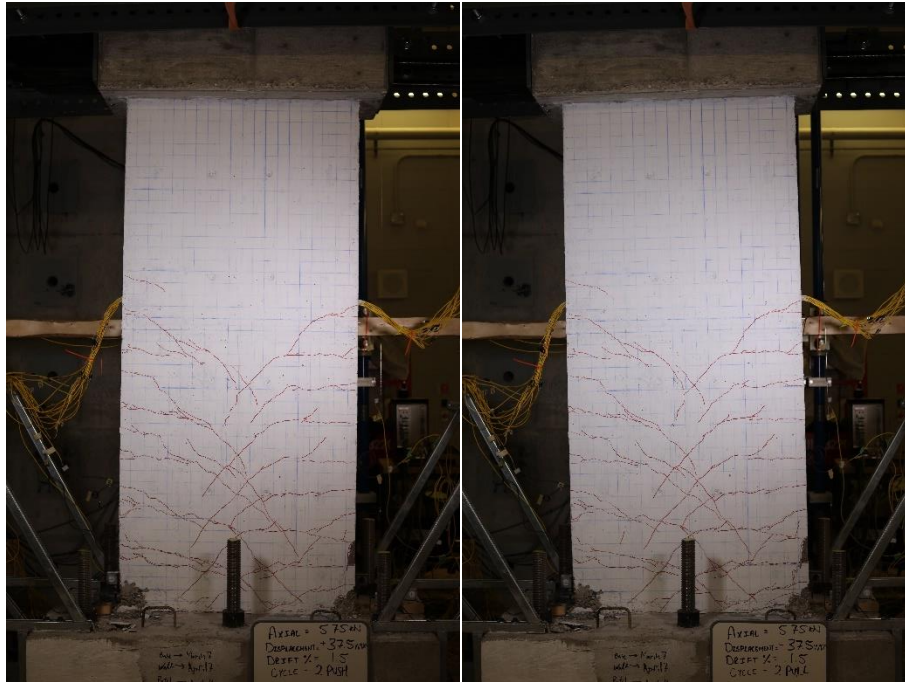


Figure 4.11: Wall at 1.5% Drift (Push on the Left, Pull on the Right)

The ninth displacement of 50mm (2.0% drift) resulted in crushing on the compression sides of both push and pull cycles (see Figure 4.12). The compression side of the push cycle had significant loss of concrete cover up to a height of 300mm from the base. The cracks along the base began to widen. On the tension side of the push cycle, the cracks at 250mm and 300mm above the base were noticeably larger and wider into the web of the wall than the other cracks; the flexural and shear cracks appeared wider along the entire crack rather than solely in the boundary region at this point of the test. The same was also noticed on the tension side of the pull cycle for the cracks at 150mm, 200mm, and 300mm above the base. When the lateral load was removed, only the cracks above 550mm from the base closed; cracks below 550mm from the base of the wall did not recover fully.

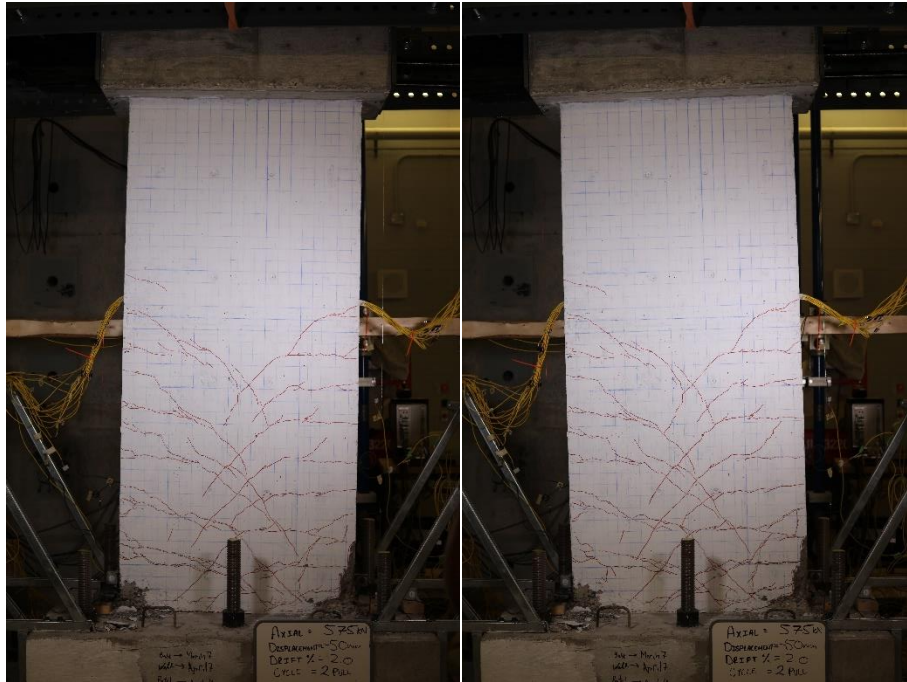


Figure 4.12: Wall at 2.0% Drift (Push on the Left, Pull on the Right)

The tenth displacement cycle of 62.5mm (2.5% drift) was the last cycle where cracks were recorded as the wall ultimately failed during the second cycle of this displacement (see Figure 4.13). During this cycle, the majority of the concrete cover on the boundary regions spalled leaving the stirrups and longitudinal bars exposed for approximately 200mm above the base at both toes of the wall. The concrete within the boundary region at the toe of both sides of the wall was crushing. The concrete within the boundary regions was beginning to reduce in area and stiffness. At this point, rebar buckling was visible in the boundary regions. During the second push cycle, the longitudinal bar near the end of the wall in the boundary region on the back of the tension side ruptured. During the following pull cycle, the longitudinal bar in the boundary region on the back of this tension side also ruptured. During the 75mm (3.0% drift) and 87.5mm (3.5% drift) cycles, spalling and crushing continued as the lateral strength reduced (see Figure 4.14 and Figure 4.15). The test was continued until the lateral load capacity reached 50% of the maximum lateral load (see Figure 4.16).

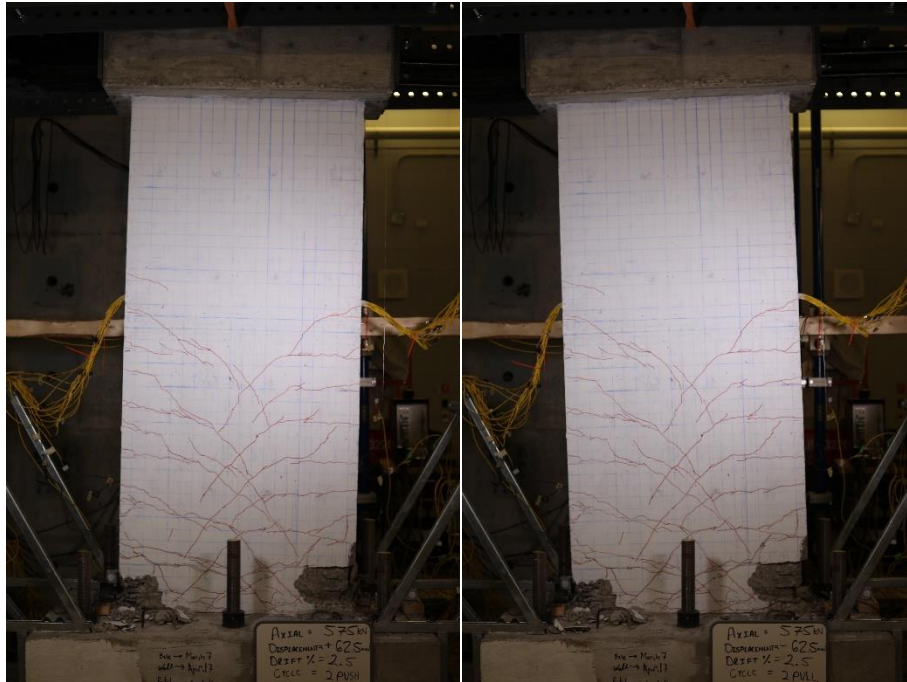


Figure 4.13: Wall at 2.5% Drift (Push on the Left, Pull on the Right)



Figure 4.14: Wall at 3.0% Drift (Push on the Left, Pull on the Right)

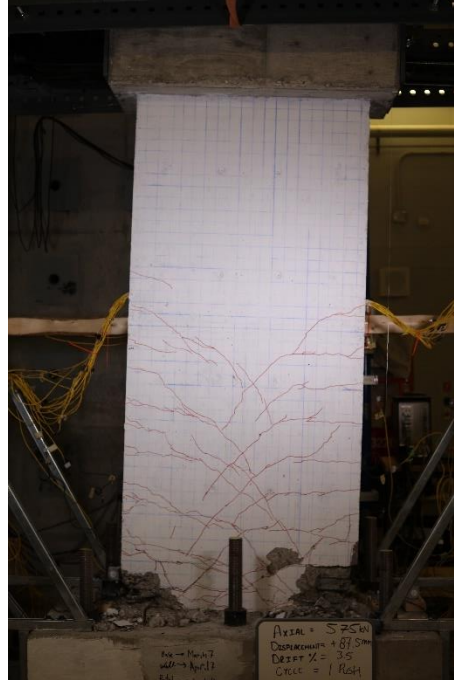


Figure 4.15: Wall at 3.5% Drift (Push cycle only)

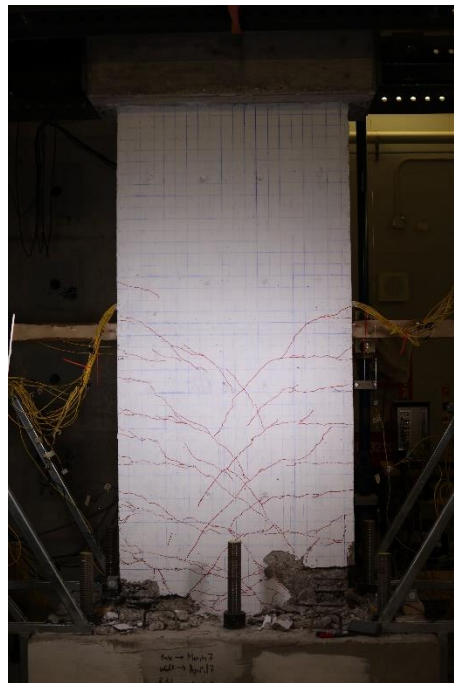


Figure 4.16: Wall Returned Back to Starting Position at the End of the Test

4.4.1.1 Load-Displacement Response

Figure 4.17 provides the lateral load-displacement response. Prior to yielding of the vertical reinforcement, the wall exhibited negligible to minimal residual displacements. Up to 0.5% drift (12.5mm), the wall maintained a linear-elastic response.

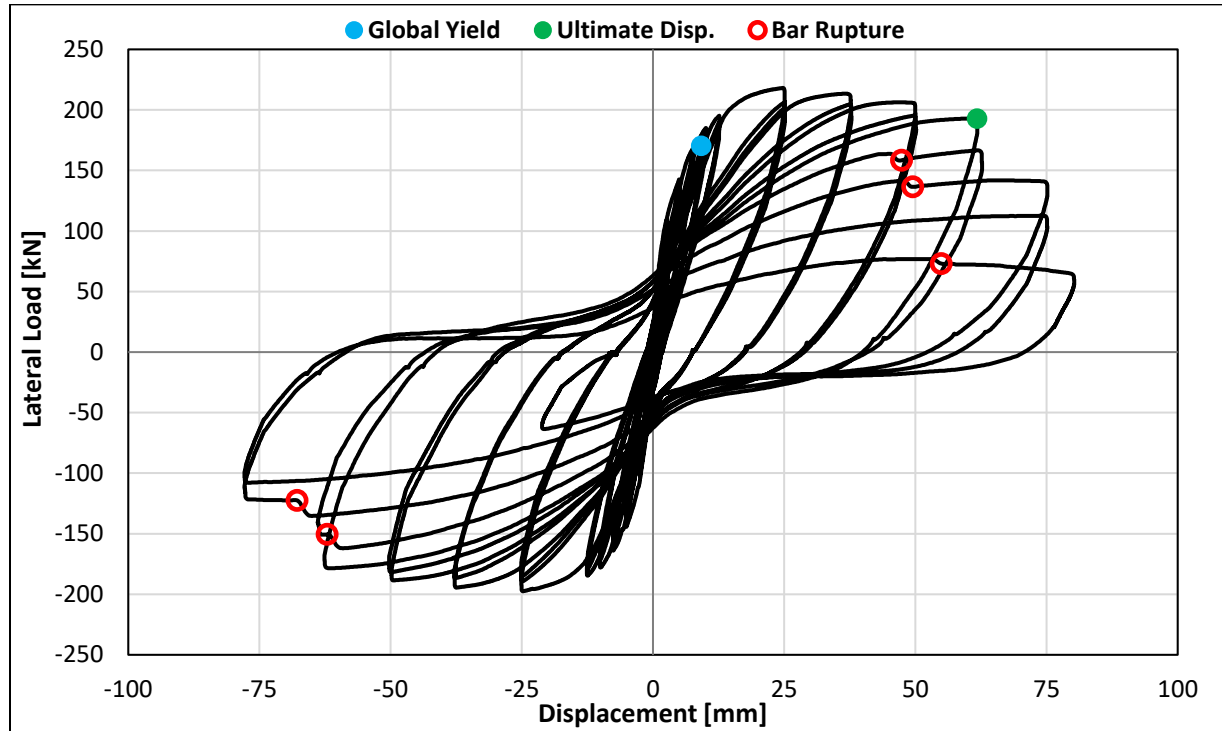


Figure 4.17: Load-Displacement Response of CP1

The presence of applied axial loading is known to create added recovery during unloading. The presence of 10% of the wall's axial capacity as applied axial loading had an effect on the hysteretic response. This is evident by the pinching on the lateral load-displacement response. Pinching is identified in Figure 4.18 by the narrowing of the hysteretic response around the origin. Up to and including 25mm (1.0% drift) displacement level, the cycles were repeated three times; thereafter the cycles were repeated twice. The stiffest response to each new displacement level was capture during the first cycle, after which, each subsequent cycle had degradation in stiffness. Notable reasons for the stiffness degradation are due to the fatigue of the steel and reduced contact area of concrete from increased crack development.

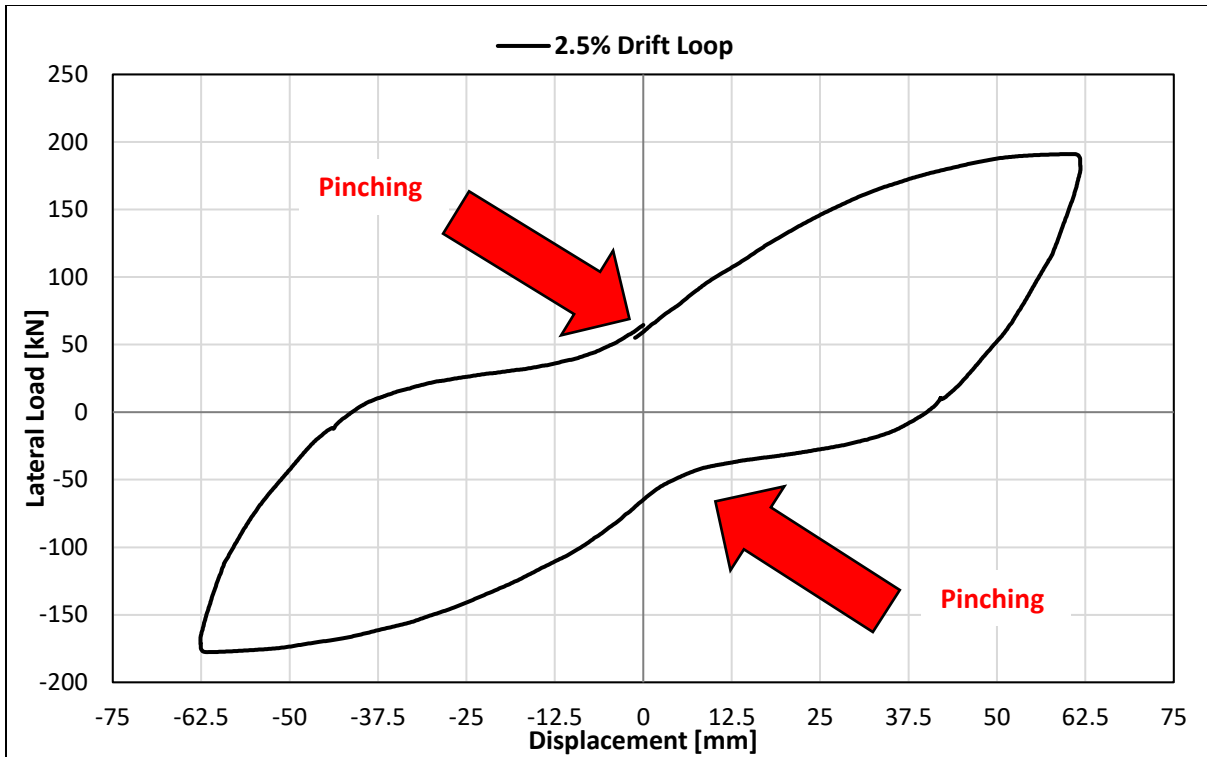


Figure 4.18: Identification of Pinching

Overall, the damage experienced by the wall continually developed after the first cracks appeared. The cracks were flexure-shear dominant, and this was apparent by the horizontal cracking spanning from the boundary region into diagonal cracks into and across the web of the wall. The damage was contained within the bottom half of the wall, with the majority of the damage taking place within a height of 550mm from the base of the wall. Residual drifts were visible when the wall was unloaded and some cracks began to remain visible after the cycles of 25mm (1.0% drift) of displacement, and these cracks continued to remain visible throughout the remainder of the test.

The ultimate displacement was established as the cycle prior to fracture of the reinforcement. After the first cycle for +/- 62.5mm (2.5% drift), the lateral load capacity experienced a 22% reduced on average from the average maximum load. This cycle was then deemed the maximum displacement prior to failure. The second cycles of +/- 62.5mm (2.5% drift) resulted in a 25% reduction of lateral load capacity and was determined as the failure point of the test. This was also the cycle that corresponded to fracture of the first reinforcing bars. The

test was continued until a substantial reduction in lateral load resistance was evident, which arose during loading to 3.5% drift (87.5mm). The loading was terminated and returned to zero prior to attaining 87.5mm. This was also necessary to prevent the vertical actuator coming into contact with threaded rods securing it. The load resistance at this stage had reduced to greater than 50% of the maximum. Values corresponding to key points in the lateral load-displacement response are summarized in Table 4.3 and Table 4.4.

Note that yielding was determined using the Park Method (see Figure 4.19). To determine the yield point from Figure 4.17 using the Park method [41], first the maximum force is determined, and a horizontal line is drawn, after which 75% of the maximum force ($0.75 \cdot F_{max}$) is calculated and a corresponding horizontal line is drawn. A diagonal line is then drawn from the origin and travels through the intersection of the $0.75 \cdot F_{max}$ line and the Load-Displacement Response. The line is extended with the same slope until it intersects the maximum lateral load line. A vertical line is then drawn from the intersection of the diagonal line and the maximum line and extended down to the horizontal axis. The intersection of this vertical line and the Load-Displacement Response is the yield point.

For the positive side of the graph (see Figure 4.19), 218.13kN was determined to be the maximum, 75% of this is roughly 163.6kN. Finding the closest point on the graph to 163.6kN, a point was located at 162.6kN at a displacement of 7.84mm. The slope of this point from the origin was found to be 20.74. The yield displacement is then calculated as $218.13/20.74 = 10.52\text{mm}$.

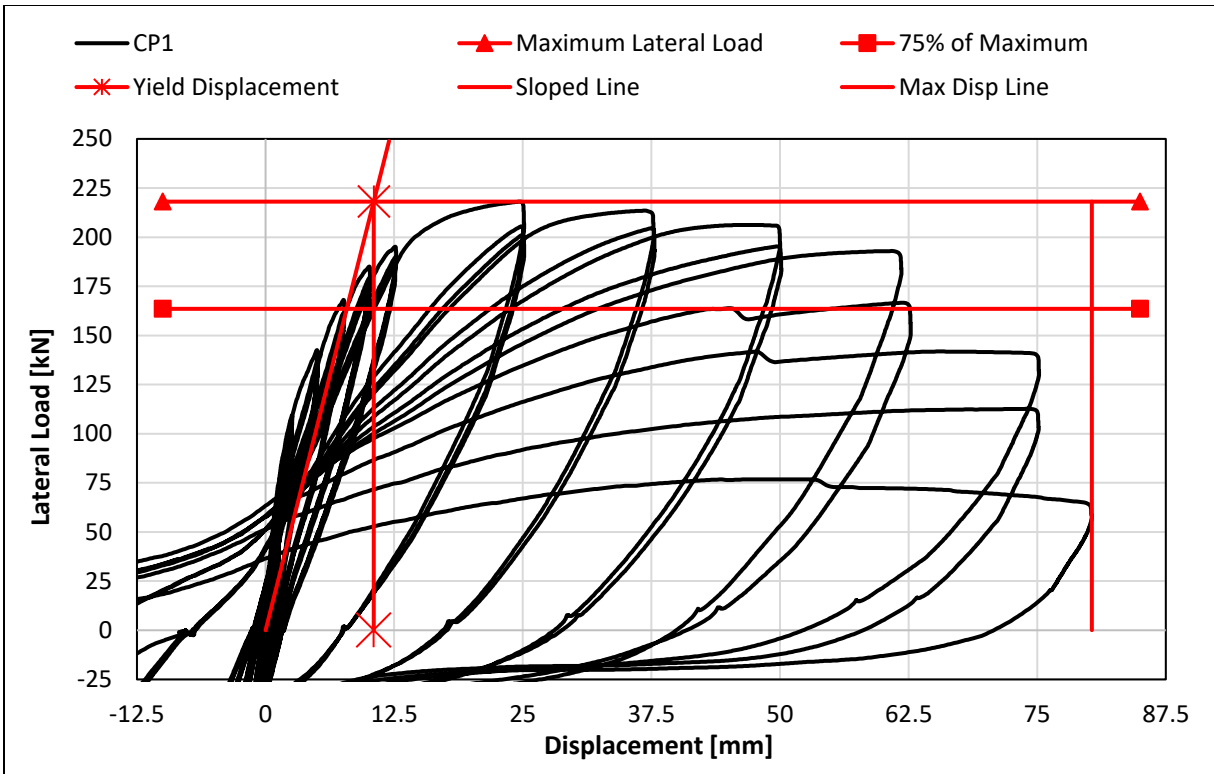


Figure 4.19: Overlay of Park Method on Load-Displacement Response of CP1

Table 4.3: Yield, Peak, and Ultimate Drifts and Loads from Figure 4.17

	Yield			Peak			Ultimate		
	Disp [mm]	Drift [%]	Load [kN]	Disp [mm]	Drift [%]	Load [kN]	Disp [mm]	Drift [%]	Load [kN]
Pos +	10.52	0.42	179.96	24.74	0.9917	218.13	61.81	2.48	182.20
Neg -	7.95	0.32	160.91	24.77	0.9929	197.49	62.61	2.51	169.73
Avg.	9.23	0.37	170.43	24.76	0.99	207.81	62.21	2.49	175.96

Table 4.4: First and Second Reinforcement Fractures from Figure 4.17

	(+62.5mm)	(-62.5mm)	Average
	First Fracture	Second Fracture	
Disp @ fracture [mm]	45.19	-59.09	52.14
Force @ fracture [kN]	163.83	-162.01	162.92
% change in Capacity	24.90	17.97	21.43
Disp @ max after fracture [mm]	61.75	-63.44	62.60
Force @ max after fracture [kN]	166.59	-150.12	158.36
% change in Capacity	23.63	23.99	23.81

Five boundary reinforcing bars ruptured by the end of testing; all four of the outer bars and one of the interior bars on the back of the tension side during the final push to 87.5mm (3.5% drift). The bars on the back of the wall ruptured during the last cycle of 62.5mm (2.5% drift). The bars on the front ruptured during the first cycle of 75mm (3.0% drift).

4.4.2 Cracking Sequence

This section provides a sequential discussion of the cracking and crack patterns throughout testing. The wall was subjected to quasi-static reverse cyclic lateral loading and applied axial loading. The lateral loading was applied in “push” (positive) and “pull” (negative) directions. The orientation of the wall has the North (front) face facing the camera. West is referred to as the push direction and East is referred to as the pull direction. Through the load displacement response below, the East and West sides of the wall will be referred to as either the compression side or tension side of the wall depending on which type of action is being applied by the actuator. A table of the entire cracking sequence can be found in Appendix M and the photos can be found in Appendix K and Appendix L.

The load-displacement response of the test was based on the displacement of the cable potentiometer located at the mid-height of the cap beam (CP1). This potentiometer was secured

to the instrumentation frame which was secured to the foundation block. This ensured that any sliding over rocking of the foundation would not affect the displacement output of CP1.

The test initiated with the application of 575kN of axial loading to the top beam of the wall. After the axial load had been fully applied and maintained without any fluctuations, the lateral load was applied. As previously stated, CP1 was used to determine the displacement of the cap beam, however the force of the actuator was controlled based on these displacements. The process of lateral loading was as follows: load was applied and the reading of CP1 was monitored until the desired displacement was achieved. At that point, the loading was stopped, marking the peak of the push cycle. Thereafter, the actuator was unloaded until it zero load was recorded. Then the pull cycle began. Once the desired displacement was achieved, the loading was paused, followed by unloading the actuator to zero force. This completed one full cycle of lateral loading.

In measuring cracks, the length was based on their horizontal distance across the wall. The crack widths were taken as the largest width along the cracks. This generally occurred in the boundary region, approximately 50 – 100mm from the edges of the wall. The locations of the cracks were taken as the vertical distance from the base of the wall to the crack and rounded to the nearest 50mm to correspond to the grid on the wall in the web section (50mm by 50mm squares). In the boundary region, this grid was 40mm in height and 60mm in width to match the numerical model analyzed in VecTor2.

Testing commenced with a first lateral displacement of 1.25mm (0.05% drift). No cracking was observed during and after the first push/pull cycles of 1.25mm (0.05% drift). After the cycles were completed, the anchor rods were checked to ensure they were properly anchored. They were retightened to minimize sliding and rocking of the foundation. Once the rods were tightened, another issue arose with one of the rollers used to prevent twisting of the cap beam. The roller closest to the actuator, on the west side of the wall, had to be removed as it added significant friction during contact with the cap beam. This was evident by a jump in the force recorded by the actuator during the pull cycle. Once these issues were rectified, the remaining cycles of 1.25mm (0.05% drift) were completed. Prior to proceeding to the second displacement, the instrumentation frame needed to be adjusted slightly as it was close to contacting one of the

beams holding the rollers. Once this was corrected, the test recommenced to a displacement of 2.5mm (0.1% drift). All three cycles were run, and no cracking was observed.

Note that after the first cracks were observed, subsequent cracks were marked, recorded, and measured during the final push and pull cycles. This ensured adequate cracking could be developed as well as reducing stop/start times of the testing. The third displacement was 5mm (0.2% drift). During the first push cycle, four cracks were observed on the tension side of the north face of the wall. During the first pull cycle of 5mm (0.2% drift), four cracks were also observed on the tension side of the north face of the wall.

The crack widths were measured during the third push cycle of 5mm (0.2% drift). The first crack was along the base, approximately 100mm long and 0.10mm wide. The second was a flexural crack about 300mm above the base, horizontal through the boundary region while slightly diagonal in the web region, with a total length of about 500mm and crack width of 0.20mm. The third crack surfaced 400mm above the base and extended up to 450mm above the base before travelling diagonally into the web region. This crack was 350mm long and 0.25mm wide. The fourth crack was 650mm above the base and was horizontal across the boundary region. The crack was 200mm long and 0.10mm wide. When the push cycle was returned to zero force, the cracks were closed and only able to be identified by the red marker used to distinguish the cracks.

During the third first pull cycle of 5mm (0.2% drift), four additional new cracks were observed. The first crack surfaced at the base and was 150mm long and 0.10mm wide. The second crack was at 200mm above the base; it was 350mm long and 0.15mm wide. The third crack occurred at 300mm above the base and was 350mm long and 0.15mm wide. The fourth crack occurred at 650mm above the base and was 250mm long and 0.10mm wide. The cracks on the tension side of the wall during this pull cycle were mainly horizontal, except for the crack that was 150mm above the base, which was diagonal. Compared to the cracks present on the compression side (compression during the pull cycle), the cracks were horizontal and diagonal. When the horizontal actuator was unloaded, all cracks had closed and were no longer visible aside from the red marker outlines.

During the third push cycle for 7.5mm (0.30% drift) the previous cracks at the base, 300mm, 400mm, and 650mm, grew in length and width. Three new cracks formed on the tension face at 100mm, 550mm, and 850mm above the base. Starting at the base, the crack remained 150mm long and 0.10mm wide. The new crack at 100mm above the base was L-shaped; extending vertically from 100mm to 150mm from the base, and horizontally out 50mm. Some spalling was observed but was deemed to be loose SCC in the repaired region. The crack at 300mm above the base grew diagonally across the wall to a length of 550mm and grew in crack width to 0.25mm. The crack at 400mm above the base also grew diagonally across the wall to a length of 550mm with a width of 0.25mm. The new crack at 550mm above the base grew diagonally through the boundary to a length of 200mm and a width of 0.10mm. The crack at 650mm above the base grew in length horizontally across the wall to 350mm and the width remained unchanged at 0.10mm. The final new crack at 850mm above the base grew horizontally to 300mm and had a width of 0.1mm. After the force was returned to zero, all cracks on the tension face were fully recovered and closed, with the exception of the spalled area.

During the third pull cycle for 7.5mm (0.30% drift) the previous cracks at the base, 200mm, 300mm, and 650mm, grew in length and crack width. Three new cracks formed on the tension face at 450mm, 550mm, and 800mm above the base. The crack at the base remained the same, with a length of 150mm and width of 0.10mm. The crack at 200mm above the base grew diagonally into the web of the wall to a length of 600mm and had a width of 0.25mm. The crack at 300mm above the base remained the same with a 350mm length but did grow in width to 0.20mm. The new crack at 450mm above the base grew diagonally through the web and merged with the cracks at 200mm and 300mm above the base; the crack width was 0.10mm and the total length was 250mm. The new crack at 550 above the base grew horizontally through the boundary region and diagonally into the web to a total length of 450mm with a width of 0.15mm. The crack at 650 above the base remained the same length of 250mm but grew in width to 0.15mm. The new crack at 850mm above the base grew horizontally to a length of 300mm across the boundary region and had a width of 0.15mm. After the wall was returned to zero load, all cracks on the tension face were fully recovered and closed.

During the third push cycle to 10mm (0.40% drift) the previous cracks at the base, 100mm, 300mm, 400mm, 550mm, 650mm, and 850mm grew in length and crack width. Two new cracks formed at 775mm and 1100mm above the base. The crack along the base grew horizontally to a length of 300mm and to a width of 0.30mm. The crack at 100mm above the base remained the same and spalling did not continue in this cycle. The crack at 250mm above the base grew diagonally to a length of 650mm and remained at a crack width of 0.25mm. The crack at 400mm above the base grew diagonally in length to 600mm and in width to 0.35mm. The crack at 550mm above the base did not change in length at 200mm, however, it was close to merging with the crack at 400mm above the base; the width of the crack grew to 0.15mm. The crack at 650mm above the base grew diagonally into the web to a length of 550mm and to a width of 0.15mm. The new crack at 775mm above the base grew horizontally across the boundary region to a length of 200mm and began to spread diagonally through the last 50mm of the crack and had a width of 0.15mm. The crack at 850mm above the base grew diagonally through the web to a length of 550mm and to a width of 550mm. The new crack at 1100mm above the base grew horizontally across the boundary region to 150mm. When the wall was unloaded and returned to zero load, the cracks recovered and closed.

During the third pull cycle to 10mm (0.40% drift), the previous cracks at the base, 200mm, 300mm, 450mm, 550mm, 650mm, and 850mm grew in length and crack width. Three new cracks formed at 150mm, 950mm, and 1100mm above the base. The crack along the base grew horizontally in length to 300mm and to a width of 0.25mm. The new crack at 150mm above the base grew horizontally and diagonally across the boundary region to a length of 200mm with a width of 0.10mm. The crack at 200mm above the base remained unchanged in length of 600mm and width of 0.25mm. The crack at 300mm above the base grew diagonally through the web to a length of 600mm with a width of 0.25mm. The crack at 450mm above the base remained unchanged. The crack at 550mm above the base grew diagonally in length through the web to 550mm and grew in width to 0.20mm. The crack at 650mm above the base grew diagonally through the web region to a length of 500mm and had a crack width of 0.20mm. The crack at 800mm grew diagonally into the web to a length of 450mm and remained at the same width of 0.15mm. The new crack at 950mm above the base grew horizontally through the boundary region

to a length of 350mm and to a width of 0.10mm. The final new crack at 1100mm above the base grew horizontally into the boundary region to a length of 150mm and to a width of 0.10mm. When the lateral load was removed, all cracks recovered. Note that on the compression side of this pull cycle, the area of spalling appeared to have enlarged as a portion of the concrete began to debond just above the toe of the wall; this was the only area which did not recover fully.

During the third push cycle to 12.5mm (0.50% drift), the previous cracks at the base, 100mm, 300mm, 400mm, 550mm, 650mm, 775mm, 850mm, and 1100mm grew in length and crack width. Two new cracks formed at 250mm and 1000mm above the base. The crack along the base grew across the construction joint to a length of 400mm and remained at the same width of 0.30mm. The spalled section expanded and loose pieces debonded. The new crack at 250mm above the base grew horizontally through the boundary region to a length of 200mm with a width of 0.15mm. The crack at 300mm above the base remained the same length but the width increased to 0.30mm. The crack at 400mm above the base remained the same. The crack at 550mm above the base also remained the same. The crack at 650mm above the base grew diagonally into the web to a length of 600mm but remained at a width of 0.15mm. The crack at 775mm above the base grew diagonally in length to 350mm and merged with the crack at 650mm above the base; the width was unchanged at 0.15mm. The crack at 850mm remained the same. The new crack at 1000mm above the base grew horizontally through the boundary and diagonally into the web to a length of 250mm and a width of 0.15mm. The crack at 1100mm above the base grew horizontally to a length of 200mm and remained at a width of 0.10mm. When the lateral load was unloaded, all cracks recovered and closed except for the spalled area.

During the third pull cycle to 12.5mm (0.50% drift), the previous cracks at the base, 150mm, 200mm, 300mm, 450mm, 550mm, 650mm, 850mm, 950mm, and 1100mm grew in length and crack width. No new cracks formed on the tension face of the pull cycle for 12.5mm (0.50% drift). The crack along the base remained 300mm long but opened significantly (0.60 mm) compared to the previous cycle. The crack at 150mm above the base remained unchanged. The crack at 200mm above the base grew diagonally to the base of the wall to a length of 650mm and opened to a width of 0.30mm. The crack at 300mm above the base continued diagonally through the web to a length of 700mm and remained at the same crack width. The crack at

450mm above the base remained unchanged. The crack at 550mm above the base remained at the same length but opened to a width of 0.25mm. The crack at 650mm above the base continued diagonally through the web region to a length of 600mm and remained at the same crack width. The crack at 800mm continued diagonally into the web to a length of 500mm and remained at the same width. The crack at 950mm above the base continued diagonally into the web to a length of 400mm and stayed at the same width. The crack at 1100mm above the base grew horizontally through the boundary region and into the web to a length of 250mm and to a width of 0.10mm. When the lateral force was removed, all cracks were recovered and closed.

For brevity and focusing on the critical section of the wall, the following will focus on the cracking details within the lower 550mm of the wall.

During the third push cycle to 25mm (1.0% drift), the previous cracks at the base, 100mm, 250mm, 300mm, 400mm, 550mm, 650mm, 775mm, 850mm, 1000mm, and 1100mm grew in length and crack width. Two new cracks formed at 1250mm and 1400mm above the base. No new cracks formed in following cycles and only existing cracks further developed and began to merge. Spalling was observed and crushing began to surface on the compression side of this cycle. The crack along the base opened up significantly to a width of 3mm. The crack at 100mm above the base remained unchanged, however, more spalling arose. The crack at 250mm above the base of the wall opened to 0.35mm. The crack at 300mm above the base of the wall opened up to 1mm. The crack at 400mm above the base of the wall grew diagonally to a length of 800mm and opened to a width of 0.5mm. When the lateral load was removed from the actuator, the residual drift was noticeable, however, all cracks recovered and closed.

During the third pull cycle to 25mm (1.0% drift), the previous cracks at the base, 150mm, 200mm, 300mm, 450mm, 550mm, 650mm, 850mm, 950mm, and 1100mm grew in length and crack width. One new crack formed at 1250mm above the base. In parallel with the final push cycle to 25mm (1.0% drift), no new cracks formed in following cycles, only existing cracks further developed and began to merge. Along the base, the crack grew to 500mm in length and opened to a width of 1.5mm. The crack at 150mm above the base opened to a width of 0.60mm. The crack at 200mm above the base opened to a width of 1.5mm. The crack at 300mm above the base opened to 0.35mm. There was vertical cracking between 200mm and 300mm above the

wall. Crushing also surfaced on the compression side of the wall for this cycle. When the lateral force was removed, the wall had visible residual displacement and not all the cracks recovered and closed. At the base, the crack remained 1.0mm wide, at 150mm above the base the crack remained 0.20mm wide, and at 200mm above the base the crack remained 0.35mm wide.

During the second push cycle to 37.5mm (1.5% drift) the cracks being focused on at the base, 100mm, 250mm, 300mm, 400mm, and 550mm all grew in width. For the remainder of the test, these cracks didn't experience more crack lengthening just additional crack opening. More spalling occurred on the tension and compression side during this cycle. The crack at the base remained the same width. The crack at 250mm above the base opened significantly to 1.0mm. The crack at 300mm above the base opened to 1.5mm wide. The crack at 400mm above the base opened to 0.80mm wide. Significant crack opening occurred in the bottom quarter of the wall. Residual drift was noticeable, and the cracks identified did not recover fully and remained open. When the lateral force was removed, the crack at the base remained 2.0mm wide, at 250mm the crack remained 0.40mm wide, at 300mm the crack remained 0.80mm wide, and at 400mm the crack remained 0.15mm wide.

During the second pull cycle to 37.5mm (1.5% drift), the previous cracks at the base, 150mm, 200mm, 300mm, 450mm, and 550mm grew in crack width. As stated above for the remainder of the test, these cracks didn't experience more crack lengthening just additional crack opening. At the base the crack opened to 2.5mm. At 150mm above the base the crack opened to 1.5mm. At 200mm above the base the crack opened to 2.5mm. At 300mm above the base the crack opened to 0.80mm. The crack at 450mm above the base opened to 0.60mm. When the lateral force was removed from the actuator all of the following cracks remained open, with slight crack width recovery. At the base the crack remained at 2.5mm, at 150mm the crack width was 1.0mm, at 200mm the crack width was 1.5mm, at 300mm the crack width was 0.15mm, and at 450mm the crack width was 0.15mm.

During the second push cycle to 50mm (2.0% drift) the cracks being focused on at the base, 100mm, 250mm, 300mm, 400mm, and 550mm all grew in width. More spalling occurred though the longitudinal boundary reinforcement was not entirely exposed during these cycles at either end of the wall. At the base a significant piece of concrete spalled leaving a 25mm open

gap at the toe. The crack at 250mm above the base continued to widen to 1.5mm. The crack at 300mm above the base opened to 2.0mm wide. The crack at 400mm above the base remained the same width. The crack at 550mm above the base of the wall increased to 1.5mm. Cracks above 550mm from the base of the wall recovered, however, the cracks below remained open. When the lateral force was removed, the crack at the base remained 20mm wide, at 250mm the crack remained 0.60mm wide, at 300mm the crack remained 0.50mm wide, at 400mm the crack remained 0.30mm wide, and at 550mm the crack remained 0.3mm wide. Compared to the residual crack width from 37.5mm (1.5% drift), the wider openings at 400mm and 550mm likely allowed the crack at 300mm to recover more than it previously did.

During the second pull cycle to 50mm (2.0% drift) the previous cracks at the base, 150mm, 200mm, 300mm, 450mm, and 550mm grew in crack width. From the base up to 300mm a significantly large portion of cover spalled leaving a 65mm tall by 5mm - 10mm deep gap exposed. The existing crack at the base could be seen through the spalled section and was 3.0mm wide. The spalling occurred into the crack at 150mm above the base and this crack remained at 1.5mm wide. At 200mm above the base the crack opened to 3.0mm. At 300mm above the base the crack opened to 1.5mm. The crack at 450mm above the base opened to 0.80mm. When the lateral force was removed from the actuator noticeable damage was visible and additional areas susceptible to spalling became apparent. All of the following cracks remained open and had minimal closing. At the base the spalled section remained at 65mm high, at 150mm the crack width was 1.5mm, at 200mm the crack width was 2.5mm, at 300mm the crack width was 1.0mm, and at 450mm the crack width was 0.40mm.

Significant spalling was experienced when increasing the displacement from 50mm (2.0% drift) to 62.5mm (2.5% drift). This spalling removed the entire cover of the boundary regions on the front and back of the wall. This exposed all of the boundary longitudinal reinforcement at the base and the transverse reinforcement up to 200mm from the base. The exposed longitudinal bars were buckled during this stage of the test, and this can be attributed to a combination of post-yielding, high compressive loading, and fatigue from the repetition of the load cycles. As the test continued, the bars on the tension sides of the wall would straighten and then elongate

further until they ruptured. On the compression side, the bars would compress and buckle. The direction of buckling followed the direction of lateral loading.

During the final push and pull cycles to 62.5mm (2.5% drift), two boundary reinforcement bars ruptured. During the second push cycle, the bar on the tension side, on the back of the wall, ruptured. On the following pull cycle, the bar on the opposite tension side, on the back of the wall, also ruptured. This was deemed to be the end of the test as the wall had failed. The test continued to capture the post-failure response until the remaining boundary bars ruptured. During the following first push cycle to 75mm (3.0% drift) the front side outer most boundary bar on the tension side ruptured. On the first pull cycle following this, the front outer most boundary bar on the opposite tension face also ruptured. This was the point where all exterior boundary reinforcement had ruptured. The internal boundary reinforced buckled, however, only one bar ruptured before the test was terminated. During the first cycle to 87.5mm (3.5% drift) the interior boundary bar on the back side of the wall ruptured. The test was halted as the threaded rods in the top beam came into contact with the vertical actuator. This was intended to prevent any damage to the actuator.

4.4.3 Lateral Displacements

Four cable potentiometers at different locations along the height of the wall were used to measure lateral displacements. The cable potentiometers were located at the middle of the cap beam, CP1 (2495mm), just below the cap beam, CP2 (2200mm), the midpoint between the actuator and base of the wall, CP3 (1247.5mm), and CP4, 100mm above the base of the wall. This section focuses on the displacements recorded by cable potentiometers CP2, CP3, and CP4. The lateral load-displacement hysteretic responses obtained from CP2 and CP3 are similar in shape to the response obtained from CP1 (see Figure 4.20 and Figure 4.21). The difference was the smaller displacements from CP2 and CP3 as they were located at lower heights than CP1. CP4, however, had a notably different hysteretic response as it recorded the sliding displacements of the base of the wall (see Figure 4.22).

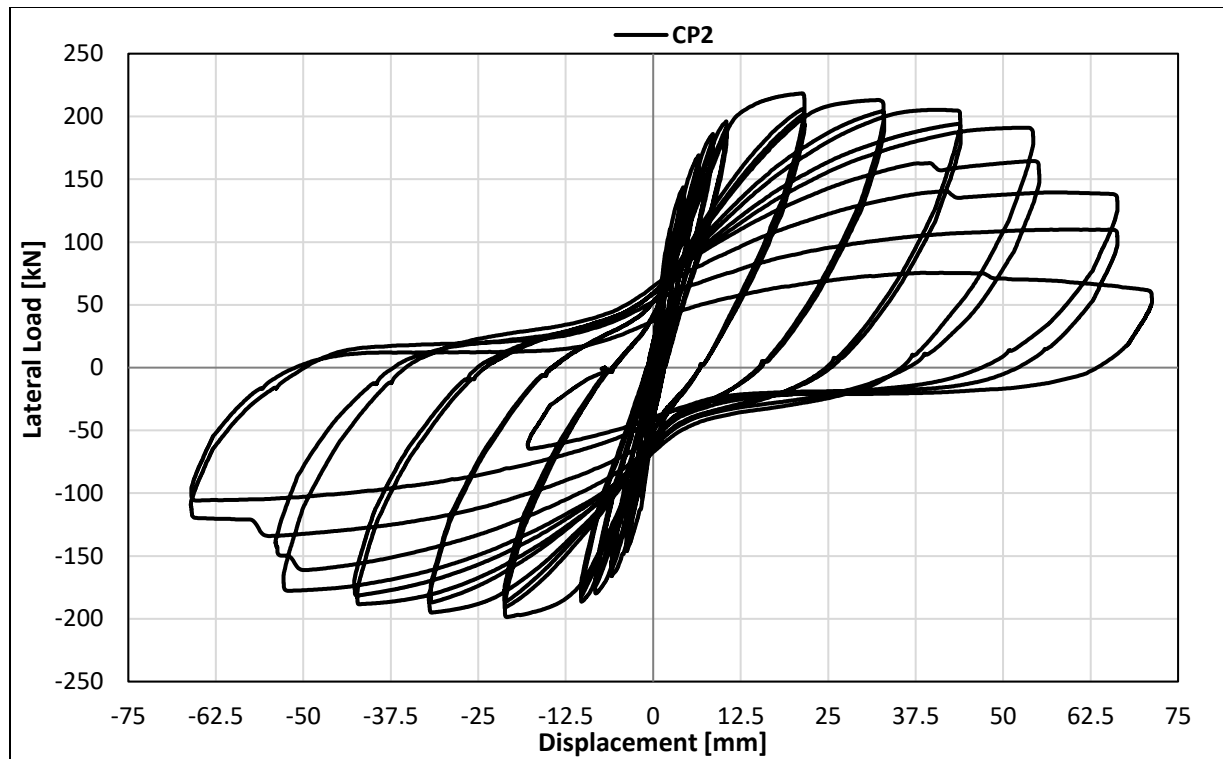


Figure 4.20: Load-Displacement Response of CP2

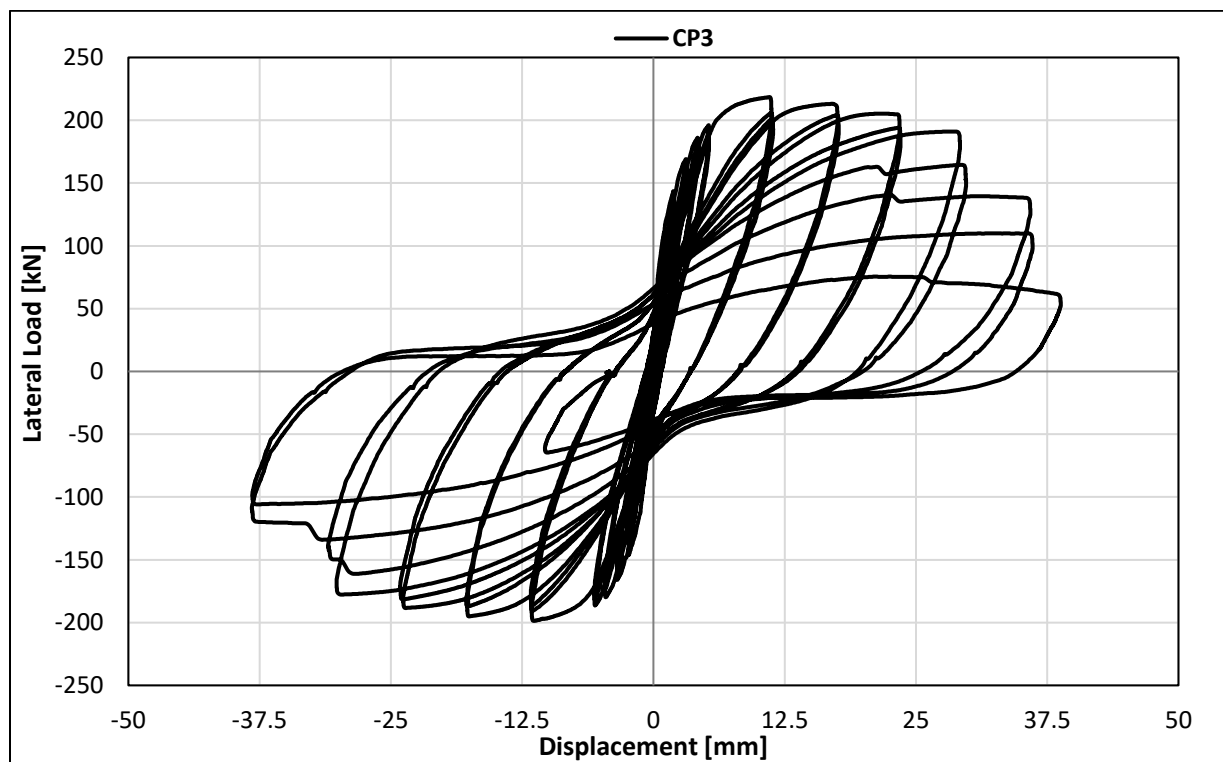


Figure 4.21: Load-Displacement Response of CP3

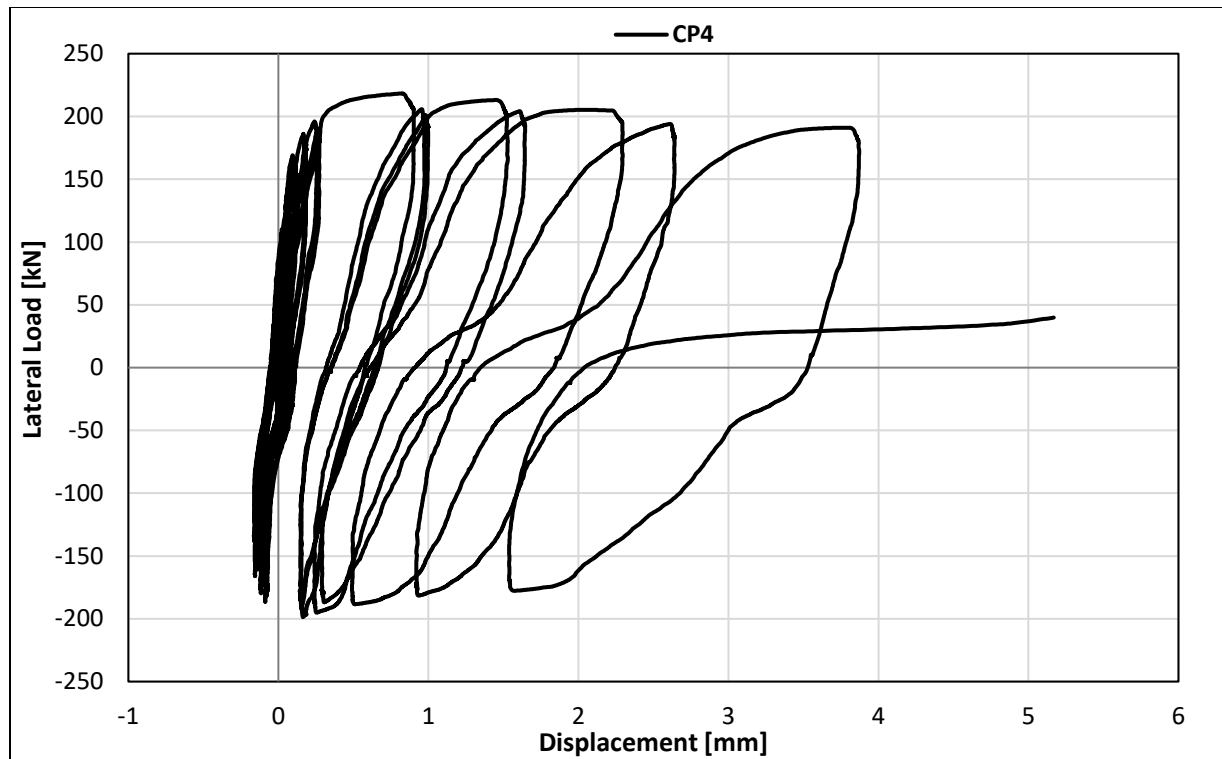


Figure 4.22: Load-Displacement Response of CP4

A significant component of the displacements recorded by CP4 is the result of concrete crushing at the location of the screw that secured the wire for this potentiometer. CP4 pulled out of the wall during the second cycle of 62.5mm (2.5% drift). Therefore, in Table 4.5 and Table 4.6, the displacements are denoted by N/A. There was a significant increase in the displacement values, however, this was due to CP4 being pulled out by the crushed concrete. This was not representative of only sliding of the wall and was neglected. The values in Table 4.5 are plotted in Figure 4.23. The response indicates the curvature through the varied displacements recorded by the CPs that corresponded to the positive and negative cycles for 62.5mm.

The range of sliding displacement at the base of the wall was between +/- 3.5mm up to the end of the first 2.5% drift cycle. When CP4 pulled out and it was no longer possible to determine the sliding displacements. The percentage of displacement of the wall for both cycles of 62.5mm (2.5% drift) is listed in Table 4.6. Just below the cap beam, the wall displaced an average of 85% of the maximum displacement. At the mid-height of the wall, it displaced an

average of 48% of the maximum displacement. After the first cycle, the base of the wall slid an average of 4% of the maximum displacement.

Table 4.5: Lateral Displacements from CPs at 2.5% Drift

	Drift of 2.5%	Force	CP1	CP2	CP3	CP4
		[kN]	[mm]	[mm]	[mm]	[mm]
1st cycle	62.5 +	182.2	61.8	-54.3	29.2	3.9
	62.5 -	-169.7	-62.6	52.8	-30.2	1.5
2nd cycle	62.5 +	156.0	62.7	-55.2	29.8	7.4
	62.5 -	-143.1	-63.8	53.9	-30.9	N/A

Table 4.6: Percentage of Drift Compared to CP1

	Drift of 2.5%	CP1	CP2	CP3	CP4
		[%]	[%]	[%]	[%]
1st cycle	62.5 +	100.0	87.9	47.2	6.3
	62.5 -	100.0	84.4	48.2	2.5
2nd cycle	62.5 +	100.0	88.0	47.5	N/A
	62.5 -	100.0	84.5	48.5	N/A

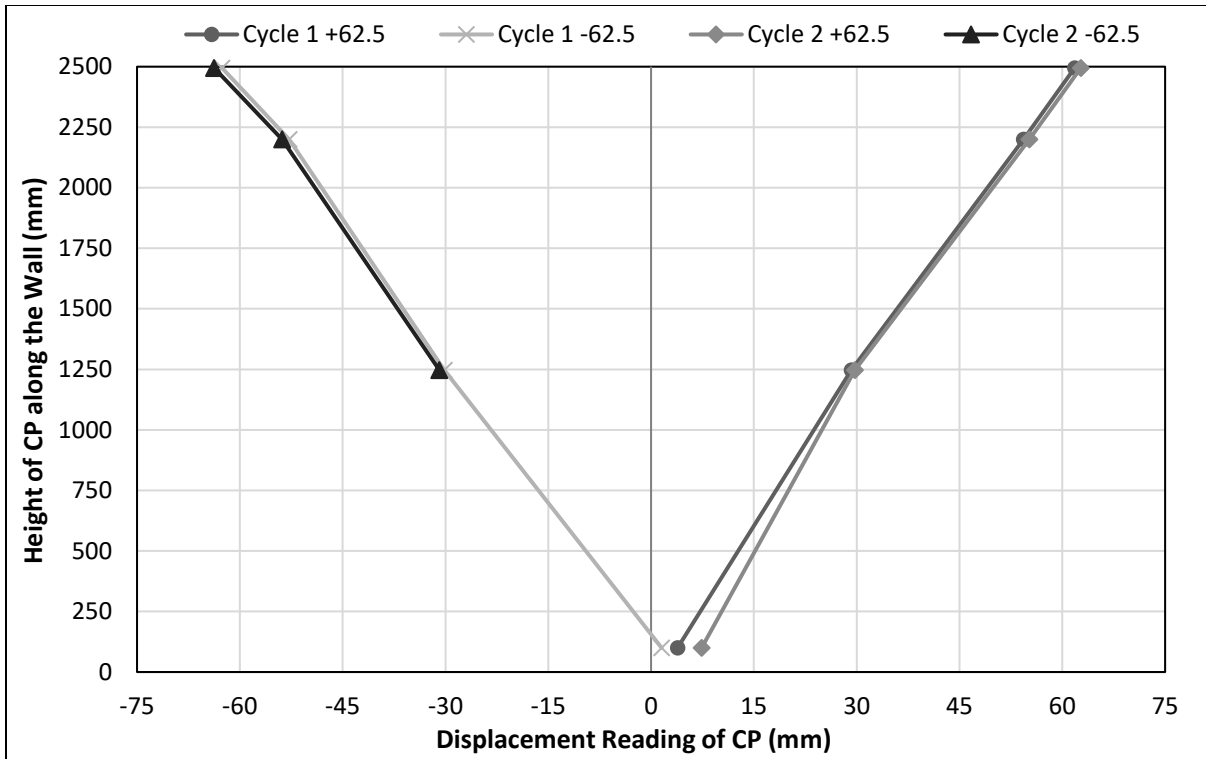


Figure 4.23: Displacement Readings Along Height of the Wall Based on Table 4.5

4.4.4 Vertical Displacements

Vertical displacements were monitored along the sides and back of the wall.

The pair of cable potentiometers on the sides of the wall (CP5 and CP6) measured vertical displacements. CP5 and CP6 were located at the center of the 150mm thickness of the wall and were 100mm above the base of the wall. CP5 and CP6 recorded slip of the wall relative to the foundation, buckling, and elongation that occurred just above the base of the wall. Figure 4.24 and Figure 4.25 provide the responses.

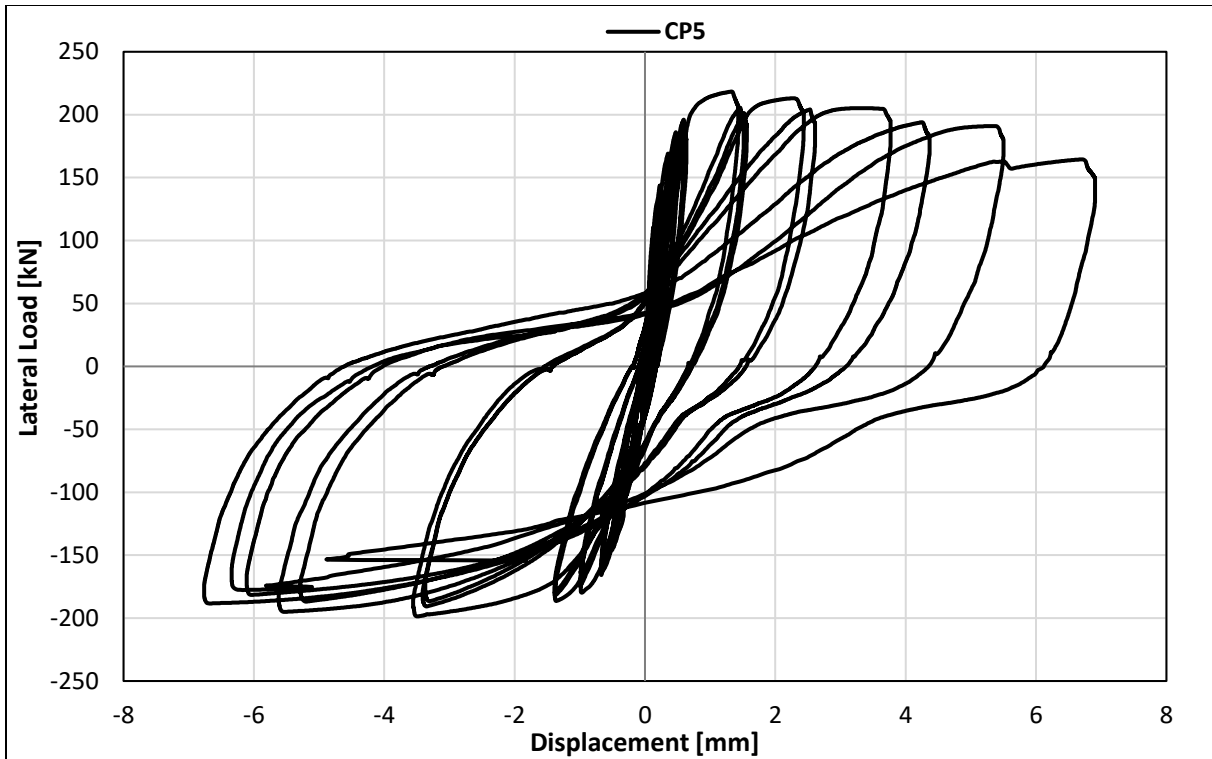


Figure 4.24: Load-Displacement Response of CP5

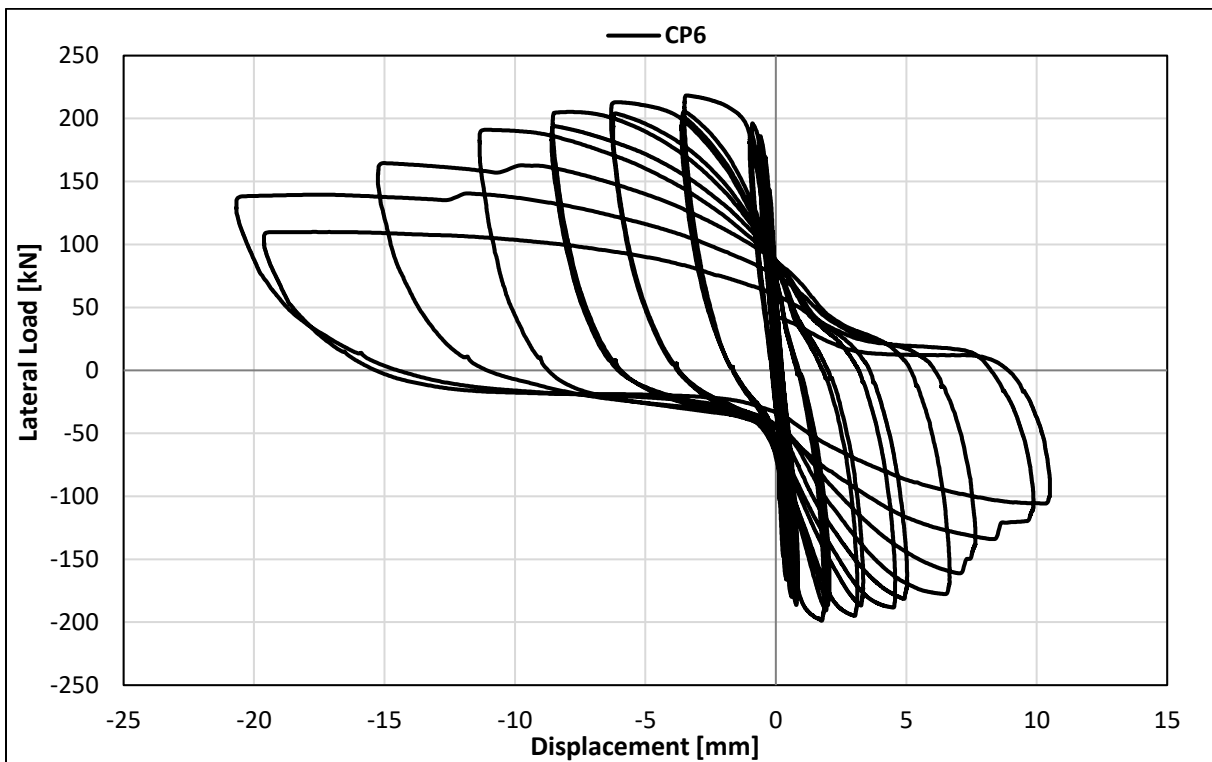


Figure 4.25: Load-Displacement Response of CP6

In addition, on the sides of the wall two cable potentiometers (CP7 and CP8) were secured to the foundation and attached to the bottom of the cap-beam to measure the vertical displacement of the entire wall height during testing. CP7 and CP8 were attached at the ends of the cap beam, 90mm from the edges of the wall, and were 75mm from the middle of the wall. Due to localization of cable potentiometers in these areas, CP7 and CP8 were placed at the same distance away from the center of both sides of the wall to ensure readings were recorded within the same plane. Figure 4.26 and Figure 4.27 illustrated the responses.

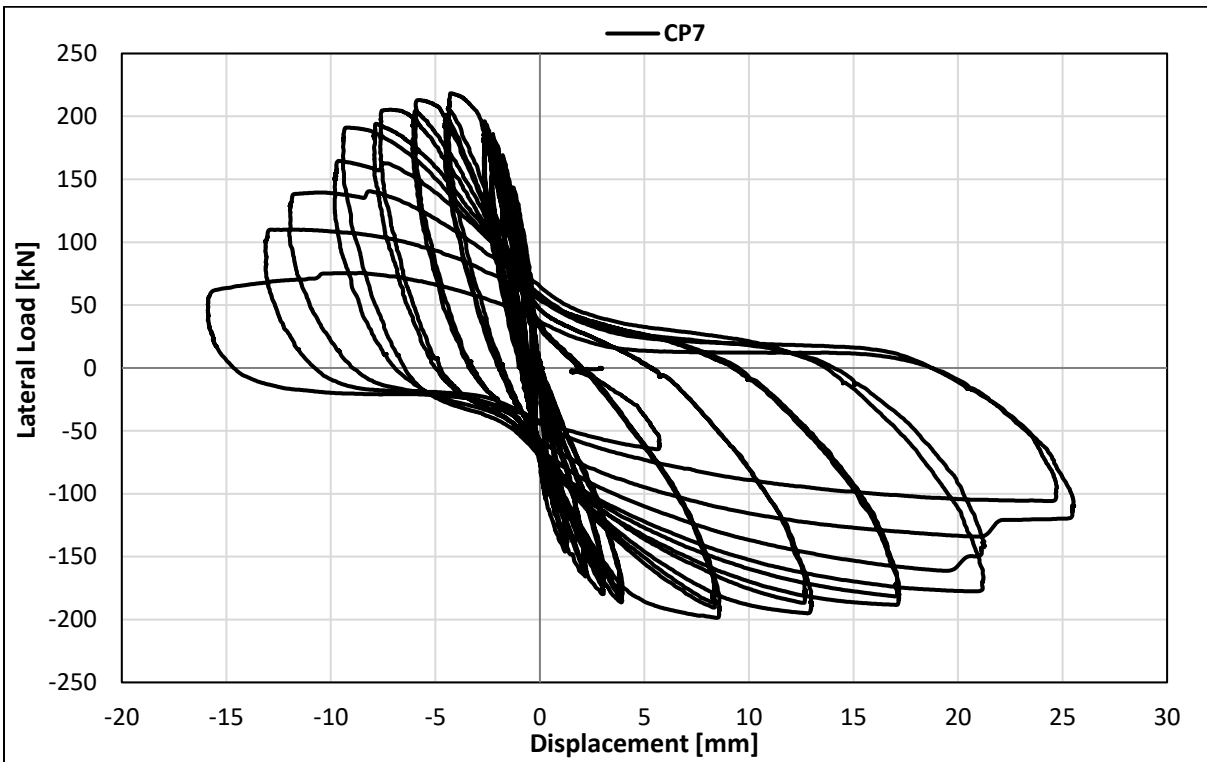


Figure 4.26: Load-Displacement Response of CP7

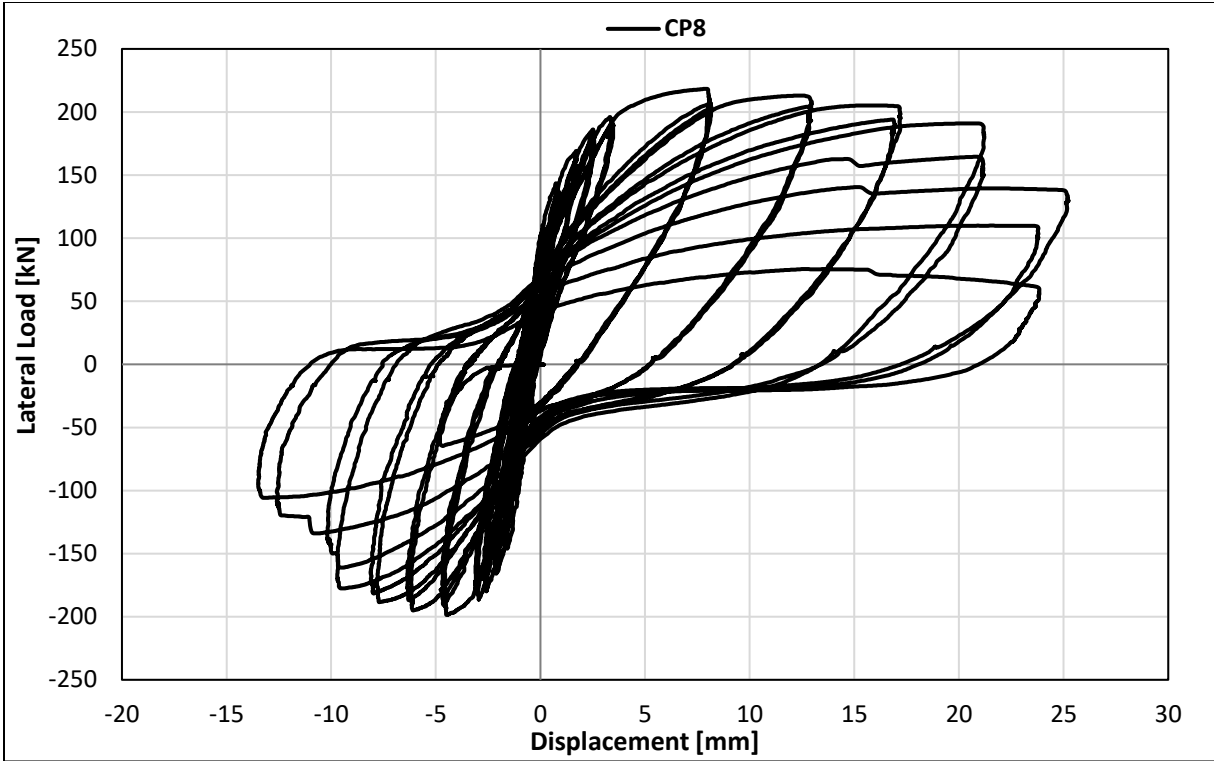


Figure 4.27: Load-Displacement Response of CP8

On the back of the wall, four cable potentiometers attached within the plastic hinge region were used to monitor vertical and diagonal displacements. Two cable potentiometers (CP9 and CP10) monitored vertical displacements about an 800mm x 850mm area. This area was enlarged compared to the testes by Morcos [3] and Soto [4] to avoid contacting any longitudinal or transverse reinforcement. The responses are shown in Figure 4.28 and Figure 4.29. The two other cable potentiometers within this area were CP11 and CP12 and measured diagonal displacements within the same area. These four readings were used to determine the shear strain in the plastic hinge.

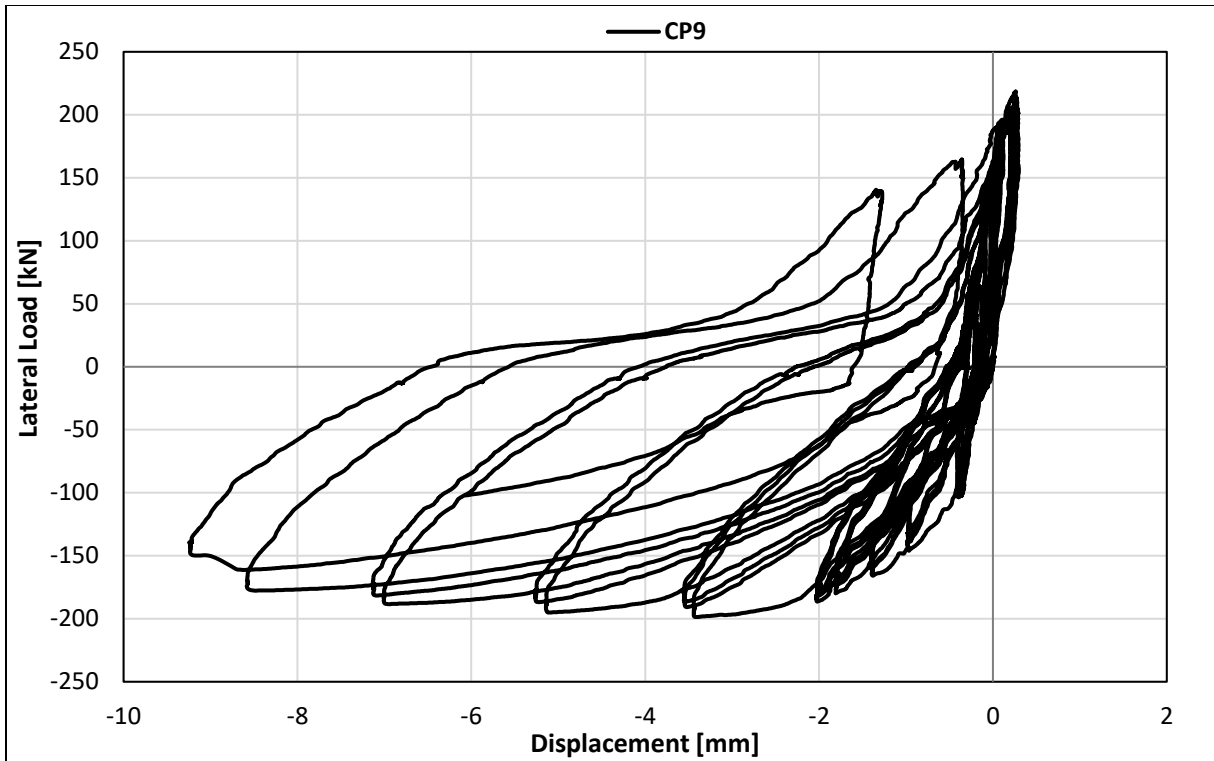


Figure 4.28: Load-Displacement Response of CP9

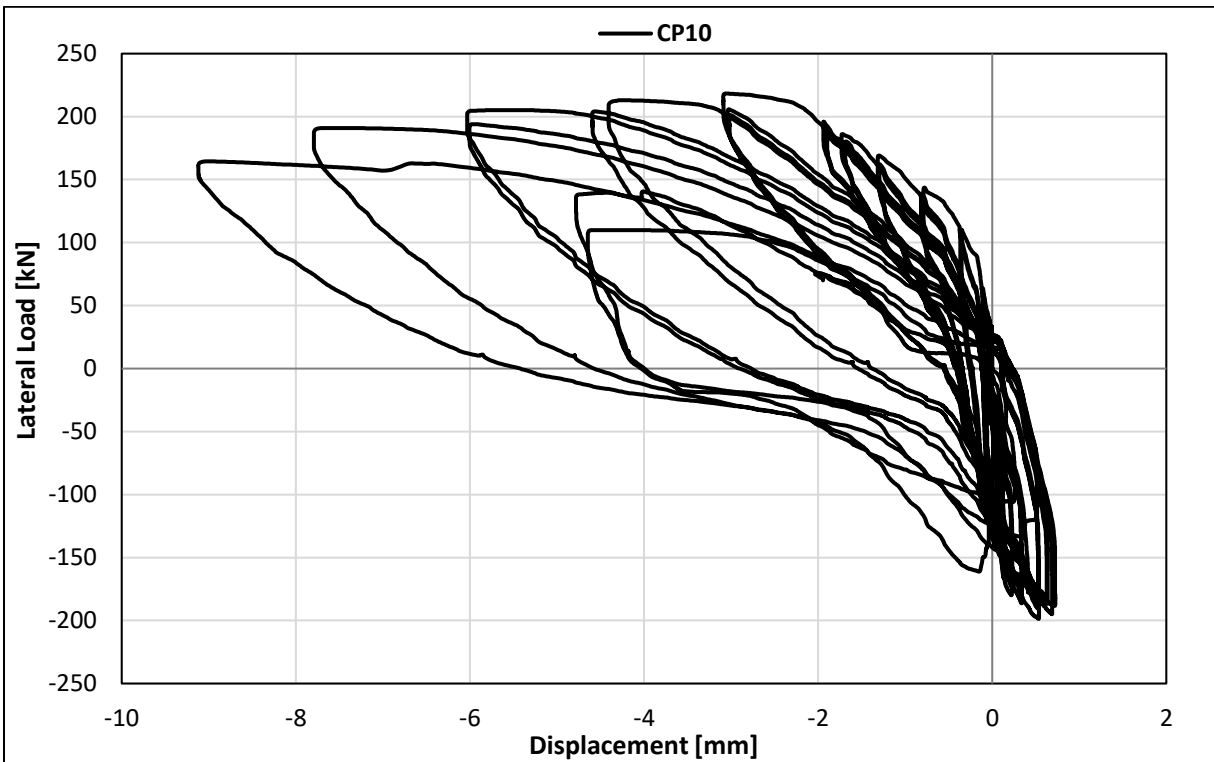


Figure 4.29: Load-Displacement Response of CP10

The displacements for the positive and negative cycles were nearly identical for all pairs of vertical instrumentation. Up to lateral displacements of 12.5mm (0.50% drift), the responses remained linear with minimal residual displacement. For CP5 and CP6, up until 62.5mm (2.5% drift), they responses were identical, however, due to concrete crushing and spalling in the boundary region, CP5 was disconnected to prevent damage. For CP7 and CP8, the results were similar in both the positive and negative directions. The same was also true for CP9 and CP10.

The results demonstrate that displacements on the tension sides during push and pull cycles resulted in larger displacements than the compressive faces. There was minimal vertical displacement on the compressive side until after the onset of concrete crushing reinforcement buckling. The presence of axial load is visible by the noticeable pinching in the responses. This pinching became more evident with each increasing lateral displacement.

4.4.5 Out-of-Plane Displacements

Out-of-plane displacements were monitored by one cable potentiometer located at the mid-height of the distance between the base of the wall and centre of the cap beam, an elevation of 1247.5mm. This cable potentiometer (CP15) was located at the same height as CP3, which measured the lateral displacement at the same height. CP15 was mounted to a 2 in x4 in timber which was bolted onto the instrumentation frame. Similar to all the other instruments attached to this frame, the readings were not affected by sliding or rocking of the foundation. CP15 (see Figure 4.30) illustrates that as the wall was subjected to increasing lateral displacements, the wall experienced greater out-of-plane displacements. A combination of reasons could have caused this behaviour; however, the most likely cause was that the cap beam was not leveled correctly during casting. The weight of concrete on the cap beam resulted in slightly shifted the formwork during casting. Due to this, the wall was leveled correctly but the cap beam was not and titled in one direction. Thus, upon applying the axial load to the cap beam, there was an initial out-of-plane displacement, which is evident in the response when the lateral load was initially at zero force. Thereafter, the out-of-plane displacement at approximately mid-height of the wall increased as the lateral loading increased. It is probable that this was the result of twisting of the wall during the lateral loading.

