

Experimental Testing and Reliability Analysis of Repaired SMA and Steel Reinforced Shear Walls

by

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A Thesis

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ABSTRACT

Superelastic Shape Memory Alloys (SMAs) are being explored as alternative reinforcing materials to traditional deformed steel reinforcement for seismic applications. The main advantage is the ability of the SMA to recover large nonlinear strains, which promotes the self-centering phenomenon. The primary objective of this research is to present the performance, before and after repair, of slender reinforced concrete shear walls, one reinforced internally with SMAs in the boundary zones within the plastic hinge region and other control wall reinforced with conventional steel only. The repair procedure included removal of damaged concrete within the plastic hinge region, replacing fractured and buckled reinforcement, followed by shortening of the SMA reinforcement in the boundary zones of SMA wall. The removed concrete was replaced with self-consolidating concrete, while the concrete above the plastic hinge region remained intact.

The SMA reinforced concrete shear wall (before and after repair) exhibited stable hysteretic response with significant strength, and displacement and energy dissipation capacities. In addition, the walls exhibited pinching in the hysteretic response as a result of minimizing the residual displacements due to the restoring capacity of the SMA reinforcement. The results demonstrate that SMA reinforced components are self-centering, permitting repairing of damaged areas. Furthermore, the SMA reinforcement is re-usable given its capacity to reset to its original state. The length of the SMA bars in the original and repaired wall, in addition to the presence of starter bars in the original wall, were significant factors in the location of failure of the walls.

The conventional steel wall prior to repair was unstable due to large residual displacements experienced during the original test. After repair the wall exhibited ratcheting in hysteretic response but with significant strength. The conventional wall, before and after repair, dissipated more energy than the SMA wall. This was the result of the wider hysteretic loops with reduced

punching, but at the cost of large residual displacements. The starter bars in the conventional wall before repair controlled the location of failure, while the presence of couplers in the plastic hinge region was the main factor in determining the failure location in the repaired conventional wall.

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CHAPTER 1

INTRODUCTION

1.1 Background

A significant amount of money is spent repairing structures after a seismic event; this amount can exceed the cost of constructing a new building. Thus, in an attempt to reduce the cost of repairs, extensive research is being performed on novel materials and techniques that could minimize structural damage during seismic events.

According to modern building codes, structures subjected to a major earthquake are expected to undergo large displacements to dissipate the energy of the earthquake, such that the structural materials yield and respond in the inelastic range. This requires large ductility capacity that could result in major structural and non-structural damage. Consequently, extreme earthquakes could cause instability of a structure due to the large imposed displacements, which may cause the structure to lose its load carrying capacity and collapse.

In the design of a structure for seismic events, engineers have the following choices: to provide enough stiffness and strength so that the response remains in the elastic range, or to provide enough ductility so that the structure can withstand large inelastic displacements beyond its lateral-load carrying capacity. The first solution is very expensive for high seismic regions, while the latter is more cost effective, but it carries the consequence of permanent damage. In North America, building codes are designed in such a way that a structure can withstand a minor to moderate earthquake without notable structural damage, while remaining in the elastic region, and can sustain (major) earthquake without collapse, but with major structural damage. Building codes have evolved through several changes in response to deficiencies of structures in major

earthquakes. The changes have been reflected in better estimations of the seismic hazard and in the detailing practice of the lateral-load resisting structural components.

New materials, structural elements, and techniques are being developed to have the capacity to sustain large inelastic displacements due to severe earthquakes, but with the additional characteristic to restore the structure to its original position. This phenomenon results in a self-centering structure that can be serviceable after experiencing a major earthquake. In addition, the structure would require minimal repairs. The respective material or technique should be evaluated after any repairs to ensure that the same structural behavior and capacity exists as it was originally designed for. Otherwise, the deformation capacity, load carrying capacity, and ductility of the structure would have to be re-evaluated. If the characteristics stated above are not verified, the structure could experience failure in future earthquakes.

1.2 Scope of Research

Shear walls are commonly used structural elements to resist the lateral forces acting on the structure due to seismic activity and/or wind loading. Shear walls provide lateral resistance to a structure, stiffens the structure, and ensures the stability of multiple floors. Shear walls act as a cantilever beam, fixed at the base, which carries loads to the foundation level as illustrated in Figure 1.. They are subjected to the variable shear forces that attain its maximum at the base, bending moments that causes tension at the loading edge and compression on the opposite edge, and vertical compression due to the gravity loading of the structure.

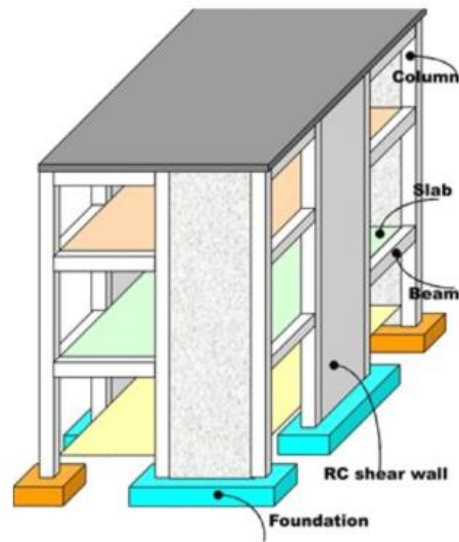


Figure 1.1: Typical Shear Wall (Engineers Daily, 2014)

To ensure ductility, building codes and/or design standards require minimum reinforcement in both the longitudinal and transverse directions of the web section of the walls. The boundary elements at the edge of the shear walls contain longitudinal reinforcement that is tied as a column to enhance strength and ductility. A cross section with typical reinforcement is illustrated in Figure 1.2. In general, shear walls are designed to undergo large inelastic deformations and dissipate significant energy.

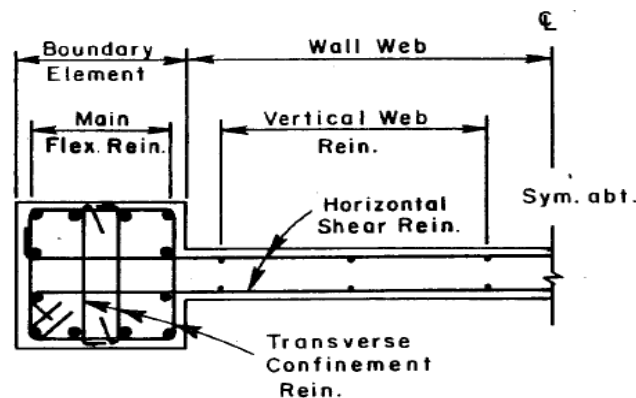


Figure 1.2: Typical Shear Wall Reinforcement Layout (Fiorato, 1983)

The strength and behavior of a slender shear wall is typically governed by the reinforcement in the boundary region. During severe lateral loading, the reinforcement at the boundary yields to dissipate energy, which results in large permanent deformations of the overall structure. Such structures cannot be repaired economically and, therefore, demolition becomes the only alternative. The focus of this study is to address the problems associated with large permanent deformations and ensure that structures are serviceable after minimum repair. Specifically, research was conducted on hybrid-reinforced shear walls, with shape memory alloy as the principal reinforcement at the boundary regions and traditional deformed reinforcement in the web section of the wall.

Shape Memory Alloys (SMAs) are a new group of alloy that is being researched and applied in both engineering and non-engineering applications. The non-engineering applications include medical and orthodontic equipment, fighter jets, shoes, and clothing. Shape memory alloy is a smart material that exhibits the unique property of recovering its deformation after being subjected to extreme straining.

SMA exhibits two distinct crystalline structures or phases: Martensite and Austenite. Martensite is a relatively soft and easily deformed phase, which exists at lower temperatures. Austenite is the stronger phase that occurs at high temperature. One of the unique characteristics of SMAs in the full Austenite phase is its pseudo-elasticity. In this phase, the material transforms from Austenite to Martensite by applying external stress without modifying the temperature. The material re-transforms back into the Austenitic state upon unloading (Figure 1.3). Shape memory alloy can fully recover up to 10% strain. Pseudo-elasticity exists in the alloy when it is in its full austenitic phase.

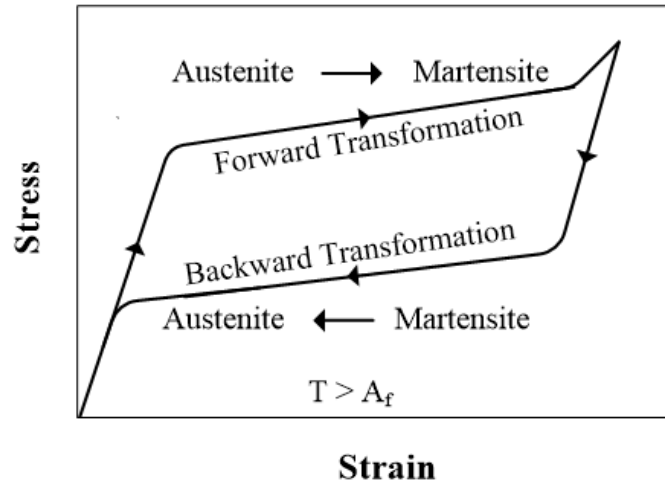


Figure 1.3: Austenite Stress-Strain Relationship (Abdulridha, 2013)

When SMA is applied in structural applications, the following properties are of interest: forward transformation σ_L , reverse transformation σ_{UL} , initial elastic modulus E_i , residual strain ϵ_R , and the equivalent viscous damping ζ_{eq} . The forward and reverse transformation stresses correspond to 3% strain in the SMA during loading and unloading, respectively, as illustrated in Figure 1.4.

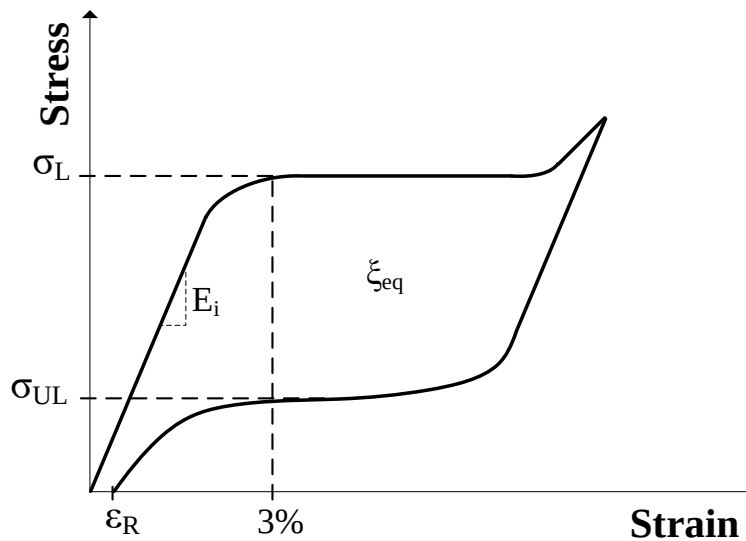


Figure 1.4: SMA Properties for Structural Applications (McCormick, 2007)

Forward transformation (σ_L) is a stress level at which the martensitic phase transformation initiates. At this stage, the SMA experiences a significant stiffness change. At the reverse transformation stress (σ_{UL}), the SMA returns to the austenitic phase, defining the size of the hysteresis loop. The initial elastic modulus (E_i) provides an indication of the expected behaviour of the structure during a seismic event. The initial stiffness affects the magnitude of the structure's acceleration when a lateral load is applied. The residual strain (ϵ_R), referred to as a plastic deformation, provides a measure of the re-centering capabilities of superplastic SMAs. Viscous damping (ζ_{eq}) is the measure of damping provided by the hysteretic behaviour of SMA and is used to compare the energy dissipation with other energy dissipating systems.

Nickel-Titanium (NiTi) alloy is the most common type of SMA. It consists of the following composition: 55.9% Nickel and 44.1% Titanium, with transformation temperature ranging between -92 degree Celsius and 110 degrees Celsius. NiTi SMA is corrosion-resistant and provides a shape-memory strain limit of 8%. Beyond 8%, residual strains begin to accumulate, but are significantly smaller than those developed in traditional deformed reinforcing steel.

The main goal in designing a structure is to ensure public safety and performance during its life span, including post-repair performance. The service life of a structure depends highly on the environmental conditions, material properties and variations in loading. However, when planning and designing, information is rarely complete and often uncertain. Therefore, the only way to ensure an estimation of the performance of a structure is through structural reliability analysis, which involves probabilistic concepts. In the past, before the development of probability analysis, structures were designed deterministically. High margins for resistance were applied, based on past cases of evaluating structural performance under different conditions. This approach was not only highly conservative (not economical), but it did not provide any estimation of the level of

safety. Essentially, it did not take into account the sensitivity of any of the design parameters. Therefore, structural reliability analysis was introduced to address the concerns stated above. This approach leads to a rational design that is economical and provides a level of safety for a given service life for the structure.

1.3 Research Objectives

The main goal of this research is to repair the plastic hinge region of previously tested hybrid-SMA reinforced and traditionally reinforced shear walls and to study their performance under reverse cyclic loading. The focus is to demonstrate that a structure with SMA reinforced shear wall, which experiences a seismic event, can be serviceable after repairing, without compromising the seismic performance. In addition, reliability analysis is proposed using a first-order reliability method to determine the values of the appropriate reliability index for SMA shear walls before and after repairing.

To achieve the main target of this research project, the following project tasks were completed:

- Repairing of two shear walls which were previously subjected to reverse cyclic loading: a control wall that was fully reinforced by conventional reinforcement and a companion wall that was reinforced with SMAs in the plastic hinge region.
- Investigation of the effects of high strength self-consolidating concrete as a repair material in the plastic region.
- Investigation of the effect of shortened SMA bar lengths on the deformation recovery capacity of the shear wall.
- Investigation of the different types of mechanical couplers for reinforcement.
- Comparison of the performance of shear walls before and after repairing.
- Numerical analysis for the reliability of shear walls.

1.4 Novelty of the Research

The research work presented herein has a number of novel contributions, including:

- Repairing damaged SMA shear walls.
- The use of high-strength, self-consolidating concrete as a full depth repair material for plastic hinge regions in slender concrete shear walls.
- The use of SMA as principal reinforcement over only a portion of the plastic hinge length.
- Reliability analysis of repaired shear walls reinforced with a hybrid of SMAs and traditional steel reinforcement

1.5 Thesis Layout

Background of innovative ways to reduce permanent deformations in structures after seismic activity, the scope of research concentrating on hybrid shear walls with SMA reinforcement, and the research objectives are presented in Chapter 1. Chapter 2 contains a literature review of different repair techniques for damaged shear walls, application of SMA reinforcement, and reliability methods for different structural components. The experimental procedure for repairing shear walls is briefly discussed in Chapter 3. Chapters 4 and 5 contain test results and discussion of experimental results, respectively. Chapter 6 introduces reliability analysis of SMA shear walls before and after repair. Finally, Chapter 7 provides conclusions drawn from this study, and future research work.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Significant research has been conducted on repairing shear walls using different materials/techniques to re-evaluate the structural properties after the repairs are performed. Repairing shear walls with current known materials and techniques, such as FRPs, steel plates, or replacing damaged reinforcing bars and concrete can result in substantial recovery of the structural properties. However, there has been limited research focusing on repairing hybrid walls with SMA reinforcement and re-evaluation of the structural properties, and no research known to the author has been reported on the overall structural reliability before and after the repairing of such walls. This chapter presents a snapshot of the literature available for repairing of damaged shear walls, the mechanism and structural applications of shape memory alloys, and introduction to basic reliability and the reliability analysis of different reinforced concrete structural components.

2.2 Repair/Retrofit of Reinforced Concrete Shear Walls

Fiorato et al. (1983) conducted an experimental study on the behavior of earthquake-resistant structural walls before and after repair. The goal of the study was to investigate damage experienced by the structural walls due to extreme lateral loading and to evaluate different repair techniques for the damaged walls. The test walls were scaled down to one-third of five-story walls, omitting the floor slabs at each level. The walls consisted of a column boundary element at each end and a web element in the middle, representing a barbell shape (Figures 2.1 a) and b)).

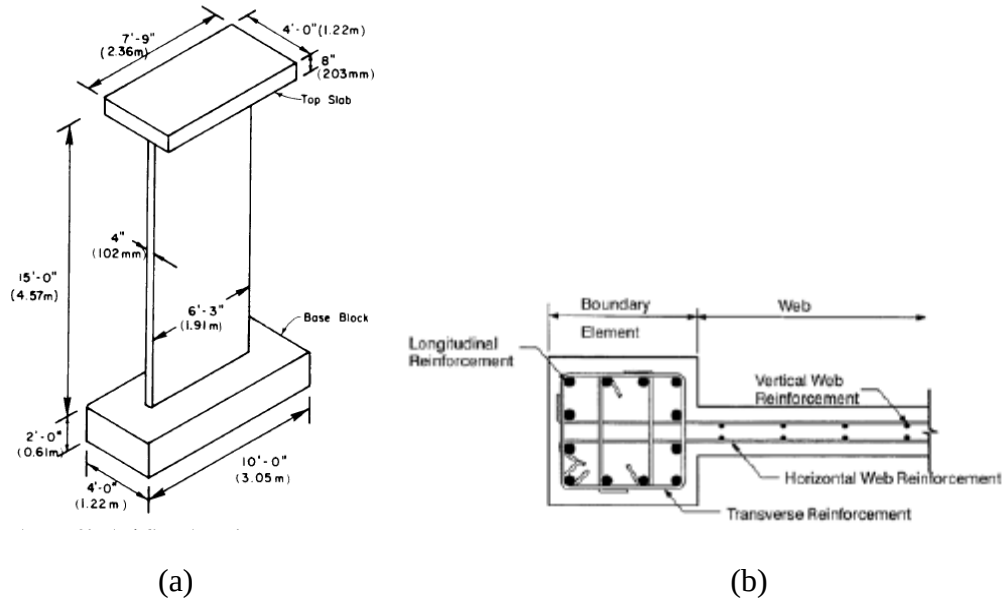


Figure 2.1: Test Specimen: (a) Nominal Dimensions; and (b) Reinforcement Details (Fiorato et al., 1983)

Table 2.1 provides the material properties and imposed axial load for each set of walls that were first tested, repaired, and then retested.

Table 2.1: Properties of Test Specimen (Fiorato et al., 1983)

Specimen	Shape	Axial load, psi	f'_c , psi	f_y for ρ_p , ksi	Reinforcement, percent			
					ρ_f	ρ_h	ρ_n	ρ_s
B5		—	6570	64.4	3.67	0.63	0.29	1.35
B5R*		—	6205	64.4	3.67	0.63	0.29	1.35
B9'		545	6395	62.3	3.67	0.63	0.29	1.35
B9R**		450	7510	62.3	3.67	0.42	0.20	1.35
B11'		—	7800	63.2	3.67	0.63	0.29	1.35
B11R**		—	6175	63.2	3.67	0.63 [†]	0.29 [‡]	1.35

*Repaired specimen.

[†]Modified reversing load history (MR loading).

[‡]2 No. 5 bars at 45 deg in each direction added in lower 3 ft of web. 1000 psi = 1.0 ksi = 6.895 MPa.

ρ_f = ratio of main flexural reinforcement area to gross concrete area of boundary element.

ρ_h = ratio of horizontal shear reinforcement area to gross concrete area of a vertical section of wall web.

ρ_n = ratio of vertical web reinforcement area to gross concrete area of a horizontal section of wall web.

ρ_s = ratio of effective volume of confinement reinforcement to the volume of core in accordance with Eq. (A.2) of ACI 318-77.

After testing the original walls B5, B9, and B11, it was observed that for all the specimens, the lower portion of their web was severely damaged and the confined boundary elements remained

intact. The damaged webs of each specimen were repaired using different techniques, which were cost effective and capable of restoring the strength of the wall. Wall B5 was repaired by replacing the damaged concrete in the web with new concrete to its original thickness. For Wall B9, the web thickness was increased as part of the repair. For Wall B11, supplementary reinforcement was added to the web, and then the concrete in the web was replaced to its original thickness. The repaired walls were tested under reverse cyclic loading and their performance was compared to the original walls in terms of load and deformation characteristics. The following results were obtained: the strength and deformation capacity of the repaired walls were the same as the original walls, the stiffness of the repaired walls was approximately 50 percent of that of the original walls, the deformation capacity of Wall B9 was increased, and for Wall B11 the shear distortions were reduced with the increase in deformation capacity.

Lefas and Kotsovos (1990) studied the performance of reinforced concrete walls to investigate the effects of the loading history and repair techniques on the structural characteristics of the walls. Three identical large-scale slender walls (SW31R, SW32R, and SW33R) were tested to failure then repaired with two different techniques and re-tested under different levels of horizontal cyclic loading. The test specimens represented a critical story element of a structural wall with a rectangular cross section. Wall SW33R was repaired only by replacing the damaged concrete in the compressive zone. For Walls SW31R and SW32R, epoxy resin was injected in major cracks in the web section, with replacement of the damaged concrete in the compressive zone. The test results indicated that the addition of epoxy resin to heal major cracks contributed to only a marginal improvement in strength, while just repairing the damaged regions of the compressive zone was sufficient to restore strength to a reasonable level. Overall, it was concluded that the compressive zone is the main contributor to shear resistance.

Vecchio et al. (2002) examined the structural characteristics of repaired shear walls in terms of strength, stiffness, and energy dissipation. Two large scale, squat wide-flanged walls originally tested under quasi-static reverse cyclic loading suffered heavy damage in the web wall. Both walls had the same H-cross section shape, built with stiff top and bottom slabs. The walls were repaired by replacing the damaged concrete in the web while preserving the original reinforcement (Figure 2.2). In addition, the concrete over a short length of the flange wall centered about the web wall was also repaired in Wall DP2. The testing of the repaired shear walls concluded that removing and replacing the damaged concrete can fully restore the strength, post cracking stiffness, ductility, and energy dissipation capacity (Figures 2.3 and 2.4). Also, it was observed that the behavior of repaired walls was highly affected by previous loading history, repair sequence, repair material properties, and residual damage in unrepaired zones.

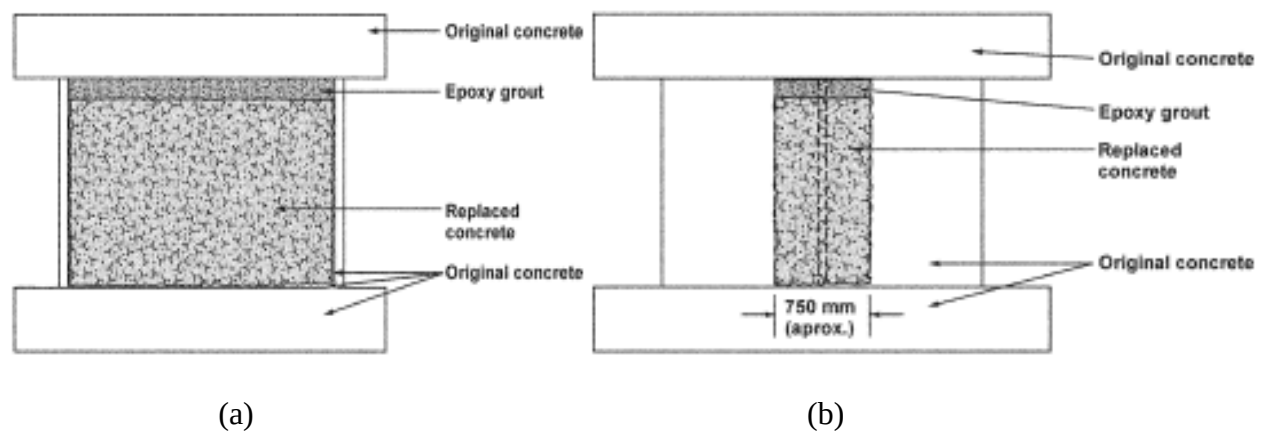


Figure 2.2: Repair Scheme of Specimen: (a) DP1; and (b) DP2 (Vecchio et al., 2002)

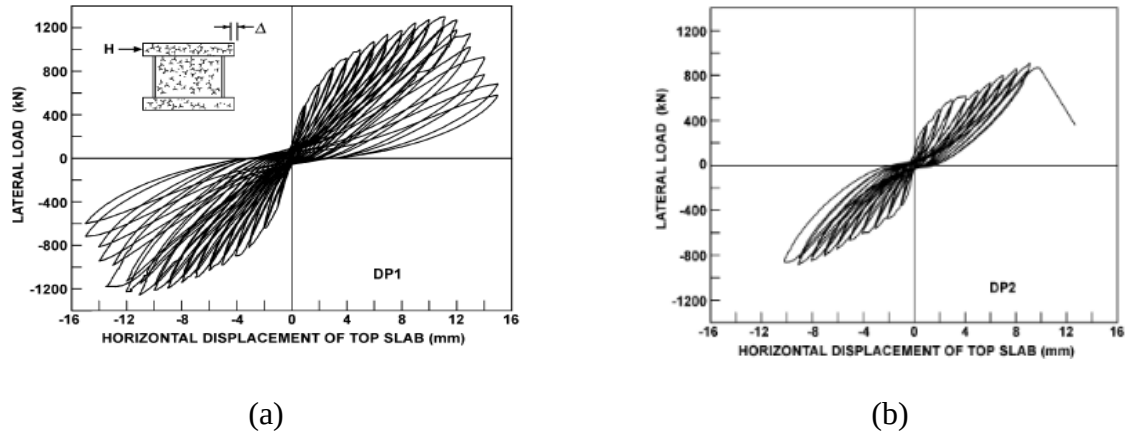


Figure 2.3: Load-Displacement Response of Original Walls: (a) DP1; and (b) DP2 (Vecchio et al., 2002)

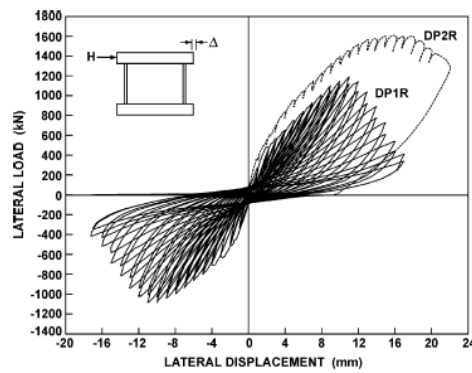


Figure 2.4: Load-Displacement Response of Repaired Walls (Vecchio et al, 2002)

Antoniades et al. (2003) investigated the effects of repaired reinforced concrete (RC) walls strengthened with fiber-reinforced polymers. Experimental studies were conducted on five low-slenderness walls designed in accordance with the Eurocode (Eurocode 8, 1998) provisions. The walls were initially subjected to reverse cyclic loading to failure, due to which they suffered heavy damage in critical regions, including fracture of some longitudinal bars. The repair process involved lap-welding of fractured longitudinal bars with new segments of reinforcement in the plastic hinge region and replacing damaged concrete with high-strength mortar. After repairing, four of the walls were strengthened by wrapping with an FRP jacket and adding FRP strips at the

wall edges for enhancement of shear and flexural capacities, respectively (Figure 2.5). One of the repaired walls was not retrofitted with FRP in order to examine the strength and deformability due to the repair procedure only.

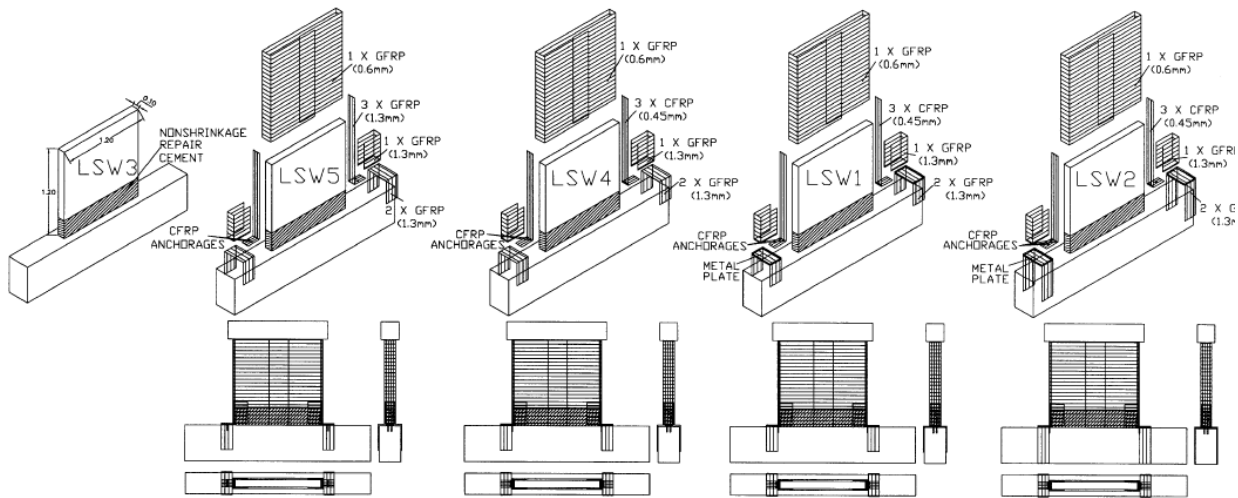


Figure 2.5: Repair and Strengthening Procedures Applied to Each Wall
(Antoniades et al., 2003)

The walls were retested and their behavior was observed. The repaired only wall specimen experienced a full recovery in strength, but not in stiffness and energy dissipation capacity. For the repaired and FRP retrofitted walls, the strength increased up to 30% with respect to the original wall, but the energy dissipation could not be fully restored to that of the original walls.

Riva et al. (2004) investigated the structural behavior of a full-scale repaired RC structural wall. The shear wall was part of a four-storey building, designed according to Eurocode 8 (1998), with one underground storey and a box foundation. The wall was tested under reverse cyclic loading to failure, which was controlled by sliding shear. The repair process of the wall consisted of replacing yielded reinforcement with new reinforcing bars using mechanical couplers and replacing damaged concrete (Figure. 2.6). The repaired wall was retested according to the loading history of

the original wall. However, the test was terminated following a few cycles at the yield displacement, prior to failure at the location of the reinforcing bar coupler. The results indicated that the repair was effective up to yield. After yielding, compression bearing stress concentrated at the end of the reinforcing bar coupler, consequently causing failure in shear. The results were not satisfactory, but they highlighted necessary improvements for the adopted repair strategy.

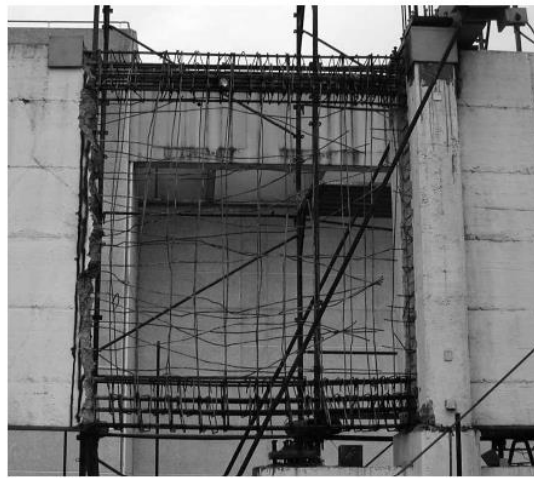


Figure 2.6: Critical Region After Concrete Removal (Riva et al., 2004)

Elnashai and Pinho (1998) investigated methods of repair and retrofitting capable of individually affecting single structural characteristics, such as stiffness, overall strength, and ductility. Several scale models of approximately 1:25 that were previously tested under reverse cyclic loading by Elnashai and Salama (1992) were repaired and retrofitted using selective techniques. The repaired and retrofitted walls were retested under reverse cyclic loading to failure. The researchers concluded the following:

Stiffness only intervention: the best results were obtained by using steel plates at the edges to maximize geometric eccentricity, thus reducing flexural cracks.

Strength only intervention: The best results were obtained by using a longer lever arm and smaller steel plates with less yield strength to minimize reduction in ductility.

Ductility only intervention: The best results were obtained by minimizing the spacing between steel plates and maximizing the height of the plates, as illustrated in Figure 2.7.