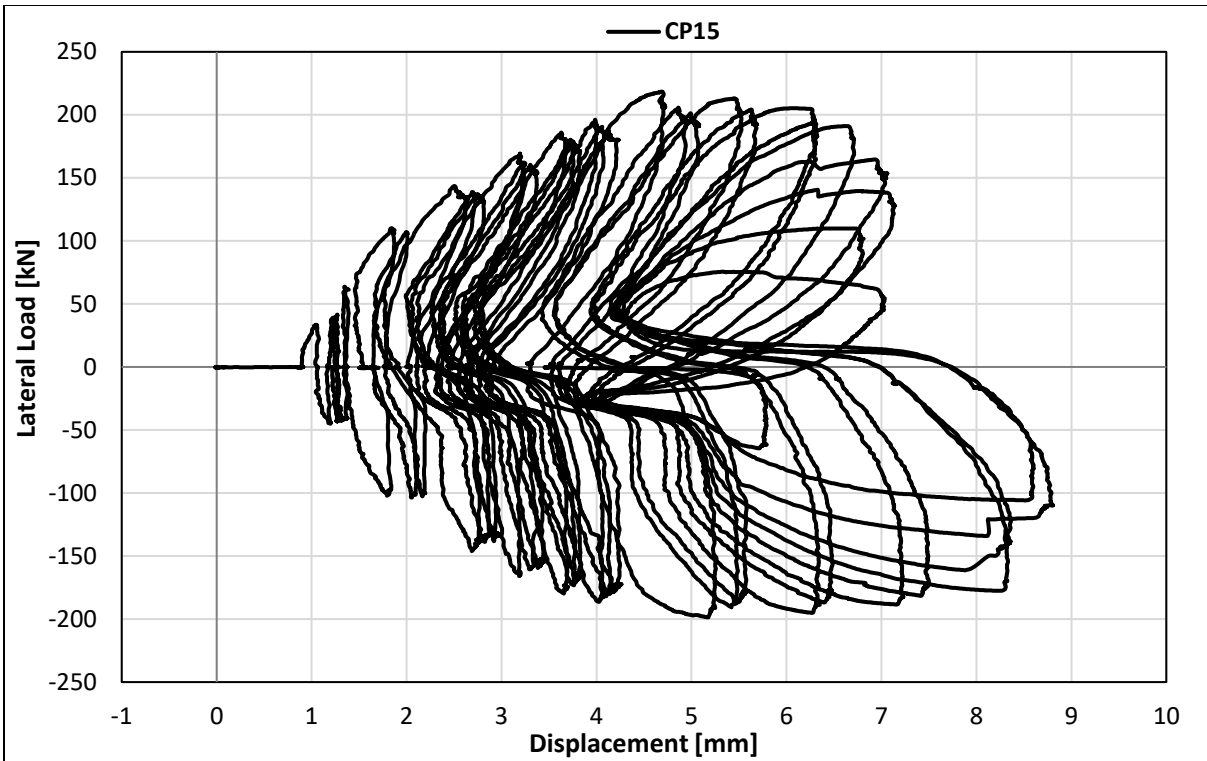


Figure 4.30: Load-Displacement Response of CP15

4.4.6 Rigid Body Displacements

Rigid body displacements refer to displacements that arise from the entire wall displacing including the foundation due to base sliding or base rocking. Two cable potentiometers (CP13 and CP14) were attached to the rigid testing frame. The lower of the two, CP13, was attached to the base foundation and measured sliding of the foundation block. The higher one, CP14, was connected at the same height as the top threaded rods in the cap beam. CP14 captured the lateral displacement of the wall including base sliding and rocking. The results are shown below in Figure 4.31 and Figure 4.32. Two linear potentiometers (LP1 and LP2) were attached to near the corners at the base of the foundation to measure uplift or rocking. These results are shown in Figure 4.33 and Figure 4.34.

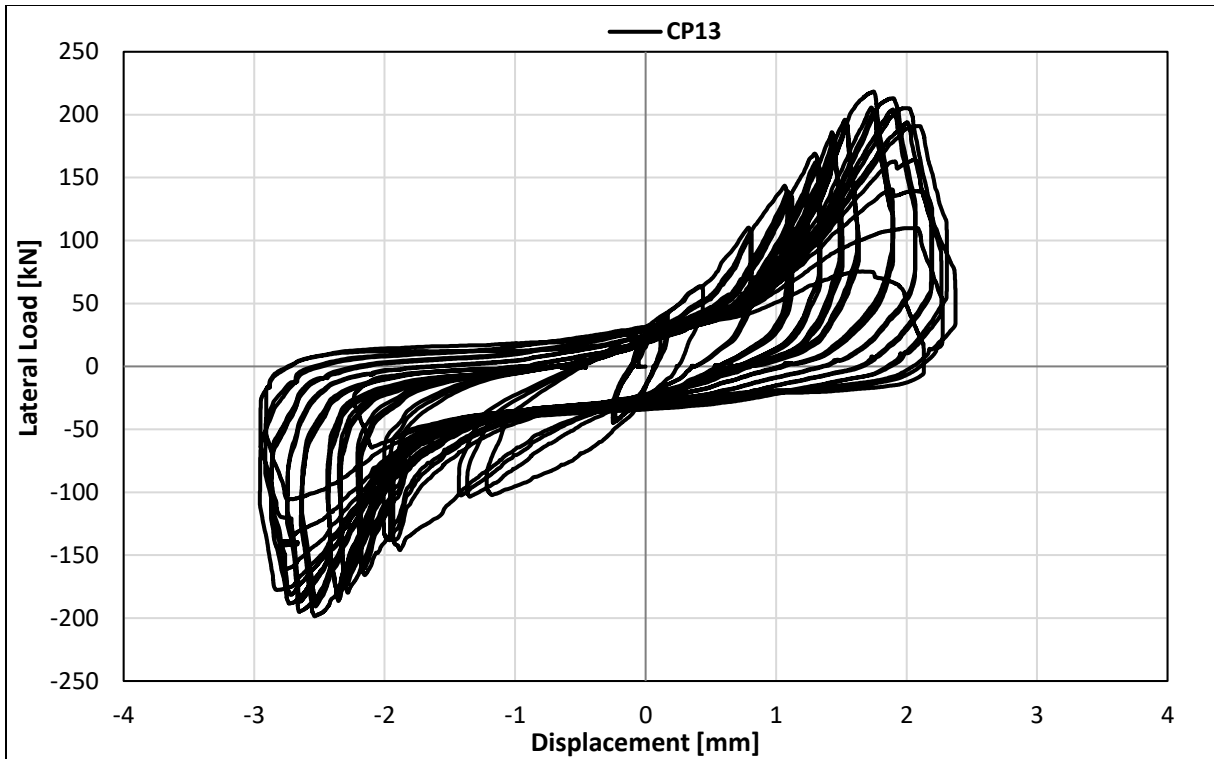


Figure 4.31: Load-Displacement Response of CP13

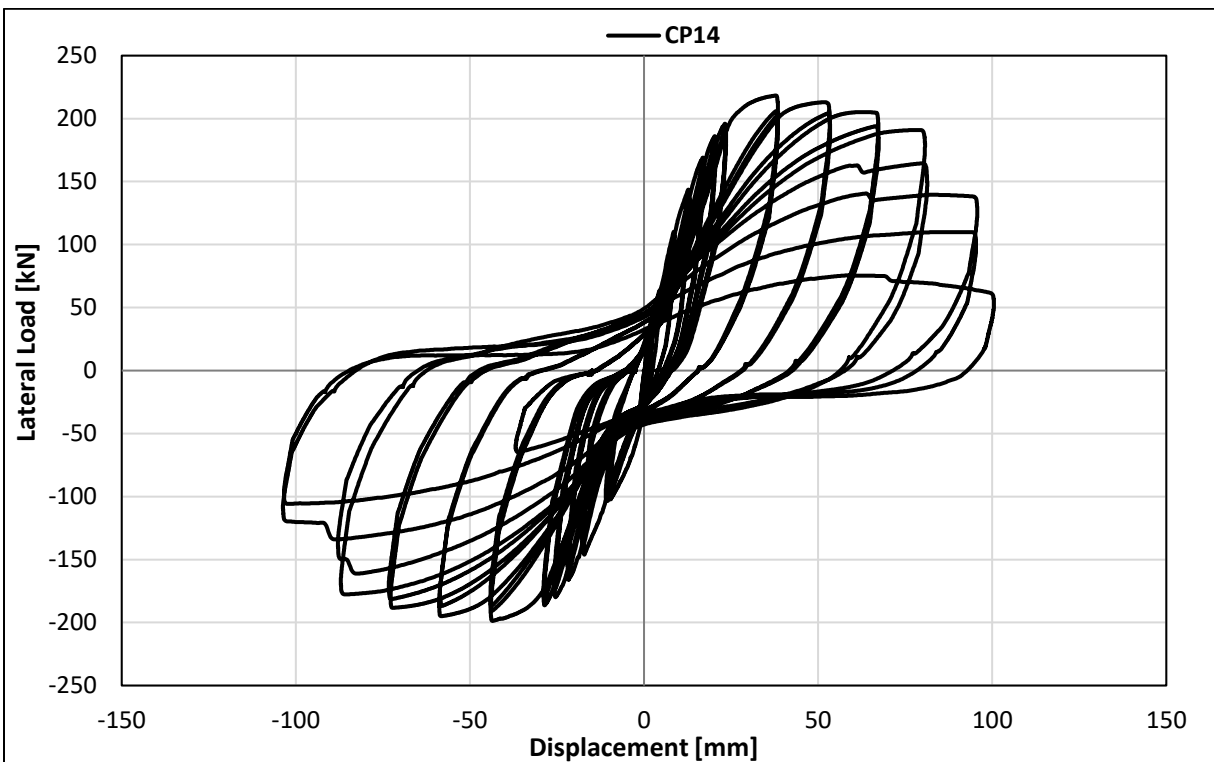


Figure 4.32: Load-Displacement Response of CP14

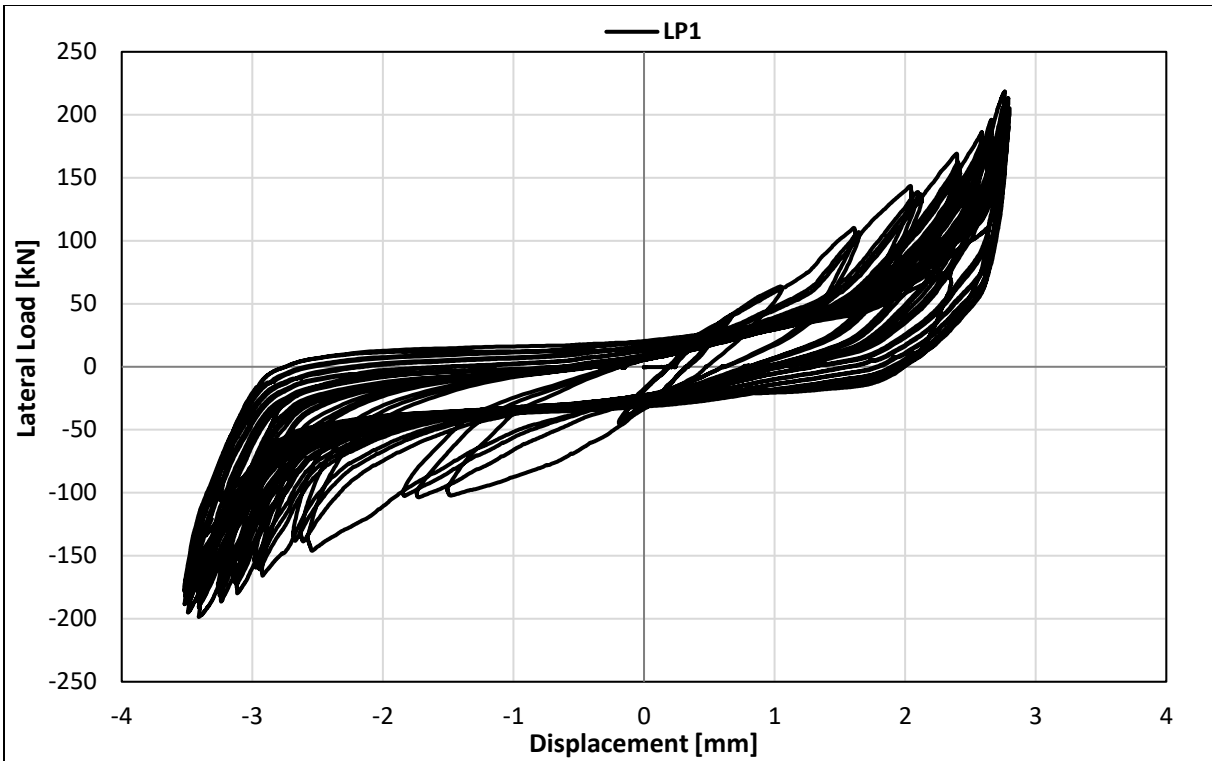


Figure 4.33: Load-Displacement Response of LP1

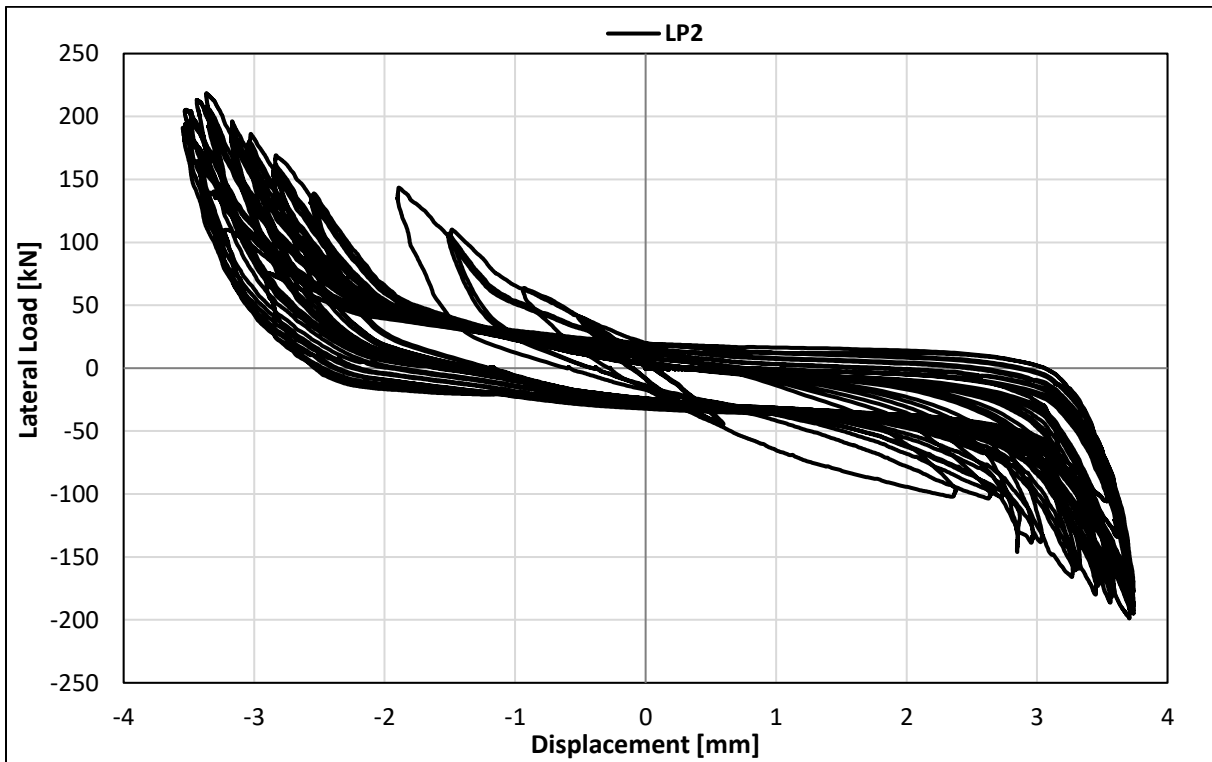


Figure 4.34: Load-Displacement Response of LP2

The results illustrate that the foundation experienced some minor sliding during testing, even after the rods were retightened. The foundation also developed a hairline crack near the middle of the foundation and through the middle anchor rods. This could be the result of possible irregularities from casting of the wall and it being level with the strong floor at the test location. There was a slight gap beneath one of the corners of the foundation which would indicate that there was not full contact of the foundation with the floor. This was observed even after the application of axial loading. The maximum sliding displacement for the push cycles was 2.5mm and the maximum sliding for the negative cycles was 3mm. The maximum uplift based on LP1 and LP2 was 3.7mm and 3.6mm, respectively. The uplift was not visible during testing however it became present as the lateral displacements increased.

4.4.7 Reinforcement Strains

A total of 33 strain gauges were attached to various reinforcing bars throughout the wall. The strain gauges were concentrated within the boundary regions. The purpose of the strain gauges was to monitor the reinforcement strains within the plastic hinge region. The three locations of the strain gauges were: the boundary regions, the front web region, and three front stirrups. Each boundary region vertical bar was equipped with 3 strain gauges. Only one side of the web had each bar equipped with two strain gauges. Three stirrups (the first, third, and fifth from the base) were equipped with one strain gauge; directly in the middle of the wall. Out of the 33 strain gauges, SG7, SG27, and SG30 were damaged during the casting of the wall. These strain gauges were at heights of 50mm above the base, 300mm above the base, and 650mm above the base. The strain gauges on the vertical bars were positioned at 50mm above the base of the wall, 400mm above the base, and 750mm above the base. SG7 – 9 and SG14 – 16 were located within the web only at heights of 50mm and 400mm above the base. For the exact locations refer to Section 3.5.1.2 and Figure 3.12.

During testing, rupturing of the reinforcement occurred at or just below the bottom strain gauges in the boundary region. The majority of the strain gauges recorded reliable readings up to yielding of the reinforcing bar. The stirrups did not rupture during testing. Based on comparisons of the strain readings from the back and front of the wall, there were some slight variations in strain in the bars located on the back side of the wall.

The reinforcement strains at the performance points of yield, peak, and ultimate are listed in Table 4.7 and Table 4.8. From the material properties of the reinforcing steel, the strain at yielding as determined from the uniaxial tension tests was 0.23%. In both tables, it is evident that at global yielding of the wall, or just before 0.4% lateral drift, the boundary bars were yielding or at the onset of yielding.

Table 4.7: Reinforcement Strain at Performance Points for Positive Cycles

Performance Points	Force [kN]	Strain corresponding to Positive Cycles [%]					
		Exterior Boundary		Interior Boundary		Stirrup	Middle Web
		SG12	SG18	SG13	SG28	SG32	SG15
Yield	185.1	-0.1945	0.3971	-0.0768	-0.1020	0.0428	0.0692
Peak	218.1	-0.0294	0.1290	-0.1272	-0.1171	0.1046	0.2417
Ultimate	182.2	-	0.0176	-	-0.0799	0.1110	0.6861

- strain reading unavailable

Table 4.8: Reinforcement Strain at Performance Points for Negative Cycles

Performance Points	Force [kN]	Strain corresponding to Negative Cycles [%]					
		Exterior Boundary		Interior Boundary		Stirrup	Middle Web
		SG12	SG18	SG13	SG28	SG32	SG15
Yield	-160.9	0.3491	-0.1005	0.1998	0.2178	0.0097	0.0586
Peak	-197.5	0.1040	-0.0119	0.1512	0.4762	0.0762	0.2367
Ultimate	-169.7	-	0.0298	-	0.1156	0.0974	0.5651

- strain reading unavailable

The two exterior boundary gauges, SG12 and SG18, located at the ends of the wall, captured yielding as expected. In Figure 4.35 and Figure 4.36, there is a noticeable pinching in the response due to the axial loading. After yielding there was a significant increase in the strain until the gauges exceeded their range limits.

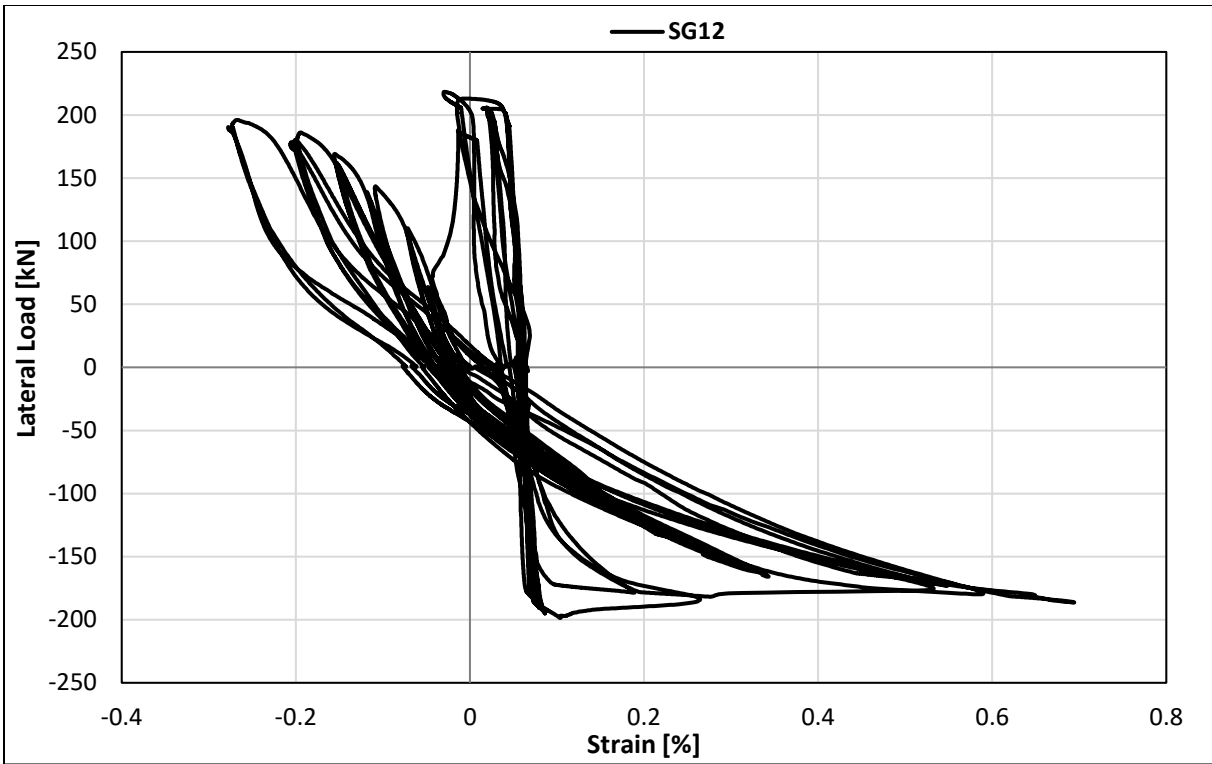


Figure 4.35: Strain Gauge Response for SG12

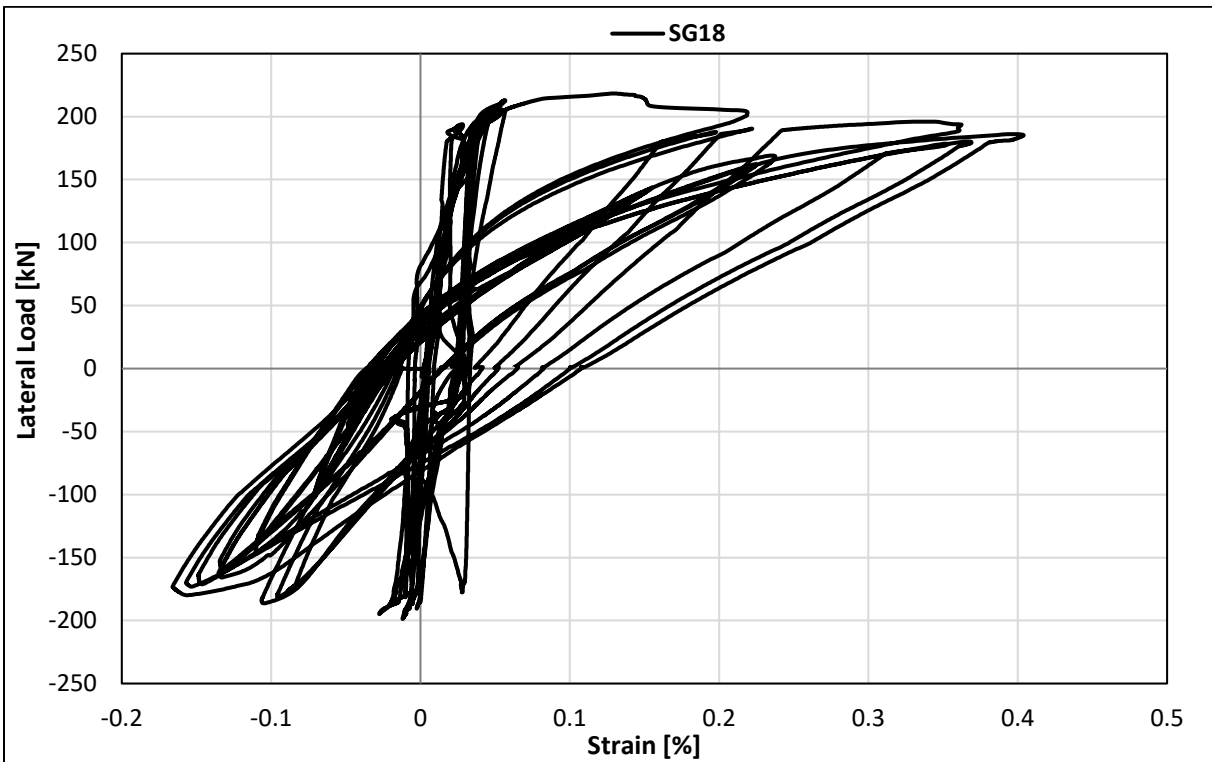


Figure 4.36: Strain Gauge Response for SG18

The strain gauges SG13 and SG28 were located on interior boundary bars at the same height, on the same side of the wall, but on different bars. SG28 was on the front of the wall and SG13 was on the back of the wall. The values listed in Table 4.8 at global yielding of the wall demonstrate that the two strain gauges recorded strains that were close the reinforcement yield strain, but that the two bars did not experience the same level of straining. Figure 4.37 and Figure 4.38 illustrate the differences in the response. Although it is expected, that due to the symmetry of the wall, these gauge readings should be similar. However, the location of cracking relative to the location of the strain gauges, the placement of the reinforcing bars in their proper location, and twisting of the wall are all probable reasons leading to the difference in response.

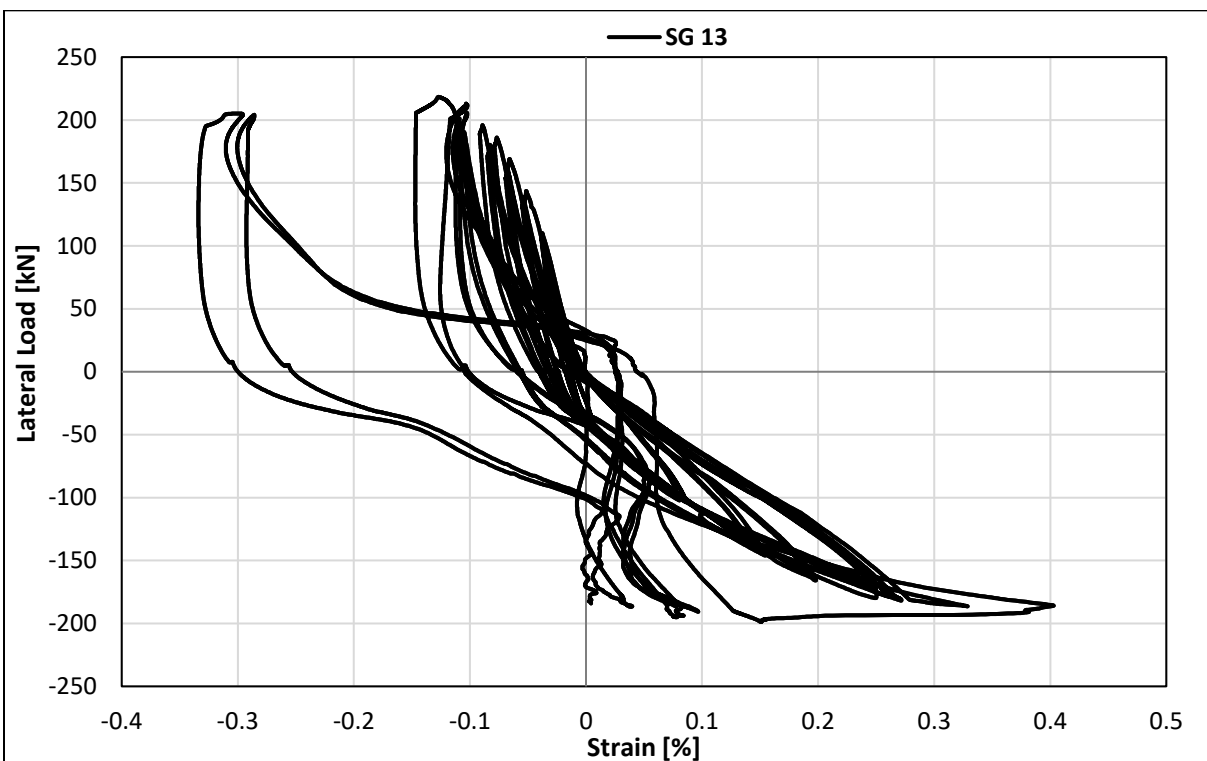


Figure 4.37: Strain Gauge Response for SG13

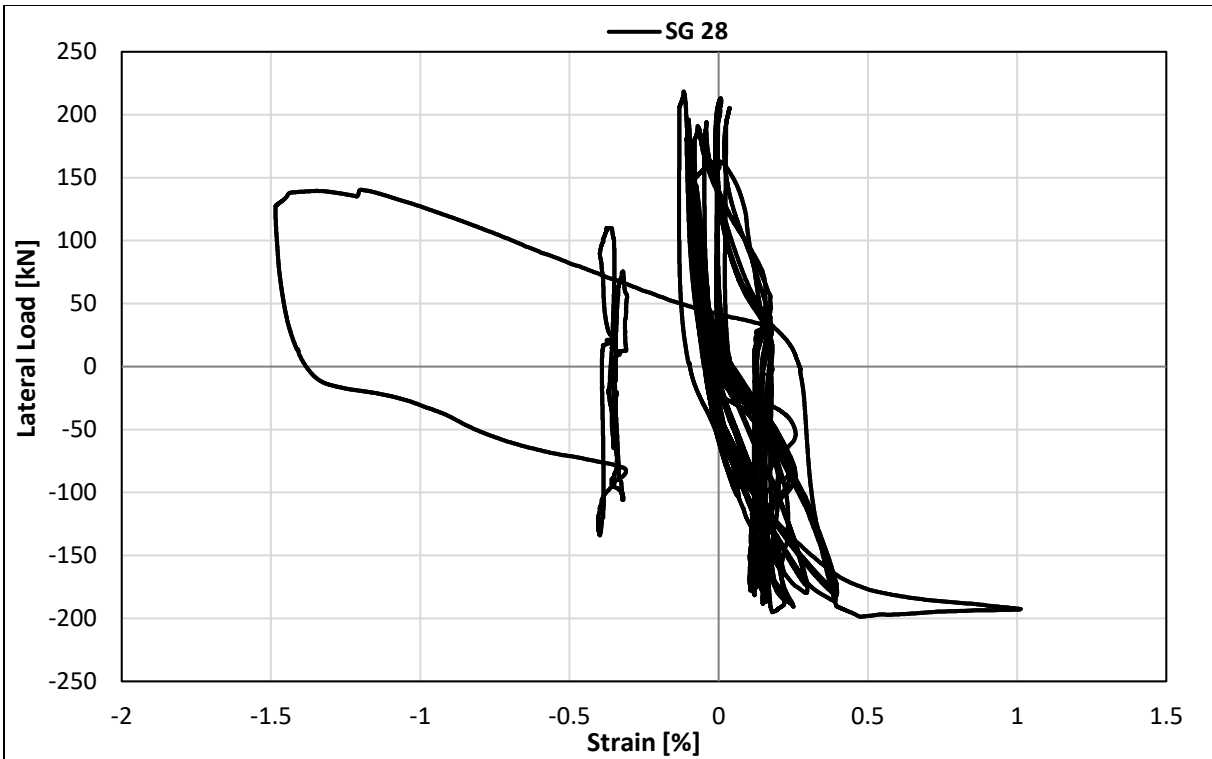


Figure 4.38: Strain Gauge Response for SG28

Strain Gauge SG32 was located on one of the stirrups. Figure 4.39 demonstrates that the bar at this location was not subjected to significant stress. As listed in Table 4.7 and Table 4.8, the stirrup did not yield, nor did it reach half the strain corresponding to yielding.

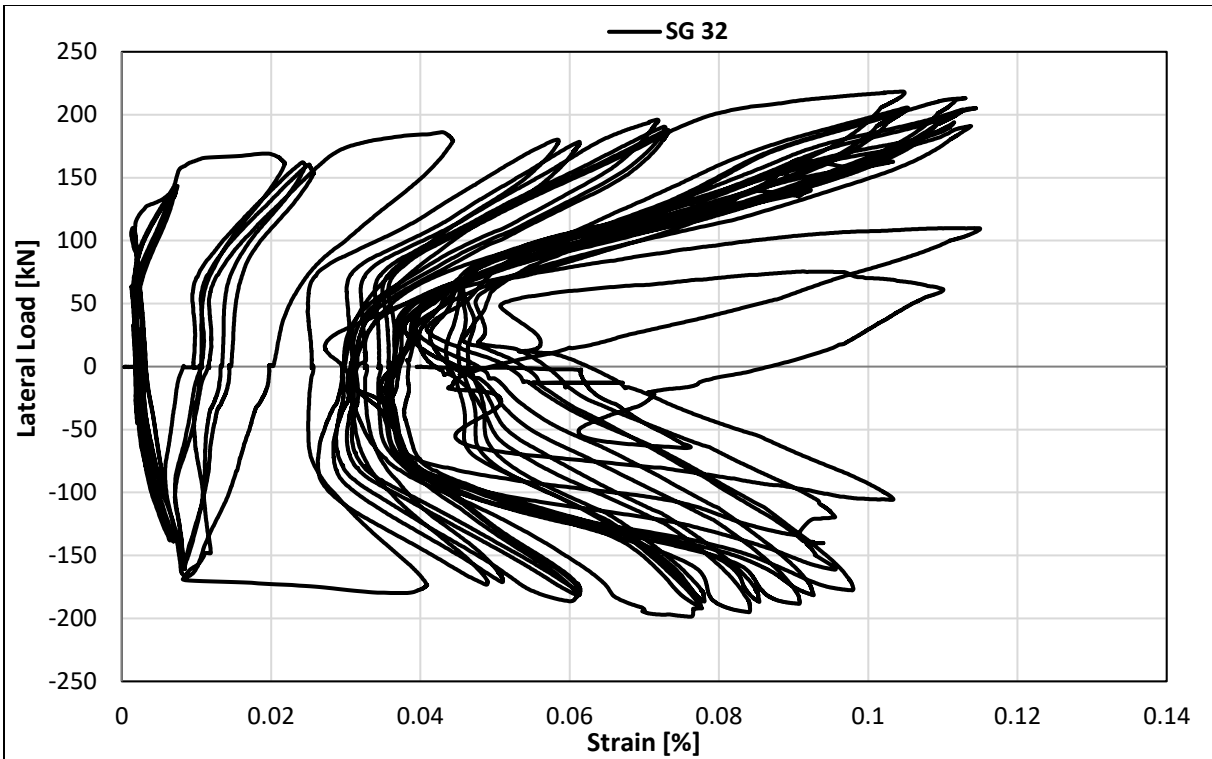


Figure 4.39: Strain Gauge Response for SG32 (Third Stirrup)

The final strain gauge, SG15, was located on the middle web longitudinal bar, 50mm above the base. Figure 4.40 illustrates that this bar experienced tensile strains during positive and negative cycles due to its location. Yielding of the middle bar did not occur until the peak recorded lateral loading at 1.0% drift. This is observed in Figure 4.40, Table 4.7, and Table 4.8, as the strain at the peak for SG15 is larger than 0.23%.

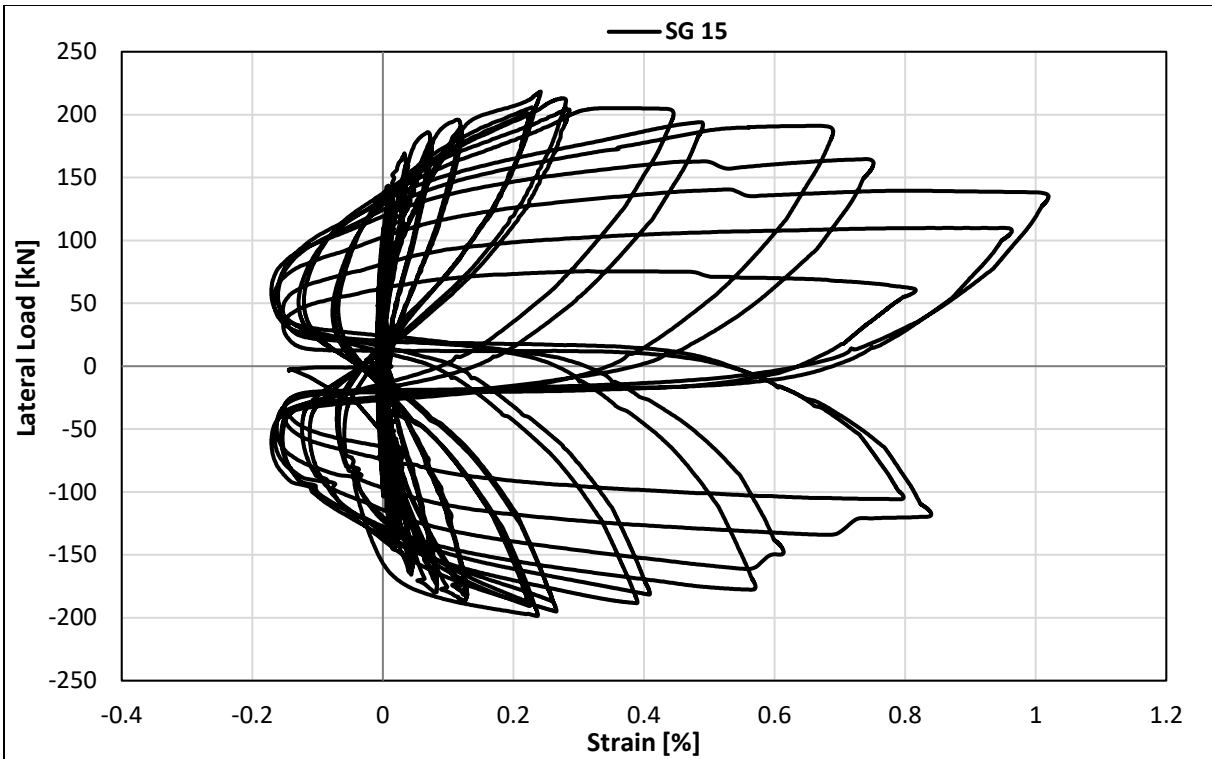


Figure 4.40: Strain Gauge Response for SG15 (Middle Web Bar)

5 Discussion of Results

5.1 Introduction

This section of the thesis discusses the experimental results of the shear wall. This section will examine the findings and compare them with results available in the literature to verify the accuracy of the test. The main addition to this research relative to past research was the application of axial loading, and this will be investigated and compared to previous tests to confirm the recovery effects due to axial loading, crack recovery, and damage control.

5.2 Stiffness and Strength

The observed strength and stiffness of the wall was greater than the previous walls tests by Abdulridha [1], Zaidi [2], Morcos [3], and Soto [4]. This is due to the presence of applied axial loading. Applying an axial load imposed a continuous compressive force on the wall which required more force to displace it. Essentially, axial loading increases stiffness, which is well known and incorporated in the analysis of shear walls subjected to seismic loading.

The repetition from reverse cyclic loading is known to impact the strength of the wall. The repeated cycles cause stiffness and strength degradation. The strength degradation is illustrated in Table 5.1; as the first cycle requires a larger force to reach the desired displacement compared to the second or third cycles. This is the result of the continuation of damage such as crack propagation during the first cycle cracks that are not as well developed as they are during the second or third loading cycles. Due to the repetitions of cycles, fatigue can occur and influence the strength of the materials.

Table 5.1: Maximum Force in the Positive Direction from 25mm to 75mm Displacement

Displacement (mm)	25.00	37.50	50.00	62.50	75.00
	Force (kN)				
Cycle 1	218.39	213.06	205.24	191.01	139.51
Cycle 2	205.77	204.18	194.05	162.84	109.92
Cycle 3	201.26				

At 1.25mm and 2.5mm displacement levels there was no stiffness degradation, as the force required to reach the desired displacements increased for each positive cycle. The last positive cycle for 5mm displacement is where the degradation began. From 5mm to 25mm displacement, the change in force required to attain the same displacement for cycle two was approximately 4% less than the force in the first cycle. In the third cycle, it was approximately 5% less force than what was required for the first cycle. After 25mm, there were only two cycles of imposed loading per displacement level. For the 37.5mm and 50mm displacement levels, the average degradation was 4.5% compared to the force required in the first cycles. It wasn't until the first longitudinal bar ruptured that there was a noticeable change in the force. At 62.5mm, the force required in the second cycle was 15% less than the force in the first cycle. At 75mm, the force required in the second cycle was 21% below the force in the first cycle. Although it was not overly apparent at the start of the test, after reviewing the test data, there is stiffness degradation between 0.2% - 1% drift through all three cycles, and between 1% - 2% after both cycles. There was a noticeable change in force required in the second cycles for 2.5% and 3%, as the stiffness degraded significantly. This can also be attributed to crushing of the concrete and buckling of the reinforcement within the toe and boundary regions. Though concrete provides virtually no tensile resistance, on the compression sides of the wall it provides resistance to the lateral load. Due to the crushing, however, there would be significantly less resistance, which is captured by the reduction in required lateral force and loss of strength. To coincide with the loss of concrete, rebar also buckled. The reinforcement is mainly engaged in tension, however, the buckling aided in increasing the level of damage that the bars sustained. This continued damage aided in fatiguing the bars and ultimately reducing the lateral resistance.

Strength change was also apparent. The maximum force was reached in the first cycle of 1.0% drift. Table 5.2 lists the maximum force required to reach the desired displacement and the change in force compared to the maximum. For 37.5mm, the first cycle required 5.3kN less force than the force needed to reach 25mm. Table 5.3 shows the percentage of the maximum force that is required to attain the different displacements. At 75mm, only 63.9% of the maximum force was required. At 62.5mm, during the second cycle where the first bar ruptured and the wall ultimately failed, there was a 25.1% reduction in force.

Table 5.2: Force Loss per Cycle compared to Maximum of 218.13 kN

	Force loss compared to max				
Strength change %	25.00	37.50	50.00	62.50	75.00
Cycle 1	0	5.3	13.1	27.4	78.9
Cycle 2	12.6	14.2	24.3	55.5	108.5
Cycle 3	17.1				

Table 5.3: Percentage of Force Compared to Maximum Force

	Strength required compared to max				
Strength change %	25.00	37.50	50.00	62.50	75.00
Cycle 1	100	97.6	94.0	87.5	63.9
Cycle 2	94.2	93.5	88.9	74.6	50.3
Cycle 3	92.2				

The secant stiffness as defined by Morcos [3], is the damage state of the wall against the initial condition. Soto [4] defined it as the ratio of peak load of the current cycle to corresponding displacement in the positive direction. The secant stiffness characterizes the nonlinear behaviour of the wall. Essentially, for Morcos [3] and Soto [4] as the displacement increased, the secant stiffness decreased. This is displayed in Figure 5.1 The same was not initially true for SFW, at 0.05%, 0.1%, and 0.2% drift, there was a steady lateral incline instead of the decline that was observed by the shear walls without applied gravity loading. After 0.2% drift, as the lateral drift in the positive direction increased, the secant stiffness decreased. Compared to the work of Morcos [3] and Soto [4], the secant stiffness for this wall was overall higher than the other two walls. This is again a result of the increase lateral load required at each cycle due to the axial loading.

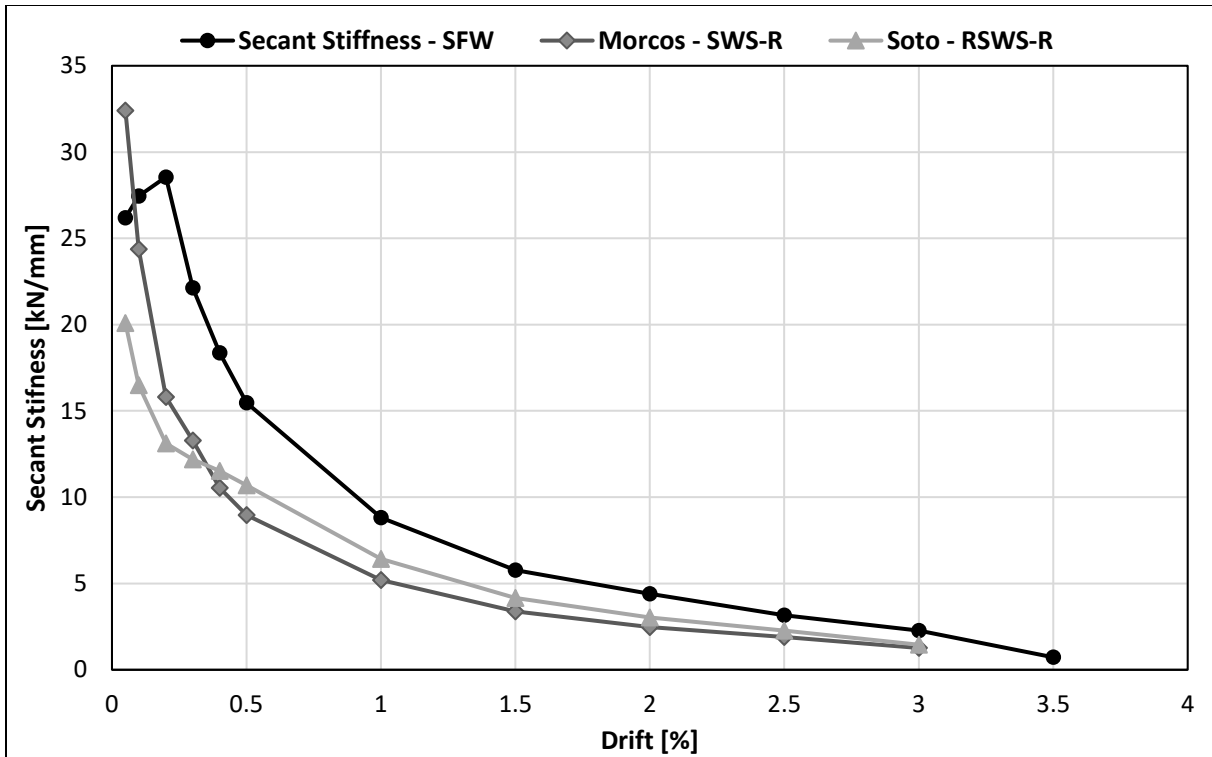


Figure 5.1: Stiffness Degradation [3] [4]

5.3 Ductility and Recovery

Ductility in this experiment was deemed to be the ability of the wall to sustain the lateral load without a significant reduction in lateral resistance. Due to the continuous repetitions, there is a decline in the overall ductility of the wall. The first loading cycle resulted in the larger lateral force resistance, and then each subsequent cycle at the same displacement required a reduction in lateral load to achieve it. The wall experienced a flexural failure where boundary bars ruptured in tension and the concrete crushed and boundary bars buckled in compression after substantial concrete cover loss. This is representative of a ductile response to the loading due to the yielding of the longitudinal reinforcing bars followed by rupture. The wall did not fail prematurely. An example of earlier failure would have been due to a shear sliding failure. Displacement recovery was calculated using Equation 5.1. Here recovery is defined as the ratio between the difference between the maximum displacement of a cycle and the residual displacement after unloading to the maximum displacement.

Equation 5.1

$$Recovery (\%) = \frac{\Delta_{max} - \Delta_{residual}}{\Delta_{max}} * 100$$

The recovery-drift responses for the positive and negative directions of loading are illustrated in Figure 5.2. [Note that the recovery was calculated during the final cycle of each drift level.]

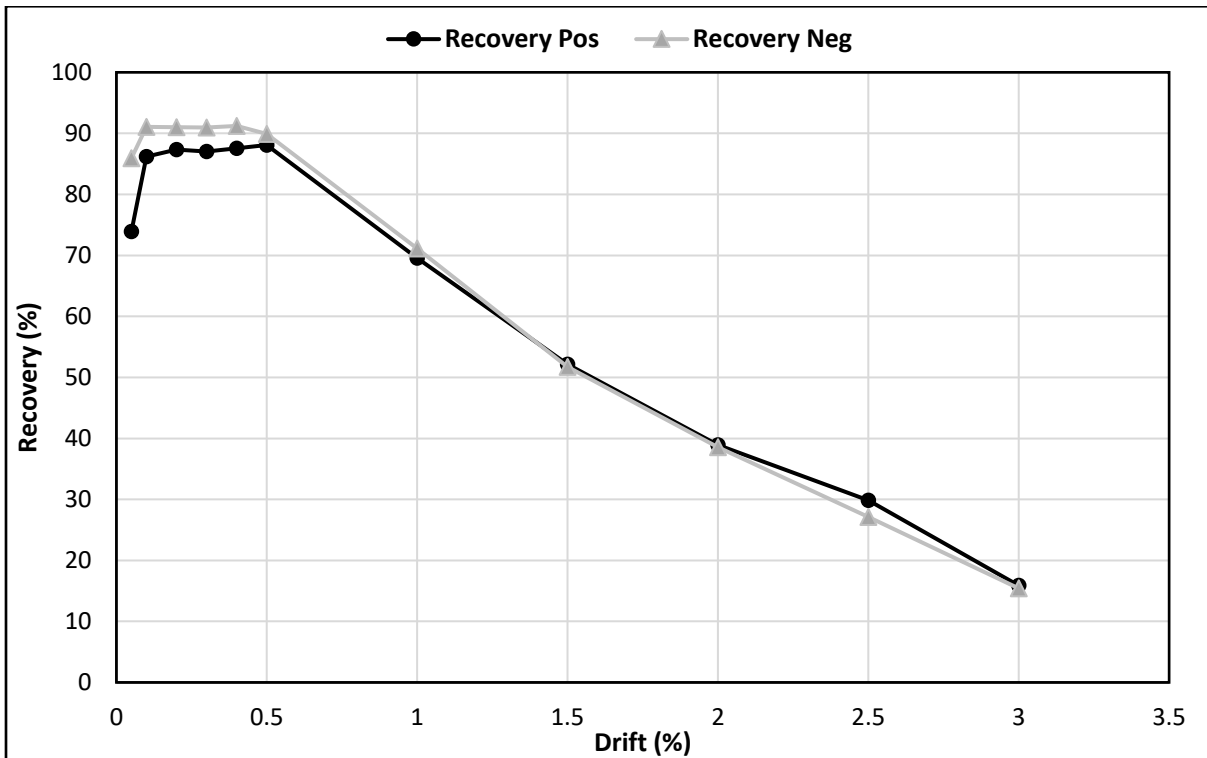


Figure 5.2: Percentage of Displacement Recovery for Positive and Negative Cycles

Drift recovery was reduced after the reinforcement in the boundary region had yielded. Up to 0.5% drift, after three cycles, on the positive loading side, the wall recovered on average 85% of the displacement. At 1.0% drift, the wall recovered 70% after the three cycles. At 1.5% drift, the wall recovered 53.7% of the 37.5mm after two cycles. From this point forward, the drift was recovered linearly in subsequent drift cycles. The walls recovery was reduced by 10% at each drift level from 1.5% drift to 3% drift until the wall failed. The relationship between recovery and

drift was approximately negatively linear after 0.5% drift. Every 0.5% increase in drift beyond 0.5% drift, resulted in a linear decline to 3% drift. This averaged to a 13% decrease each 0.5% drift from 0.5% drift to 3.0% drift.

The recovery during loading to the negative side of the wall was higher between 0-0.5% drift cycles. The wall recovered an average of 91% of the displacement. After 0.5% drift, the negative cycles paralleled the positive cycles.

There is a correlation with displacement recovery, yielding, and crack closing. Cracking, as stated previously, initiated at 5mm displacement (0.2% drift). Global yielding as determined by the Park Method, of the reinforcement occurred at approximately 9.2mm (0.37% drift). Up to yielding the recovery response in Figure 5.2 is plateaued. After the onset of yielding, the steel is no longer elastic and does not have the ability to return to its original form. This follows the material properties. Due to the axial loading, however, the cracks fully recovered and closed to 25mm of lateral displacement (1.0% drift). As cracks began to remain open due to the increased crack development and damage, the recovery also continued to diminish. The linear decline in recovery and lack of crack recovery also bears a similar relationship; at each displacement the cracks opened wider and the residual cracks remained wider. The increase in crack width after unloading also linearly increased as recovery linearly decreased.

The recovery of concrete walls is limited due to the ability of reinforcing steel to recover after yielding. Figure 5.3 provides the peak-residual displacement response. After yielding, the residual displacement increased significantly. As mentioned previously, the reinforcement did buckle on the compression sides of the wall when the concrete crushed. This buckling created added strains on the compression sides of the wall and increased the fatigue within the steel during load reversals. This fatigue effect added to the steels reduced recovery capabilities.

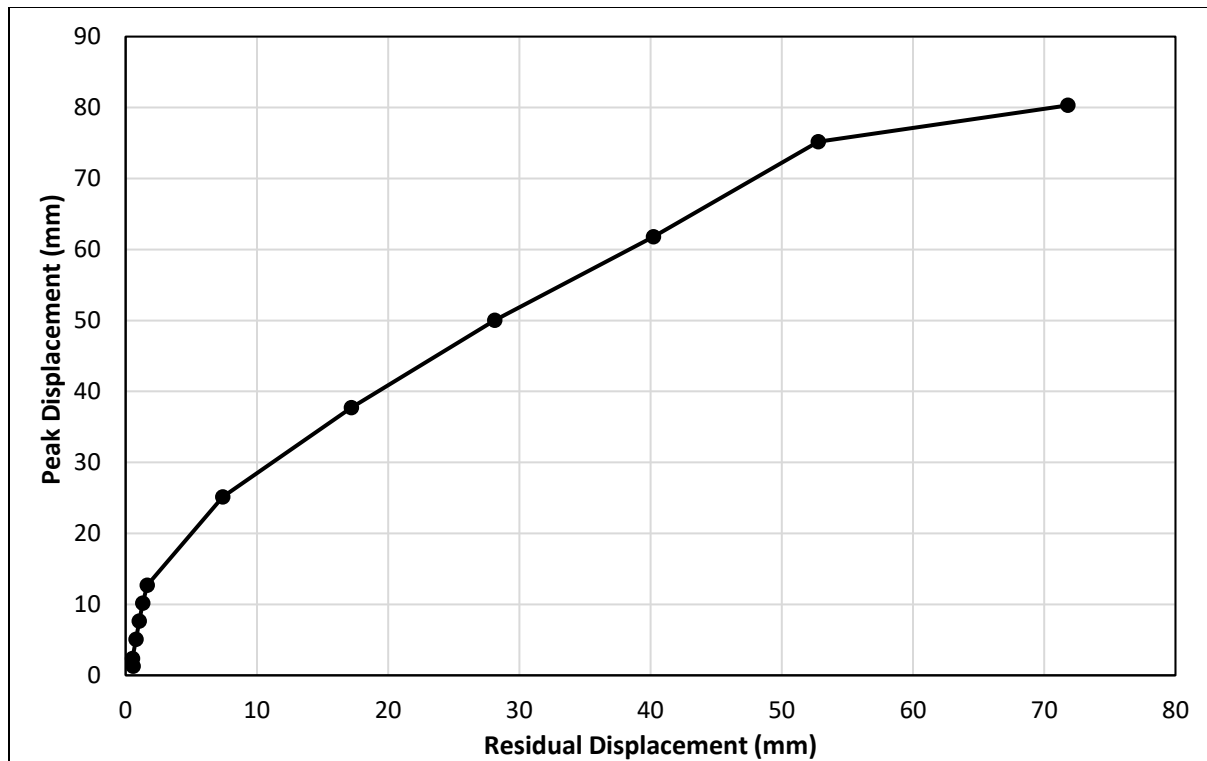


Figure 5.3: Peak-Residual Displacement Response

5.4 Lateral Displacements and Loading

During the first cycles of the test, there were some discrepancies regarding the applied lateral load and the recorded lateral displacement. This is evident on the first point of the recovery response in Figure 5.2 above. CP1, which was used to monitor the lateral displacements, was secured to a steel C-Channel test frame that was mounted on the foundation of the wall. This test frame came in contact with the steel cross beam with the mounted rollers close to the front of the wall. This was evident by a notable scratching noise that came from the testing frame during the application of the first series of displacements. This was corrected by pausing the test and adjusting the bolts holding the mounting frame. Though visibly it did not affect the position of CP1, the scraping could likely have been the reason for the noticeable changes in recovery at 0.05% drift.

The lateral loading applied to the wall at early stages of the testing was mainly from the horizontal actuator, but as the displacements increased throughout the testing, the actuator applying the axial loading contributed more significantly to the lateral loading. The axial load was

applied to the middle of the cap beam, and the vertical actuator was bolted to the cap beam. The top of the actuator was fastened to the loading frame and was permitted to swivel about its connection points. As the test progressed, the vertical actuator travelled, and the swivel heads rotated to be compliant with the cap beam. As a result, the axial loading was applied at an angle. Thus, as the wall displaced laterally, the vertical actuator imposed a vertical and horizontal load on the cap beam, which required trigonometry to solve for these force components. The initial results from the actuators load cell required correction using the horizontal displacement data from CP1 and the vertical displacement data from CP7 and CP8. CP1, which recorded the lateral displacement of the middle of the cap beam, and the rotation of the cap beam, determined from CP7 and CP8, were used to determine the lateral displacement at the swivel point of the actuator near the cap beam. In addition, the difference of CP7 and CP8 divided by their horizontal separation (1180mm) were used to determine the rotation of the cap beam. The average of CP7 and CP8 were used to calculate the change in vertical length of the actuator between its swivel points. With these measurements, the horizontal and vertical components from the actuator were calculated. The load-displacement responses were modified to account for the added horizontal force from the vertical actuator. A derivation for the corrected lateral load correction is provided in Appendix G. The results of this derivation are shown in Figure 5.4.

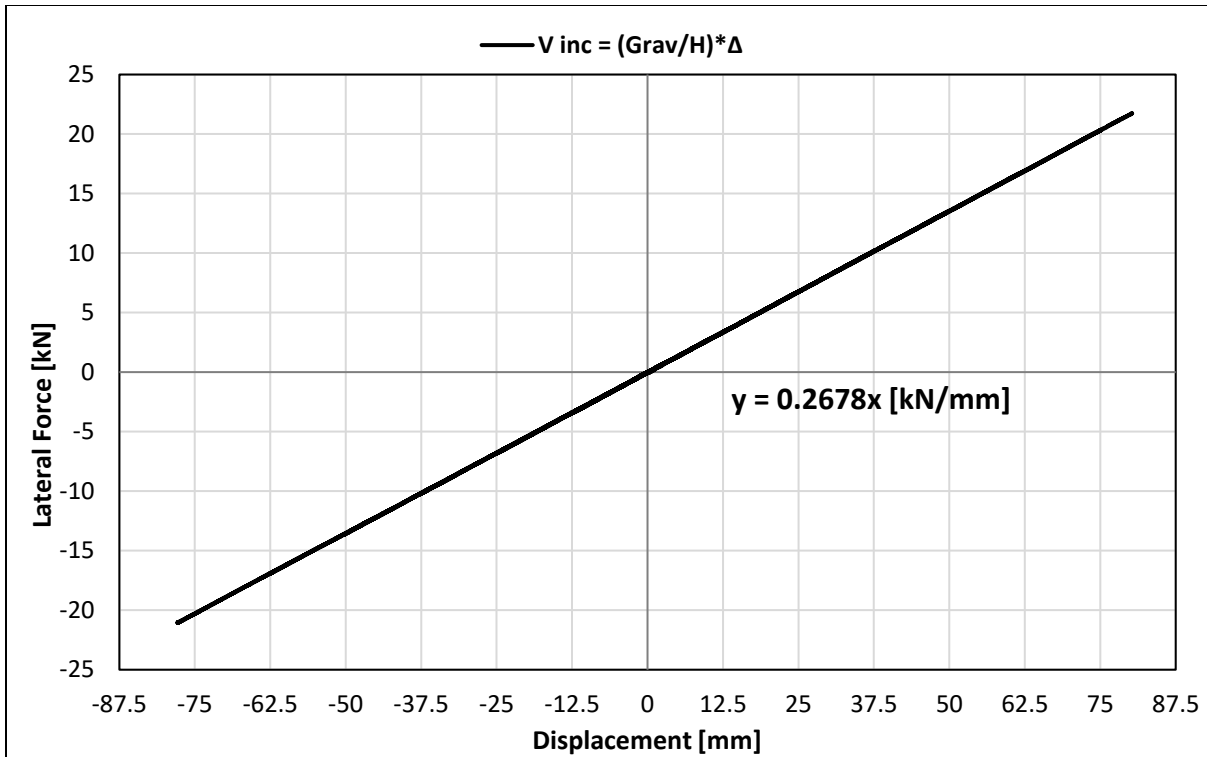


Figure 5.4: Linear Relationship for Lateral Load Correction

Figure 5.4 provided the incremental lateral load arising from the vertical actuator as a function of the lateral displacement recorded by CP1. The additional lateral force is linearly proportional to the lateral displacement of the wall. At zero lateral displacement, the lateral force correction is also zero. However, at 81.2mm of lateral displacement, the lateral load increased by 21.7kN. The relationship between the lateral load from the vertical actuator during testing was a linear relationship with the lateral displacement with a slope of 0.2678 kN/mm. At any displacement throughout testing, the corrected lateral load was a combination of the load lateral recorded by the horizontal actuator plus the lateral displacement multiplied by 0.2678. At the maximum lateral load, corresponding to 25mm, the lateral from the horizontal actuator was 211.4kN, with an added 25mm multiplied by 0.2678 kN/mm to provide another 6.8kN from the vertical actuator. Thus, the corrected maximum load was 218.13kN.

The lateral displacements during testing were not consistent every cycle due to the force controlled nature of the test. The target displacements were based on a visual inspection of the readings from CP1. When the target displacement was achieved, the load was stopped and then reversed to hit the target displacement in the opposite direction. Unlike the modelling software

which can run to the same displacement every cycle, there were some variations during testing. None of these variations were greater than $\pm 5\%$ and, therefore, the testing was consistency executed.

5.5 Vertical Displacements

The vertical displacements of the wall were greater on the tension sides during positive and negative cycles, and smaller on the compression sides during positive and negative cycles. Until the concrete spalled and crushed on the compression sides of the wall, there was not a significant change in displacement at the compression toes of the wall. Once the concrete was crushed and cover spalled off, the steel was exposed and began to buckle. Although there was ample confinement from the stirrups and ties, the concrete supported the majority of the compressive force. The steel did not rupture in compression, which is expected, however it did experience more damage due to buckling. This was observed during the testing.

Due to these vertical displacements, there was significant rotation of the cap beam. The rotation of the cap beam was established from the vertical displacement's readings from CP7 and CP8 as illustrated in Figure 5.5 and Figure 5.6.

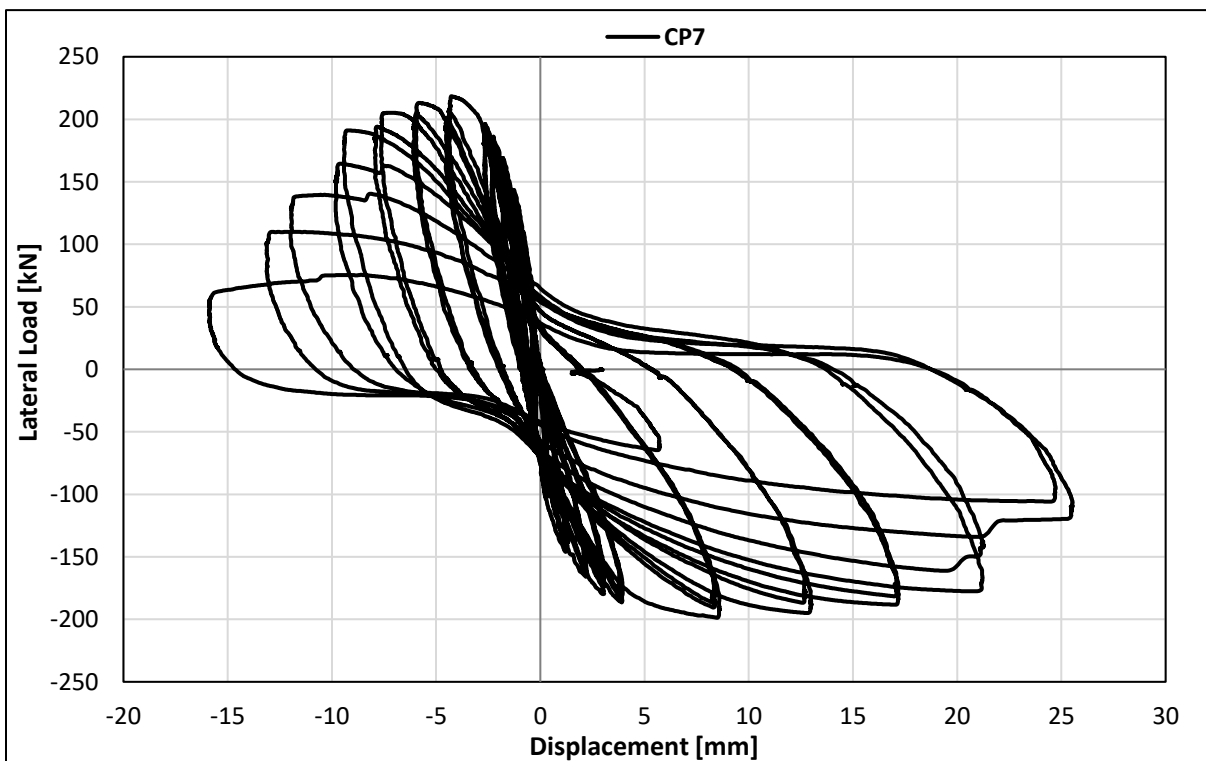


Figure 5.5: Load-Vertical Displacement from CP7

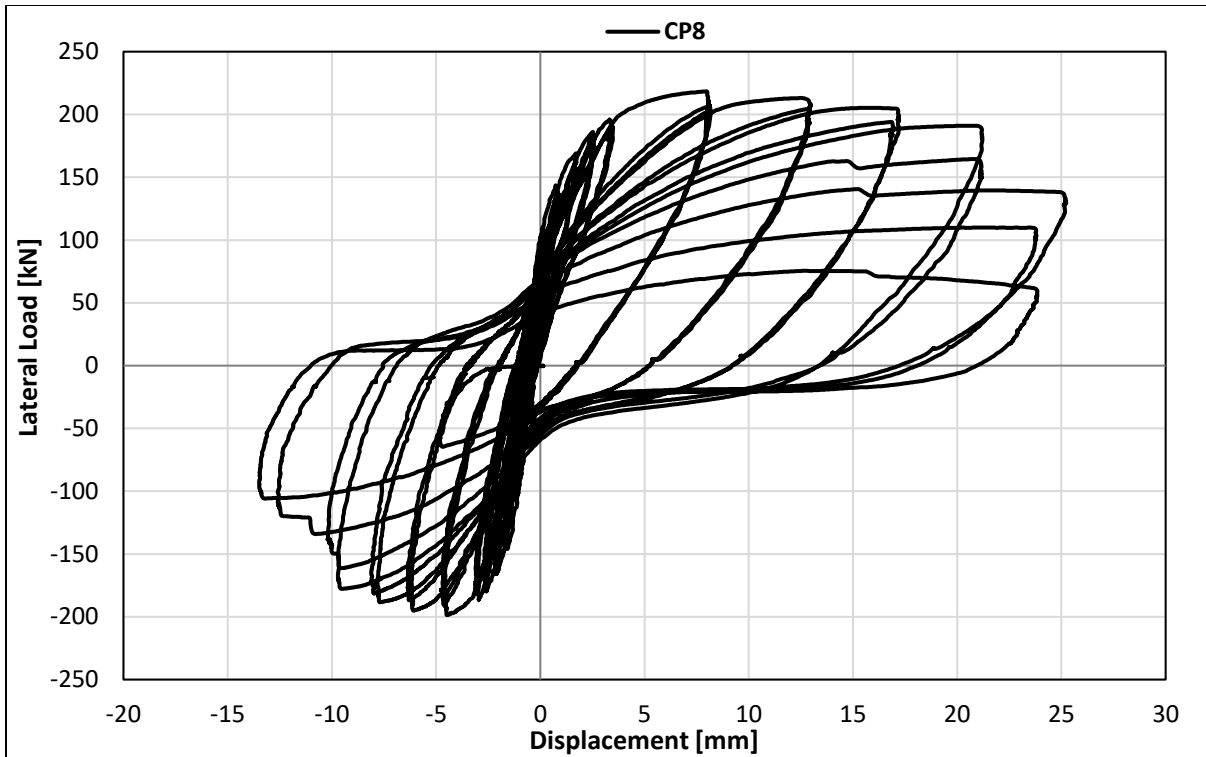


Figure 5.6: Load-Vertical Displacement from CP8

Both CP7 and CP8 provided similar responses, but on opposite sides of the vertical axis. When the difference of the two is taken, see Equation 5.2, as seen in the graph below, the average change in height of the middle of the cap beam can be observed. The rotation was established from the difference between readings from CP8 and CP7 and then divided by the length separating the two cable potentiometers, approximately 1180mm (Equation 5.3). The respective responses based on these calculations are provided in Figure 5.7 and Figure 5.8.

Equation 5.2

$$\text{Change in height of middle of cap beam} = \frac{(\text{CP8} + \text{CP7})}{2}$$

Equation 5.3

$$\text{Cap Beam Rotation, } \theta = \frac{(\text{CP8} - \text{CP7})}{1180 \text{ mm}}$$

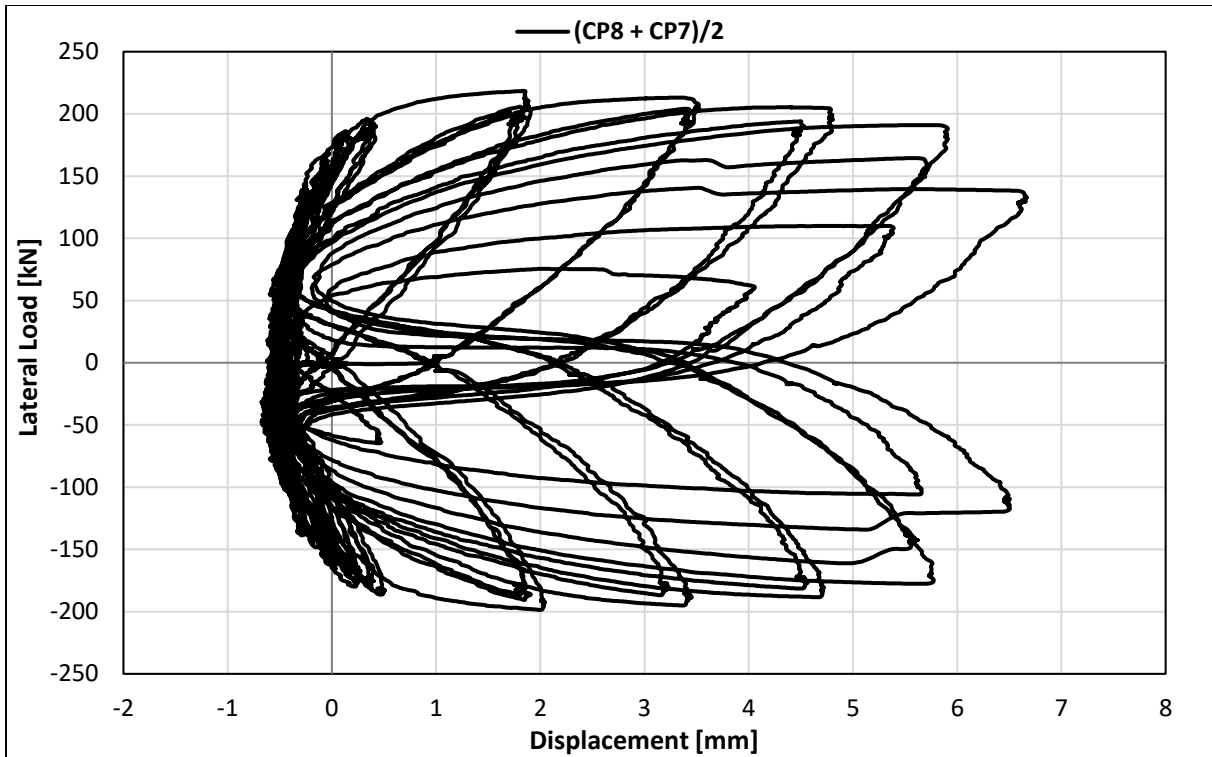


Figure 5.7: Average Displacement of Center of Cap Beam

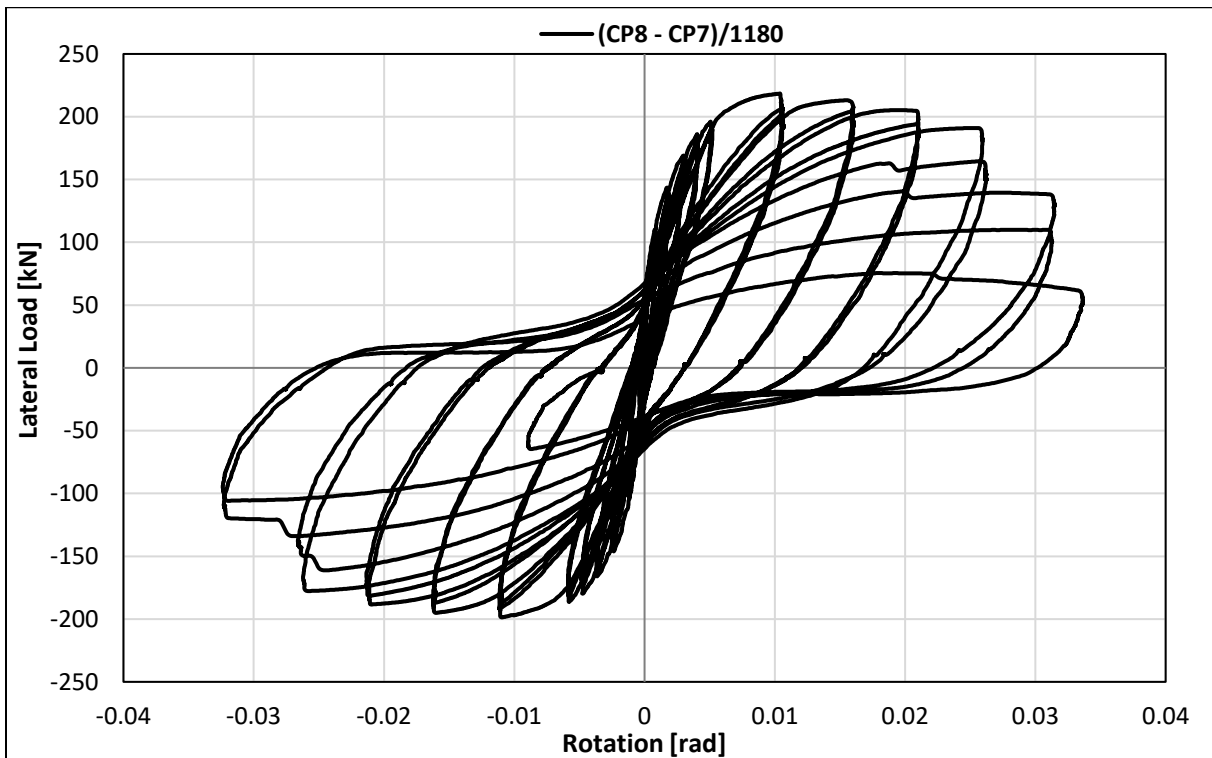


Figure 5.8: Load-Rotation of Cap Beam

5.6 Out-of-Plane Displacements

The out-of-plane displacement of the wall was likely magnified by the application of axial loading combined with the cap beam being slightly tilted due to a casting discrepancy. This caused the wall to lean forward adding compressive forces to the front of the wall and tensile forces to the back of the wall. The added compressive forces on the front of the wall likely preserved the bars from rupturing at the same time as the bars on the back side of the wall. In the same way, that the added tension forces on the bars on the back led them to rupture before the bars on the front.

Although there was an out-of-plane force and possible added compression to the front of the wall and added tension to the back of the wall, the boundary bars on the compression sides during push and pull cycles buckled during the 75mm (3.0% drift) cycles. The added tensile force was likely only present and applied to the tension sides of the walls during push and pull cycles, likely leading the back side bars to rupture first.

5.7 Shear Distortions

Shear distortion was determined from the displacements captured from CP9-CP12. Referring to Chapter 3 Experimental Program, these potentiometers were located within the plastic hinge region in an 850mm by 800mm region. The height was increased from 800mm to 850mm due to an issue with contacting a stirrup when drilling into the wall. The approximate method for calculating shear distortions was determined by Oesterle et al. [12] [13] and shown below in Equation 5.4. The equation is used to approximate the shear distortions (straining). This is calculated by taking the change in lengths of the diagonal CPs in either direction. In Equation 5.4, d is the original diagonal length, δ is the change in diagonal length, h is the original height of the region and l is the original width of the region.