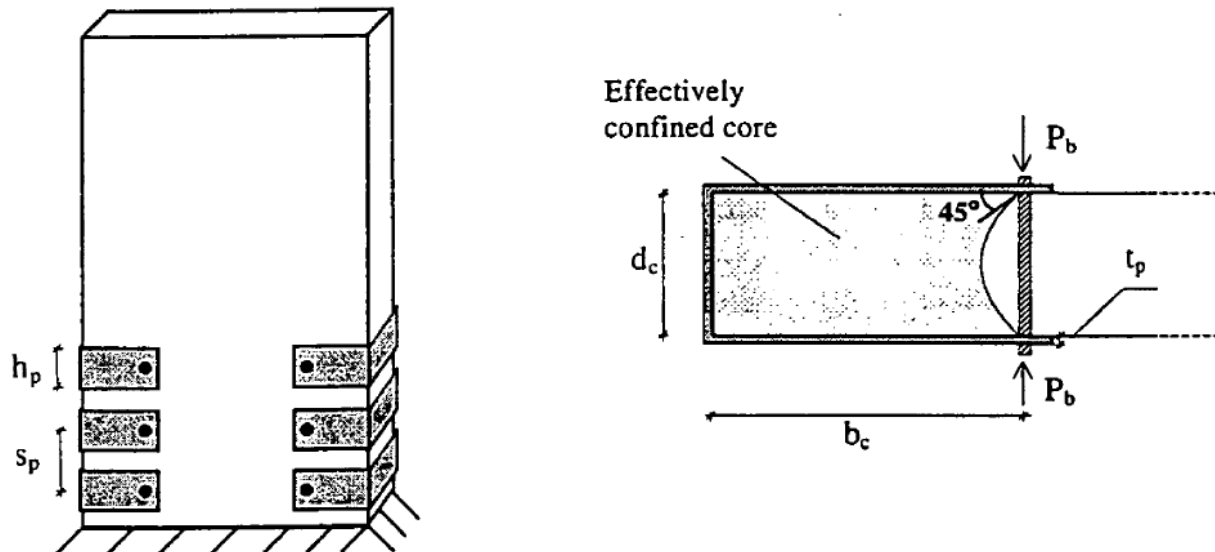


Figure 2.7: Ductility-Only Intervention Strategy (Elnashai and Pinho, 1998)

2.3 Response of Shape Memory Alloys During Cyclic Loading

DesRoches et al. (2004) focused on testing sets of various SMA wires and bars to examining their mechanical characteristics. The tests were performed under both quasi-static and dynamic cyclic loading. Mechanical properties sensitive to seismic activity such as residual strain, forward and reverse transformation stresses, and energy dissipation (viscous damping) were investigated and compared between SMA wires and bars. The tests demonstrated that both wires and bars followed idealized super-elastic behavior (Figure 2.8). The residual strain for both wires and bars increased as the cyclic strain increased. However, no relationship was found between residual strain and bar diameter. Higher strength and equivalent viscous damping were found in wires compared to bars, but the energy dissipation was still low for super-elastic SMAs to be considered for damping applications.

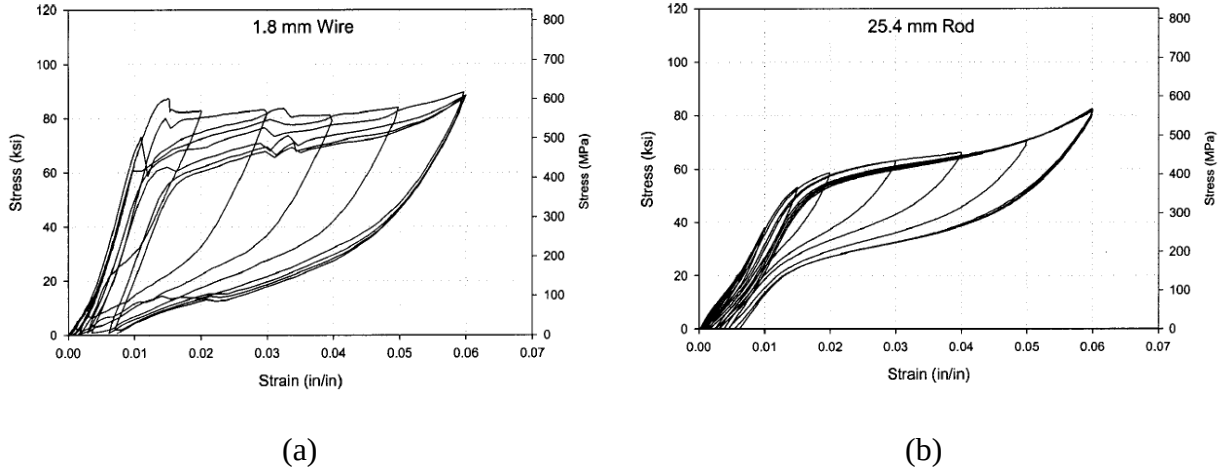


Figure 2.8: Stress–Strain for Nitinol Shape Memory Alloy Wire Subjected to Quasi-Static Cyclic Loading: (a) 1.8 mm Diameter; and (b) 25.4 mm Diameter (DesRoche et al., 2004)

Under dynamic cyclic loading, it was observed that an increase in strain rate resulted in an increase of both loading and unloading stress, but the increase in unloading stress was significantly higher. This resulted in a narrow hysteresis curve, due to which the viscous damping capacity of both wires and bars decreased (Figure 2.9).

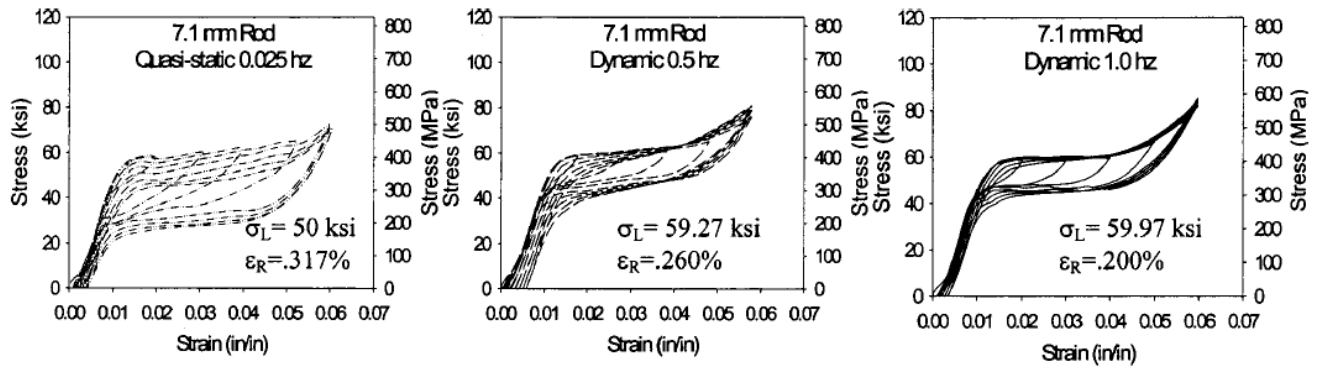


Figure 2.9: Stress–Strain for 7.1 mm Diameter Nitinol Shape Memory Alloy Bar Subjected to Quasi-Static and Dynamic Cyclic Loading (DesRoche et al., 2004)

The results obtained by DesRoches et al. (2004) were confirmed by Fugazza (2005). The researcher investigated the cyclic properties of shape memory alloys to determine their

applicability in earthquake engineering. Tests were conducted on two SMA wires and one SMA bar. The SMA wires were provided by two different manufacturers (CNR-IENI and Memory Corp.), with diameters of 1.00 mm and 0.76 mm, respectively. Static and dynamic loadings were applied to the test specimens using a 100 kN, MTS 810 multi-purpose servo-hydraulic testing apparatus. During testing, the mechanical properties of SMAs were observed, and demonstrated effective stiffness, re-centering capabilities, and transformation. Under static loading (Figure 2.10), superplastic behavior was observed in both the wires and the bar. However, the cyclic properties of the two wires were not the same. The different compositions used by the manufacturers resulted in the Memory Corp. wire displaying a higher reduction of forward transformation stress compared to the CNR-IENI wire.

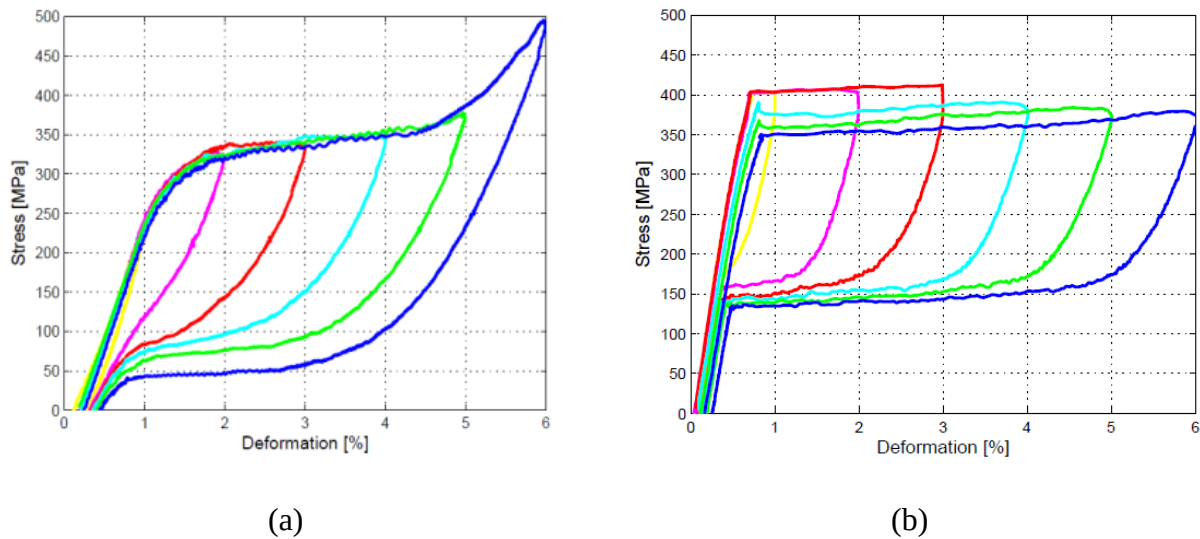


Figure 2.10: Stress–Deformation for Wires in Static Test: (a) CNR-IENI; and (b) Memory Wires (Fugazza, 2005)

Under dynamic loading (Figure 2.11), an increase in the strain rate of the wires resulted in a higher stress levels at which forward and reverse transformations occur. However, the unloading stress was observed to be much higher compared to loading stress.

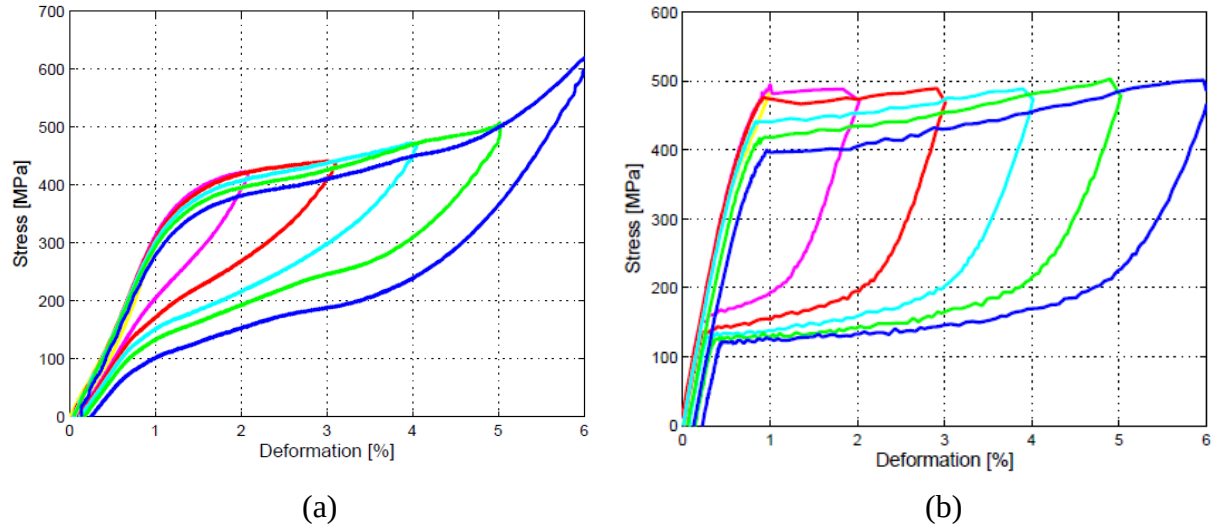


Figure 2.11: Stress–Deformation for Wires in Dynamic Test: (a) CNR-IENI; and (b) Memory Wires (Fugazza, 2005)

2.4 Application of Shape Memory Alloys

2.4.1 Reinforcement in Reinforced Concrete

Youssef et al. (2008) studied the seismic behavior of a moment-resisting frame. Moment-resisting frames are designed to perform as a strong column and weak beam in order to develop a plastic hinge in the beam near the face of the column. At the plastic hinge, the longitudinal bars are allowed to yield to dissipate earthquake energy. The yielding of the bar causes permanent deformation of the frame, resulting in severe damage to the entire structure. Shape memory alloys, due to its super-elasticity were investigated as reinforcement at the beam-column joint (BCJ). Two frames were tested: one with conventional steel reinforcement, named JBC-1 (Figure 2.12); and the other with SMAs, referred to as JBC-2 (Figure 2.13). The SMAs in JBC-2 were connected to conventional steel using mechanical couplers (Figure 2.14). The specimens were tested under reverse cyclic loading and their performances were compared. After testing, it was observed that Frame JBC-2 had very low residual strains at the BCJ compared to Frame JBC-1, as shown in the

hysteretic curves presented in Figure 2.15 and Figure 2.16. In the Frame JBC-1, the plastic hinge was developed in the beam at the face of the column; while in Frame JBC-2, the plastic hinge was shifted away from the column to the mid-height of the beam. In terms of energy dissipation, Frame JBC-1 dissipated more energy than Frame JBC-2, but at the cost of a large storey drift.

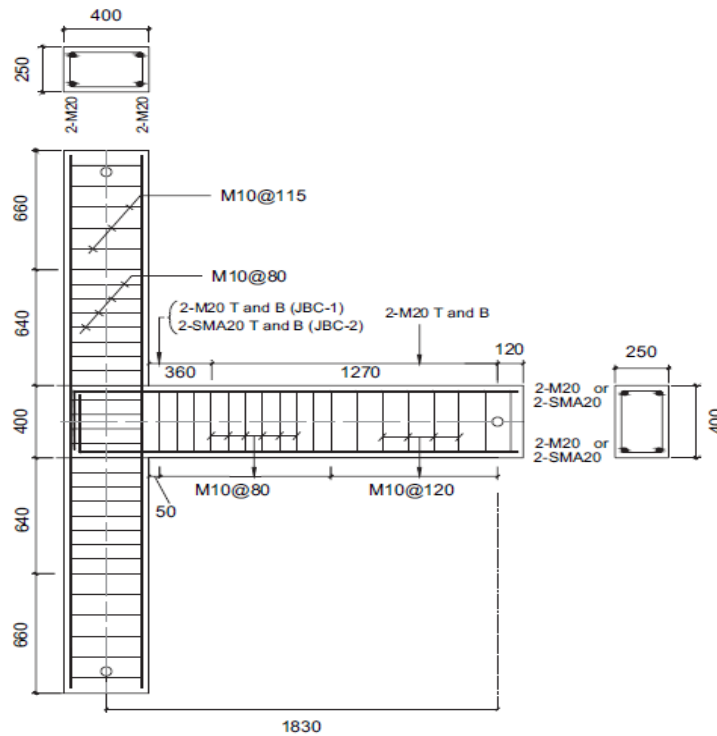


Figure 2.12: Beam-Column Joint with Conventional Steel Reinforcement (JBC-1)
(Youssef et al., 2008)

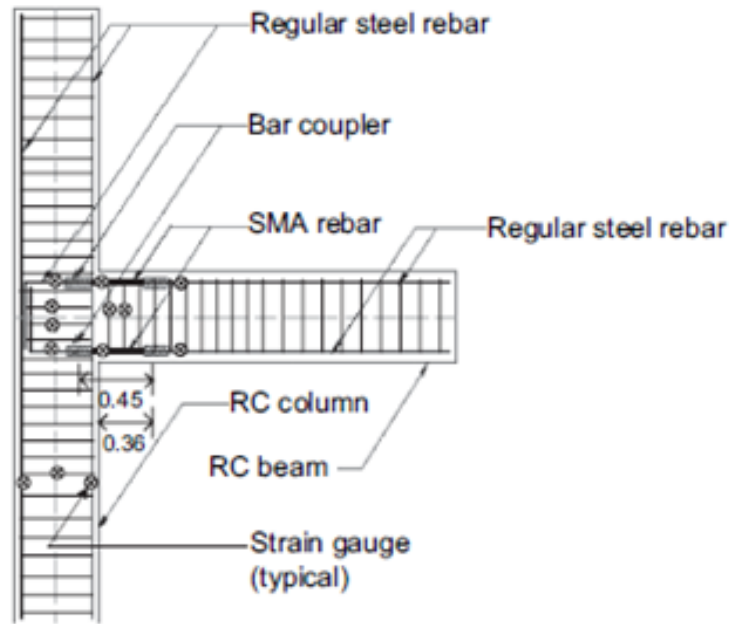


Figure 2.13: Beam-Column Joint with SMA Reinforcement (JBC-2)
(Youssef et al., 2008)

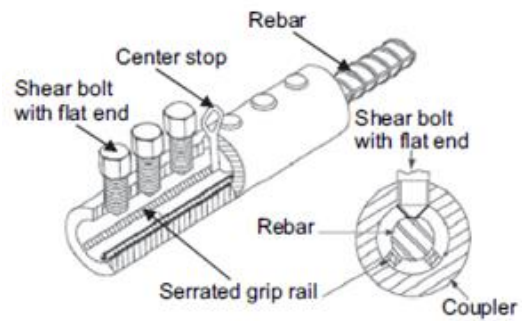


Figure 2.14: Mechanical Coupler (Youssef et al., 2008)

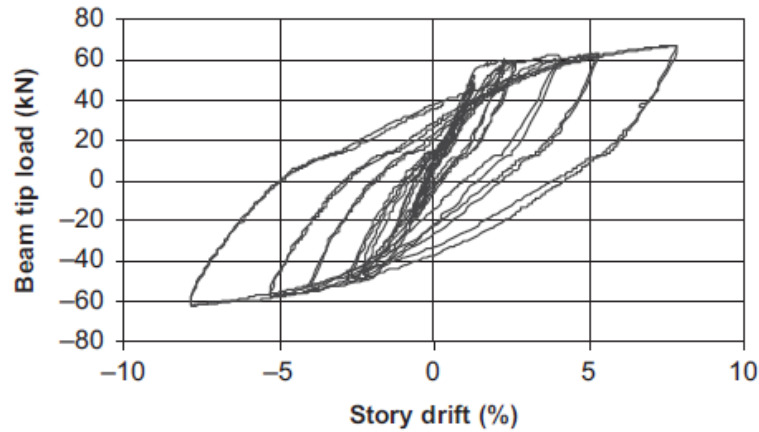


Figure 2.15: Load-Storey Drift of JBC-1 (Youssef et al., 2008)

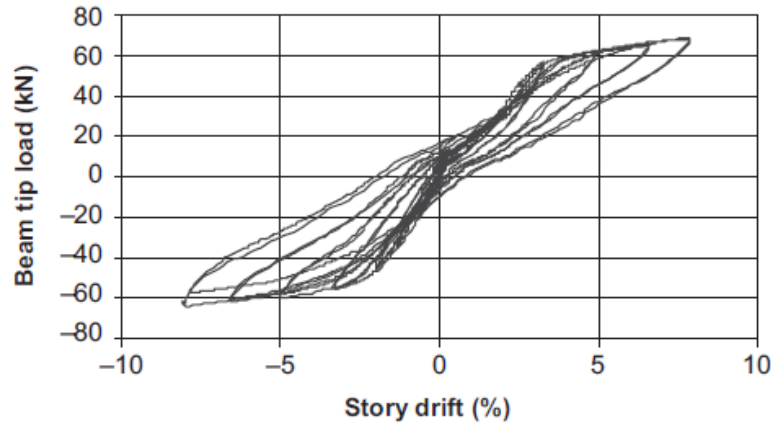


Figure 2.16: Load-Storey Drift of JBC-2 (Youssef et al., 2008)

Saiidi and Wang (2006) investigated the application of shape memory alloys as reinforcement in columns. The main objective of the study was to investigate a decrease in residual displacement of concrete columns due to the super-elastic property of SMA, and also to study the seismic performance and damage of an SMA column repaired using engineered cementitious composites (ECC). A quarter-scale column reinforced with shape memory alloy in the plastic hinge region (Figure 2.17 (a)) area was tested on a shake table to simulate earthquake loading. After the test, the damaged column was repaired using ECC in the plastic hinge area, and the column was retested. The test results showed that the columns reinforced with SMA are able to recover almost

all of their post yielding deformation as demonstrated in Figure 2.17 (b). The use of ECC significantly reduced concrete damage, thus requiring minimum repair even after a strong earthquake.

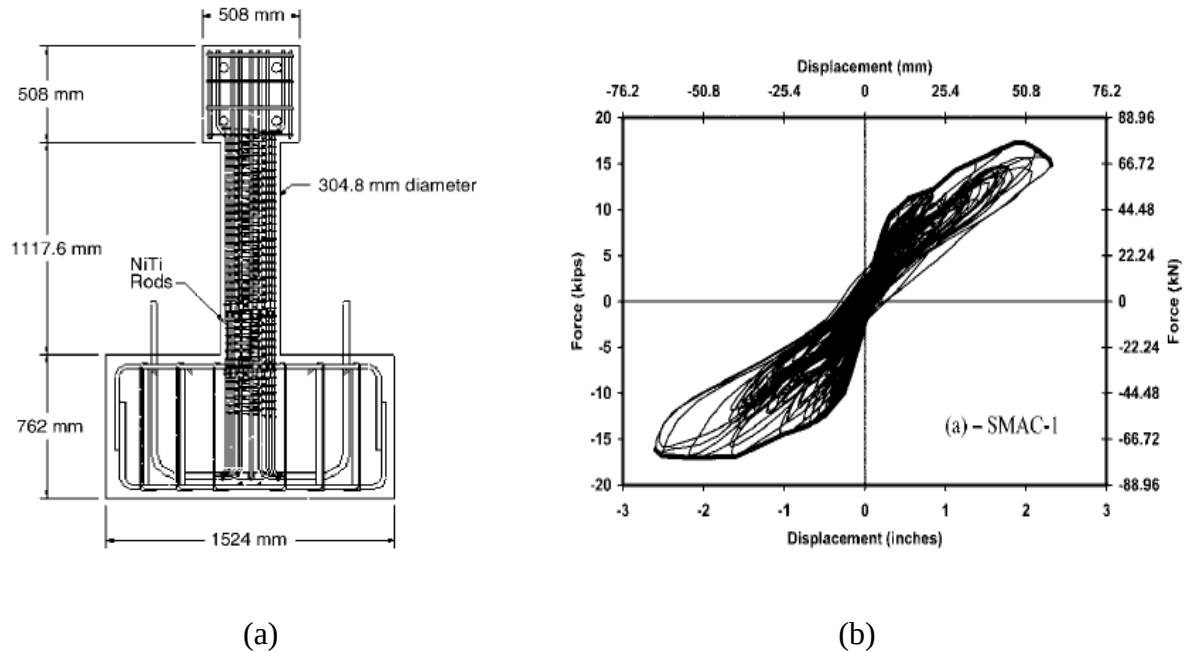


Figure 2.17: Column Reinforced with SMAs: (a) Nominal Dimension; and (b) Measured Cumulative Hysteresis Curves (Saiidi and Wang, 2006)

Effendy et al. (2006) focused his research on using shape memory alloy bars as an external structural bracing system for low-rise shear walls (Figure 2.18). The purpose of using an SMA bar as a structural brace was to increase the ductility and energy dissipation capacities of the walls, since conventional low-rise shear walls lack both of these structural properties. Two different types of SMA bars were used in this study: super-elastic SMA and Martensite SMA. Super-elastic SMA recovers its shape upon removal of stress, while Martensite SMA also possesses the ability to recover its shape but through heating. Three low-rise shear walls (Table 2.2) were tested under reverse cyclic loading, two of which used two different types of SMA bars for external bracing on

both sides of the wall (Figure 2.19). The walls were designed with height, length and width of 1.0 m, 2.0 m and 0.12 m, respectively. The external SMA bracing was placed at 25 degrees from the horizontal on the two walls.

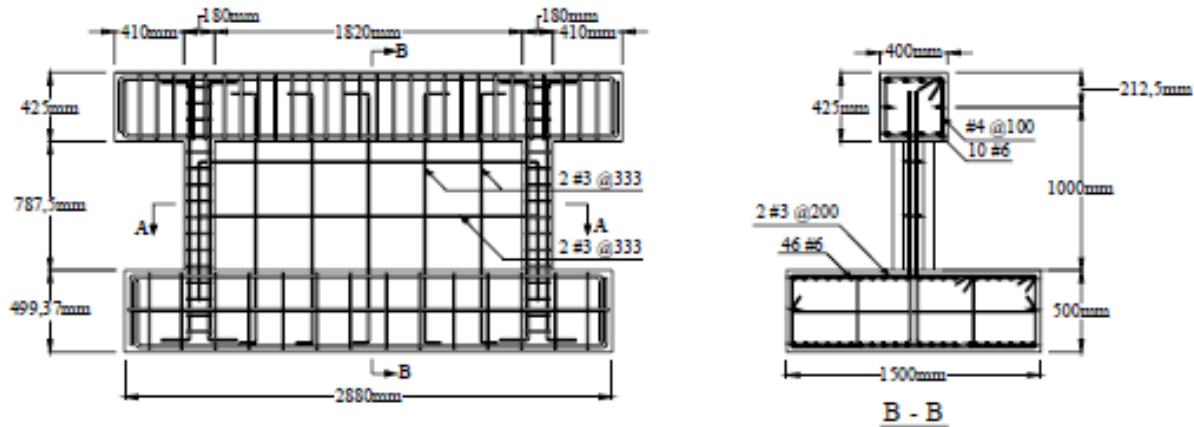


Figure 2.18: Dimensions and Reinforcement of SMAC (Effendy et al., 2006)

Table 2.2: Steel Ratios of Test Specimen (Effendy et al., 2006)

Specimen	Steel Ratio (%)		SMA Bars (mm ²)
	Horizontal	Vertical	
SMAC	0.48	0.48	N/A
SMAS	0.24	0.24	4 x 127 (Superelastic)
SMAM	0.24	0.24	4 x 127 (Martensite)

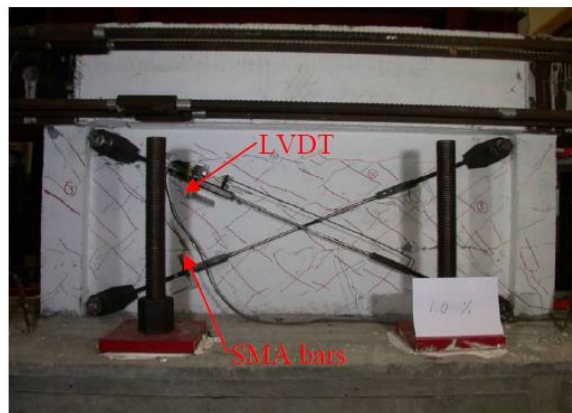


Figure 2.19: Position of SMA bars and LVDTs on the SMA bars (Effendy et al., 2006)

The test results indicated that the shear strength of the walls was affected by the SMA bars. The wall with Martensite SMA had much greater residual displacement compared to the wall with super-elastic SMA. Also, during testing, it was observed that both specimens with SMA bracing did not suffer failure in the boundary elements compared to the specimen with no external bracing, which failed at the bottom of the boundary element. However, the energy dissipation capacity was less than expected due to the buckling of SMA bars at the ultimate loading stage. The test results for the three specimens are summarized in Table 2.3.

Table 2.3: Ductility and Maximum Shear Force (Effendy et al., 2006)

	Δ_c (mm)	Δ_y (mm)	Δ_m (mm)	Δ_u (mm)	μ	Maximum shear force(ton)
SMAC	1.73	4.97	10.03	13.93	2.80	89.67
SMAS	1.41	3.63	6.45	11.41	3.14	114.66
SMAM	1.46	3.03	5.07	10.91	3.60	105.41

Note: Δ_c = cracking displacement

Δ_y = yield displacement

Δ_m = displacement at maximum shear force

Δ_u = displacement corresponding to 80% of the maximum shear force in the descending branch

μ = ductility (Δ_u / Δ_y)

Saiidi et al. (2007) studied the behavior of reinforced concrete structures when reinforcement is fully or partially replaced by SMA. Experimental studies were conducted on small scale concrete beams reinforced with various reinforcement ratios from 0.1 to about 0.9%. Eight reinforced concrete beams were constructed and tested under cyclic loading. All the beams were identical in shape but different in type and amount of reinforcement at mid-span as shown in Table 2.4. The experimental results indicated that the average residual displacement in beams reinforced with SMA was less than one-fifth of that of steel-reinforced beams. However, due to SMA's low modulus of elasticity, the SMA beams had less stiffness compared to steel reinforced beams.

Overall, it was concluded from the results that a hybrid system which combines SMA with steel is a better choice for the design.

Table 2.4: Test Variables (Saiidi et al. 2007)

Specimen	Reinforcement Type	Reinforcement at midspan	Reinforcement ratio (ρ) at midspan
BNL1	NiTi	1 bar, $\phi=6.400$ mm ($\phi=0.250$ in.)	0.0012
BNL2	NiTi	2 bar, $\phi=6.400$ mm ($\phi=0.250$ in.)	0.0023
BNH1	NiTi	1 bar, $\phi=9.500$ mm ($\phi=0.375$ in.)	0.0026
BNH2	NiTi	2 bar, $\phi=9.500$ mm ($\phi=0.375$ in.)	0.0052
BSL1	Steel	1 bar, $\phi=9.500$ mm ($\phi=0.375$ in.)	0.0026
BSL2	Steel	2 bar, $\phi=9.500$ mm ($\phi=0.375$ in.)	0.0052
BSH1	Steel	1 bar, $\phi=12.700$ mm ($\phi=0.500$ in.)	0.0047
BSH2	Steel	1 bar, $\phi=12.700$ mm ($\phi=0.500$ in.)	0.0094

Abdulridha and Palermo (2014) conducted an experimental study using shape memory alloys as reinforcement for ductile reinforced concrete shear walls. Two, ductile shear walls were constructed identically: a hybrid SMA-steel reinforced wall (W2-NR); and a conventional steel reinforced wall (W1-SR). In Wall W2-NR, SMA was provided as the principal reinforcement in the boundaries over the plastic hinge. The walls were tested under reverse cyclic loading with no axial load until failure. The results demonstrated that the hybrid SMA wall was significantly more effective at re-centering, and less residual displacements were recorded after subjecting the wall to large drifts, compared to the conventional wall. However, both walls experienced similar displacement capacity and failure due to rupturing of the reinforcing bars near the base of the wall. For W1-SR, the steel reinforcement ruptured in the boundary zones; while for W2-NR, the steel reinforcement in the web section of the wall ruptured and the SMAs remained intact in the

boundary zones. The hybrid SMA wall had lower initial stiffness and dissipated less energy. Also, the yield and peak load capacities were comparable. The hysteretic curves for W1-SR and W2-NR are presented in Figures 2.20 and 2.21, respectively.