Equation 5.4

$$\gamma_{avg} = \frac{\delta_1 d_1 - \delta_2 d_2}{2 * h * l}$$

Figure 5.9 illustrates that there is some pinching effect about the origin. The shear distortions were terminated at 3.0% drift when crushing in the toe regions dislodged the screws securing the potentiometers. Table 5.4 lists the shear distortion values for the various performance points. During negative cycles, higher shear distortion values were observed.

Table 5.4: Shear Distortions

	Positive Cycle ($\times 10^{-3}$ rad)	Negative Cycle ($\times 10^{-3}$ rad)
Yield	0.595	-0.492
Peak	1.500	-1.802
2.5% Drift	4.439	-4.942
Maximum	5.311	-6.307

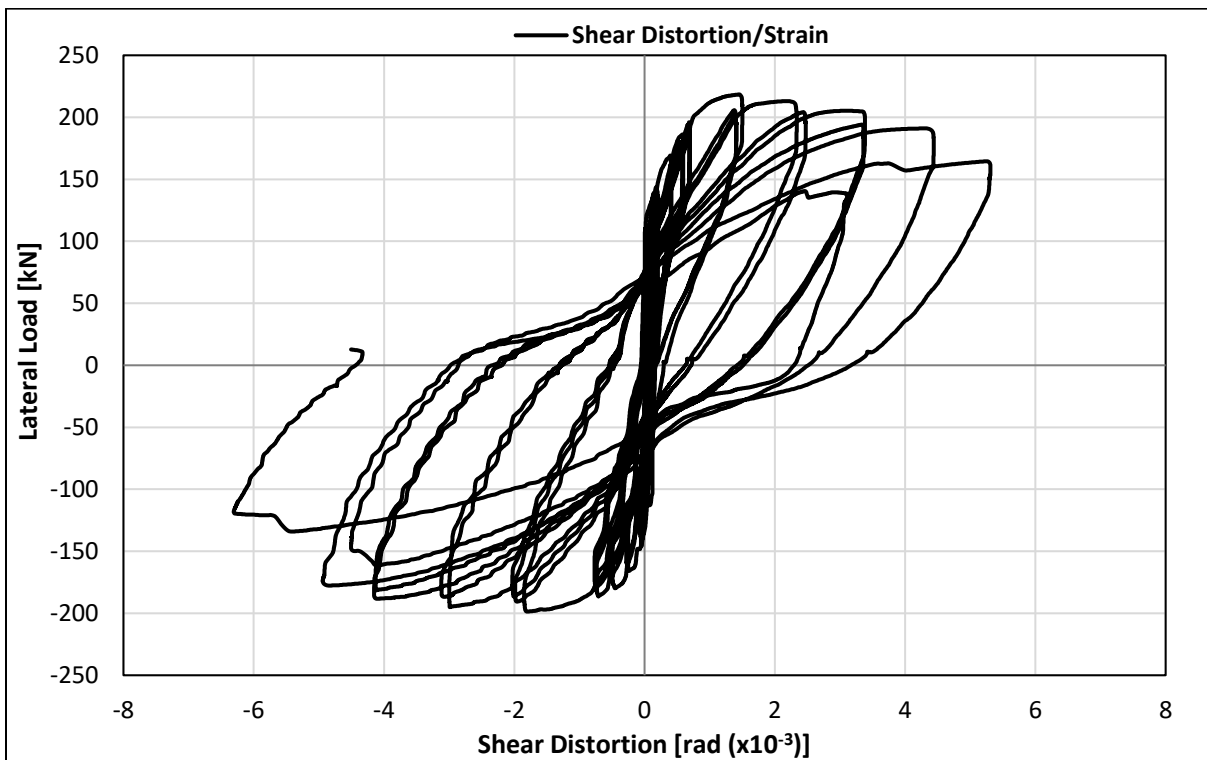


Figure 5.9: Load-Shear Distortion

5.8 Application of Axial Loading

The application of axial loading is a key objective in this research. The loading was applied by a hydraulic actuator suspended from a loading frame and secured to the top of the wall using

threaded rods. The following sections discuss key components of the axial loading that impacted the testing of the wall.

The axial loading was applied at the beginning of the test which led to loosening of the threaded rods that secured the foundation to the strong floor. These rods were retightened, however, based on the rigid body displacements, there was some foundation sliding throughout testing. After the test was completed, two of the threaded rods were found to be loose as a result of the added compressive forces from the axial loading. To rectify this, post-tensioned rods may be necessary to ensure adequate anchorage to the strong floor. Another possible solution would be to enlarge the diameter of the threaded rods used to reduce the space within the holes in the foundation and strong floor.

The threaded rods that were cast into the concrete were integral in maintaining maximum contact between the face of the actuator head and the steel plate on the top of the cap beam of the wall. The short coming of these rods, however, was encountered during the first positive cycle to 75mm displacement. Up to 75mm, the rods were clear from contacting the swivel head of the top actuator; however, during loading to 87.5mm displacement, there was near contact between the head of the actuator and the rods. The cycle was paused prematurely to avoid damaging the actuator. To solve this issue, the rods should be cut shorter in future tests to ensure the vertical actuator can rotate freely at large displacement levels.

Due to the lean of the wall, the actuator also had a lean. The actuator was free to rotate about the two swivel heads, which allowed it to remain in contact with the cap beam of the wall. However, the lean of the swivel head contacting the cap beam likely caused additional load to be applied as additional tension stresses on one face and additional compressive stresses on the opposite face of the wall. This is difficult to capture or calculate using the given instrumentation. To rectify this issue, during casting, the wall level will need to be monitored and a more adequate bracing frame will need to be applied between the base of the cap beam and the wall formwork.

5.9 Energy Dissipation

The hysteretic energy dissipation was calculated from the load-displacement response of the first cycle for each target displacement (drift). The presence of applied axial loading causes the hysteretic response to pinch about the origin and reduces the steepness of the unloading

branches (Figure 5.10). This pinching is representative of recentering, but it is also representative of higher elastic potential energy. Visibly this is present when there is a large area beneath the loops or through the reduction in steepness of the unloading branch to zero load.

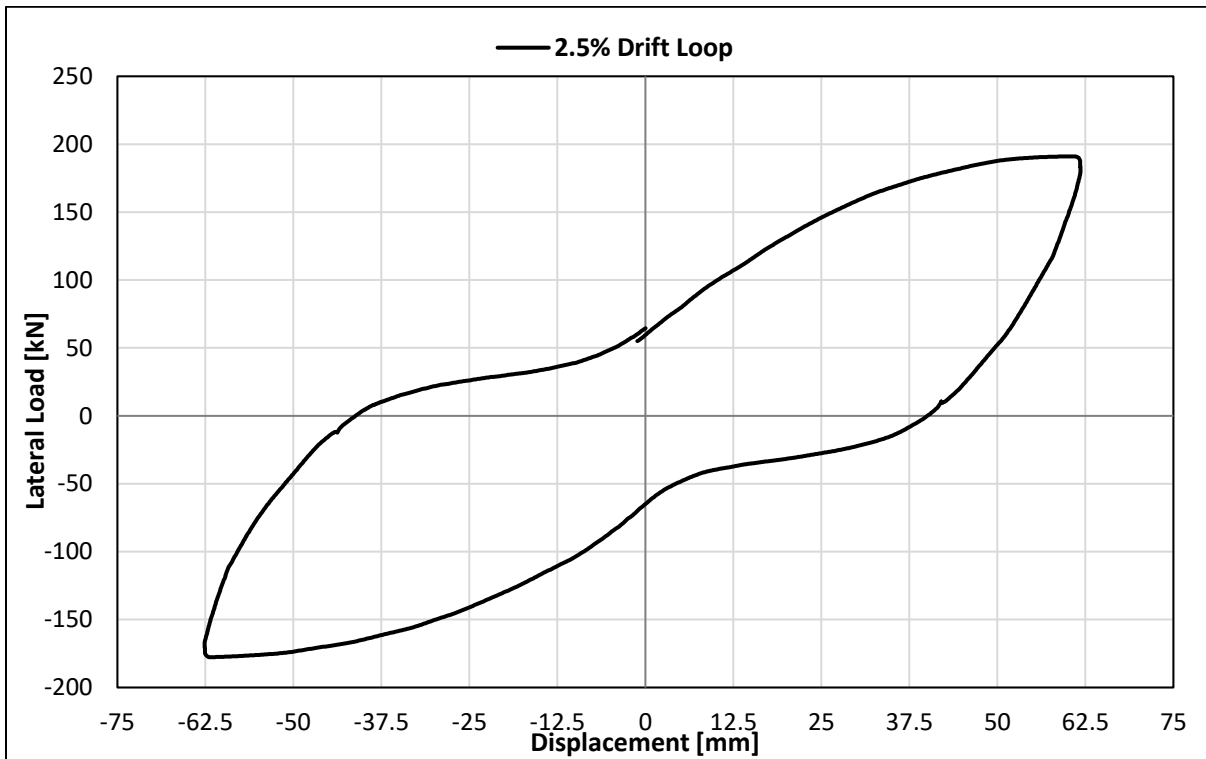


Figure 5.10: Load-Displacement Response at First +/- 2.5% Drift Cycle

The hysteretic energy and stored elastic potential were calculated using the Reiman Sum method. The Reiman Sum method calculates the area of two points by averaging the height of both points multiplied by the distance between the points. Figure 5.11 outlines the areas for hysteretic (dissipated) energy (in black) and stored elastic potential energy (in red).

The stored elastic potential energy is the area of the red lines in Figure 5.11, taking the Reiman Sum of the descending branch of the hysteretic loop gives the stored elastic potential. The elastic potential energy was taken only from the area under the peaks to the horizontal axis in quadrants 2 and 4 (Figure 5.10).

Hysteretic (dissipated) energy is the area within the hysteretic loop. To calculate this, the Reiman Sum of the ascending branch to the maximum point of the curve was subtracted by the

Reiman Sum of the descending branch. The total energy is the area of the hysteretic loop in all four quadrants (Figure 5.10).

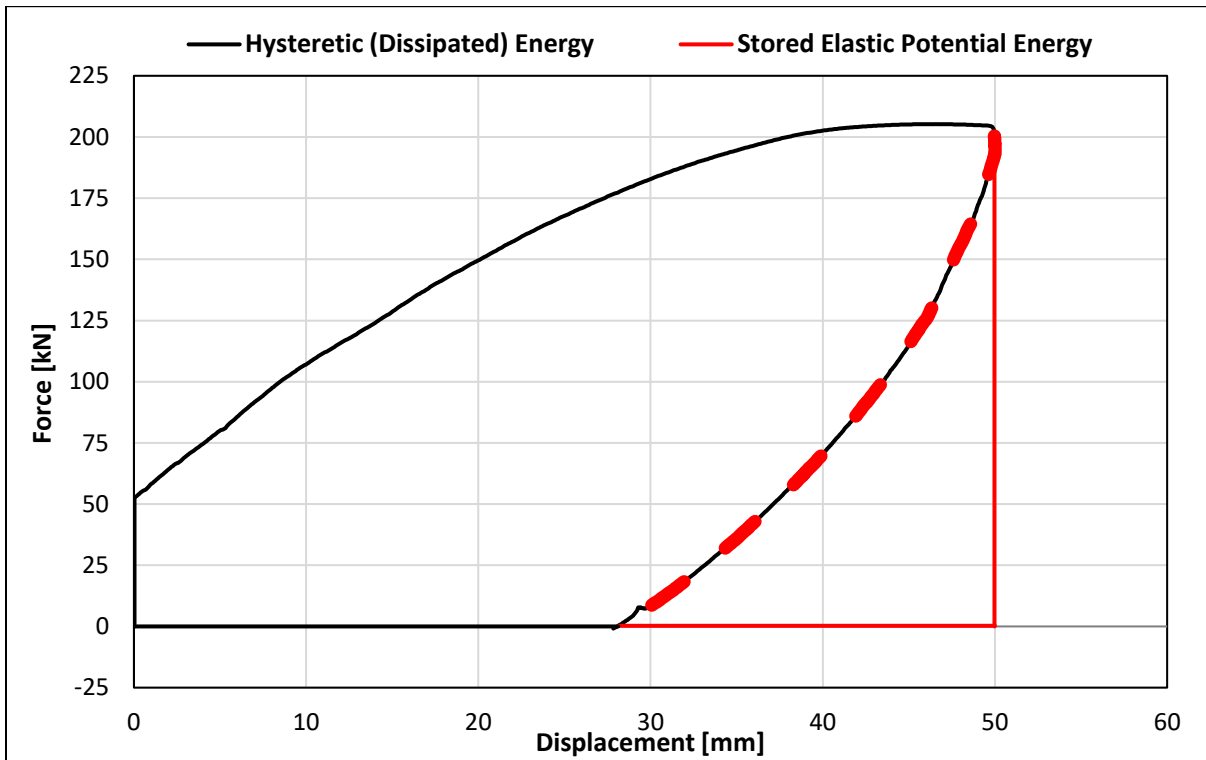


Figure 5.11: Areas of Hysteretic Energy and Stored Elastic Potential for Positive 2.0% Drift Cycle

The total energy that was dissipated continually increased throughout testing as illustrated in Figure 5.12. The total energy increased linearly throughout the test. Although the negative cycle was not recorded for 3.5% drift, it would likely have less total energy than 3.0%. This can be interpreted from Figure 5.13 as there is a steep decline when progressing from 3.0% to 3.5% drift.

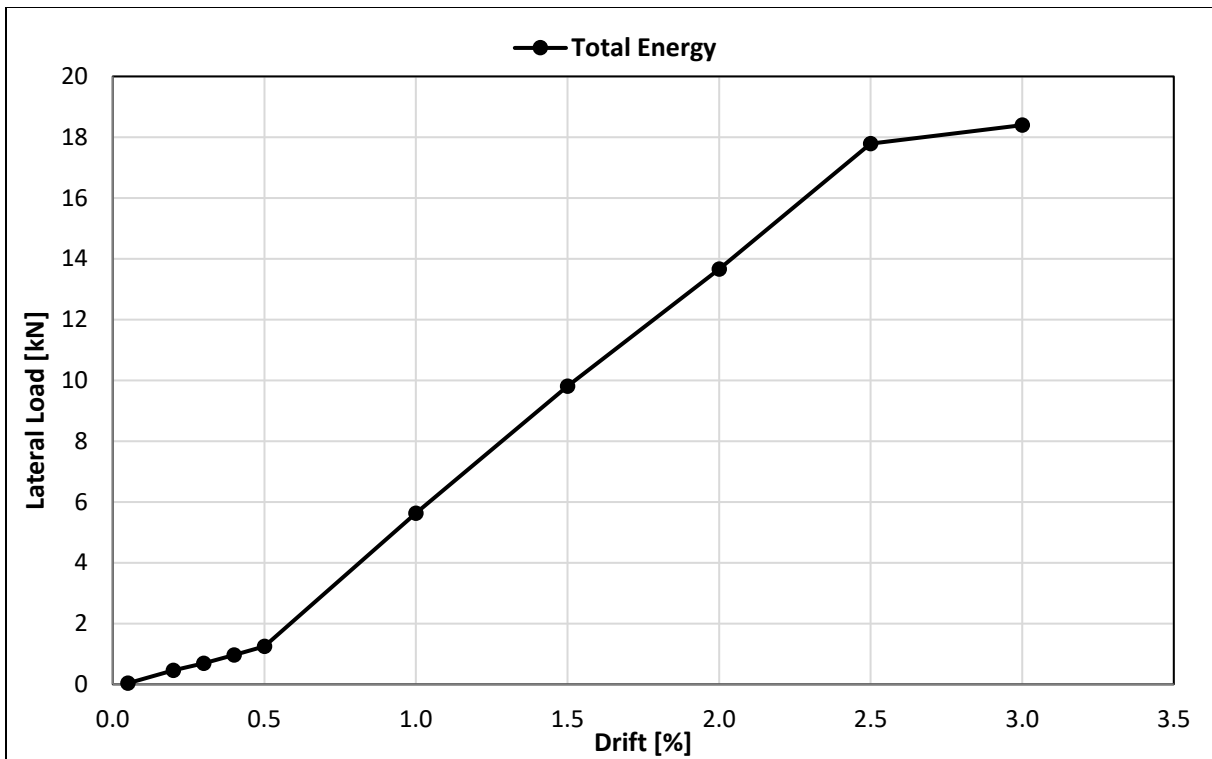


Figure 5.12: Total Energy

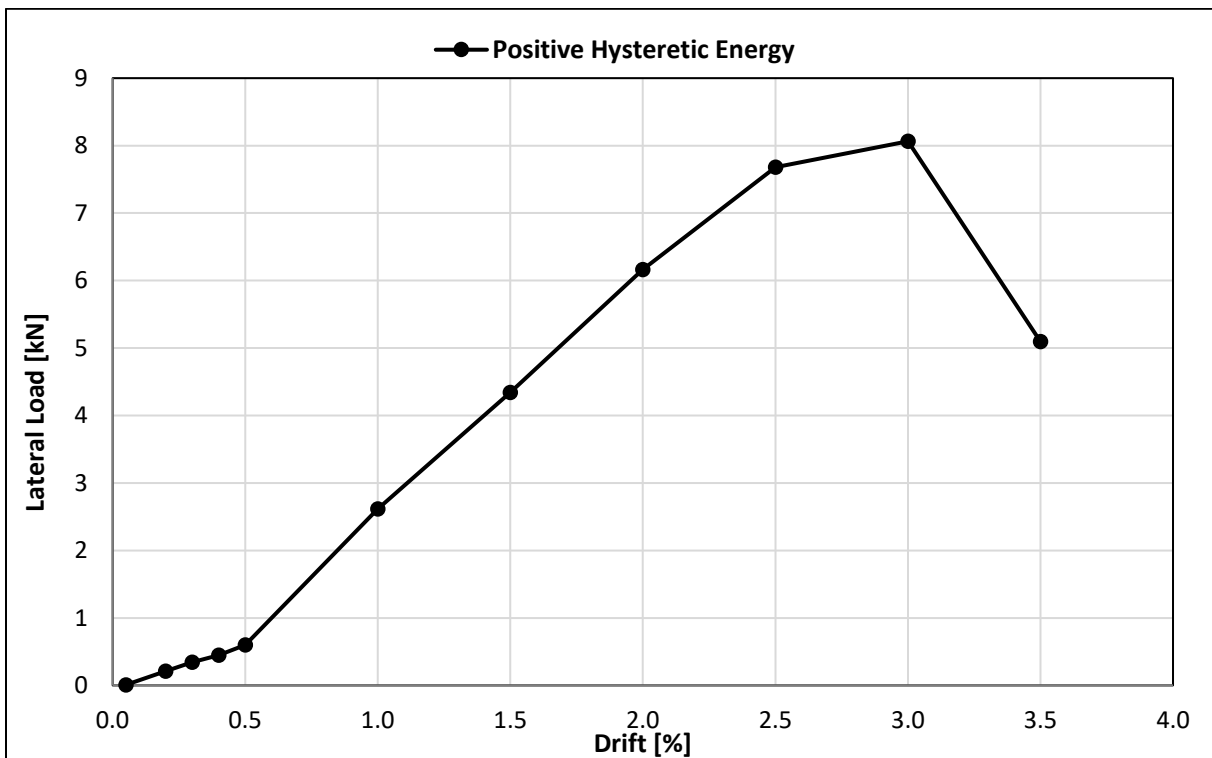


Figure 5.13: Hysteretic Energy Dissipation for Positive Cycles

The total energy is reliant on the amount of lateral loading required to displace the wall. The total energy for this experiment is higher than previous testing done by Abdulridha [1] and Morcos [3] and this is due to the higher lateral as a result of the applied axial loading. This is expected as the high axial load increased the stiffness of the wall and increased the total energy. As the lateral loading decreased from rupturing of the reinforcement, the slope of the linear increase in Figure 5.12 and Figure 5.13 reduced. The steepness of the 0.50% drift to 2.5% drift is not maintained to 3.0% and ultimately would have been further reduced at 3.5% as shown in Figure 5.13.

The total energy, illustrated in Figure 5.12, is based on the values listed in Table 5.5. In Table 5.5, the values for 0.10% drift were not calculated due to a discontinuity in the response (see Figure 5.14). The energy for 3.5% drift during the negative loading was also not recorded as the test was terminated at the end of the positive 3.5% cycle.

Table 5.5: Energy Throughout Testing

Drift (%)	Positive Side ($\times 10^3$ Nm)		Negative Side ($\times 10^3$ Nm)		Total Energy ($\times 10^3$ Nm)
	Dissipated Energy	Stored Elastic Potential	Dissipated Energy	Stored Elastic Potential	
0.05	0.009	0.010	0.020	0.021	0.035
0.10	-	-	-	-	-
0.20	0.210	0.272	0.235	0.293	0.464
0.30	0.343	0.479	0.326	0.518	0.692
0.40	0.447	0.713	0.486	0.707	0.965
0.50	0.598	0.943	0.607	0.914	1.247
1.00	2.617	1.424	2.722	1.315	5.631
1.50	4.343	1.378	4.574	1.449	9.808
2.00	6.165	1.580	5.861	1.447	13.664
2.50	7.682	1.466	7.617	1.263	17.794
3.00	8.064	0.969	7.877	0.698	18.398
3.50	5.094	0.167	-	-	6.545

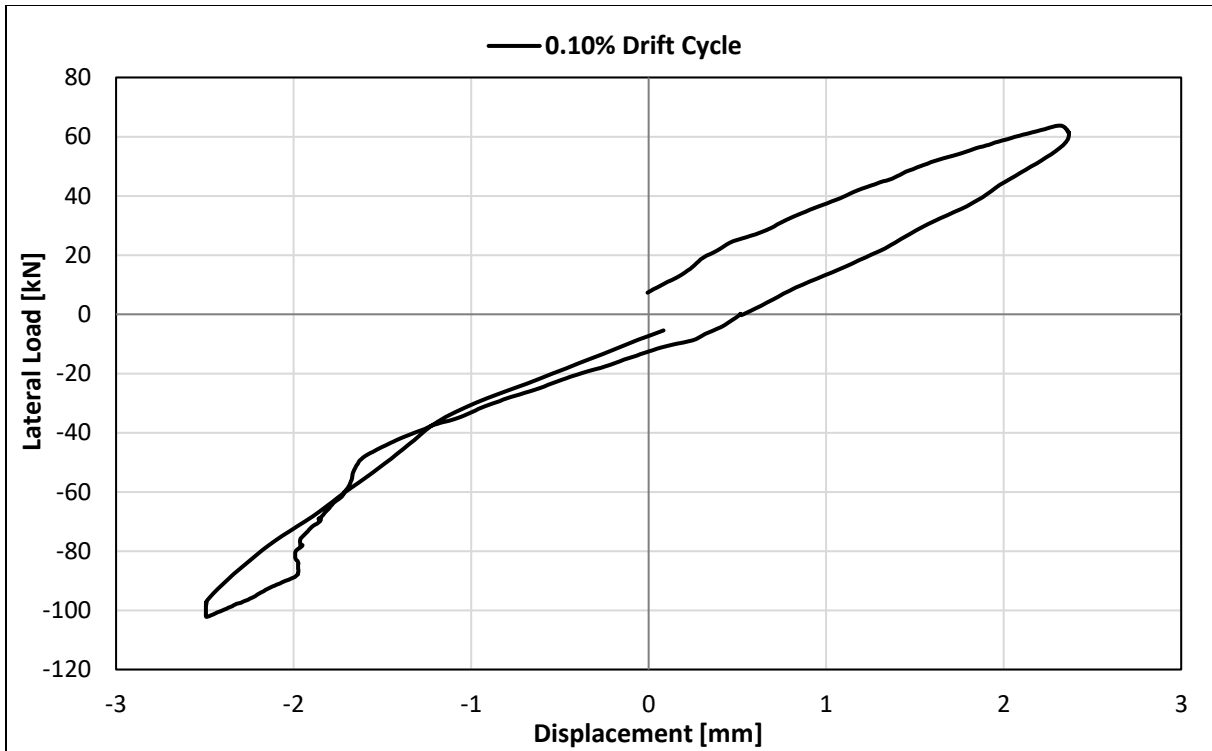


Figure 5.14: Load-Displacement Response at First Cycle for +/- 0.10% Drift

6 Numerical Modelling of Shear Wall

6.1 Introduction

Numerical modelling was conducted on this shear wall, using Nonlinear Finite Element Analysis (NLFEA) through FormWorks and VecTor2 (Version 4.4). The purpose of the model was to determine the behaviour of the shear wall prior to testing and verify/update the model after testing. The initial model was based on the walls constructed by Morcos [3] and Soto [4]. This original model was modified to account for the changes in the design/construction of the shear wall and the dimensions and material properties. Several models were developed in the preliminary phase prior to construction to observe the impacts of different percentages of axial loading. Once the test frame was designed and finalized, the cap beam dimensions changed and, thus, the model was modified.

6.2 Model Development

Several models were constructed based on the reinforced concrete shear wall tested by Morcos [3]. The initial model consisted of the same 2200mm tall by 1000mm long by 150mm thick wall, and 400mm high by 400mm wide, by 1600mm long cap beam. The concrete compressive strengths and steel yield values were adopted from those reported Morcos. These values were input into the model to ensure all variations of the models had the same properties. The only portion of the wall not modeled to scale was the foundation block, which was significantly large (1700mm wide by 1400mm deep) within the model to ensure adequate stiffness during reverse cyclic analyses. Note that these models were used prior to construction of the actual wall and prior to finalizing the dimensions.

Different axial loads were applied to observe the walls' behaviour. The axial loads were based on percentages of the wall's cross-section compressive capacity ($f'_c A_g$). The loads used were 0%, 2.5%, 5%, 7.5%, and 10% of the wall's capacity. Prior to completing cyclic analyses of the various loadings, monotonic pushover analyses were run to observe the effects of increasing the axial as illustrated in Figure 6.1. From the pushover results shown below, a pattern can clearly be observed from 0% axial loading to 10% axial loading. The amount of lateral load required to reach the same displacements increased. This result was expected as increasing the axial load increasing the stiffness, thus more force is required to displace the structure.

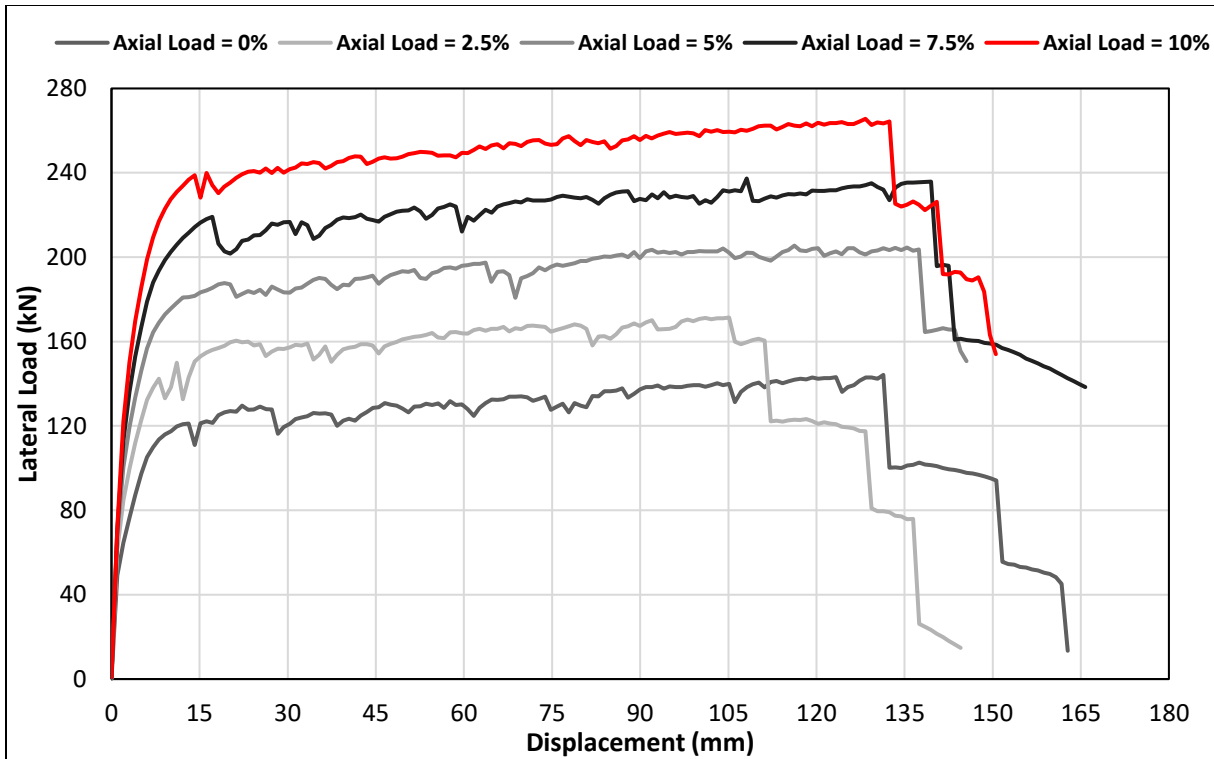


Figure 6.1: Monotonic Pushover Analyses from Original (initial) Finite Element Model

A comparison of the reverse cyclic responses to the same axial load levels was completed. The comparison of 0% and 10% axial load is shown in Figure 6.2. [Note, the other reverse cyclic responses are available in Appendix K]. The necessity to review the variations in varying axial load affects the recentering of the wall residual displacements. The response with 0% axial load represents the highest potential for large residual displacements; whereas the 10% axial load is intended to promote the higher recentering. These models were also used to compare the behaviour of the wall with and without axial loading and to assess the seismic performance and effects with applied axial load. Specifically, from Figure 6.2, two differences were observed. Similar to the pushover responses, with an increase in axial loading, there was an increase in lateral load resistance. The second difference was the in shape of the load-displacement loop.

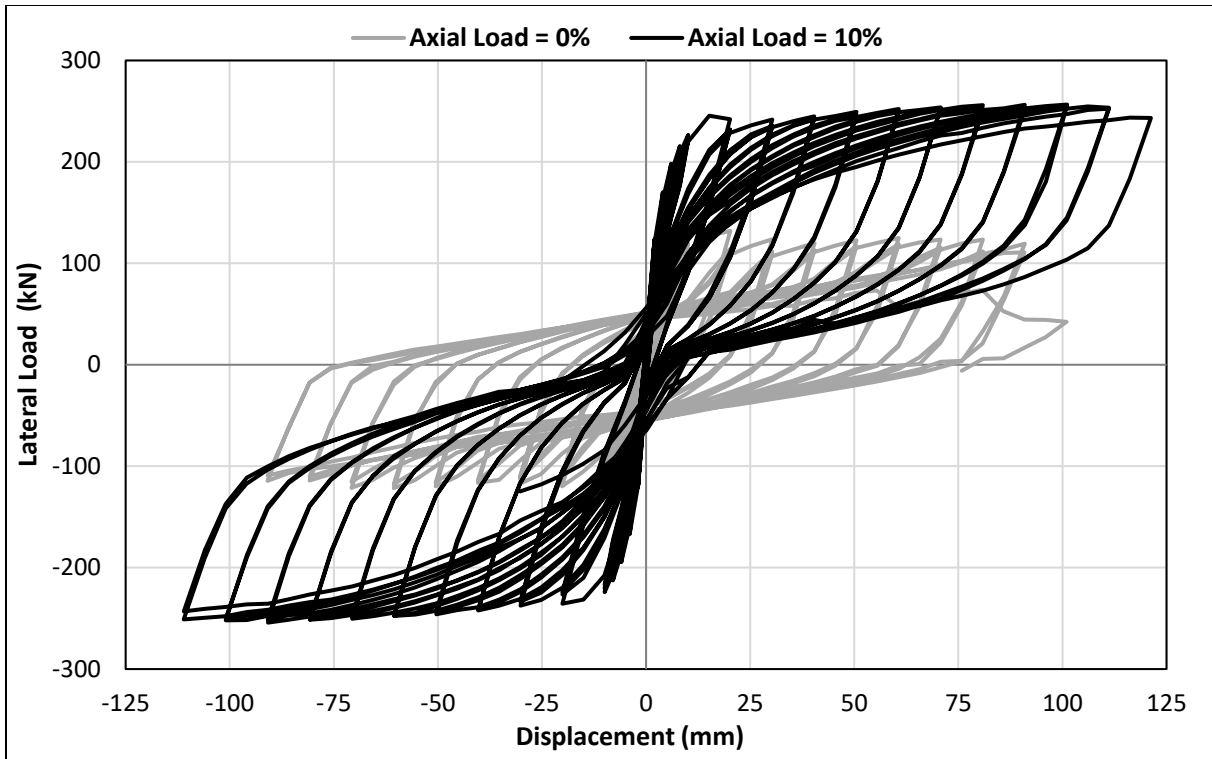


Figure 6.2: Comparison of Reverse Cyclic Analyses from Original Model for 0% and 10% Axial Loading

With no axial loading, the loops are much wider and their descent from each peak is steep. With 10% axial loading, there is noticeable pinching about the origin and a gradual slope from the peak load to zero load. The pinching effect is a sign of poorer energy dissipation; a function of the area under the hysteretic curves.

6.3 Finalized Models

Once the dimensions of the cap beam were finalized due to the test setup and size of actuators, the original models were modified to account for these updates. The focus of the updated models was on 7.5% and 10% axial loading. These models were developed using the expected concrete compressive strength for the wall and expected reinforcing steel yield strength. When the actual material properties were established from the concrete cylinder compression tests and uniaxial steel tension tests, they were input into the models. After testing was completed, additional finite element models were considered to capture some behavioural features that arose during the experimental testing.

The initial models were organized in the following manner: the concrete zones defined by rectangular elements were constructed; discrete truss bars to represent the longitudinal reinforcement in the wall were located at the interfaces of the concrete rectangular elements; bond properties were assigned to the truss bars; pin support conditions were applied at the base of the foundation in the vicinity of the anchor rods; and finally, the loading was applied. [Note that the transverse reinforcement including stirrups and ties were modelled as smeared within the rectangular elements.]. For these models, the loading was applied at two locations and in two different ways. The axial loading was initially applied across the length of the entire cap beam; however, this was modified to only the width of the actuator head (400mm) to replicate the test conditions more realistically. The lateral loading was applied as a displacement at the middle of the cap beam in increments of 1.25mm. Within VecTor2, the displacement was controlled using a loading factor to meet the same displacements used within the loading protocol for testing. The model was displacement controlled, which permitted the analysis to capture that post-peak response.

In the updated FE model, changes were made to the method for defining the boundary region and the associated area of reinforcement. Five versions of models were created. The first four versions are discussed within this section and the fifth version is discussed in Section 6.6.1.5. A brief explanation of the first three models is given in the following paragraphs with respect to Figure 6.3. Figure 6.3 (a) illustrated that the boundary region initially had a clear cover of 40mm. This cover was the length of the concrete to center of the outer longitudinal boundary bar and it contained no smeared reinforcement. Due to this, the blue region, defining the boundary region, was only 160mm wide. This resulted in the boundary region containing a higher amount of smeared reinforcement in the out-of-plane direction to account for the legs of the anti-buckling ties. This resulted in a higher lateral load resistance and stiffness. The cover was modified to 20mm as shown below in Figure 6.3 (b). One additional change was made to remove the triangulation of the elements, which is not evident in Figure 6.3 (c). This change in length in the cover reduced the concentration of smeared out-of-plane reinforcement in the boundary region and confined the boundary steel on all sides. Ultimately, further modifications were made for the fourth version of the model.

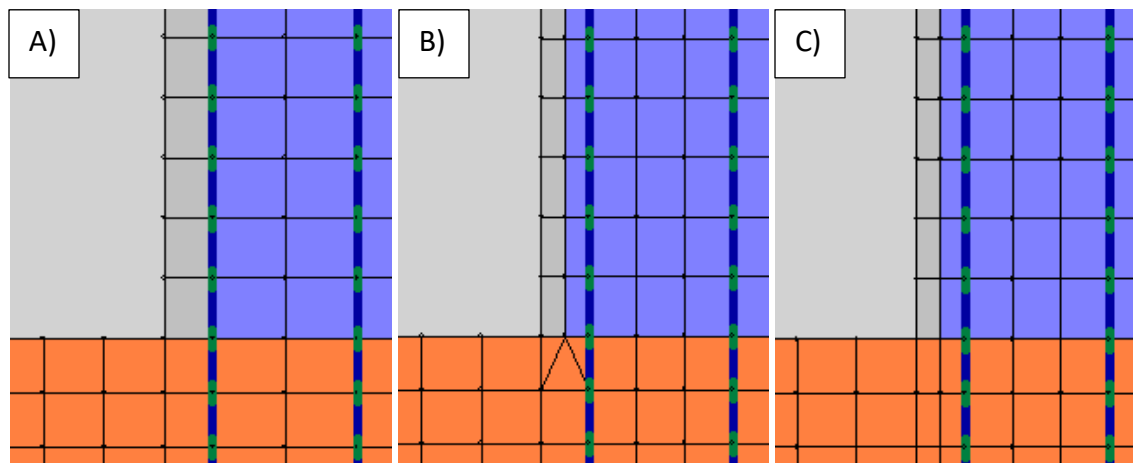


Figure 6.3: Changes to the Boundary Region: (a) Before Testing V1; (b) After Testing V2; and (c) V3

In the fourth and final version of the model, a modification was made to reduce the longitudinal reinforcing bar area within the boundary and web regions of the wall. The area of the bars in the boundaries was reduced by 10% to account for the material that is removed to permit proper bonding of the strain gauges to the bar; this is depicted by the yellow truss bars in Figure 6.4. The bars were grinded and sanded down to create a smooth surface for the strain gauges to be placed on. In the web region, the bar area was reduced by only 5% instead of 10%; shown with green truss bars in Figure 6.4. This was the result of only one side of the web bars having strain gauges, which means only half of the bars had an area reduction. A more thorough discussion of the reinforcement area reduction is provided in Section 6.3.1.

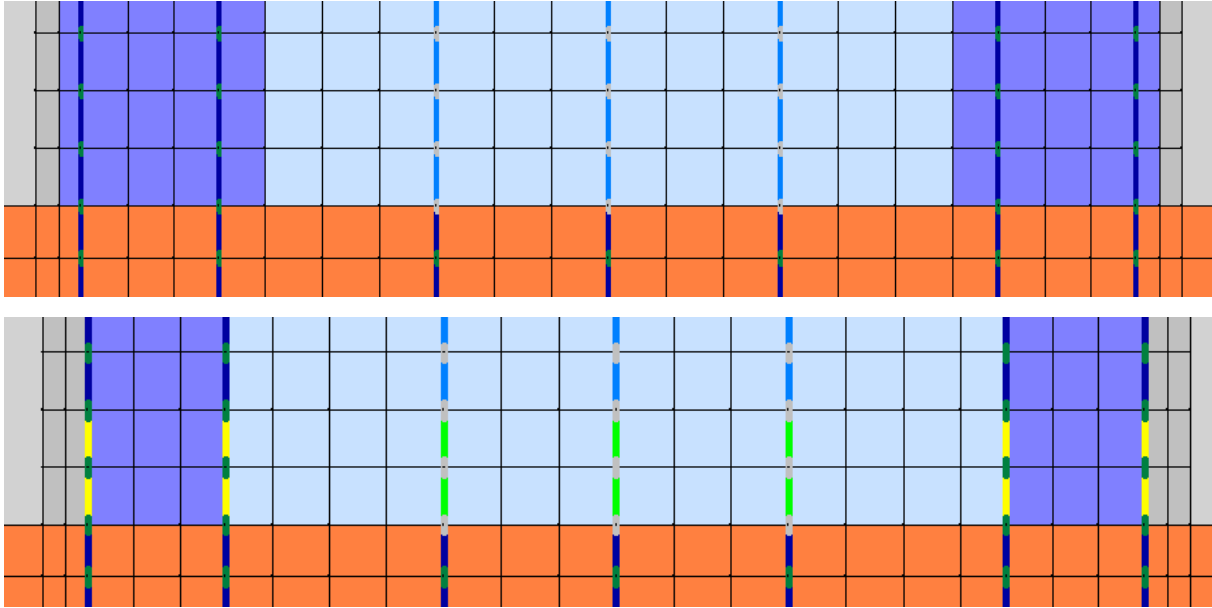


Figure 6.4: Model V3 (top) and Modification to Boundary and Web Regions in Model V4 (bottom)

Another change was made to the out-of-plane smeared reinforcement in the boundary regions. Initially, the boundary regions were 160mm in length, however this was an overestimation due to the spacing of the longitudinal bars. The cover from the first model to the most recent model, was changed from 40mm to 20mm, and then back to 40mm. After reviewing the Models V1, V2, and V3, it was determined that 40mm of cover better represented the wall and was used in Model V4. The boundary area was taken as the area between the two vertical bars or 120mm, as illustrated in the comparison of the top and bottom models in Figure 6.4. The percentage of out-of-plane smeared reinforcement increased within this region, and the web region was then increased to a length of 680mm. Overall, the cover was returned back to 40mm, the boundary regions were reduced to 120mm, and the web was enlarged to 680mm.

6.3.1 Area Reduction of Boundary Reinforcement

Multiple models were created with varying amounts of cross section area reductions in the longitudinal reinforcement. The reduction in bar area was in effect from a height of 0 to 100mm above the base. This change was made to represent the reduction in area of the reinforcing bars due to the placement of strain gauges. The intent was to simulate the effect of sanding and

grinding of the bars at the strain gauge locations. The purpose was to see if this reduction in area would reduce the strength and stiffness of the model response and to capture failure at the same location as observed during testing.

Initially the full area of the reinforcing bars was considered (Figure 6.5 (a)), but as illustrated in Figure 6.5 (b) and (c), there are area reductions depicted by the red and yellow truss bar elements. The reduction used was between 2.5% and 12.5% of the bar's cross-sectional area. In terms of the steel, the area was reduced in the range of 200mm² to 175mm². This reduction only took place in the longitudinal bars in the boundary regions as these were the bars that ruptured during testing. Ultimately, the closer to 12.5%, the earlier the model failed, however, it would not experience any stiffness reduction. The reason for the model overestimating the stiffness was due to the use of rectangular elements, which are not capable of capturing the P- Δ effect arising from the displacement of the axial load. The concrete elements were changed to quadrilateral elements and this resulted in a reduction of the stiffness; the larger the percentage of bar area reduction, the more abruptly the stiffness reduced. The final version used in Model V4 is shown in Figure 6.5 (c) with a 10% reduction in area. This change in reinforcement area ultimately improved the response of the model and captured a failure mechanism that was similar to that observed during testing.

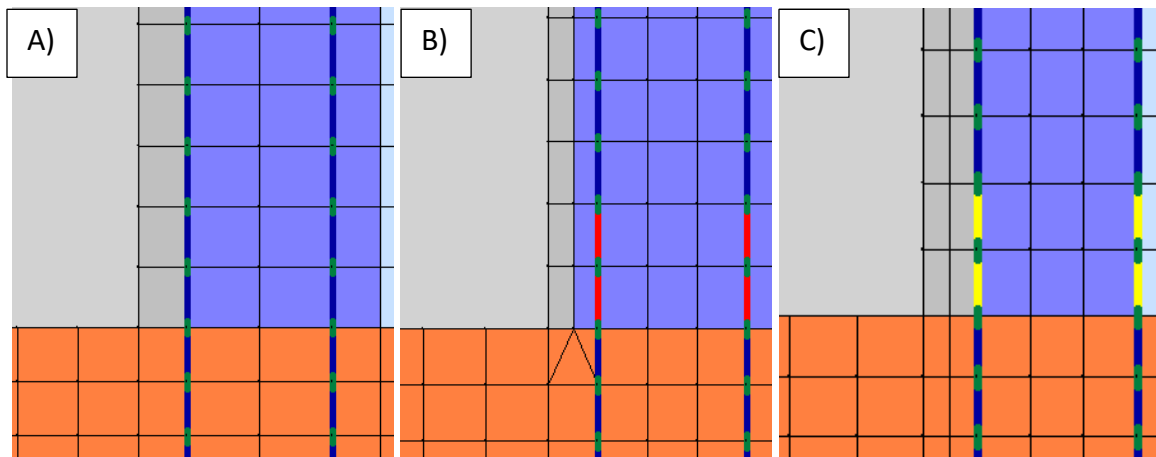


Figure 6.5: Bar Reduction: (a) 0% Reduction; (b) 12.5% Reduction; and (c) 10% Reduction

6.3.2 Modelling Parameters

The initial models were run using the default constitutive models with VecTor2. After some preliminary analysis, two of the default models were updated. Shown in Table 6.1, the

Concrete Tension Stiffening and the Concrete Hysteretic Response Models were changed. The default model for Tension Stiffening is the Modified Bentz 2005 Model and for the Concrete Hysteretic Response is the Nonlinear w/ Plastic Offsets. Tension Stiffening was updated to Modified Bentz w/ Local Fracture and Hysteretic Response was changed to Palermo 2002 (w/ Decay). Changing these constitutive models resulted in the model failing at a drift of 3.0%, specifically the Tension Stiffening Model. Prior to making these changes, as illustrated in Figure 6.1, the original model failed at approximately 5% drift.

Table 6.1: Modelling Parameters Used

Concrete Models	
Compression Pre-Peak	Hognestad (Parabola)
Compression Post-Peak	Modified Park-Kent
Compression Softening	Vecchio 1992-A(e1/e2-Form)
Tension Stiffening	Modified Bentz w/ Local Fracture
Tension Softening	Nonlinear (Hordijk)
FRC Tension	SDEM - Monotonic
Confined Strength	Kupfer / Richart
Dilation	Variable - Isotropic
Cracking Criterion	Mohr-Coulomb (Stress)
Crack Stress Calc	Basic (DSFM/MCFT)
Crack Width Check	Agg/2.5 Max Crack Width
Crack Slip Calc	Walraven
Creep and Relaxation	Not Considered
Hysteretic Response	Palermo 2002 (w/ Decay)
Reinforcement Models	
Hysteretic Response	Bauschinger Effect (Seckin)
Dowel Action	Tassios (Crack Slip)
Buckling	Akkaya 2012 (Modified Dhakal-Maeka)
Bond Models	
Concrete Bond	Eigehausen
Analysis Models	
Strain History	Previous Loading Considered

Strain Rate Effects	C: n/c S: n/c
Structural Damping	Not Considered
Geometric Nonlinearity	Considered
Crack Spacing	CEB-FIP 1978 - Deformed

6.4 Organization of the Models

Smeared reinforcement within the concrete elements refers to regions of well distributed reinforcement. With this approach the reinforcement is averaged over the elements. Smeared reinforcement is modeled as perfectly bonded to the concrete, and can be set in X (horizontal), Y (vertical) or Z (out-of- plane) directions. Discrete reinforcement refers to specifically defined regions of reinforcement. In the model it is defined as linear truss bar elements which have the properties of the reinforcement within its location. The discrete reinforcement can be assigned different bond properties based on the surrounding confinement pressure, or to account for possible slip. For these models, the transverse reinforcement (stirrups and ties) was modelled as smeared reinforcement and only the longitudinal bars within the wall were modeled as discrete reinforcement.

The models were created through the following process. First, the actual constructed walls were used to define the concrete regions. The concrete regions were refined given that not every location within the wall contained the same amount of smeared reinforcement. The discrete reinforcement was defined next. The locations of all the vertical reinforcement were retrieved from the AutoCAD drawings and then defined within the model. The mesh was defined after this point. The discrete reinforcement causes a redefinition of the nodes of elements. No nodes are allowed to pass through the discrete reinforcement, instead, the nodes of the concrete elements must coincide with the nodes of the discrete truss bars. After the model was created, the discrete bars were redefined to reduce any triangulations of the mesh.

6.4.1 Prior to Experimental Testing

Beginning with the concrete regions, the model had six concrete regions as shown in Figure 6.6. Focusing primarily on the wall, there were four regions that governed the model, which are identified in Table 6.2.

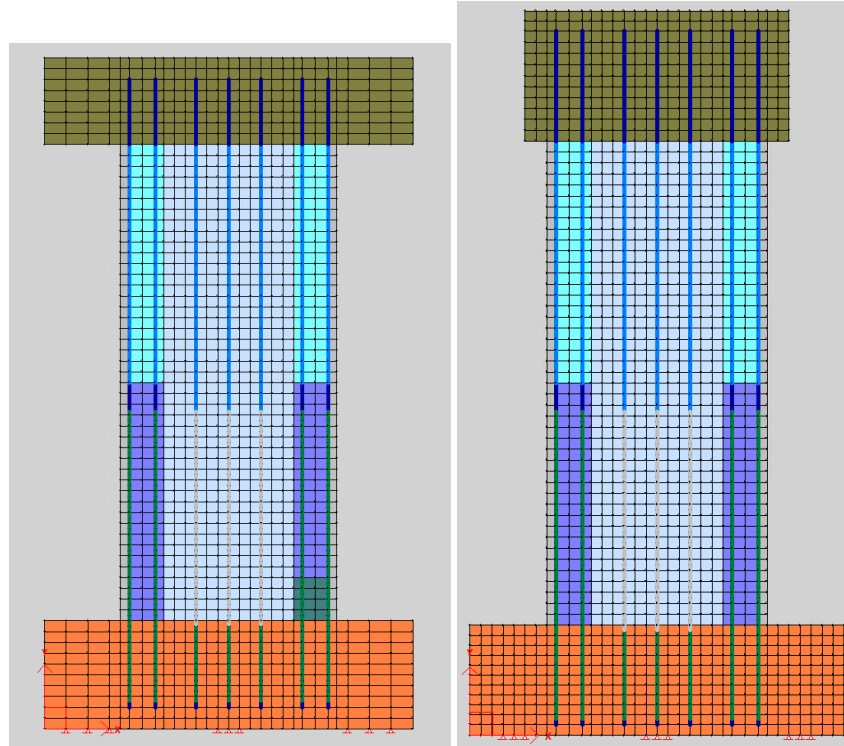


Figure 6.6: Original Model (left), New Model V1 (right)

Table 6.2: Region Dimensions for VecTor2 Model

Region in Model	Height [mm]	Length [mm]	Smeared Reinforcement Ratio, X/Z
Boundary Cover	2200	40	X = 0%, Z = 0%
Boundary Region (plastic hinge)	1100	160	X = 2.67%, Z = 1.78%
Boundary Region (above plastic hinge)	1100	160	X = 1.78%, Z = 0.89%
Web	2200	600	X = 0.89%, Z = 0%

All regions identified in Table 6.2 were 150mm in thickness. The first regions were the cover on either side of the exterior boundary reinforcement and these regions were 40mm in length and 2200mm in height. The second region was the bottom half of the boundary region, which had a higher volume of smeared transverse reinforcement due to the closer spacing of the anti-buckling ties, and this region was 160mm in length and 1100mm in height. The third region was the top portion of the boundary zone, with the same dimensions as the lower boundary but a lower volume of smeared reinforcement. The fourth region of the wall was the web; this region

only had smeared reinforcement in one direction since due to the presence of the stirrups that wrapped around the boundary region. This region was 600mm in length by 2200mm in height.

The model was redefined one more time after testing, and this is summarized below. The discretized reinforcement used within the model was located at the specific location of the longitudinal reinforcement within the boundaries and web. Since the model is two-dimensional, one truss bar element was used to represent all the reinforcement within the thickness of the wall at the location of the reinforcing bar. In this model, at the location of each longitudinal reinforcing bar, one truss bar was used to model two 10M bars. The truss bars spanned from the base foundation to the middle of the cap beam. To modify the bond properties of the truss elements, imperfect nodes were added at the top of the plastic hinge region to allow for the bond constitutive model to be applied to the truss elements within the plastic hinge region. If these imperfect bonds were not included, the model would assume perfect bond along the entire length of the truss element which is undesirable. The discretized trusses were located within the foundation, cap beam, boundary regions and web. The exterior boundary bars were located 40mm from the edges of the wall. The interior boundary bars were located 120mm from the exterior boundary bars. Within the web, one bar was located in the middle and the other two bars were spaced 150mm on either side. This was to match as accurately as possible the spacing of the reinforcement within the wall. All truss bars were assigned the area of two 10M bars. The bars located in the boundary region were subjected to higher confinement than the bars within the web.

The finite element mesh consisted of plane stress rectangular elements. The rectangles provide a less stiff element compared to the constant strain triangle and thus are better for modelling reinforced concrete [42]. The accuracy of the rectangular elements is better when squares or rectangles with ratios of 3:2 or less are used and degrade when the ratio is larger than 3:2 [42]. This can have an impact on the analysis results. In addition, a finer mesh provides better accuracy, however, there is a point where the computational power required to run the analysis is too great and is undesirable. For these models, the average mesh size where possible was 50mm by 50mm. This mesh was selected by Morcos [3] as it provides adequate accuracy for localized stresses and aids in isolated singular cracks. Further refining this mesh size would likely

exceed the number of elements that the program permits and would also not provide any notable improvement in the results. The locations where the mesh size was less than 50mm by 50mm were specifically over the zones defining the concrete cover.

6.4.2 After Experimental Testing

Changes made to model in Figure 6.6 are illustrated in Figure 6.7, and the changes made to the model initially defined in Table 6.2 and redefined in Table 6.3. As noted in Section 6.3, the observed differences between Model V1 in Figure 6.6 and Model V3 in Figure 6.7 are the clear cover lengths. Also noted in Section 6.3, in Figure 6.7 the update was the removal of the triangular mesh elements.

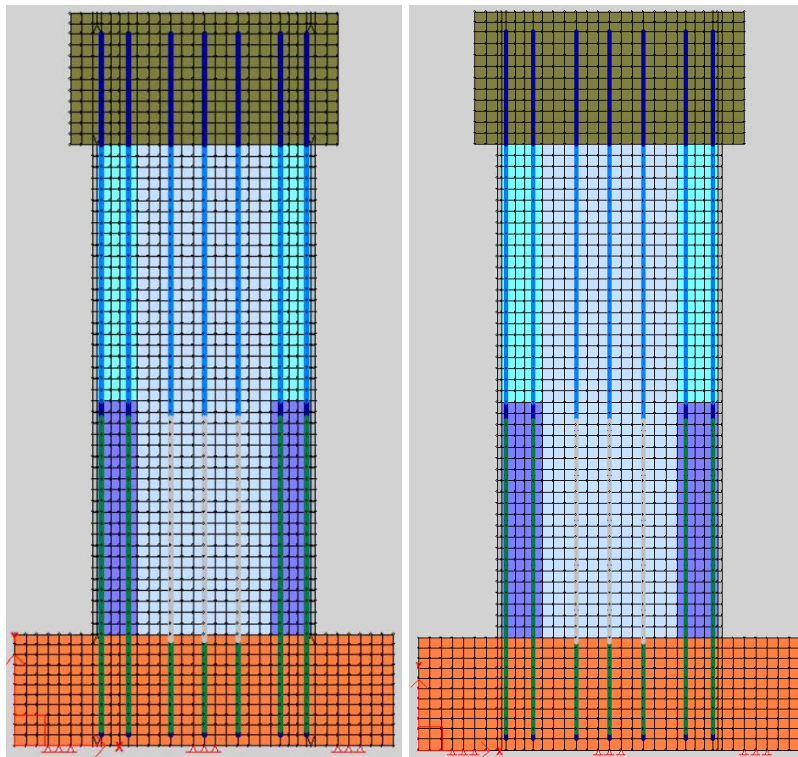


Figure 6.7: Model Created After Experimental Testing, V2 (left), V3 (right)

Table 6.3: Updated Region Dimensions and Smeared Reinforcement Ratios

Region in Model	Height [mm]	Length [mm]	Smeared Reinforcement Ratio, X/Z
Boundary Cover	2200	20	X = 0%, Z = 0%
Boundary Region (plastic hinge)	1100	180	X = 2.67%, Z = 1.48%
Boundary Region (above plastic hinge)	1100	180	X = 1.78%, Z = 0.75%
Web	2200	600	X = 0.89%, Z = 0%

6.4.3 Final Iteration After Testing

As noted above, the models were continuously updated to provide a response that was in satisfactory agreement with the experimental results. For reasons that have been noted in the discussion and in this chapter, the numerical model was not able to capture the external forces imposed by the lean of the wall and the P- Δ effect. However, the final iteration (Model V4) was able to capture the stiffness degradation and better represents the tested wall.

As discussed in Section 6.3, changes were made to the boundary regions, concrete cover, web length, and smeared reinforcement ratios. Listed in Table 6.4 are the updates to Table 6.3. One distinct change which is shown in Figure 6.8 is the protrusion of the cap beam, which is also shown in Figure 6.9. This protrusion was included for two reasons: first was to more realistically capture the P- Δ effect of the axial load which is described in detail in Section 6.6; second, also discussed in Section 6.6.1.3 is to account for the lateral load component from the vertical actuator. On the top of this protrusion (Figure 6.9) is a lateral force of 0.338 kN. This force scales linearly with the level of lateral displacement of the wall that was imposed by the horizontal actuator. Similar to testing, the larger the lateral displacement, the larger the lateral force that was applied. The slope of the relationship between the lateral load from the vertical actuator and the lateral displacement (Lateral Load Correction from Section 5.4) is 0.270. When multiplied by 1.25mm, the displacement used in the models to simulate the lateral loading, the additional lateral load from the vertical actuator is 0.338 kN, which is shown in Figure 6.9. This captured the added lateral load that arose during testing. In addition, the vertical loading was imposed by the vertical actuator was spread across the top of the protrusion.

Table 6.4: Finalized Model Regions and Smeared Reinforcement Regions for Model V4

Region in Model	Height [mm]	Length [mm]	Smeared Reinforcement Ratio, X/Z
Boundary Cover	2200	40	X = 0%, Z = 0%
Boundary Region (plastic hinge)	1100	120	X = 2.67%, Z = 2.22%
Boundary Region (above plastic hinge)	1100	120	X = 1.78%, Z = 1.11%
Web	2200	680	X = 0.89%, Z = 0%

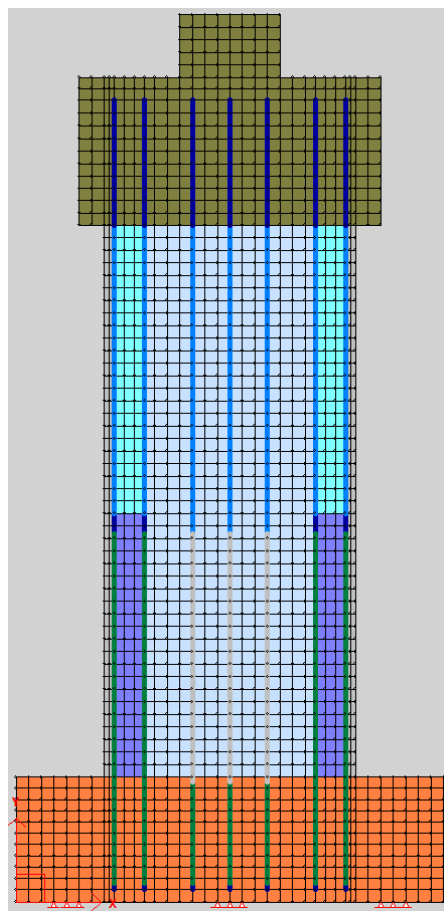


Figure 6.8: Finalized Model V4

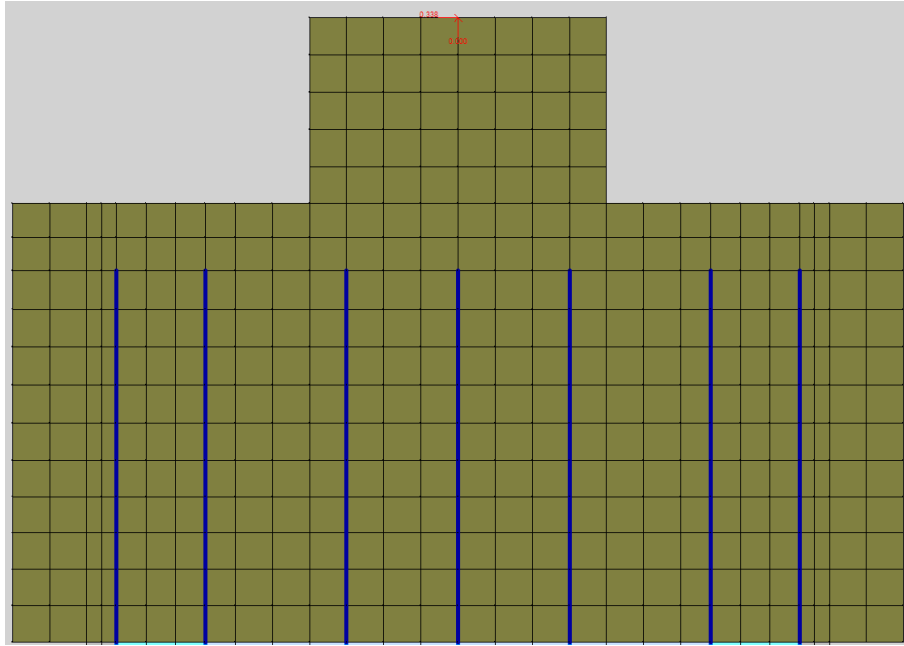


Figure 6.9: Updated Cap beam for Model V4

6.5 Finite Element Analysis Results

From the initial models, it was evident that the presence of axial loading on the shear wall improved the recentering ability. The higher the axial load, from 0% - 10%, the higher the recentering. This is visually apparent by the pinching in the lateral load-displacement responses or by the amount of residual drift that was present at the end of each positive and negative loading cycle. Another result observed, and as expected, is that the lateral load resistance increased as axial load increased. This was also observed through the higher lateral force required to reach yielding and failure in the models with each axial load increase.

6.5.1 Load Displacement Response

The models simulated combined lateral reverse cyclic loading and axial loading. The lateral loading was located at the center of the cap beam, 2495mm from the base of the wall. The axial loading was located across the width of the actuator head, 400mm wide, along the protrusion on top of the cap beam as shown in Figure 6.9. The added lateral load was also located in the middle and top of this protrusion.

A monotonic pushover analysis was conducted to determine the yield displacement of the wall. The pushover considered the combined lateral and axial loading. It was run until rupture

of the tension side reinforcement. Using the pushover results, the yield displacement was determined to be 6.83mm (0.274% drift). The yield force was approximately 169kN. The method for determining the yield point, the Park method, is shown in Figure 6.10. The Park method was used as defined in Section 4.4.1.1.

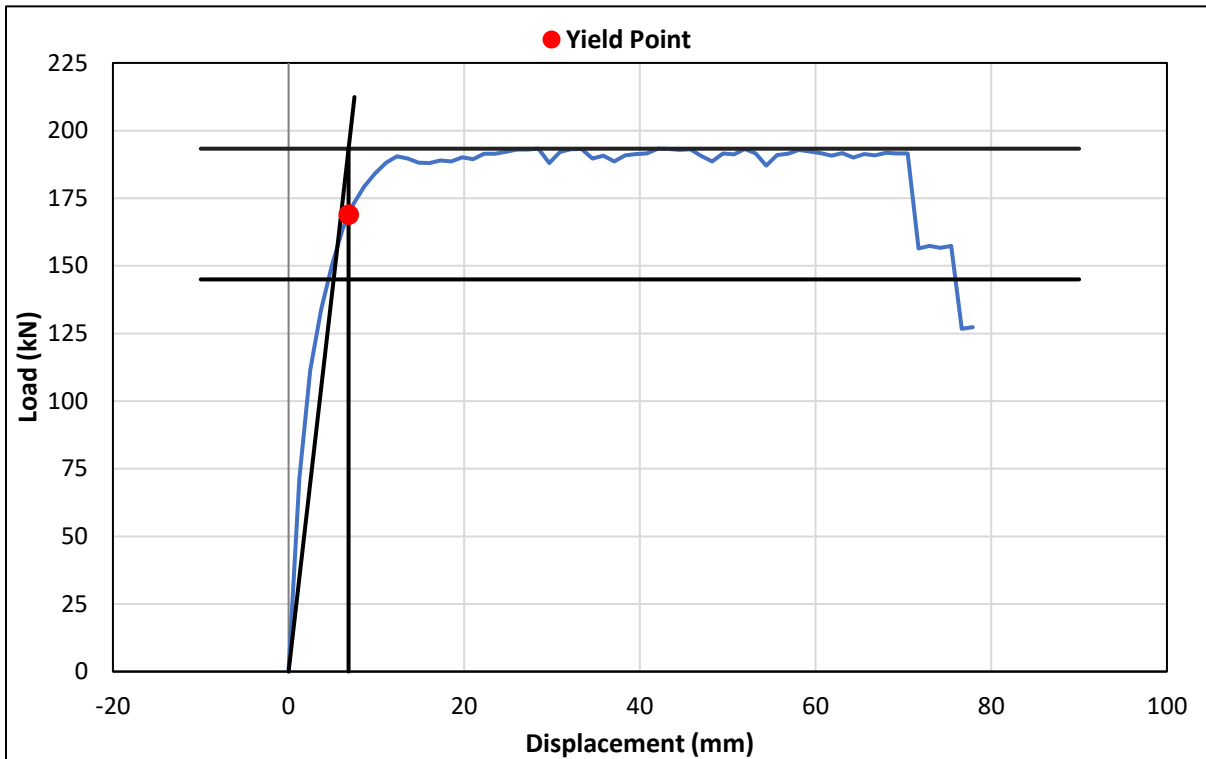


Figure 6.10: Yield Point Derivation from Pushover Analysis

After which, a reverse cyclic analysis was used to simulate a seismic event on the various shear wall models. The reverse cyclic loading protocol was similar to that used during testing, with 3 repetitive cycles up to 25mm (1.0% drift) followed by two cycles thereafter. Using the reverse cyclic results, the yield displacement for the wall was 7.10mm (0.285% drift) and the yield force was 170kN. The method for determining the yield point for the cyclic response is shown in Figure 6.11, using the Park method as defined in Section 4.4.1.1.

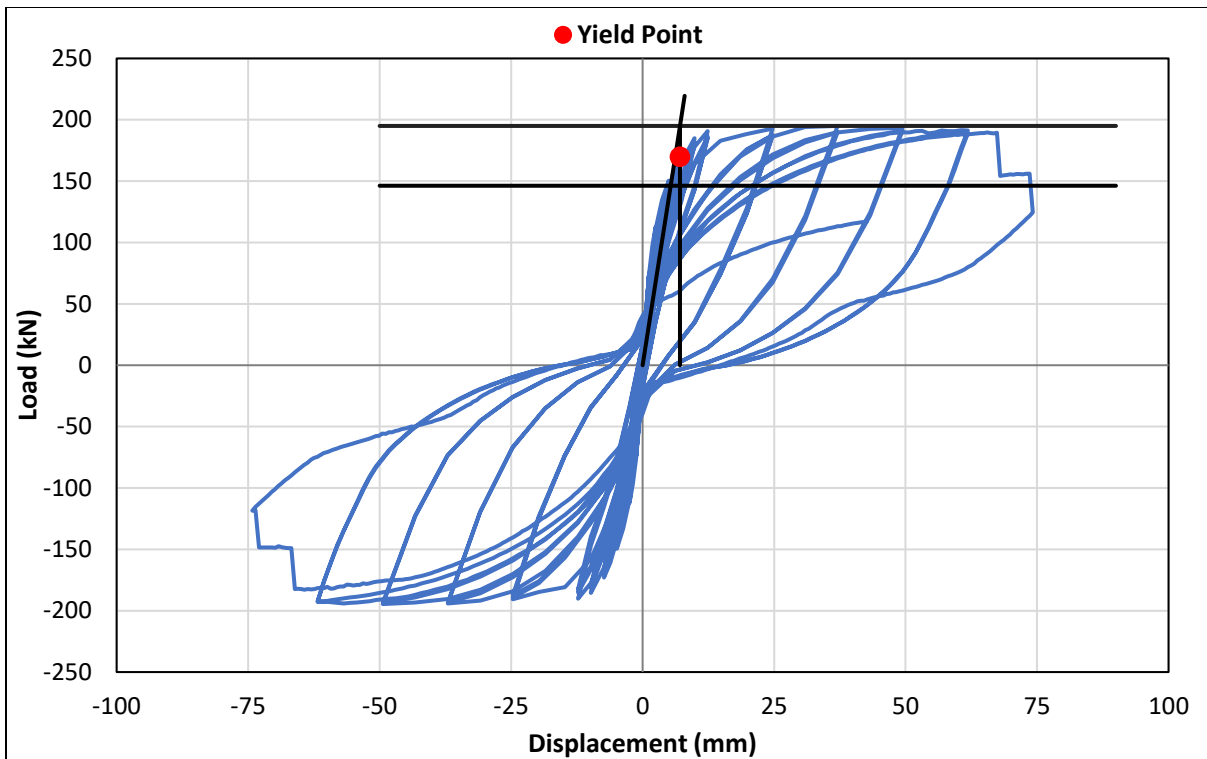


Figure 6.11: Yield Point Derivation from Reverse Cyclic Analysis

6.5.2 Comparison to Experimental Results

The reverse cyclic response exhibited key behavioral features which were present in the experimental response. There was notable pinching between the positive and negative cycles. There was a stiffer response during the first cycle of a new displacement and then subsequent cycles of the same displacement resulted in stiffness degradation visible by a reduction in required force.

It should be noted that there was one distinct difference between the FE model and the experimental test. The major difference was how the axial loading was applied. In the FE model, the axial load maintained a constant load perpendicular to the cap beam, regardless of the rotation of the cap beam. During testing, the actuator head remained perpendicular to the cap beam; however, the axial load also applied added lateral forces due to the rotation of the cap beam. These added lateral loads were initially not captured within the models. They were included in the development of Model V4 (Figure 6.8 and Figure 6.9). In Model V3, after 1.0% drift, there were discrepancies in the analysis results (see Section 6.6.2). The final model, Model V4, was nearly identical to the experimental results up to 0.5% drift on the positive side and up

to 2.0% drift on the negative side. The most noticeable difference between the model and the experimental results are the degradation of the stiffness during larger displacements (see Figure 6.12). The stiffness degradation of the model is more gradual than that of the experimental results for the positive loading of Figure 6.12, but is quite similar until 2.0% drift for the negative loading. A significant difference between the model results and experimental testing was the unloading and residual displacement. The model predicted greater recovery, leading to smaller residual displacements and greater pinching. This difference was further investigated in Section 6.6.1.5.

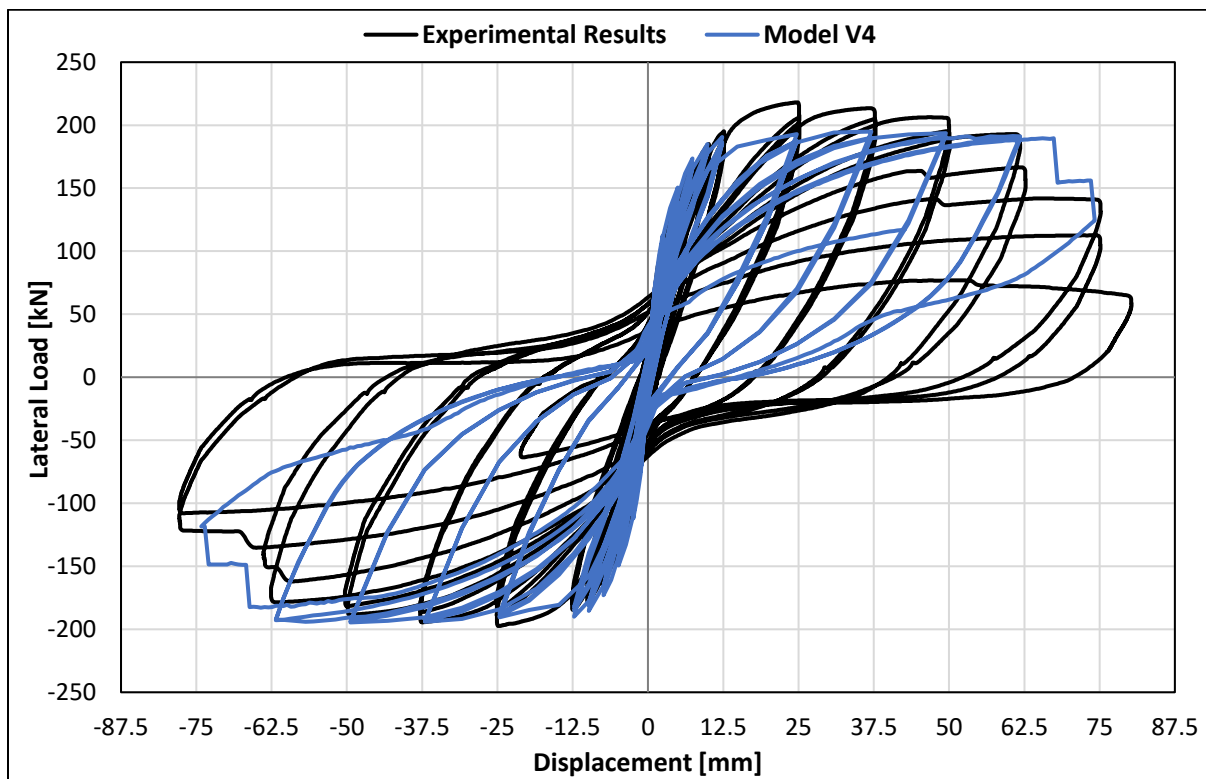


Figure 6.12: Hysteretic Response from Experiment Testing and Model

From the experimental results, the wall had a higher maximum load of 218.13kN on the positive side, and -197.49kN on the negative side, whereas the model had a maximum load of 195kN on the positive side, and -194.6kN on the negative side. The maximum lateral load from testing occurred at 25mm displacement (1.0% drift) whereas the model captured the maximum

load at 37.5mm (1.5% drift). This can likely be attributed to the way the load is applied in the model compared to how it was applied during testing.

There are no noticeable drops in the load-displacement response from reinforcement rupturing which occurred multiple times during testing; every significant drop corresponded to rupture of reinforcement. In the model, it was only captured at the end of analysis, and the reinforcement sustained more loading than during testing.

The yield stress of the reinforcement (440 MPa) was achieved at 7.5mm displacement (0.3% drift) in the model. The yielding stress in the boundary longitudinal bars was reached after all three 7.5mm displacement cycles. At 10mm displacement the yielding stress in the boundary longitudinal reinforcing bars exceeded the yield stress of 440 MPa. In the Model, the boundary bars on both sides of the wall ruptured during the last full cycle, which was the first cycle of 75mm. In Figure 6.12, there are four significant drops in the final cycle, two on either side, and these drops represented the rupturing of the reinforcement. The experimental wall, however, experienced rupture of two of the outside bars in the boundary zone during the second cycle of 62.5mm (one at each end of the wall) and the remaining two outer boundary bars ruptured during the first cycle of 75mm (one at each end of the wall). Unlike the model, only one inside boundary bar ruptured, and this was during the first 87.5mm cycle. Essentially, the model assumes the boundary bars all rupture at the same displacement, and this can be attributed to the symmetry of the model, and the lack of symmetry with the experimental wall.

In Figure 6.13, the pushover and reverse cyclic responses of the model are displayed. These results illustrate that the pushover response provides an acceptable envelope of the hysteretic curve.

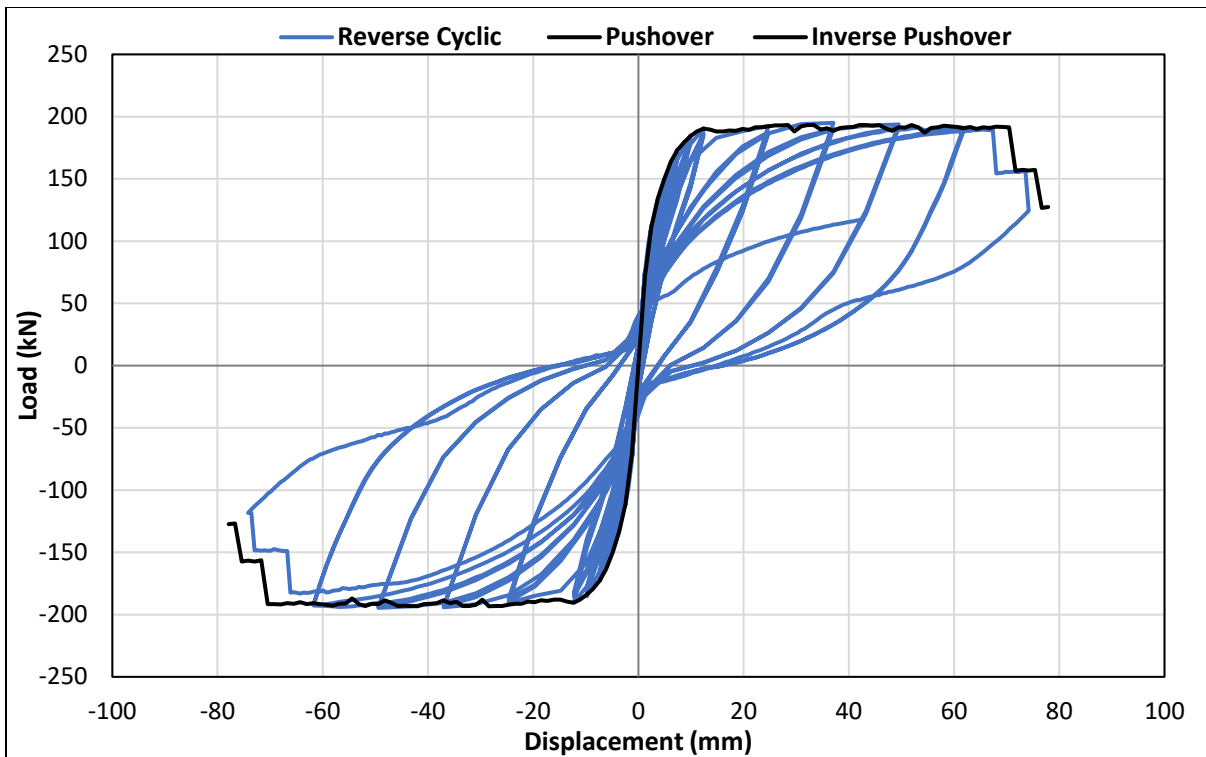


Figure 6.13: Reverse Cyclic Response compared to Pushover Response

6.5.3 Energy Dissipation

Energy dissipation is determined by the size (area within) of the hysteretic loops from the lateral load-displacement response. Refer back to Section 5.9 for the definition of how dissipated energy was calculated. The higher the axial load that was applied, the more of a pinching effect on the response. The correlation between applied axial load and energy dissipation is apparent in the model responses without applied load and with applied load (see Figure 6.2). As the axial load is increased, the width of the loops decreases and thus so does the energy dissipation capacity.