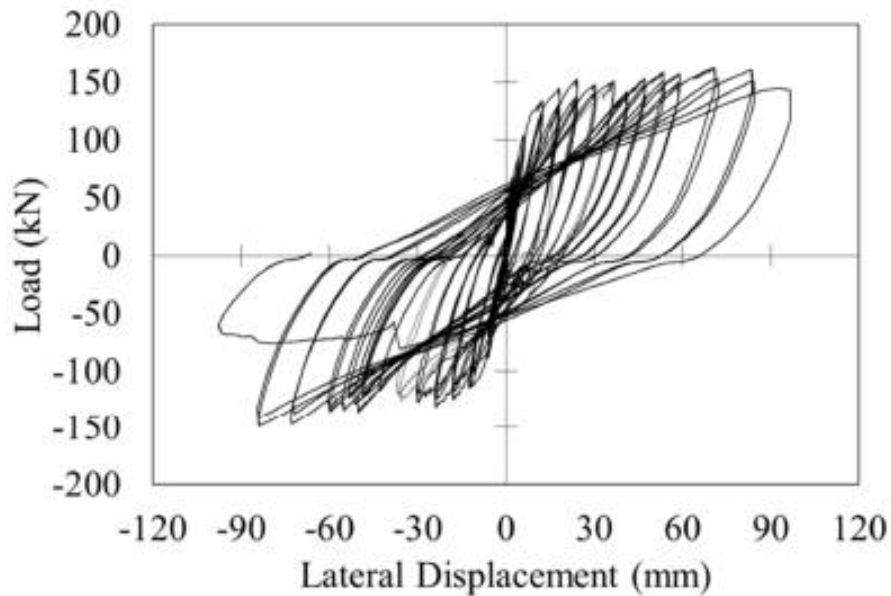


Figure 2.20: Load-Displacement Response for W1-SR (Abdulridha and Palermo, 2014)

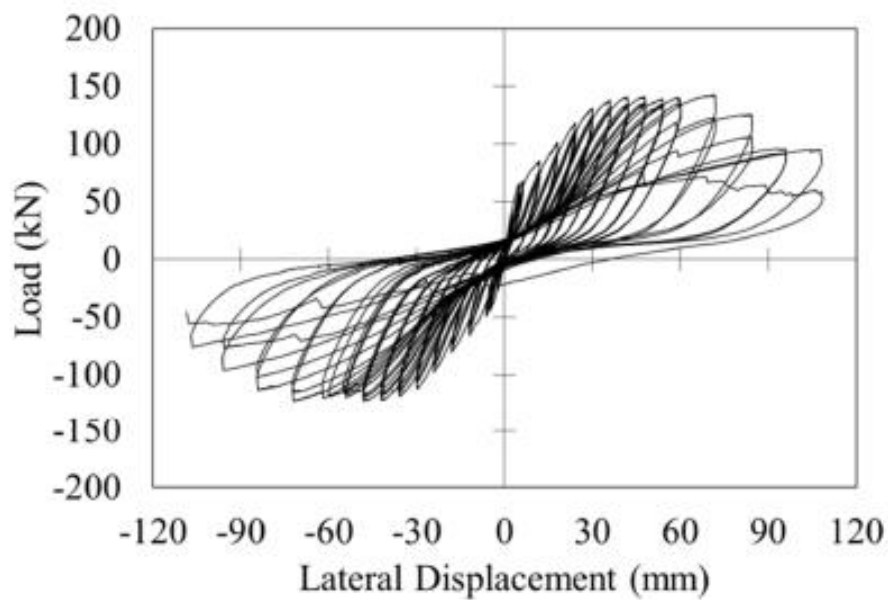


Figure 2.21: Load-Displacement Response for W2-NR (Abdulridha and Palermo, 2014)

2.5 Structural Reliability Analysis

A deterministic approach is used when the load and the corresponding resistances are known; while on the other hand, a return period concept considers only the probability of loading exceeding a certain limit. In reality, at a given point in time, the actual loading is uncertain, as illustrated by the histogram of a floor loading shown in Figure 2.22.

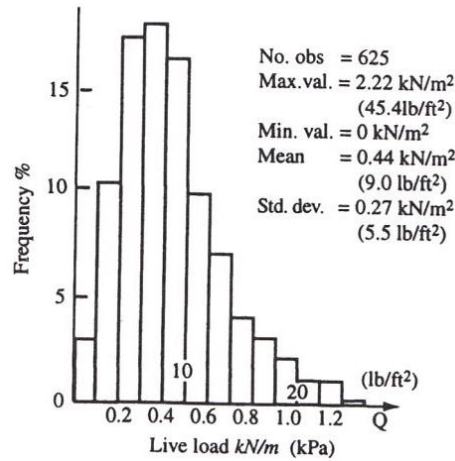


Figure 2.22: Floor Loading Histogram (Melchers, 1999)

This histogram shows that the probability of the floor loading lying between 0.6 and 0.7 kPa is only about 7%. To tackle these problems in engineering design, the load and the resistance are treated as the random variables. The probability that the random variable X takes on a value less than or equal to x (a specific value) is given by:

$$P(X \leq x) \equiv F_x(x) = \int_{-\infty}^x f_x(\varepsilon) d\varepsilon \quad (2.1)$$

where $F(x)$ is the cumulative distribution function of X , and $f_x(x)$ is the probability density function. A distribution is defined by a number of properties known as moments which may include a mean or expected value, a variance and standard deviation, and skewness.

The applied load (S) and the resistance (R) of a structure fluctuate with time and have uncertain value at any point in time. Over time, the loads have the tendency to increase and the resistance to decrease. Accordingly, the probability density function $f_S()$ and $f_R()$ become wider and flatter with time, and the mean values of load and resistance also change, as illustrated in the Figure 2.23.

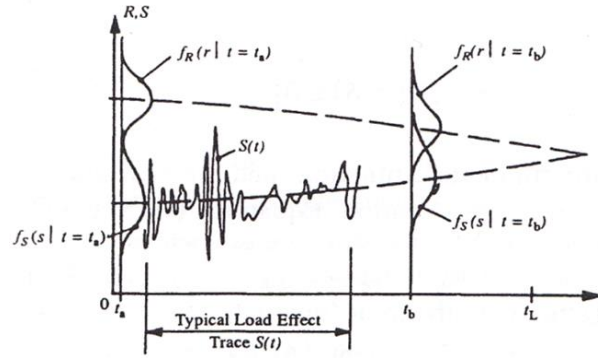


Figure 2.23: Load and Resistance Distribution as Function of Time (Melchers, 1999)

2.5.1 Basic Reliability

The safety of a structural element is considered to be violated when the resistance R is smaller than the applied load S . Hence, the probability (p_f) of failure of the structural element can be stated using the following form:

$$p_f = P(R \leq S) = P(R - S \leq 0) = P\left(\frac{R}{S} \leq 1\right) \quad (2.2)$$

Equation 2.2 can be rewritten in a general form as:

$$p_f = P[G(R, S) \leq 0] \quad (2.3)$$

where $G(R, S)$ is defined as a “limit state function”

In general, f_r and f_s for resistance and load, respectively, can be expressed as a joint density function $f_{RS}(r, s)$, which is the probability that an event occurs for given values of both r and s .

The probability for the joint density function becomes:

$$p_f = P(R - S \leq 0) = \iint_D f_{RS}(r, s) dr ds \quad (2.4)$$

When R and S are independent. The above can be represented by Figure 2.24.

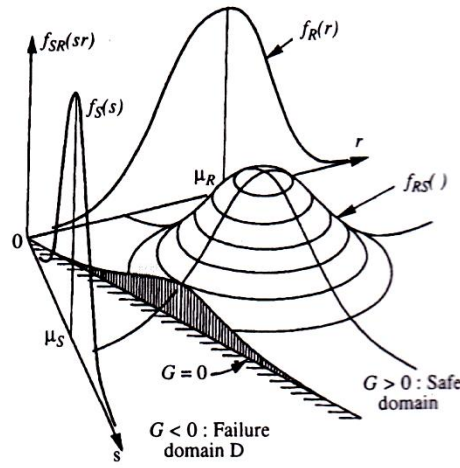


Figure 2.24: Joint Density Function $f_{RS}(r, s)$ (Melchers, 1999)

Figure 2.24 illustrates the joint density function $f_{RS}(r, s)$ with density function f_r and f_s and the failure domain D expressed by Equation 2.4. For the special case when R and S are independent, Equation 2.4 can be written in a single form as:

$$p_f = P(R - S \leq 0) = \int_{-\infty}^{\infty} F_R(x) f_s(x) dx \quad (2.5)$$

2.5.2 Multi-Dimensional Integration

In this section, the computational aspect of the integrals mentioned in the previous sections will be discussed. There are three ways in which the multi-dimensional integration might be performed:

- 1) Direct integration (only possible in some cases)
- 2) Numerical integration, such as the Monte Carlo technique
- 3) Transforming the integrand into a multi-normal joint probability density function

2.5.2.1 Direct and Numerical Integration

Analytic integration of complex integrals is only possible in very special cases that are of very limited practical interest. A numerical solution of integration can be easily obtained using numerical methods such as the trapezoidal rule, Simpson rule, and Gauss-Hermite quadrature formulae.

The computational demands of numerical integration increase rapidly with dimension n of the integration space. Also, to conduct numerical integration, the region must be confined to simpler regions, such as the hypercube and n -dimensional solid sphere.

2.5.2.2 The First-Order Reliability (FOR) Method

Direct or numerical integration methods are very expensive. The difficulty increases with an increase in random variables (x) and with the integration of a nonlinear multi-dimensional joint probability. They require an extensive amount of calculations and repetitions. The first order reliability method is developed to easily approximate the probability of failure of such functions in the area of structural reliability.

The probability density function $f_x ()$ in the integrand can be simplified. Equation 2.4 can be generalized as:

$$p_f = [G(X) \leq 0] = \int \dots \int_{G(x) \leq 0} f_x(x) dx \quad (2.6)$$

The first step in simplification is to transfer all the variables to their standardized normal space with zero moment and unit variance. The transformation can be applied by using the well-known transformation method:

$$Y_i = \frac{X_i - \mu_{x_i}}{\sigma_{x_i}} \quad (2.7)$$

Where Y_i has $\mu_{x_i} = 0$ and $\sigma_{x_i} = 1$. This transformation changes the joint probability density function $f_x(x)$ as illustrated in Figure 2.25 to the transformed space $y = (y_1, y_2)$ illustrated in Figure 2.26.

The Limit state function $G(X)$ is a random function consisting of more than two basic random variables:

$$G(X) = Z(X) = a_0 + a_1X_1 + a_2X_2 + \dots + a_nX_n \quad (2.8)$$

Generally $G(X) = 0$ is also a nonlinear multi-dimensional function. Using first order Taylor series expansion about some point x^* , $G(x) = 0$ can be linearized. The $G(x) = 0$ at some point x^* can be simplified to $G_L(x) = 0$ (Figure 2.23).

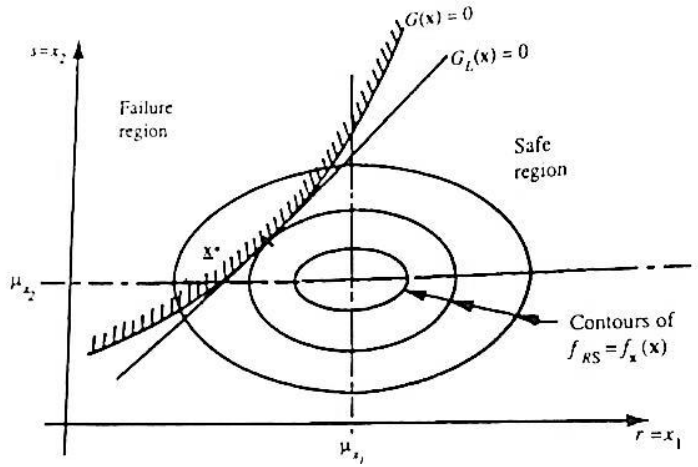


Figure 2.25: Joint Probability Density Function in Original Space (Melchers, 1999)

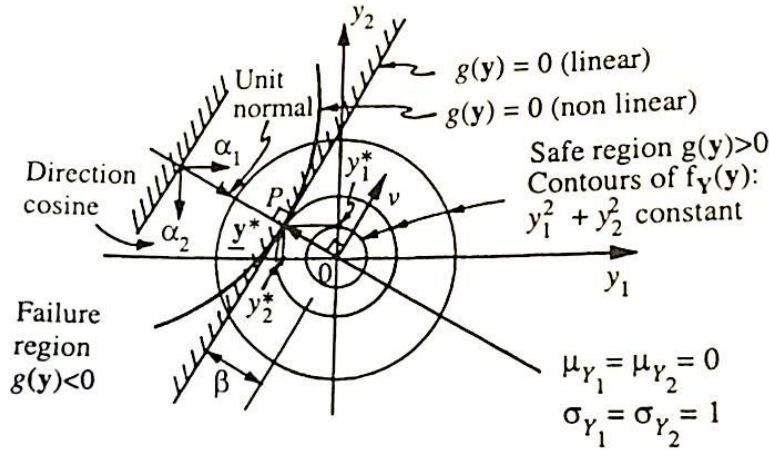


Figure 2.26: Joint Probability Density Function in Transformed Space (Melchers, 1999)

The distance β in Figure 2.26 corresponds to the shortest distance from the origin in the y space to the limit state surface $g(y) = 0$. The distance β is given by:

$$\beta = \min(\sum_{i=1}^n y_i^2)^{1/2} \quad (2.9)$$

where y_i represents the coordinates of any point on the limit state surface. The point for which Equation 2.9 is satisfied is called the “checking or design point y^* ”.

When both resistance R and load effect S are each normal random variables the limit state equation is the safety margin ($Z = R-S$) and the probability of failure is:

$$p_f = \phi(-\beta) \quad (2.10)$$

$$\beta = \frac{\mu_z}{\sigma_z} \quad (2.11)$$

where β represents the safety or reliability index and $\phi ()$ is the standard normal distribution function. β is a direct measure of the safety of the structural element; greater β represents higher safety or lower nominal failure probability.

2.5.3 Load Effect Modelling

There are two types of loads that a structure needs to withstand: natural phenomena and man-imposed. The loads that occur due to natural occurrences are wind, wave, snow and earthquake, while man-imposed loads include dead and live loads.

As discussed in the previous section, the magnitude of most loads varies with time and with location. This means that, at a given point in time, the actual loading is uncertain. Therefore, perfect models are not possible due to the uncertainty of loads. Instead, approximate models are developed.

The statistical properties of the load can be estimated for the load due to natural phenomena if previous data are available over a period of time. If instantaneous records are available then a complete, time-dependent reliability analysis may be possible by estimating the probability density function. On the other hand, for a man-imposed load, it is rare to see sufficient long-term data to form a probabilistic model.

2.5.4 Resistance Modelling

There are many uncertainties associated with the strength properties of the structure that can affect the resistance. Uncertainties associated with reinforcing steel and concrete according to literature are stated below.

2.5.4.1 Reinforcing Steel

According to Sobrino (1993), factors that affect the mechanical behavior of reinforcing steel bars are:

1. Variability of the material strength (which depends on the material itself, fabrication process, manufacturer, etc.)
2. Variability of section's geometry
3. Material degradation (e.g. corrosion)
4. Load history (fatigue phenomena)
5. The method of definition of conventional strength parameters (yield and ultimate), and their experimental evaluation (type of test, velocity of the load application, etc.).

A vast amount of research has been performed in developing a probability model for reinforcing steel, specifically by Mirza and MacGregor (1979). Most researchers have focused on the yield and ultimate strengths; by knowing these properties it is possible to define the steel behavior expressed by the stress and strain curve. For the probability distribution, it is recommended to use the same type of distribution for both the ultimate and yield strengths. Most researchers recommend using log-normal distribution (Sobrino, 1993).

2.5.4.2 Properties of Concrete

The uncertainties related to concrete are of great interest, but they have little influence on the structure compared to reinforcement properties. This is due to a higher concern for structure ductility in the design process.

The heterogeneous composition of concrete causes more variability in the mechanical properties of concrete. According to Dawid et al. (2012), the uncertainties of mechanical properties depend mostly on the following factors:

1. Material properties of cement, aggregates, etc.
2. Concrete composition (water, cement ratio, cement quantity, etc.)
3. Execution (mixing, transporting, placing, curing and hardening, etc.)
4. Testing procedure (type and dimensions of specimens, velocity of the load application, etc.).
5. Concrete being in the structure rather than in a control specimen (in-place concrete may not be well represented by test cylinders or cubes due to differences in consolidation and/or curing conditions).
6. Maintenance, material degradation, etc.

The factor that is analyzed the most in concrete probabilistic models is compressive strength (f'_c). The others factors, such as tensile strength, modulus of elasticity, and ultimate strength are usually defined by empirical relations (EC-2, 2004). Compressive strength is the main parameter for quality control of concrete throughout the life of the structure. It is also an important parameter in the building code for acceptance and rejection criteria.

Since 1970, researchers have been developing and improving a probabilistic model for compressive strength of unconfined concrete. The model that is used the most by researchers to

analyze f'_c is based on the studies by Ellingwood (1980), and Mirza and MacGregor (1982) as presented in Table 2.5.

Table 2.5: Statistical properties by Ellingwood et al (1980), and Mirza and MacGregor (1982)

Load type	Mean/nominal	Covariance	Probability distribution
Dead load	1.05	0.10	Normal
Live load			
Point-in-time load	0.273	0.674	Weibull
Transformation to load effect	1.00	0.206	Normal
Wind Load			
Point-in-time velocity	0.0804	1.001	Weibull
Transformation to load effect	0.68	0.22	Lognormal
Concrete compressive strength, f'_c	1.16	0.14	Normal
Steel yield strength, f_y	1.115	0.064	Lognormal
Modulus of elasticity of reinforcing steel, E	1.0		Deterministic
Total area of reinforcement	1.01	0.04	Shifted lognormal*

2.5.5 Application of Reliability Analysis in Reinforced Concrete Structures

Lu et al. (1994) evaluated the reliability of reinforced concrete beams designed under the provisions of the ACI building code (ACI, 1989). To examine the reliability of beams, modern full-distribution reliability methods and refined mechanical models were used. Reliability analysis was conducted on a typical spandrel beam and an interior beam of the continuously-supported, one-way floor system represented in Figure 2.27.

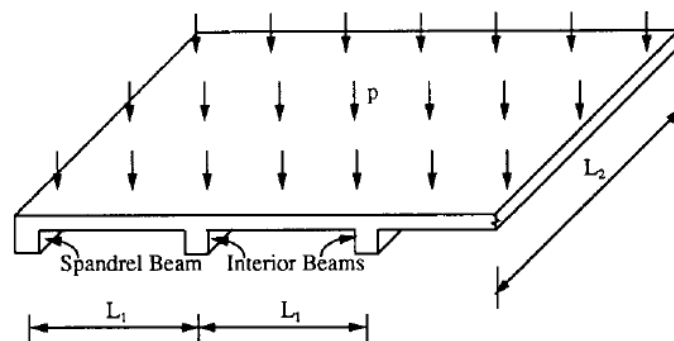


Figure 2.27: One-way and Beam System (Lu et al., 1994)

The spandrel beam was subjected to bending moment, shear force and torque; while the interior beam was subjected only to bending moment and shear. The total load per unit length, moment, shear and torque were calculated using Equations 2.12 and 2.13. These loads should be less than the resistances as required by the ACI code (1989) and expressed by Equations 2.14, 2.15 and 2.16 for moment, shear and torsion, respectively.

$$q = \frac{1}{2}pL_1 + \gamma_D whb_w \text{ (Spandrel beam)} \quad (2.12)$$

$$q = pL_1 + \gamma_D whb_w \text{ (Interior beam)} \quad (2.13)$$

$$M_u = \frac{1}{12}qL_2^2 \leq \phi \rho f_y b_w d^2 \left(1 - \frac{\rho f_y}{2(0.85)f'_c}\right) \quad (2.14)$$

$$V_u = \frac{1}{2}qL_2 \leq \phi \left(2\sqrt{f'_c} b_w d + f_y d \frac{A_v}{s}\right) \quad (2.15)$$

$$T_u = \frac{1}{2}mL_2 \leq \phi \left(0.8\sqrt{f'_c} x^2 + \alpha_t x_1 y_1 f_y \frac{A_t}{s}\right) \quad (2.16)$$

The parameters are defined in Table 2.6.

For the combined effect of shear and torsion for the spandrel beam, the following expressions were considered:

$$T_u \leq \phi \left(\frac{0.8\sqrt{f'_c} x^2 y}{\sqrt{1 + \left(\frac{0.4V_u}{C_t T_u}\right)^2}} + \alpha_t x_1 y_1 f_y \frac{A_t}{s} \right) \quad (2.17)$$

$$V_u \leq \phi \left(\frac{2.0\sqrt{f'_c} b_w d}{\sqrt{1 + \left(\frac{2.5 C_t T_u}{V_u}\right)^2}} + f_y d \frac{A_v}{s} \right) \quad (2.18)$$

where h and b_w represent the height and width of the beam, and m is the distributed torque along the beam, which is equal to $\frac{1}{24}mL_1^2$, as suggested by the ACI code (1989).

The structural reliability of the reinforced concrete beam as a system and for the individual components was considered in this study. The multiple failures in flexure, shear and torsion were determined by representing the beams as a series system. The reliability analysis was conducted using First Order Reliability Method (FORM) and Second Order Reliability Method (SORM) by using CALREL software. The basic random variables and their marginal distribution were based on the study by Mirza and MacGregor (1979) are stated in Table 2.6.

Table 2.6: Description of Random Variables (Lu, 1994)

X_i	Description	Distribution	Mean	COV
h_s	Slab thickness	Normal	Nominal	0.070
h	Beam height	Normal	Nominal	0.010
d	Effective beam depth	Normal	Nominal	0.020
b_w	Beam width	Normal	Nominal	0.020
$A_{\sim}$	Tension reinforcement area	Normal	Nominal	0.040
$A_{v/s}$	Shear stirrup area per unit length	Normal	Nominal	0.040
$A_{t/s}$	Torsional stirrup area per unit length	Normal	Nominal	0.040
f_y	Yield strength of steel	Beta	48.8/71.0 ksi	0.107/0.093
f'_c	Compressive strength of concrete	Normal	3.125/ 3.800ksi	0.180
C	Measure of concrete splitting strength	Normal	Nominal	0.180
L	Live load intensity	Type I largest	Eq. (16)	0.250

x_1	Short center-to-center dimension of stirrup	Normal	Nominal ~	0.100
y_1	Long center-to-center dimension of stirrup	Normal	Nominal ~	0.060
B_f	Flexure model uncertainty	Normal	1.10	0.120
B_v	Shear model uncertainty (ACI model)	Normal	1.20	0.112
B'_v	Shear model uncertainty (Zsutty's model)	Normal	1.09	0.120
B_t	Torsion model uncertainty	Normal	1.03	0.063
B_{vt}	Shear-torsion interaction model uncertainty	Normal	1.00	0.060

The critical limit state function for flexure was considered for the beam when moderately reinforced as defined by Equation 2.19. For shear, Zsutty's model stated in Equation 2.20 was found to be more accurate for predicting the shear capacity than the ACI model according to previous literature (Lu et al., 1994). Equation 2.21 and Equation 2.22 are the limit state functions for torsion and combined shear-torsion.

$$g1 = B_f A_s f_y \left(d - \frac{A_s f_y}{1.7 f'_c b_w} \right) - M \quad (2.19)$$

$$g2 = B_v 10 \sqrt{f'_c} b_w d - V \quad (2.20)$$

$$g3 = B_t (4.0 \sqrt{f'_c} x^2 y) - T \quad (2.21)$$

$$g4 = B_{vt} \left(\frac{A_t}{s} + \frac{1}{2} \frac{A_v}{s} \right) - \frac{A_t^{req}}{s} - \frac{1}{2} \frac{A_v^{req}}{s} \quad (2.22)$$

The random variable parameters of Equations 2.19-2.22 are defined in Table 2.6 above. The component reliabilities indices for flexure, shear, and torsion in terms of live-to-dead ratio were computed. Two different material strengths were considered in the reliability analysis, $f_y = 40$ ksi

and 60 ksi, and $f'_c = 3$ ksi and 4 ksi, respectively. The reliability index for flexure and shear are presented in Figure 2.28. Figure 2.29 provides reliability analysis for torsion and combined shear and torsion. Comparison for different failure modes are provided in Figure 2.30.

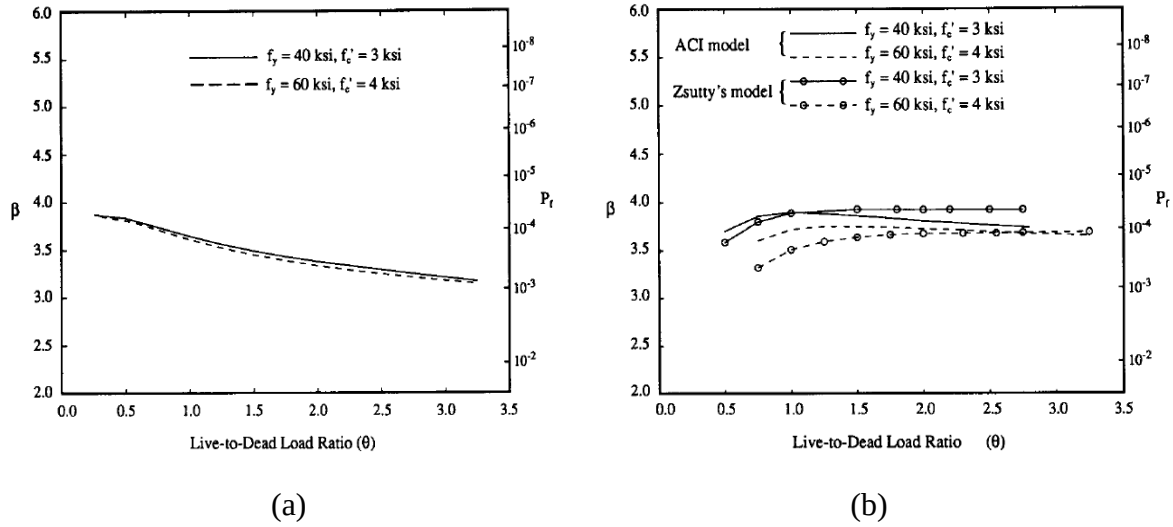


Figure 2.28: Reliability Index vs. Live-to-Dead Load Ratio for: (a) Flexure; and (b) Shear (Lu, 1994)

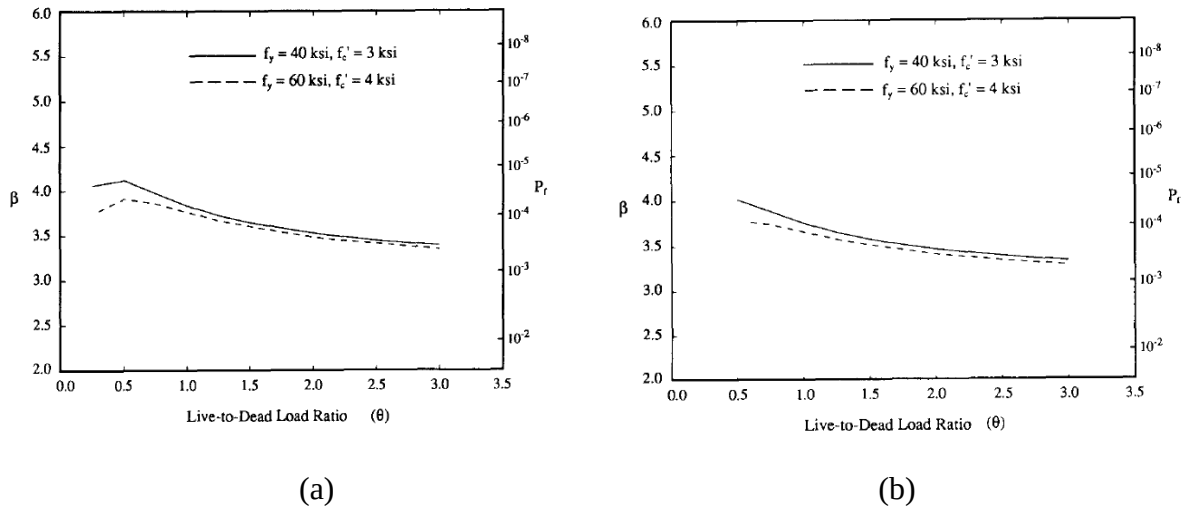


Figure 2.29: Reliability Index vs. Live-to-Dead Load Ratio: (a) Torsion; and (b) Shear-Torsion (Lu, 1994)

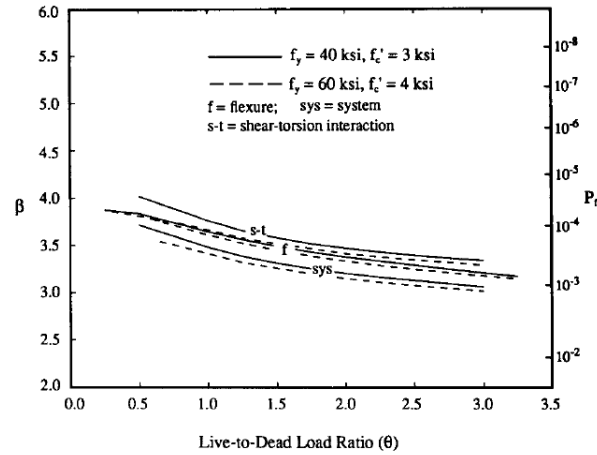


Figure 2.30: Comparison of reliability for various failure modes (Lu, 1994)

From the reliability analysis, the following conclusions and remarks were made:

- 1) For all the failure modes, the reliability index was confined to a comparatively slim band and it varies slowly over a large variety of design parameters.
- 2) The components and system reliability indices were most sensitive to live load, material strengths and model uncertainty.
- 3) Designs using lower strength steel and concrete had a slightly larger reliability compared to designs adopting higher strength materials.
- 4) The flexure failure mode was found to regulate the reliability of the beam in the case of large live-to-dead ratios.
- 5) The reliability index corresponding to the shear-torsion failure mode was most insensitive to the total lateral reinforcement ratio and to the torsion-to-shear reinforcement ratio.

Zou and Hong (2011), through reliability analysis, calibrated the concrete resistance factor ϕ_c for FRP-confined RC columns. Fiber-reinforced polymer (FRP) jackets confine concrete resulting in a significant increase in strength and ductility of unconfined concrete. Currently, FRPs are being employed in existing structures for rehabilitation purposes. The effectiveness of FRP is due to its

unique material and chemical properties, high tensile strength, and corrosion resistance and durability. Materials such as reinforcing steel, FRP, and concrete need to be considered when analyzing FRP confined columns. These three materials behave differently under applied load; their stress-strain relationship must be considered when evaluating the capacity of the columns. One of the key issues in modeling the stress-strain relationship of FRP-confined concrete is to have an accurate definition of the ultimate state condition, which depends highly on the availability of test results, the measured hoop tensile strain of confining FRP jackets, and accurate and standardized tensile characterizations of the FRP materials (Lam and Teng, 2002; De Lorenzis and Tepfers, 2003; and Teng and Lam, 2004).

The probabilistic models used in this study for the compressive strength of unconfined concrete, the yield strength of reinforcing steel, the total reinforcement area, and column cross-sectional dimensions were all based on studies by Ellingwood (1980), and Mirza and MacGregor (1982). The dimensions of the column cross-section, as well as the elastic modulus of the reinforcing steel, were treated as deterministic variables due to their effect on the estimated reliability, which is insignificant (Zou and Hong, 2011). The probabilistic model for the FRP jacket (f_{FU}) was based on the experimental results reported previously in the literature.

According to CSA S806-02 (Design and Construction of Building Components with Fiber-Reinforced Polymers, 2002), building components with FRP shall be designed in such a way that their factored resistance is greater than or equal to the effects of factored loads. The capacity of reinforced concrete columns can be defined by an interaction diagram with the vertical axis representing the axial load capacity P_r and the horizontal axis representing the bending moment capacity M_r . For short, RC column, P_r and M_r can be evaluated using the equivalent rectangular stress block method (MacGregor, 1997).

The factored load was calculated as specified in the NBCC (2005):

$$S_{DT} = \alpha_D D_n + \alpha_L L_n + \alpha_W W_n \quad (2.23)$$

where α_D , α_L and α_W represent the load factors for dead, live and wind loads, respectively. D_n , L_n , and W_n are the specified nominal loads. To develop the limit state function, X_r denoted the vector of random variables of column resistance and X_s denoted the vector of random variables representing the total load effects. Then, the limit state function $g(X_r, X_s, \text{ and } \xi_c)$ was defined as (Hong and Zhou, 1999):

$$g(X_R, X_S, \xi_c) = \xi_c \sqrt{(P_R)^2 + \left(\frac{M_R}{h}\right)^2} - \sqrt{(A_D + A_L + A_W)^2 + \left(\frac{M_D + M_L + M_W}{h}\right)^2} \quad (2.24)$$

where ξ_c is the modeling error; A_D , A_L , and A_W represent the axial loads due to dead, live and wind, respectively; and M_D , M_L and M_W denote the bending moments due to dead, live and wind, respectively. Failure occurs when the limit state function reaches zero; in other words, when the load is greater than the resistance. After the limit state function was established, the following algorithm was used to evaluate the reliability of the FRP column:

- 1) Calculate factored design axial load (A_{deg}) and factored design bending moment (M_{deg}) for the given design load eccentricity (e_{dge}) using the ERSB (Equivalent Rectangular Stress Block) recommended in CSAS806-02 (2002) or CSA A23.3-04 (2004).