Observing the hysteretic loops for 2.0% drift in Figure 6.14, it can be noted that the experimental loops are wider than the loops of the model. The larger pinching in the model predictions resulted in less effective energy dissipation ability and a higher stored elastic potential.

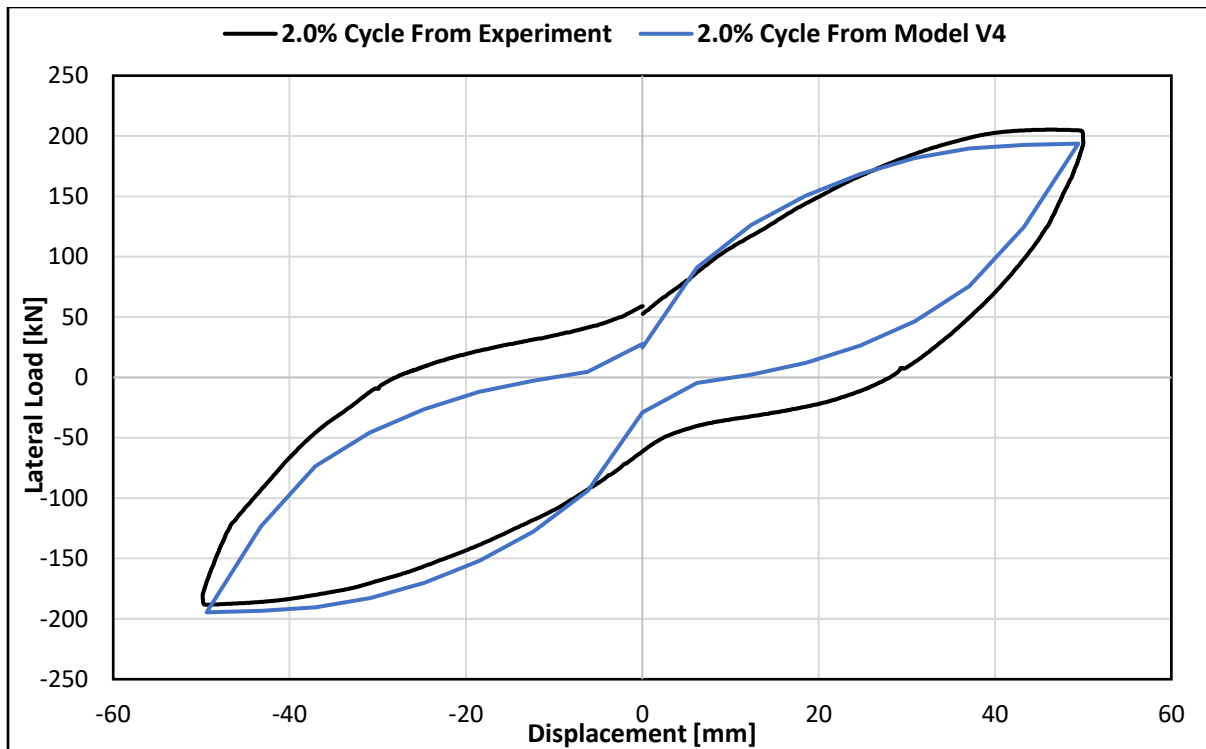


Figure 6.14: Comparison of Lateral Load-Displacement Response for Experiment and Model at 2.0% Drift

Figure 6.15 illustrates that the experimental wall had more total energy that was dissipated compared to the model. The model had more stored elastic potential energy, this is seen by the high amount of pinching and area under the descending branch of the hysteretic curve. The same is evident in comparing the positive cycles only in Figure 6.16. In Figure 6.16, at 3.0% drift, however, the model converges with the experimental results. In Figure 6.12, there is a noticeable decline in the lateral load required to achieve 3.0% drift for the experimental wall, whereas the model maintained the majority of its strength until the end of the first 3.0% cycle where it failed.

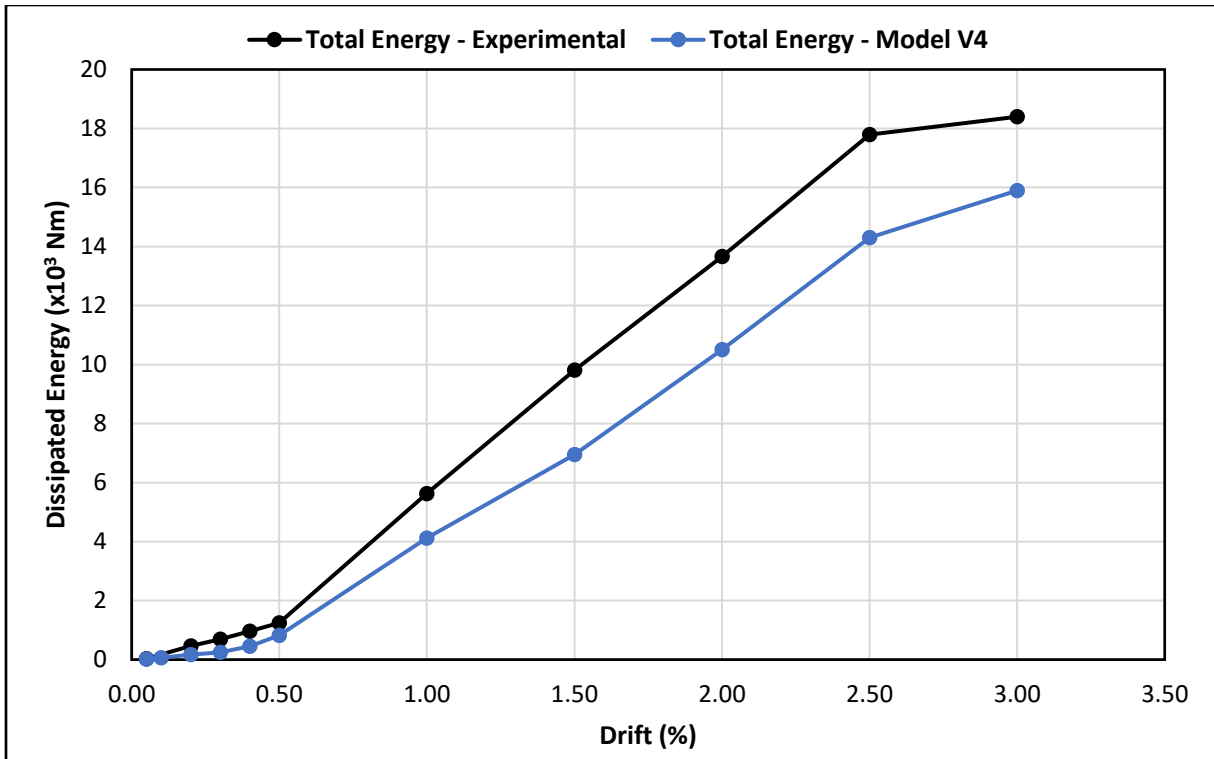


Figure 6.15: Comparison of Total Energy for Experiment and Model

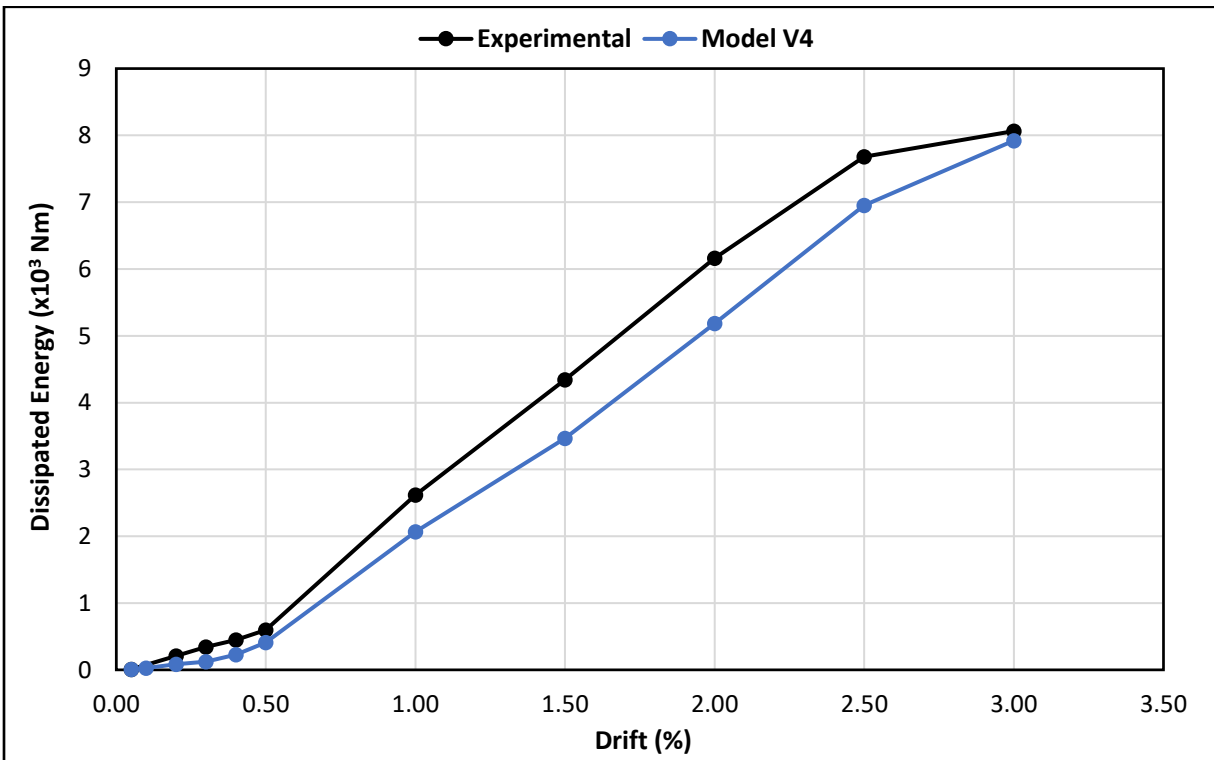


Figure 6.16: Comparison of Hysteretic Energy for Positive Cycles

In Figure 6.17, the stored elastic potential energy is shown. As higher drift levels are reached, the stored elastic potential energy increases significantly in the model compared to the experimental wall. This can be related back to Figure 6.14 and the amount of pinching that is present in the model compared to experimental wall.

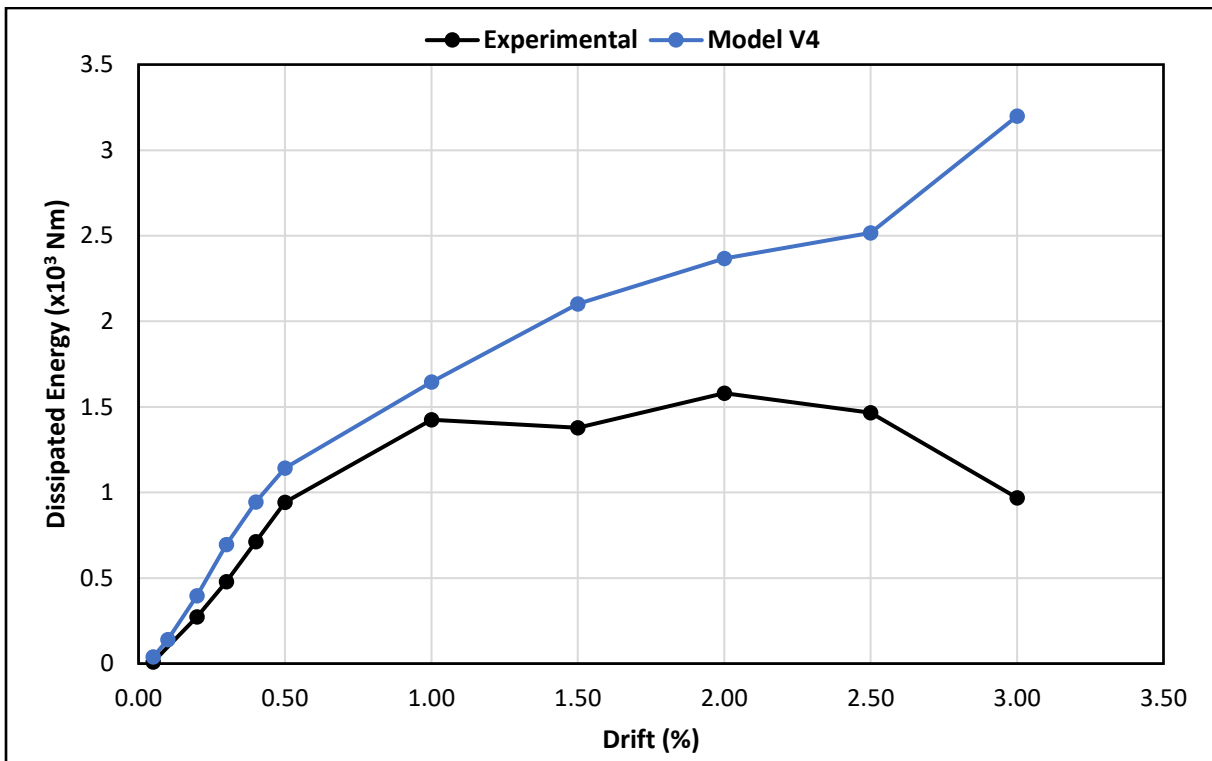


Figure 6.17: Comparison of Stored Elastic Potential Energy for Positive Cycles

6.5.4 Recentering and Recovery

The presence of axial loading on shear walls has been thoroughly discussed, however, it is noteworthy that axial loading typically improves recentering and the recovery of residual drift. In numerical models and testing without axial loading, the hysteretic loops are typically much wider. Though this signifies better energy dissipating capabilities, shear walls typically sustain both lateral loading and axial loading and this response needs to be quantified. In addition, there is significant residual displacements where axial loading is not considered. Though the wider loops refer to better energy dissipation, the ability to recover cracks and recenter is improved when axial loading is applied.

The model predicted a behaviour that was slightly different from the experimental results. One of the reasons was the way in which the axial load was applied. As previously discussed, the axial load in the experimental wall did not always remain vertical, but rather was perpendicular to the cap beam during its rotation, whereas the model maintained vertical axial loading until failure. The recentering in the model was greater with smaller residual displacement. Referring to Figure 6.14, at 2.0% drift the model recovered to approximately 10mm, whereas the experimental wall recovered to 30mm. There was greater recentering observed in the models, however it is a sign of underestimating the actual energy dissipation. The model also predicted a symmetric response, whereas to due discrepancies in construction and the test setup asymmetry arises. Note that the overestimation of recentering in the model is probably associated with local effects, such as crack closing or the accumulation of reinforcement straining at the crack. This was further investigated in Section 6.6.1.5.

6.5.5 Crack Patterns

The model predicted an array of flexural and shear cracks starting at a displacement of 1.25mm. This was earlier than when the cracks visually surfaced during testing. The cracks developed along the height of the wall, higher above the base than observed in the experimental wall. The cracking within the model was concentrated within the bottom half the wall, with wider cracks forming at the base as early as 0.5% drift (12.5mm displacement).

In terms of identifying specific cracks, the model did predict and replicate the increased damage sustained within the bottom 400mm of the wall. Emphasis was placed on the crack that occurred at the base and foundation interface. This crack propagated during testing and was captured by the model. The model also demonstrated cracking extending to a maximum height of 1750mm above the base, however, during testing, the maximum height reached of cracking was only 1400mm above the base. The model accurately replicated the crack directions. Figure 6.18 illustrates that all cracks were relatively horizontal and did not propagate through the length of the web at 0.2% drift, which was similar in the model.

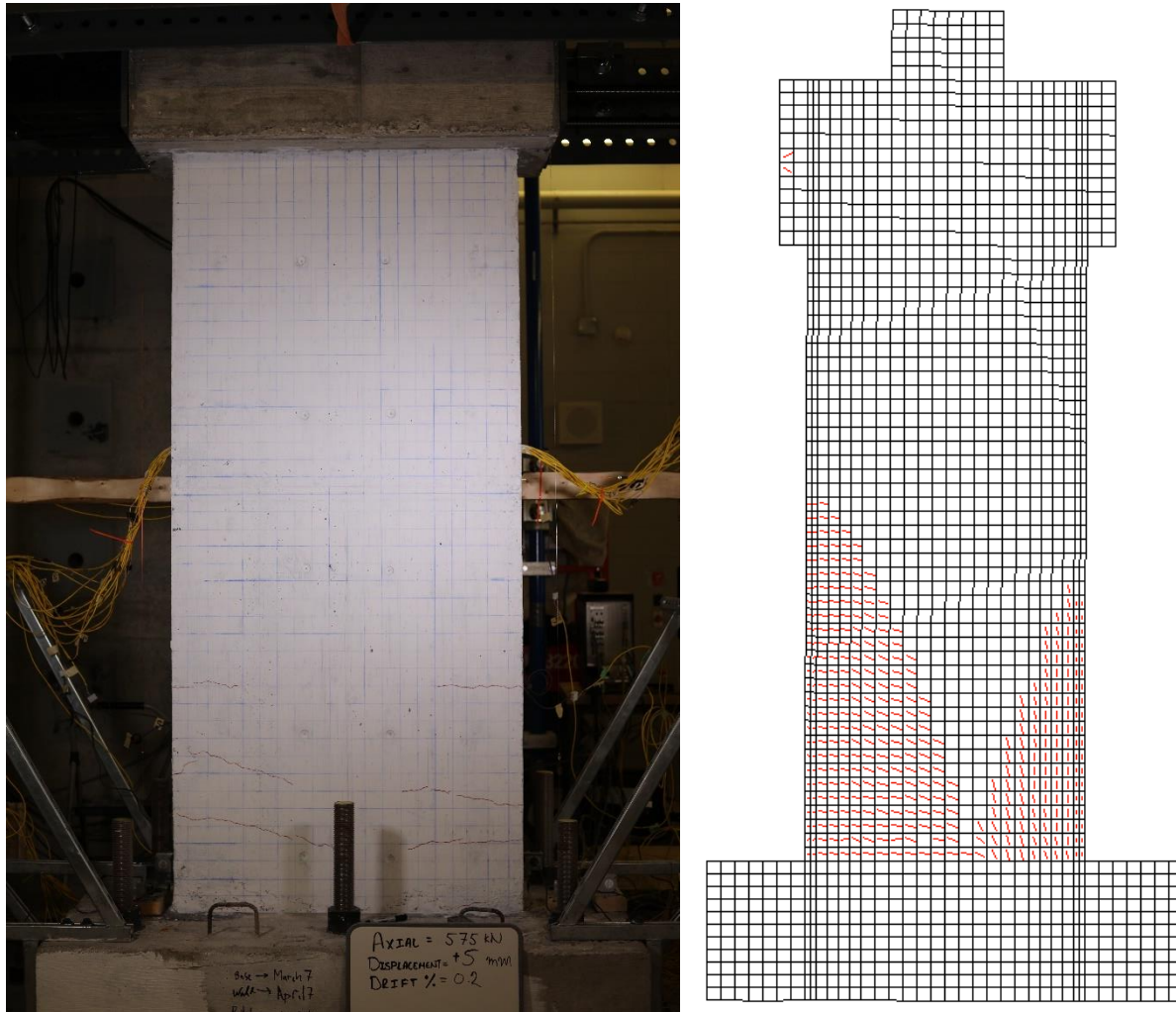


Figure 6.18: Cracking Comparison at 0.2% Drift

As seen in Figure 6.19, the cracks appear horizontal through the boundary regions and then extend diagonally through the web region. As cracks appear in the web, they propagate at various angles. The lower the crack in the wall, the more acute the angles appear; the higher in the wall, the steeper the cracks become in the web.

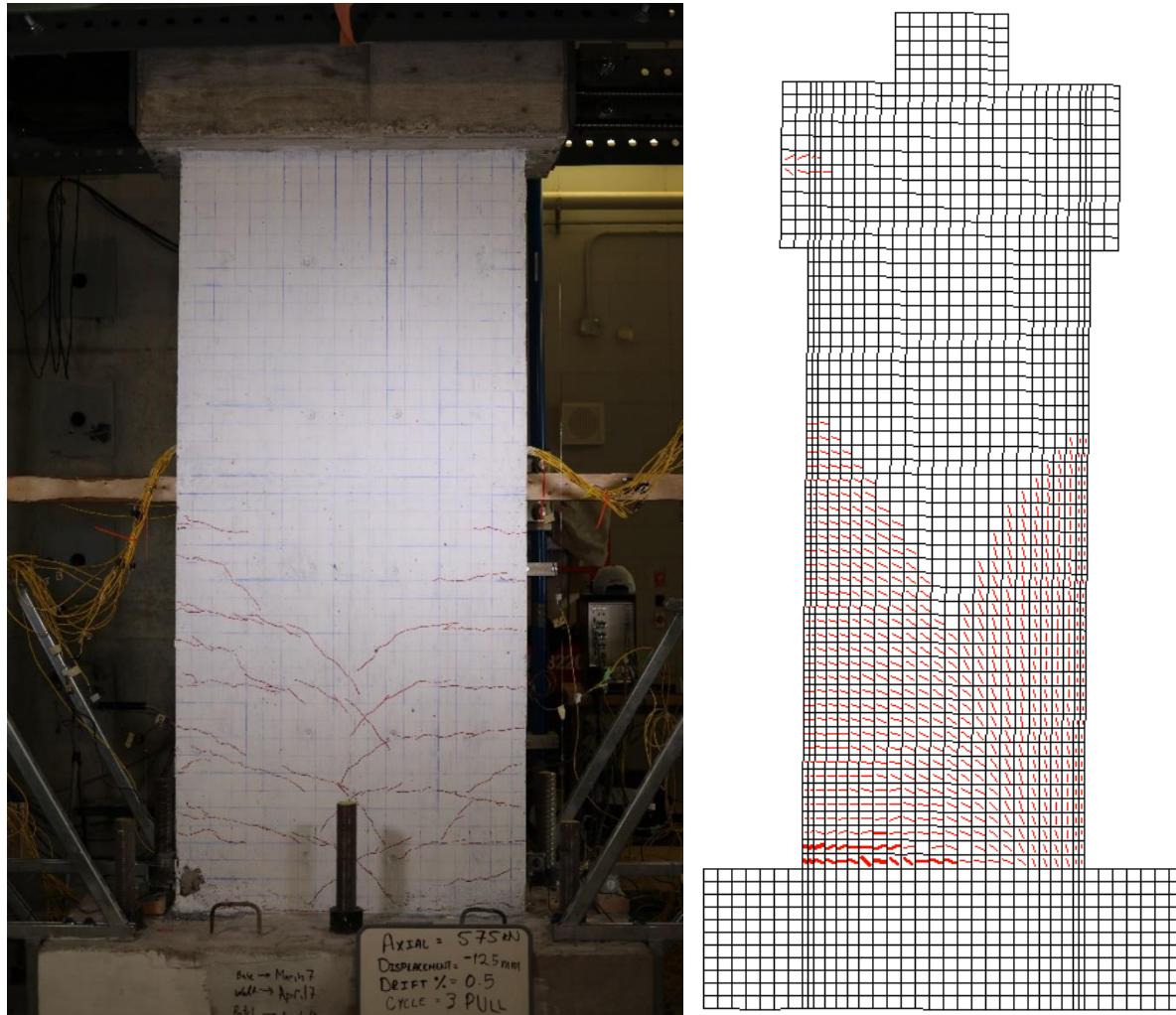


Figure 6.19: Cracking Comparison at 0.5% Drift

Figure 6.20 illustrates that the steepness of the cracks in the web at 1.0% drift are not as severe as the steepness of the cracks in the web for 2.0% drift (see Figure 6.21). The steepness of the cracks in the web noticeably increase from Figure 6.18 to Figure 6.19, to Figure 6.20 and to Figure 6.21 in both the experimental wall and the model. Overall, the model satisfactorily predicted the propagation of cracks throughout the loading.

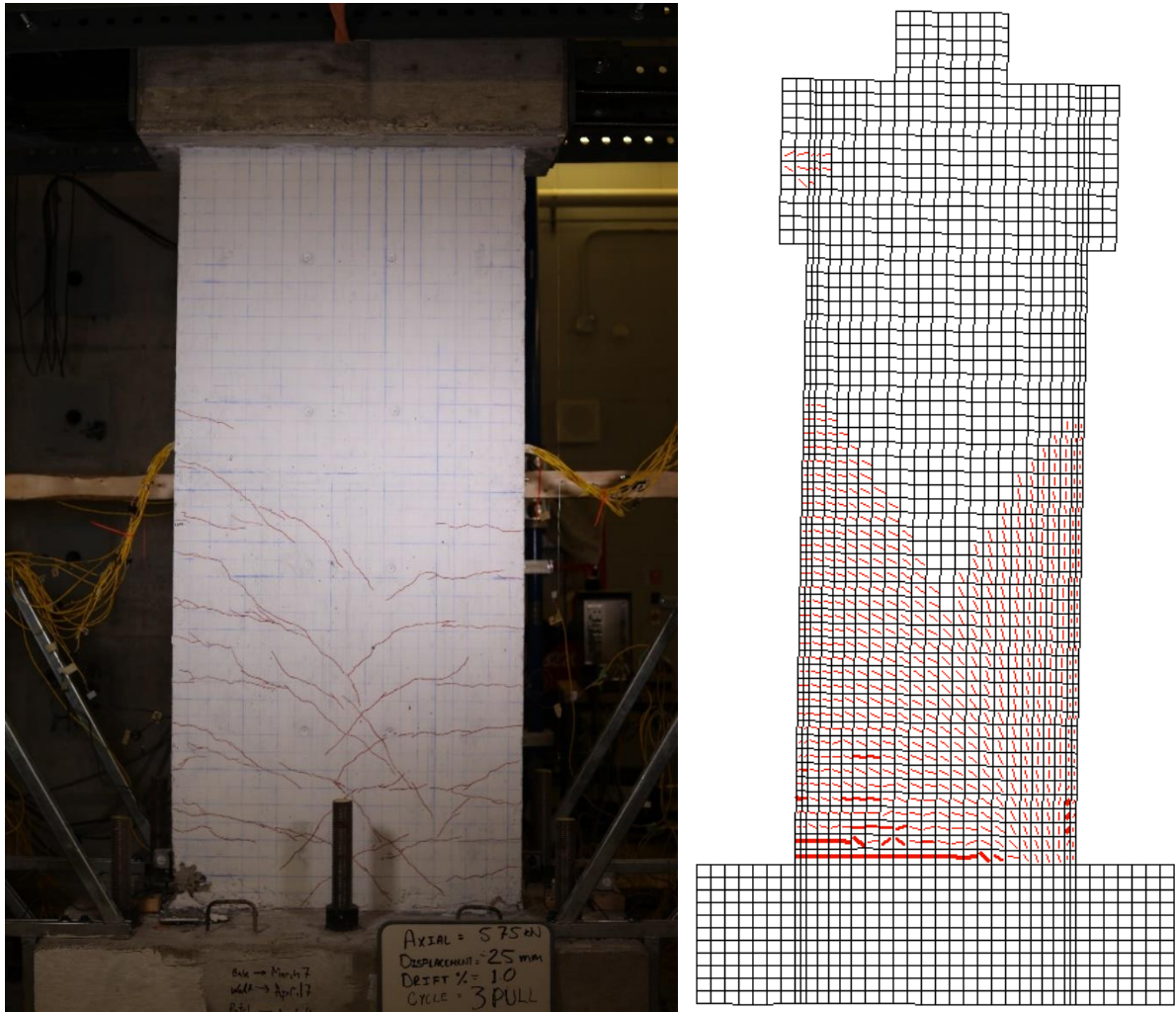


Figure 6.20: Cracking Comparison at 1.0% Drift

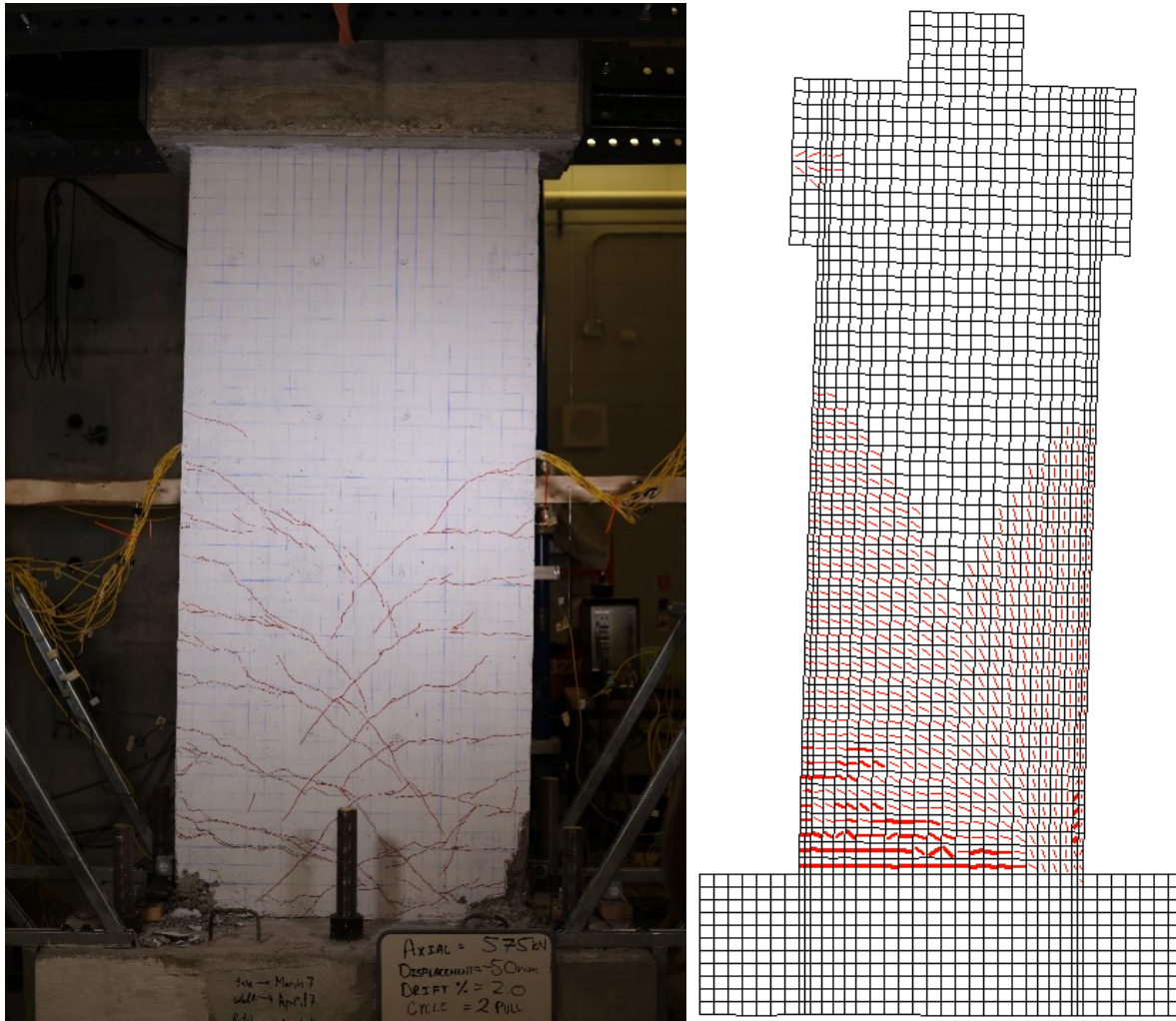


Figure 6.21: Cracking Comparison at 2.0% Drift

6.6 Discrepancies, Errors, and Modelling Discontinuities

Throughout the finite element modeling process a variety of models were constructed. Modifications included the application of the loading, the diameter of the reinforcing bars in the boundary regions, and the length of the boundary areas. Some of the default material models were changed and these changes did not result in significant improvements in the initial models. The initial models (V1, V2, and V3) did not accurately predict the walls behaviour after 1.0% drift. After this point, there were some noticeable differences as the lateral load reached its peak at 3.5% drift, which was not the case for the experimental response. There were specific effects that were not captured by the initial models and this section provides a discussion to highlight them.

All the models assume the wall is symmetric and there were no construction errors. As previously discussed, there was a leveling issue with the cap beam during casting due to its weight. This leveling issue created a tertiary out-of-plane load that added tensile stresses to the back side of the wall and compressive stresses to the front of the wall. It is not possible to directly capture that effect in the 2D model used in this study. This out-of-plane force increased as the test progressed and resulted in the boundary bars fracturing first on the back side of the wall and later on the front side of the wall.

Another discrepancy between the test setup and the initial models was the method used to apply the axial load. The models assumed that regardless of the level of lateral displacement and the rotation of the cap beam, the axial loading is applied vertical to the cap beam at all times. In reality, the head of the vertical actuator rotated with the cap beam and applied an added lateral load and a vertical axial load, resulting in an increase in total lateral force at each displacement level. Essentially, the axial loading did not remain in the perfect vertical position due to the lateral displacement and rotation of the cap beam. The lateral displacement of the vertical actuator resulted in the “P delta” or “P- Δ ” effect (see Figure 6.22). The initial models with rectangular elements did not capture this secondary P- Δ moment that is generated by the lateral displacement of the wall. The model with quadrilateral elements was able to account for this moment. Furthermore, given that the initial models did not explicitly model the horizontal force from the vertical actuator, the additional base moment and base shear from this effect was not captured. This was later captured in the model with the addition of the protrusion to the cap beam.

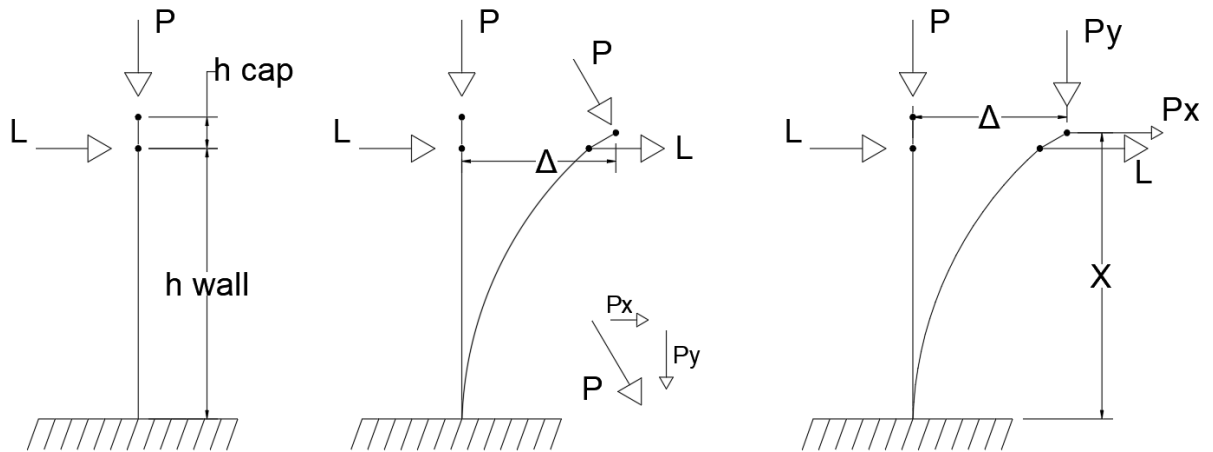


Figure 6.22: P-Δ Effect

As the lateral displacement of the wall increased, the secondary moment generated by the axial load also increased. This secondary moment was negligible at the early stages of loading but would greatly increase beyond 1.0% drift. Since this was not originally captured, the models predicted an overstrength, however, this was the result of the model not capturing the P-Δ effect and the effect of the horizontal force from the vertical actuator. The overall base moment on the wall is provided in Equation 6.1. The original models would only capture the first component of the base moment.

Equation 6.1

$$M_{Overturning} = (L * h_{wall}) + (P_y * \Delta) + (P_x * X)$$

Reference is made to the research from Rao et al., [18] where the cyclic response of their shear wall would be better simulated by the Model V3. This is due to the method for applying the axial load that they used is much closer to the way VecTor2 applies the load compared to the test setup in this research. They placed 8.5 tons directly on top of their cap beam and it maintained a constant vertical load. The head of actuator in this research was free to rotate and travel horizontally with the wall, resulting in a pendulum-like motion during testing. This motion

magnified the base moment due to the horizontal component arising from the vertical actuator and the P- Δ effect of the translational of the axial load. As noted above the modifications to the final model were intended to capture these effects.

As discussed above, another discrepancy in the initial models was not capturing the added lateral load from the rotation of the head of the vertical actuator. The experimental results were corrected and discussed in the results section of this thesis and in detail in the appendix. Although this did not have a significant impact on the lateral load resistance, there was a significant base overturning moment generated due to the long lever arm from the base of the wall to the pin joint of the head of the actuator attached to the top of the cap beam.

The out-of-plane lean in the wall also played a critical role in magnifying the stresses on the back and front sides of the wall. Shown in Chapter 4 and Chapter 5, CP15 recorded the out-of-plane displacement of the wall during testing. It was evident that during positive and negative cycles, the wall, on average, was displacing in one direction. VecTor2, as a two-dimensional software, is not able to explicitly simulate this out-of-plane displacement. Essentially, due to this action, additional tensile stresses were imposed on the back face of the wall and additional compressive stresses on the front face of the wall. During testing, the additional tensile stresses lead to the longitudinal bars in the boundary regions first rupturing on the back face of the wall.

Figure 6.23 and Figure 6.24 illustrate the out-of-plane lean in both the vertical actuator and the wall. Figure 6.24 provides an enlarged graphic of the effect of this lean and the additional moment that may be generated at the base of the wall. Equation 6.2 provides an estimate of the moments that is developed in the out-of-plane direction due to a displacement at the approximate mid-height of the wall, which would increase the tensile stresses in the reinforcement in the boundary regions on one side of the wall. This additional stress is not captured in the 2-D models of this study, is a probable reason why reinforcement fracture arose sooner during testing than it was captured in the models.



Figure 6.23: Out-of-Plane Lean of the Wall and Actuator during Testing

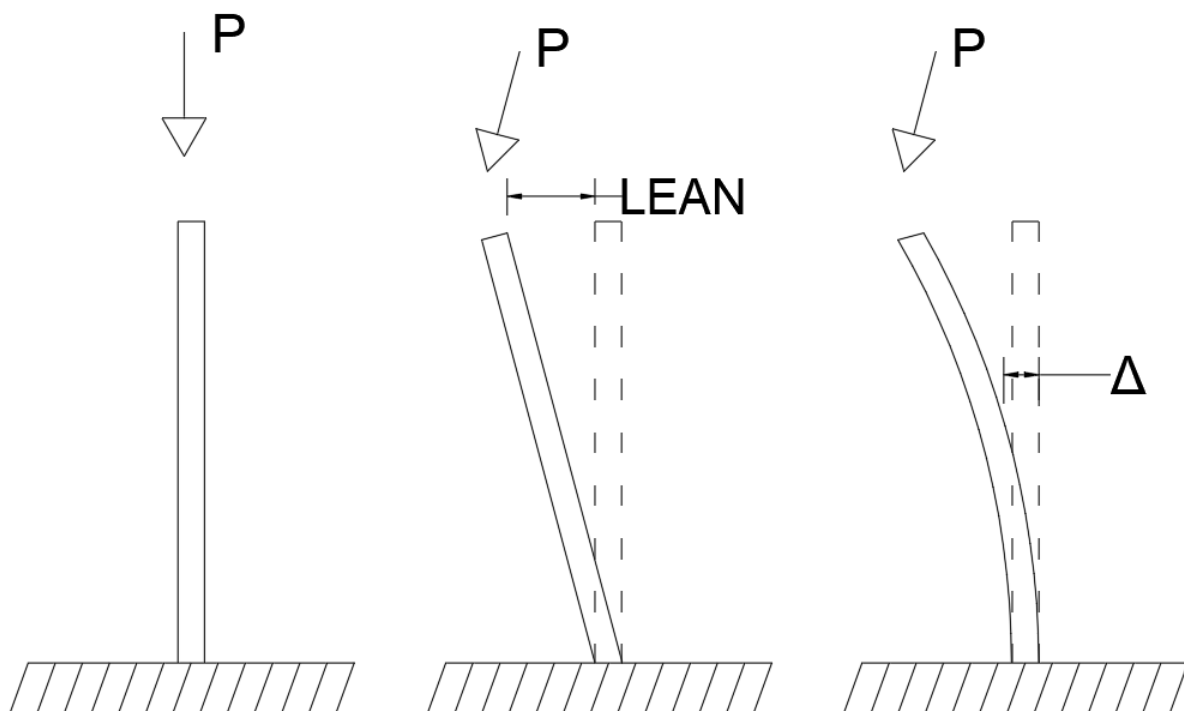


Figure 6.24: Enlarged View of Out-of-Plane Lean in the Wall

Equation 6.2

$$M = \frac{6 * EI}{L^2} * \Delta$$

6.6.1 Other Models

This section presents modifications made to the initial models (V1, V2, and V3) to attempt to improve the analysis response in comparison to the experimental response. In the following sections, discussions are provided on the significant discrepancies between the models and the experimental results and the rationales why they are not captured by the models. Multiple simulation models of this experiment were developed to try predicting the walls' response prior to testing. After comparing the pre-test predicted results and the experimental results, it was deemed that the response of the initial models (V1, V2, and V3) overestimated the capacity of the wall, however the model only exhibited overestimations after 1.0% drift. Several changes were made to the models to better replicate the experimental results; however, these changes also served the purpose of identifying different types of effects in the model. Essentially a parametric study was conducted to observe how the models were affected by changes to certain parameters and changes made to the finite element model.

6.6.1.1 7.5% Axial Load

As previously discussed, 7.5% axial loading was initially the target axial loading for testing and assumed in the modelling. The main reason for this target was due to the compressive capacity of the actuators that were considered for the test setup. During the planning, it was determined that larger actuators were going to be used for testing, which permitted the application of 10% axial loading. It also provided an opportunity to compare 10% axial loading, which is generally the high end for slender concrete shear walls, with previous testing with 0% axial loading. The 10% axial load would provide the higher degree of recentering while 0% axial load provides the higher degree of residual displacements. Key differences in the model responses from this change were less pinching, lower maximum lateral load, increased maximum

displacement, and increased residual displacements due to the lower axial loading. Refer to Figure 6.1 to view the effect of 7.5% and 10% axial loading.

6.6.1.2 Crack Spacing

A parameter that was investigated, which is not visible in Figure 6.3, was the maximum crack spacing. VecTor2 can determine the crack spacing, this is denoted by S_x and S_y . S_x and S_y are crack control parameters that indicate the spacing of cracks parallel to the y-axis (S_x) and to the x-axis (S_y) [42]. When defining concrete regions, these parameter values are automatically set to 0, which the model then identifies them as 1000mm. These input values were changed to match the width of the wall, 150mm; the manual states that if there is no in-plane reinforcement, the maximum crack spacing is considered equal to the element thickness [42]. Using 150mm instead of 1000mm resulted in a better representation of the experimental results. The purpose of making this change was to observe if the model would fail earlier due to the crack spacing change. It was observed that the model with $S_x = S_y = 150\text{mm}$ had tighter pinching in the load-displacement response and failed at 87.5mm (3.5% drift). The model with the default S_x and S_y failed at 100mm (4% drift) and had larger residual displacements. The maximum lateral force experienced by both models was identical.

6.6.1.3 Lateral Load from Vertical Actuator

To try and model the added lateral load from the axial actuator, a heavily reinforced section was added to the center of the cap beam (see Figure 6.25).

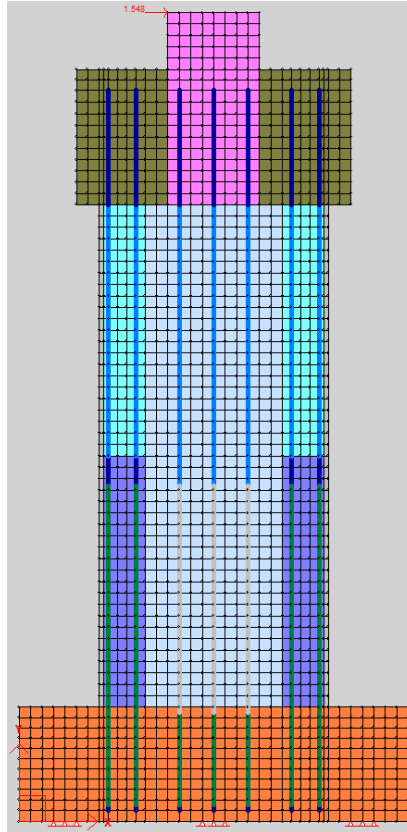
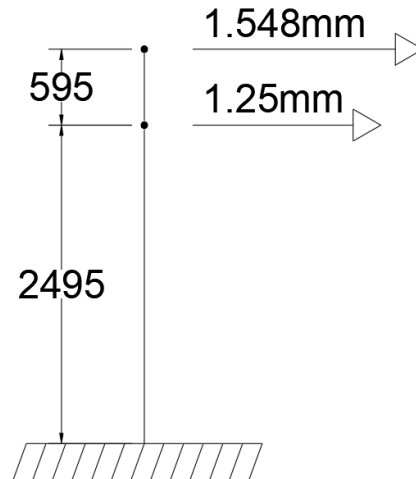


Figure 6.25: Model Simulating Actuator Head

In this model, the pink region extends 250mm above the top of the cap beam, to approximately where the pin is located on the actuator head. In the first trial model to understand the impact of this loading, rather than imposing a lateral load at the middle of the cap beam, the lateral load was applied at the top of the pink region to observe how the longer lever arm with lateral loading may affect the cyclic results of the wall. Shown in Figure 6.25 is a displacement value of 1.548mm. This value was used to ensure the center of the cap beam was displacing the correct amount. Essentially, using similar triangles, and knowing that the target displacement at a height of 2495mm was 1.25mm, the displacement at the top of the actuator head was calculated as illustrated in Figure 6.26.



$$\text{To Determine Displacement} = \frac{1.25}{2495} * (2495 + 595) = 1.548\text{mm}$$

Figure 6.26: Method to Determine Displacement Value

The results from this model, however, were inconclusive. The results provided a similar cracking pattern and the same level of pinching in the load-displacement response. Ultimately this was intended to observe the effect of the lateral load from the vertical actuator. An expected result from this model was a lower maximum lateral force due to the larger lever arm; the maximum lateral load from this model was less than 200kN, whereas the model without this modification predicted a maximum lateral force of approximately 220kN. Otherwise, there was no apparent difference in applying the lateral displacement at a higher location than the original location.

6.6.1.4 Quadrilateral Elements vs Rectangular Elements

The final modelling adjustment made to the type of elements from rectangular to quadrilateral. Visibly, the model does not appear to look any different, however, there were some notable differences in the response. The intention of using quadrilateral elements was to capture the P-Δ effect. The rectangular element model does not capture P-Δ. Unlike rectangular elements, VecTor2 divides each quadrilateral element into two constant strain triangles. The strain within one of these quadrilaterals is then the average of the strains of the two triangular elements [42]. When defining these elements, however, care must be taken to ensure the model

is either all quadrilateral elements or none, as the model will not run if quadrilateral elements are in contact with rectangular elements.

Two different models, (Rectangular and Quadrilateral) outside of V1 - V5, were built and analyzed to observe the effect of these elements. Both models were modifications of Model V3 (Figure 6.7 (right)). Both models had no reduction in reinforcement cross sectional area and were run only to compare their reactions. These were not intended to achieve the experimental response of the wall. Other reasons for these two models were to observe and compare the P- Δ effect and stiffness decay from both element types

The quadrilateral model was analyzed with the default constitutive models with the exception of the Concrete Hysteretic Response. The Concrete Hysteretic Response was changed to Palermo 2002 (w/ Decay),

From the initial comparison, shown in Figure 6.27, the model with quadrilateral elements sustained a lower peak lateral load and began to decay after reaching the peak, whereas the model with rectangular element model experienced a continual increase in lateral load to failure. The post-peak decay is representative of the P- Δ effect. Both models overestimated the ductility of the wall, however, the model with quadrilateral elements demonstrated a slow decay in response. The overestimation of the ductility in the results is because the Modified Bentz w/Local Fracture Tension Stiffening Model was not used. When the quadrilateral element model is compared to the experimental results (Figure 6.28), the model accurately simulates the results up to 1.5% drift, whereas the rectangular element model which is only accurate to 1.0% drift.

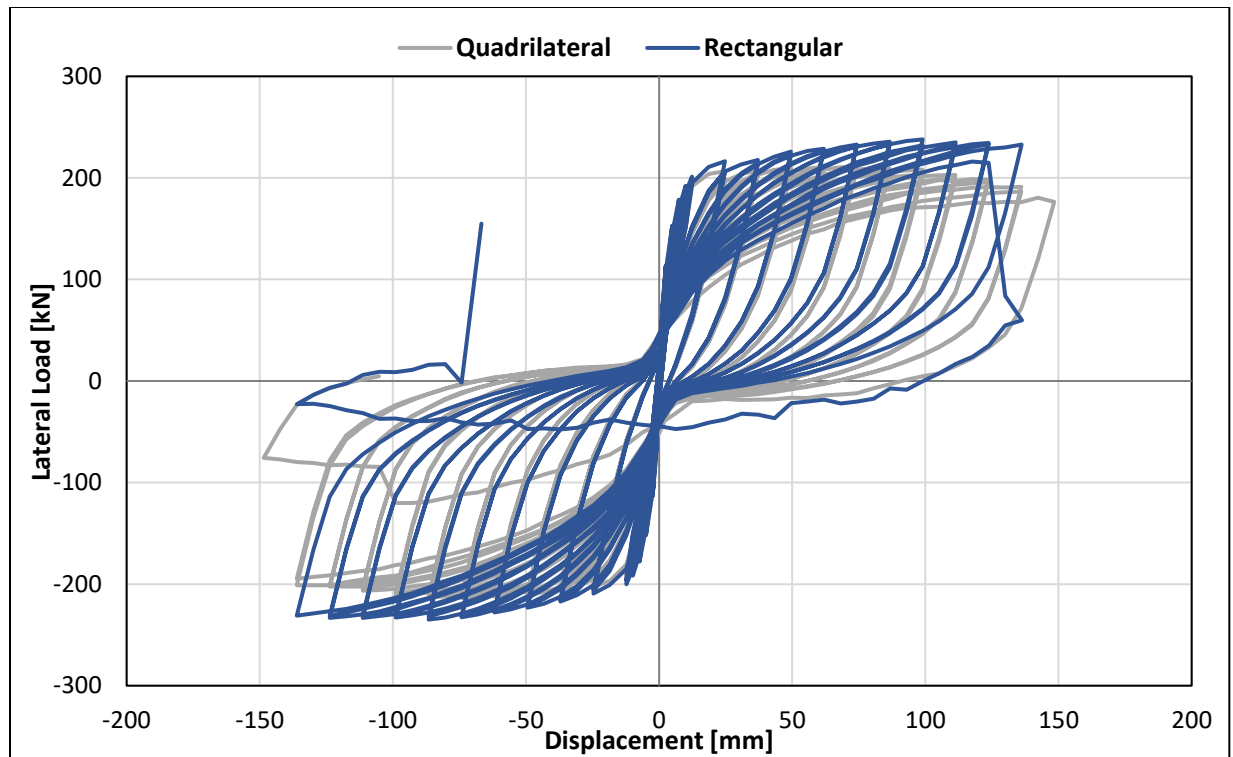


Figure 6.27: Quadrilateral Elements vs Rectangular Elements Modelling Results

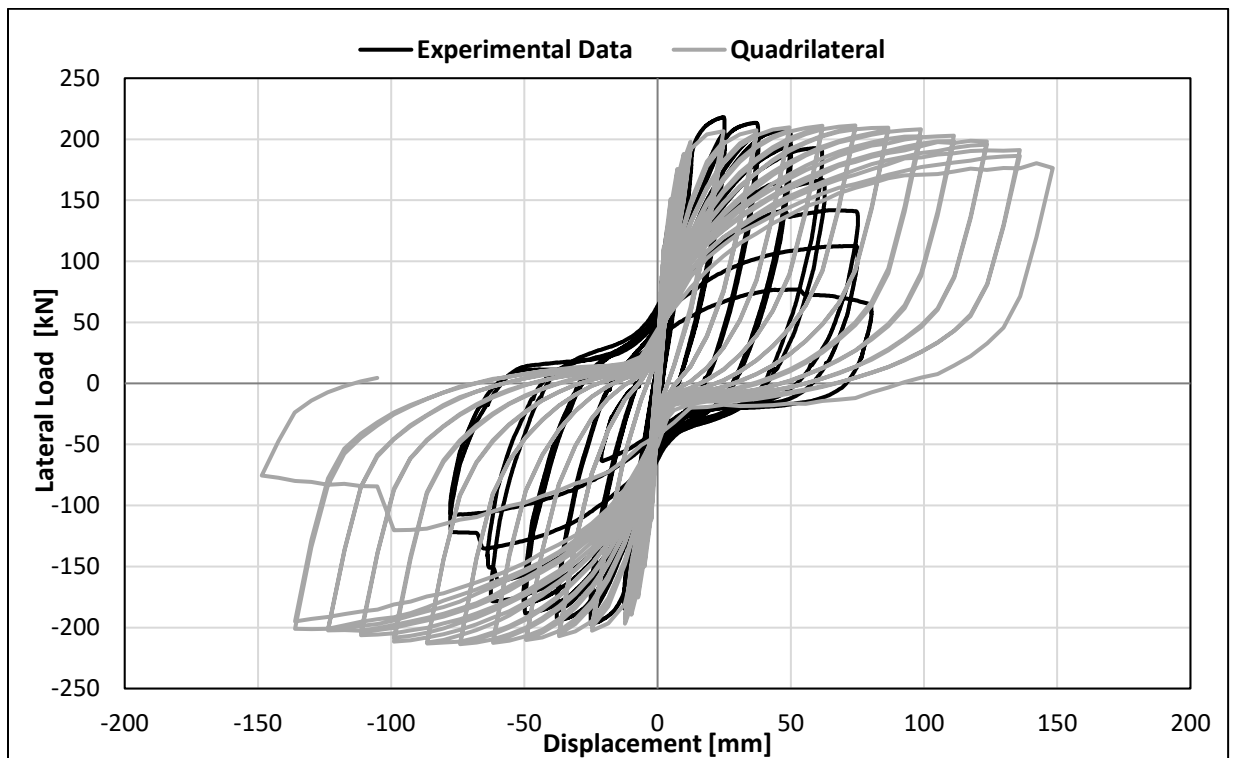


Figure 6.28: Quadrilateral Element Results vs Experimental Results

6.6.1.5 Reinforcement Models – Hysteretic Response Change – Model V5

The following model was named Model V5, rather than referring to it as a secondary model as it provided a significant improvement in response relative to the other secondary models. The following sections discuss the two main improvements in Model V5, including the energy dissipation and the residual drift.

In this model (Model V5), the hysteretic response of the reinforcement and reinforcement cross sectional area were modified relative to the final model (Model V4). Referring to Table 6.1, the reinforcement model for the hysteretic response used in Model V4 was the Bauschinger Effect (Seckin) and is a default VecTor2 model. In Model V5, the built-in model of Seckin (w/ Local Accumulation) was selected for the reinforcement hysteretic response. Otherwise, Model V4 and Model V5 incorporated the same constitutive models, including the Modified Bentz w/Local Fracture Tension Stiffening Model and the Palermo 2002 (w/ Decay) Model for the Concrete Hysteretic Response. The only other difference between Model V4 and Model V5 was the reduction in reinforcement area in the longitudinal reinforcing bars. In Model V4 there were reduced by 10% and in Model V5 there was 2.5% reduction. Figure 6.29 illustrates the response from Model V5.

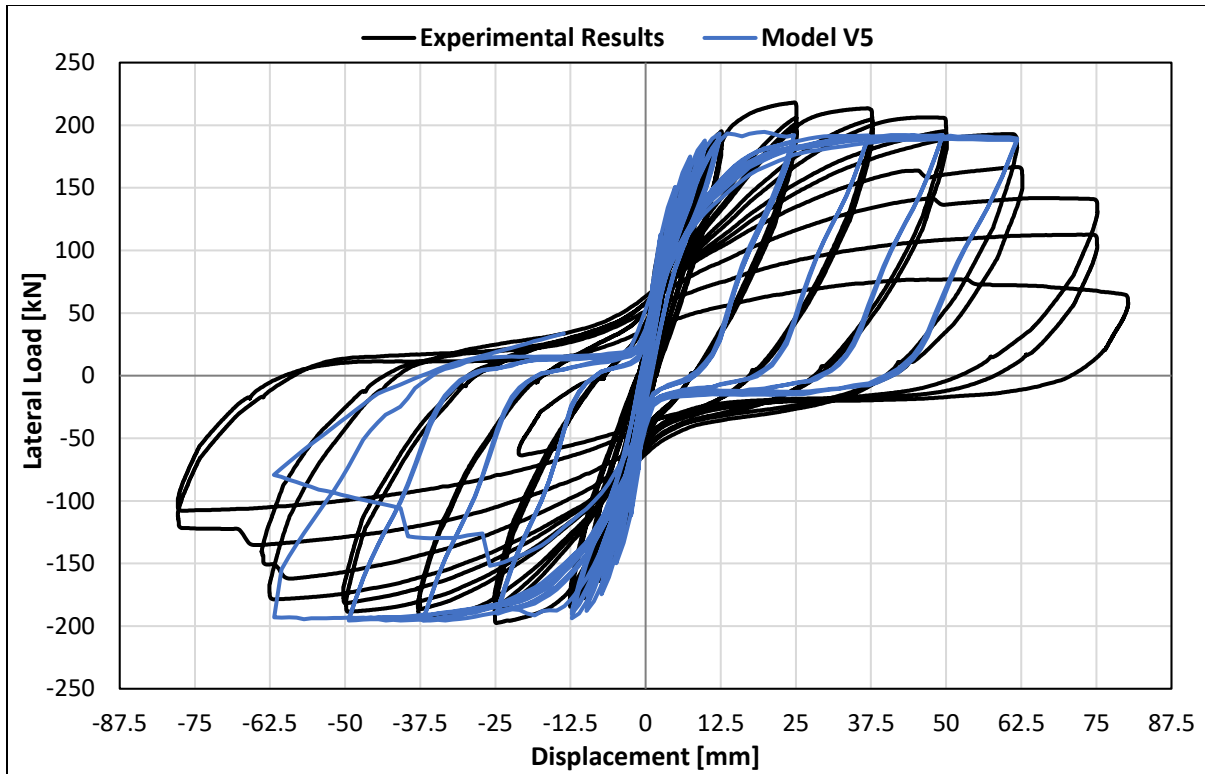


Figure 6.29: Comparison of Model V5 to Experimental Results

Figure 6.30 illustrates the close agreement in energy dissipation and residual displacement between Model V5 and testing at the 2.0% drift level. This is apparent as the hysteretic loop is significantly wider around the origin, which implies better greater energy dissipation, reduced stored elastic potential energy, and a steeper descending unloading response.

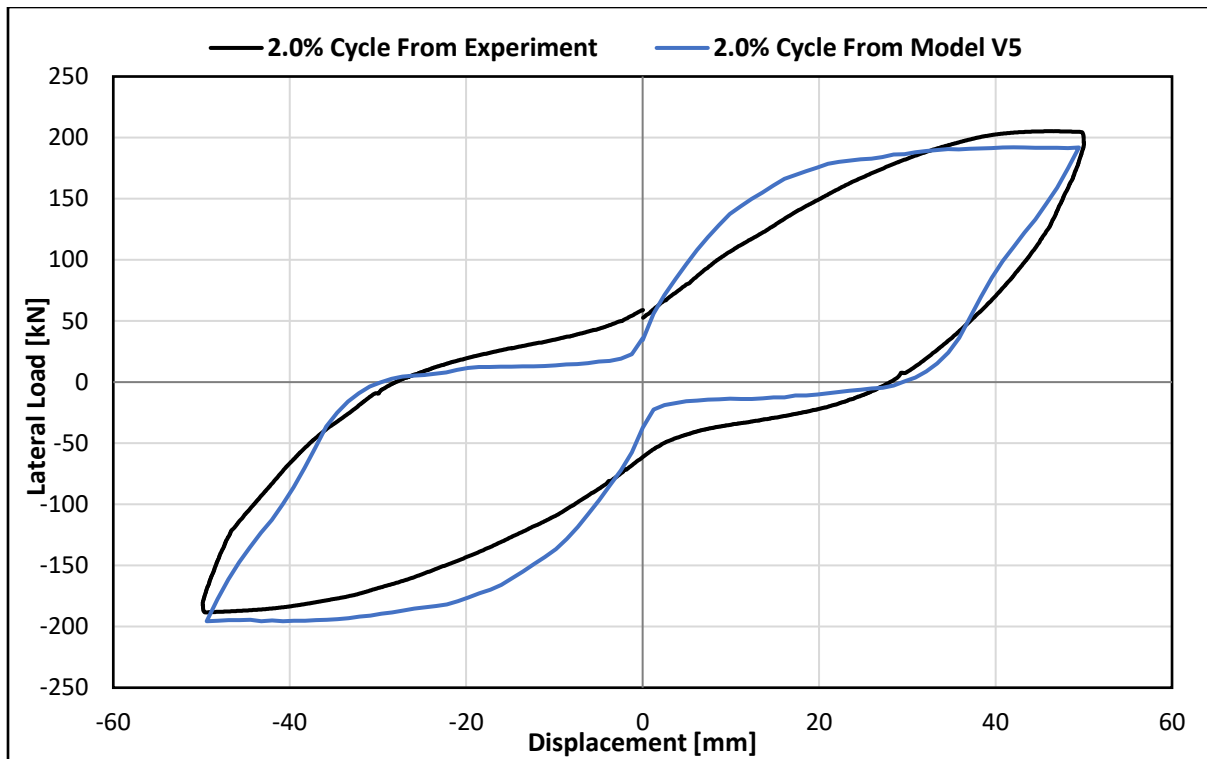


Figure 6.30: Comparison of Response for Experiment and Model V5 at 2.0% Drift

Compared to Model V4 (Figure 6.31), there is less pinching in Model V5. This is due to the adoption of the Seckin (w/ Local Accumulation) Model in V5. This model accounts for the local accumulation of reinforcement straining that occurs at the cracks rather than the average effect that would occur in Model V4. Due to the high axial load, the V4 model with the Bauschinger Effect (Seckin) underestimated the local straining of the reinforcement that takes place at the cracks and, thus, it overestimated the degree of cracking closing during unloading. These effects have an impact on the residual displacements, where Model V4 underestimated significantly the residual displacements recorded during testing.

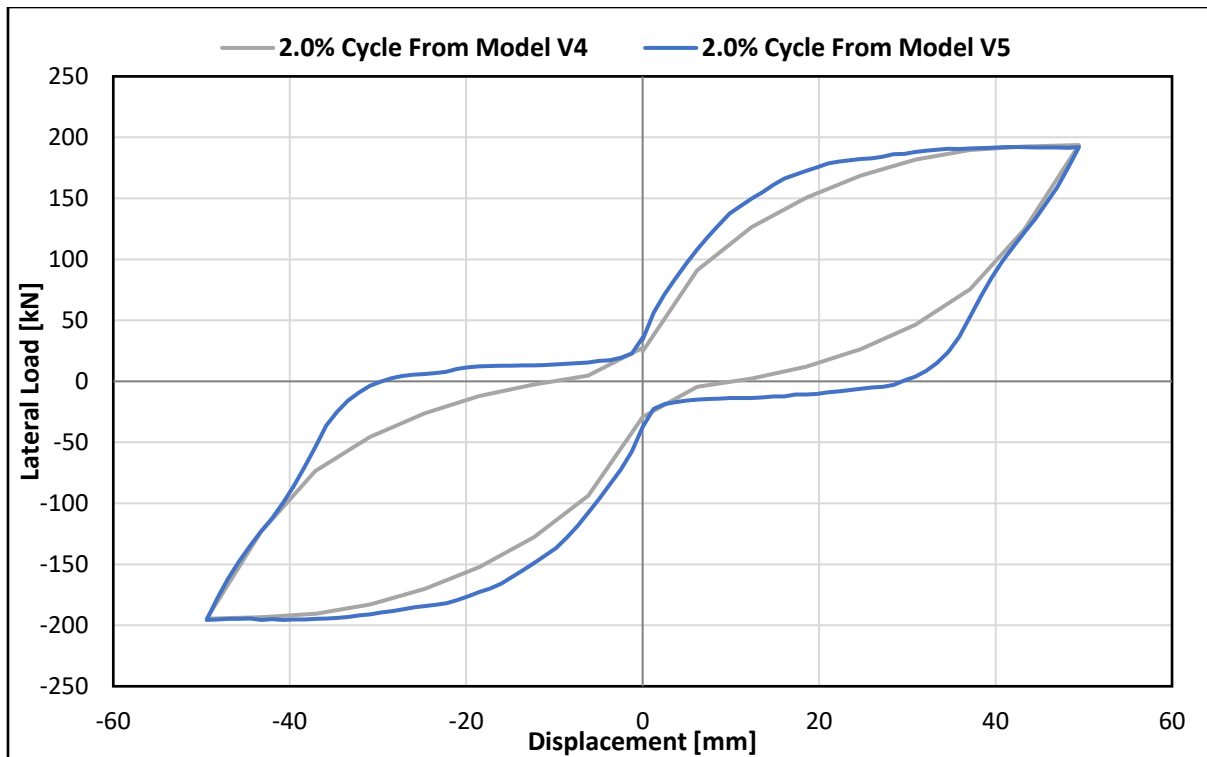


Figure 6.31: Comparison of Response Between Model V4 and Model V5

The Tension Stiffening Model was based on the Modified Bentz w/ Local Fracture. This model was incorporated with the quadrilateral elements and 2.5% reduced longitudinal reinforcement area in the boundary regions, and 1.25% reduced longitudinal reinforcement area in the web region in Model V5. This reduction was only over the first two truss bar elements from the base of the wall. The 10% reduction in the boundary reinforcement as was used in Model V4, led to a premature failure in Model V5. This needs to be further investigated. Note, as previously discussed, the reduction in reinforcement area considers the effect of placing strain gauges on the reinforcement.

The improvements in Model V5 compared to Model V4 are notable. The slope of the descending branches of the hysteretic response in Model V5 (Figure 6.29) matches the experimental response more effectively than in Model V4 (Figure 6.12). The wider loops better represent the response from testing; however, there is still significant pinching occurring about the origin which requires more investigation. The reason why Model V5 is considered a secondary model in this thesis compared to Model V4, is that more analyses are required to develop Model V5. However, it does provide a promising improvement to Model V4.

Comparing the results presented in Figure 6.32, Figure 6.33, and Figure 6.34, to Figure 6.15, Figure 6.16, and Figure 6.17, it is evident that Model V5 better replicates the experimental results. Model V4 underestimated the amount of total energy and overestimated the amount of stored elastic energy.

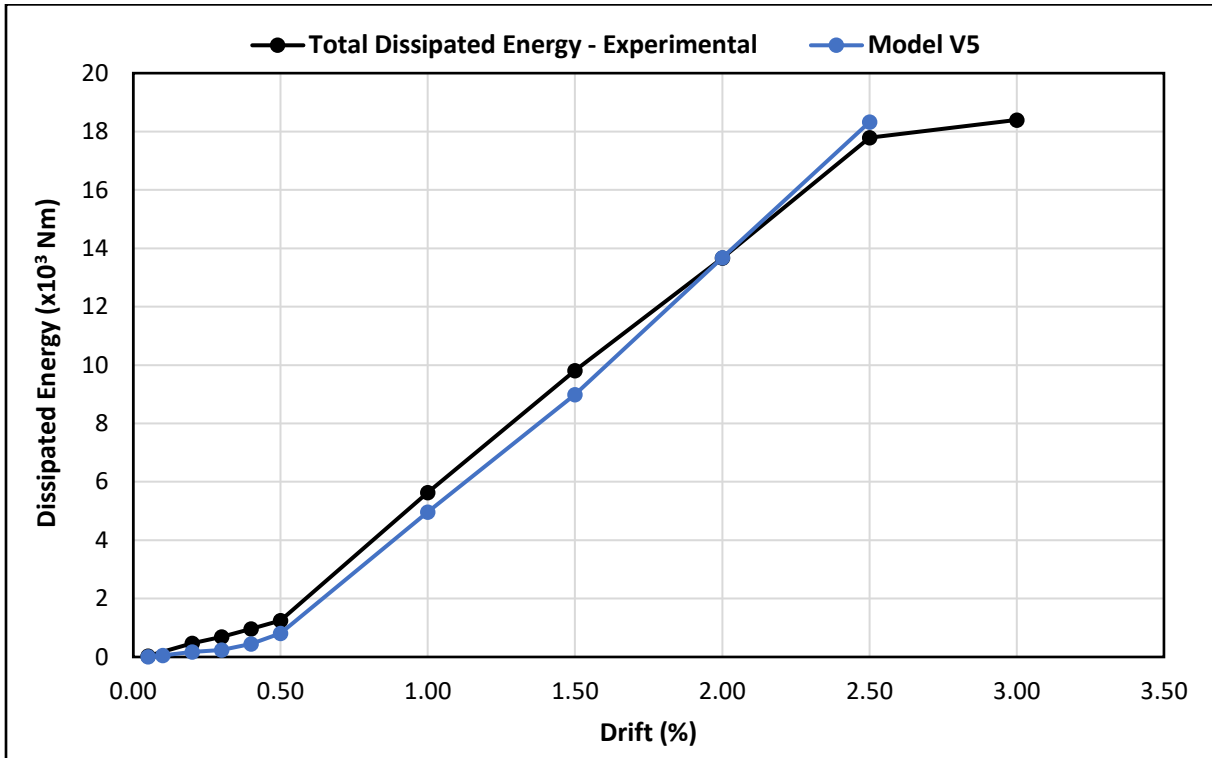


Figure 6.32: Comparison of Total Energy for Experiment and Model V5

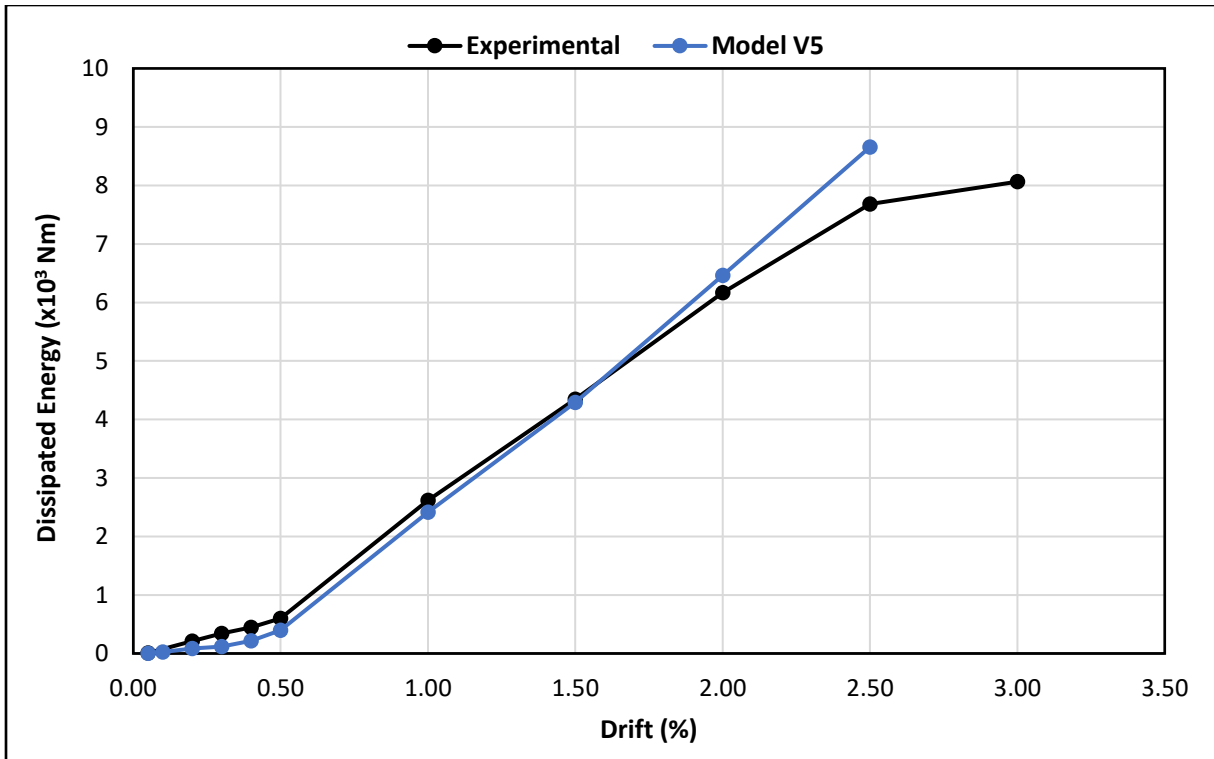


Figure 6.33: Comparison of Hysteretic Energy for Positive Cycles (Model V5)

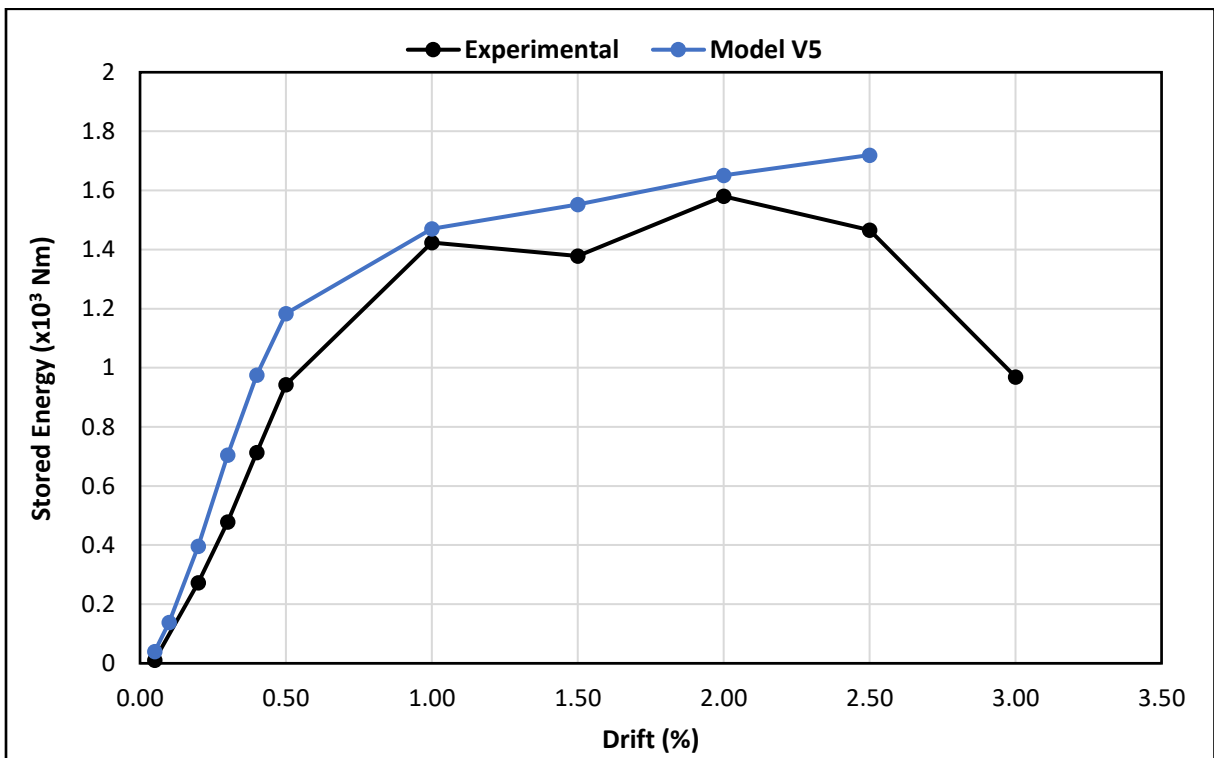


Figure 6.34: Comparison of Stored Elastic Potential Energy for Positive Cycles (Model V5)

Figure 6.35 illustrates the peak-residual displacement response. Model V4 predicted significant smaller residual displacements, whereas Model V5 accurately captured the level of residual displacements up to the last loading cycle of 2.5% where the model failed. Model V5 captured the strength degradation arising from $P-\Delta$ due to the use of quadrilateral elements. On the positive loading side, Model V5 predicted a peak lateral load of 194.6 kN. Overall, this model provides improvements to the residual drift and the energy dissipation.

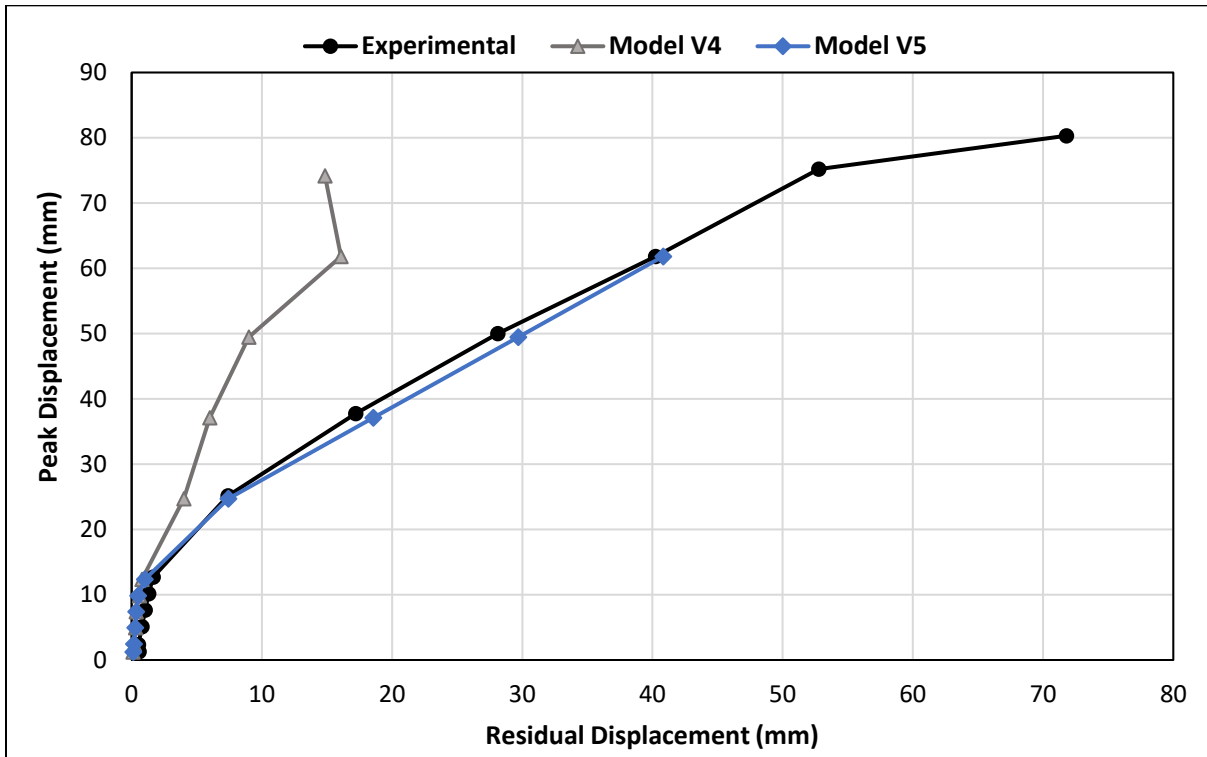


Figure 6.35: Peak vs Residual Displacement for Model V4, V5, and Experimental Results

6.6.2 Model V3 - Load Displacement Response

Recall that Model V3 was similar to Model V4 with the exception that Model V4 incorporated quadrilateral elements, and Model V3 incorporated rectangular elements, otherwise there were no other differences. This section provides the results from Model V3 as a comparison to the results from Model V4 and V5.