- 2) Calculate nominal axial forces (A_{Dn}) and nominal bending moments (M_{Dn}) due to dead load using Equations 2.25 and 2.26, respectively. Use the given ratios α_D , α_L and α_w to determine nominal axial force due to live load (A_{Ln}), nominal axial force due to wind load (A_{Wn}), nominal bending moment due to live load (M_{Ln}), and nominal bending moment due to wind load (M_{Wn}), by equating the ratios (i.e. $A_{Wn}/A_{Dn} = M_{Wn}/M_{Dn} = \alpha_w$).

$$A_{Dn} = A_{deg} / \max((1.25 + 1.5\alpha_L + 0.4\alpha_w), (1.25 + 0.5\alpha_L + 1.4\alpha_w)) \quad (2.25)$$

$$M_{Dn} = e_{deg} A_{Dn} \quad (2.26)$$

3) Calculate the mean value of the random variables using the provided nominal values and statistics.

4) Conduct reliability analysis using the FORM limit state function stated in Equation 2.24.

Reliability analysis was conducted on a circular column with a diameter of 450 mm. Different values of reinforcement ratio ρ_s (1.5%, 2.5% and 3.5%), dead to live load ratio (α_L), and dead to wind load ratio (α_W) were considered.

The following conclusions were drawn from the results:

- 1) The resistance factor ϕ_c was equal to 0.65, leading to higher reliability than the 50-year target reliability index of 3.0, which was used to select factored design loads in the NBCC (2005).
- 2) The reliability index was higher for small load eccentricities compared to larger eccentricities.
- 3) The reliability index was insensitive to the reinforcement ratio and the layers of FRP wrapping.
- 4) For an efficient design, it is recommended to use ϕ_c equal to 0.75 for FRP columns to achieve the 50-year target reliability index of about 3.5.

Wang and Hong (2002) conducted a reliability assessment of a concrete column under axial load and biaxial bending. In this study, the reliability of columns designed according to the current design code was compared with the reliability indices obtained for the axial load and uniaxial bending.

The same algorithm, statistics, and limit state function used by Zou and Hong (2011), as mentioned above were also applied for the reliability analysis in this study. However, after applying the ERSB for eccentricity in each direction, the author used the reciprocal load method (Bresler, 1960) provided in Equation 2.27 to calculate the strength of the column under biaxial bending. For the

limit state function, the moment component is split into x and y directions as shown in Equation 2.28.

$$1/P_A = 1/P_{nx} + 1/P_{ny} - 1/P_u \quad (2.27)$$

$$g(X_R, X_S, \xi_c) = \xi_c \sqrt{(P_R)^2 + \left(\frac{M_x}{h}\right)^2 + \left(\frac{M_y}{b}\right)^2} - \sqrt{(A_D + A_L)^2 + \left(\frac{(M_{xD} + M_{xL})}{h}\right)^2 + \left(\frac{(M_{yD} + M_{yL})}{h}\right)^2} \quad (2.28)$$

For the reliability analysis, a square and rectangular section with different reinforcement ratio as illustrated in Figure 2.31 and Table 2.7 were studied.

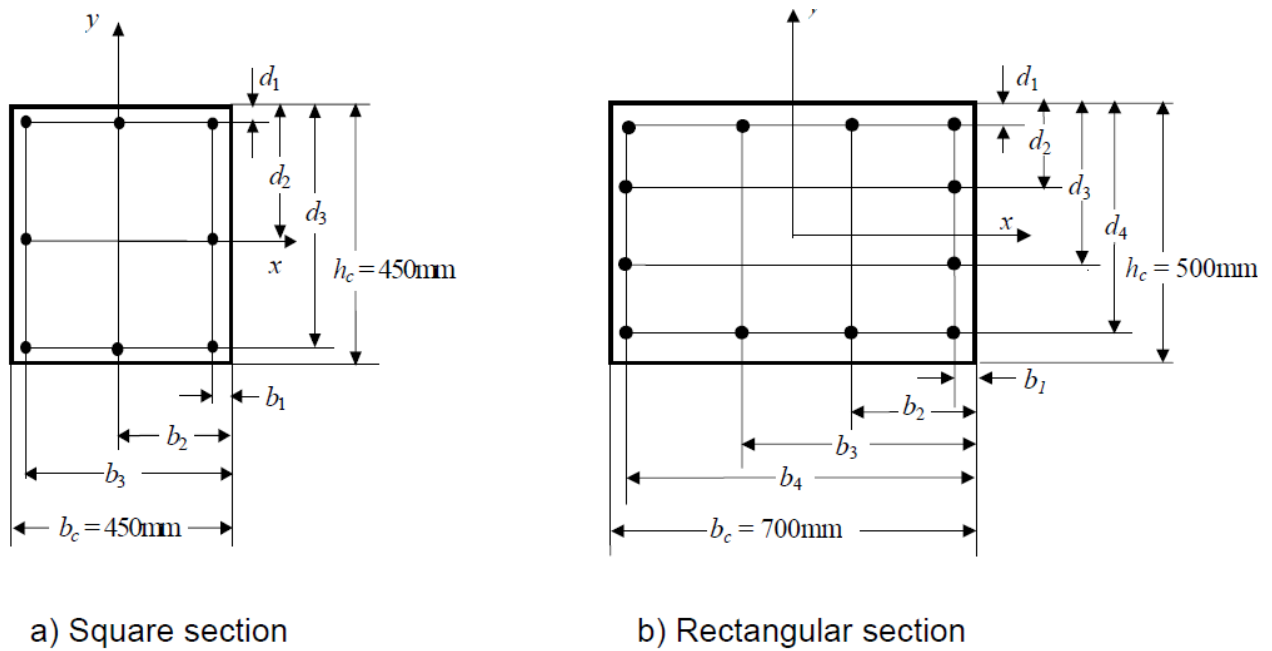


Figure 2.31: Cross Section of Reinforced Concrete Column: (a) Square; and (b) Rectangular (Wang and Hong, 2002)

Table 2.7: Values of Material Properties and Geometry Characteristics of the Column (Wang and Hong, 2002)

Variable	Value
Concrete compressive strength, f_c'	30, 60 or 90 Mpa
Steel yield strength, f_y	400 Mpa
Modulus of elasticity of steel, E	2.01×10^5 Mpa
Reinforcement ratio ρ_s (d_i and b_i , $i=1,2,3$, are in mm)	
Figure 2a (square section)	1.2% (8No. 20, $d_1=60$, $d_2=225$, $d_3=390$, $b_1=60$, $b_2=225$, $b_3=390$); 2.0% (8No. 25, $d_1=62.5$, $d_2=225$, $d_3=387.5$, $b_1=62.5$, $b_2=225$, $b_3=387.5$); 2.8% (8 No. 30, $d_1=65$, $d_2=225$, $d_3=385$, $b_1=65$, $b_2=225$, $b_3=385$)
Figure 2b (rectangular section)	1.3% (8No. 20 + 4No. 25, $d_1=52.5$, $d_2=150$, $d_3=350$, $d_4=447.5$, $b_1=52.5$, $b_2=200$, $b_3=500$, $b_4=647.5$); 2.2% (4No. 25 + 8No. 30, $d_1=55$, $d_2=150$, $d_3=350$, $d_4=445$, $b_1=55$, $b_2=200$, $b_3=500$, $b_4=645$); 3.3% (4No. 35 + 8No. 30, $d_1=57.5$, $d_2=150$, $d_3=350$, $d_4=442.5$, $b_1=57.5$, $b_2=200$, $b_3=500$, $b_4=642.5$);

From the results, the following conclusions were reached:

- 1) The reliability levels for biaxial bending vary in a wide range compared to that of uniaxial bending.
- 2) The reliability indices were insensitive to the reinforcement ratio.
- 3) Beta increased as the permanent load and compressive strength increases.
- 4) The CSA design code is adequate for designing RC columns under axial load and biaxial bending. However, for some cases, the code can be very conservative for biaxial bending.

2.6 Summary

This literature review was divided into four parts: (1) different materials/techniques for repairing damaged shear walls and their impact on structural characteristics; (2) influence on the cyclic response of shape memory alloys due to material composition of SMA and due to different types of loading (static and dynamic); (3) applications of shape memory alloy in reinforced concrete structures; and (4) basic concepts of structural reliability with examples related to reinforced concrete structures.

Repairing shear walls by replacing damaged concrete and reinforcement leads to almost full recovery of strength and deformation capacities as stated by Fiorato et al. (1983). The results were also confirmed by Leaf and Kotsovos (1990) and Vecchio et al. (2002). These two studies highlighted that repairing the damaged regions of the compressive zone was sufficient to restore structural characteristics to a reasonable level. However, there was no literature presently available that studied the performance of hybrid-SMA reinforced shear walls, repaired by replacing damaged reinforcement, and by replacing heavily damaged concrete with high strength self-consolidating concrete.

The properties of SMA have been widely studied by numerous researchers. Different material compositions SMA and type of loading can highly effect both forward and reverse transformations. This current research also investigates the effect of cyclic response due to length of the SMA reinforcement present in hybrid-SMA wall.

The use of SMAs in moment resisting reinforced concrete components leads to low residual deformations of the overall structure after cyclic loading. This was concluded by studying the literature on SMA as reinforcement in the critical regions of moment resisting components. Youssef et al. (2008) investigated a moment resisting frame with SMA as reinforcement at the beam-column joint. Saiidi and Wang (2006) investigated the application of shape memory alloys as reinforcement in columns. Abdulridha and Palermo (2014) conducted an experimental study using shape memory alloys as reinforcement for ductile reinforced concrete shear walls. All these authors came to the same conclusion, that reinforced concrete members reinforced with SMAs provided significant strength with almost full recovery of displacements. This leads to an assumption that hybrid-SMA structures can be serviceable after minimum repair due to mitigation

of residual displacements. However, experimental testing is required to ensure repaired, hybrid-SMA reinforced structural components have reasonable structural characteristics after repair.

Safety of a structure is violated when the resistance is less than the applied loads. Uncertainties are associated with both the resistance and loads. Without considering uncertainties in the design, a structure can have a high probability of failure due to variations in resistance and loading over time. To address this, safety factors are introduced in the design process which incorporates uncertainty. Safety factors are derived by transforming experimental data and site observations into statistical parameters. Extensive research has been focused to develop safety factors for reinforced concrete structures, which not only results in an economical design but includes margins for uncertainties. SMA is a new material in Civil Engineering, which requires research to establish safety factors before they can be used in the design of real structures.

In the current research, hybrid-SMA reinforced shear wall is repaired by replacing damage concrete and fractured and buckled reinforcing steel. Reliability analysis is conducted for failure in flexure mode for walls before and after repair. Also a framework is provided to establish statistical moments from the experimental data.

CHAPTER 3

EXPERIMENTAL PROCEDURE

3.1 Introduction

To examine the applicability of hybrid SMA-steel reinforced concrete structures, two shear walls were previously tested to compare the behavior of an SMA-steel hybrid shear wall to a wall reinforced with conventional steel only. After completion of the original tests, both walls were repaired and retested in this research study to compare the behavior of the repaired shear walls. This chapter describes the details of the original walls, damage sustained in the original testing of the walls, and the repairing strategy for the walls.

3.2 Original Walls

Two ductile shear walls were constructed and tested under reverse cyclic loading by Abdulridha (2013). The walls were built on a stiff reinforced rectangular base foundation with a length of 1700 mm, height of 500 mm, and width of 1400 mm to provide full fixity to the strong floor of the laboratory. The walls contained a boundary zone at each end of the wall, and had a height-to-width ratio of 2.2. Overall, the walls had a rectangular shape with a height of 2200 mm, a length of 1000 mm and a thickness of 150 mm. In addition, a heavily reinforced top beam was provided to distribute the lateral load uniformly along the wall. The original walls were named W1-SR and W2-NR, where W represented shear wall, SR denoted the control wall reinforced with conventional steel only, and NR represented the hybrid wall with SMA reinforcement in the boundary regions. For the current research, the walls were renamed RW1-SR and RW2-NR, where the leading R denotes “repaired”. Figure 3.1 provides the dimensional details of the original and repaired walls.

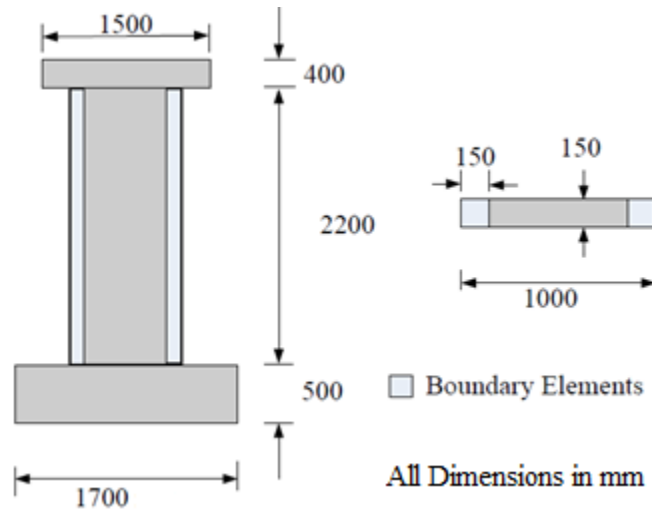


Figure 3.1: Dimensions of Test Shear Walls (Abdulridha, 2013)

The walls were reinforced with two layers of orthogonal steel reinforcement in the web region. The vertical and horizontal reinforcement in the web were spaced uniformly at 150 mm in both directions, leading to reinforcement ratios of 0.88%. The vertical reinforcement in the web consisted of three (3) pairs of 10M deformed bars, while the horizontal reinforcement consisted of 15 pairs of 10M deformed bars. To prevent buckling of the vertical reinforcement in the end boundary zones, 10M closed ties were spaced at 70 mm in the plastic hinge region and 100 mm above the plastic hinge region. Both walls, W1-SR and W2-NR, were identically reinforced with the exception of the plastic hinge region within the boundaries areas where the conventional steel reinforcement used in W1-SR was replaced by 12.5 mm-diameter super-elastic SMA bars in W2-NR. The Nitinol (SMA) bars were connected to conventional bars in the foundation and above the plastic region into the wall using mechanical couplers. The total length of the SMA bars was 1200 mm from center to center of the couplers. In order to prevent sliding at the interface between the wall and the foundation during testing, four (4) pairs of 10M starter bars were provided. The concrete compressive strength for W1-SR and W2-NR on their respective days of testing was 30.5

MPa and 31.6 MPa. Reinforcement details are given in Figures 3.2 and 3.3, while material properties are listed in Table 3.1.

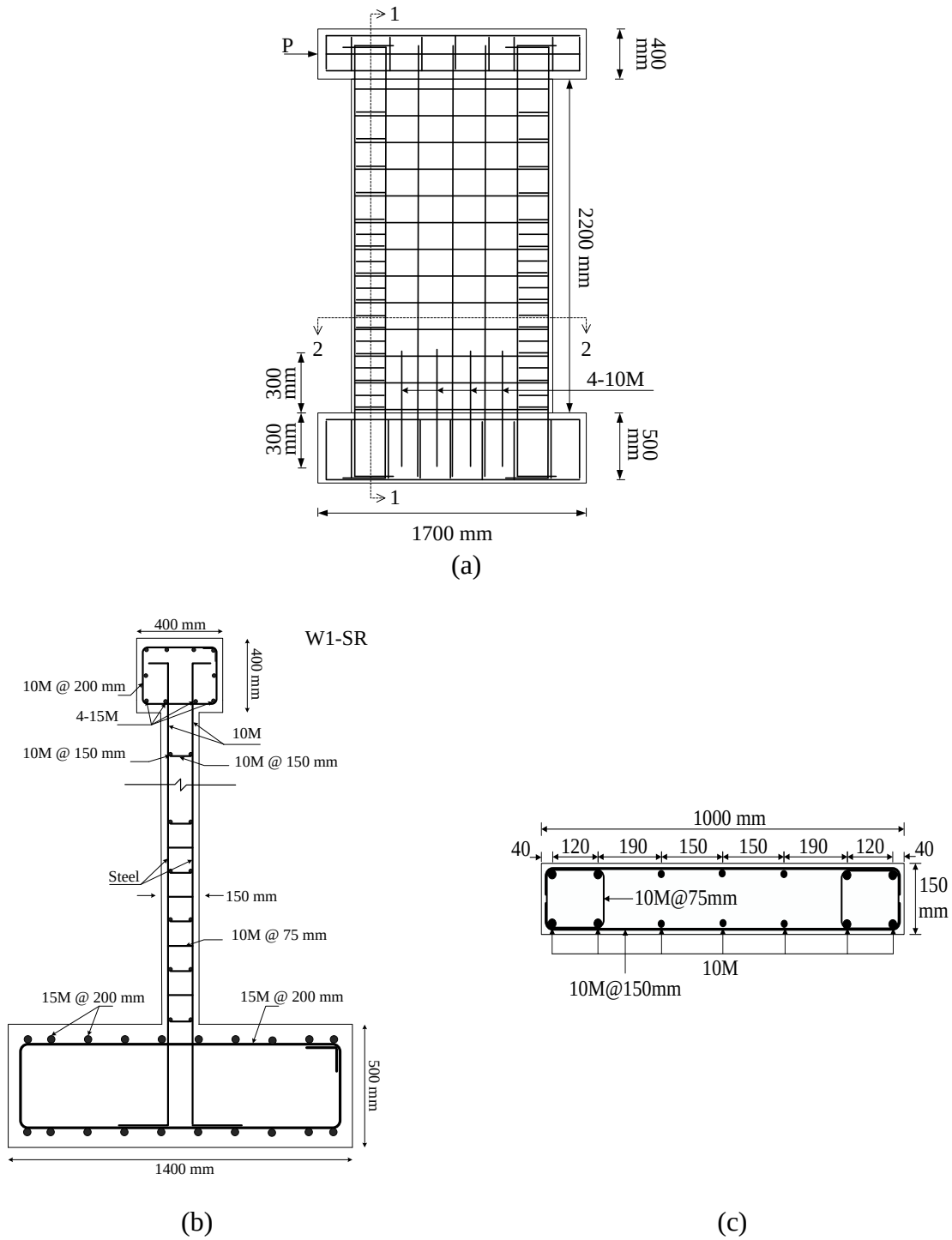
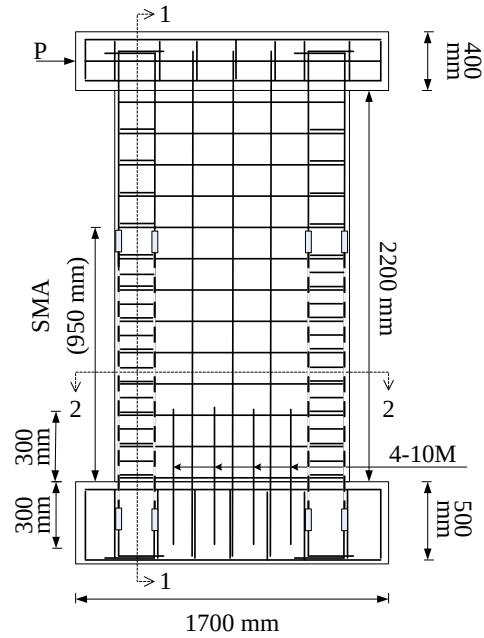
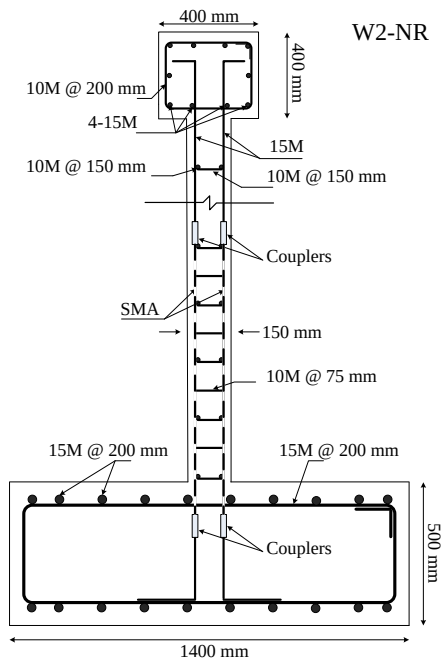


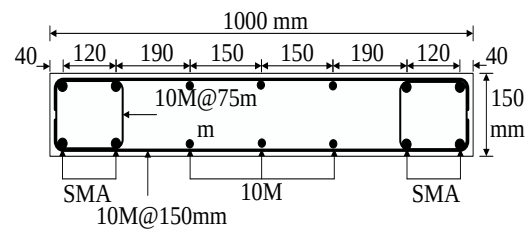
Figure 3.2: Reinforcement Details for Wall W1-SR: (a) Front View; (b) Section 1-1; and (c) Section 2-2 (Abdulridha, 2013)



(a)



(b)



(c)

Figure 3.3 Reinforcement Details for Wall W2-NR: (a) Front View; (b) Section 1-1; and (c) Section 2-2 (Abdulridha, 2013)

Table 3.1: Material Properties of Original Walls (Abdulridha, 2013)

Specimens	Concrete	Vertical Reinforcement				Transverse Reinforcement	
	f'_c (MPa)	Web		Boundary		ρ (%)	f_y (MPa)
		ρ (%)	f_y (MPa)	ρ (%)	f_y (MPa)		
W1-SR	30.5	0.88	425	1.33	425	0.88	425
W2-NR	31.6	0.88	425	1.68	380(NiTi)	0.88	425

Typical stress-strain responses of the concrete are provided in Figures 3.4 and 3.5 for W1-SR and W2-NR, respectively. Figure 3.6 illustrates the typical stress-strain response for the 10M reinforcement used by Abdulridha (2013) in both walls. The typical response of the SMA used in Wall W2-NR is provided in Figure 3.7.

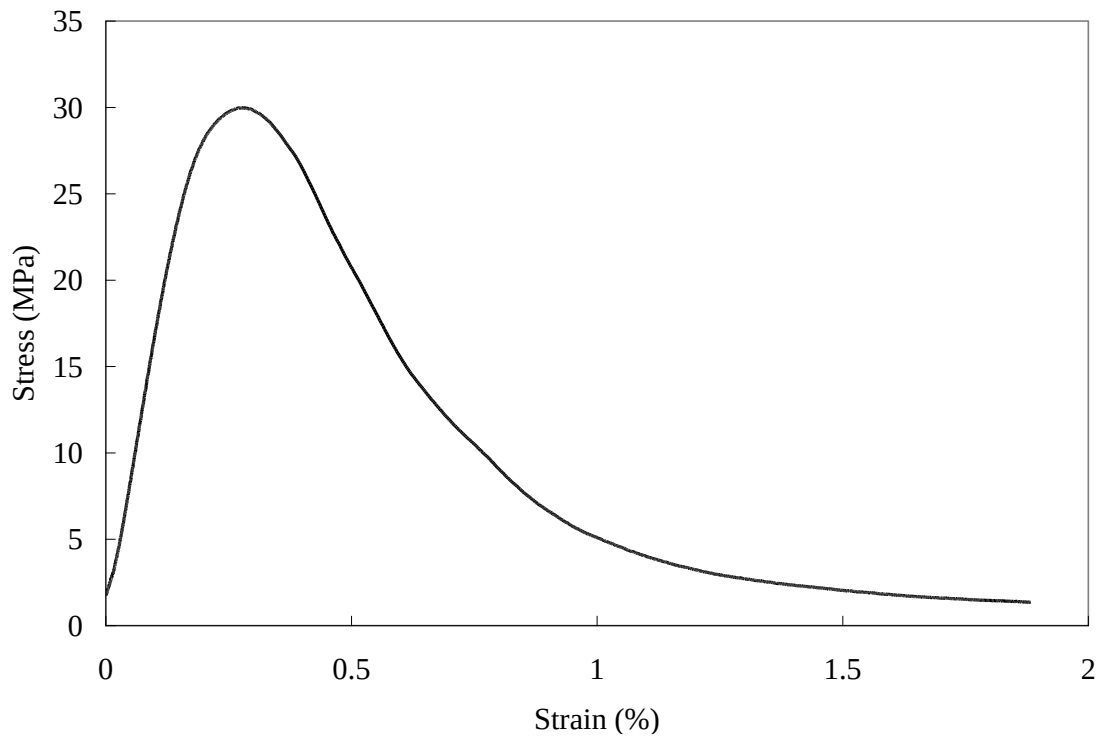


Figure 3.4: Typical Stress-Strain Response of Concrete Used in Wall W1-SR (Abdulridha, 2013)

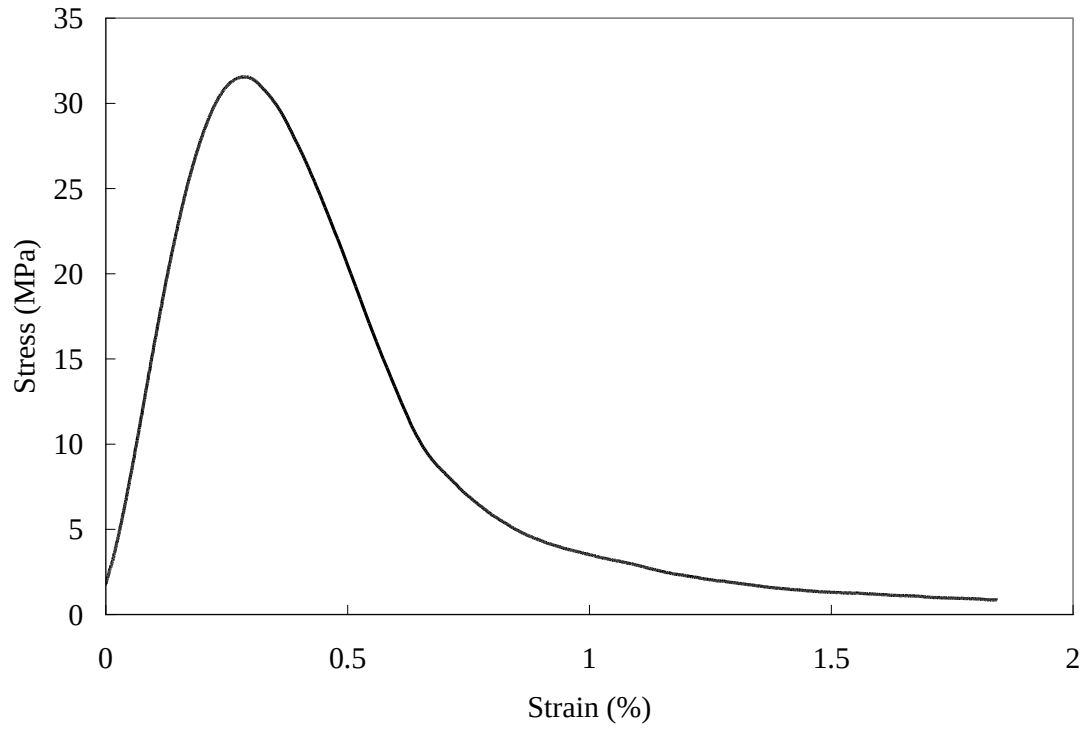


Figure 3.5: Typical Stress-Strain Response of Concrete Used in Wall W2-NR (Abdulridha, 2013)

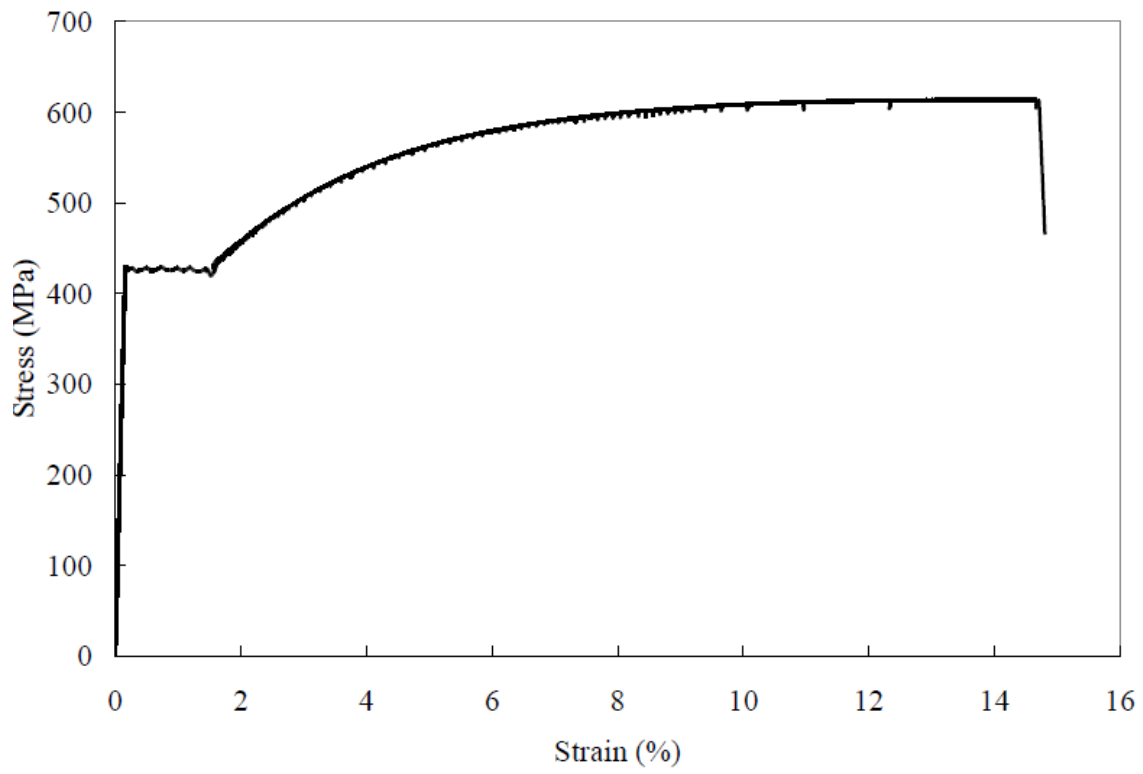


Figure 3.6: Typical Stress-Strain Response of 10M Reinforcement (Abdulridha, 2013)

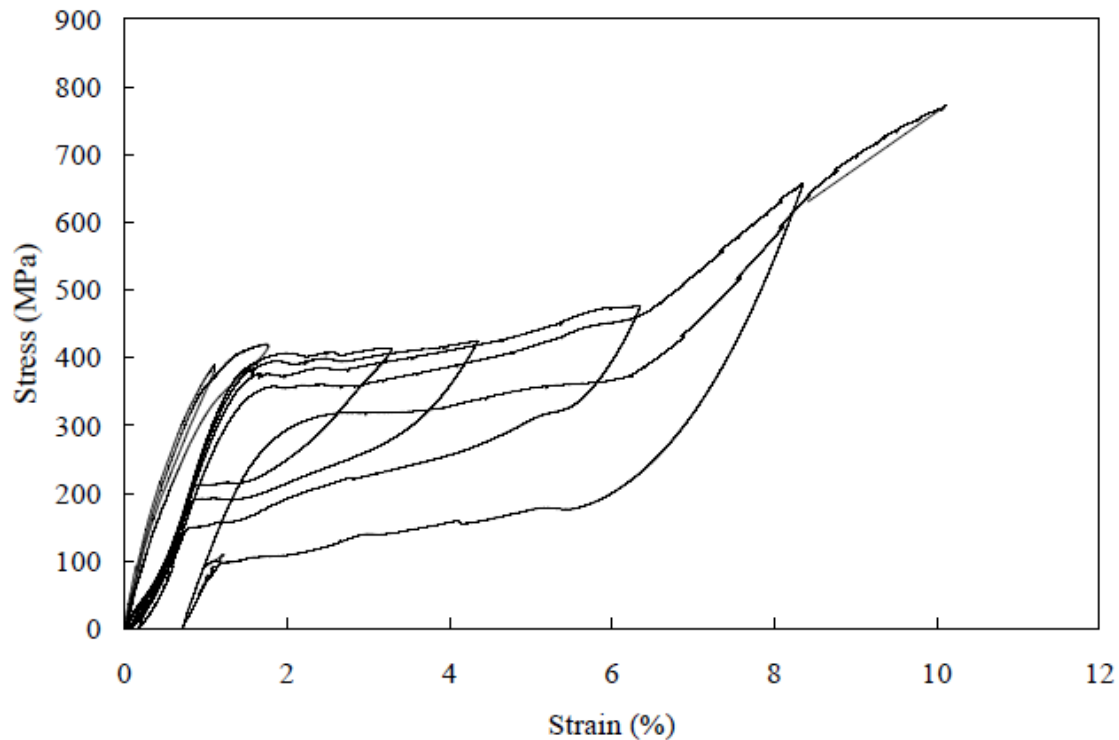


Figure 3.7: Typical Stress-Strain Response of SMA Reinforcement (Abdulridha, 2013)

3.3 Repairing Procedure

3.3.1 Preparations for Repairing

After testing of the original walls (Abdulridha, 2013), it was observed that both walls suffered extensive damage in the plastic hinge region. Wall W1-SR had several cracks spread throughout the wall; while Wall W2-NR had fewer but larger cracks due to the smooth surface of the SMA bars. Both walls experienced a major crack just above the location of the termination of the starter bars. The concrete had also spalled in both walls. Figure 3.8 illustrates the damage experienced by original walls after testing.



(a)



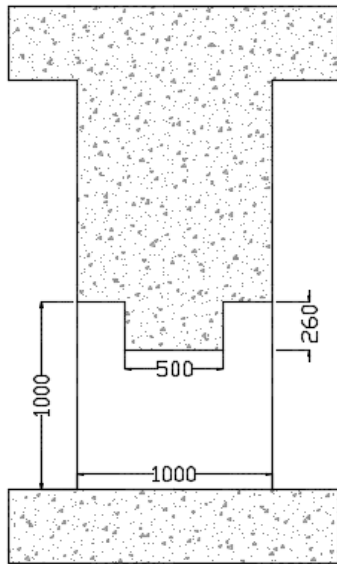
(b)

Figure 3.8: Damaged Experienced by Walls: (a) Wall RW2-NR; and (b) Wall RW1-SR

Prior to initiating the repairing process, the walls were braced using external supports (Figure 3.8 (b)). The supports consisted of two, 4" x 4" timber columns placed at the ends of the wall, and two pairs of 4" x 4" timber columns placed on both sides of the walls. To provide out-of-plane stability, four inclined studs (4" x 4") were connected to the laboratory strong floor and to the top of the wall at each corner. Thereafter, the repairing process commenced.

3.3.2 Removing Damaged Concrete

The first step in the repairing process involved removing damaged concrete in the plastic hinge region using an electric jackhammer. For Wall RW2-NR, the concrete in the boundary region was removed up to the location of the coupling between the SMA with conventional steel to examine the adequacy of the modified mechanical coupler introduced by Abdulridha (2013). The extent of the concrete removed (Figure 3.9 (a)) from both walls was similar to allow for a consistent comparison of the test results of the repaired walls.



(a)



(b)

Figure 3.9: Concrete Removal: (a) Schematic of Extent of Critical Area for Concrete Removal; and (b) Exposed Reinforcement at the Wall Base

3.3.3 State of W2-NR and W1-SR Reinforcement

After the concrete was removed from the critical area, it was evident in both walls that a number of vertical reinforcing bars were fractured and others experienced buckling. In Wall W2-NR, three web vertical bars fractured, while the others experienced buckling. This damage was concentrated

along the major crack adjacent to the termination of the starter bars. In addition, the SMA bars in the boundaries regions suffered moderate buckling along the same major crack. In general, the mechanical couplings at the top of the plastic hinge region of the boundaries zones were intact with no slippage of the reinforcement, with the exception of one where an SMA bar fractured just below the coupler. Figure 3.10 illustrates the state of the reinforcement in Wall W2-NR after the original test.

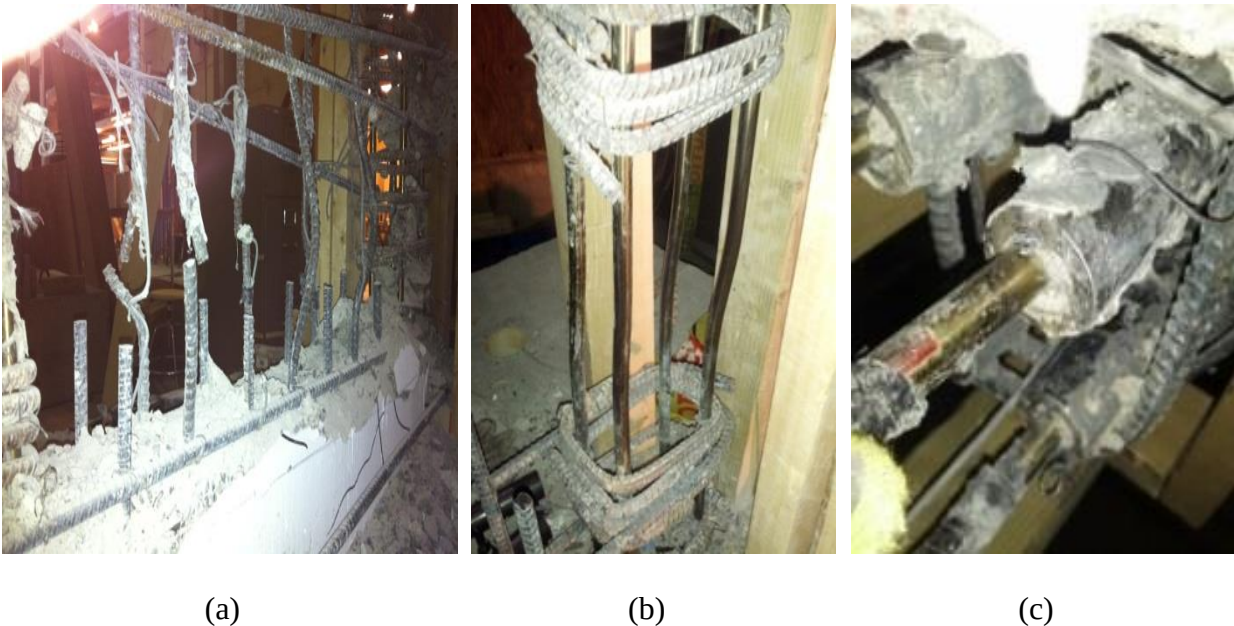


Figure 3.10: Condition of Reinforcement in W2-NR after the Original Test: (a) Fractured and Buckled Web Bars; (b) Buckled SMA Bars; and (c) Fractured SMA Bar at Coupler

The longitudinal reinforcement in the boundary zones of Wall W1-SR experienced both fracture and buckling. The left boundary zone experienced significantly more buckling than the right zone, while in the right boundary zone all the longitudinal reinforcement fractured. However, in the web only buckling of longitudinal reinforcement was evident. All the damage in the boundary zones and web was concentrated adjacent to the termination of the starter bars. Figure 3.11 illustrates the state of the reinforcement in Wall W1-SR after the original test. After the buckled and fractured

reinforcing bars were removed, the wall was vertically aligned to its original position using the overhead crane in the structures lab. Note, the SMA reinforced wall was vertically self-aligned after the original test due to self-centering phenomenon. After reinforcement in Wall RW1-SR was aligned, new reinforcing bars were added.

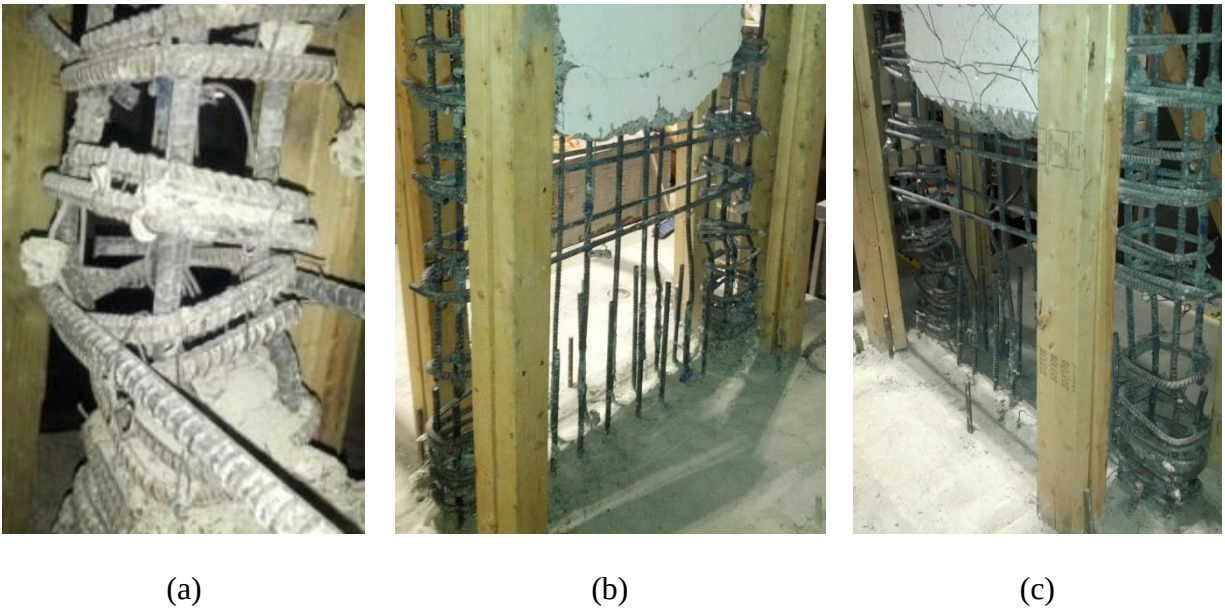


Figure 3.11: Condition of Reinforcement in W1-SR After the Original Test: (a) Buckled Reinforcement in Boundary Zone; (b) Buckled Reinforcement in Web; and (c) Fractured Reinforcement in Boundary Zone

3.3.4 Removing and Replacing Reinforcement

Once the damaged concrete was removed, fractured and buckled reinforcing bars were cut using a circular grinding saw. The starter bars that were originally provided in the web section of the wall to resist shear sliding at the foundation were also removed in both walls. The presence of starter bars resulted in higher stiffness at the base of the wall. This led to a major crack that surfaced just above the starter bars where the original failure occurred.

In Wall RW2-NR, the SMA bars were shortened from 950 mm to 424 mm from the base of the wall. The shortened length of the SMA bars led to the investigation of the effect on deformation

recovery in hybrid walls due to the length of the SMA bars. In the web, only 90 mm of the 10M reinforcing bars from the base and 115 mm from the top of the removed concrete remained, through which mechanical couplers were connected. The total length of each new segment of 10M reinforcing bar installed in the web was 540 mm. In the boundary zones, 500 mm of new segments of 15M conventional reinforcing bars replaced the length of the SMA bars that were removed. Once all the fractured and buckled reinforcing bars were replaced with new segments of reinforcing bars, the transverse reinforcement and anti-buckling ties in the boundary zones were replaced at 100 mm and 70 mm, respectively. The extent of repair of RW2-NR is provided in Figure 3.12.

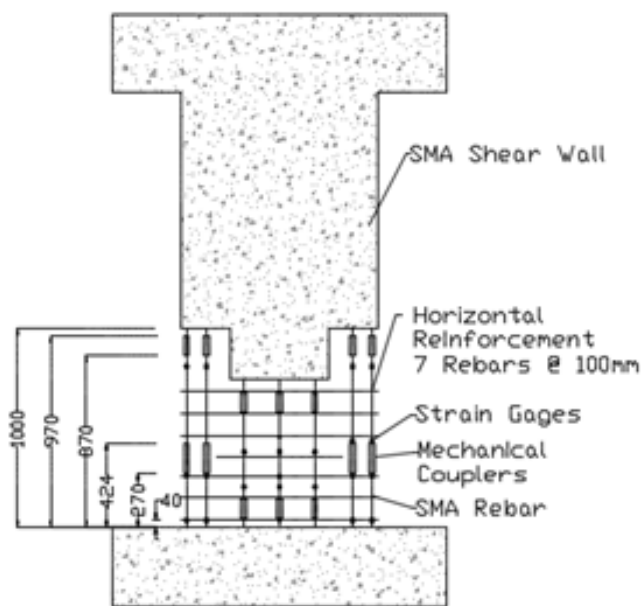


Figure 3.12: Reinforcement Layout in the Repaired Region of RW2-NR

Wall RW1-SR was repaired in a similar manner as Wall RW2-NR; fractured and buckled reinforcing bars were replaced in the boundary zones and web section of the wall. To facilitate placement of the four-inch mechanical couplers, 76 mm in length of the 10M vertical reinforcing bars remained at the base of the wall and 185 mm remained at the top from where the concrete was

removed in the web, respectively. The reinforcing bars in the web were replaced by 500 mm lengths of new segments of 10M reinforcing bars. In the boundary zones, 600 mm of fractured and buckled bars were also replaced. After the installation of the new segments of longitudinal reinforcement, the transverse reinforcement and anti-buckling ties in the boundary zones were replaced at a spacing of 100 mm and 75 mm, respectively. Figure 3.13 provides the extent of repair of RW1-SR.

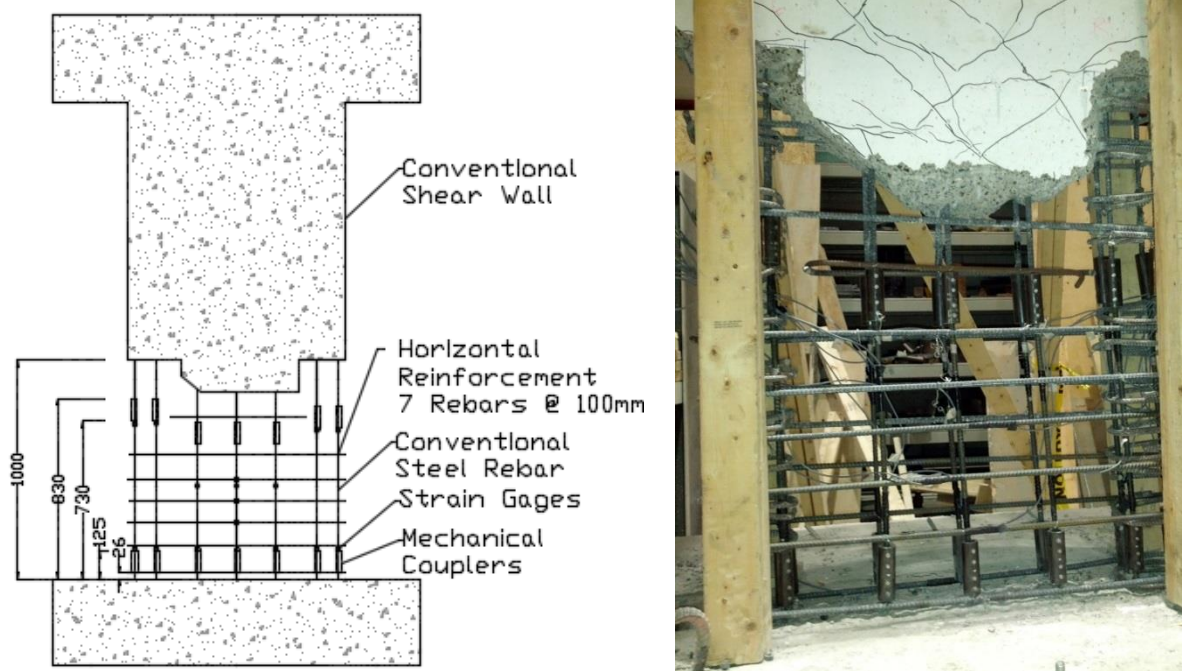


Figure 3.13: Reinforcement Layout in the Repaired Region of RW1-SR

Initially, the repair region of RW1-SR was intended to be similar to RW2-NR. However, two large inclined cracks located 1 m from the base of the wall penetrated through the thickness to the other side of the wall, exposing the reinforcing bars, as illustrated in Figure 3.14. The concrete containing this crack was also removed to ensure that stress would not develop in this damaged region during testing.



Figure 3.14: Inclined Crack in Wall RW1-SR

3.3.5 Coupling of Reinforcement

To ensure full connection with no slip between the remaining deformed reinforcement and the new segments of deformed reinforcing bars during testing, two different types of connections were tested in a 600 kN MTS universal testing machine. The first connection consisted of welding new segments of reinforcing bars to existing reinforcement in the wall to maintain uniform concrete cover. The other connection made use of a screw-lock mechanical coupler.

While testing Weld Test 1, it was noted that stress concentration developed at the front of the weld between the two reinforcing bars due to misalignment of the bars. This resulted in failure at the location of the weld before the reinforcing bars reached their ultimate strength and displacement capacities. Two other weld test trials (Weld Test 2 and Weld Test 3) included additional welding to the joint. Both trials provided satisfactory results; the reinforcing bars experienced their ultimate capacity and failure initiated at the center of the reinforcing bar. Figure 3.15 illustrates the failure locations of the weld tests.



Figure 3.15: Tensile Test Performed on Welded and Coupling Connections

The use of mechanical couplers (Figure 3.16) produced similar results to Weld Test 2 and Weld Test 3. The reinforcing bar reached its ultimate capacity without slip inside the coupler. The stress-strain responses of the weld and coupler tests are provided in Figure 3.17.



Figure 3.16: Four-inch Mechanical Coupler

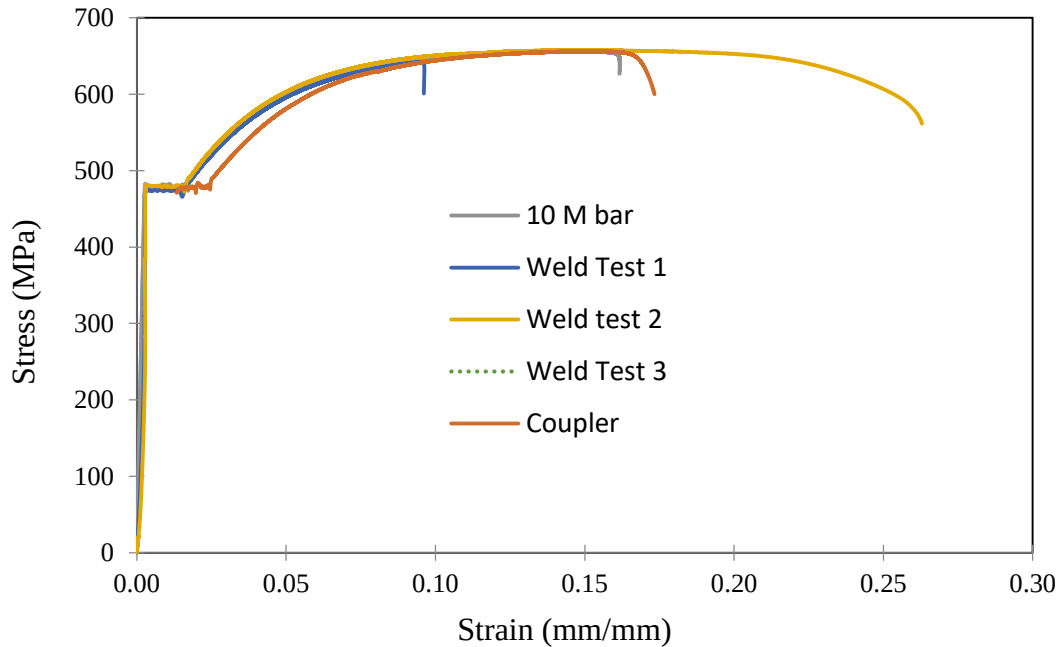


Figure 3.17: Comparison of Stress-Strain Responses of Welded and Coupling Connections

Mechanical couplers are relatively easy and simple to install; both bars are inserted inside the couplers until they reach the midpoint where a separating pin is located. Then, using an impact wrench, the bolts are tightened until the heads shear off which indicates full connectivity. The mechanical couplers ensure that reinforcing bars are aligned, eliminating stress concentrations that were evident at the joint of Weld Test 1. Furthermore, welding would be difficult to apply in congested areas inside the wall. As a result, four-inch mechanical coupler illustrated in Figure 3.16 was used to connect the deformed 10M reinforcing bars.

To connect the new segments of conventional reinforcing steel to the SMA bars that remained in the wall, the same coupling technique as that introduced by Abdulridha (2013) was applied in repairing Wall RW2-NR. The connection consisted of inserting the SMA bar through the entire length of the modified six-inch coupler. The couplers were modified by adding three additional rows of bolts, with each row consisting of six bolts. After the SMA was inserted in the couplers,

the bolts were tightened and a 15M bar was welded at the end of the coupler. Note that 15M reinforcing bars were used in the boundary zones above the SMA bars in the original wall. The intent was to ensure that yielding would initiate in the SMA in the plastic hinge region. For this study, 15M were also used above the SMA bar segments.

This connection was tested by Abdulridha (2013) in a universal testing machine (Figure 3.18) and the results demonstrated that the SMA bar reached its yield capacity and attained its full super-elastic strain range (approximately 6%).

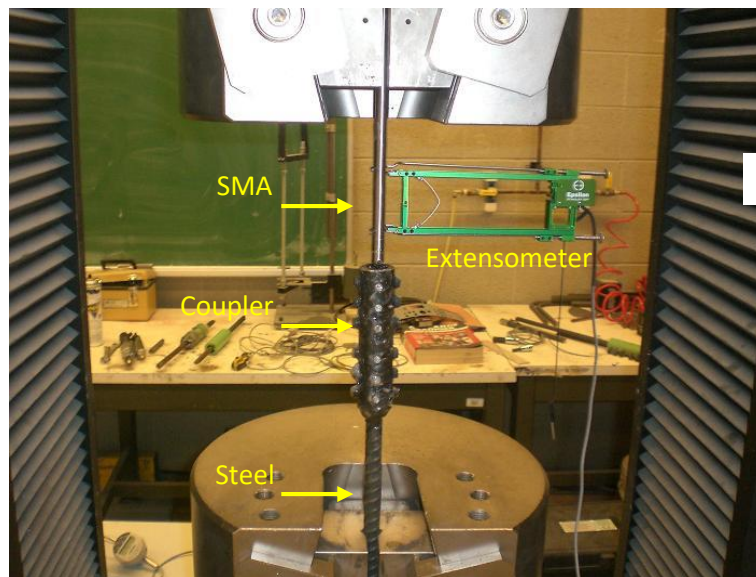


Figure 3.18: Testing of SMA Coupling Connection with Conventional Steel (Abdulridha, 2013)

3.3.6 Formwork, Casting and Curing

Before erecting the formwork, additional bars were placed throughout the wall to maintain uniform concrete cover, as illustrated in Figure 3.19. The formwork construction and erection around the perimeter were executed such that the studs supporting the walls remained in place. The formwork was designed with a small opening at the front and back for the purpose of casting the self-

consolidating concrete SCC (Figure 3.20). To prevent the formwork from bulging outwards during casting of the concrete, steel ties were inserted through the wall and tightened at the ends.



Figure 3.19: Additional Bars to Maintain Uniform Cover



Figure 3.20: Form Work for Both Walls

Once the formwork was in place, the concrete was prepared as instructed by the manufacturer. First, 2.5 L of water per bag of SCC was added to the concrete mixer located in the structures

laboratory as shown in Figure 3.21. Thereafter, the SCC was added while the concrete mixer was running. Finally, an additional 2.7 L of water per bag of SCC was added to the mixer. Once the concrete was thoroughly mixed, the concrete was poured into the small openings provided in the formwork using small plastic buckets (Figure 3.22).



Figure 3.21: Concrete Mixer in Structures Laboratory



Figure 3.22: Casting of Concrete

The day following casting, the formwork was removed and the walls were exposed for curing as presented in Figure 3.23. Curing involved placing wet burlap around the walls and rewetting it for two consecutive days. After curing, the walls were considered fully repaired and ready to be re-tested. Figure 3.24 and 3.25 illustrate the condition of the repaired SMA (RW2-NR) and conventional (RW1-SR) walls, respectively.



Figure 3.23: Curing of Repaired Shear Walls



Figure 3.24: Repaired RW2-NR



Figure 3.25: Repaired RW1-SR

3.4 Material Properties

3.4.1 Concrete

The walls were repaired in the critical regions using high strength, self-consolidating concrete (SCC). Advantages in using SCC include: simple to use and high deformability; and moderate viscosity, making it an excellent source for repairing concrete structures where vibration is not possible or when there is complex formwork. The SCC was donated by Sika for the purpose of this research. Figure 3.26 provides technical data as provided by the manufacturer.

Technical Data					
Packaging	25 kg (55 lb) bag				
Colour	Concrete Grey				
Yield	Approx. 13 L (0.46 ft³) of fresh concrete per bag.				
Shelf Life	1 year in original, unopened packaging. Store dry between 4 and 35°C (40 and 95°F), ensuring that product is not exposed to rain, condensation or high humidity. Condition material between 18 and 24°C (65 and 75°F) before using.				
Mix Ratio	Mix with clean potable water at rate of between 2.5 and 2.7 L (0.66 and 0.71 US gal.) per bag. Start with 2.5 L (0.66 US gal.) and mix to consistency required with remainder of water.				
Properties at 23°C (73°F) and 50% R.H.					
Standard Water Ratio	2.6 L/25 kg bag (0.69 US gal./55 lb)				
Mixing Time	2 to 2 min 30 s				
Application Time	25 to 30 min				
Slump Flow ASTM C1611	600 - 700 mm (24 - 28 in)				
Flow Rate ASTM C230	0 min at standard water ratio				
	240 mm (9.5 in)				
	15 min at standard water ratio				
	180 mm (7 in)				
Air Content ASTM C457	7 ± 2%				
Compressive Strength ASTM C39, MPa (psi)					
24 hrs	11 (1595)				
3 days	31 (4496)				
28 days	45 (6527)				
*Compressive Strength ASTM C39, MPa (psi)					
(tested with Sikacem® Accelerator)					
Temperature	Dosage	24 hours	2 days	3 days	28 days
-5°C (23°F)	1 bottle (150 mL)	2 (290)	3 (435)	7 (1015)	35 (5076)
-5°C (23°F)	2 bottles (300 mL)	4 (580)	8 (1160)	10 (1450)	40 (5800)
0°C (32°F)	1 bottle (150 mL)	4 (580)	6 (870)	9 (1305)	40 (5800)
0°C (32°F)	2 bottles (300 mL)	7 (1015)	10 (1450)	13 (1885)	44 (6380)
10°C (50°F)	1 bottle (150 mL)	10 (1450)	13 (1885)	20 (2900)	44 (6380)
10°C (50°F)	2 bottles (300 mL)	14 (2030)	15 (2175)	22 (3190)	45 (6527)
23°C (73°F)	1 bottle (150 mL)	16 (2320)	23 (3335)	-	-
23°C (73°F)	2 bottles (300 mL)	18 (2610)	25 (3625)	-	-
*All moulds, mixing tools and powder components were pre-conditioned to the test temperatures. Prepared test specimens were cast and then cured at the indicated test temperatures until the time of testing.					
Liquid/solids ratio (water + Sikacem® Accelerator/Sikacrete®-08 SCC) = 0.104; [2.6 L (0.69 US gal.) of liquid per 25 kg (55 lb) bag of Sikacrete®-08 SCC].					
Bond Strength CAN A23.2-6B					
7 days	2.5 MPa (362 psi) failure in substrate (Substrate 35 MPa concrete)				
Bond Strength ASTM C882					
(slant shear)	14 MPa (2030 psi) failure in mold				
Freeze/Thaw Durability ASTM C666	> 300 cycles				
Shrinkage ASTM C157	< 0.065%				
Product properties are typically averages, obtained under laboratory conditions. Reasonable variations can be expected on-site due to local factors, including environment, preparation, application, curing and test methods.					

Figure 3.26: Technical Data of Self-Consolidating Concrete (<http://can.sika.com>)

Concrete cylinders were prepared on the day of casting concrete for the walls. The cylinders were subjected to the same curing conditions as the walls. The cylinders were tested on the day of testing

at a stress rate of 0.25 MPa/s in accordance with ASTM C39 (1994). The average concrete strength for Wall RW1-SR was 76 MPa while for Wall RW2-NR it was 81 MPa. Figure 3.27 represents a typical stress-strain response of the SCC used in Wall RW2-NR.

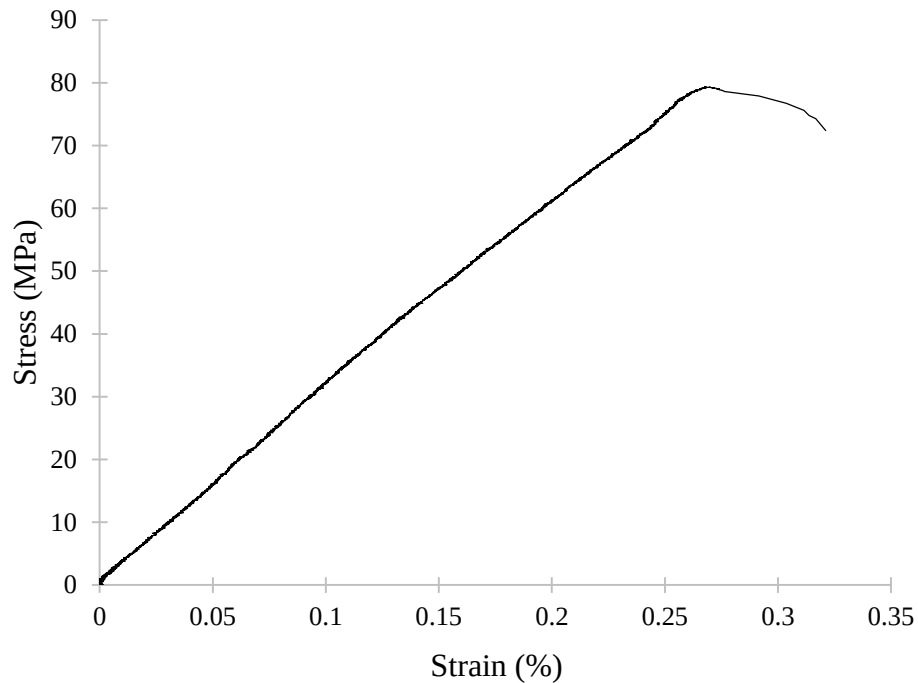


Figure 3.27: Typical Stress-Strain Response of SCC from Wall RW2-NR

3.4.2 Nitinol Rods (SMA)

The super-elastic Nitinol rods used in Wall RW2-NR were provided by SAES Smart Materials.

Table 3.2 provides the technical data as provided by the manufacturer.

Table 3.2: Material Properties of SMA Provided by SAES (Abdulridha, 2013)

Physical Properties	Density	6.5 g/cm ³
	Melting Point	1310 °C
	Modulus of Elasticity	41-75 x10 ³ MPa
	Electrical Resistivity	32 μohm-cm
	Coefficient of Thermal Expansion	11x10 ⁻⁶ /°C
Mechanical Properties	Yield Strength	379 MPa (min.)
	Ultimate Tensile Strength	1068 MPa
	Total Elongation	10% (min.)
	Permanent Set After 6% Strain	0.1%
	Transformation Temperature A_r	(-30)-10 °C
Chemical Composition	Nickel	55.8 wt.%
	Titanium	Balance
	Iron	0.05 wt.% (max.)
	Carbon	0.02 wt.% (max.)

The segment of SMA bars that were removed from Wall W2-NR to shorten the SMA in each boundary region (L1 and R1) were tested under cyclic tensile loading to assess the deformation recovery capacity after the original wall test. The testing was performed using a 600 kN universal testing machine. The stress-strain responses are provided in Figures 3.28 and 3.29 for L1 and R1, respectively. Note that L and R refer to the left and right boundary.