The model simulated combined lateral reverse cyclic loading and axial loading. The lateral loading was located at the center of the cap beam, 2495mm from the base of the wall. The axial loading was located across the width of the actuator head, 400mm wide, on the middle of the top of the cap beam.

A monotonic pushover analysis was conducted to determine the yield displacement of the wall. The analysis ran until rupture of the reinforcement on the tension side. Using the pushover results, the yield displacement was determined to be 9.22mm (0.37% drift). The yield force was approximately 191.5kN. The method for determining the yield point is shown in Figure 6.36.

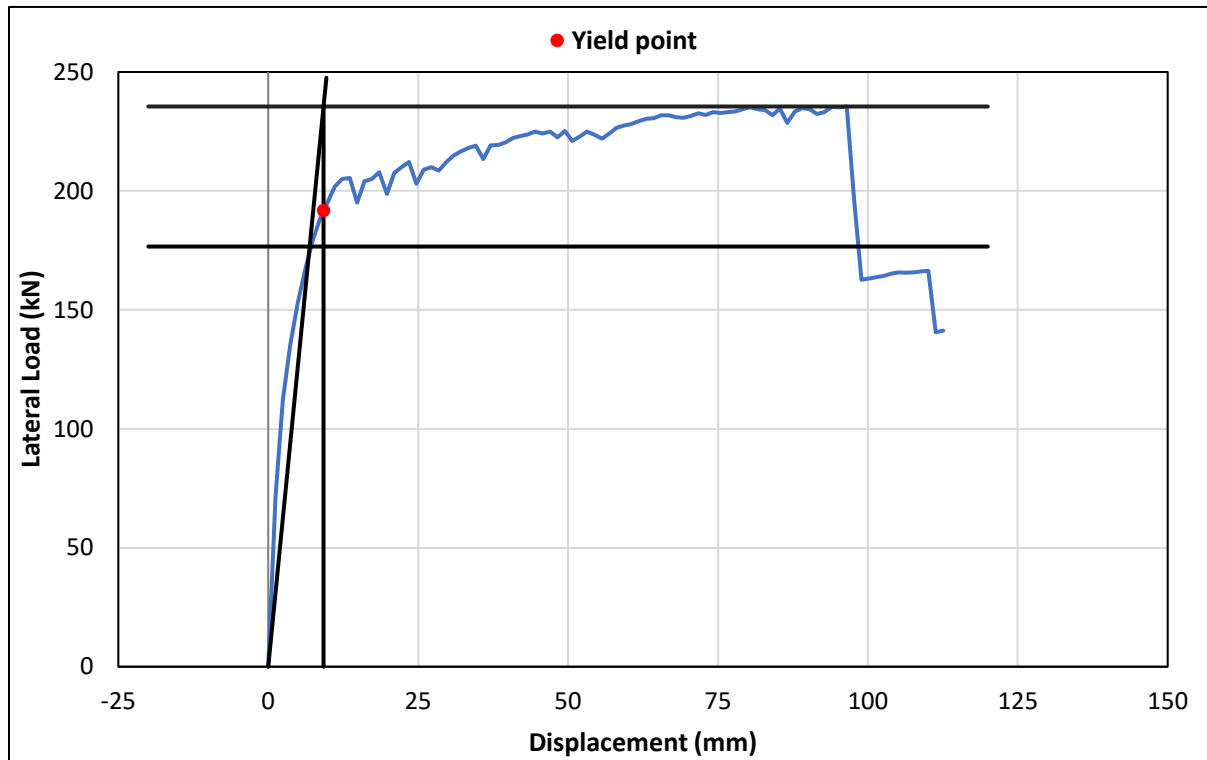


Figure 6.36: Yield Point Derivation from Pushover Analysis

Thereafter a reverse cyclic analysis was conducted to simulate a seismic event. The reverse cyclic loading protocol used was similar to that used during testing, with 3 repetitive cycles up to 25mm (1.0% drift) and two repetitive cycles after this point. Using the reverse cyclic results, the yield displacement for the wall was 8.92mm (0.357% drift) and the yield force was 186kN. The method for determining the yield point for the cyclic response is shown in Figure 6.37.

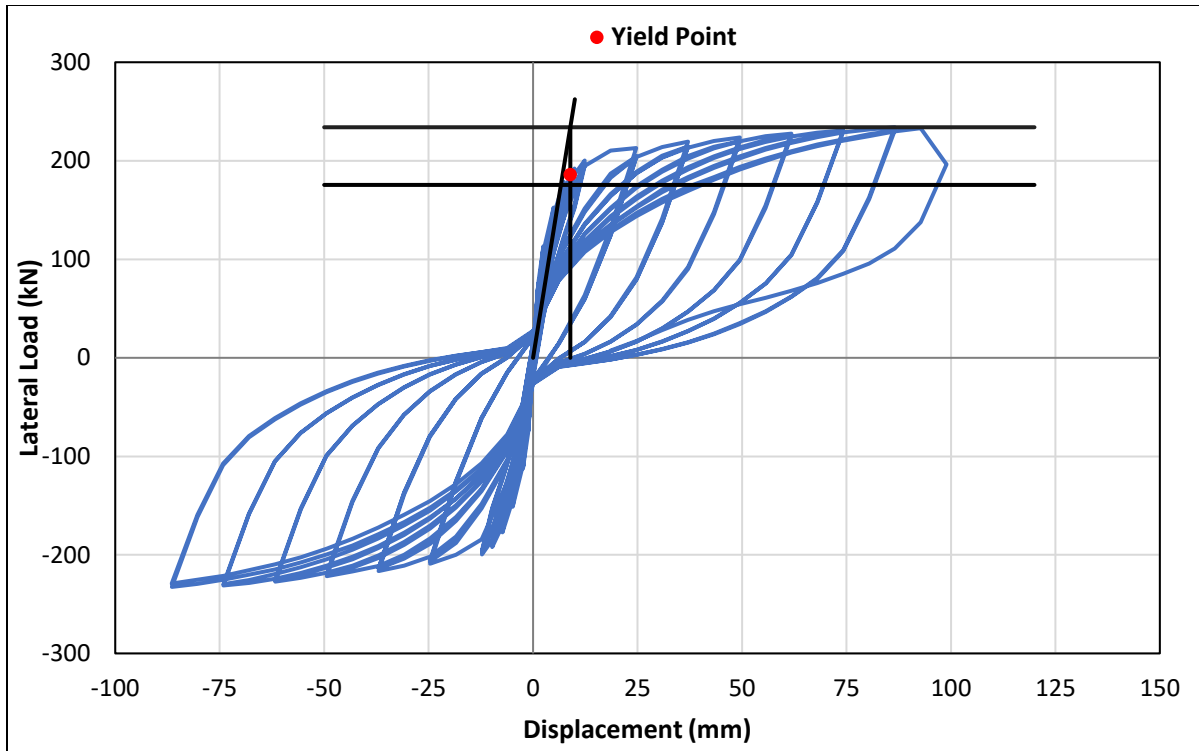


Figure 6.37: Yield Point Derivation from Reverse Cyclic Analysis

6.6.2.1 Comparison to Experimental Results

The reverse cyclic response had notable differences with the experimental response. The first is the notable pinching occurring between positive and negative cycles in the model. Similar to the experimental response, there was a stiffer response during the first cycle of a new displacement, and then subsequent cycles of the same displacement had a stiffness degradation visible by a reduction in force to each the target displacement.

Another notable difference between the model and the experimental results is that the overall stiffness does not degrade at larger displacements in the model (Figure 6.38). From the experimental results, after 25mm displacement (1.0% drift), the stiffness begins to degrade gradually, whereas for the model, the stiffness remains until the wall fails. The experimental results recorded the maximum lateral load at 25mm displacement (1.0% drift), whereas the model reached its maximum load at a lateral displacement of 50mm, thus the model overestimated the ductility. As stated, and discussed, incorporating quadrilaterals elements in Model V4 improved this response.

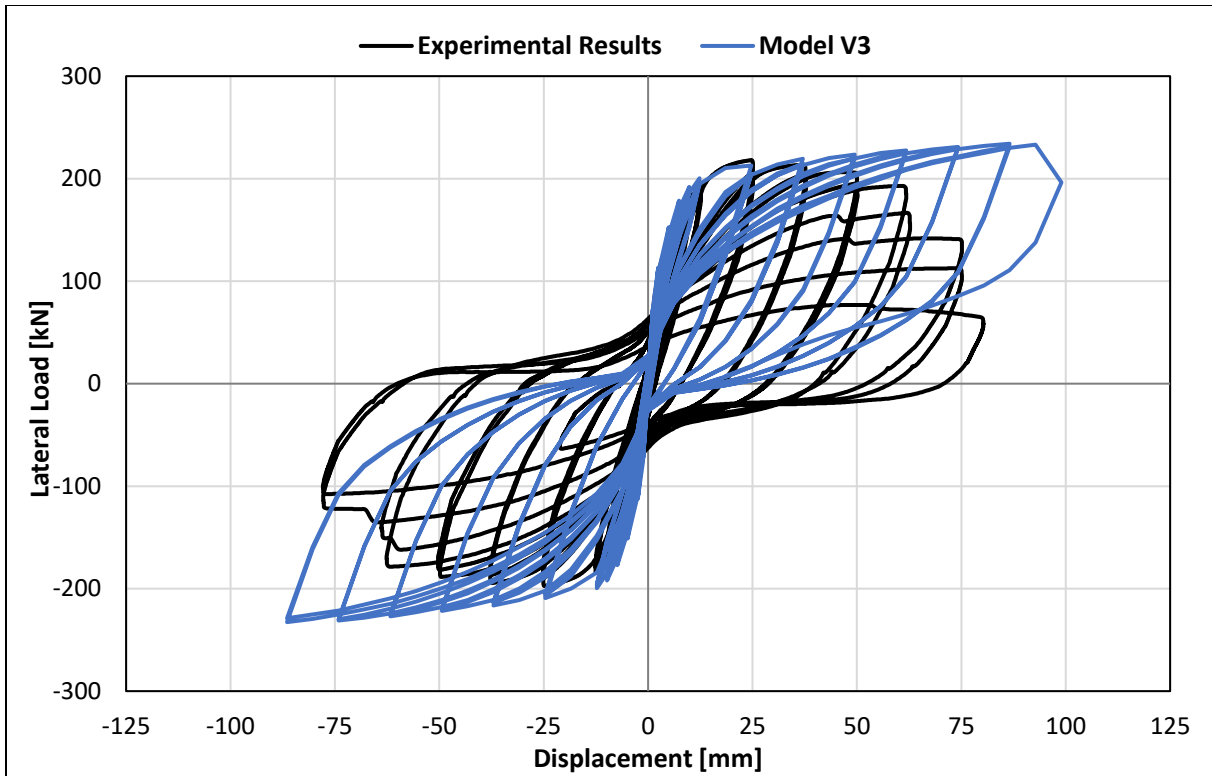


Figure 6.38: Hysteretic Response from Experiment and Model V3

The yield stress of the reinforcement, 440 MPa, was reached at 7.5mm displacement (0.3% drift). The yielding stress was exceeded after all three 7.5mm displacement cycles were completed and at 10mm displacement the stress in the boundary longitudinal reinforcing bars was greater than 440 MPa. In Figure 6.39 the pushover and reverse cyclic responses of the model are displayed. It is evident that the pushover response is an acceptable envelope of the hysteretic response.

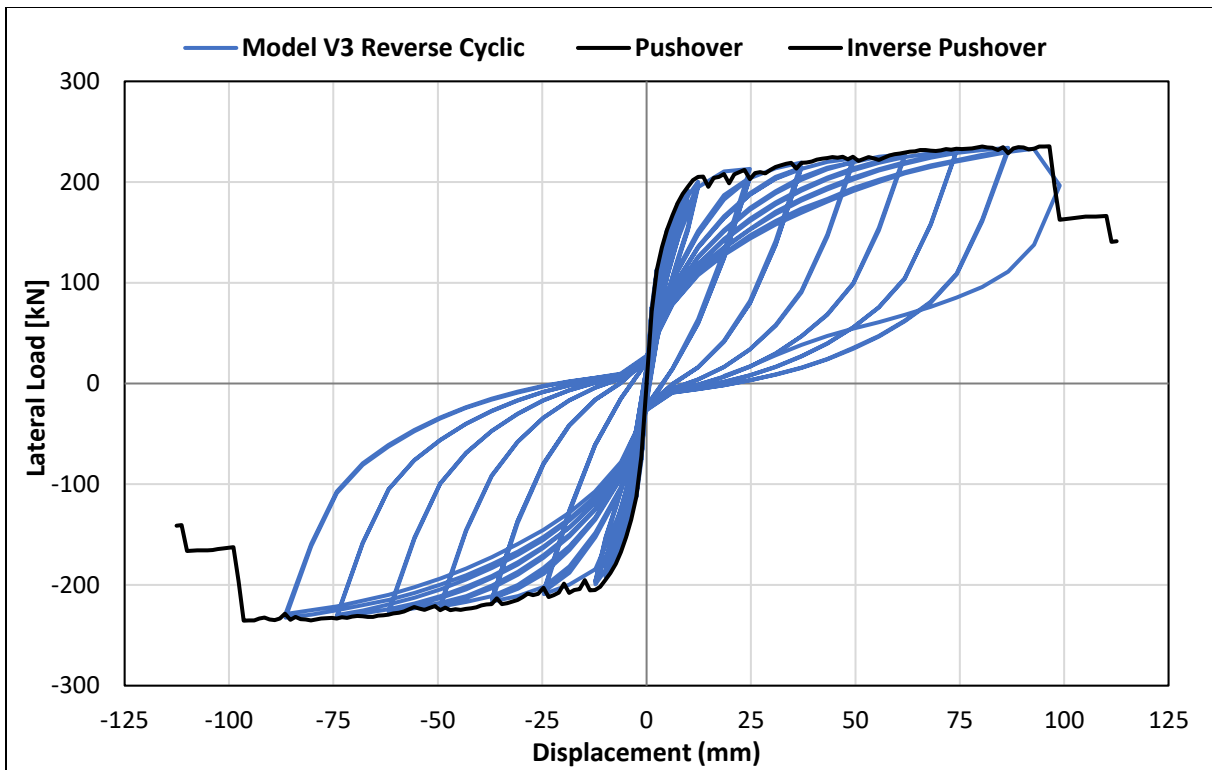


Figure 6.39: Reverse Cyclic Response Compared to Pushover Response from Model V3

7 Conclusion and Future Work

7.1 Conclusions

A slender reinforced concrete shear wall was studied to assess the performance under combined lateral and axial loading, to observe the crack recovery with applied axial loading, to quantify the effect of axial loading on self-centering, to determine the drift recovery, and to provide a control wall for future testing. The wall will serve as a control wall for a larger experimental program involving emerging materials such as SMAs (Shape Memory Alloys) and ECC (Engineered Cementitious Composites). The wall was tested under combined reverse cyclic lateral displacements and axial loading to simulate a more realistic seismic event on a shear wall in a structure. Below is a summary of the conclusions based on the results.

7.2 Shear Wall Conclusions

1. The response of the wall was different compared to previous testing that did not include axial loading. The application of axial loading created pinching in the lateral load-displacement hysteretic response. This pinching affected the energy dissipation.
 - a. Energy dissipation linearly increased throughout testing and reduced after rupture of the longitudinal reinforcement in the boundaries. The effect of axial loading increased the elastic potential energy of the wall, and this was evident in the reduced steepness of the unloading branches. Although it was expected that axial loading would reduce the energy dissipation capacity, the amount of energy dissipated was higher than previous wall testing. This was the result of a larger lateral load in the wall in this study to achieve the same displacement level as the. Noting this, larger residual drifts lead to higher energy dissipation. The axial loading caused a pinching of the hysteretic loops and a reduction in the residual drifts; thus, more elastic potential energy stored and less dissipated energy.
2. The applied axial loading promoted crack closure when the wall was laterally unloaded up to and including 0.5% drift, or just after yielding. Cracks were closed completely up to and including the 1.0% drift cycles. After this point cracks within the bottom 500mm of the wall remained opened until the end of the test, or until spalling occurred. When the

wall was unloaded between positive and negative cycles, the open cracks did reduce in width.

3. The vertical displacements of the cap beam were greater on the tension side than the compression side of the wall. Vertical displacements on the compression side increased when the concrete crushed and bars began to buckle.
4. The crack pattern during testing was flexure shear cracking. This was expected and seen in the previous testing and is a result of the type of testing and wall. The majority of cracking was contained within the bottom half of the wall which was not the case for walls tested without axial loading. The walls without axial loading experienced cracking up the entire height of the wall and experienced more damage. The presence of axial loading aided in containing the cracking within the bottom half of the wall and demonstrated improved damage control compared to the previous walls.
5. Drift recovery was maintained until 1.0% drift, thereafter the wall continually reduced its ability to recover. This was due to yielding of the reinforcement. When the wall was no longer elastic, the recovery effect was reduced. After the onset of strain hardening, there was a noticeable linear reduction in the walls' ability to recover. After the longitudinal steel ruptured at the ends of the wall at the 2.5% drift cycles, negligible recovery was observed. Thus, the ability for the wall to recover was dependent on the steel remaining within the elastic range and early stages of yielding. When the steel was well into the inelastic range or buckled, the recovery was no longer present.
6. The longitudinal reinforcement buckled due to the combination of lateral and axial loading; this was only visually observed when the concrete cover spalled off. The buckling occurred in the direction of loading.
7. Reinforcement within the boundary region began to rupture in the second cycle of 2.5% drift. This was expected given that for normal importance reinforced concrete structures, 2.5% drift generally refers to the onset of collapse. The test was terminated during the first cycle of 3.5% drift due to a 50% drop in lateral load resistance.
8. The test set up, specifically the application of axial loading, created additional lateral loading to the wall. This added lateral loading increased as the lateral displacement

increased. This added lateral loading was considered to determine the actual lateral capacity of the wall.

9. Due to the orientation of the set-up and a leveling error during casting, the cap beam was slightly off level. Although a layer of plaster was placed below the plate on the top of cap beam for the vertical actuator to bear against, the application of axial loading caused the wall to lean in the out-of-plane direction. This displacement in the wall ultimately created added tensile strain to the back side of the wall and compressive strains on the front side of the wall. This resulted in the back boundary reinforcement to rupture first.
10. Finite element modelling was more accurate at representing the experimental results when using quadrilateral elements with the Tension Stiffening Model set to the Modified Bentz w/ Local Fracture. This accuracy refers to the model predicting failure at a displacement in satisfactory agreement with the test results; however, it did not accurately represent the stiffness decay.
11. The finite element model with quadrilateral elements rather than rectangular elements was able to capture the stiffness decay. When comparing the results of both models with the different elements however, they were only accurate to 1.5% drift for quadrilateral elements and 1.0% drift for rectangular elements.
12. The model exhibited better results when a protrusion of the cap beam was incorporated with a lateral force of 0.338kN applied to the center and a 10% reduction in the reinforcement cross sectional area in the boundary bars. Model V4 contained all these features and provided the most accurate results.

7.3 Practical Summary

The practical applications of this research are to further develop the understanding of reinforced concrete structures built with conventional construction materials and to provide guidance on numerical modelling. Due to the cost of producing emerging materials such as SMA and ECC, they are not as desirable to be used over an entire structure. Though these emerging materials can provide superior improvements to the resilience of structures, having a thorough knowledge of traditional materials is just as important. This enables a more efficient use of

emerging materials combined with traditional materials to achieve a structure that meets its target performance.

Understanding the damage that can occur to a structure while in service is critical to maintaining the safety of its occupants. In modern mid-rise and high-rise buildings, being able to add more usable floor area is desirable to building owners. To do so, the vertical supporting members (columns and walls) may be reduced in width. As a result, the tributary area of the gravity loading and, in turn the axial loading, supported by these vertical members does not change. However, given a change in geometry increases the percentage of the vertical members axial load capacity that is being exhausted by the increased loading. The increased axial load on a vertical member, such as a shear wall, increases the strength and stiffness, but decreases the displacement ductility, and increase secondary moments through the $P-\Delta$ effect.

The wall presented in this thesis had dimensions of 150mm in thickness and 1000mm in length. 10% of this wall's axial load compressive capacity was 575kN based on the concrete compressive strength of $f'_c = 38.3\text{MPa}$. A 10% reduction in area, specifically in the short span direction, reduces the walls cross section to 135mm x 1000mm. This small change in the wall dimensions results in an increase of 10% to the axial compressive stress on the section. This change would promote concrete crushing and longitudinal bar buckling at a smaller lateral displacement. Understanding that a slender shear wall with generally a small cross sectional area with high axial loading leads to added recentering; however, it also leads to increased buckling and crushing effects, which was highlighted in this study.

This study also provided guidance on modelling phenomenon that impacts the simulations results. Notably, capturing the $P-\Delta$ effect through the appropriate concrete elements provided a significant improvement in the modelling results. In addition, local effects at the concrete cracks, such as the consideration of the reduced ultimate strain in the reinforcement and the accumulation of straining were critical in capturing the onset of failure that was in line with the observed results.

7.4 Recommendations and Future Work

The following recommendations are intended for furthering research on shear walls and to aid in providing suggestions that would make this process more efficient.

1. For the cap beam, ensure that a proper support frame is built on the top of the formwork. In this case, the weight of the cap beam bared against the 2 in x 4 in timber on the top of the formwork, resulting in the cap beam being off level.
2. Build formwork that has removable sections. Due to the weight of the cap beam, the wall formwork compressed and the bulk heads were broken to remove them.
3. Place the strain gauges on the inside of the reinforcing bars so that they won't be damaged by the rebar ties and the bars. Alternatively use structural health monitoring such as distributed fiber optic sensors to capture the strain profiles of the reinforcement during testing. Distributed fiber optic sensors provide continuous strain readings.
4. Determine the locations of the cable potentiometers prior to casting the concrete so that the locations do not have steel in the path
5. When casting the concrete, create areas in the formwork that are accessible for the vibrators. The formwork was congested in the bottom quarter of the wall and it was a challenge for the vibrator to vibrate the concrete from the external surfaces.
6. For this test, no sliding was observed at the base of the wall, this was observed in previous tests. It is probable that no added vertical reinforcement is required to prevent sliding when axial loading is present.
7. Future testing should consider new emerging materials such as ECC and SMA to improve the performance of shear walls to seismic loading, including drift recovery and damage tolerance.
8. Future testing of the shear walls with ECC and SMA should incorporate combined lateral and axial loading to assess improvements in recovery capacity due to the impact of the axial load.
9. Future research should investigate local phenomena, such as local fracture of the reinforcement, strain accumulation of the reinforcement at a crack, and crack closure due to axial loading.

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Appendix A – Material Drawings

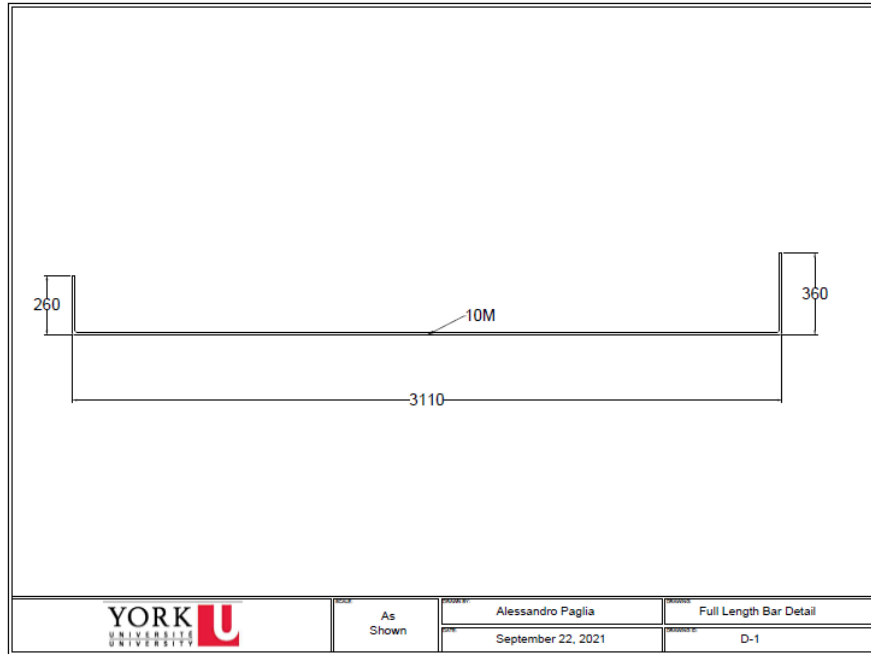


Figure 8.1: Boundary and Web Longitudinal Bar Detail

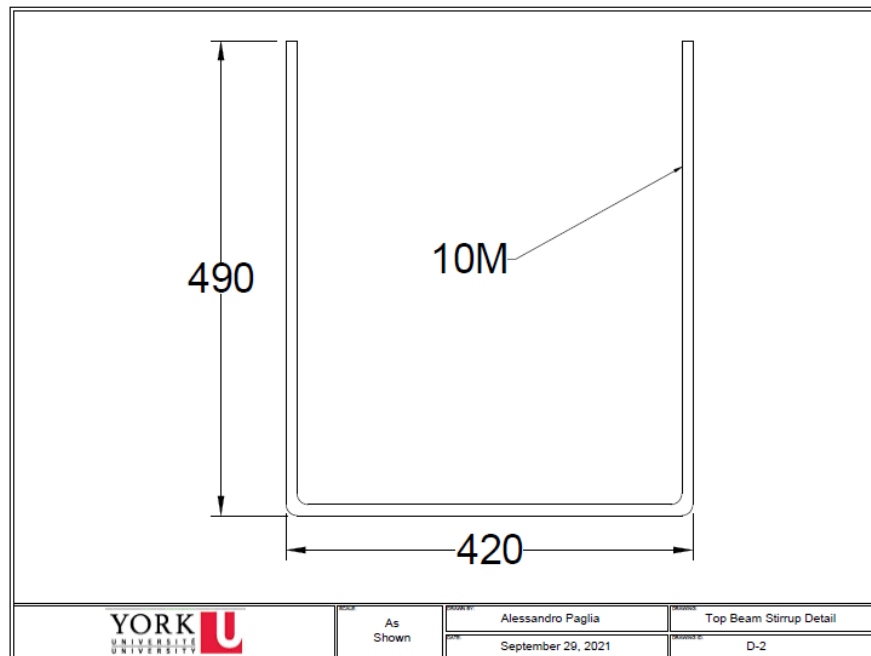


Figure 8.2: Cap Beam Stirrup Detail

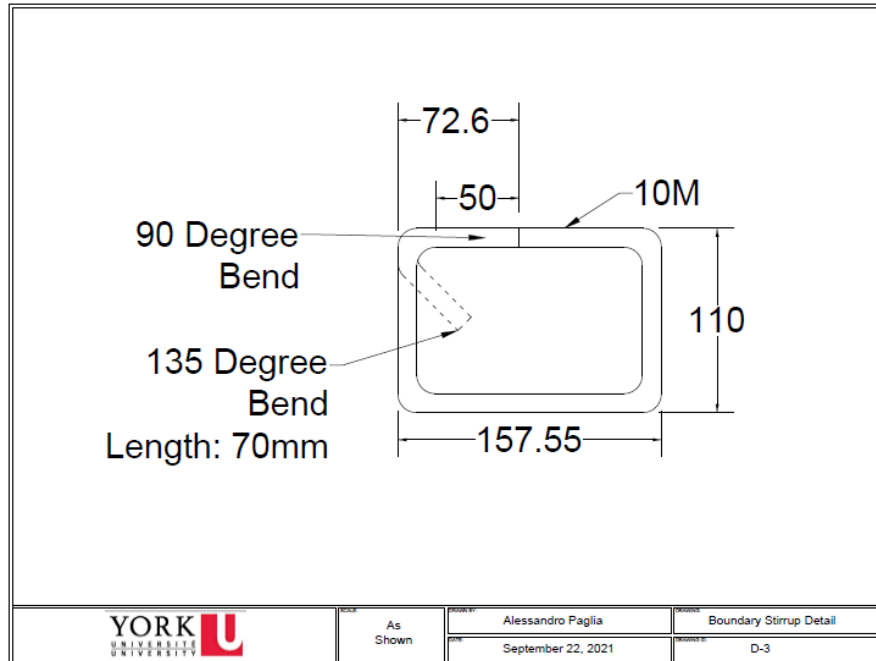


Figure 8.3: Boundary Tie Detail

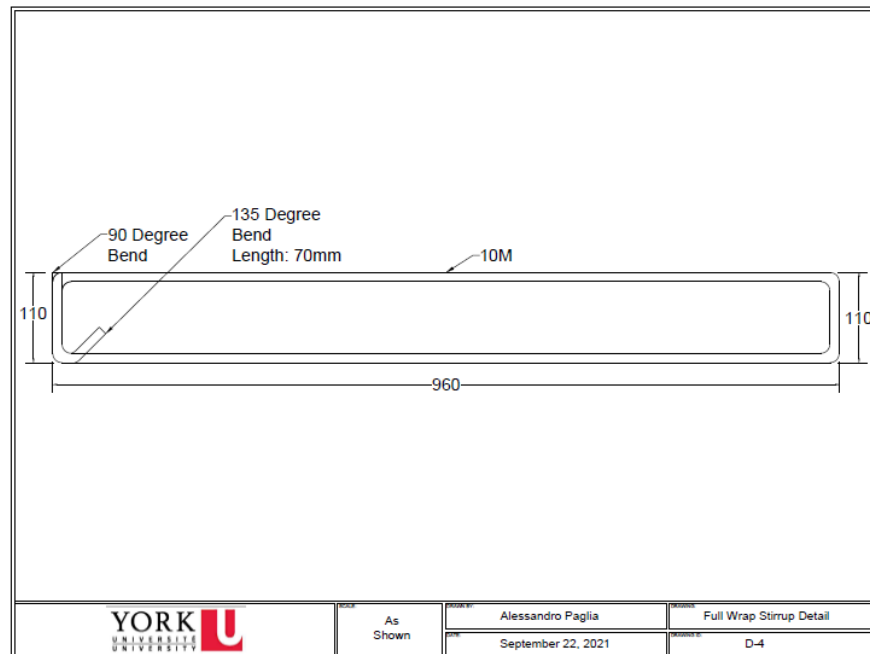


Figure 8.4: Wall Stirrup Detail

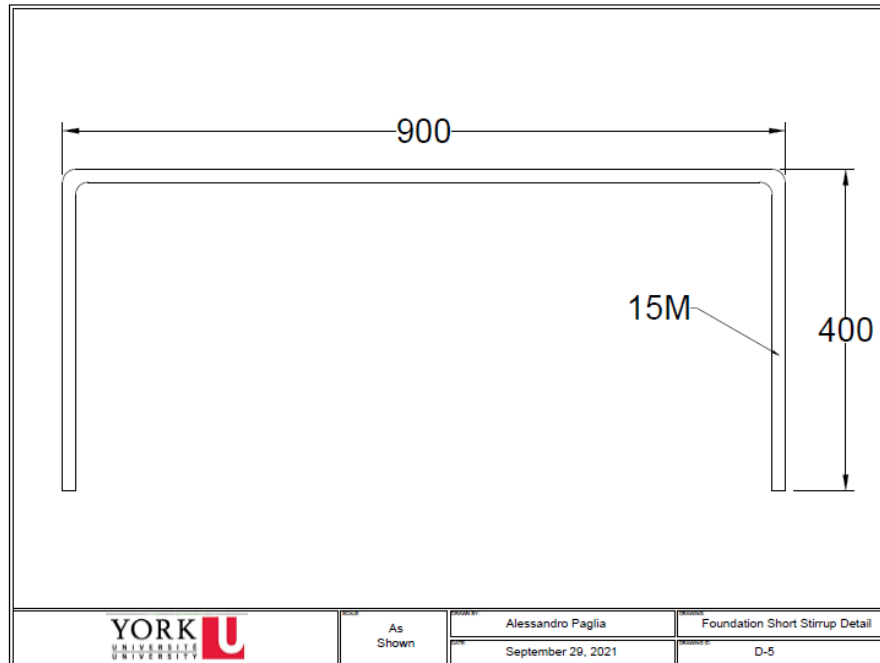


Figure 8.5: Foundation Short Direction Stirrup Detail

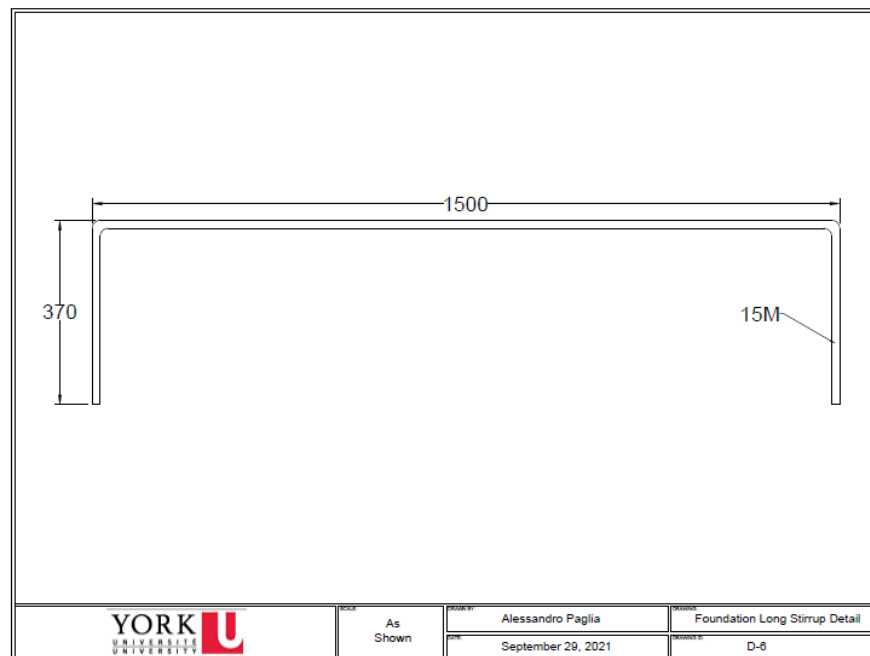


Figure 8.6: Foundation Long Direction Stirrup Detail

Appendix B – Formwork



Figure 8.7: Base Formwork with Bottom Rebar Cage



Figure 8.8: Base Formwork with Added Support Prior to Casting



Figure 8.9: Bottom of Wall Formwork with Bulkheads



Figure 8.10: Fully Erected Wall Formwork



Figure 8.11: Cap Beam Formwork



Figure 8.12: Internal View of Cap Beam Formwork

Appendix C – Construction Process



Figure 8.13: On Site Mixing for Foundation Pour provided by CeMix



Figure 8.14: Completed Foundation Pour



Figure 8.15: Cylinders Placed in Water Bath After Demolding



Figure 8.16: Completion of Cap Beam and Wall Pour

Appendix D – SCC Patch Region



Figure 8.17: Honeycombing Along Base, Front of Wall, Actuator Side



Figure 8.18: Honeycombing Along Base, Front of Wall, Non-Actuator Side



Figure 8.19: Honeycombing on Side of Wall, Non-Actuator Side



Figure 8.20: Honeycombing Along Base, Back of Wall, Non-Actuator Side



Figure 8.21: Honeycombing on Side of Wall, Actuator Side



Figure 8.22: Front of Wall After Removal of Formwork

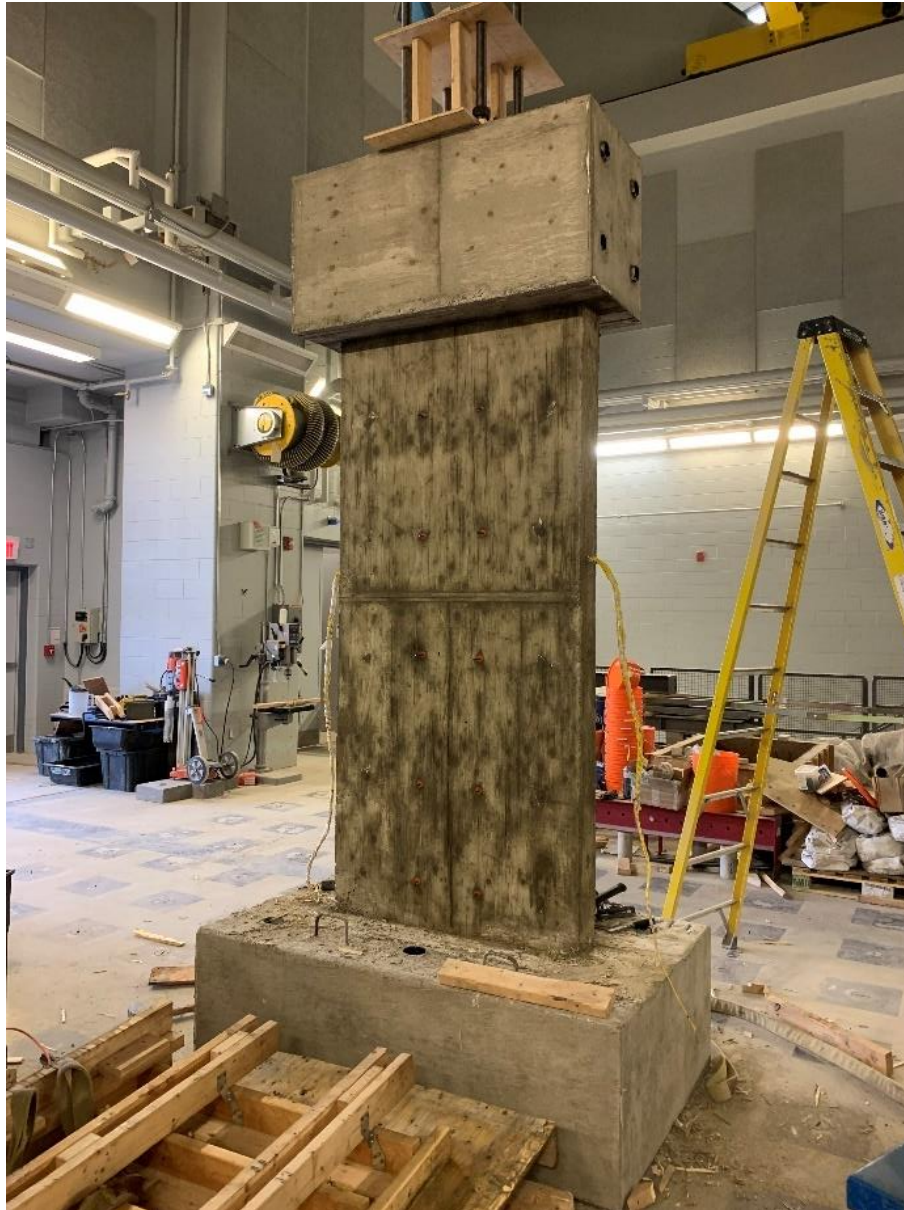


Figure 8.23: Back of Wall After Removal of Formwork

Appendix E – Material Testing

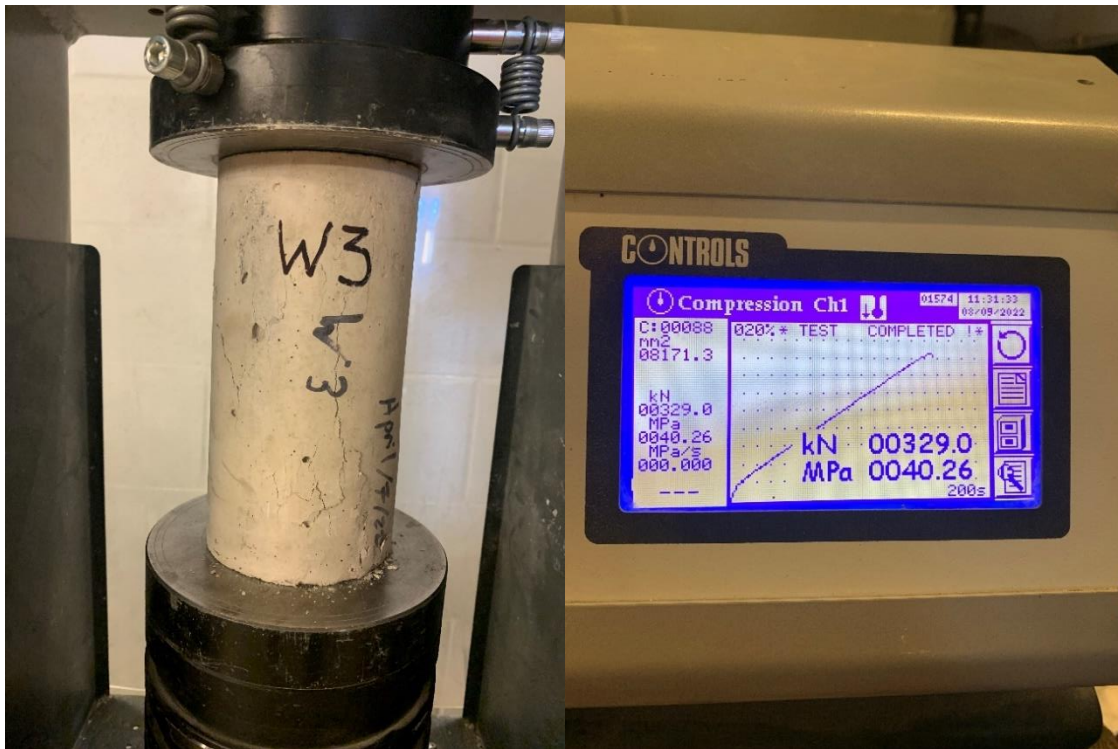


Figure 8.24: Cylinder Testing and Result for W3

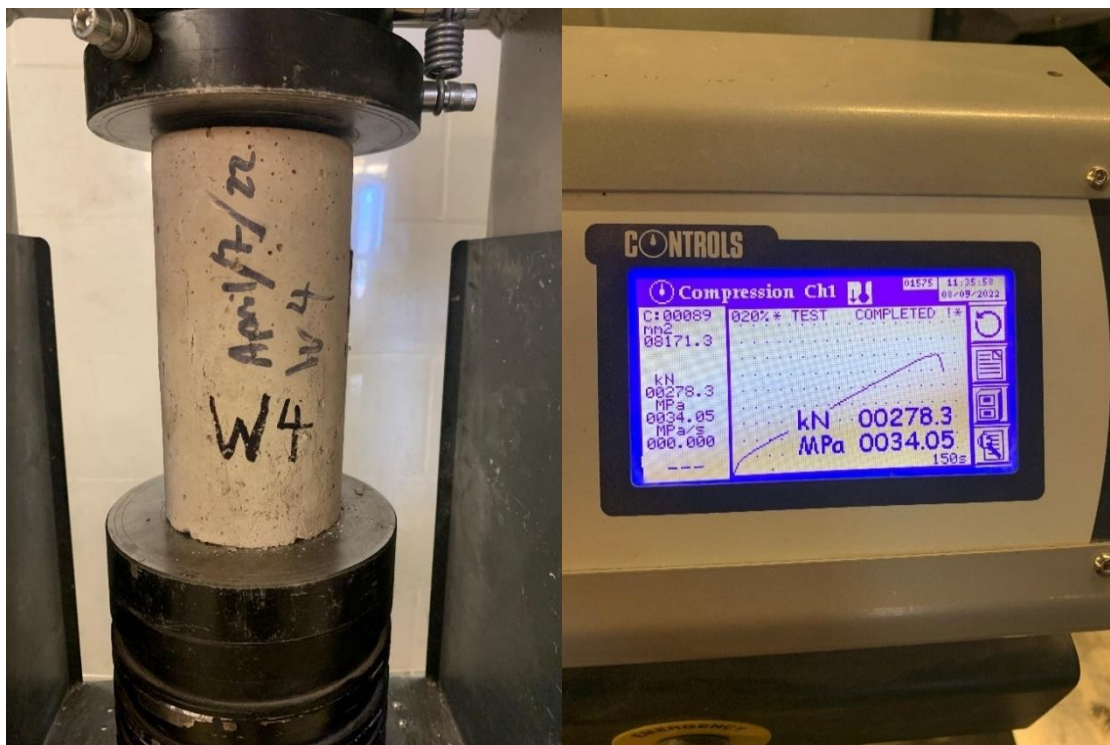


Figure 8.25: Cylinder Testing and Result for W4

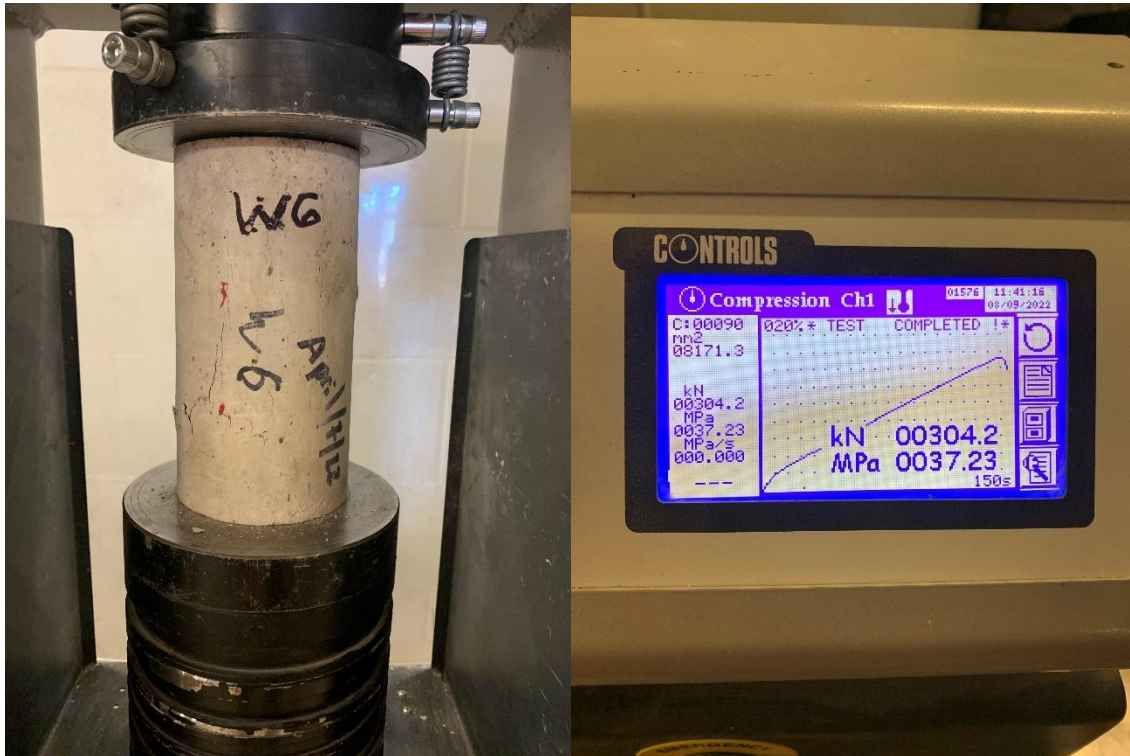


Figure 8.26: Cylinder Testing and Result for W6

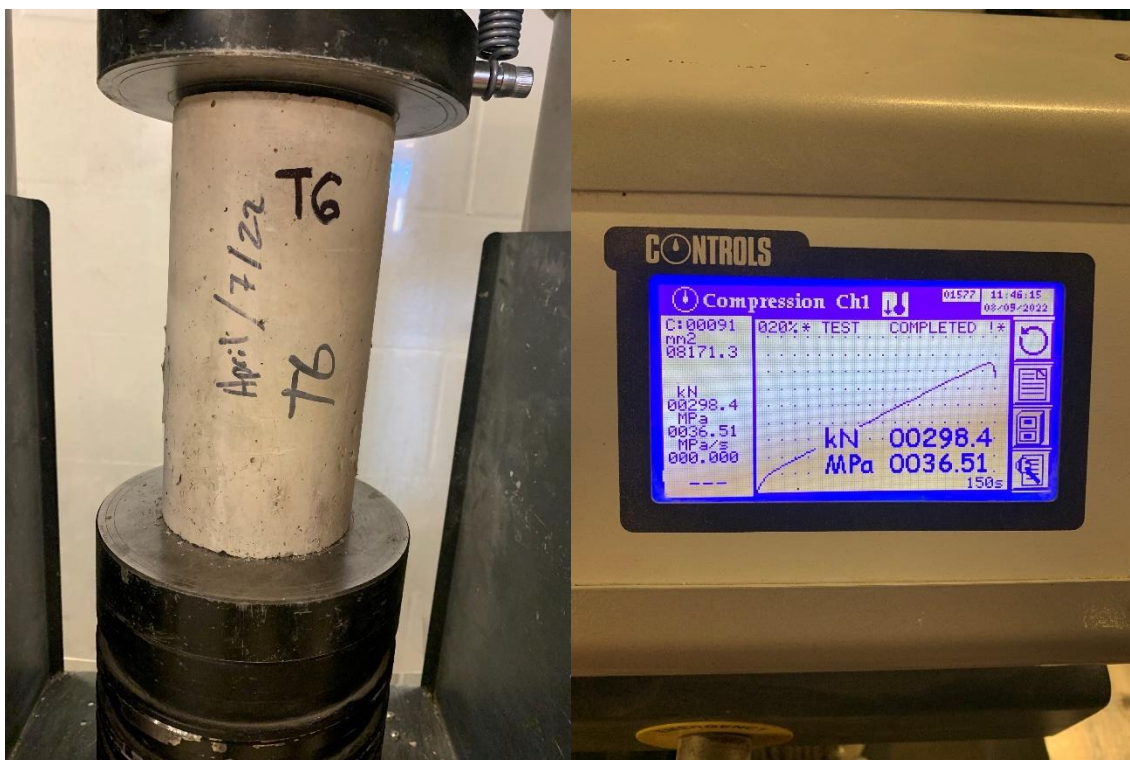


Figure 8.27: Cylinder Testing and Result for T6

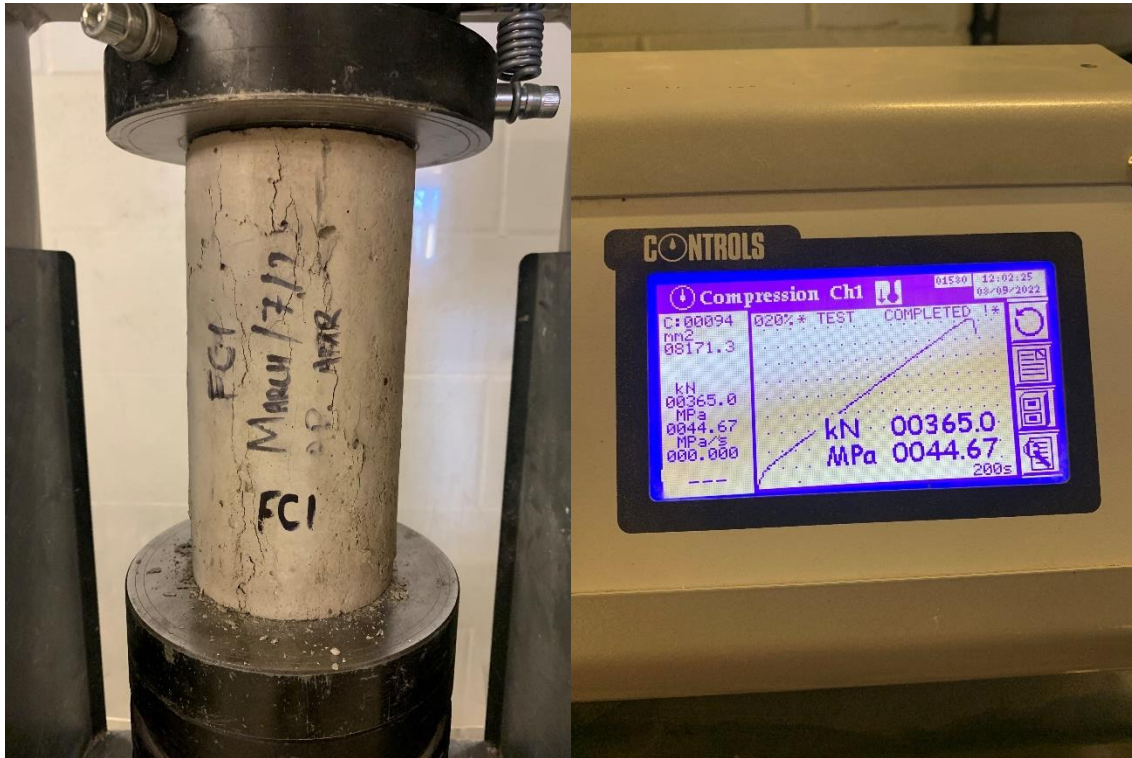


Figure 8.28: Cylinder Testing and Result for FC1

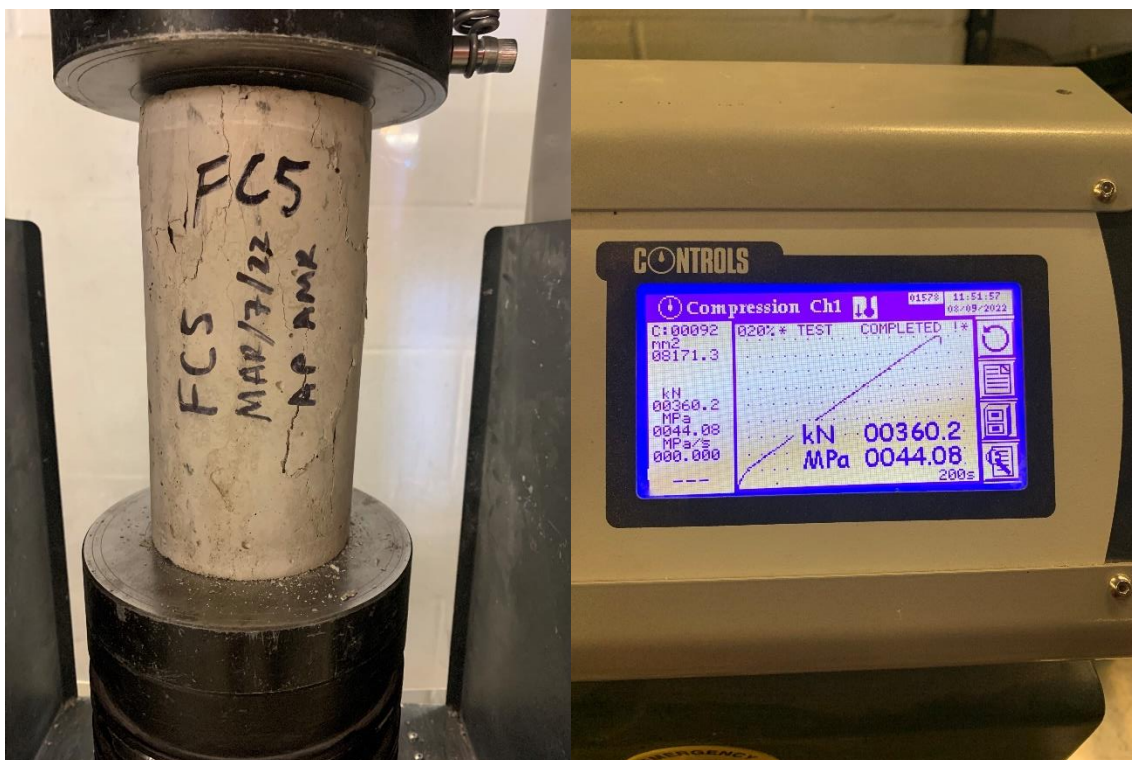


Figure 8.29: Cylinder Testing and Result for FC5

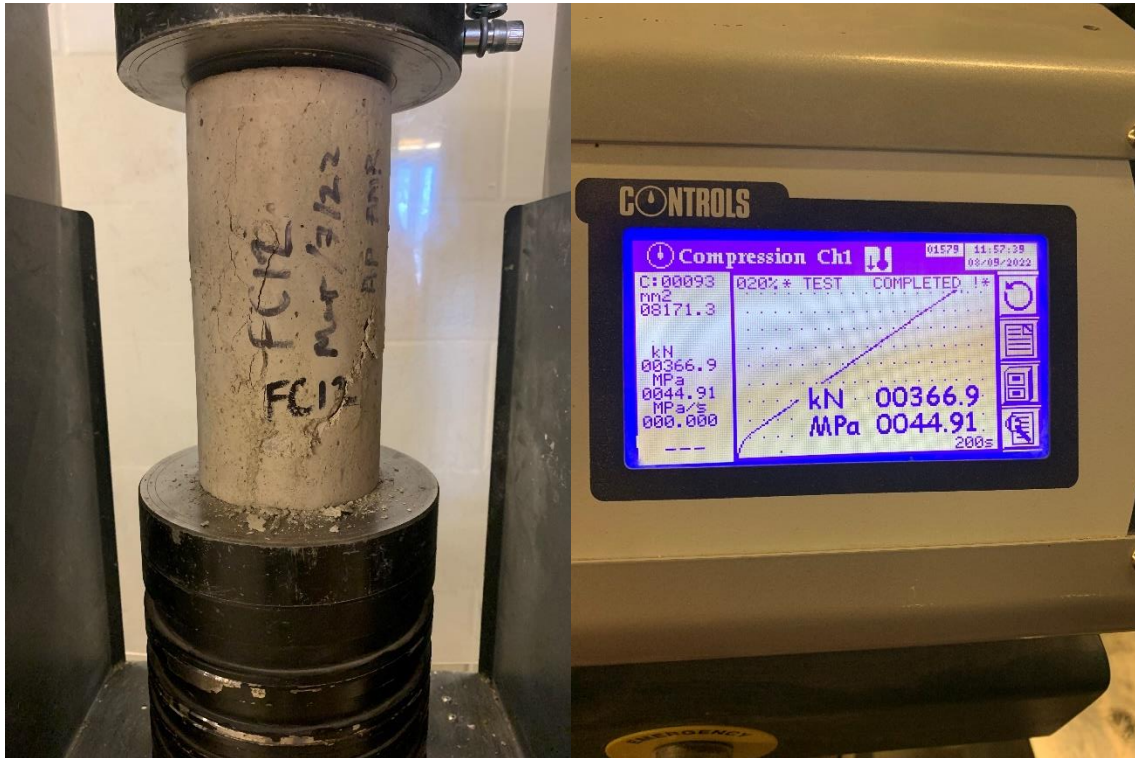


Figure 8.30: Cylinder Testing and Result for FC12

Appendix F – Instrumentation Set-Up

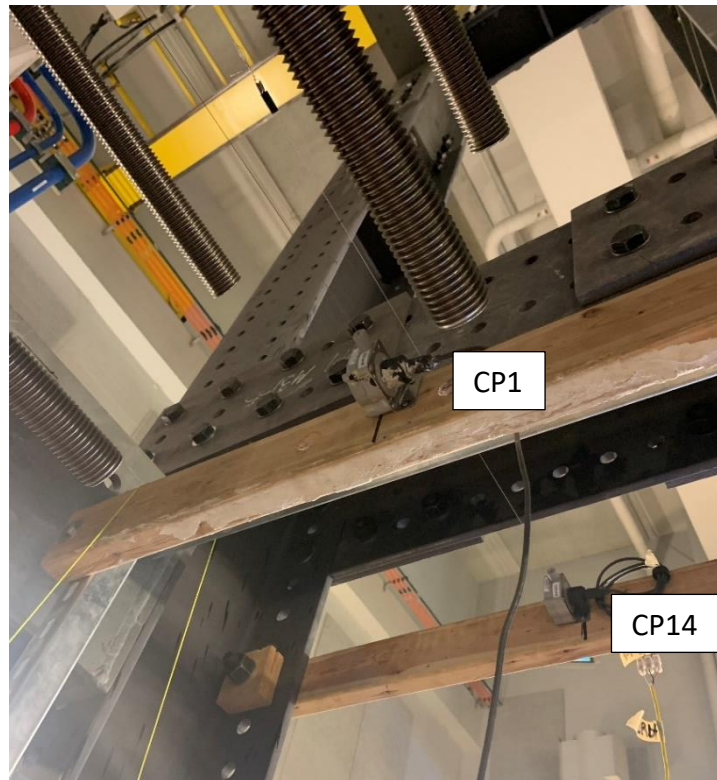


Figure 8.31: Location of CP1 and CP14



Figure 8.32: Location of CP2

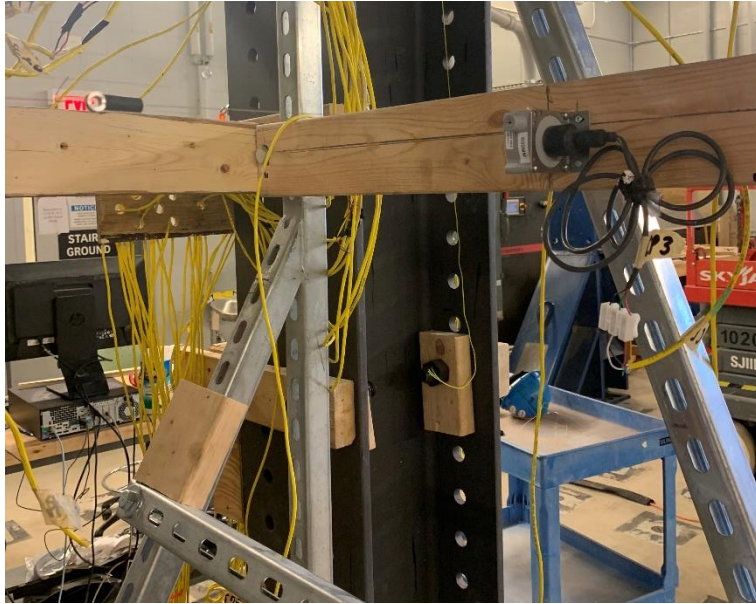


Figure 8.33: Location of CP3

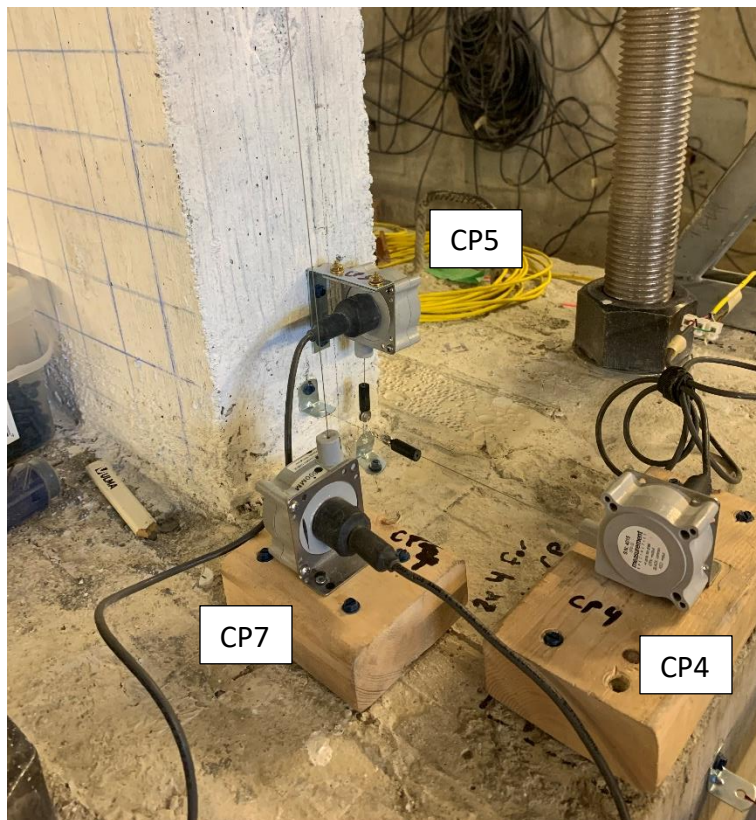


Figure 8.34: Location of CP4, CP5, and CP7

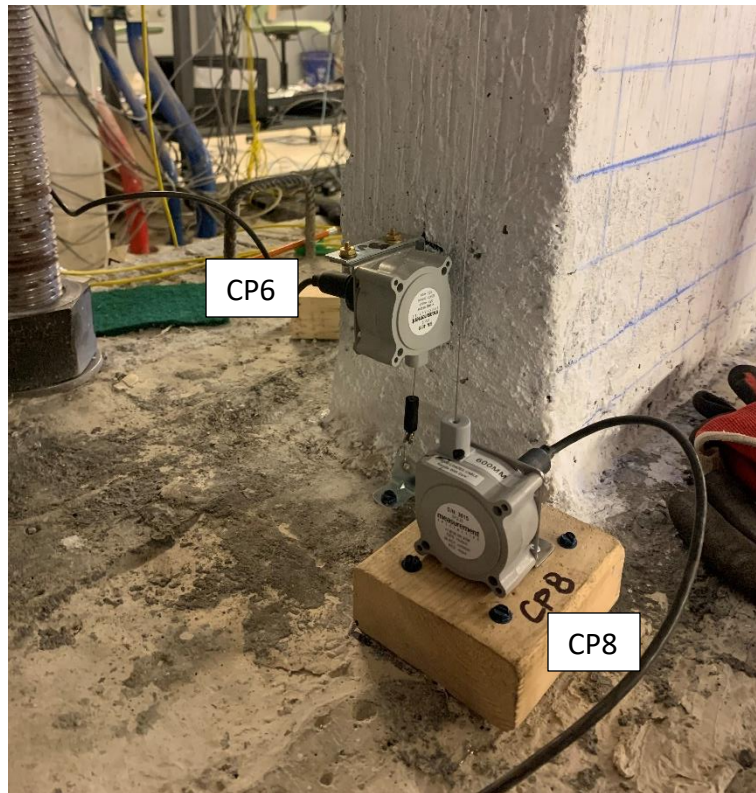


Figure 8.35: Location of CP6 and CP8

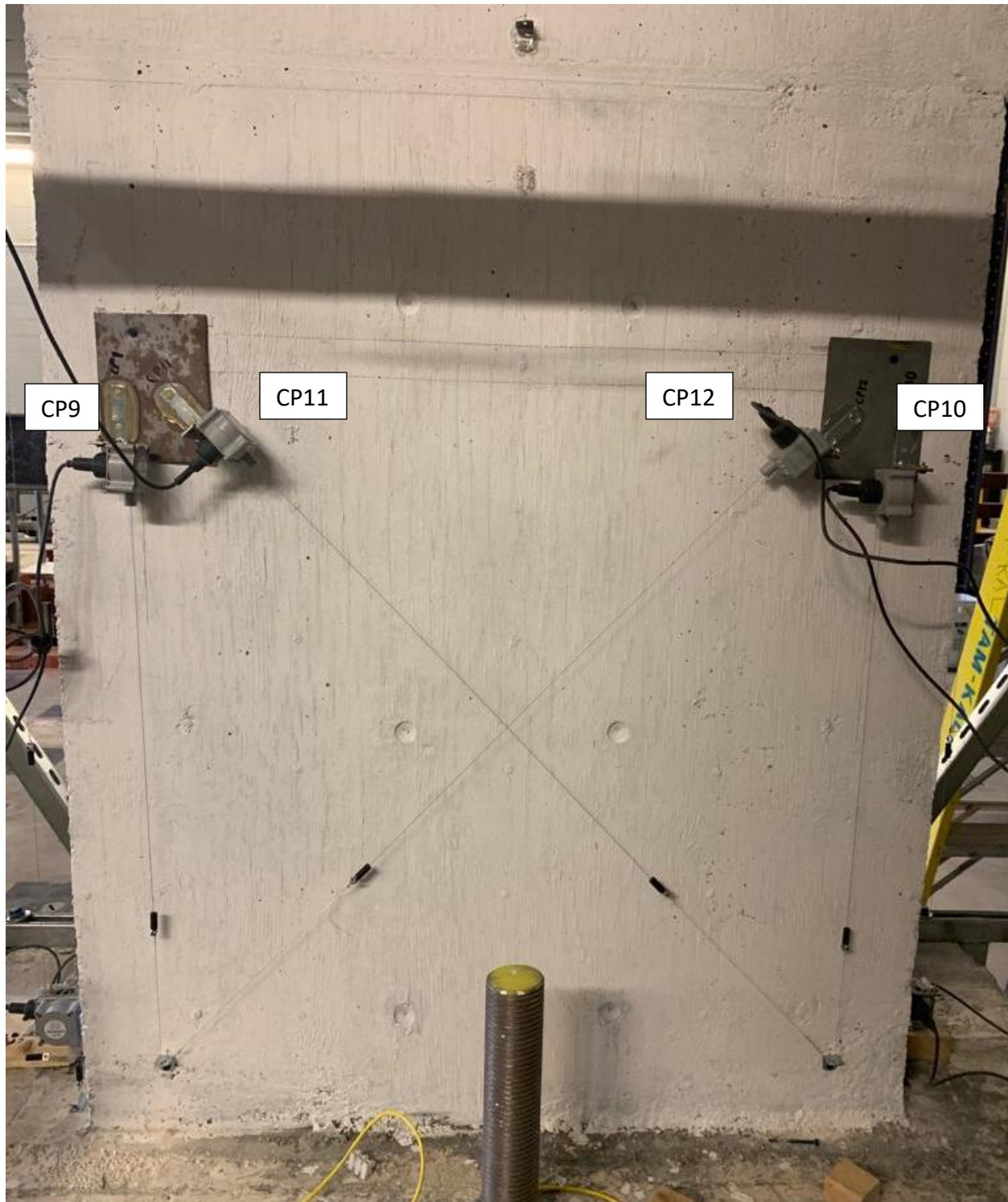


Figure 8.36: Locations of CP9, CP10, CP11, and CP12



Figure 8.37: Location of CP13



Figure 8.38: Location of CP15



Figure 8.39: Location of LP1



Figure 8.40: Location of LP2



Figure 8.41: Clear view of all Strain Gauges

Appendix G – Gravity Load Correction

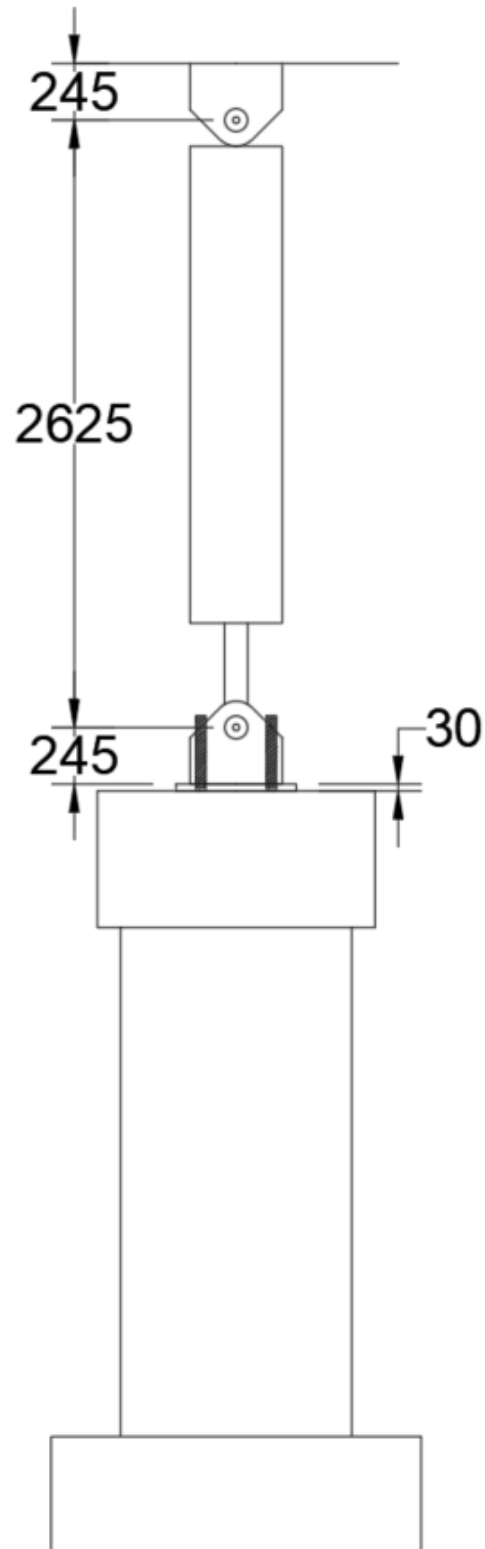


Figure 8.42: Schematic of Actuator and Wall

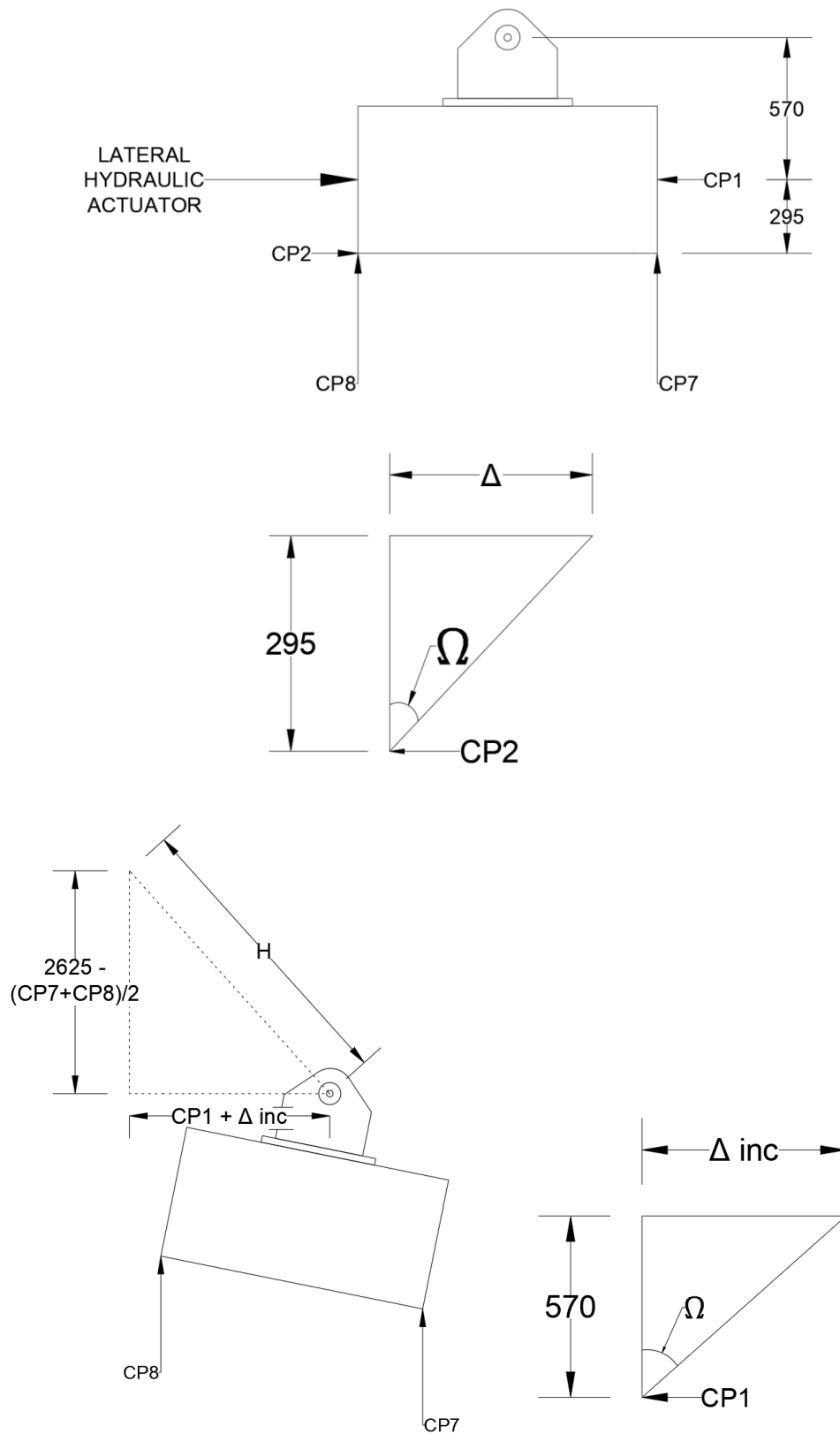


Figure 8.43: Dimensions for Lateral Load Correction

Example for Lateral Correction at Maximum

@ Peak, Lateral Load = 218.39 kN

$$CP1 = 24.74mm, \quad CP2 = 21.25mm, \quad \frac{CP7 + CP8}{2} = \frac{-4.28 + 7.95}{2} = 1.84mm$$

$$\Delta = CP1 - CP2 = 3.059$$

$$\Delta = 295 * \Omega, \quad \Omega = \frac{3.059}{295} = 0.01037 \text{ rad}$$

$$\Delta_{inc} = 570 * \Omega = 5.9109$$

$$CP1 + \Delta_{inc} = 24.74 + 5.9109 = 30.65mm$$

$$H = (2625 - 1.84)^2 + 30.65^2 = 2623.33mm$$

$$V_{inc} = \left(\frac{\text{Gravity Load}}{H} \right) * (CP1 + \Delta_{inc})$$

$$V_{inc} = \left(\frac{575}{2623.33} \right) * (30.65) = 6.72 \text{ kN}$$

Appendix H – Test Frame Set-Up



Figure 8.44: Adding First Cross Beam with Rollers



Figure 8.45: Adding Column Extensions to Columns and Upper Cross Beam



Figure 8.46: Moving Wall into Place



Figure 8.47: Securing Front Cross Beam with Rollers

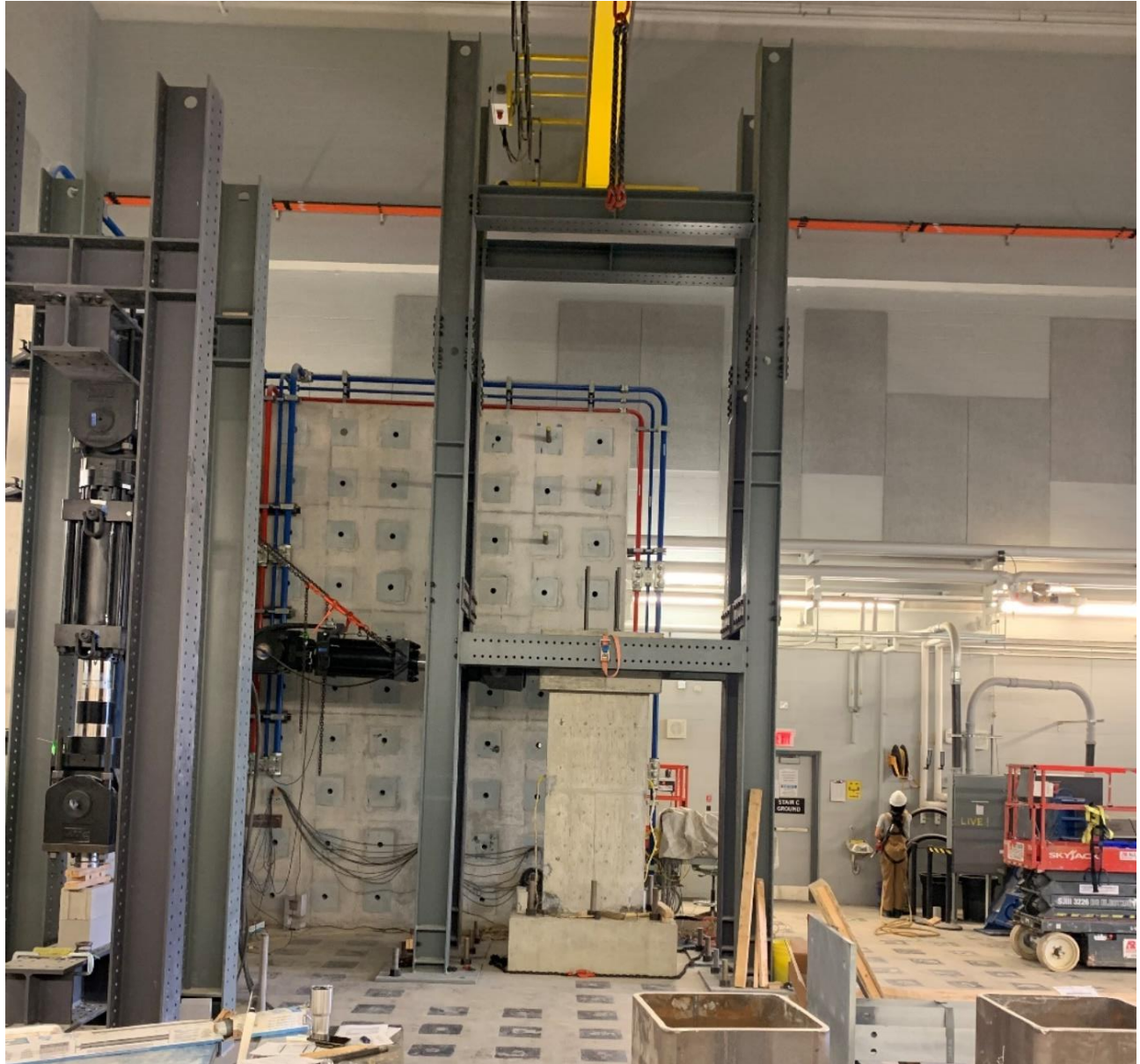


Figure 8.48: Test Frame without Gravity Actuator

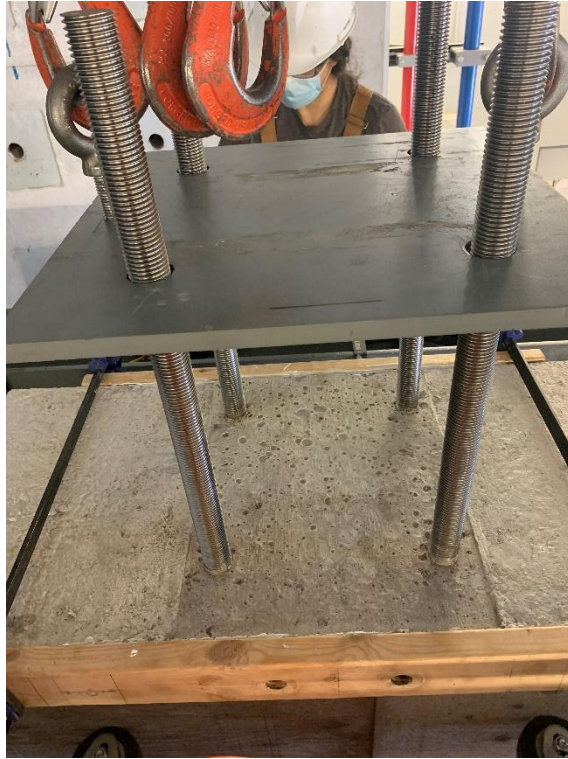


Figure 8.49: Preparing Cap Beam for Plaster under Steel Plate



Figure 8.50: Test Frame with Gravity Actuator, Prior to Adding Short Direction Beams, Top View