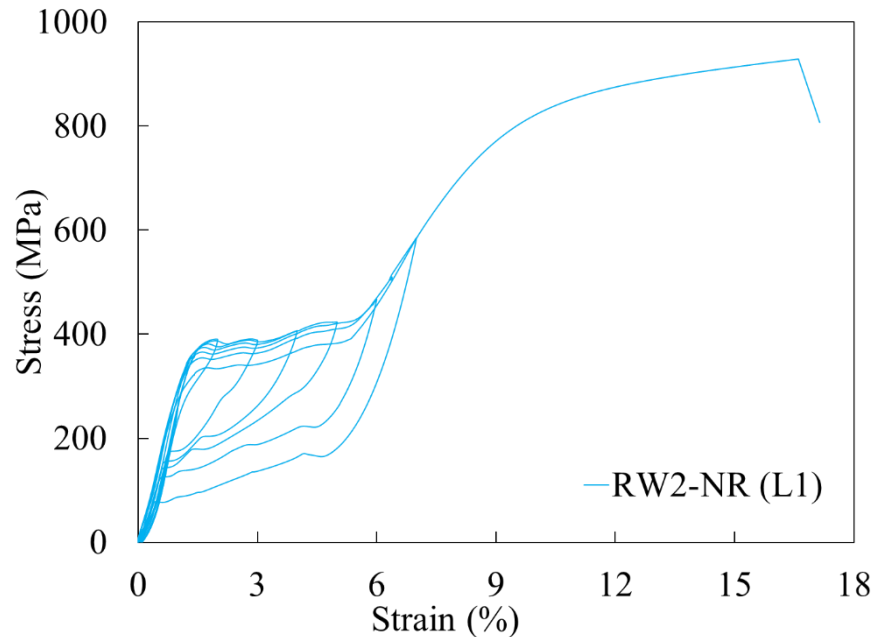


Figure 3.28: Stress-Strain Response of SMA from Left Boundary

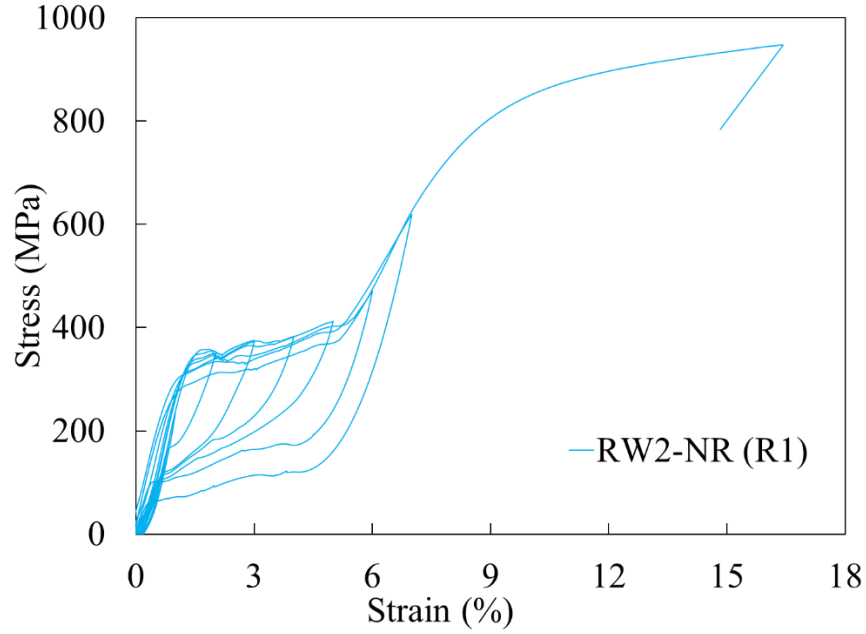


Figure 3.29: Stress-Strain Response of SMA from Right Boundary

The responses were similar to the original bar testing with a yield stress of 305 MPa and modulus of elasticity of 27 GPa, calculated using 0.2% offset method. At 7% strain, the stress in the SMA bar was 625 MPa. Beyond this strain level, the SMA bar was loaded to rupture. The ultimate (rupture) stress was 948 MPa at a strain of 16.4%. The stress-strain responses demonstrated that the SMA bar retained the capacity to recover large strains. Table 3.3 provides the properties of the SMA reinforcing bars tested as part of this research and the properties provided by Abdulridha (2013)

Table 3.3: SMA Properties Before and After the Original Cyclic Test

Designation	Yield Strength (MPa)	Yield Strain (%)	Modulus of Elasticity (GPa)
SMA (W2-NR)	380	0.82	38
SMA (RW2-NR)	305	1.1	27

3.4.3 Conventional Reinforcing Steel

Buckled and fractured 10M deformed reinforcement in both walls were replaced with similar 10M bars. In addition, any transverse reinforcement (10M) and anti-buckling ties (10M) that were removed to facilitate the repair were replaced with the same reinforcing bar type during the repair process. A standard tensile test was performed using a 600 kN MTS testing machine on the new 10M bars. Typical stress-strain response is provided in Figure 3.30. The following properties were established: yield strength of 480 MPa, ultimate strength of 650 MPa, and modulus of elasticity of 200 GPa.

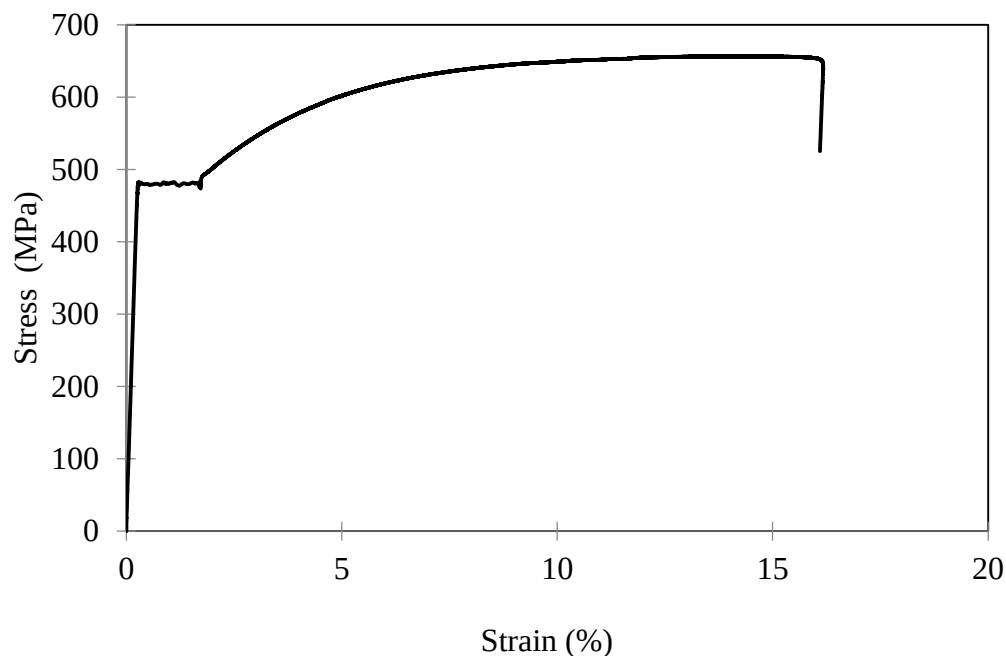


Figure 3.30: Typical Stress-Strain Response of 10M Reinforcement

The conventional steel reinforcement in the portion of the walls that was not repaired had the properties listed in Table 3.4.

Table 3.4: Original Reinforcement Properties (Abdulridha, 2013)

Designation	Yield Strength (MPa)	Yield Strain (%)	Ultimate Strength (MPa)	Ultimate Strain (%)	Modulus of Elasticity (GPa)
10M	425	0.21	615	14.72	202
15M	440	0.22	650	12.96	200
20M	472	0.23	678	13.64	205
φ=6.35mm	570	0.28	650	20.72	203

3.5 Instrumentation

3.5.1 Strain Gauges

Two types of strain gauges were used to measure the strains in the reinforcing bars throughout the walls. General purpose (F-series) strain gauges were used for the conventional deformed steel while post-yield (YF-series) strain gauges were used for the SMA bars. Prior to bonding the strain gauges on the deformed reinforcing steel, the surface was ground to make it smooth. The smooth surface was then cleaned using an acid and base solution to remove any dirt. Finally, the strain gauges were glued to the smooth surface using a glue-type product provided by the manufacturer (Figure 3.31). Once the strain gauges were bonded to the surface, lead wires were soldered to the strain gauges and electric tape was used to protect the gauges from any damage during casting of the concrete.

The lead wires for the Y-F series strain gauges used on the SMA bars were soldered first to the strain gauges due to lack of space in the wall for soldering. Thereafter, the SMA bars were cleaned using an acid and base solution. Once the surface was cleaned, the strain gauges were bonded to the SMA bars. Lastly, the electric tape was wrapped around the strain gauges to protect them during concrete casting.



Figure 3.31: Strain Gauge Preparation on 10M bar

The layout and numbering of the strain gauges for Walls RW1-NR and RW2-SR are provided in Figures 3.32 and 3.33, respectively.

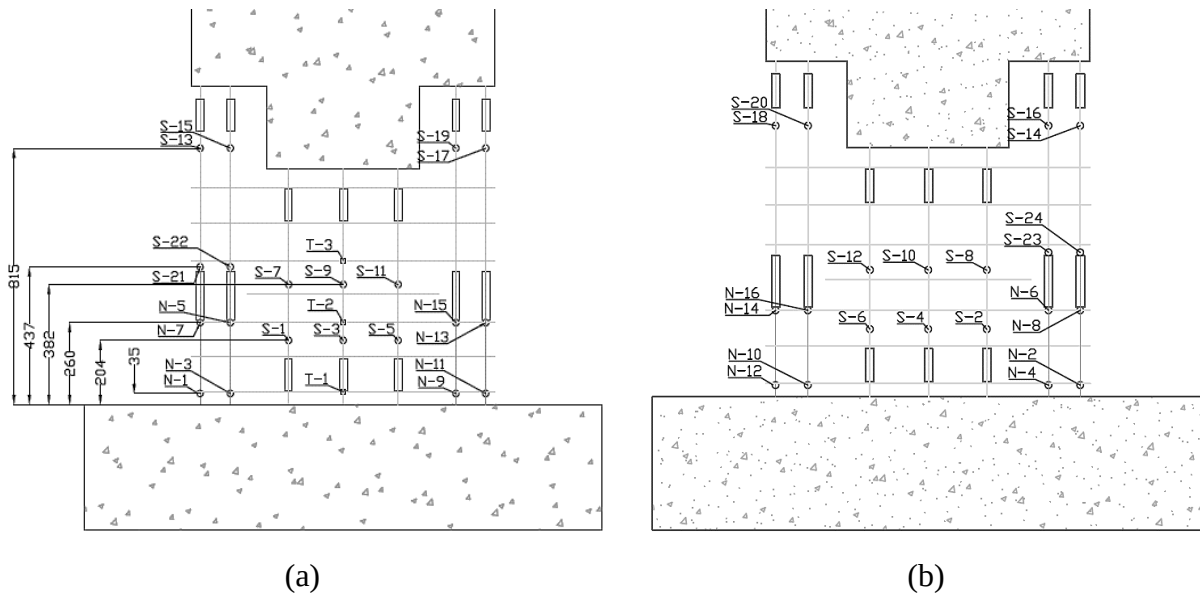


Figure 3.32: RW2-NR Strain Gauge Numbering and Layout (a) Front; and (b) Back

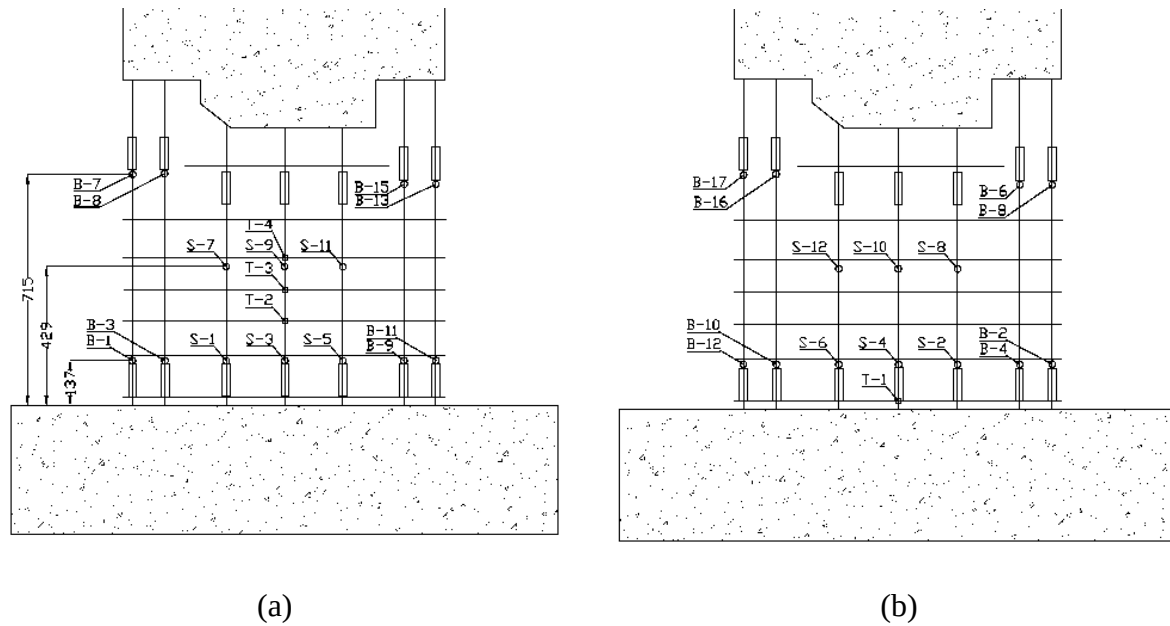


Figure 3.33: RW1-SR Strain Gauge Numbering and Layout (a) Front; and (b) Back

To record the strain profile of the SMA bars in the boundary zones of Wall RW2-NR, strain gauges were placed on the SMA bars 35 mm from the base of the wall, and below and above the six-inch modified coupler at 260 mm and 467 mm, respectively, from the base. Two additional strain gauges were placed in the boundary zones on the SMA bars at 815 mm from the base of the wall, just below the top couplers connecting the new segment of conventional steel to the existing reinforcing steel bar to measure the strains above the plastic hinge region. In the web of RW2-NR, the strain gauges were concentrated below the height of the couplers used in the boundary zones to connect the SMA bars to the conventional steel. Based on the original testing, it was assumed that a critical section would develop at this height, where stress would concentrate along a predominant crack. The strain gauges in the web were placed at 204 mm and 382 mm from the base. As for the transverse reinforcement, strain gauges were placed at 35 mm, 260 mm and 437 mm from the base.

For Wall RW1-SR, the first strain gauges were placed just above the bottom couplers at 138 mm from the base of the wall. This location was selected specifically due to the higher stiffness of the wall near the base as a result of the presence of the couplers. It was assumed that the critical section would develop just above the couplers. The second layer of strain gauges in the web was placed at 429 mm from the base; the same location of the second layer of strain gauges in the web of RW2-NR. Similar to Wall R2W-NR, two additional strain gauges were placed in the boundary zones just below the top couplers at 716 mm from the base. The strain gauges at the transvers reinforcement were installed at 35mm, 237mm, 337mm and 437 mm from the base.

3.5.2 Displacement Transducers

Linear Variable Differential Transducers (LVDTs) and Displacement Cable Transducers (DCTs) were installed at various locations to record displacements experienced by the walls. Detailed locations of the instrumentations are provided in Figure 3.34.

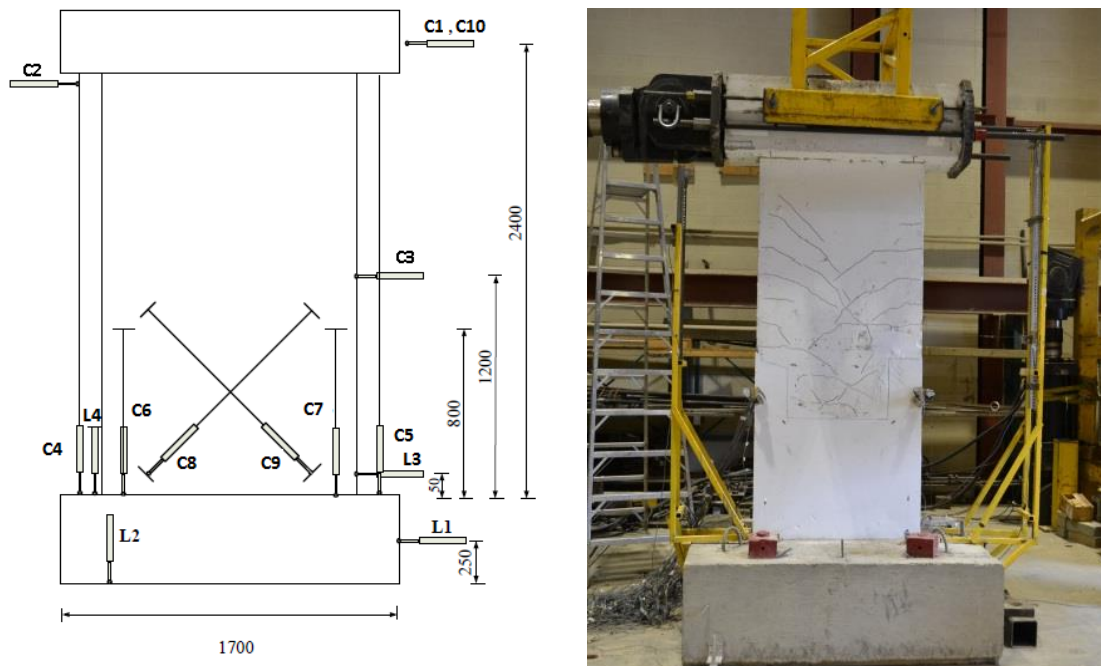


Figure 3.34: DCTs and LVDTs Layout for Displacement Measurements (Modified from Abdulridha, 2013)

The lateral displacements of the walls were measured by DCTs (C1, C2, C3, and C10). C1, C2, and C3 were supported by a light steel frame which was connected to the base of the wall, resulting in independent readings from any movement of the foundation during testing (Figure 3.35 (a), (b) and (c)). C10 was supported by a steel frame, which was fastened to the strong floor of the laboratory to produce measurements with respect to the floor (Figure 3.35 (d)).

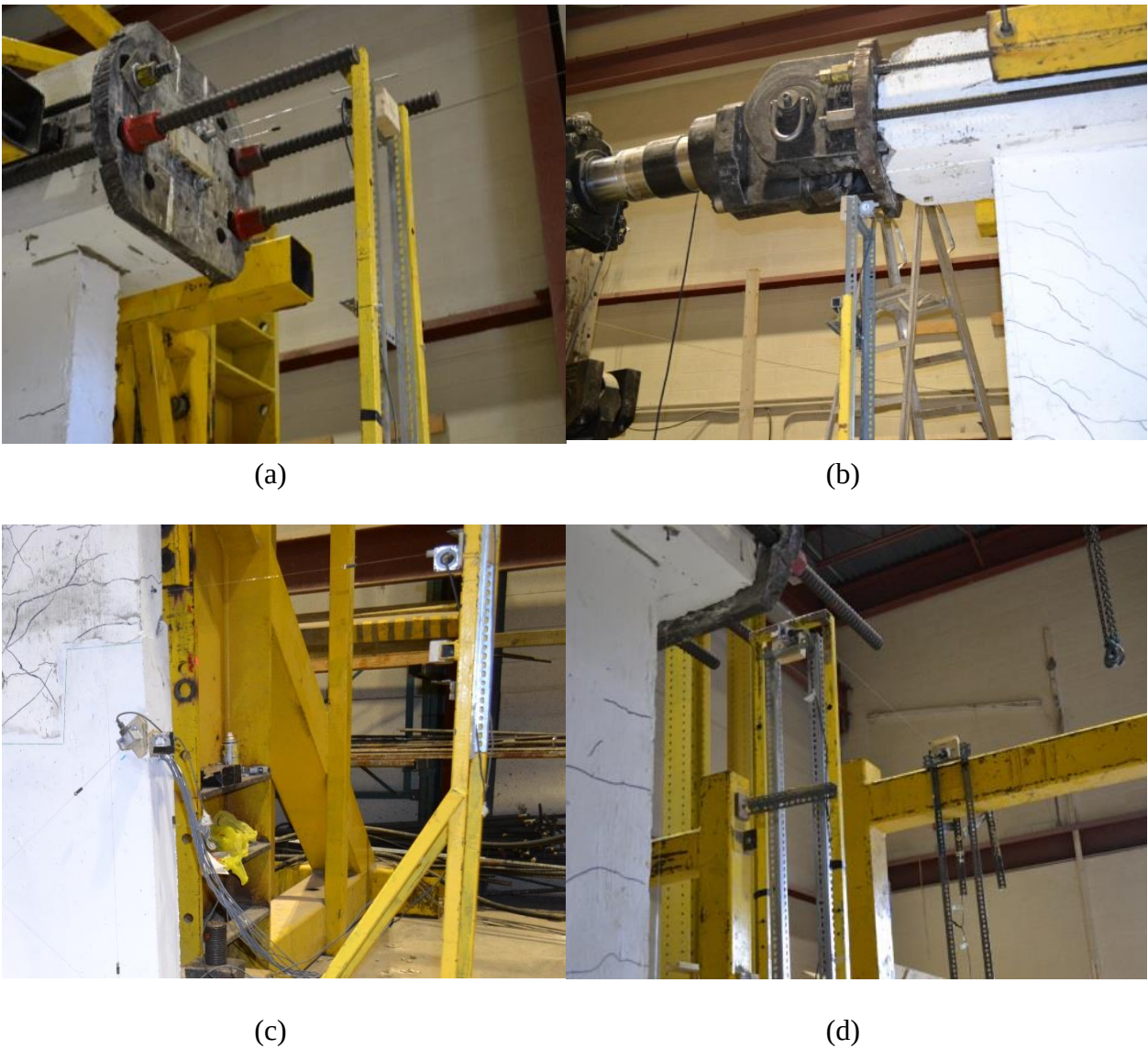


Figure 3.35: Locations of DCTs: (a) C1; (b) C2; (c) C3; and (d) C10

C1 and C2 measured the lateral displacement at the top of the wall from both ends. However, due to the loading system, C2 was placed 250 mm below the midpoint of the top beam. C3 was installed at a height of 1200 mm from the base of the wall to measure the lateral displacement at the top of the plastic hinge region.

DCTs C4 and C5 were placed at the top of the base foundation and extended to the bottom of the top of the beam to measure the vertical displacements at the ends of the wall relative to the base foundation. C6 and C7 were placed at 800 mm from the base of the wall and extended down to the top of the foundation to measure the vertical displacement at the top of the plastic hinge region at both ends. C8 and C9 were attached to the wall at 45 degrees to calculate the diagonal deformation of the plastic hinge region with a combination of recordings from both C8 and C9. Figure 3.36 illustrates the DCTS in the plastic hinge region.



Figure 3.36: DCTS in Plastic Hinge Region: (a) Front View; and (b) Side View

Four Linear Variable Differential Transducers (LVDTs) were connected to the wall. LVDT 1 was connected to the lab strong floor and to the side of the base foundation to measure sliding of the entire wall (Figure 3.37 (a)). LVDT 2 was connected to the base foundation and the lab floor to

measure uplift during testing (Figure 3.37 (b)). To measure shear sliding at the base of the wall, LVDT 3 was connected to the wall at a height of 50 mm and parallel to the loading (Figure 3.38 (b)). Lastly, LVDT 4 was used to measure vertical anchorage slip at the base of the wall (Figure 3.38 (a)).



(a)

(b)

Figure 3.37: LVDT Locations: (a) LVDT 1; and (b) LVDT 2



(a)

(b)

Figure 3.38: LVDT Locations: (a) LVDT 3; and (b) LVDT 4

3.6 Test Setup

The test setup consisted of lateral loading system in which the lateral loading was applied using a displacement controlled hydraulic actuator positioned at the center of the top loading beam (Figure 3.39). The actuator was attached to a steel frame which consisted of two I-section steel columns braced by I-section steel struts. At the base the reaction frame was connected to the strong laboratory floor with high strength bolts. The hydraulic actuator used during testing has a maximum load capacity of 1000 kN in tension and 1450 kN in compression with a maximum stroke capacity of 500 mm. To eliminate out-of-plane displacements of the walls, a lateral supporting frame was connected to the top loading beam.

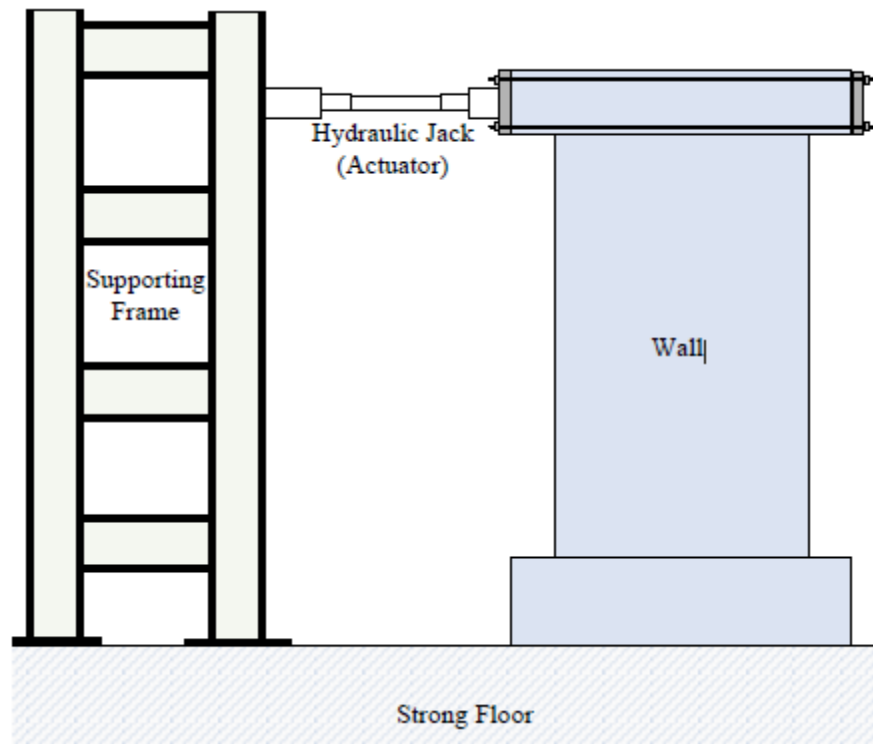


Figure 3.39: Test Setup (Abdulridha, 2013)

3.7 Test Program

The walls were tested in a sequence of reverse cycles at multiples of the yield displacement, as stated in ATC-24 (1992). Following this procedure, Abdulridha (2013) developed the loading protocol for the original walls as demonstrated in Figure 3.40. For the repaired walls in this study, the yield displacement used in the loading protocol was based on the experimental yield displacement determined by Abdulridha (2013) during the original testing which, was 27 mm and 9 mm for RW2-NR and RW1-SR, respectively. The walls were first loaded to the yield displacement in three steps with three repetitions at each displacement level. The first and second loading cycles targeted $1/3 \Delta y$ and $2/3 \Delta y$, respectively, while the third loading cycle aimed for the yield displacement. In the post-yield regime, loading was incremented by $1/3 \Delta y$ for Wall RW2-NR and Δy for Wall RW1-SR until $5\Delta y$ was achieved, with each loading cycle being imposed with three repetitions. The difference in yield displacement increment was intended to ensure a similar displacement was imposed at each load cycle. Beyond $5\Delta y$, two repetitions of loading were imposed to failure. The test protocols for Walls RW2-NR and RW1-SR are provided in Figures 3.41 and 3.42, respectively.

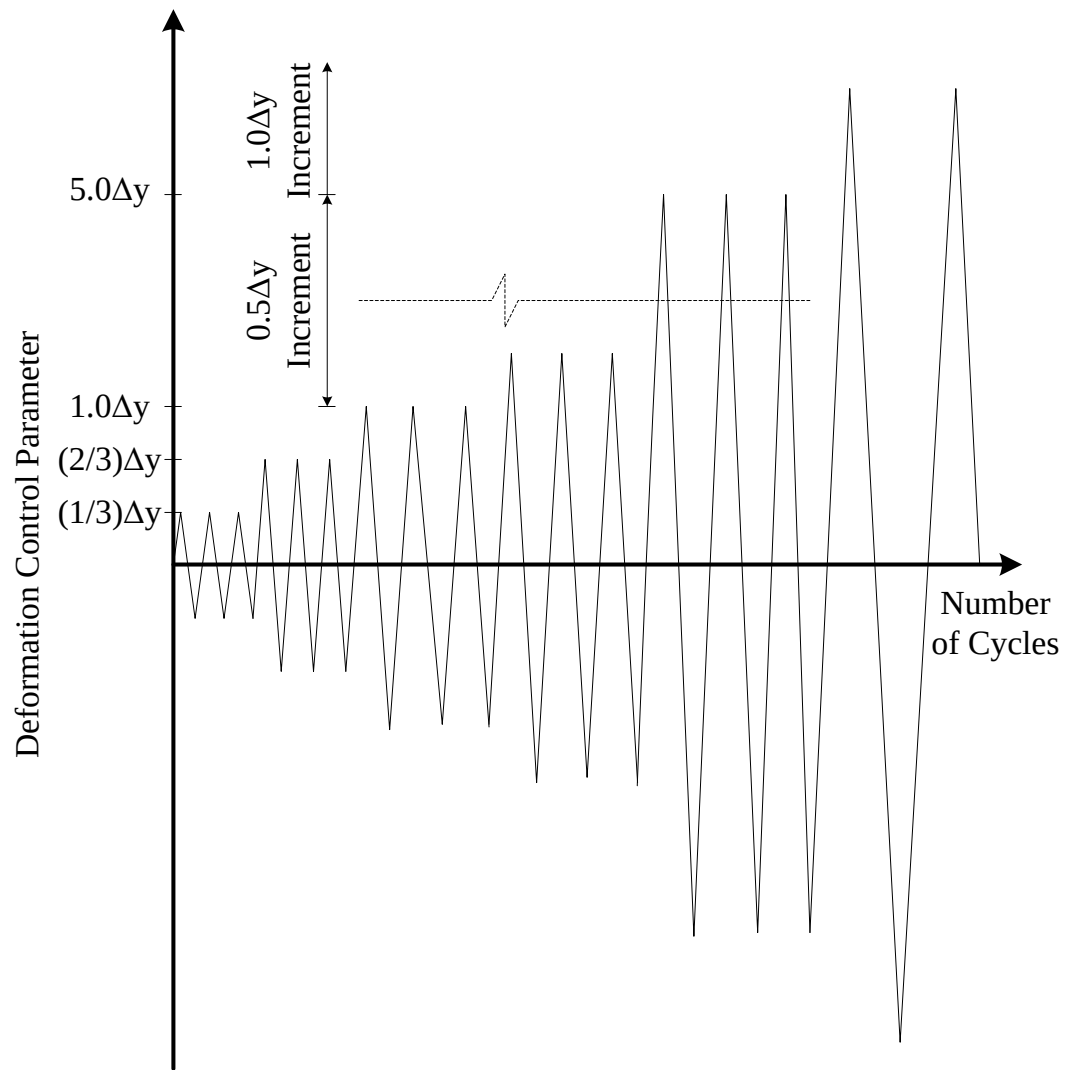


Figure 3.40: Test Protocol for Original Walls (Abdulridha, 2013)

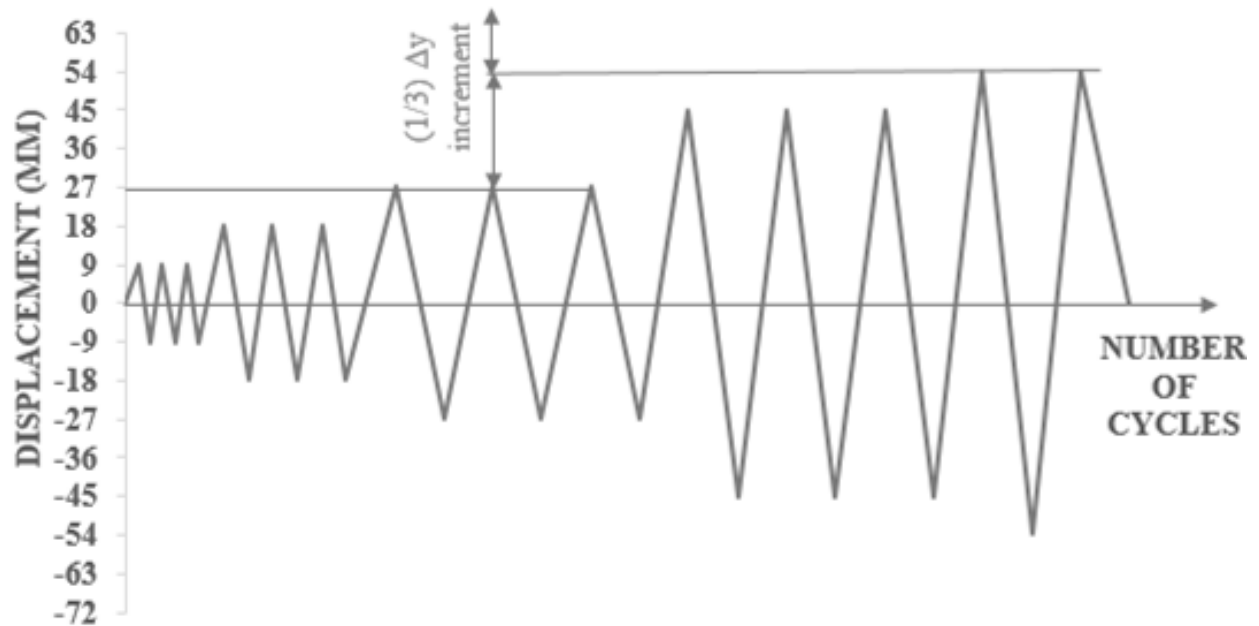


Figure 3.41: Test Protocol for Wall RW2-NR

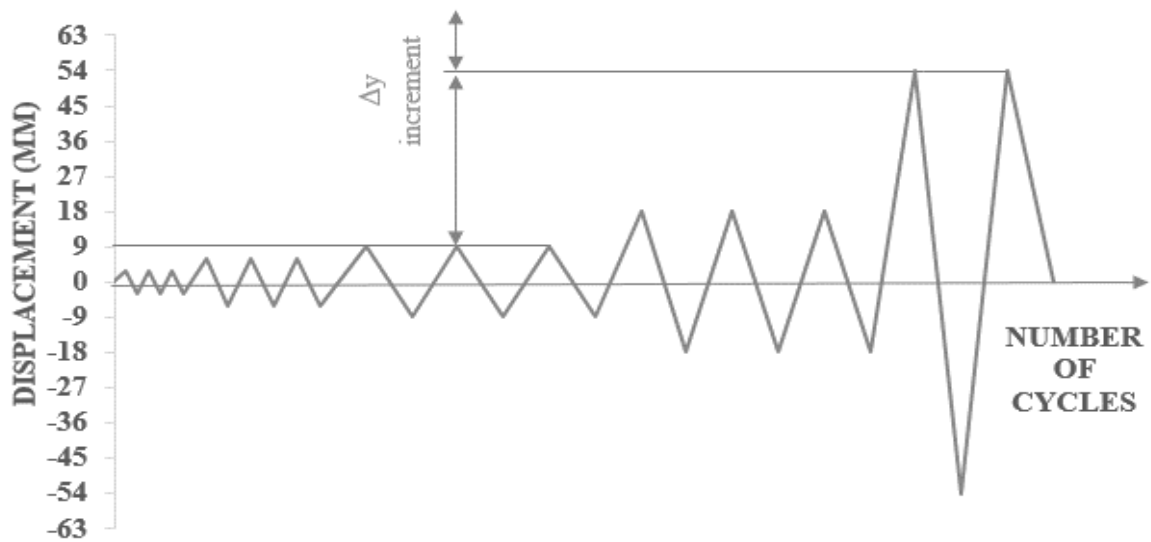


Figure 3.42: Test Protocol for Wall RW1-SR

CHAPTER 4

TEST RESULTS

4.1 Introduction

This chapter presents the experimental data recorded during testing of the repaired walls. This includes the crack patterns that were observed and crack widths that were measured and noted at the end of every loading cycle; the hysteretic load-displacement responses of both walls; the vertical displacement of the top loading beam relative to the base foundation; the diagonal displacement of the wall over the plastic hinge region; and typical strain responses of the reinforcement recorded by the strain gauges.

4.2 Crack Characteristics

The crack characteristics of both the conventional steel reinforced and hybrid SMA reinforced shear walls were monitored throughout testing. At each drift level, the flexural and shear cracks were recorded and the corresponding crack widths were measured. The walls were loaded according to the loading protocol ATC-24 (1992) as described in Chapter 3. In both walls, as the load increased, the initial flexural cracks became wider and propagated further into the wall. Also, diagonal cracking developed from the initial flexural cracks as the loading progressed.

The first flexure cracks became evident at the end of the first loading stage to 3 mm displacement for Wall RW1-SR (steel reinforced wall) and 9 mm displacement for Wall RW2-NR (hybrid SMA reinforced wall). At these load stages, the lateral load experienced by the Walls RW1-SR and RW2-NR were 66 kN and 44 kN, respectively. The initial cracks were perpendicular to the longitudinal reinforcement and spread along the repaired region.

For Wall RW1-SR, the initial flexural cracks surfaced at a height of 146 mm, 350 mm and 785 mm from the base of the wall. The first two cracks from the base of the wall propagated along the

entire width of the wall, while the third crack from the base of the wall was confined to the left repaired boundary region. In Wall RW2-NR, the initial crack pattern was similar to that of RW1-SR except one additional crack surfaced. Moreover, they appeared at slightly different heights (140 mm, 260 mm, 440 mm and 690 mm) from the base of the wall. The recovery of crack opening was evident for the crack located at 140 mm from the base. At the end of first cycle, this crack was only visible in the web region of the wall. The crack located at 260 mm from the base of the wall surfaced just below the six-inch coupler used to connect SMA rod to the deformed reinforcing steel. The crack at 440 mm from the base of the wall surfaced above the six-inch coupler. The cracks at 260 mm and 440 mm from the base propagated across the entire width of the wall. The initial crack patterns for both walls are illustrated in Figure 4.1.



Figure 4.1: Initial Flexure Crack Pattern: (a) RW1-SR; and (b) RW2-NR

The first diagonal crack was observed in Walls RW1-SR and RW2-NR at the end of the first loading repetition to 3 mm (0.13% drift) and 9 mm (0.4% drift), respectively. The diagonal crack surfaced in RW1-SR at 830 mm from the base of the wall, and in RW2-NR at 610 mm from the base of the wall (Figure 4.2). At this stage, more flexure cracks were observed in both walls.



Figure 4.2: First Diagonal Crack: (a) RW1-SR; and (b) RW2-NR

As the loading progressed towards Δ_y , further flexure and shear cracks developed in both walls and the initial flexure cracks extended in length. In Wall RW1-SR, the initial flexure cracks had width of 0.25mm. At the end of loading to 9 mm displacement, the crack that formed at 146 mm from the base of the wall had a width of 0.50 mm, while the crack located at 540 mm from the base had a width of 0.90 mm. For RW2-NR, the initial flexure cracks had width of 0.20 mm. At the end of load stage to 18 mm displacement, the width of the crack located at 260 mm from the base increased to 1.75 mm.

Throughout the test, it was evident that the crack width in the boundary of RW2-NR was significantly greater than the crack width in the boundary of RW1-SR. This was due to the smoothness of the SMA bars resulting in a weak bond to the surrounding concrete. As a result, cracks are wider and spaced farther apart relative to deformed steel-reinforced walls.

In the post-yield range of loading and at the drift level of 1.12% (27 mm displacement), both walls experienced growth in flexure and diagonal cracks. More diagonal cracks formed in Wall RW1-SR compared to Wall RW2-NR. Note that the diagonal cracks in RW1-SR were spread throughout

the repaired region, while in RW2-NR the diagonal cracks were concentrated in the web region. The crack pattern at 1.12 % drift is shown in Figure 4.3.



Figure 4.3: Post-Yield Crack Pattern at Drift Level of 1.12%: (a) RW1-SR; and (b) RW2-NR

At drift level of 1.5 % (36 mm displacement) spalling of concrete was evident at the boundary of Wall RW2-NR as shown in Figure 4.4. Spalling initiated at the crack located 260 mm from the base of the wall, which was just below the 6-inch coupler. The two probable reasons for early spalling were the weak bond between the surface of the mechanical coupler and the concrete and reduced cover caused by the presence of the mechanical coupler. At this stage of loading, Wall RW1-SR was intact with the development of more diagonal cracking, while the existing flexure cracks increased in length. Compared to Wall RW2-NR, a significantly lower increase in diagonal cracks were evident at this drift level.



(a)



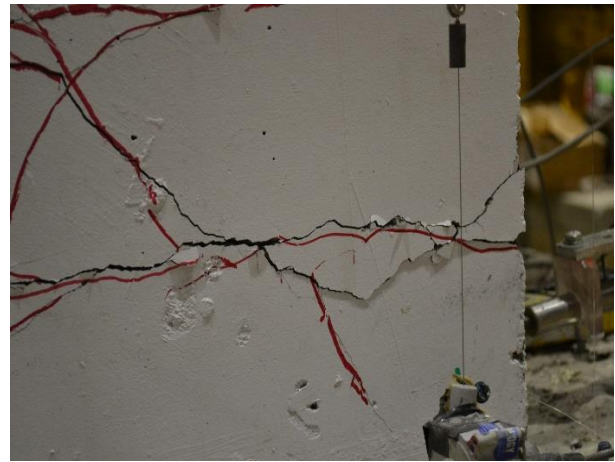
(b)

Figure 4.4: Crack Pattern at Drift Level of 1.5%: (a) RW1-SR; and (b) RW2-NR

In Wall RW1-SR, the initiation of concrete spalling became evident at the right boundary zone at the end of loading to 45 mm displacement (1.875% drift). As loading increased, spalling at this location became more critical and it was one of the reasons for the failure of the wall. The front and side view of the initial concrete spalling in Wall RW1-SR is shown in Figure 4.5.



(a)



(b)

Figure 4.5: Initiation of Concrete Spalling in RW1-SR at Drift level of 1.875 %: (a) Front View; and (b) Side View

At drift level of 2.25% (54 mm displacement), a large portion of concrete from the right side of the boundary of Wall RW2-NR spalled from the wall, exposing the bar used to maintain uniform cover in the boundary zone as shown in the Figure 4.6.



Figure 4.6: Spalled Concrete in RW2-NR at Drift Level of 2.25%: (a) Front View; and (b) Side View

As the loading continued beyond 2.25% drift, further spalling at both boundaries became evident in both walls. In Wall RW1-SR, the crack located at 146 mm from the base of the wall extended fully along the wall and controlled the behaviour of the wall to failure. For Wall RW2-NR, the crack located at 260 mm from the base became a major crack in the left boundary; while for the right boundary, the crack located at 140 mm from the base merged with the major crack from the left boundary to form the critical crack. The failure state of Wall RW1-SR is illustrated in Figure 4.7 and failure state of RW2-NR is illustrated in Figure 4.8.

At failure, Wall RW1-SR was very unstable with cracking spread throughout the repaired region. However, in Wall RW2-NR cracking was mostly concentrated in the web region. As a result, less concrete spalling was evident and the wall appeared to be more stable compared to RW1-SR. No

new cracks surfaced in the region above the plastic hinge zone that remained intact after the original test. However, existing cracks in this region re-opened during testing.



(a)



(b)



(c)

Figure 4.7: Crack Pattern at Failure of Wall RW1-SR: (a) Front view, (b) Back view; and (c) Side view



(a)



(b)



(c)

Figure 4.8: Crack Pattern at Failure of Wall RW2-NR: (a) Front view, (b) Back view; and (c) Side view

4.3 Load-Displacement Response

Linear Variable Differential Transducers (LVDTs) and Displacement Cable Transducers were installed at various locations on the wall to monitor lateral, uplift, and diagonal displacements of the walls throughout testing.

The lateral displacement recorded from Displacement Cable Transducer C1, located at the mid-height of the top loading beam, was used to develop the hysteresis response of both repaired walls RW1-SR and RW2-NR. The hysteresis response was used to establish the stiffness, yield load, ultimate load and corresponding displacement (peak displacement), ultimate displacement, and residual displacements. The hysteresis response of RW1-SR and RW2-NR are shown in Figure 4.9 and Figure 4.10, respectively.

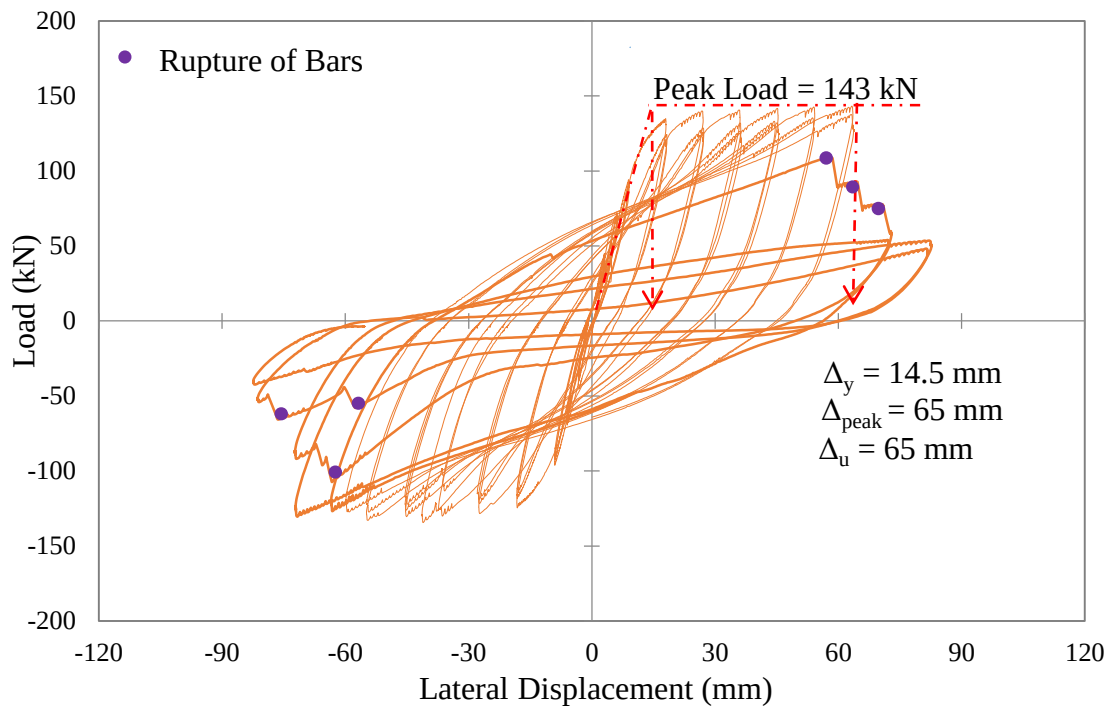


Figure 4.9: Load-Displacement Response of Wall RW1-SR

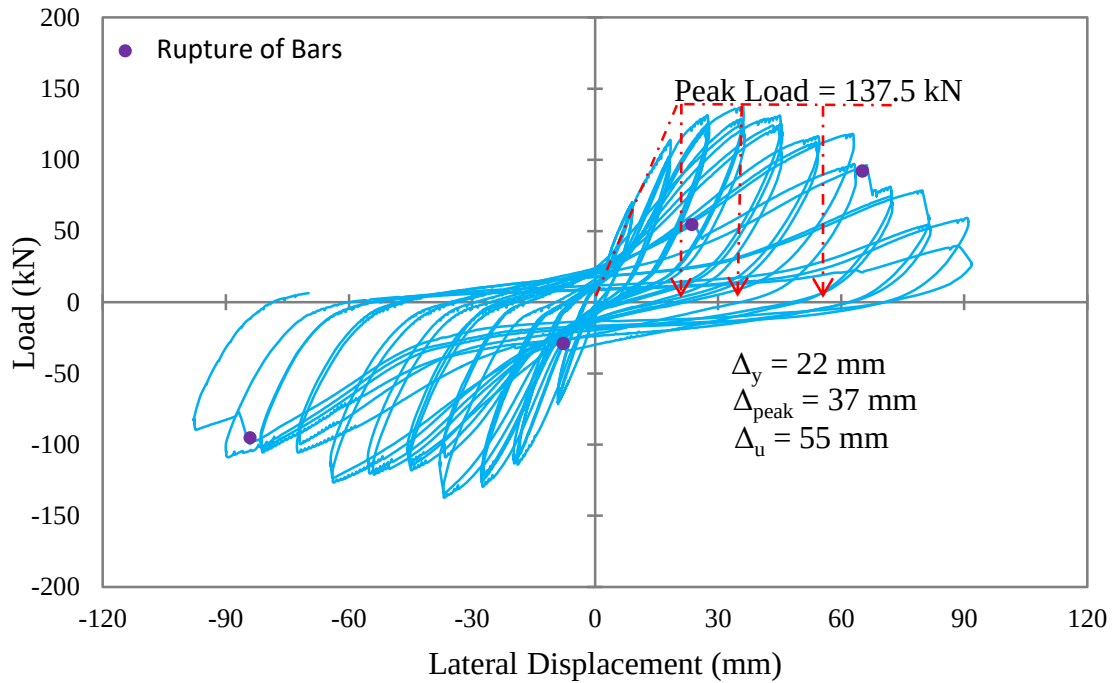


Figure 4.10: Load-Displacement Response of Wall RW2-NR

The hysteretic response of both walls prior to yielding was very similar and almost linear, indicating linear-elastic behavior. After the first repetition of loading, a reduction in strength arose during the second and third repetitions due to the decrease in stiffness caused by the cyclic loading effect. Yielding of the walls was calculated using the equivalent elasto-plastic system with a secant stiffness passing through the load-displacement response at 75% of the average nominal strength (Park 1988). The yield displacement of Wall RW1-SR was observed at displacement of 14.5 mm corresponding to 123 kN of the load. Wall RW2-NR yielded at displacement of 22 mm corresponding to 119 kN of the load. The ultimate displacement, corresponding to the displacement prior to the lateral load capacity of the wall dropping below 80% of the maximum load capacity (Park (1988)) was 65 mm and 55 mm for Walls RW1-SR and RW2-NR, respectively. The yield and ultimate laterals loads and displacements were based on the positive direction of loading.

The peak strength of the Wall RW2-NR was 137.5 kN at a displacement of 37 mm. Beyond this displacement level, Wall RW2-NR experienced a gradual reduction in strength for the remainder of the test. At a displacement level of 36 mm, the load capacity of Wall RW1-SR was similar to that of Wall RW2-NR (138 kN), however an increase in strength was experienced beyond this level in Wall RW1-SR.

Beyond the post-yield region, the ability of the SMA to recover displacements was evident in Wall RW2-NR. At a lateral displacement of 36 mm, Wall RW2-NR recovered 30 mm of displacement, while RW1-SR only recovered 14 mm of displacement. Note that at this displacement level, both walls when originally tested prior to repairing recovered similar displacements, respectively. From this, it can be concluded that the shortening of the SMA bars in RW2-NR did not affect the recovery capacity up to lateral displacement of 36 mm.

The post-yield response of Wall RW1-SR was non-linear with a near-constant plateau indicating marginal increase in strength up to the peak capacity of the wall. The peak lateral load capacity of Wall RW1-SR was 143 kN at a displacement level of 65mm. Table 4.1 summarizes the yield and peak lateral load capacities, while Table 4.2 provides the yield and peak displacements/drifts.

Table 4.1: Yield and Peak Loads of Wall RW1-SR and RW2-NR

Wall	Yield Load (kN)	Peak Load (kN)
RW1-SR	123	143
RW2-NR	119	137

Table 4.2: Yield and Peak Displacements/Drifts of Wall RW1-SR and RW2-NR

Wall	Yield Displacement (mm)	Peak Displacement (mm)	Yield Drift (%)	Peak Drift (%)
RW1-SR	14.5	65	0.92	2.63
RW2-NR	22	37	0.6	1.54%

In the post-peak regime of loading, RW1-SR developed a plastic hinge near the base of the wall. Beyond 63 mm of displacement, the wall did not maintain its lateral load capacity; for the subsequent load stage to 72 mm displacement to the push side (drift level of 3%), three bars at the left boundary ruptured, resulting in a sudden decrease in strength of about 36% as evident in the hysteretic response. While pulling the wall during the second repetition to 72 mm displacement, one bar at the right boundary ruptured. By the end of second repetition of loading at a displacement level of 81 mm on the pull side, all the bars at the right boundary had ruptured. The rupture of reinforcement at boundary is illustrated in Figure 4.11. The rupturing of the bars in the boundary zones of Wall RW1-SR occurred just above the bottom couplers; this was due to the high stress in this region due to the higher stiffness caused by the couplers. The presence of the couplers shifted the critical region to just above the couplers.



Figure 4.11: Rupture of Reinforcing Bars in Boundary Zone of RW1-SR

In the post-peak regime of loading of Wall RW2-NR, a plastic hinge also developed near the base of the wall. RW2-NR experienced a sudden degradation of strength of about 13% due to rupturing

of the SMA bar at a displacement level of 63 mm during the second repetition of loading. Rupturing of the SMA bar also affected the displacement recovery by 20 % with respect to the first repetition at a displacement level of 63 mm. During the subsequent loading stage (pushing) to 72 mm, an additional SMA bar ruptured. This significantly reduced the recovery capacity. One more SMA bar ruptured during pulling at the subsequent loading stage of 72 mm. While pushing the wall to 90 mm during the second repetition rupture of reinforcement was evident in the hysteretic response. Beyond this displacement level and up to failure of the wall, rupturing of other SMA bars and deformed vertical bars in the web section of the wall were evident (Figure 4.12). Rupturing of SMA bars occurred just below the modified 6-inch mechanical coupler.

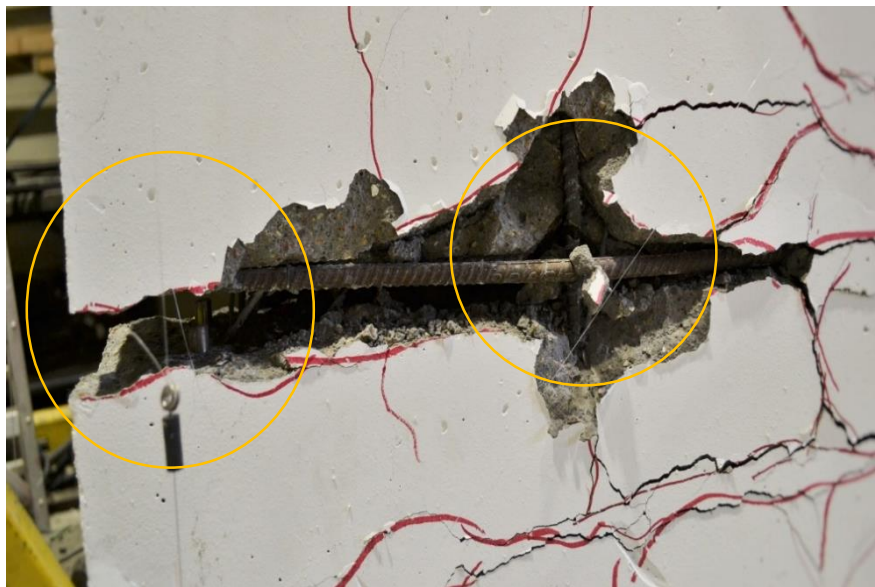


Figure 4.12: Rupture of Reinforcing Bars in Wall RW2-NR

4.4 Displacement at Mid-Height

Figures 4.13 and 4.14 illustrate the lateral displacement at the mid height of Walls RW1-SR and RW2-NR, respectively, recorded using DCT C3. At yielding, the displacement of Wall RW1-SR was 5.5 mm which represent 38% of lateral displacement at mid-height of the top loading beam.

The mid-height displacement at yielding of RW2-NR was 45% of lateral displacement at mid-height of the top loading beam of Wall RW2-NR.

At the peak lateral load, the mid-height displacements of Walls RW1-SR and RW2-NR were 46% of their respective displacement at the mid-height of the top loading beam.

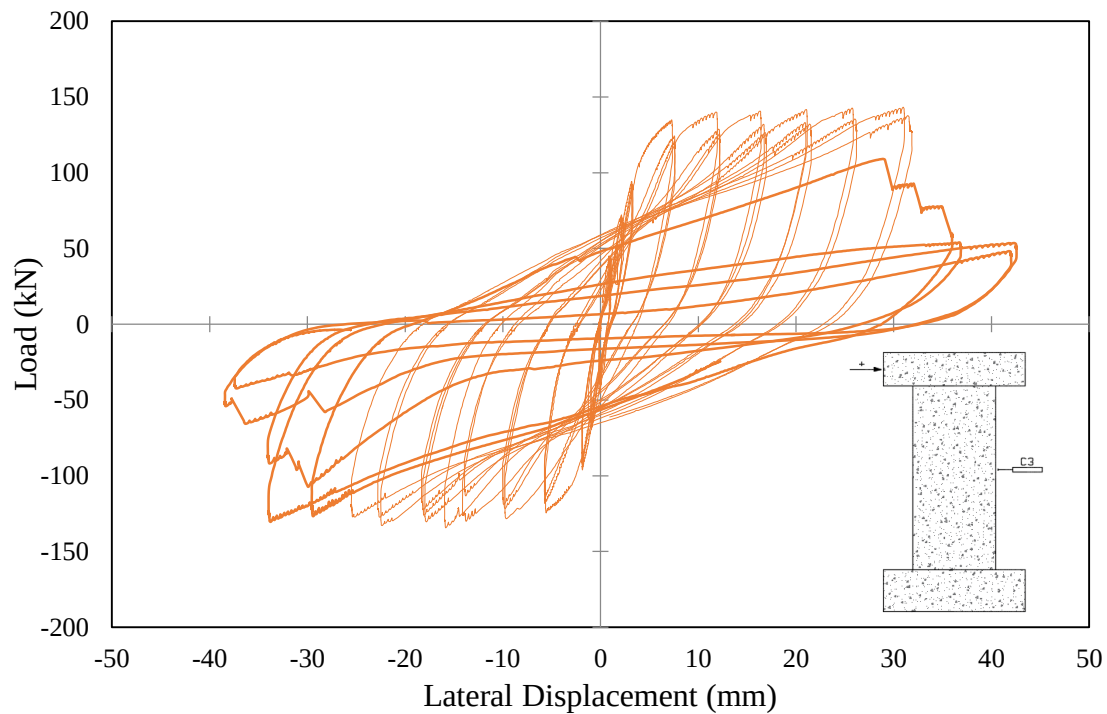


Figure 4.13: Load-Displacement Response at Mid-Height of Wall RW1-SR

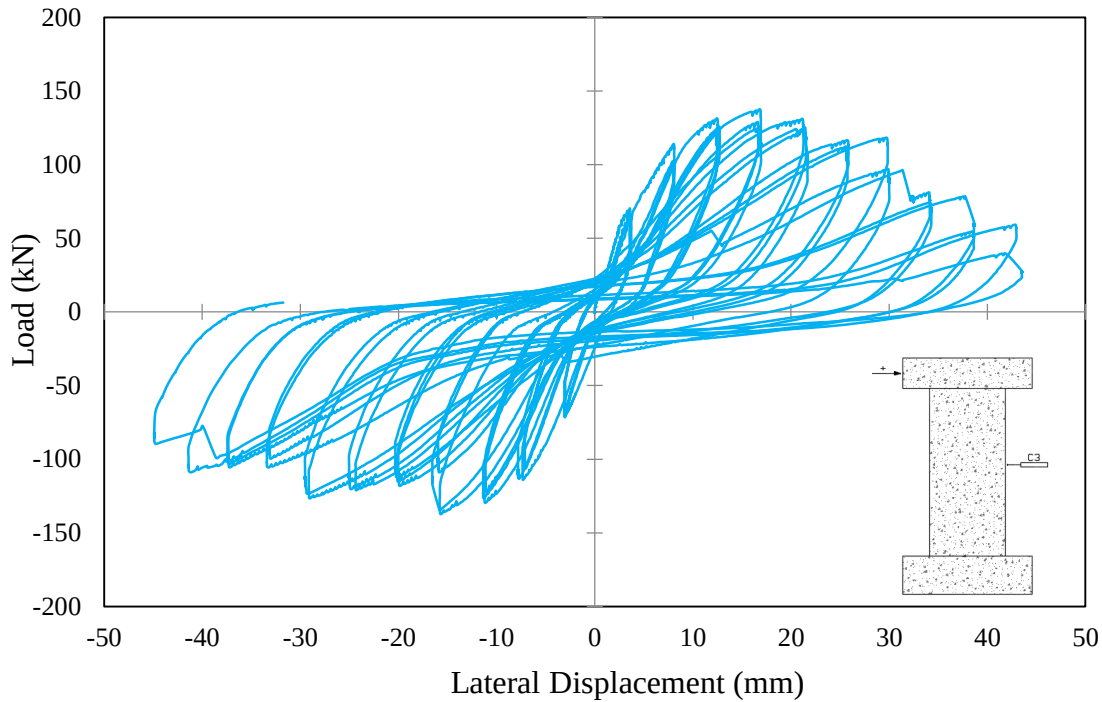


Figure 4.14: Load-Displacement Response at Mid-Height of Wall RW2-NR

4.5 Vertical Displacements

DCTs were installed on the shear walls to monitor the vertical displacements of the top loading beam and the top of the plastic hinge region relative to the base of the wall. These displacements resulted from concrete cracking and yielding of the longitudinal reinforcement. In both walls, DCT C4 measured the vertical displacement of the top loading beam, while DCT C6 measured the vertical displacement of the plastic hinge region. The hysteretic response recorded from DCTs C4 and C6 for Walls RW1-SR and RW2-SR are provided in Figure 4.15 and Figure 4.16, respectively.

At yielding of Wall RW1-SR, the vertical displacement of the top beam was 1 mm and at the top of the plastic hinge the vertical displacement was 2 mm. Similarly, at yielding of Wall RW2-NR, the vertical displacement of the top beam and plastic hinge was 4 mm and 6.9 mm, respectively. It is also noted that at yielding, the vertical displacement of both the top beam and the plastic hinge

of Wall RW2-NR is greater than RW1-SR. This can be attributed to the smooth surface of SMA bars resulting in larger crack widths and yielding at a larger lateral displacement relative to Wall RW1-SR.

At peak lateral load of Wall RW1-SR and RW2-NR, the vertical displacements of the top beam and top of the plastic hinge were approximately the same. For Wall RW1-SR, the vertical displacement of the top beam was 23 mm and at the top of plastic hinge the vertical displacement was 25 mm. For Wall RW2-NR, the vertical displacement of the top beam was 10 mm and at the top of plastic hinge the vertical displacement was 13 mm. The nominal difference between the vertical displacements of the top beam and plastic hinge in both walls confirms that all the new cracks were concentrated in the repaired zone and no new cracks were developed above the plastic hinge region.

In Figure 4.16, the self-centering phenomenon of Wall RW2-NR is evident due to the SMA bars. During load reversal, there is greater pinching in RW2-NR due to closing of the cracks.

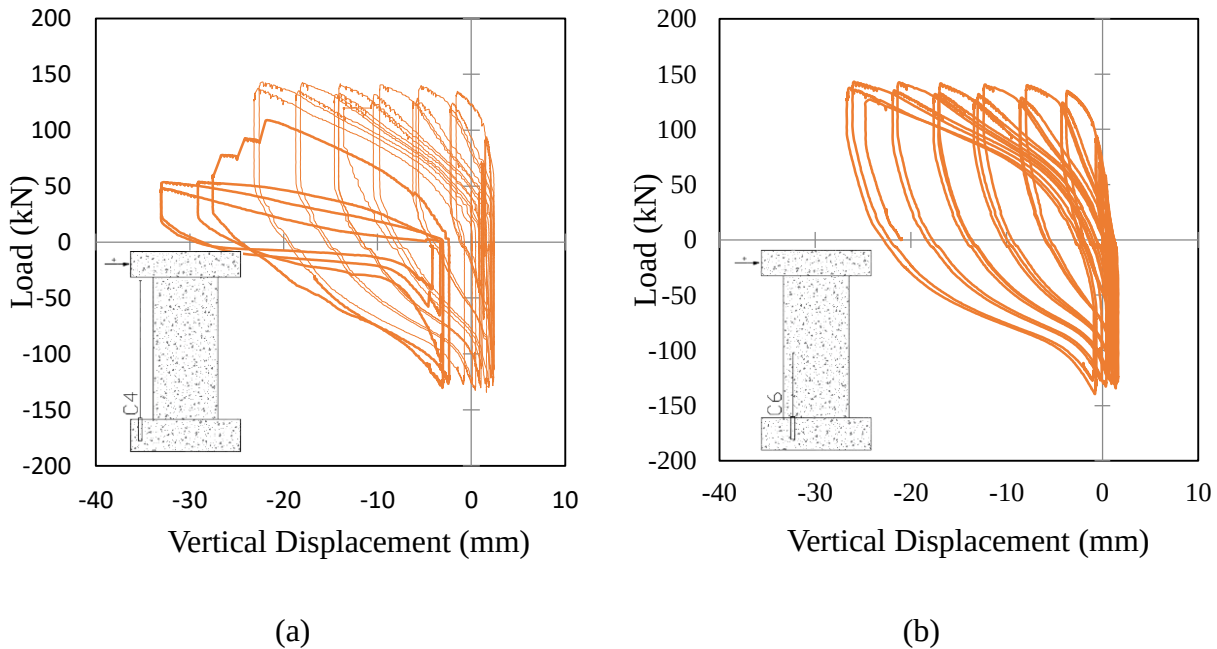


Figure 4.15: Vertical Displacement of Wall RW1-SR (a) Top Beam; and (b) Plastic Hinge

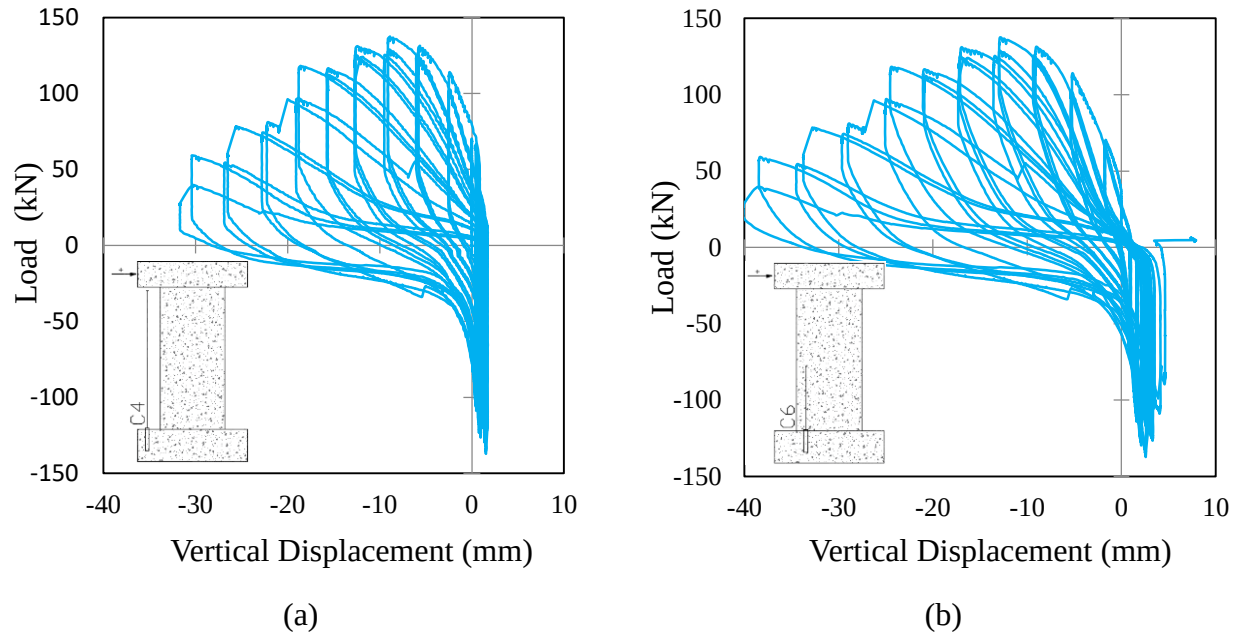


Figure 4.16: Vertical Displacement of Wall RW2-NR (a) Top Beam; and (b) Plastic Hinge

4.6 Diagonal Displacements

The diagonal displacement of the repaired plastic hinge region of Walls RW1-SR and RW2-NR was recorded during testing using DCT C8. The diagonal displacement provides an indirect measure of the shear deformation. The hysteretic load-diagonal displacement responses are provided in Figure 4.17 for Walls RW1-SR and RW2-NR.