Figure 8.51: Test Frame with Gravity Actuator, Prior to Adding Short Direction Beams, Front View



Figure 8.52: Securing Steel Plate and Threaded Rods to Cap Beam



Figure 8.53: Steel Plate Modification, Below



Figure 8.54: Steel Plate Modification

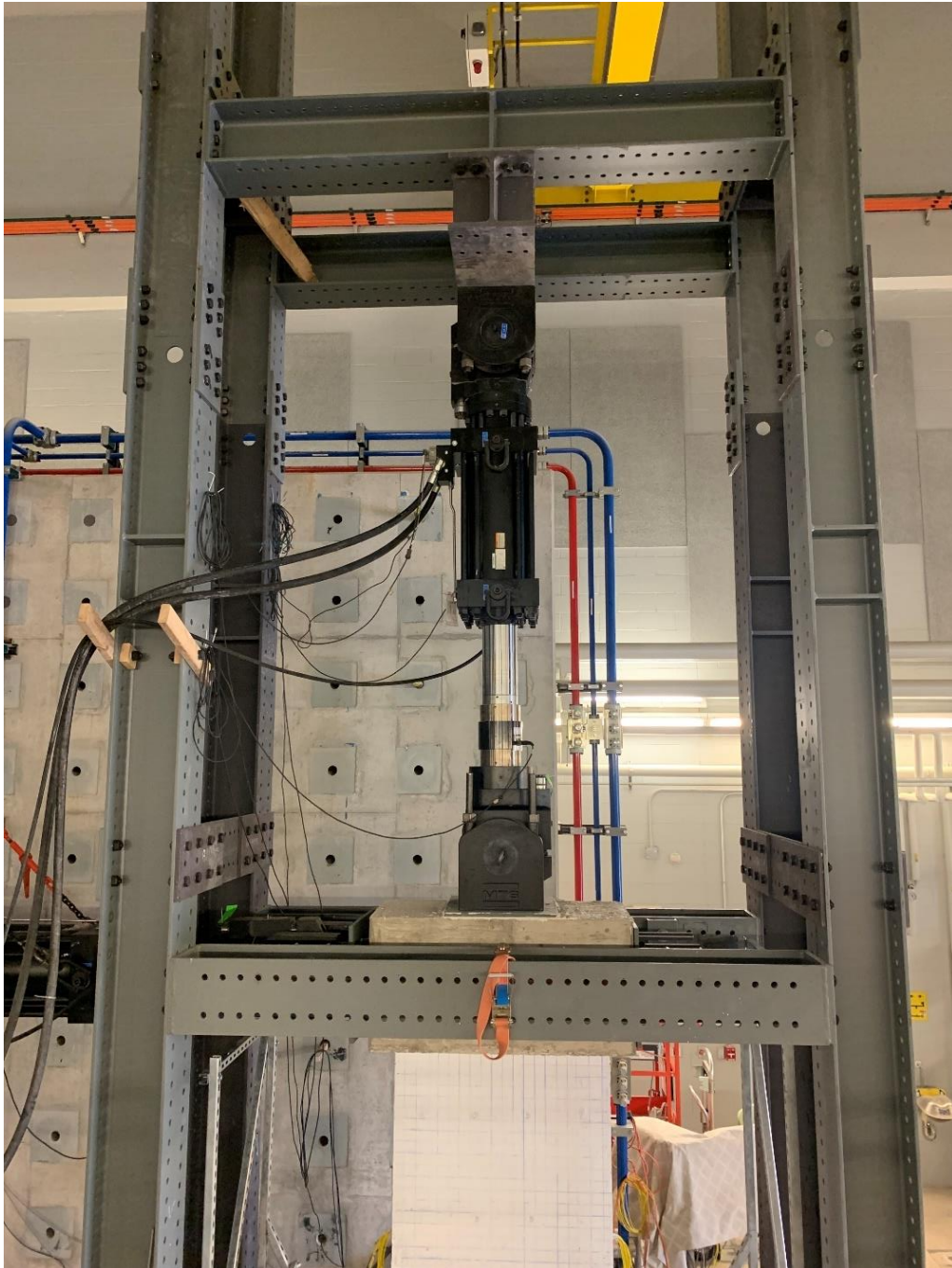


Figure 8.55: Securing Gravity Actuator to the Cap Beam

Appendix I – Strain Gauge Data

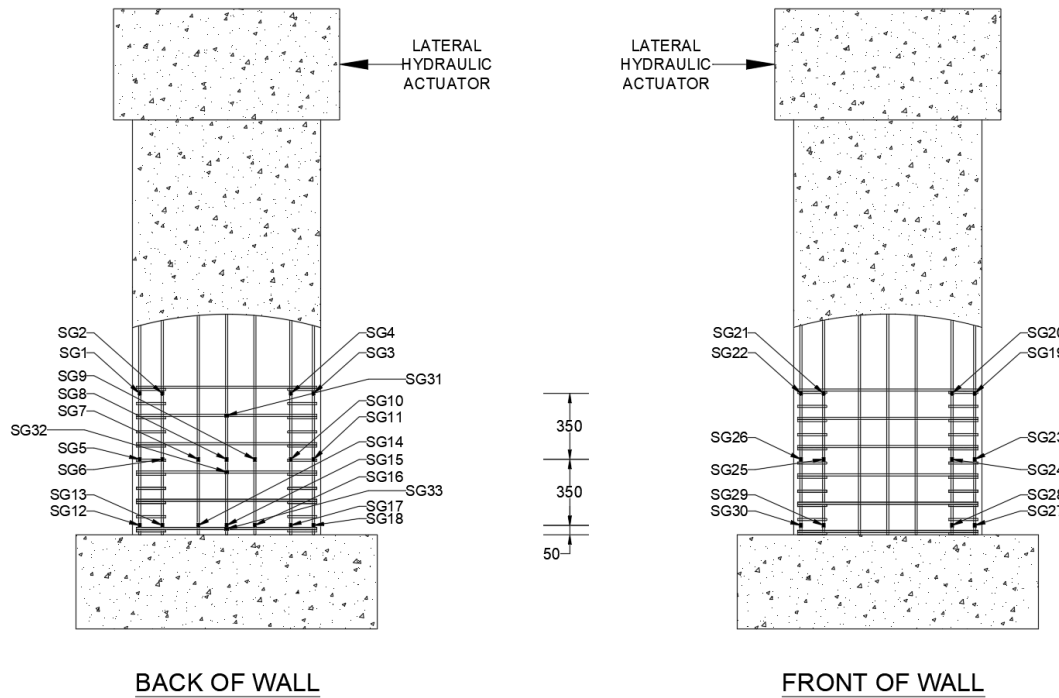


Figure 8.56: Strain Gauge Map

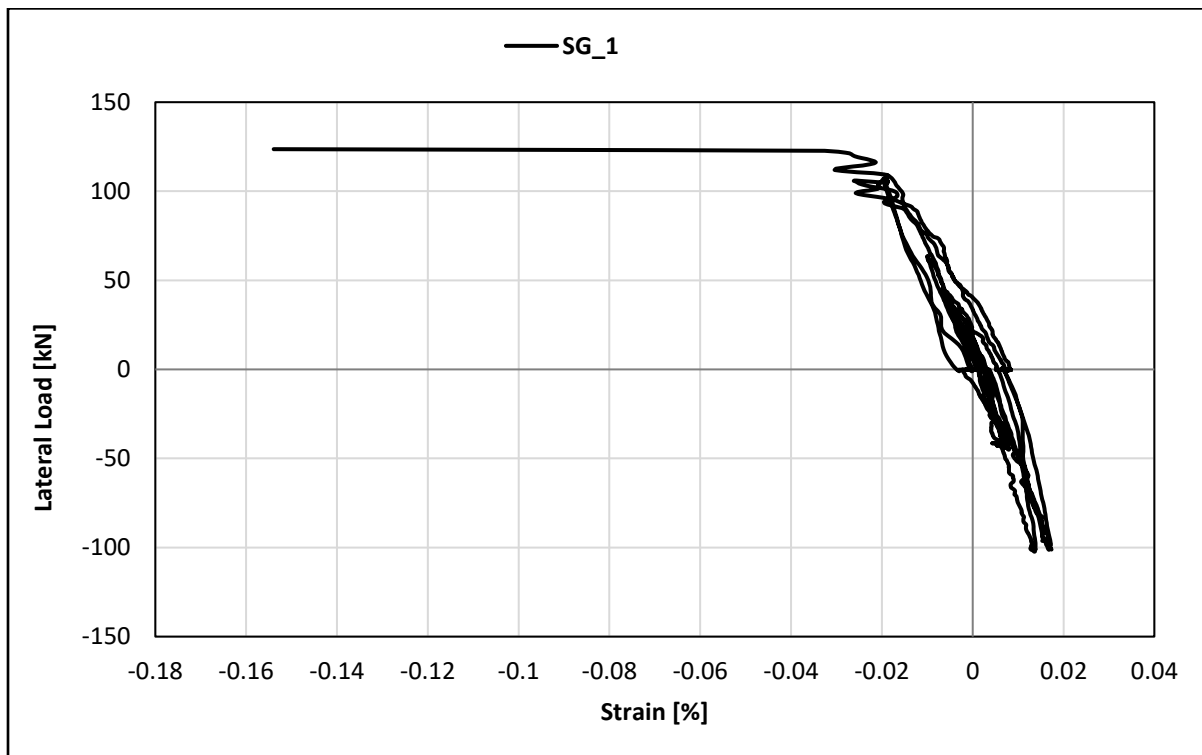


Figure 8.57: Strain Gauge Response for SG 1

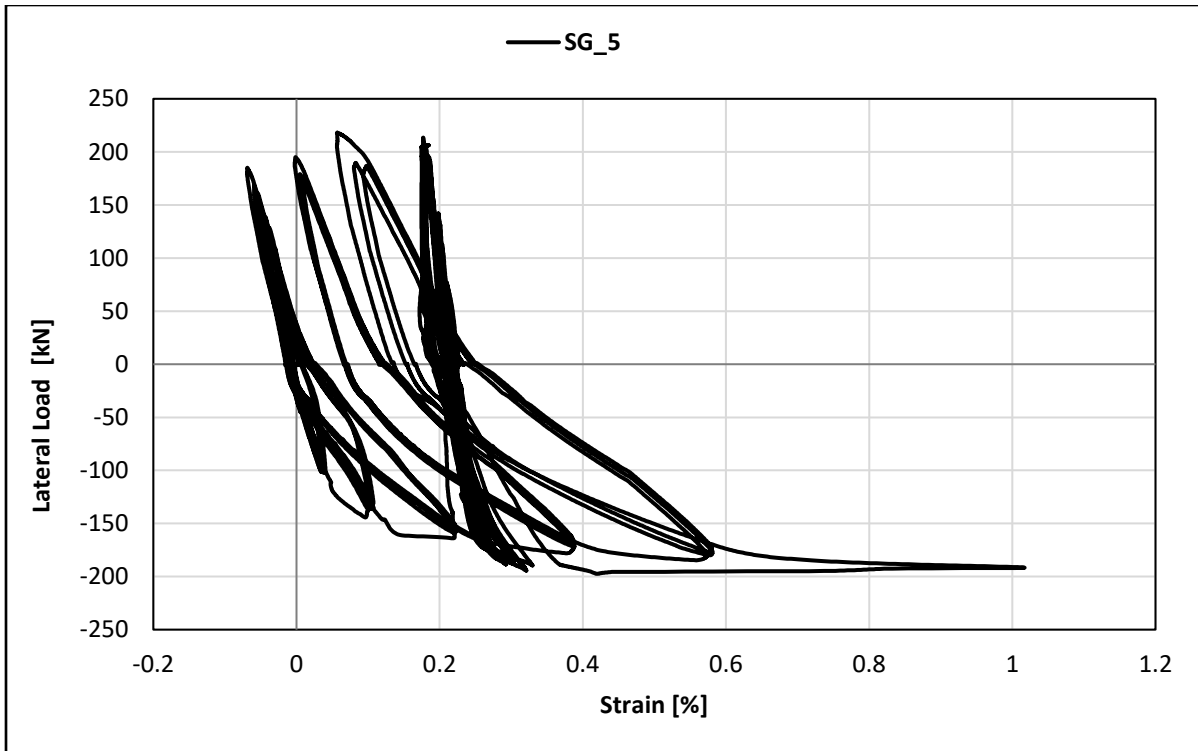


Figure 8.58: Strain Gauge Response for SG 5

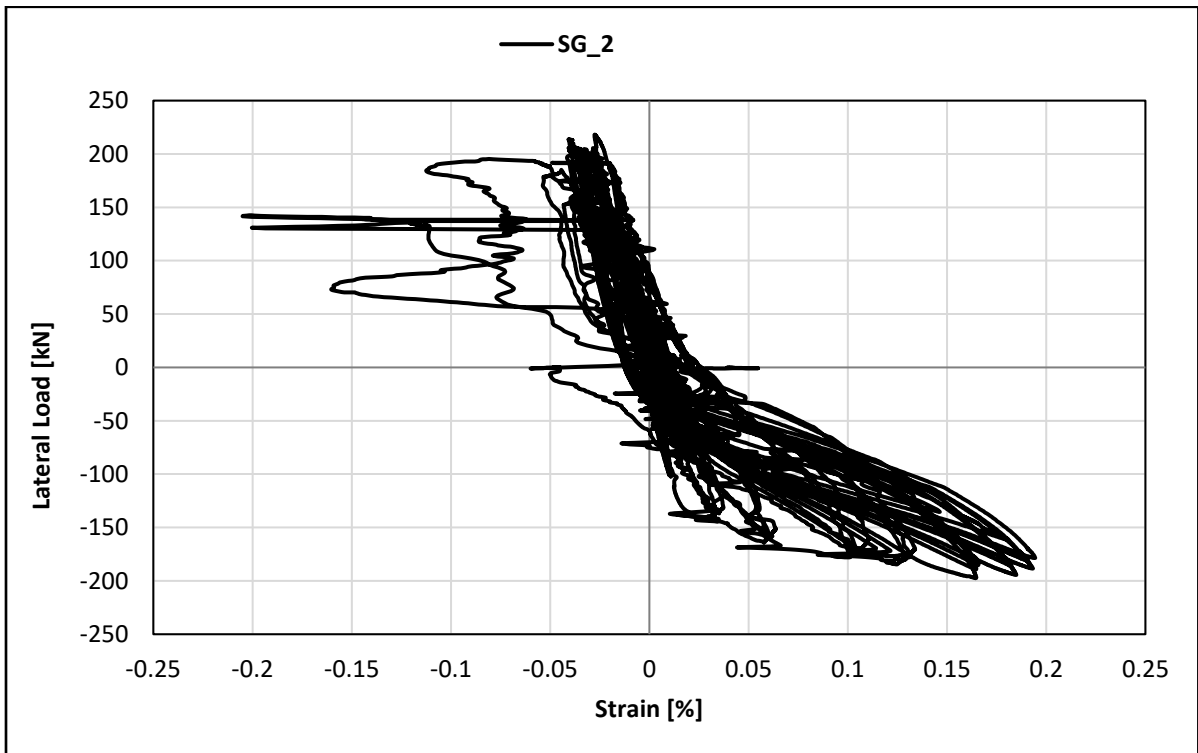


Figure 8.59: Strain Gauge Response for SG 2

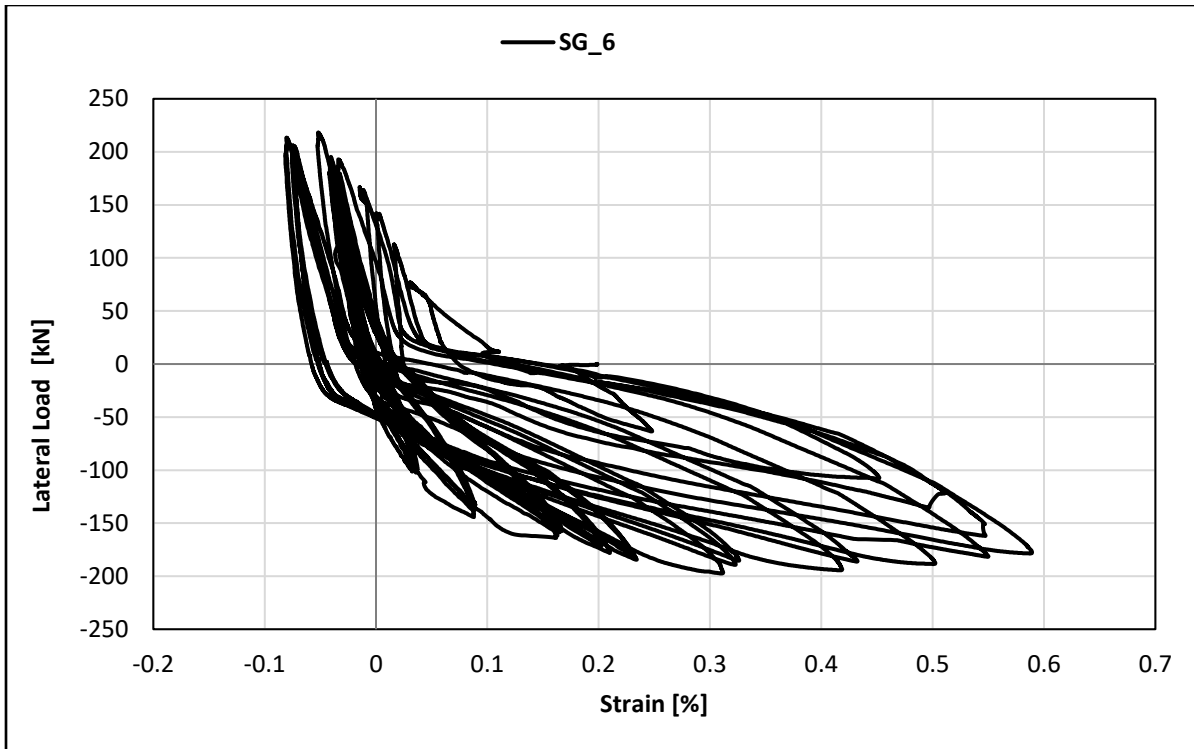


Figure 8.60: Strain Gauge Response for SG 6

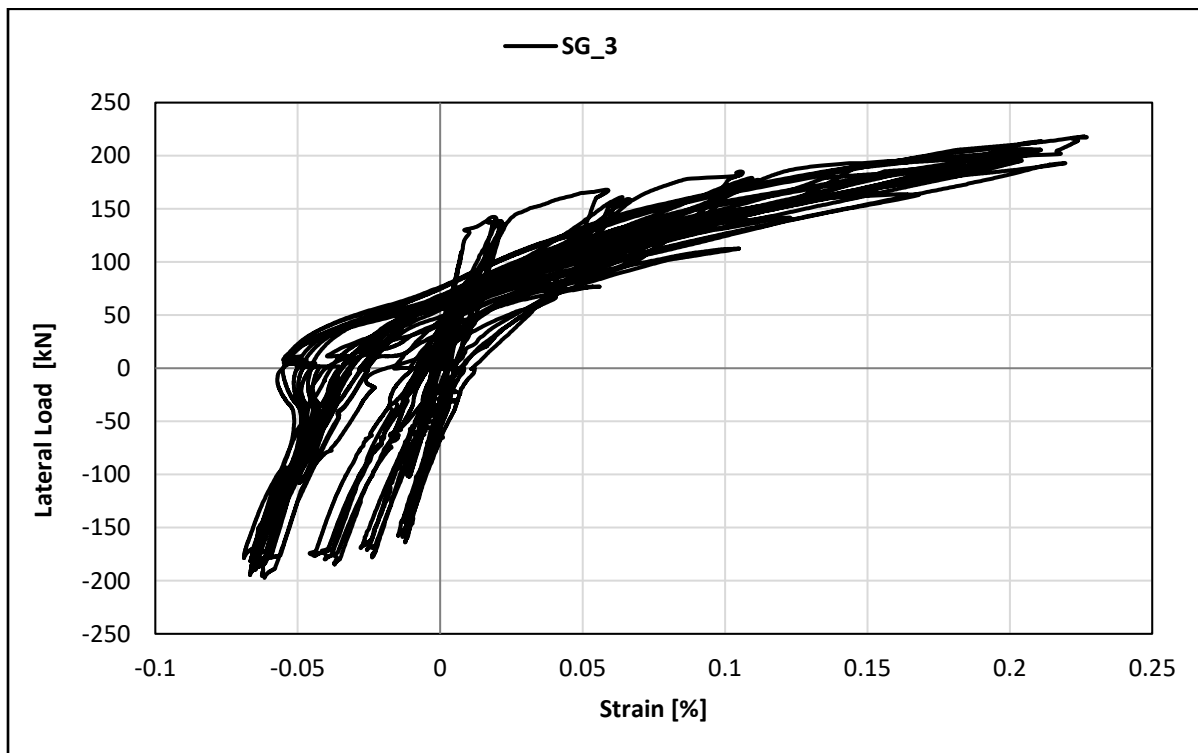


Figure 8.61: Strain Gauge Response for SG 3

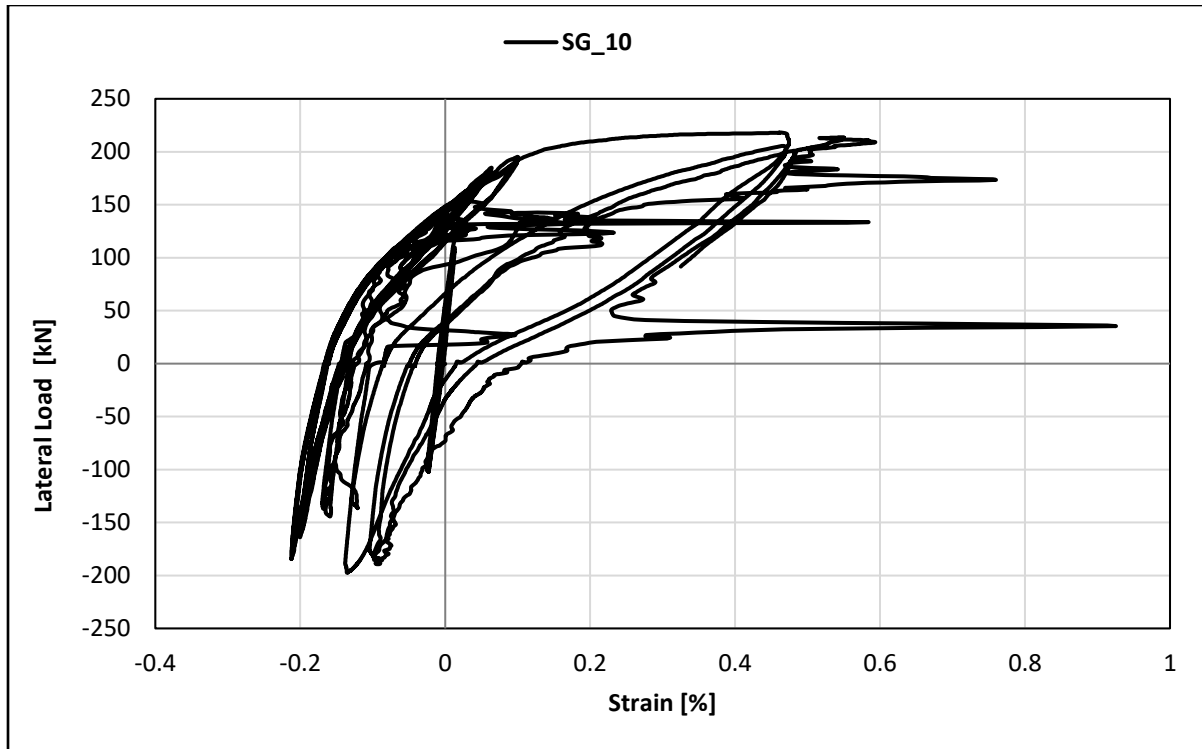


Figure 8.62: Strain Gauge Response for SG 10

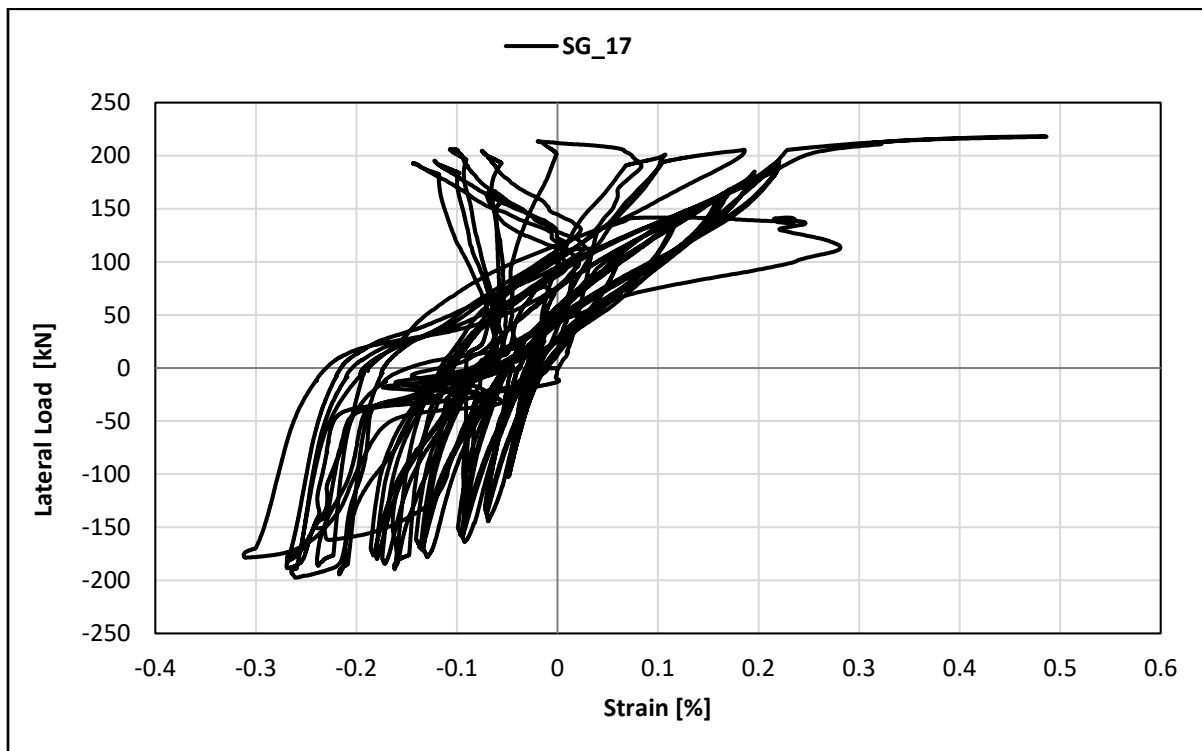


Figure 8.63: Strain Gauge Response for SG 17

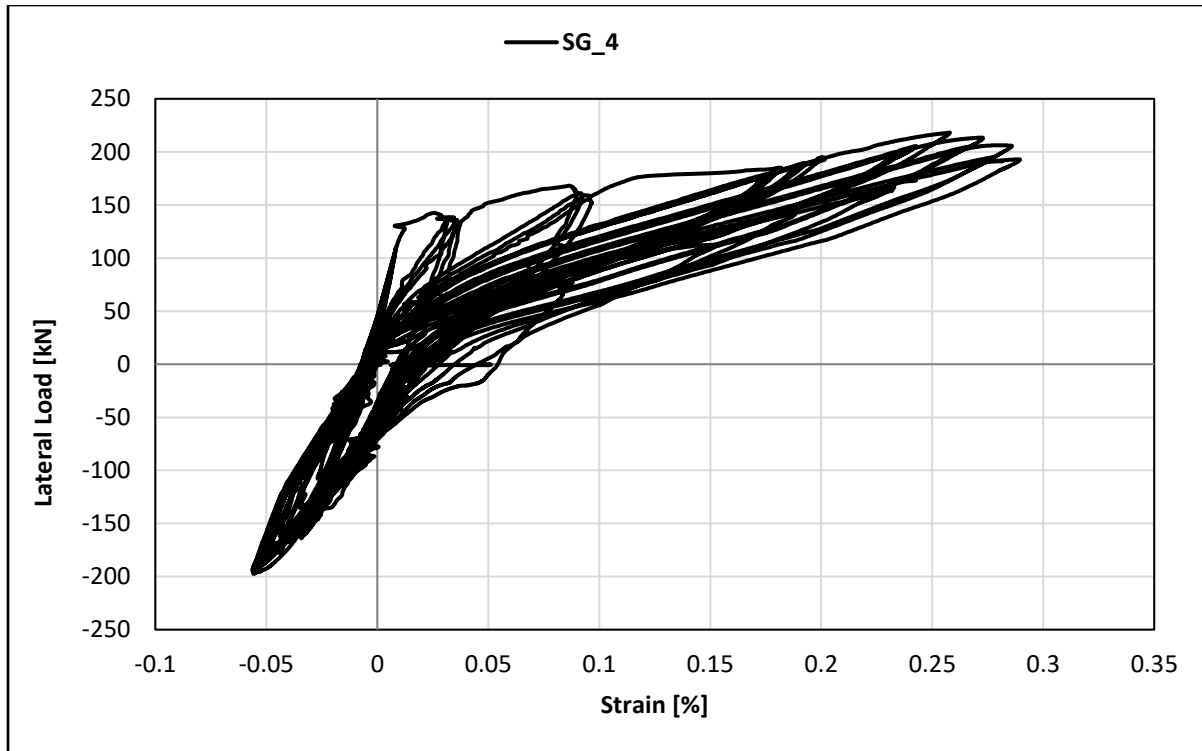


Figure 8.64: Strain Gauge Response for SG 4

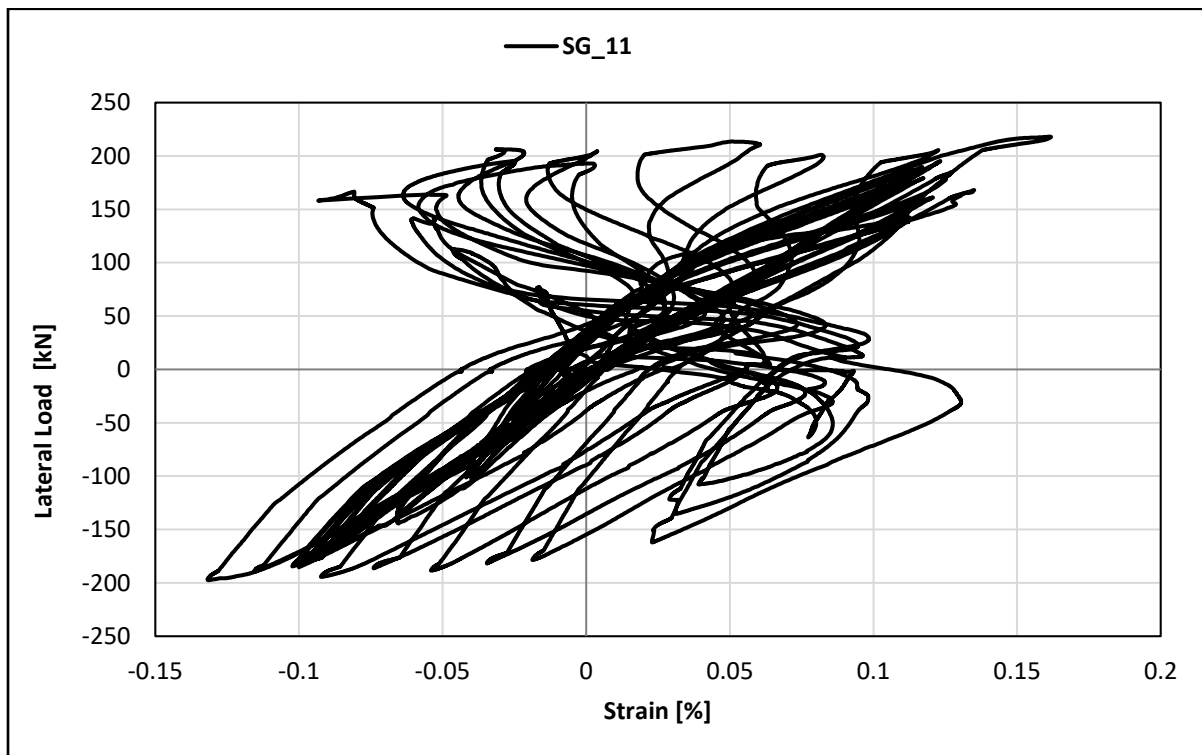


Figure 8.65: Strain Gauge Response for SG 11

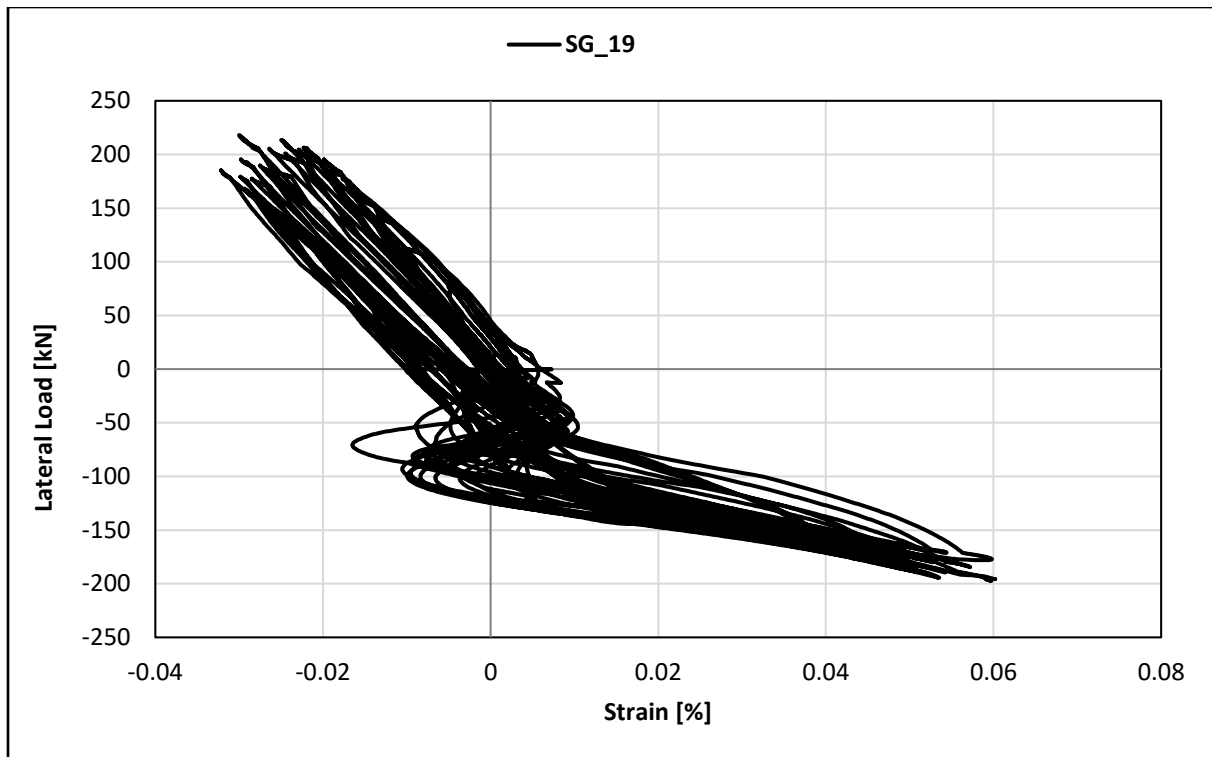


Figure 8.66: Strain Gauge Response for SG 19

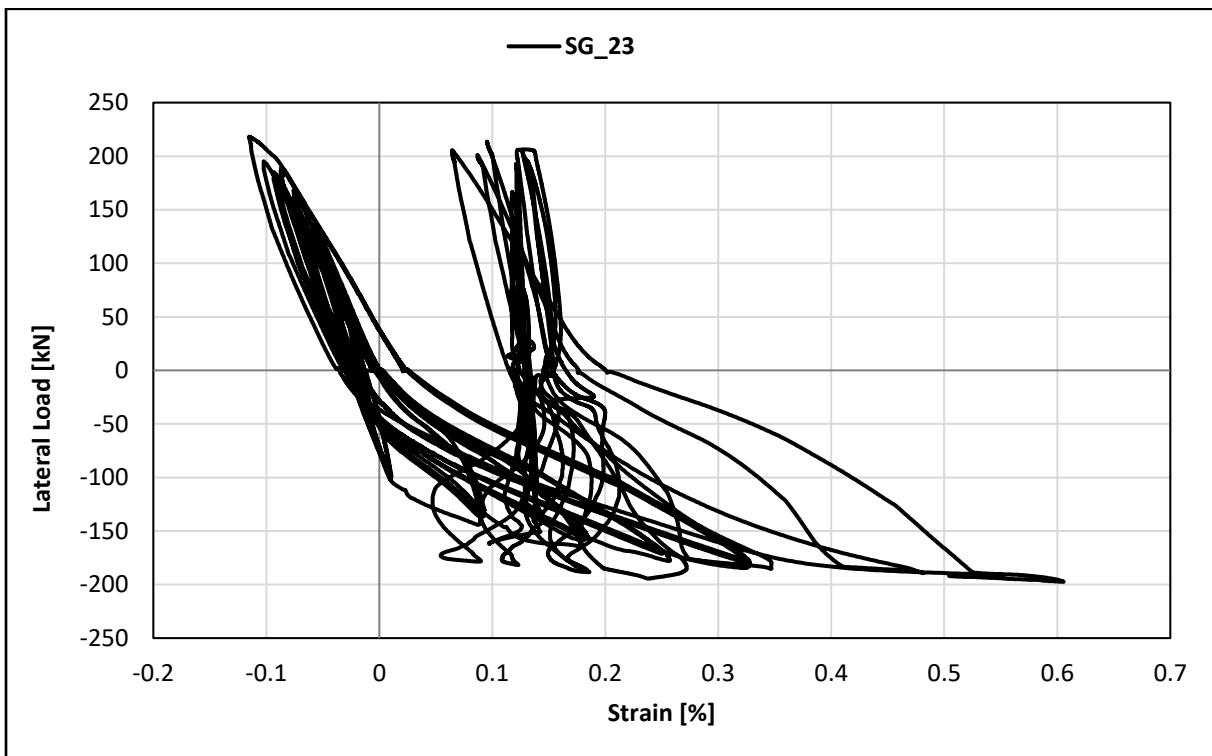


Figure 8.67: Strain Gauge Response for SG 23

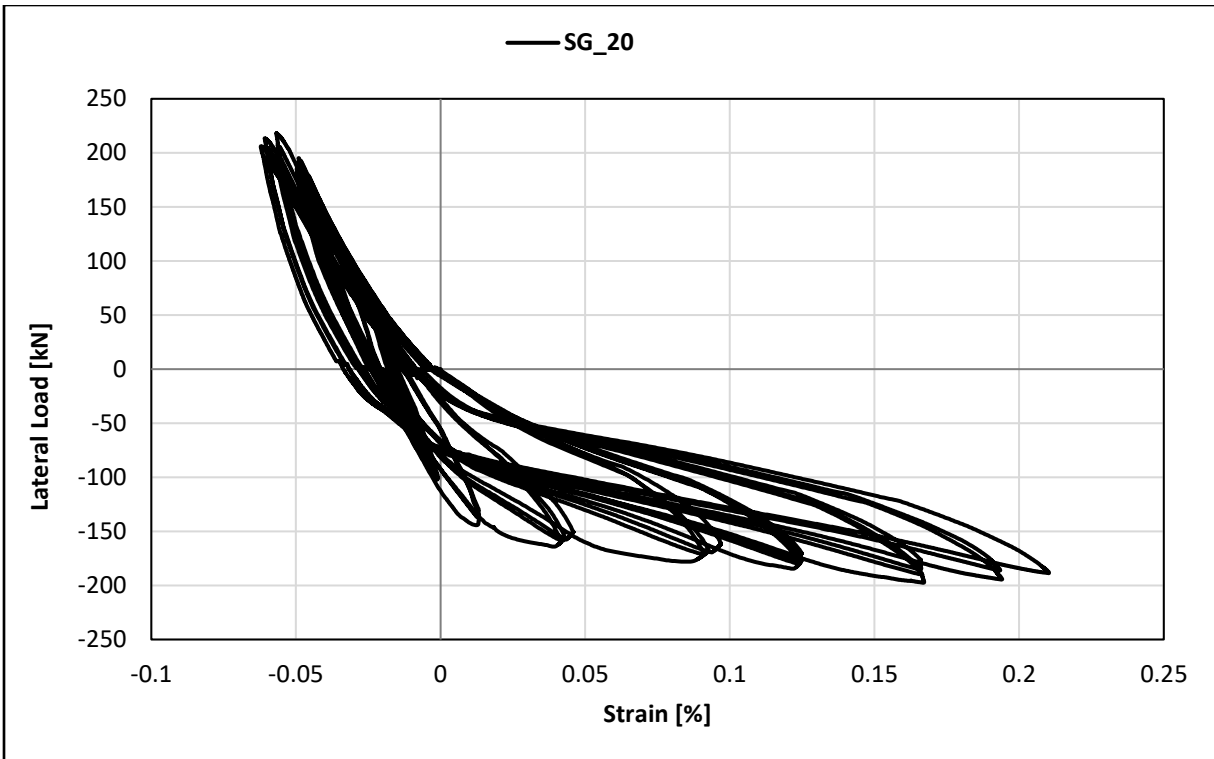


Figure 8.68: Strain Gauge Response for SG 20

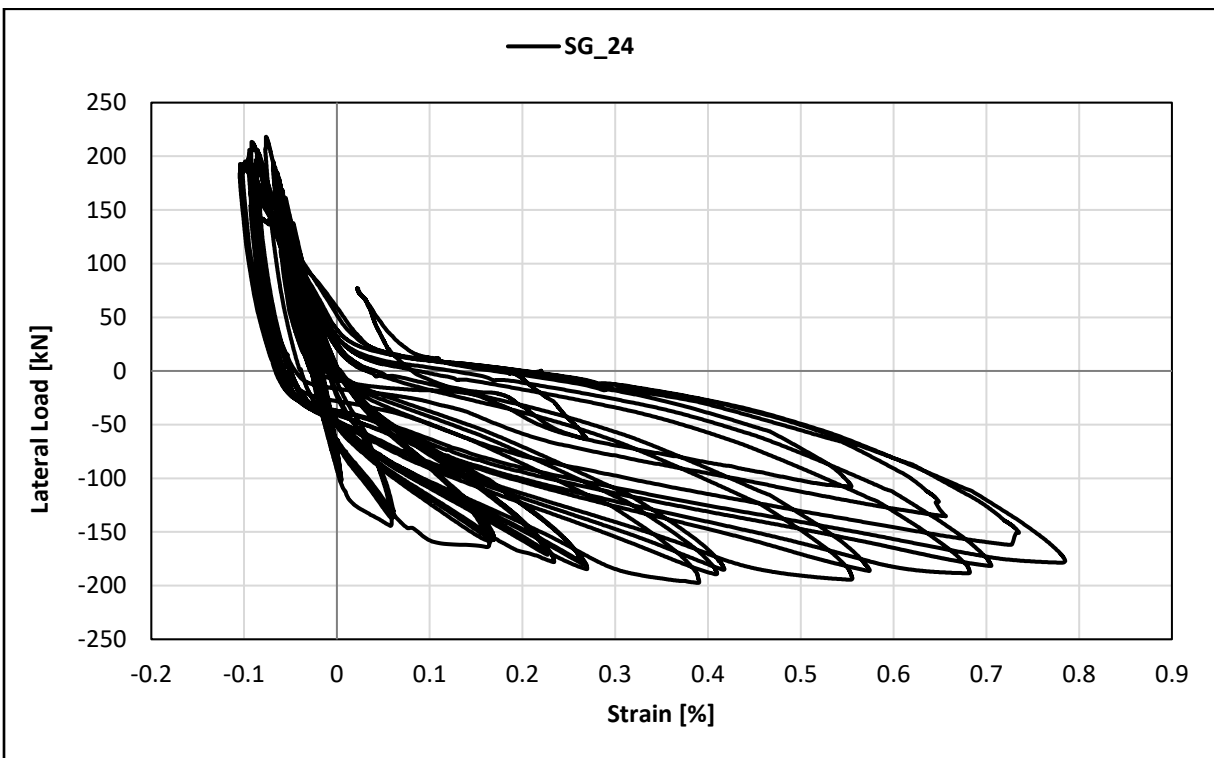


Figure 8.69: Strain Gauge Response for SG 24

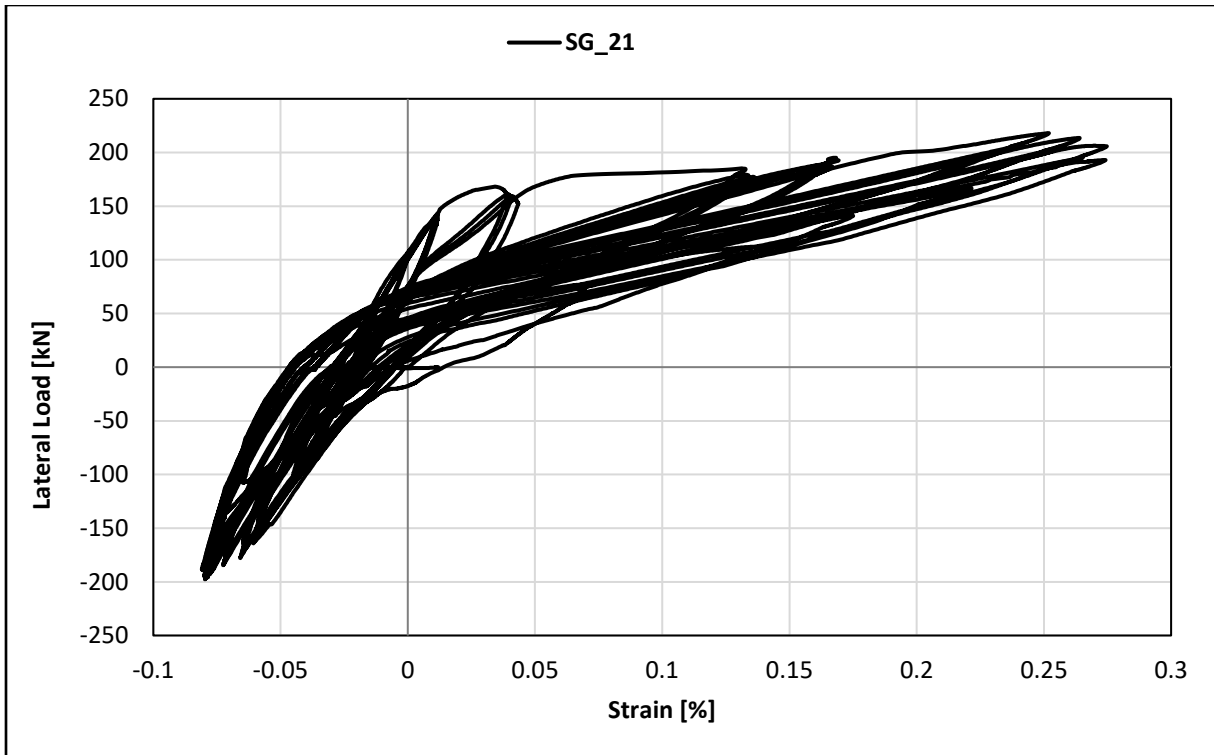


Figure 8.70: Strain Gauge Response for SG 21

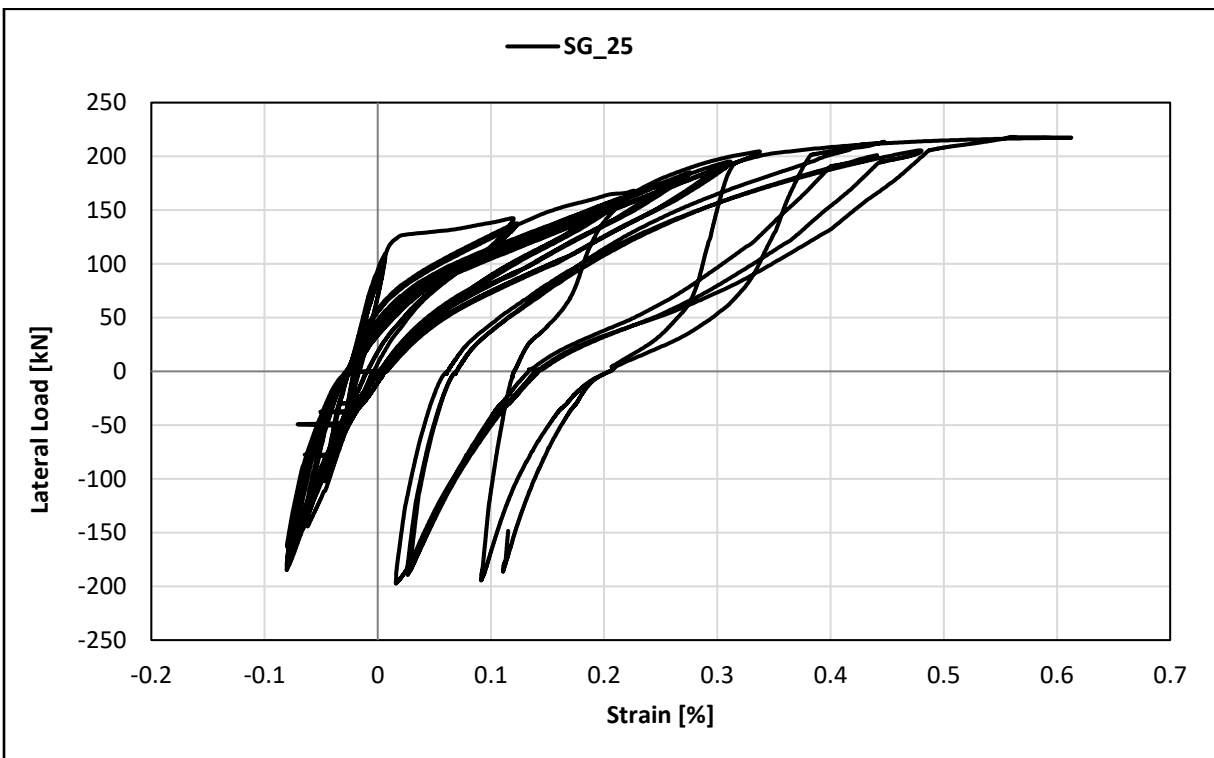


Figure 8.71: Strain Gauge Response for SG 25

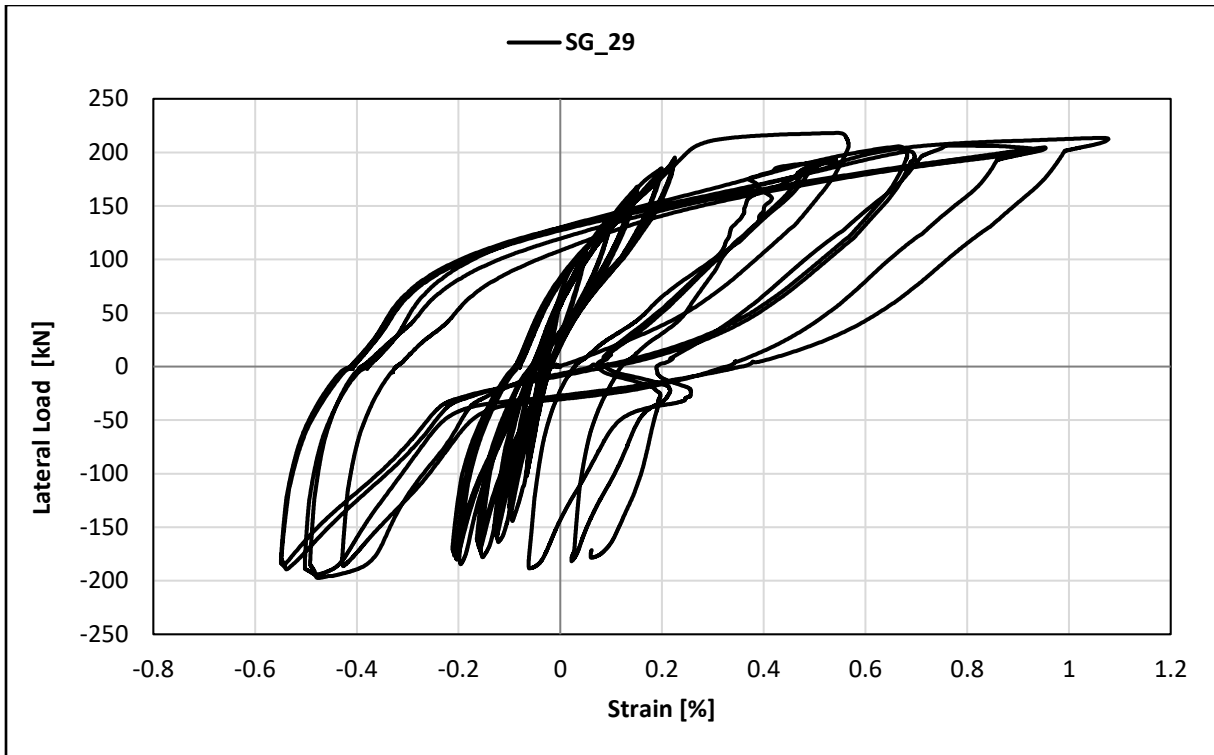


Figure 8.72: Strain Gauge Response for SG 29

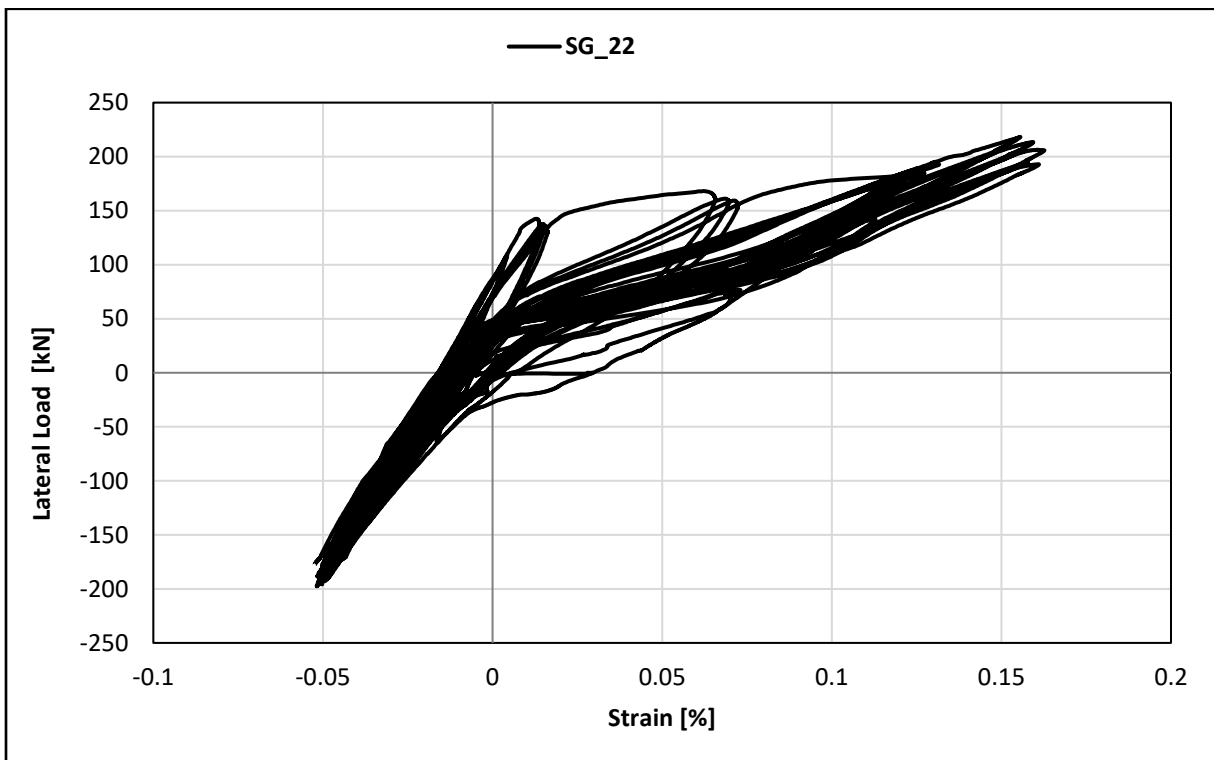


Figure 8.73: Strain Gauge Response for SG 22

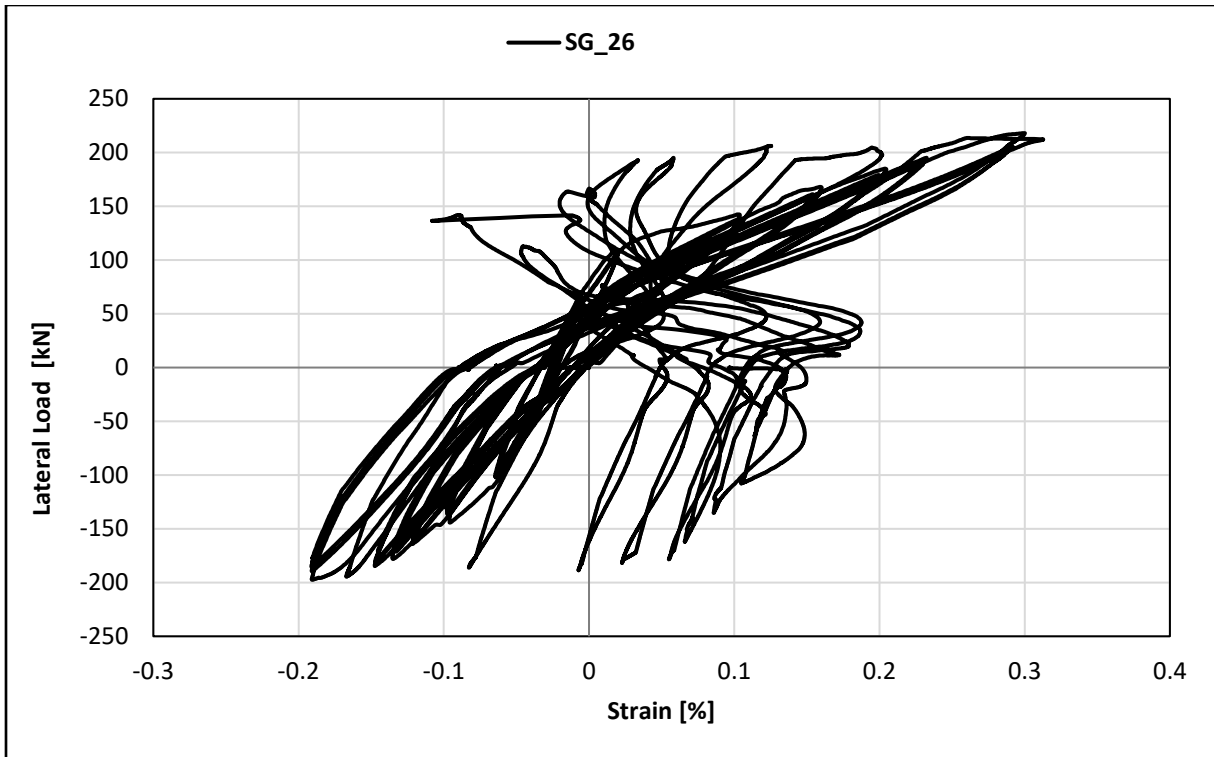


Figure 8.74: Strain Gauge Response for SG 26

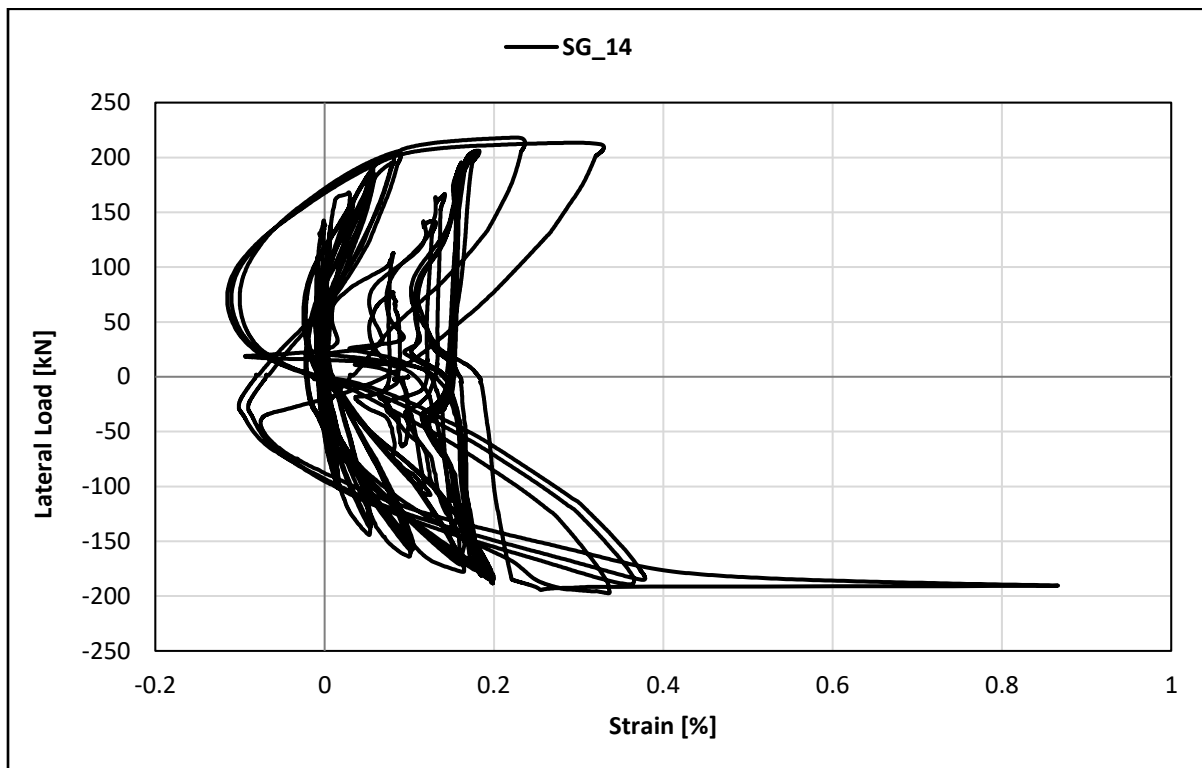


Figure 8.75: Strain Gauge Response for SG 14

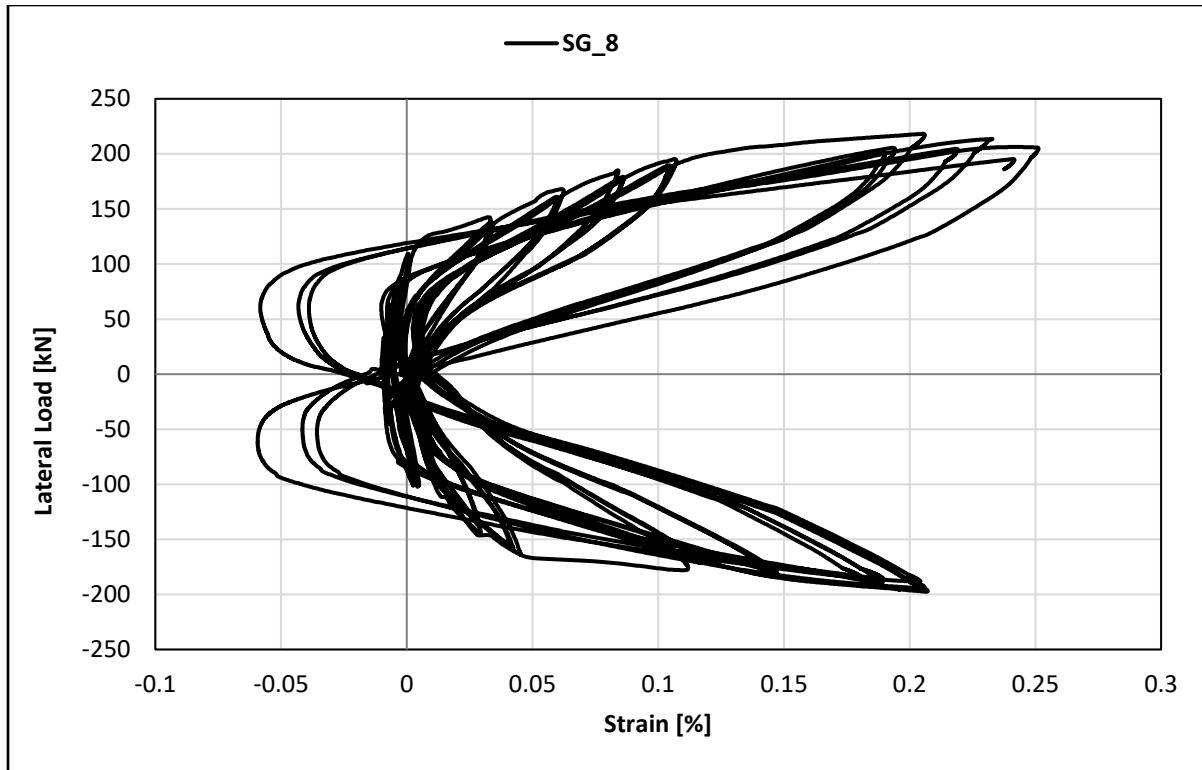


Figure 8.76: Strain Gauge Response for SG 8

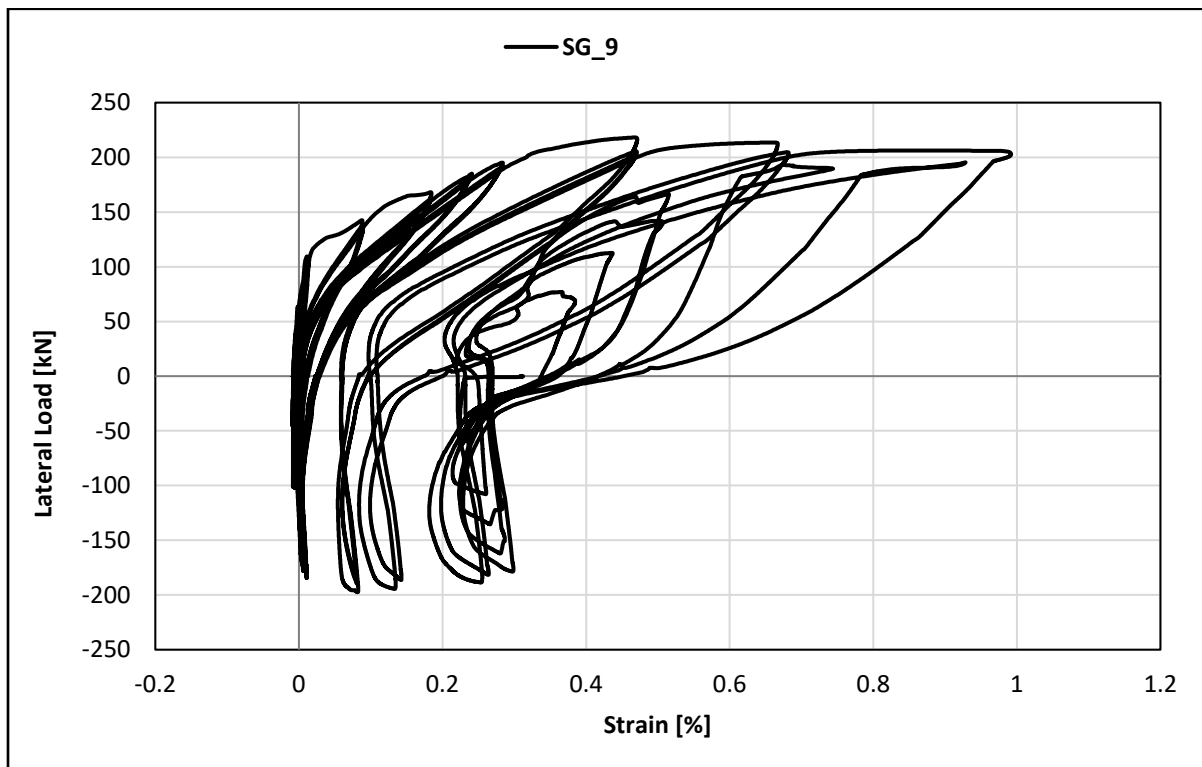


Figure 8.77: Strain Gauge Response for SG 9

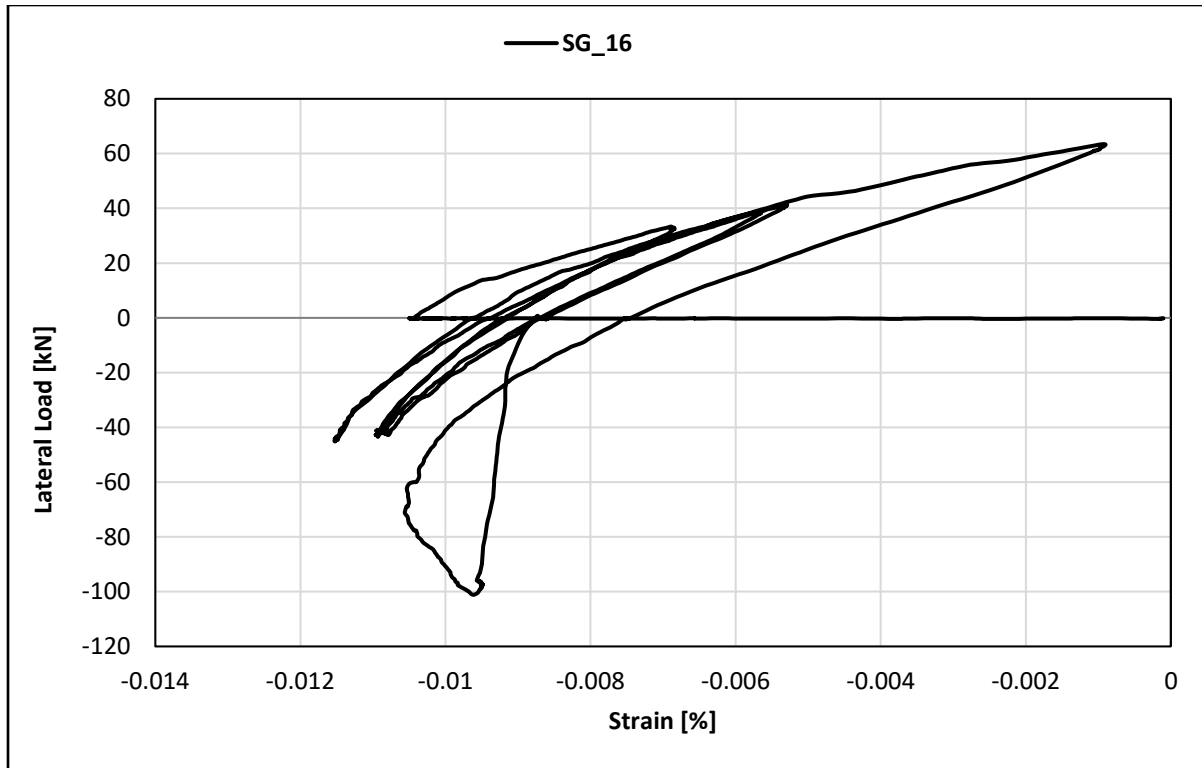


Figure 8.78: Strain Gauge Response for SG 16

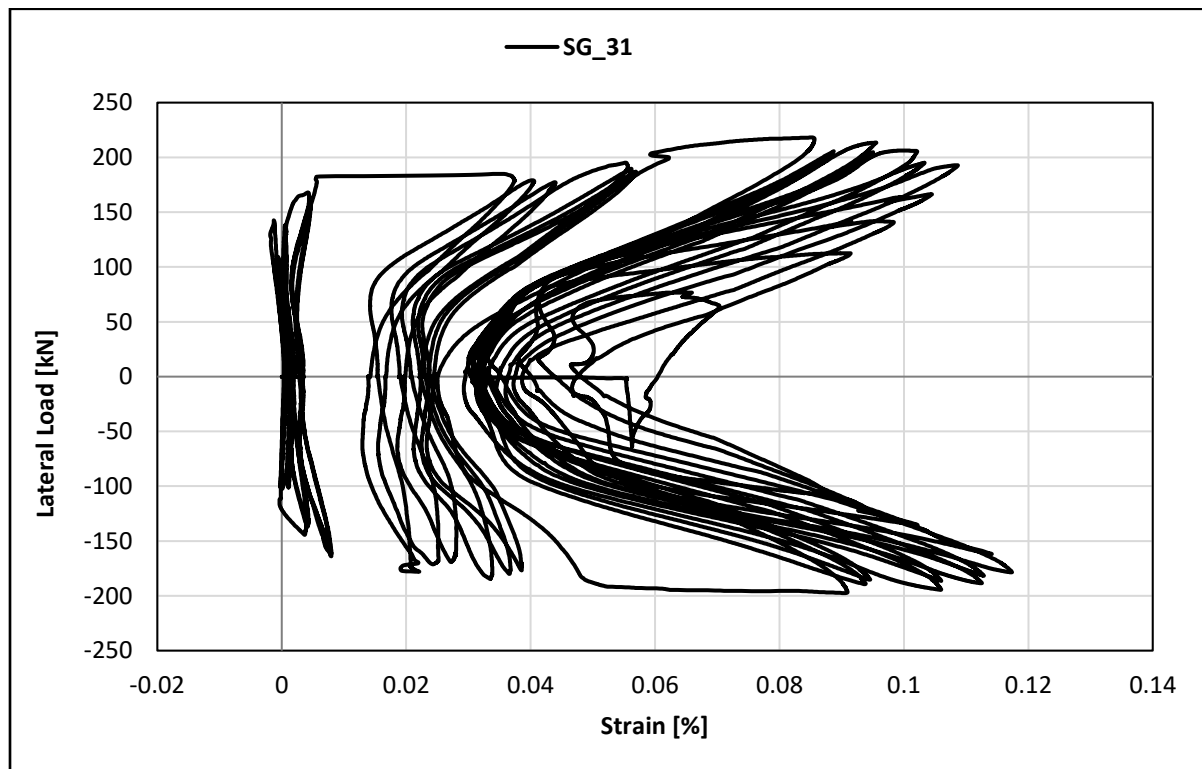


Figure 8.79: Strain Gauge Response for SG 31

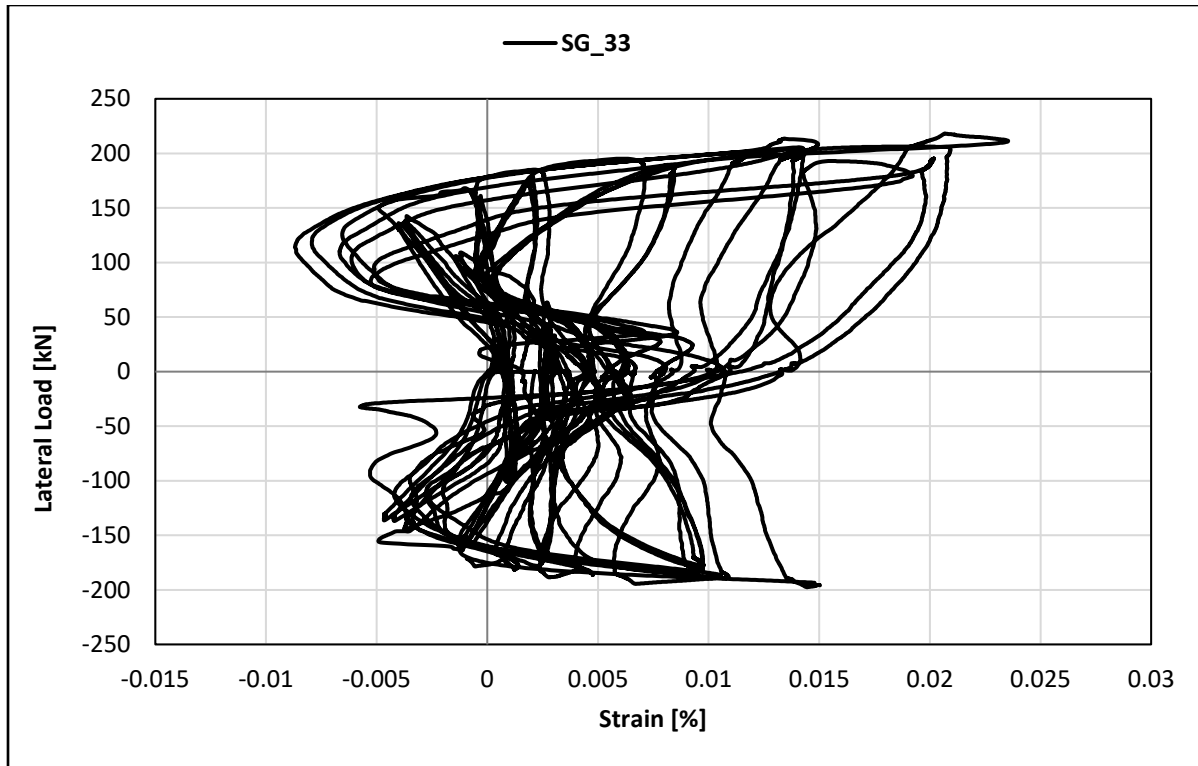


Figure 8.80: Strain Gauge Response for SG 33

Appendix J – Finite Element Modelling

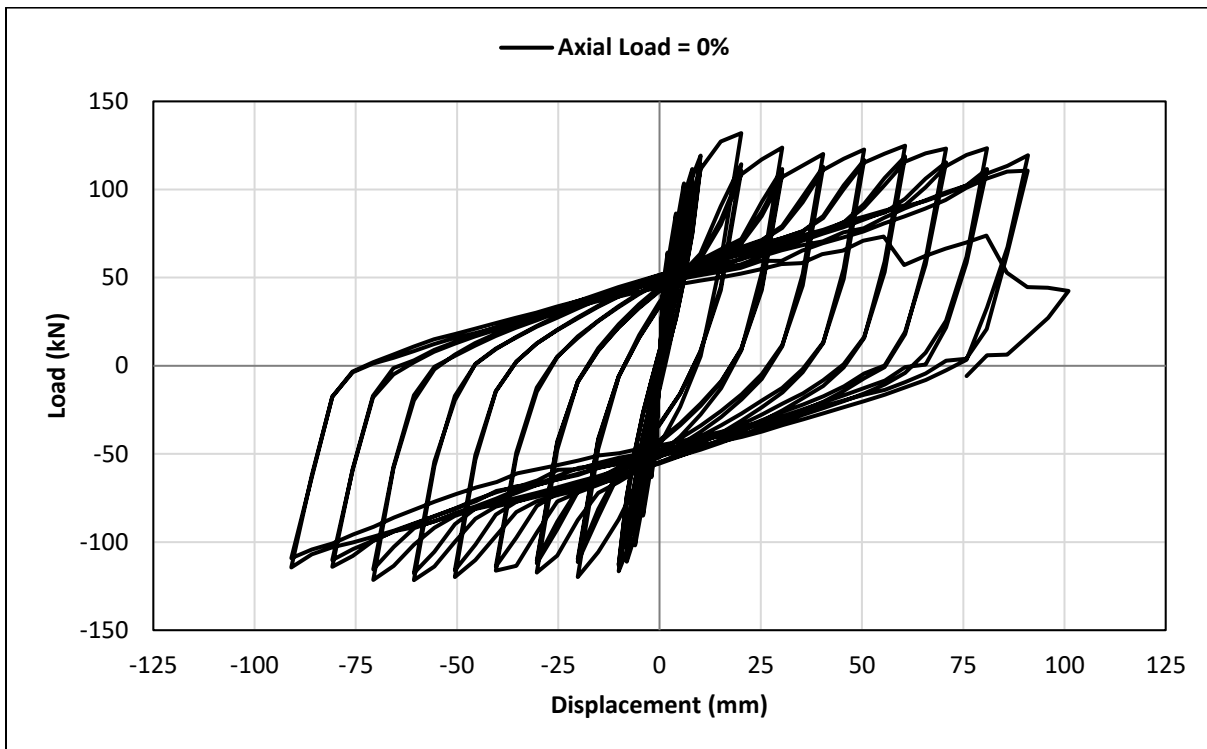


Figure 8.81: Reverse Cyclic Analysis for 0% Gravity Load using Model V1

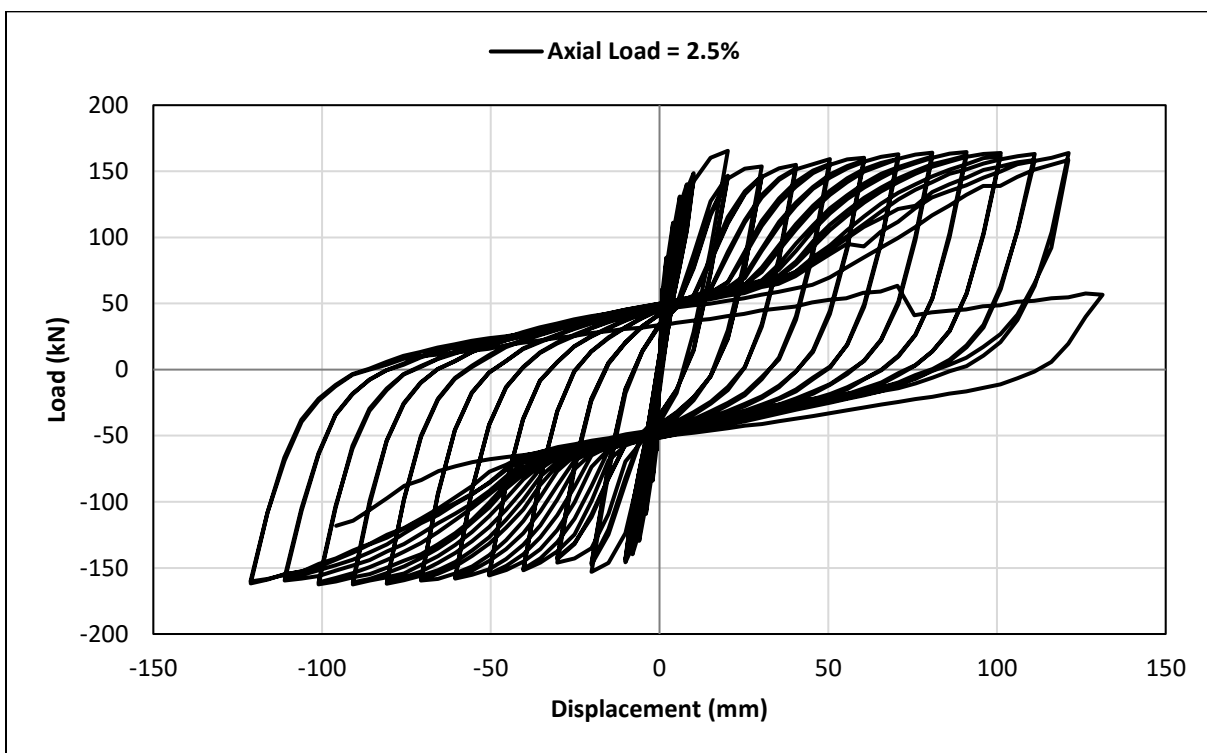


Figure 8.82: Reverse Cyclic Analysis for 2.5% Gravity Load using Model V1

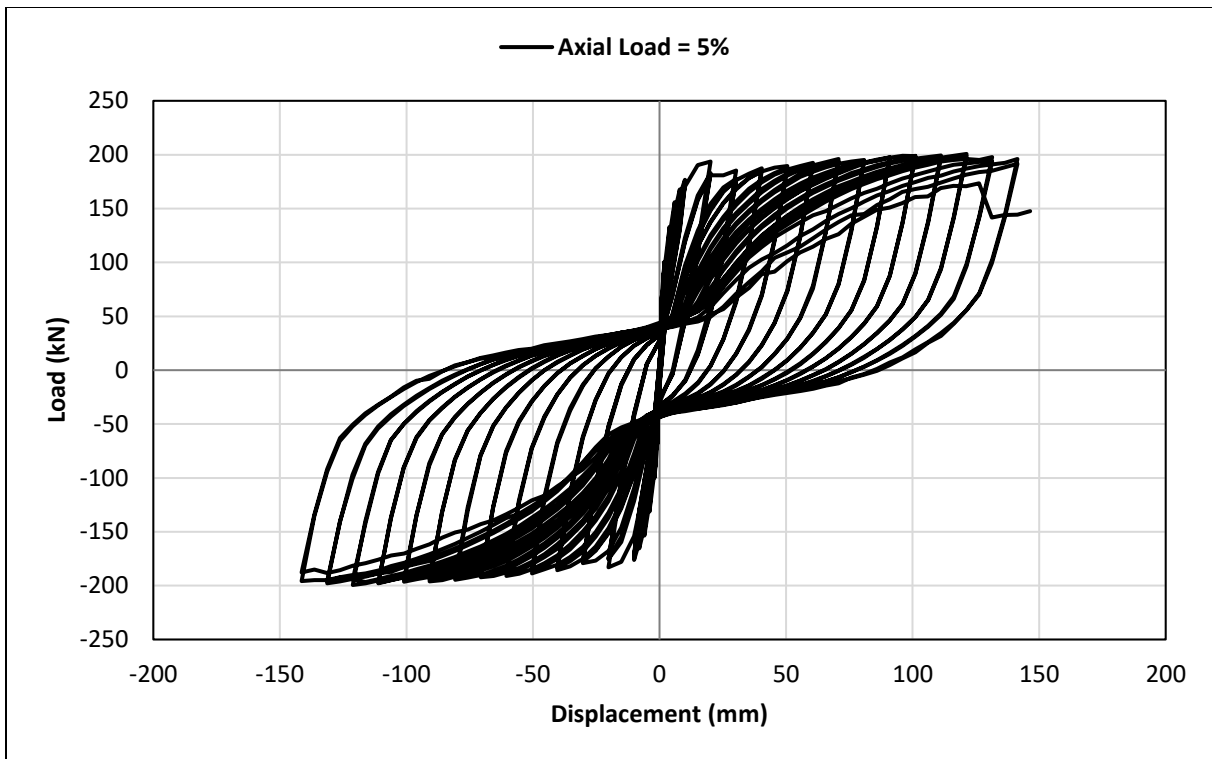


Figure 8.83: Reverse Cyclic Analysis for 5% Gravity Load using Model V1

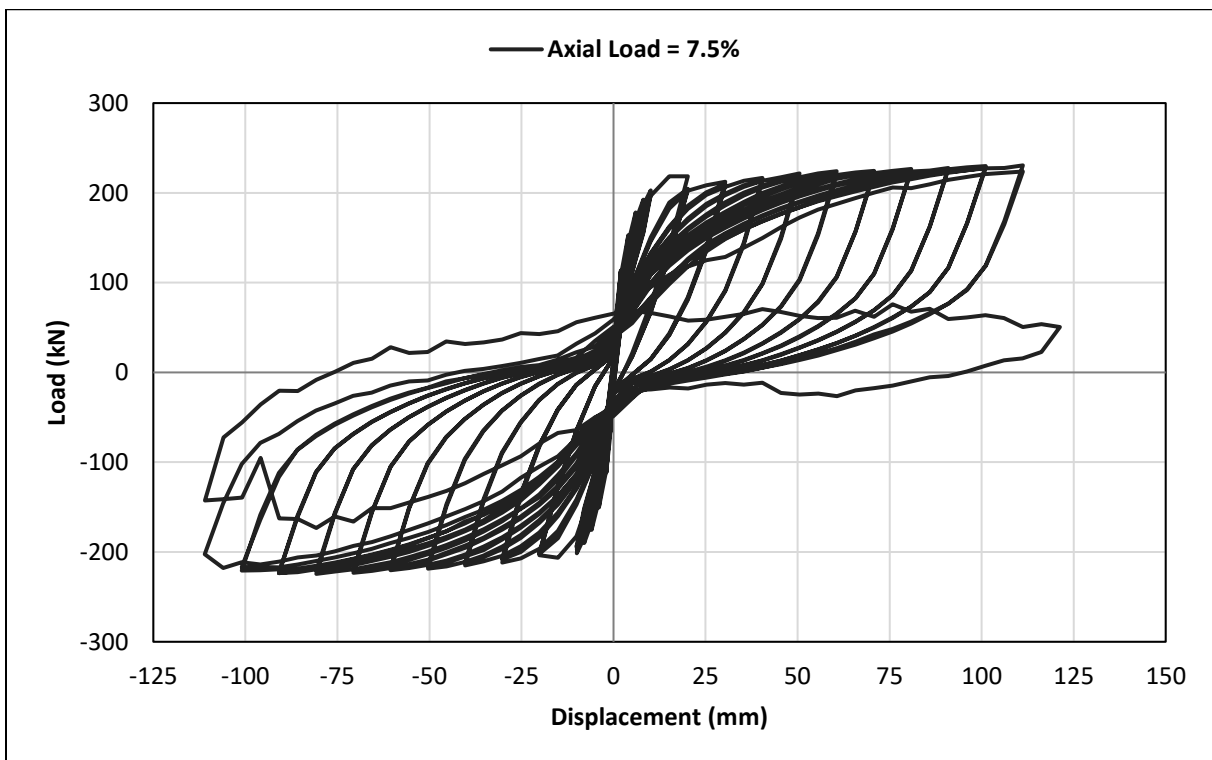


Figure 8.84: Reverse Cyclic Analysis for 7.5% Gravity Load using Model V1

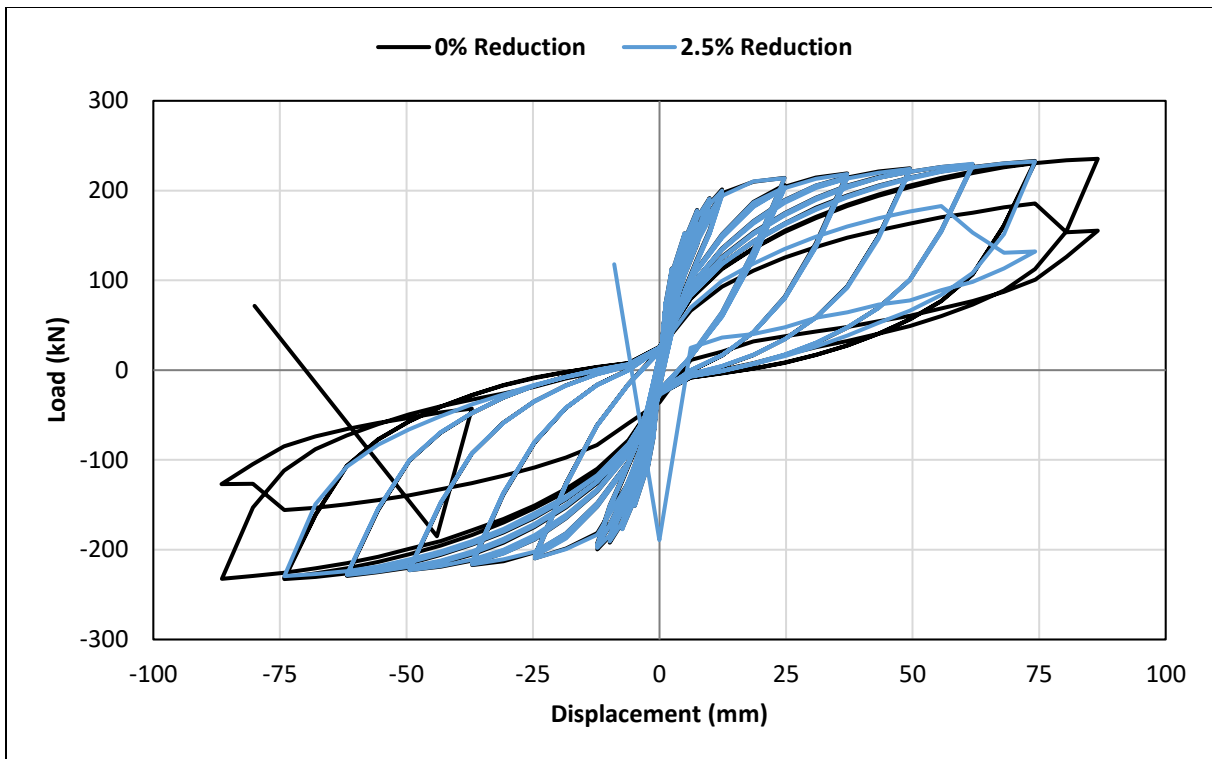


Figure 8.85: 0% and 2.5% Bar Reduction on Model V3

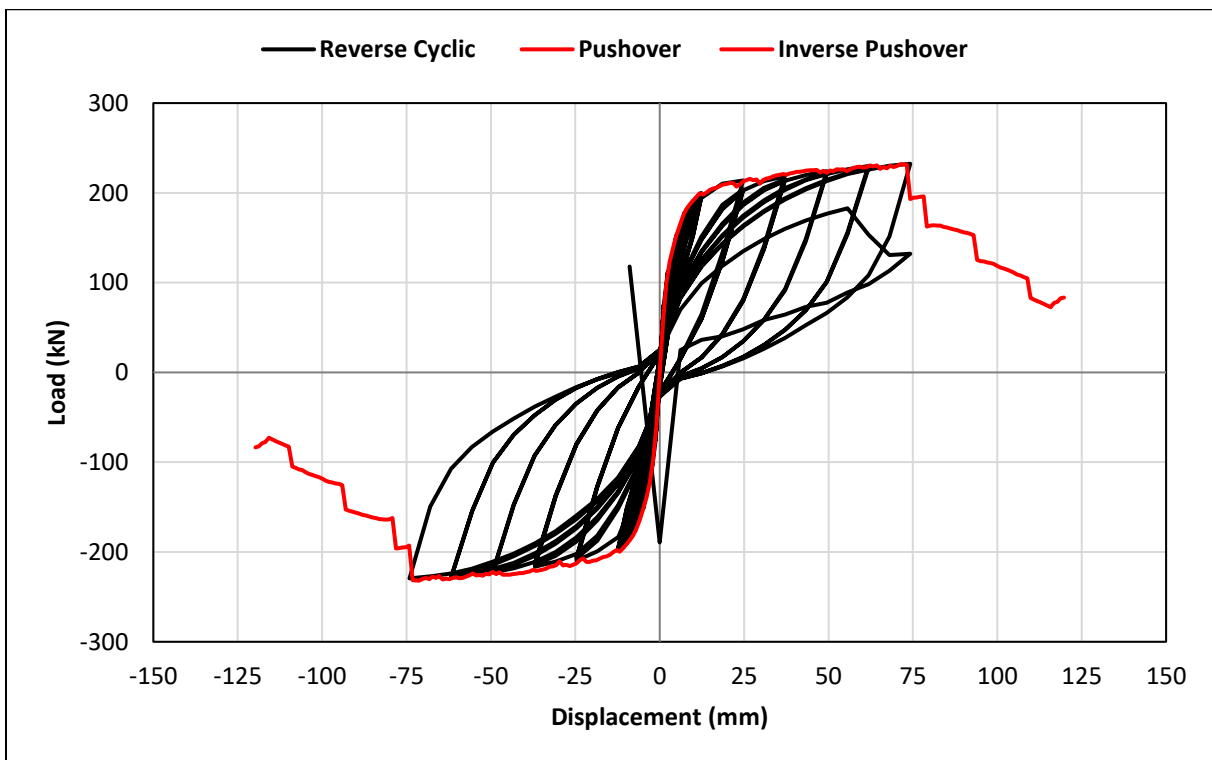


Figure 8.86: 2.5% Bar Reduction with Pushover Model V2

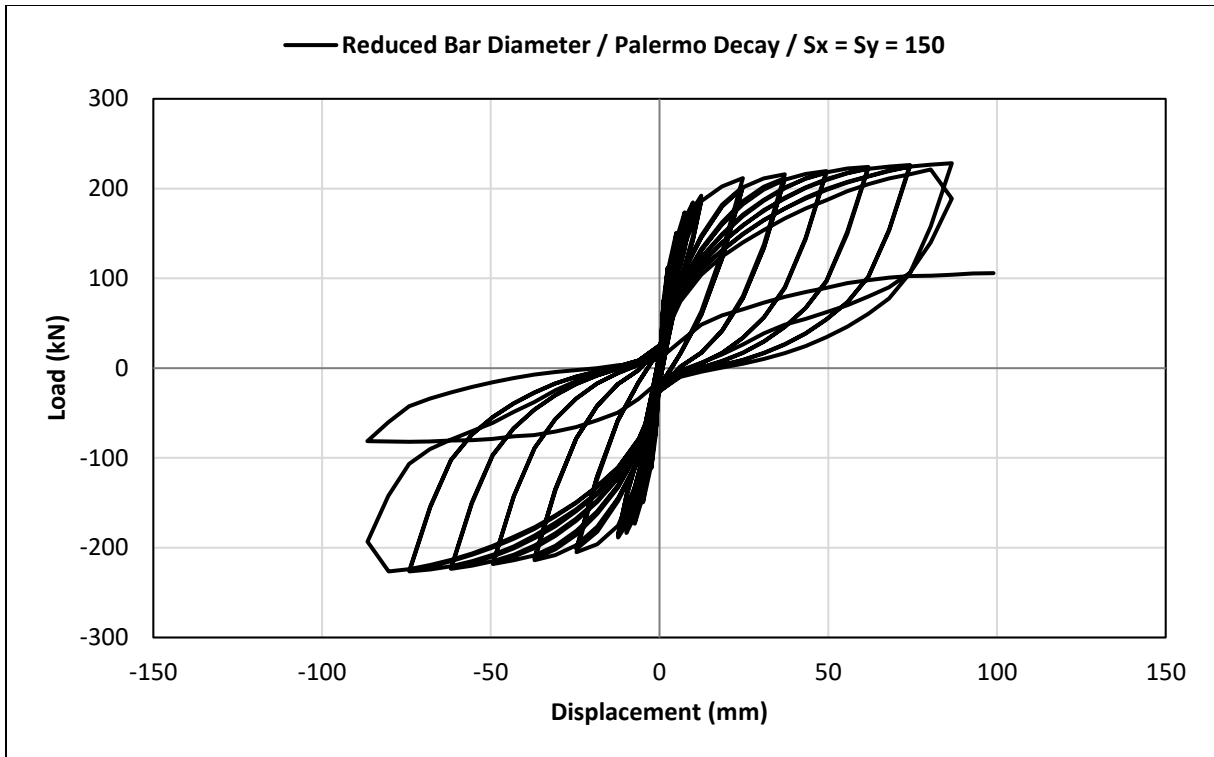


Figure 8.87: Model V2 with $S_x = S_y = 150\text{mm}$

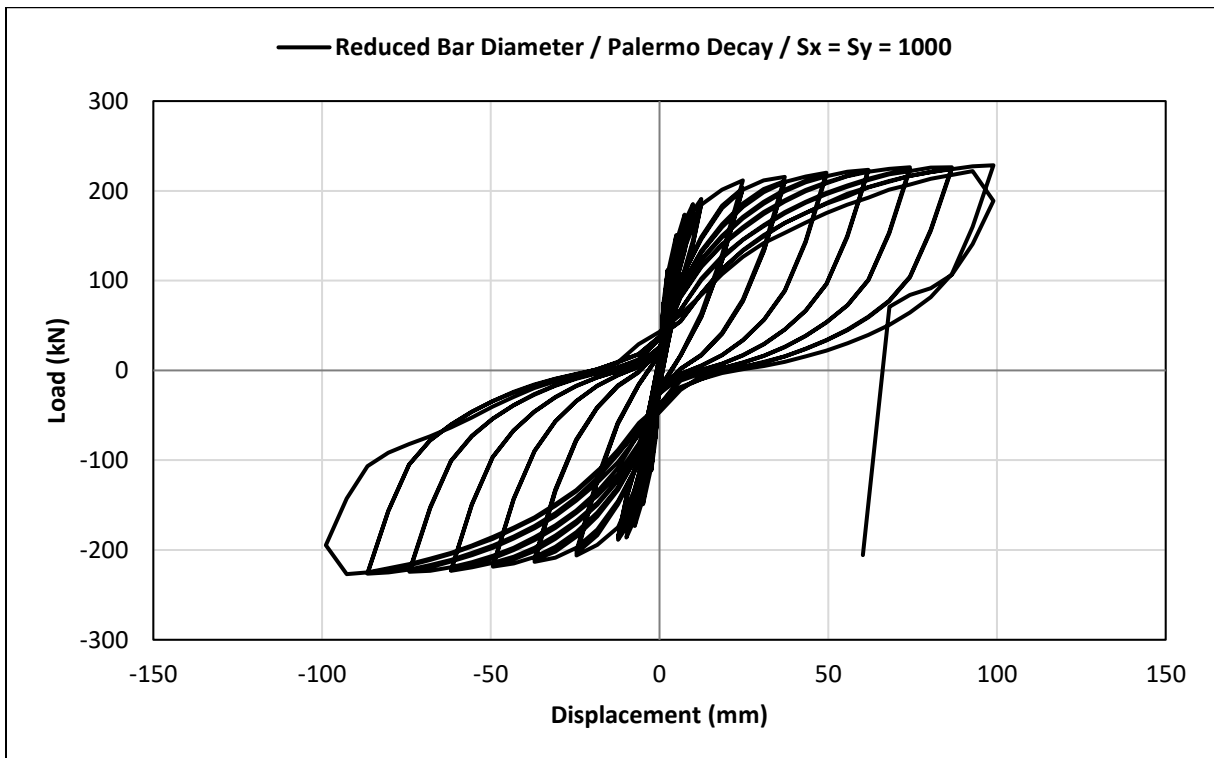


Figure 8.88: Model V2 with $S_x = S_y = 1000\text{mm}$

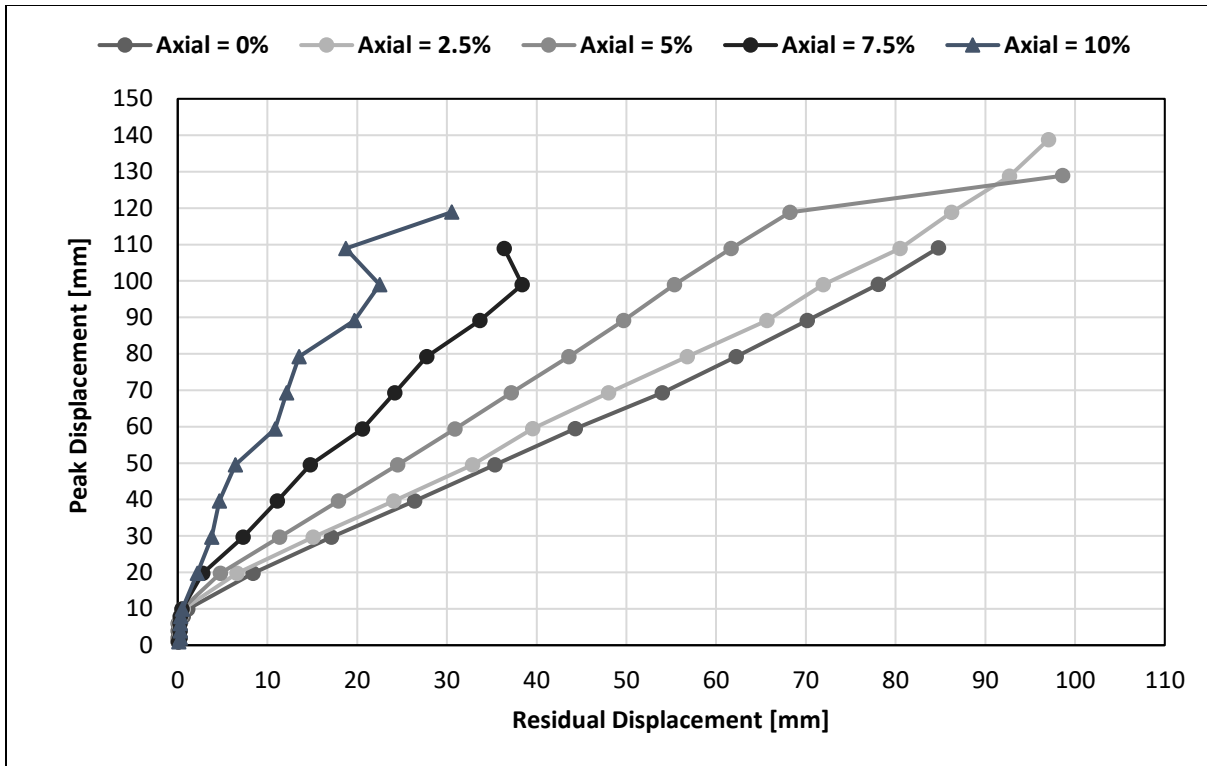


Figure 8.89: Peak vs Residual for all Load Percentages Based on Model V1

Appendix K – Cracking Timeline

The following section contains a start to finish sequence of each positive and negative cycle. The intent is to illustrate the deformation of the wall during testing with photos taken at 0 force, and at the maximum displacement of each cycle.

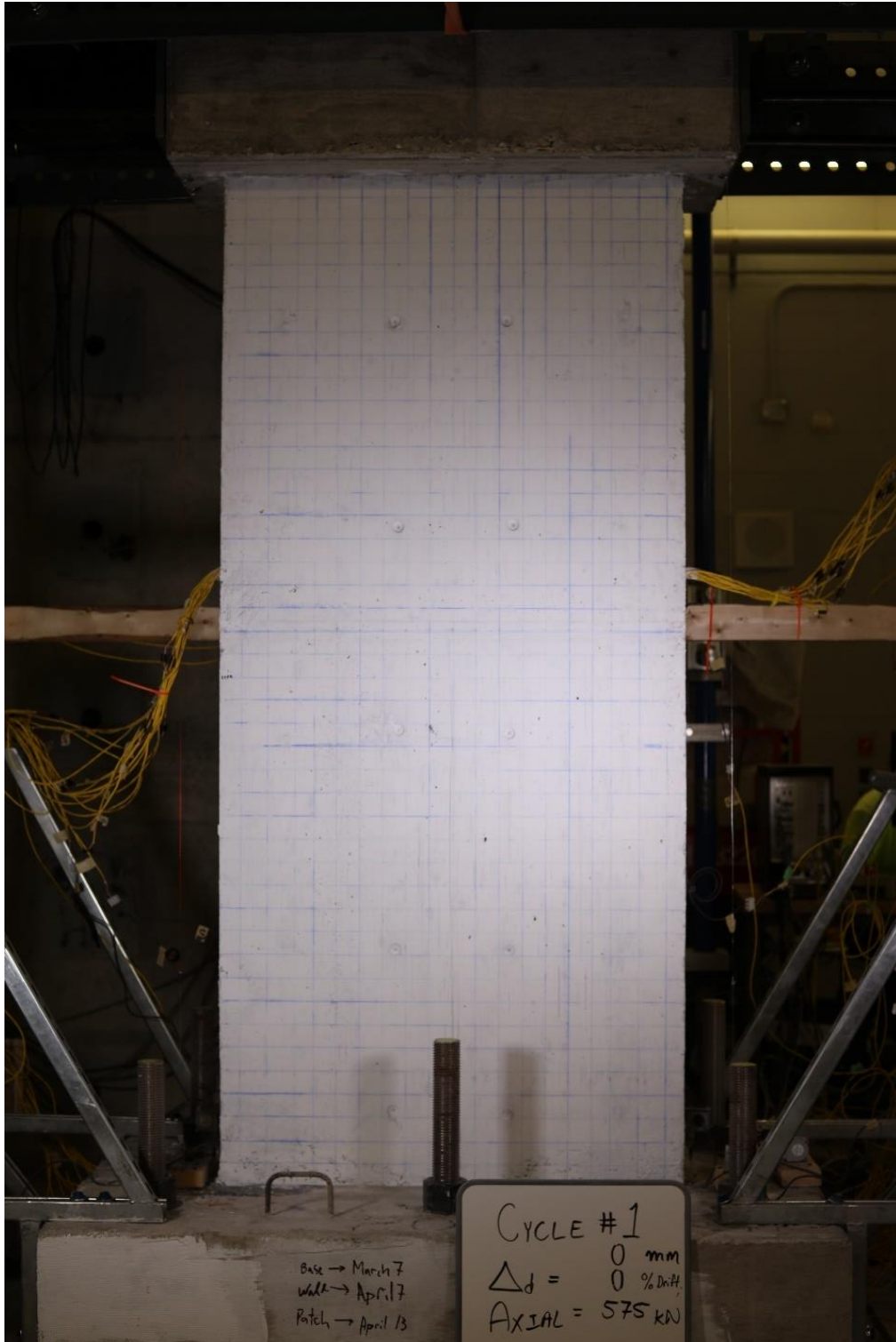


Figure 8.90: Wall With Axial Load Only

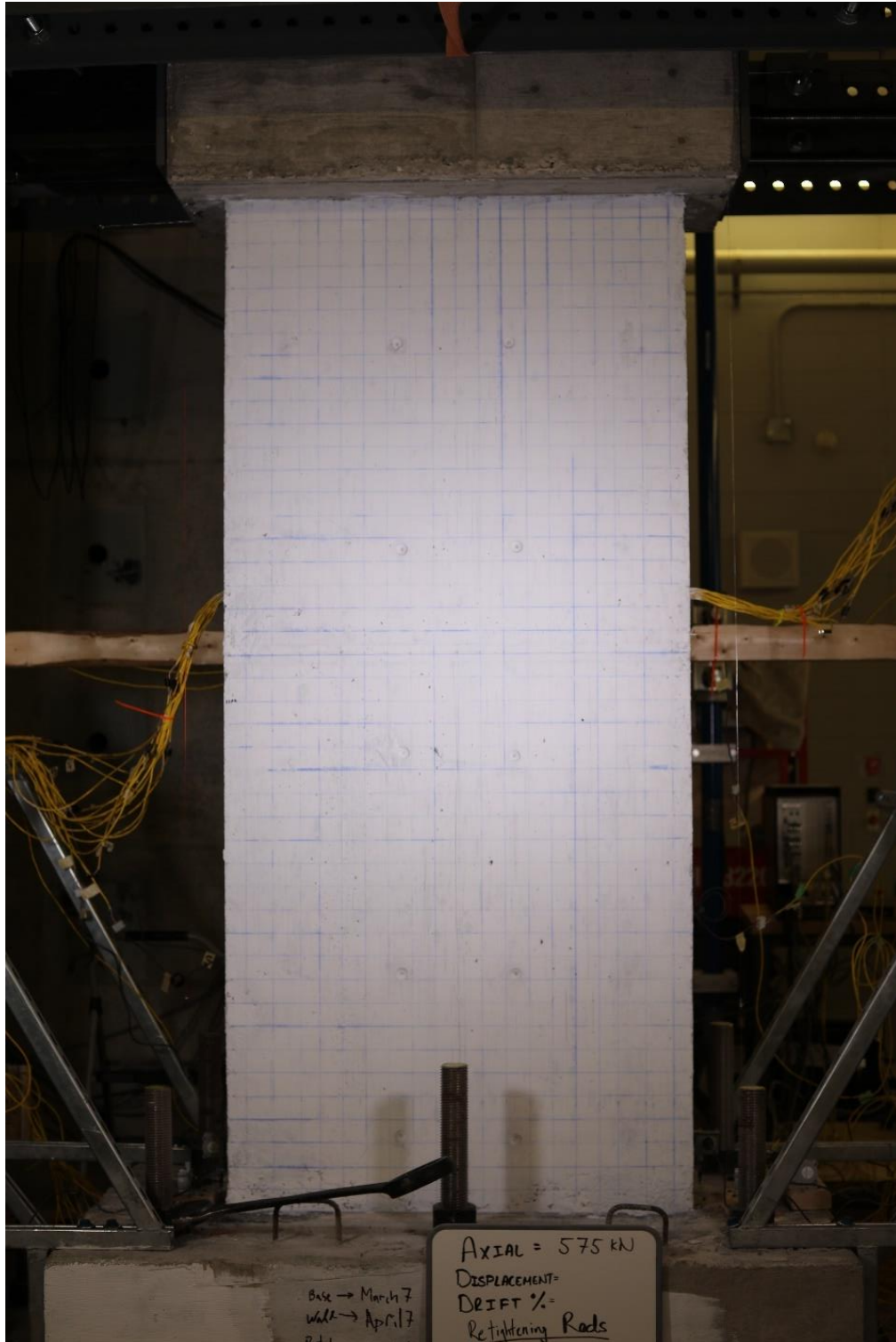


Figure 8.91: Retightening Rods after Applying Axial Loading

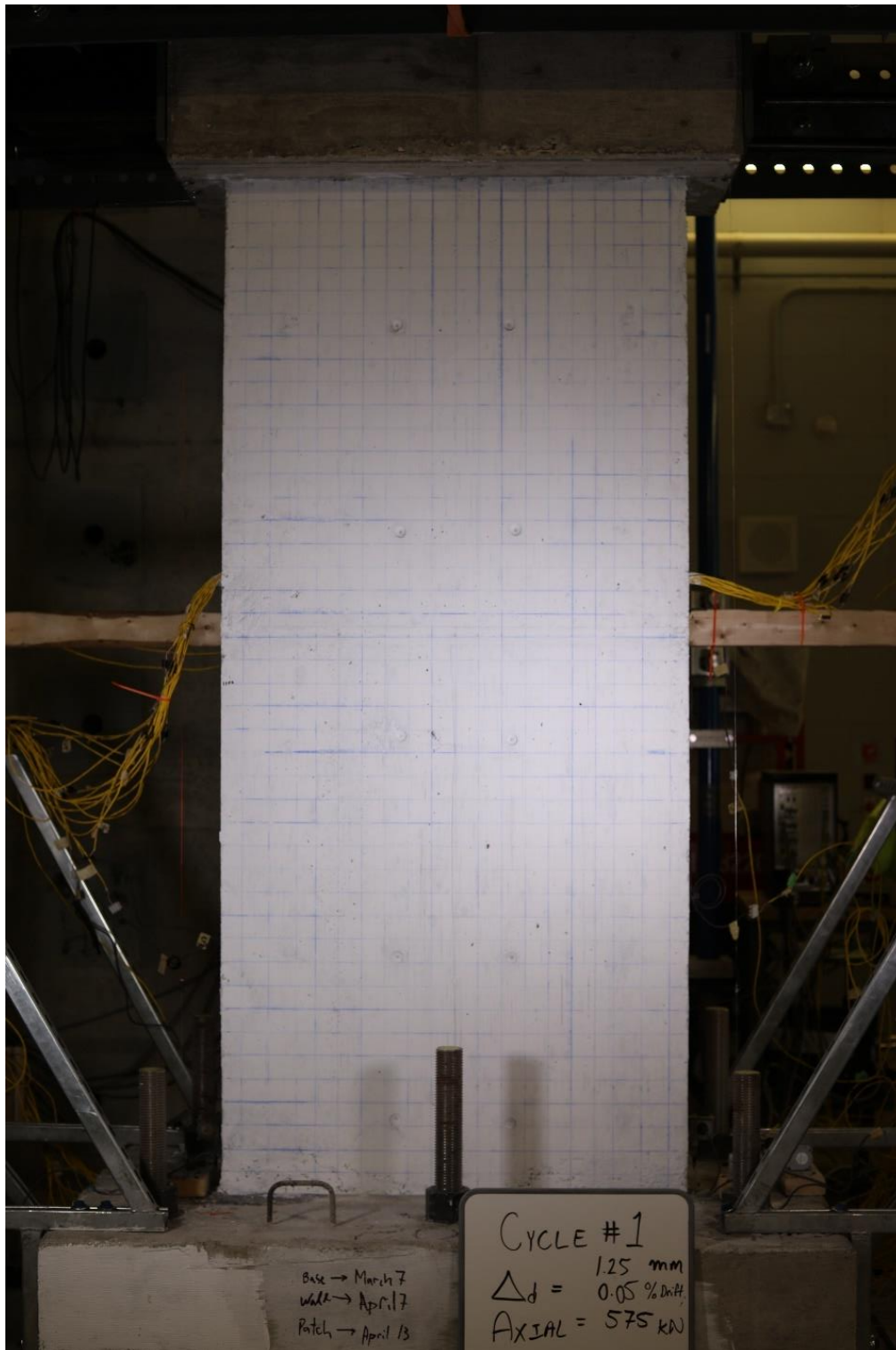


Figure 8.92: Positive 1.25mm Displacement, Cycle 3

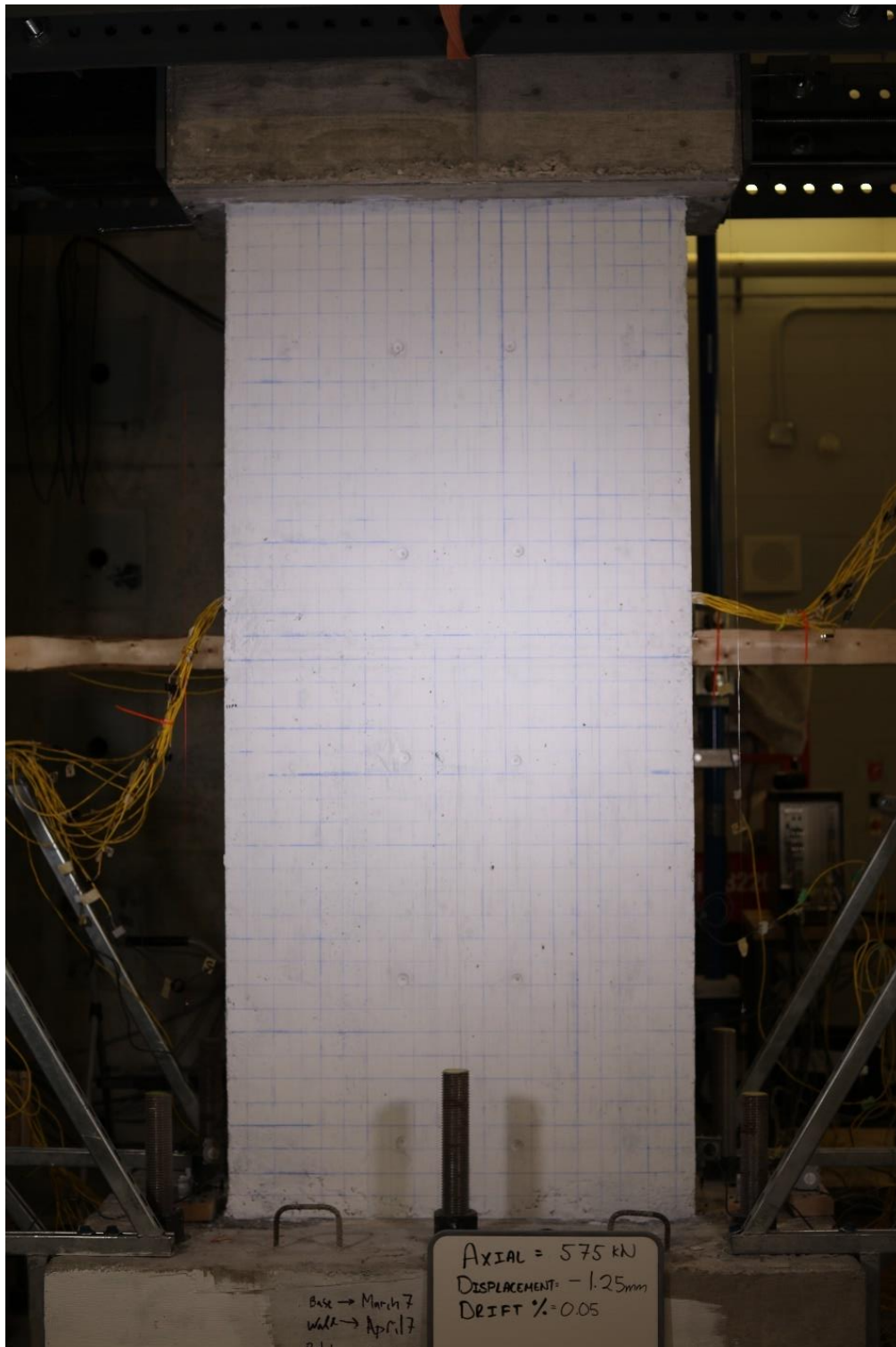


Figure 8.93: Negative 1.25mm Displacement, Cycle 3

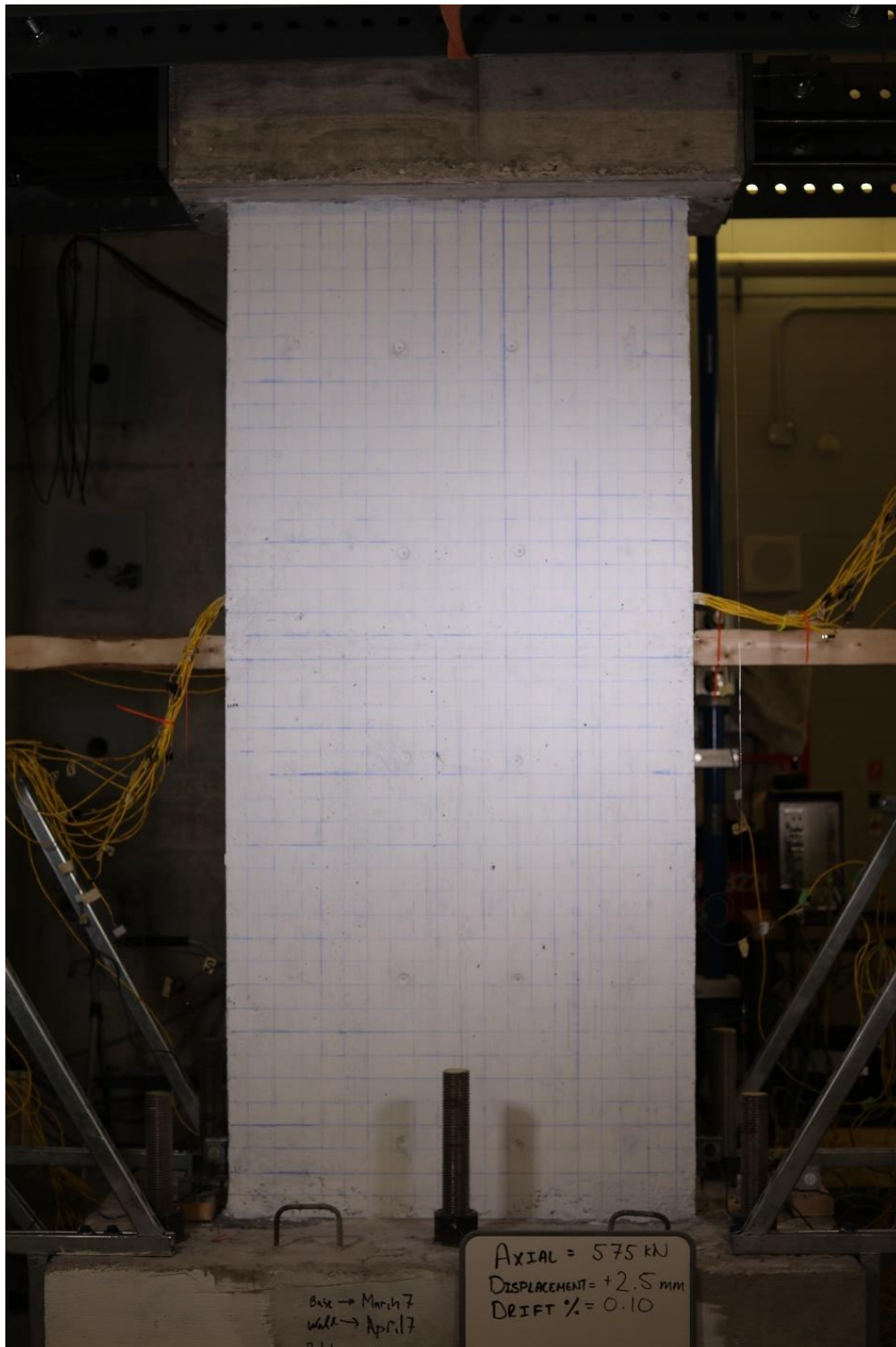


Figure 8.94: Positive 2.5mm Displacement, Cycle 3

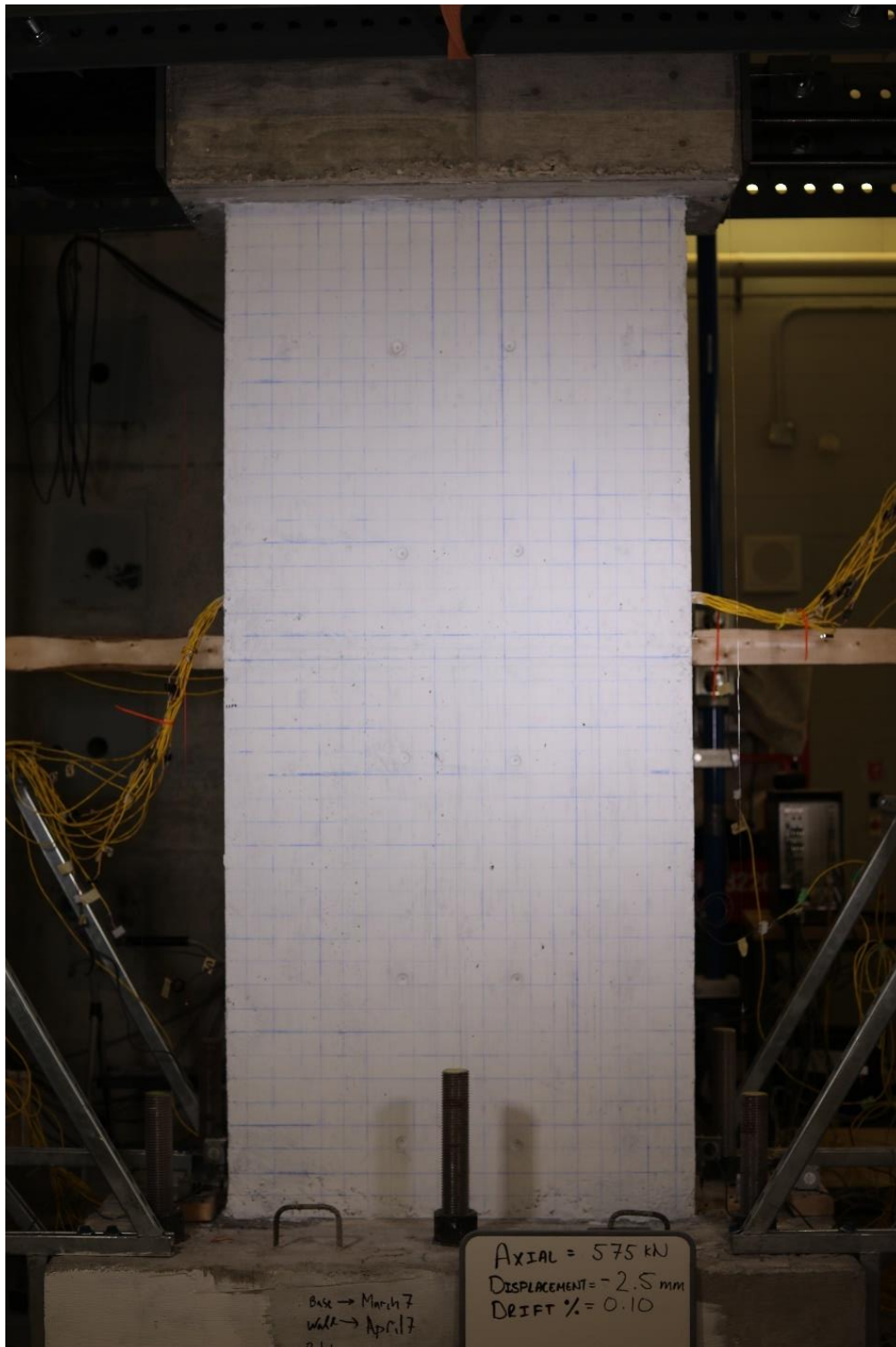


Figure 8.95: Negative 2.5mm Displacement, Cycle 3

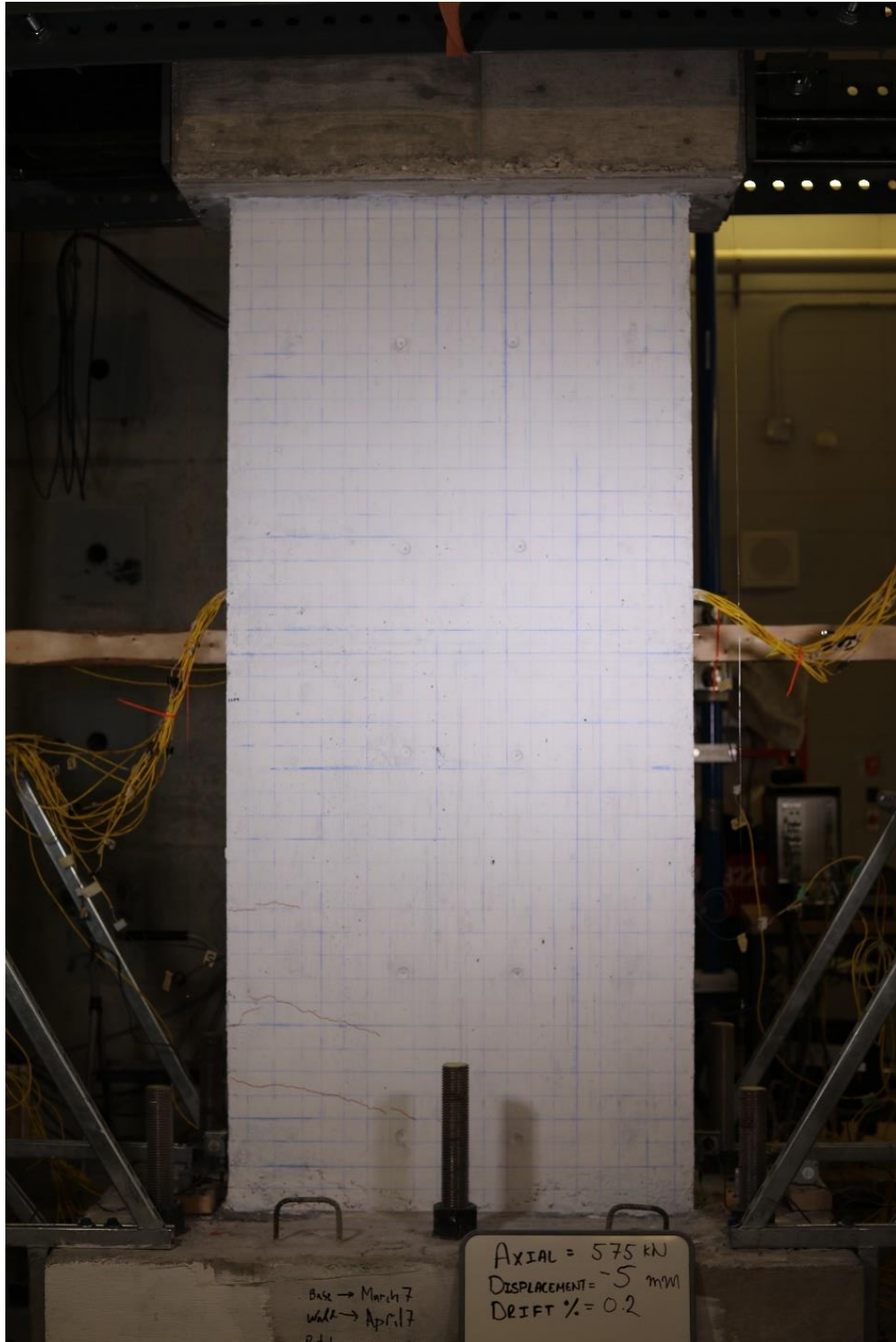


Figure 8.96: First Positive Cycle of 5mm, where initial Cracks Happened, Cycle 1

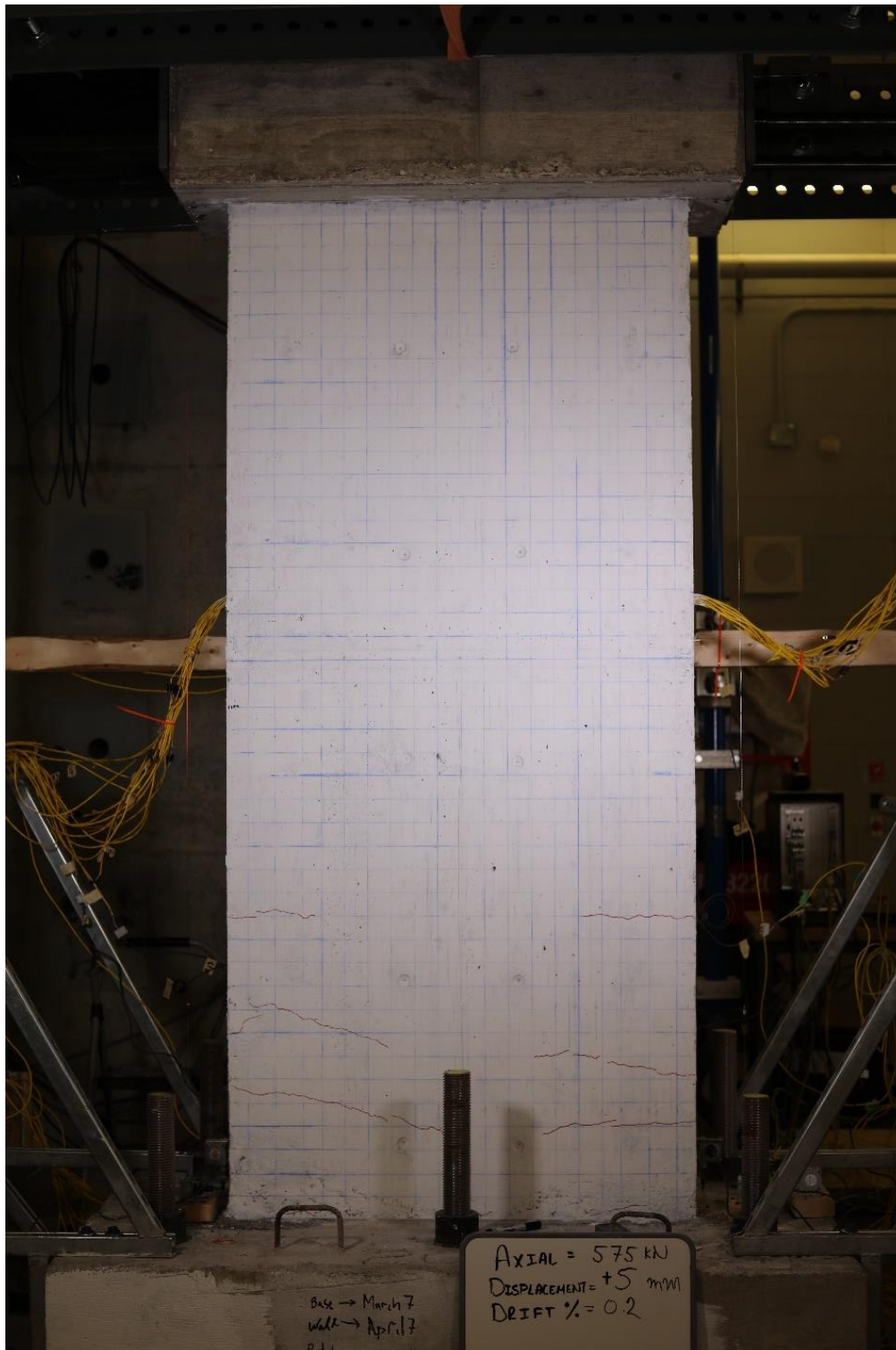


Figure 8.97: Positive 5mm Displacement, Cycle 3

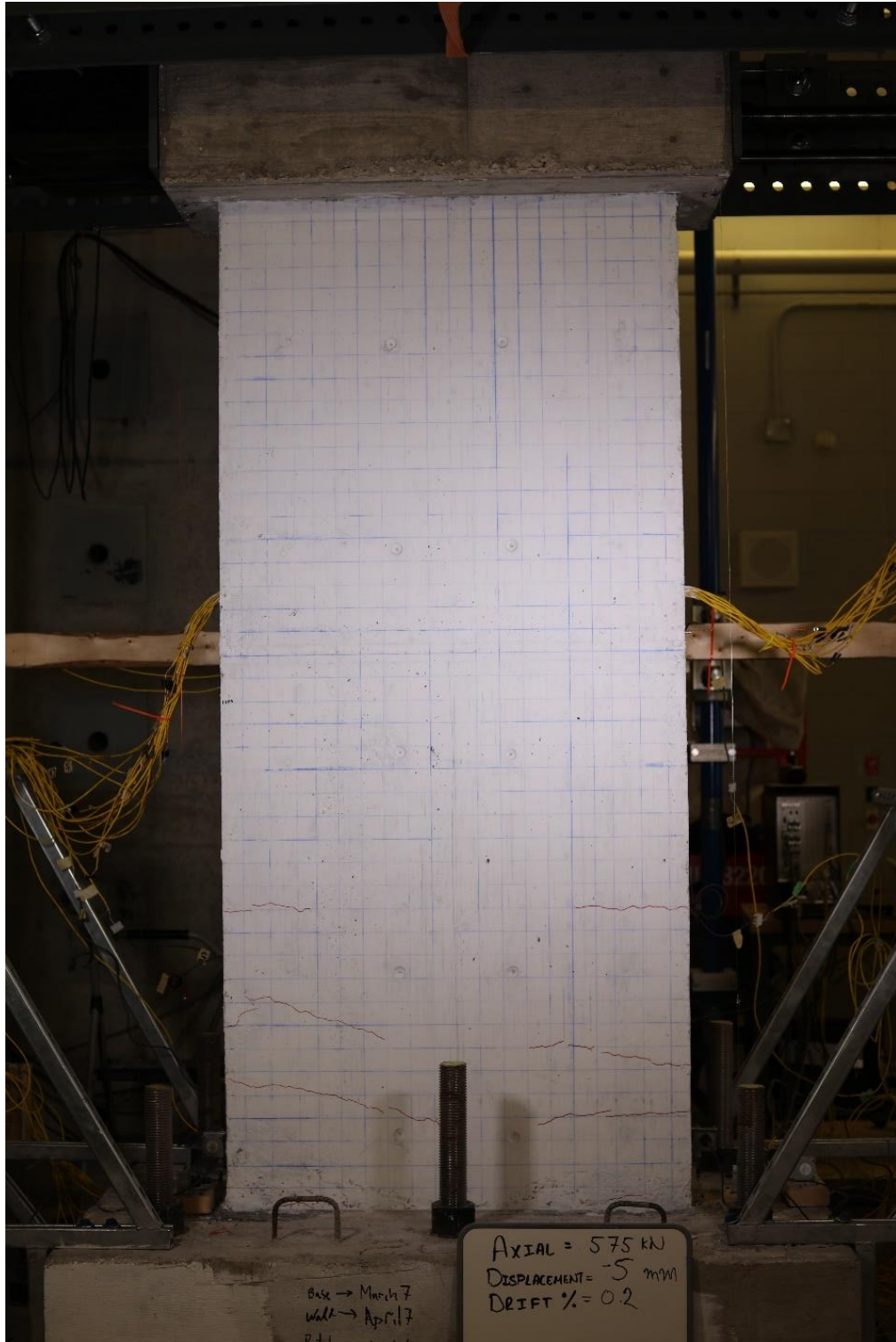


Figure 8.98: Negative 5mm Displacement, Cycle 3

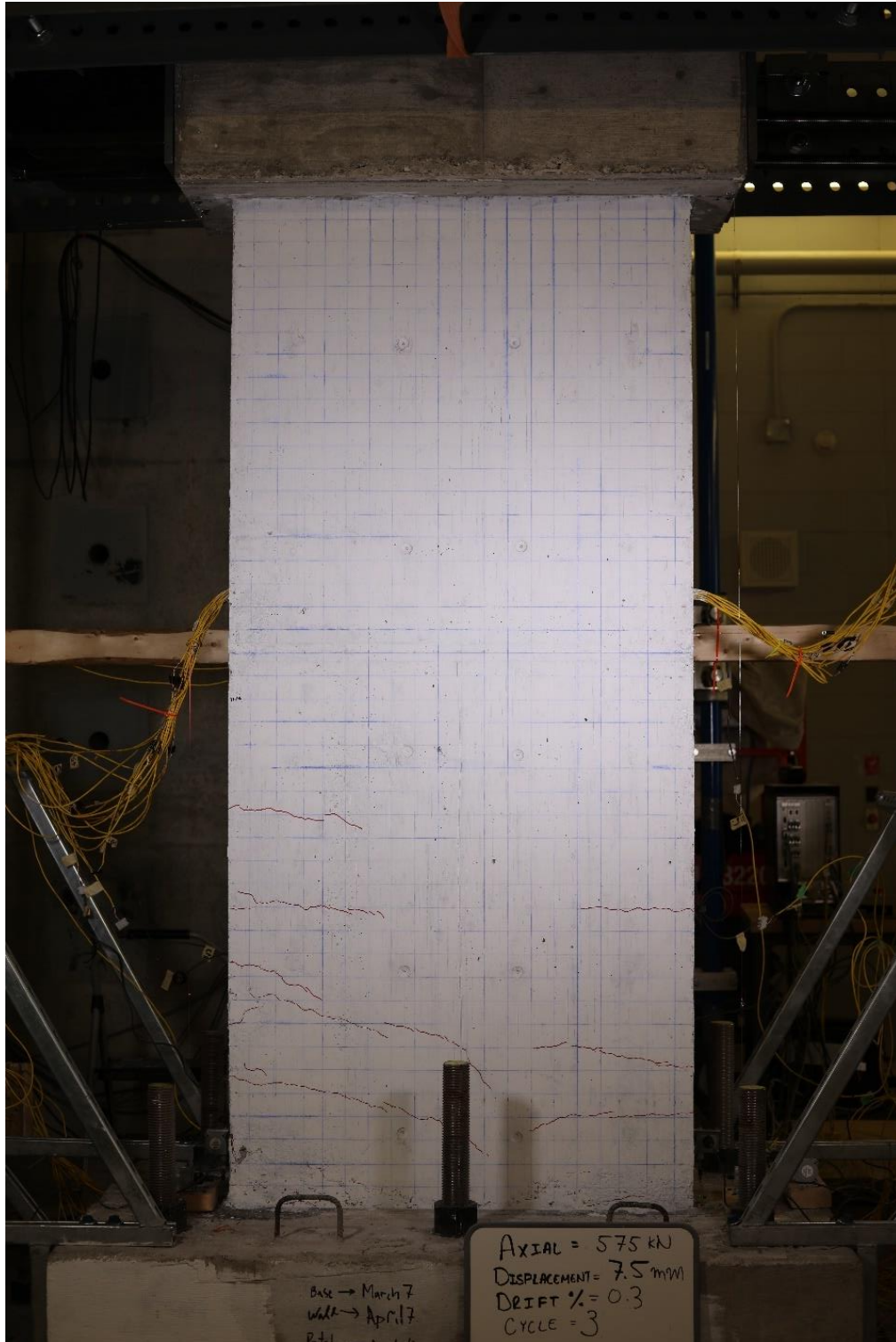


Figure 8.99: Positive 7.5mm Displacement, Cycle 3

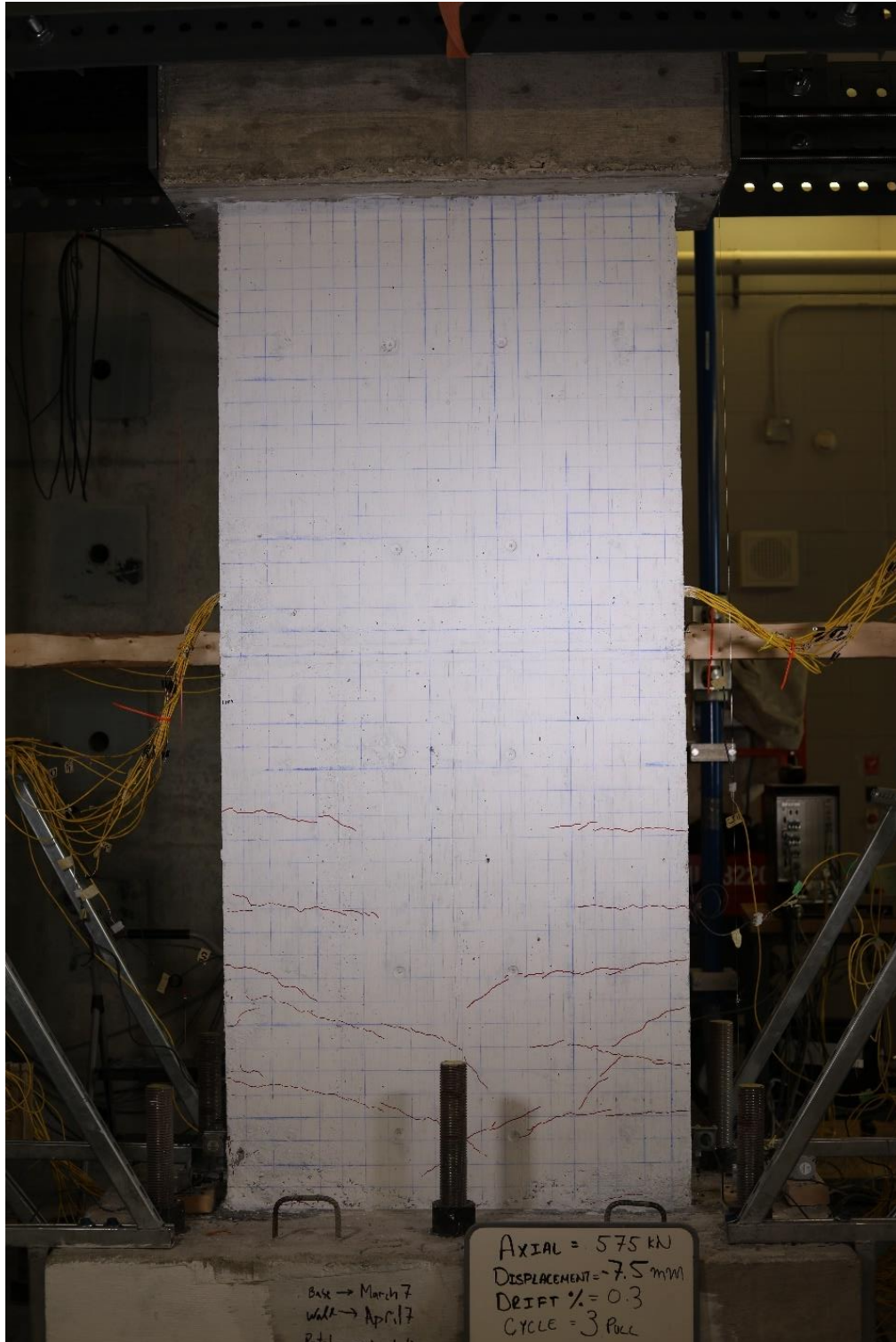


Figure 8.100: Negative 7.5mm Displacement, Cycle 3

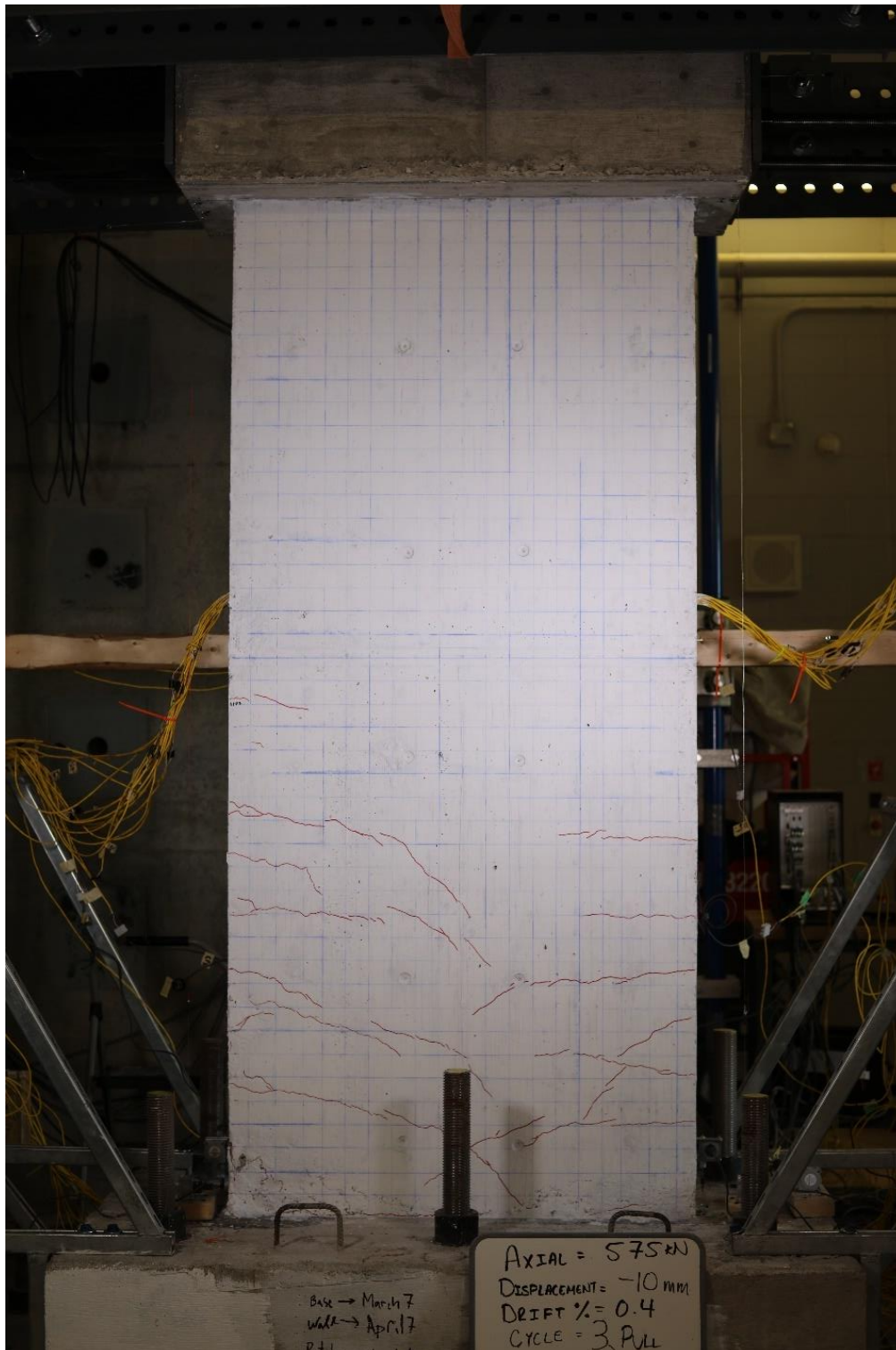


Figure 8.101: Positive 10mm Displacement, Cycle 3

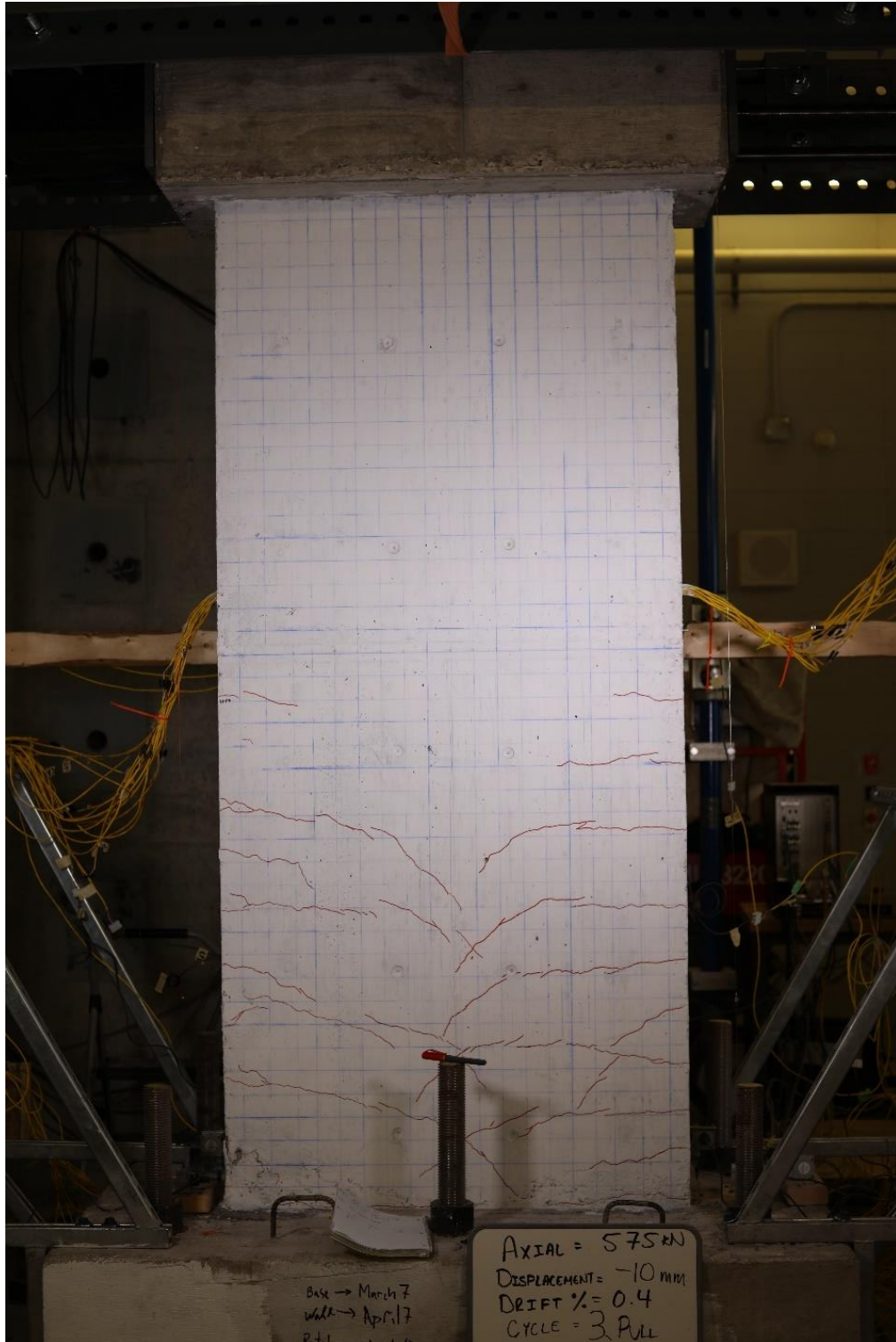


Figure 8.102: Negative 10mm Displacement, Cycle 3

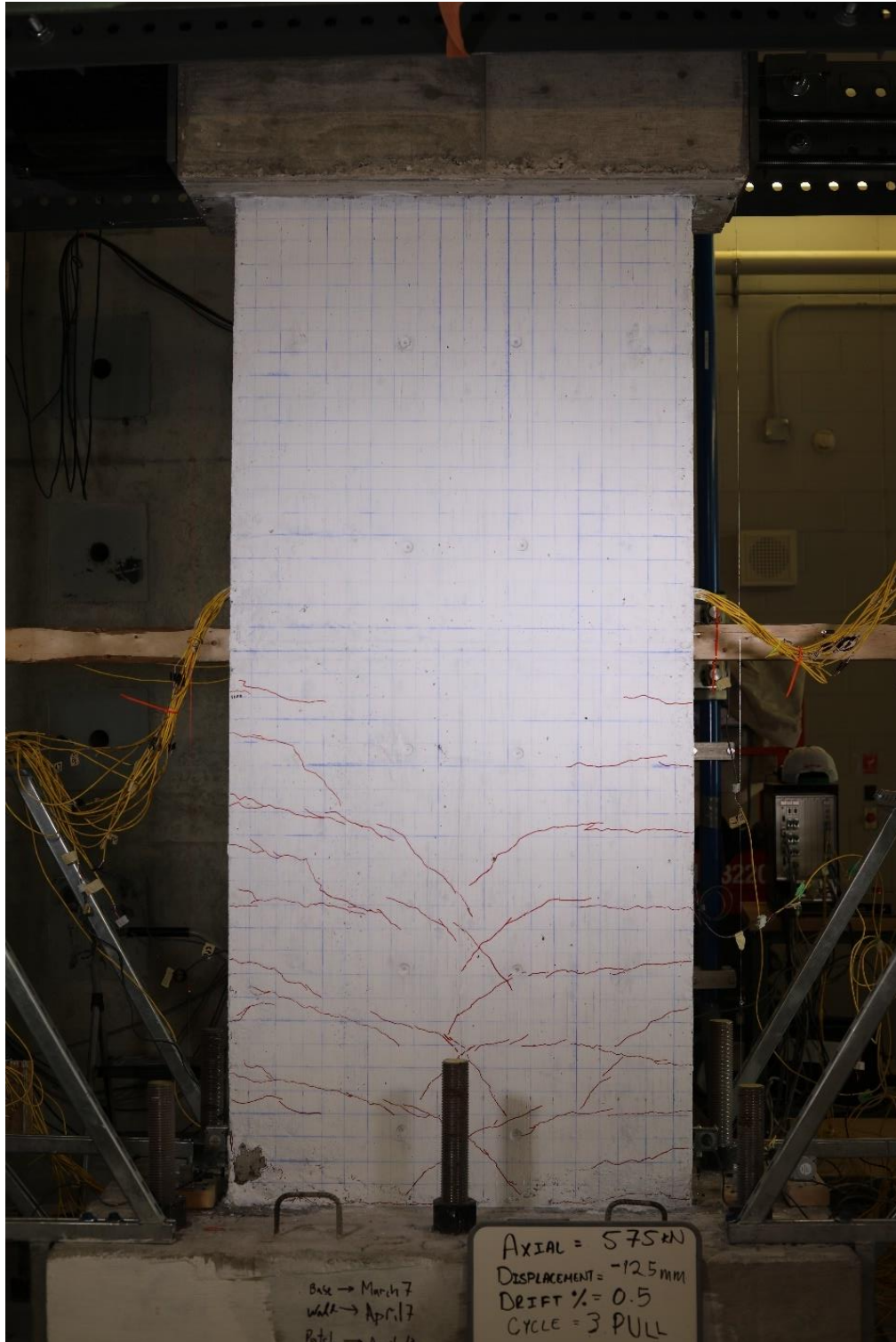


Figure 8.103: Positive 12.5mm Displacement, Cycle 3

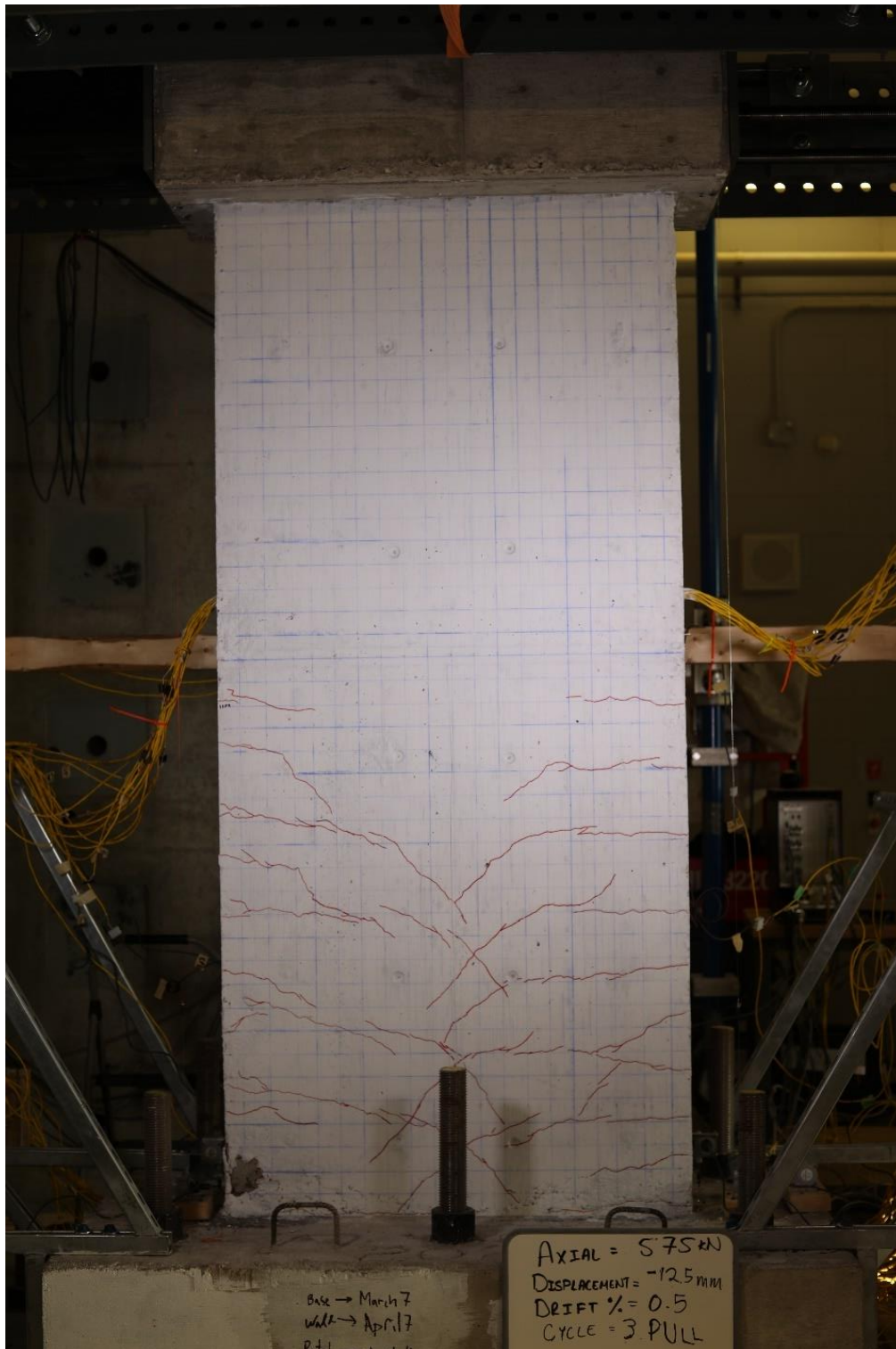


Figure 8.104: Negative 12.5mm Displacement, Cycle 3

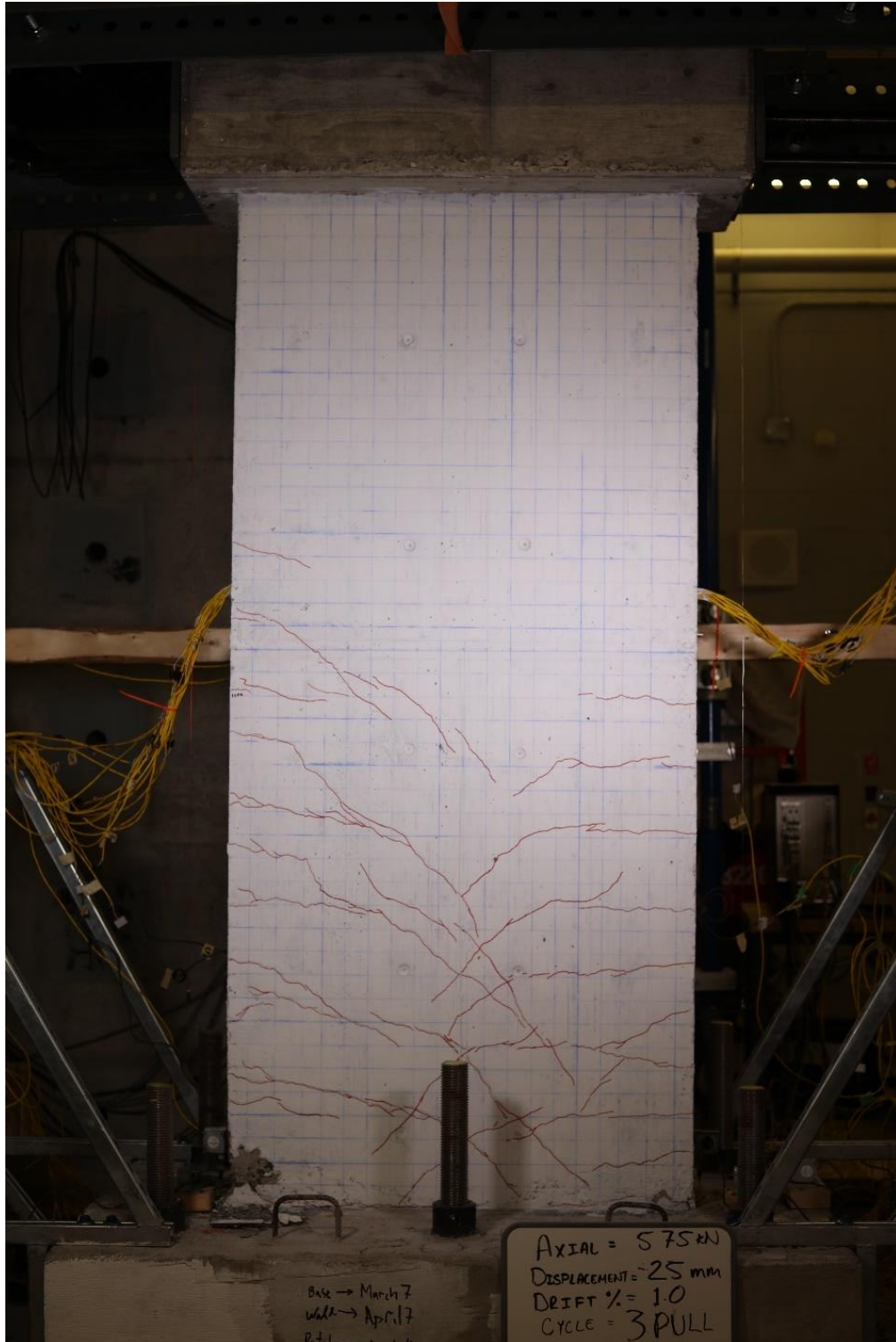


Figure 8.105: Positive 25mm Displacement, Cycle 3

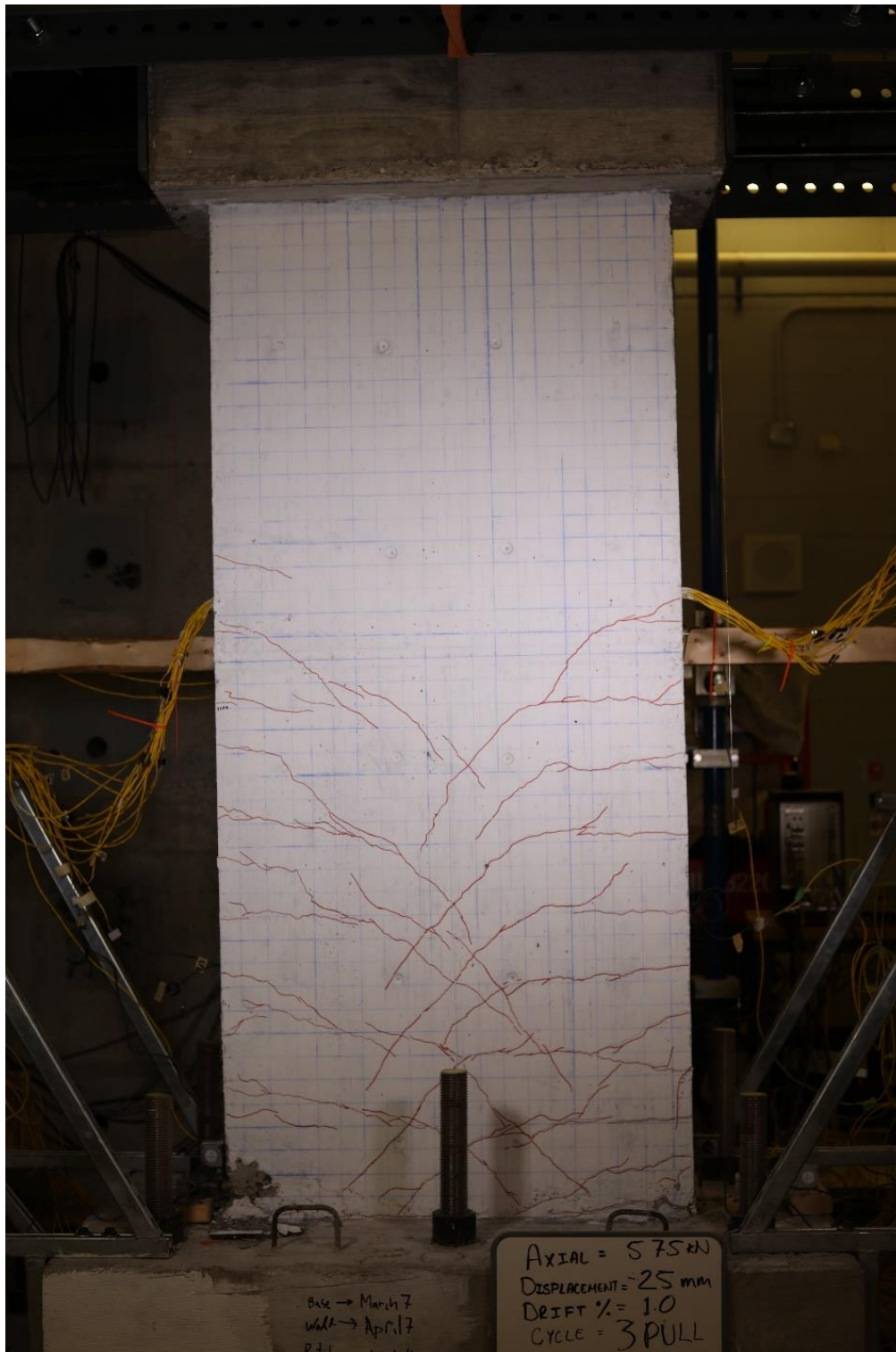


Figure 8.106: Negative 25mm Displacement, Cycle 3

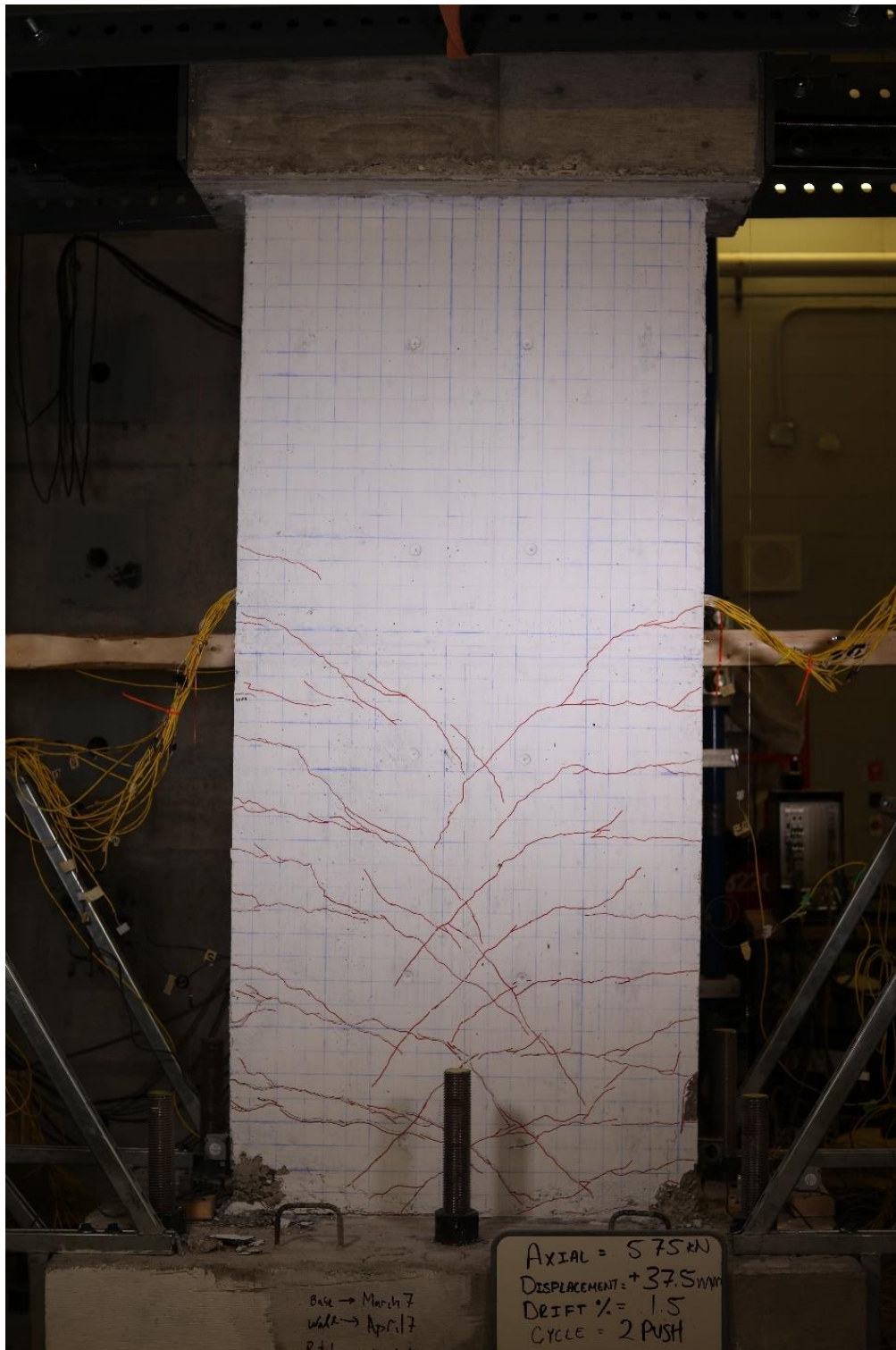


Figure 8.107: Positive 37.5mm Displacement, Cycle 2

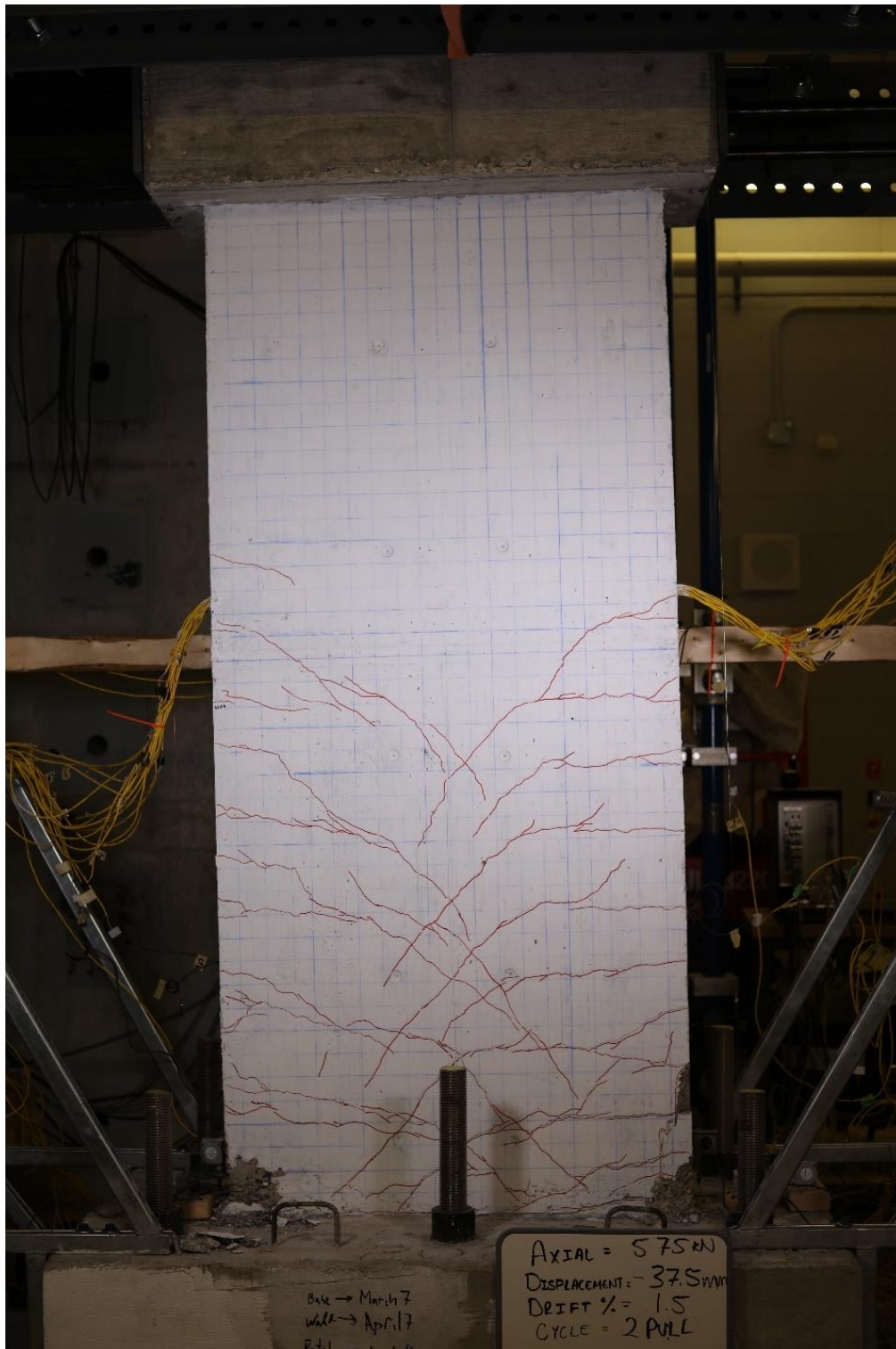


Figure 8.108: Negative 37.5mm Displacement, Cycle 2

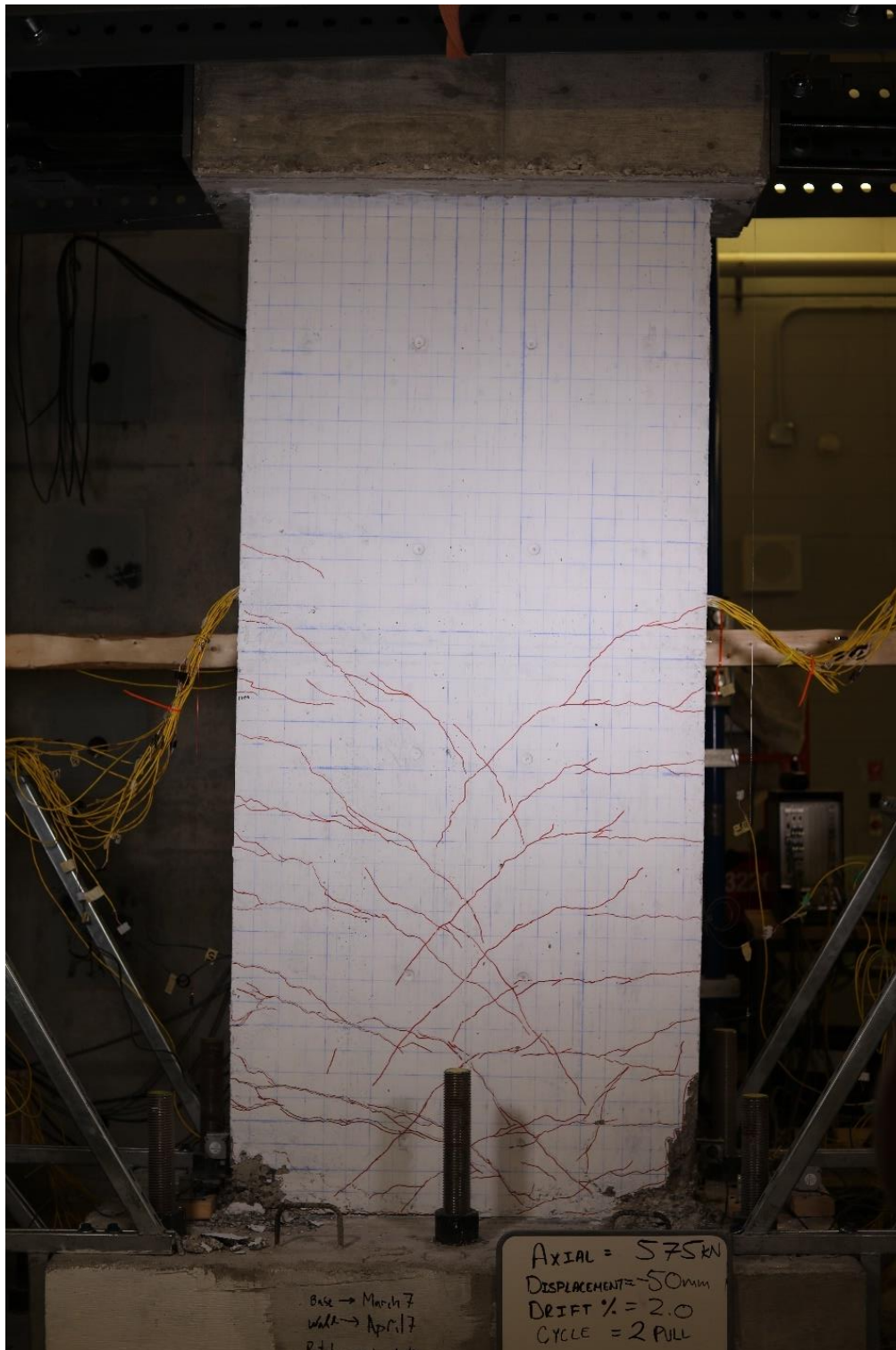


Figure 8.109: Positive 50mm Displacement, Cycle 2

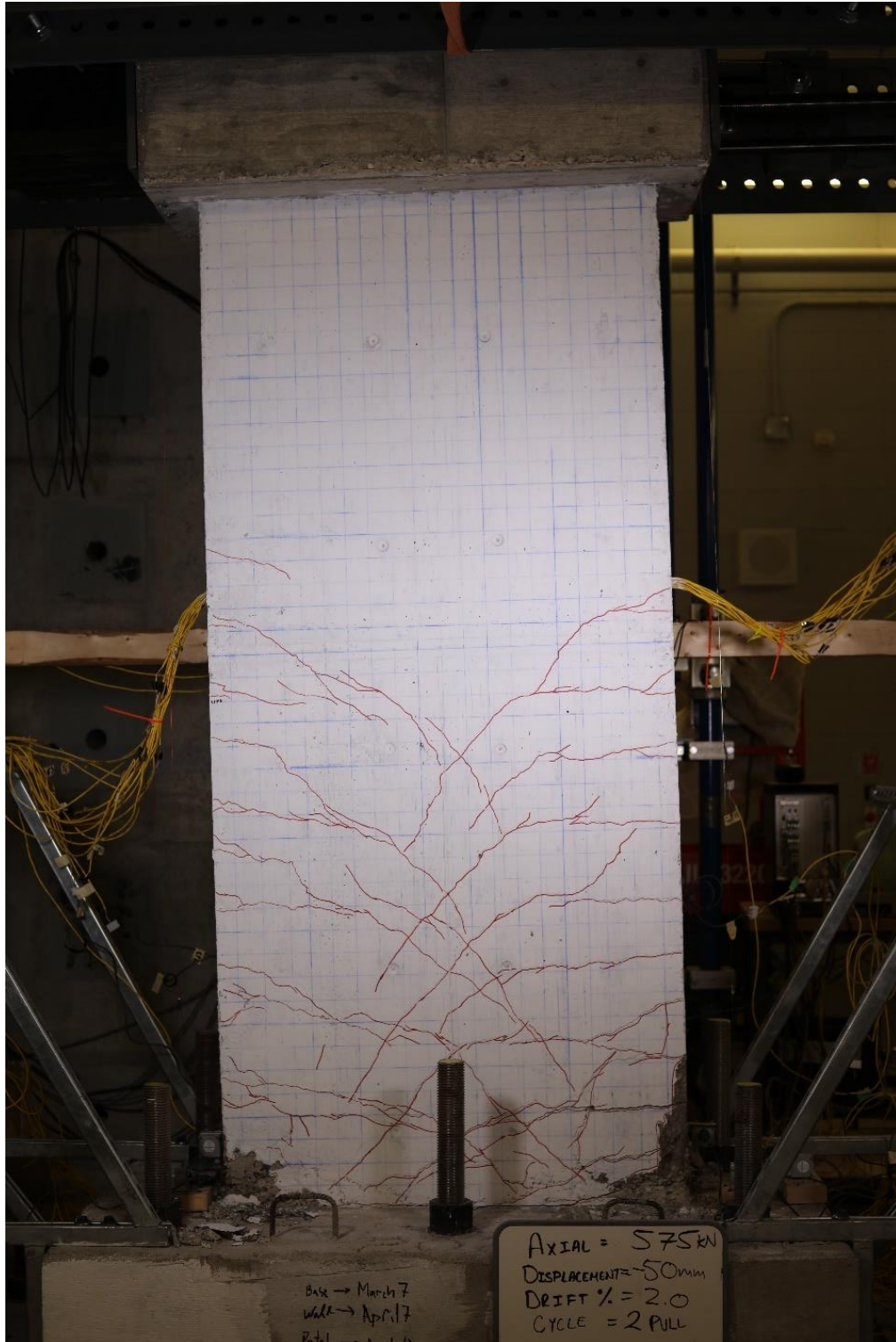


Figure 8.110: Negative 50mm Displacement, Cycle 2

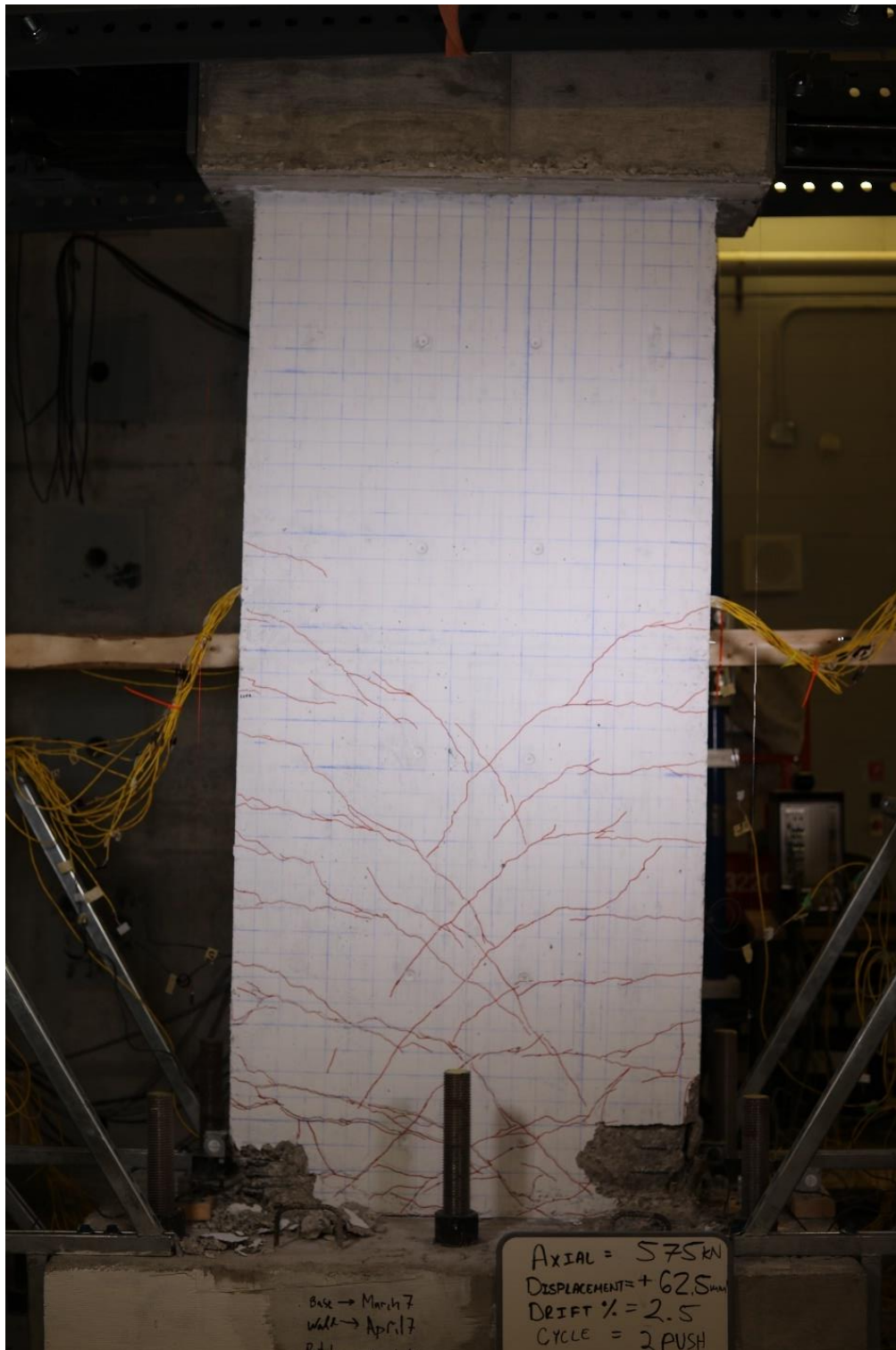


Figure 8.111: Positive 62.5mm Displacement, Cycle 2

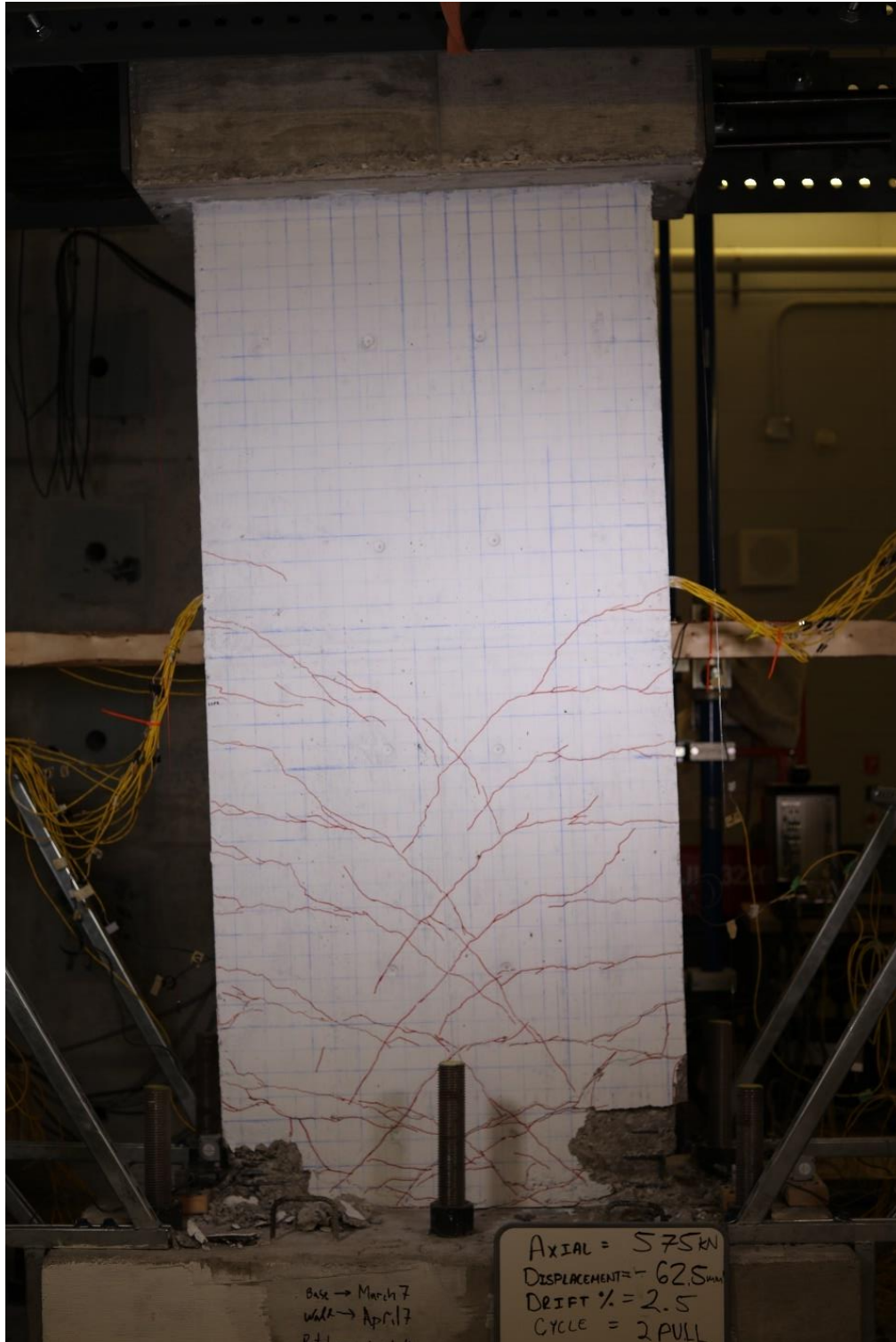


Figure 8.112: Negative 62.5mm Displacement, Cycle 2

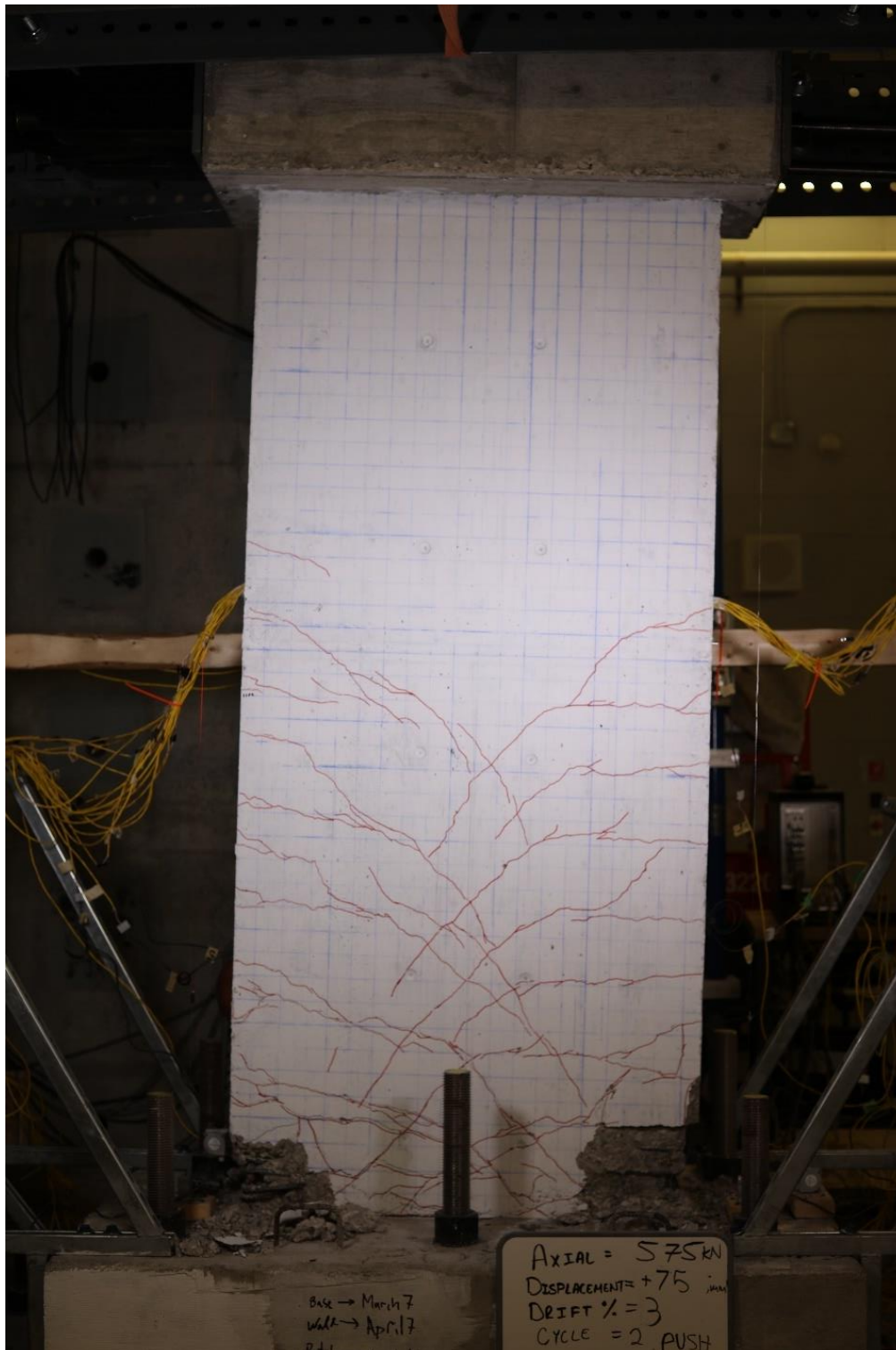


Figure 8.113: Positive 75mm Displacement, Cycle 2

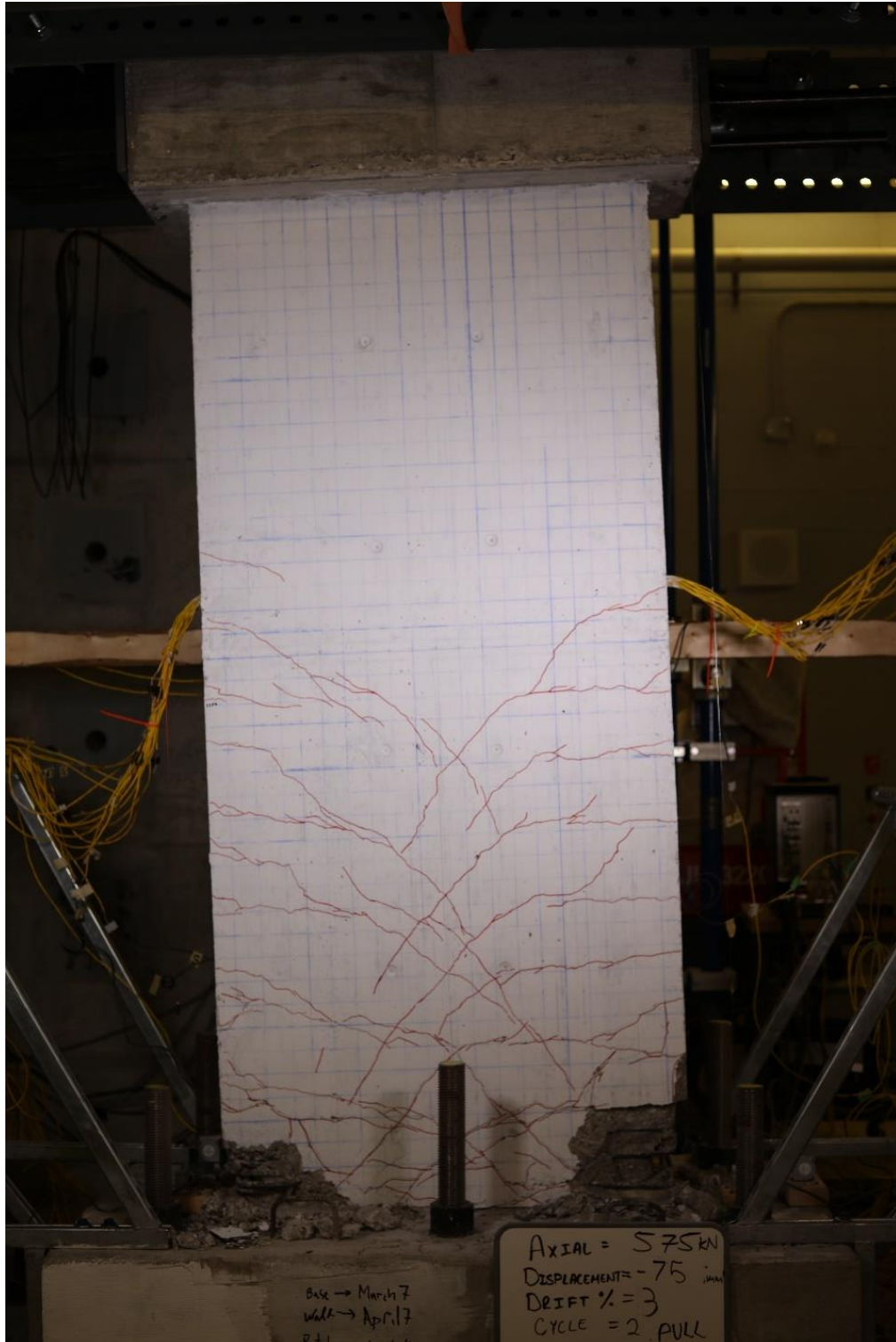


Figure 8.114: Negative 75mm Displacement, Cycle 2

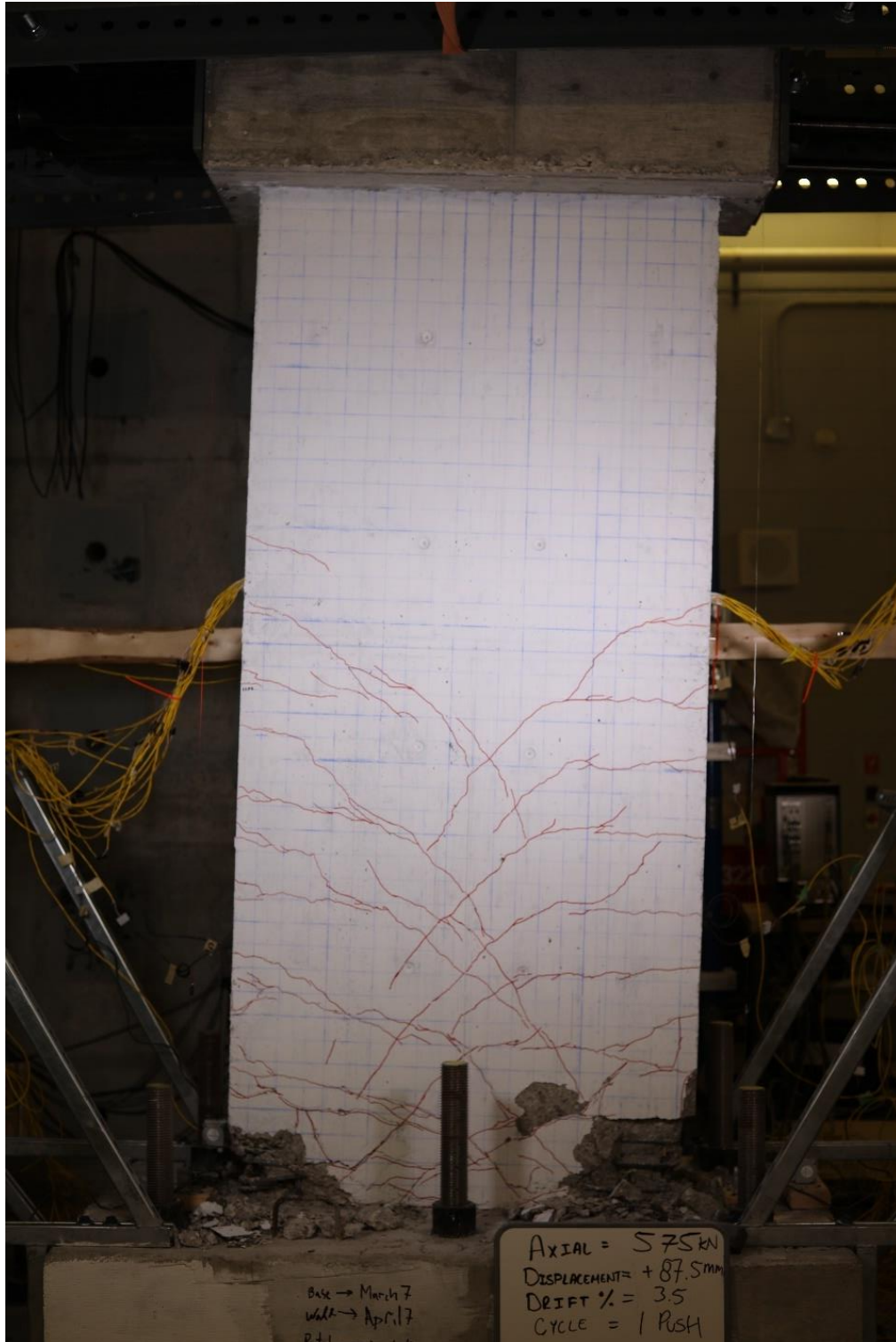


Figure 8.115: Positive 87.5mm Displacement, Cycle 1

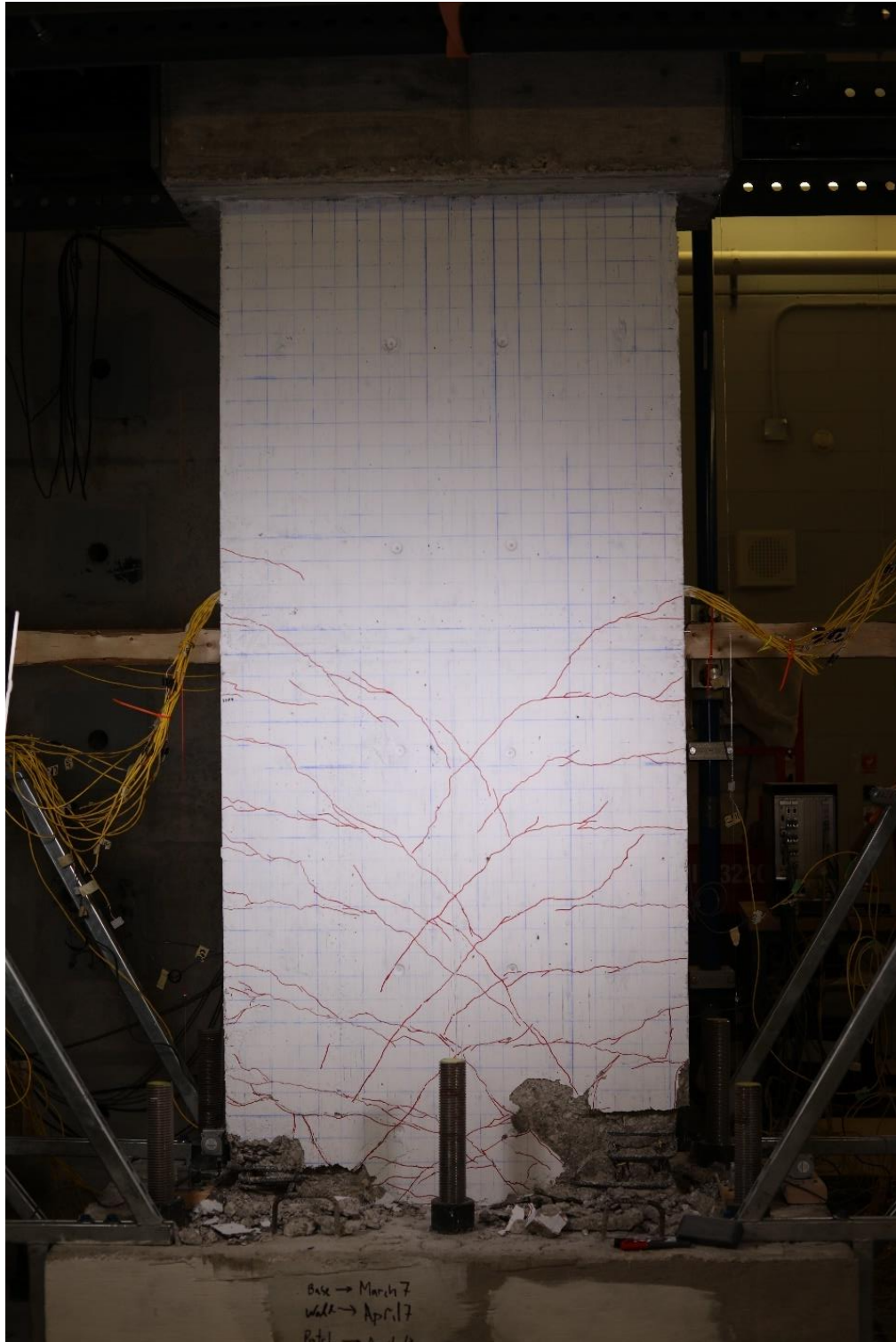


Figure 8.116: Unloaded Wall after Completed Test

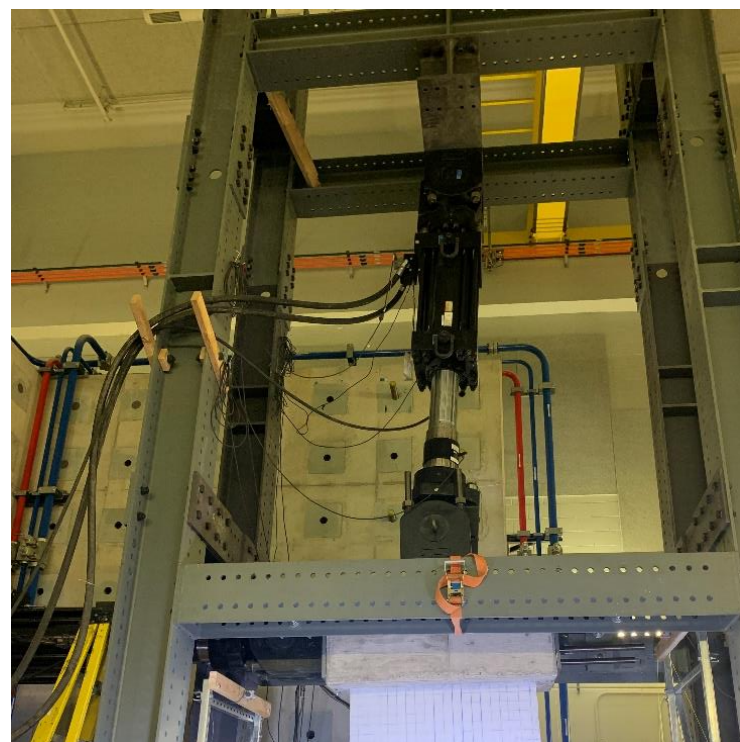
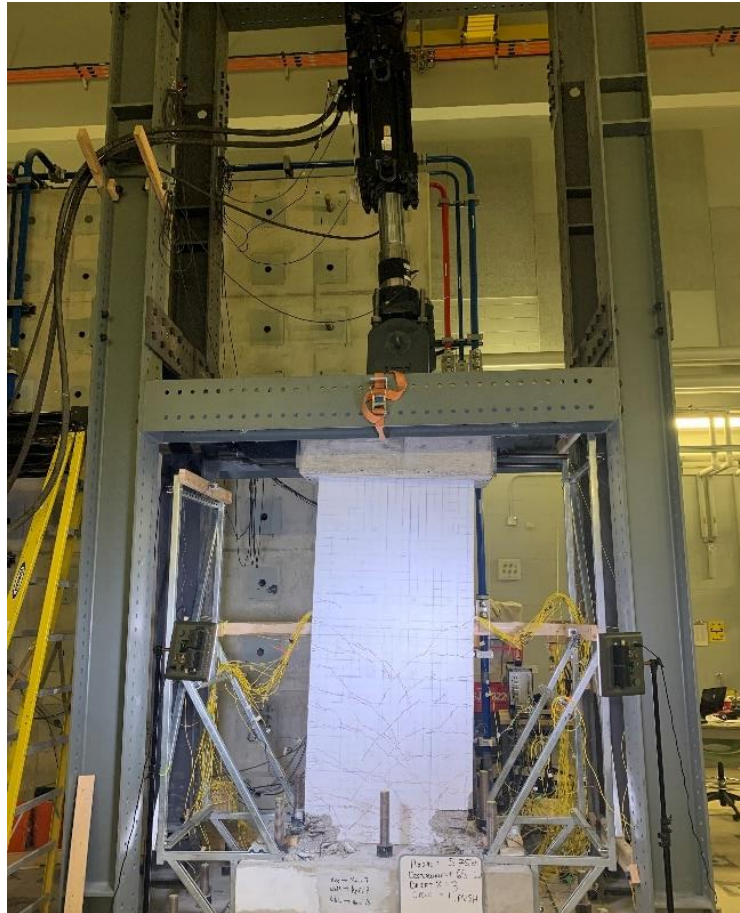


Figure 8.117: Visual Movement of Both Actuators

Appendix L – Crack Width Measurements

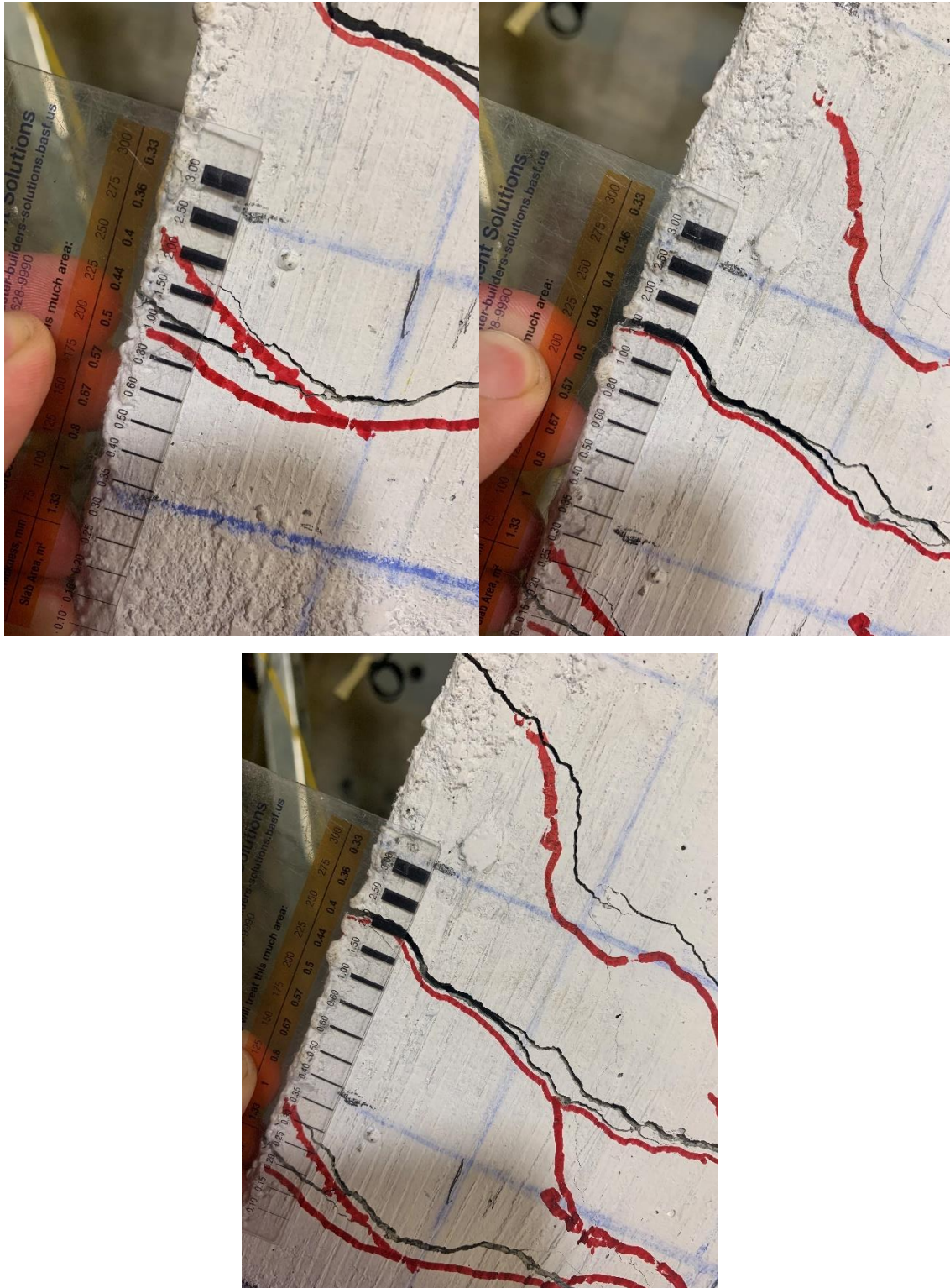


Figure 8.118: Examples of Crack Width Measurements, Actuator Side

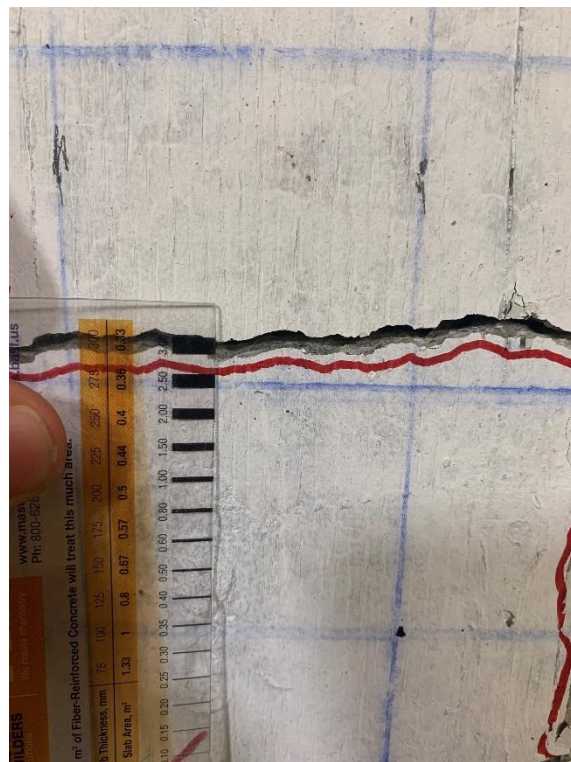
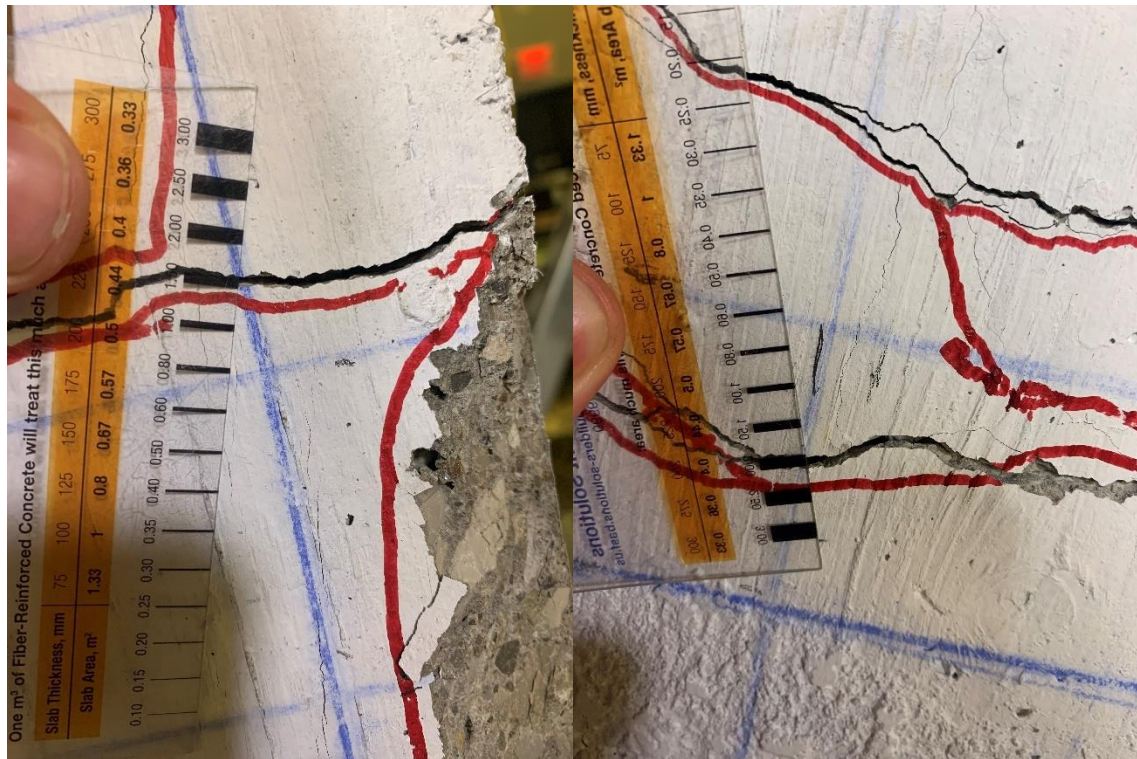


Figure 8.119: Examples of Width Measurements, Non-Actuator Side

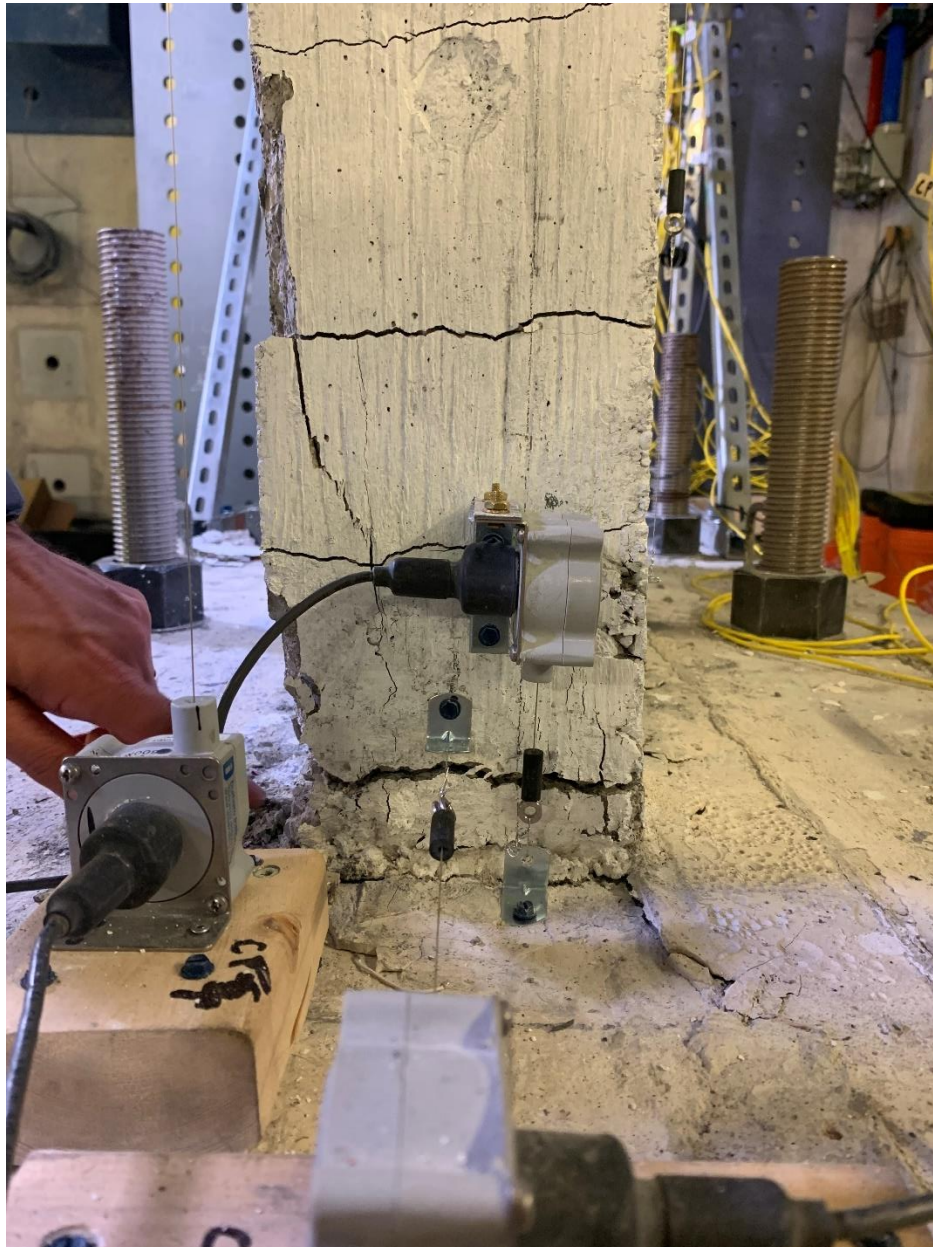


Figure 8.120: Damage to Toe of Wall, Non-Actuator Side



Figure 8.121: Rupture of Reinforcement in Boundary Region



Figure 8.122: Spalling and Crushing of Concrete at Toe, Non-Actuator Side

Table 8.1: Cracking Sequence for Push Cycles

Crack Occurrence	Displacement																						
Push Cycles	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length			
Location Above Base	1.25 mm (0.05% drift)		2.5 mm (0.10% drift)		5 mm (0.20% drift)		7.5 mm (0.30% drift)		10 mm (0.40% drift)		12.5 mm (0.50% drift)		25 mm (1.0% drift)		37.5 mm (1.5% drift)		50 mm (2.0% drift)		62.5 mm (2.5% drift)				
	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]			
0					0.1	100	0.1	100	0.3	300	0.3	400	3	500	3	500	25	500	large spalled piece	500			
100					SCC spalling		100	SCC spalling	100	SCC spalling	100	large chunk spalled off	100	large chunk spalled off (100x100)	150	large chunk spalled off (100x100)	150	large chunk spalled off (150x150)	150				
250											0.15	200	0.35	250	1	550	1.5	550	2	550			
300					0.2	500	0.25	550	0.25	650	0.3	650	1	650	1.5	650	2	650	1.5	650			
400					0.25	350	0.25	550	0.35	600	0.35	600	0.5	800	0.8	800	0.8	800	0.8	800			
550					0.1		200	0.15	200	0.15	200	0.25	300 (merged with 400 crack)	0.4	300 (merged with 400 crack)	1.5	300 (merged with 400 crack)	1	300 (merged with 400 crack)				
650							0.1	200	0.1	350	0.15	550	0.15	600	0.2	600 (merged with 775 crack)	0.25	600 (merged with 775 crack)	0.6	600 (merged with 775 crack)	Not Measured	600 (merged with 775 crack)	
775									0.15	200	0.15	350 (merged with 600 crack)	0.15	800	0.15	800	0.25	800	Not Measured	800			
850					0.1		300	0.15	550	0.15	550	0.1	800 (merged with 775 crack)	0.2	800 (merged with 775 crack)	0.15	800 (merged with 775 crack)	Not Measured	800 (merged with 775 crack)				
1000											0.15	250	0.15	300 (merged with 850 crack)	0.15	300 (merged with 850 crack)	0.15	300 (merged with 850 crack)	Not Measured	300 (merged with 850 crack)			
1100									0.1	150	0.1	200	0.15	200 (merged with 1250 crack)	0.15	200 (merged with 1250 crack)	0.15	200 (merged with 1250 crack)	Not Measured	200 (merged with 1250 crack)			
1250																0.15	550 (merged with 1100 crack)	0.15	600 (merged with 1100 crack)	0.2	600 (merged with 1100 crack)	Not Measured	600 (merged with 1100 crack)
1400																0.1	150	0.1	200	0.1	200	Not Measured	200
													Crushing of the toe beginning here (west side)		Concrete crushing on east side toe, spalling at east toe and a piece from 200 to 300 above the base spalled off on the east edge		At this point, the crack lengths didn't really change but crushing was apparent in the east toe.		More spalling at both toes. Crushing of the toe concrete. Left toe piece 200x250mm. Rebar buckling observed in areas where cover has spalled off and rebar ruptured on south side boundary.				

Table 8.2: Cracking Sequence for Pull Cycles

Crack Occurrence	Displacement																							
Pull Cycles	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length	Crack width	Crack Length				
Location Above Base	1.25 mm (0.05% drift)		2.5 mm (0.10% drift)		5 mm (0.20% drift)		7.5 mm (0.30% drift)		10 mm (0.40% drift)		12.5 mm (0.50% drift)		25 mm (1.0% drift)		37.5 mm (1.5% drift)		50 mm (2.0% drift)		62.5 mm (2.5% drift)					
	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]				
0					0.1	150	0.1	150	0.25	300	0.6	300	1.5	500	2.5	500	Spalling off, 6.5cm	Crack = 500, spalling = 200	Spalling off, 6.5cm	Crack = 500, spalling = 200				
150									0.1	200	0.1	200	0.6	400	1.5	400	Spalling off, 1.5	Crack = 400, spalling = 100	Spalling off, 1.5	Crack = 400, spalling = 250				
200					0.15	350	0.25	600	0.25	600	0.3	650	1.5	700	2.5	700	Spalling off, 3.0	Crack = 700, spalling = 50	Spalling off, 3.0	Crack = 700, spalling = 250				
300					0.15	350	0.2	350	0.25	600	0.25	700	0.35	750	0.8	750	1.5	750	Spalling off, 2.0	Crack = 750, spalling = 200				
450									0.1	250 (merged with 300 and 200 cracks)	0.2	250 (merged with 300 and 200 cracks)	0.2	250 (merged with 300 and 200 cracks)	0.2	250 (merged with 300 and 200 cracks)	0.6	250 (merged with 300 and 200 cracks)	0.8	250 (merged with 300 and 200 cracks)	1	250 (merged with 300 and 200 cracks)		
550									0.15	450	0.2	550	0.25	550	0.2	550	0.4	550	0.5	550	0.5	550		
650									0.1	250	0.15	250	0.2	500	0.2	600	0.25	700	0.25	700	0.3	700	0.3	700
800									0.15	300	0.15	450	0.15	500	0.2	650	0.2	650	0.2	650	0.2	650		
950													0.1	350	0.1	400	0.15	450	0.15	450	0.15	450		
1100													0.1	150	0.1	250	0.15	550	0.1	550	0.15	550		
1250													0.1	300 (merged with 100 crack)	0.1	300 (merged with 100 crack)	0.1	300 (merged with 100 crack)	0.1	300 (merged with 100 crack)				
											Crack at the base beginning to grow at the end of this cycle		Spalling beginning to take place along the crack above the base, in the boundary region. Crushing beginning. Residual @Base = 1.0, @150 = 0.2, @200 = 0.3		Spalling occurring at base and between crack at 200 and 300 above the base on the (Tension side). Residual @ base = 2.5, @ 150 = 1.0, @ 200 = 1.5, @ 300 = 0.15, @ 450 = 0.15		Entire corner of the wall spalled off at this point, all the way up to 300mm above base. Inside the spalled areas, the cracks are significantly large. Rebar exposed due to cover spalling. Residual @ base = 6.5cm, @ 150 = 1.5, @ 200 = 2.5, @ 300 = 1.0, @ 450 = 0.4		A large 200x200 piece of concrete spalled off at the corner of the wall. At this point, the wall was considered to have failed due to two bars rupturing. The spalling was significant enough to observe reinforcement rupturing and buckling.					