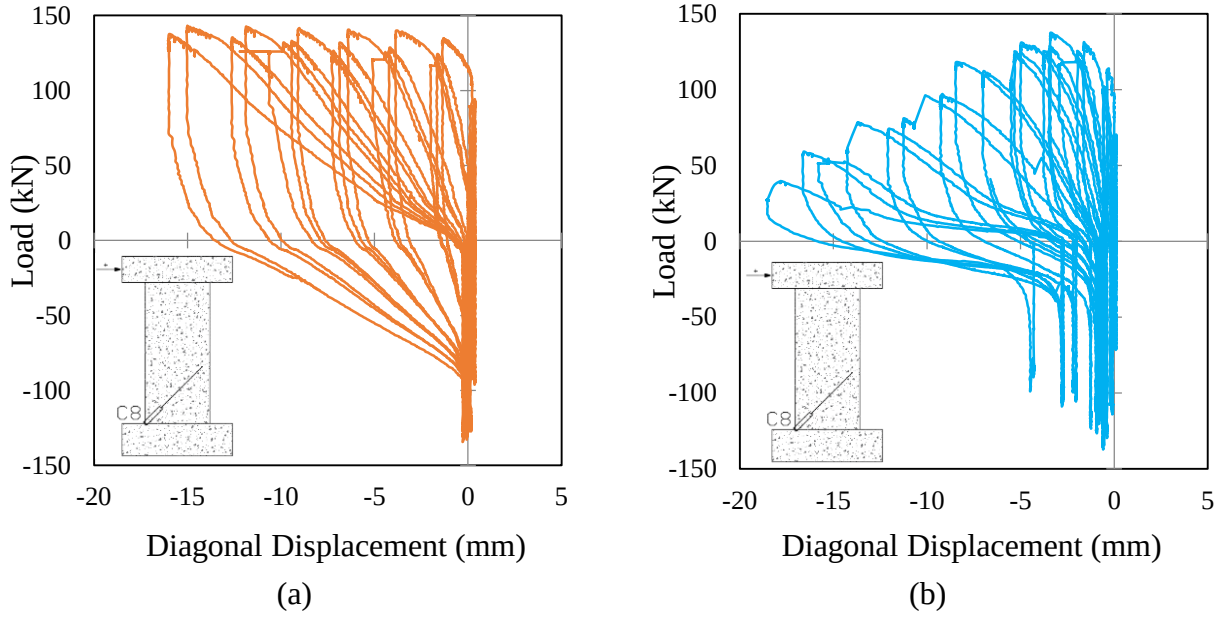


Figure 4.17: Diagonal Displacement: (a) RW1-SR; and (b) RW2-NR

For Wall RW1-SR, the diagonal displacement at yield was 0.5 mm. Beyond yielding the wall experienced nonlinear response with diagonal cracking resulting in larger diagonal displacements. At the peak lateral load, the diagonal displacement was 15 mm. Beyond this load level the diagonal displacement was not recorded due to pull-out of DCT C8 during testing. During the unloading response the diagonal cracks did not close as the response is almost constant to zero load. In addition, during load reversal, a significant lateral load is required before the wall returns to zero displacement.

The load-diagonal displacement hysteretic response of Wall RW2-NR demonstrated recovery of diagonal displacement beyond yielding and up to rupture of the SMA bars. This self-centering phenomenon led to more pinching than Wall RW1-SR. At yielding, the diagonal displacement was 1.1 mm, while it increased to 3.5 mm at the peak lateral load.

4.7 Reinforcement Strains

Electrical resistance strain gauges were attached to the reinforcing bars throughout the repaired region to measure the strain in the longitudinal and transverse reinforcement during testing. The data recorded from the strain gauges were used to construct lateral load-strain responses.

Figure 4.18 provides the load-strain response based on Strain Gauge B2 located 138 mm from the base of Wall RW1-SR just above the mechanical coupler on the longitudinal reinforcement in the boundary zone. At first crack, the strain recorded by B2 in RW1-SR was 0.017%. After first crack, a sudden increase in strain was evident due to the transfer of forces from the concrete to the reinforcement.

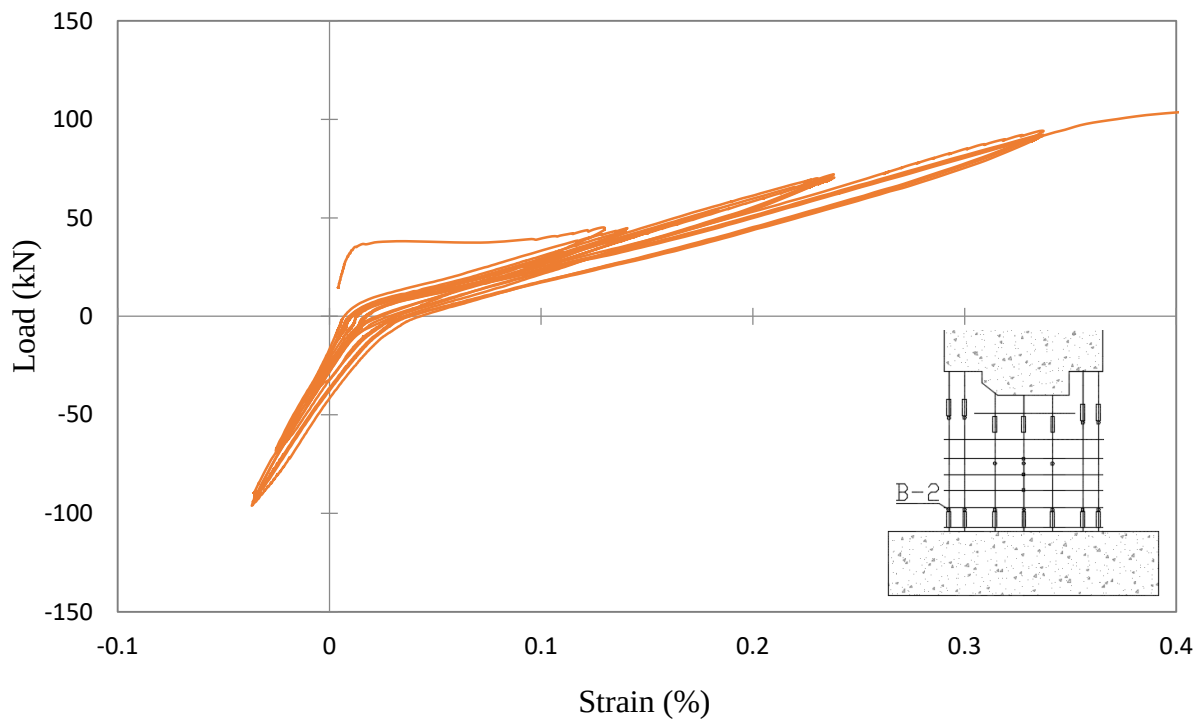


Figure 4.18: Lateral Load-Strain Response of Gauge B-2 in Wall RW1-SR

In Wall RW1-SR, at 138 mm from the base, yielding of the longitudinal reinforcement in the boundary zone and in the web occurred at approximately the same load of 93 kN. This observation

was based on Strain Gauge B-2 (Figure 4.18) and Strain Gauge S-2 installed in the web. The stress-strain response of S-2 is provided in Figure 4.19. Strain Gauge S-7 (Figure 4.20) located in the web, 430 mm from the base, also indicated yielding of the reinforcement at the same yield load. Therefore, yielding of the web reinforcement spread to 430 mm from the base of the wall in RW1-SR.

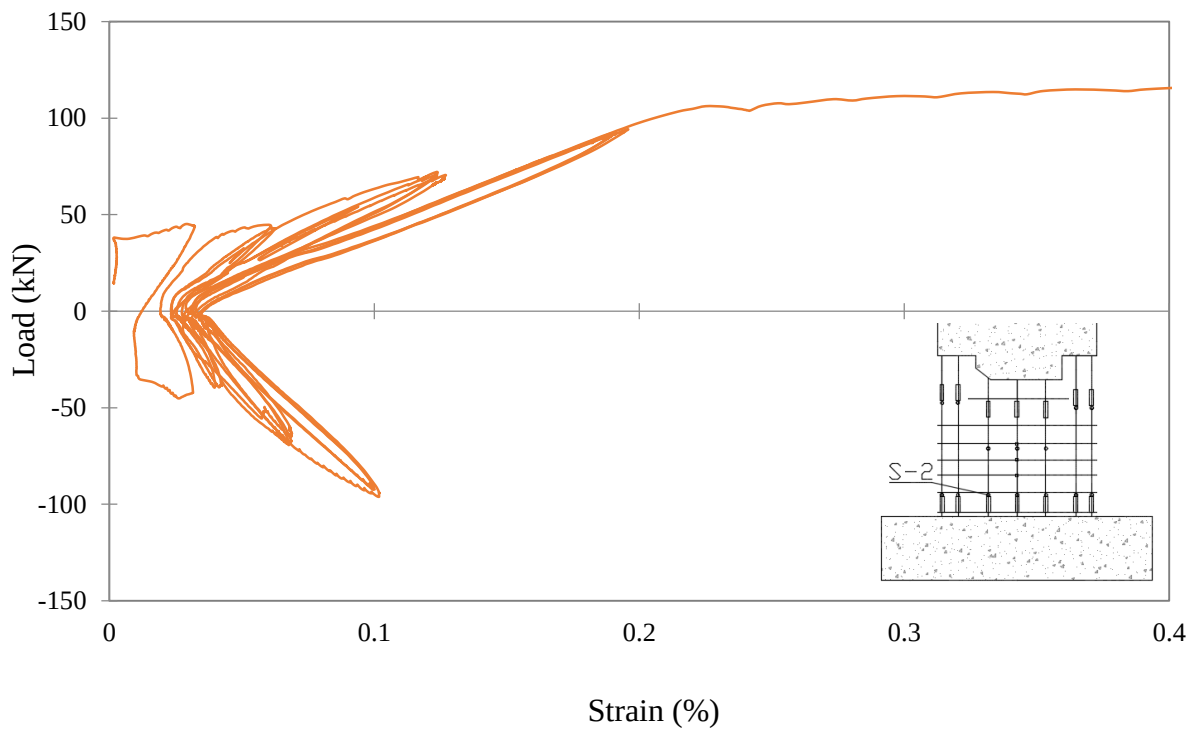


Figure 4.19: Lateral Load-Strain Response of Gauge S-2 in Wall RW1-SR

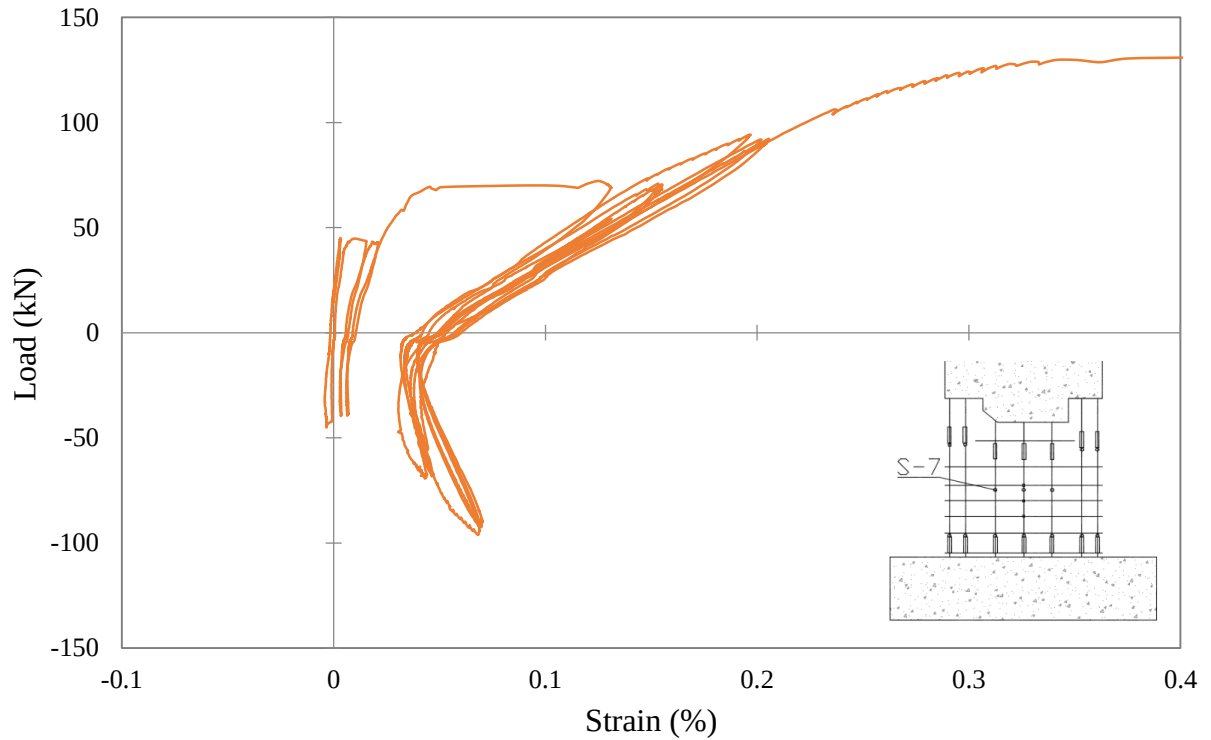


Figure 4.20: Lateral Load-Strain Response of Gauge S-7 in Wall RW1-SR

The strain gauge in Wall RW2-NR attached to the SMA reinforcement nearest to the base was did not provide reliable data. Based on data from Strain Gauge S-6 (Figure 4.21), located in the web of RW2-NR at 204 mm from the base, the reinforcement yielded at 55 kN. Comparing the yielding according to gauge S-6 with yielding of the overall wall, it can be concluded that the web reinforcement at 204 mm yielded prior to yielding of the SMA bars in the boundary. Also note, the web reinforcement in RW2-NR yielded at a lower lateral load than the web reinforcement in RW1-SR. This was due to the lower modulus of elasticity of the SMA bars leading to a lower lateral stiffness of Wall RW2-NR.

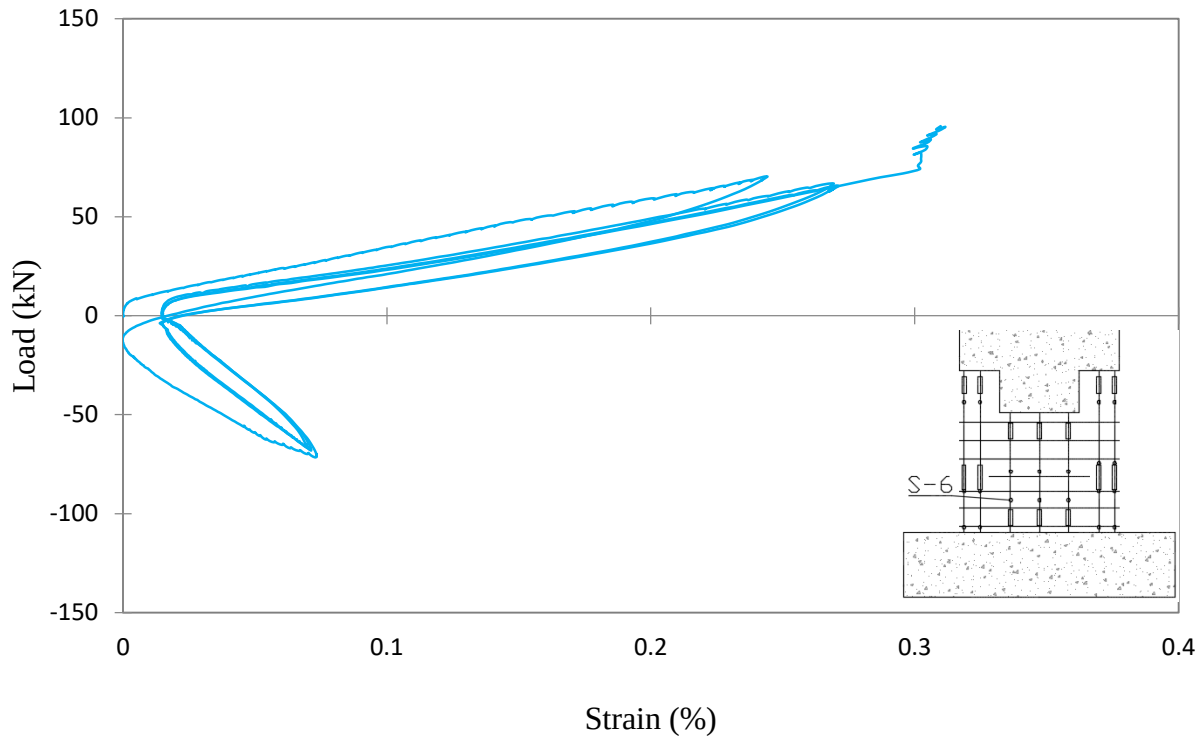


Figure 4.21: Lateral Load-Strain Response of Gauge S-6 in Wall RW2-NR

Based on Strain Gauge S-24 (Figure 4.22), yielding was not experienced in the 15M reinforcing bar located just above the six-inch coupler. The maximum strain recorded by S-24 was only 0.15%. Slightly below the height of Strain Gauge S-24, Gauge S-12 located in the web indicated that the 10M reinforcing bar yielded at 133 kN of lateral load as shown in Figure 4.23. Note that this lateral load corresponds approximately to the peak load sustained by RW2-NR. This demonstrates that plastic hinging remained in the SMA region only and it did not spread beyond the six-inch couplers. Also, S-24 exhibited a load-strain response similar to the stress-strain response of an SMA bar with both forward and reverse transformation (Austenite and Martensite transformation). This was the result of the 15M bar remaining in the elastic range with the SMAs dictating the response. A similar load-strain response was evident with Strain Gauge S-14 (Figure 4.24) located at the boundary, 716 mm from the base of the wall.

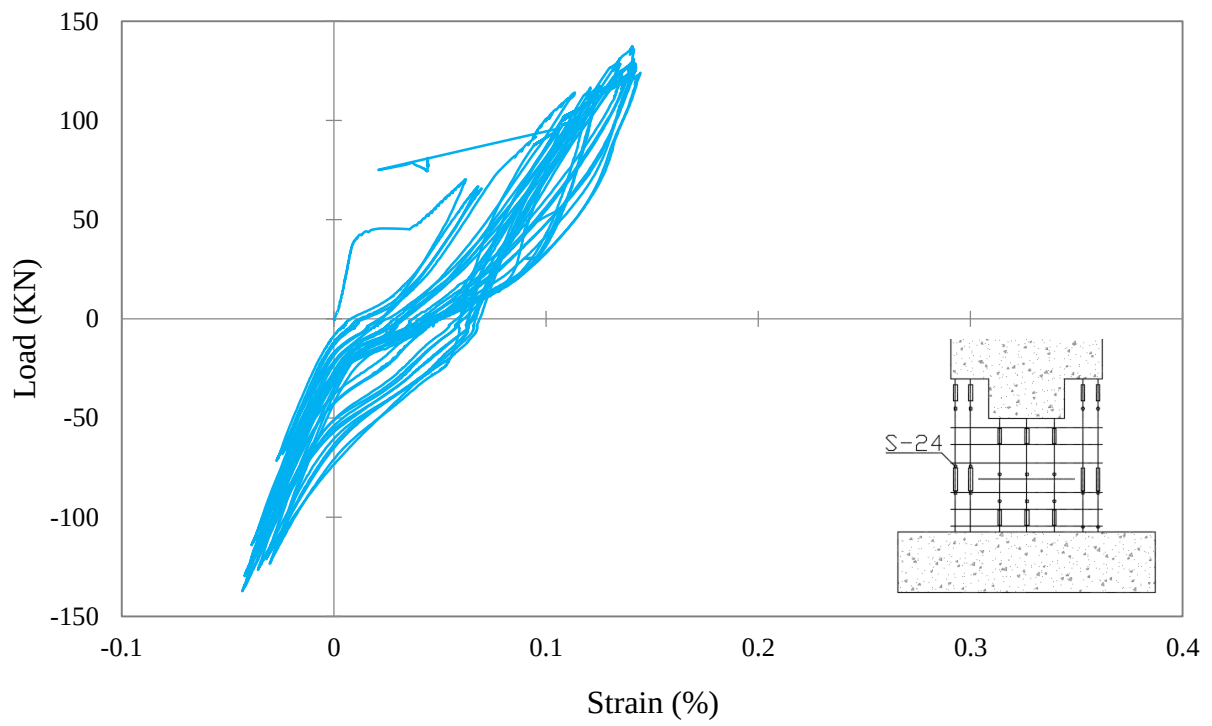


Figure 4.22: Lateral Load-Strain Response of Gauge S-24 in Wall RW2-NR

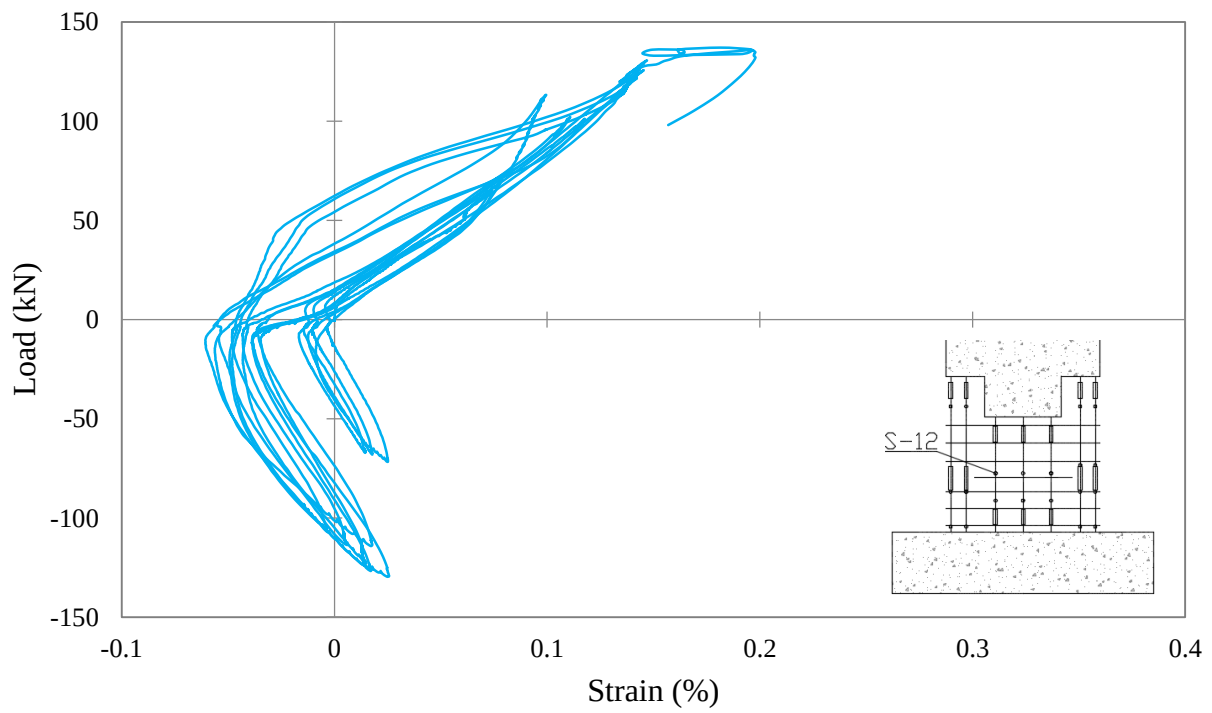


Figure 4.23: Lateral Load-Strain Response e of Gauge S-12 in Wall RW2-NR

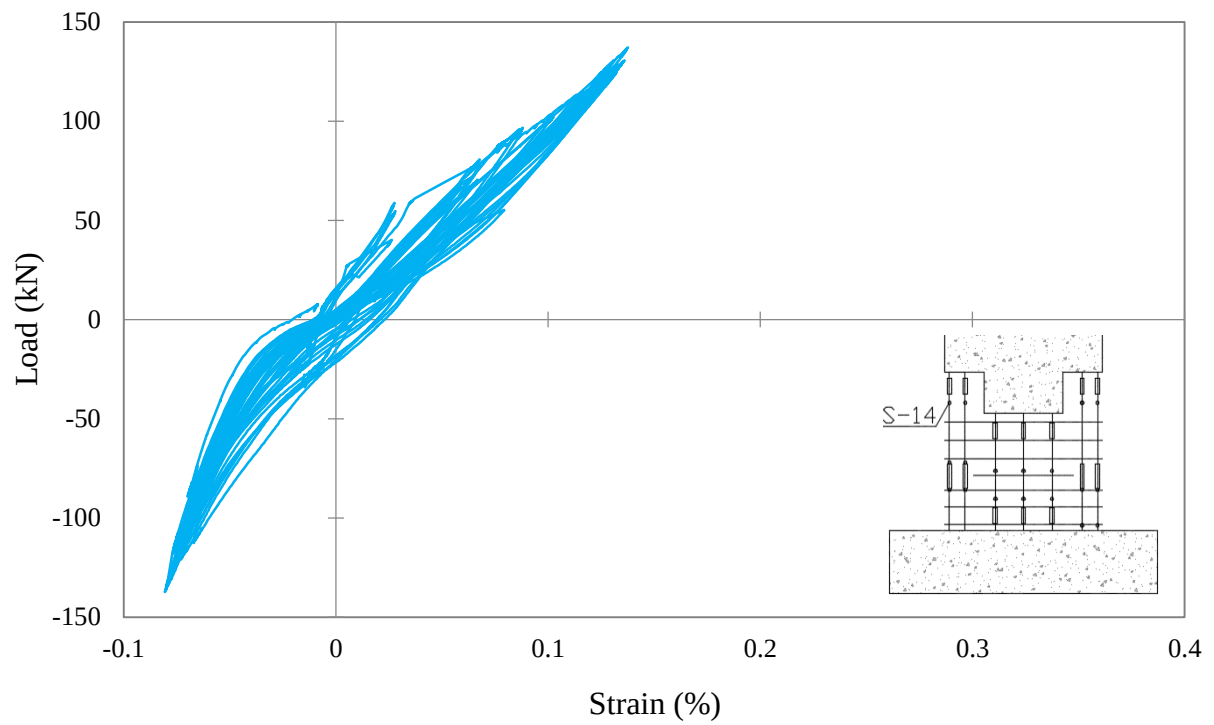


Figure 4.24: Lateral Load-Strain Response of Gauge S-14 in Wall RW2-NR

CHAPTER 5

DISCUSSION OF RESULTS

5.1 Introduction

This chapter discusses the results from testing of Walls RW1-SR and RW2-NR, with comparisons to the experimental results of the original walls (W1-SR and W2-NR) reported by Abdulridha (2013). Specifically, the following structural characteristics are discussed and compared: lateral strength, stiffness, recovery capacity, cyclic degradation, strain energy at peak lateral loads of each wall, energy dissipation, global rotation, and shear strain. Finally, the type of failures for each wall is discussed.

5.2 Structural Characteristics

To evaluate the effectiveness of repairing hybrid-SMA reinforced and conventional steel reinforced walls, the lateral load-displacement hysteretic responses of repaired walls are compared to the responses of the original walls. Figures 5.1 and 5.2 provide the lateral load-displacement hysteretic responses of Wall RW1-SR and RW2-NR, respectively, with the responses of the original walls. The responses of the repaired walls were similar to the original walls. Overall, the original walls experienced greater lateral strength and dissipated more energy compared to repaired walls. Walls RW1-SR and RW2-NR experienced slightly less pinching and slightly wider hysteretic curves compared to W1-SR and W2-SR. In addition, RW2-NR provided a reduced recovery capacity in comparison to W2-NR. The yield and peak loads and displacements for the walls before and after repair are summarized in Table 5.1. Note that the yield forces and displacements reported by Abdulridha (2013) were based on the average of the positive and negative directions of loading. The yield forces and displacements for repaired walls in this research are based on the positive direction only.

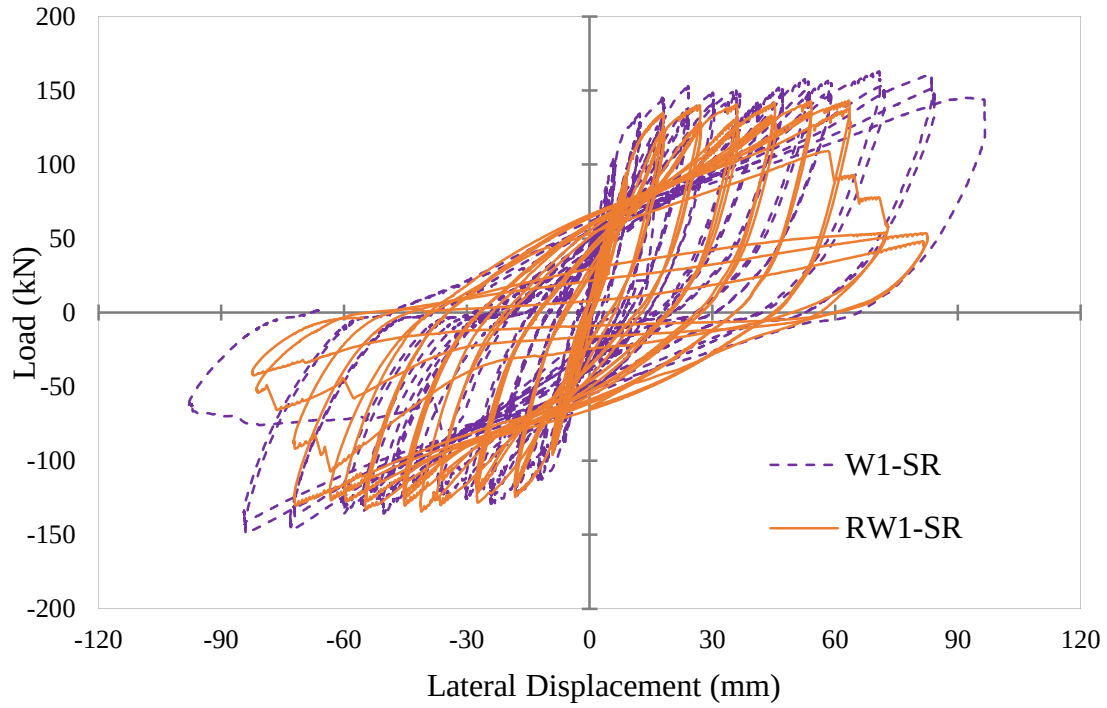


Figure 5.1: Hysteretic Load-Displacement Responses of Walls W1-SR and RW1-SR

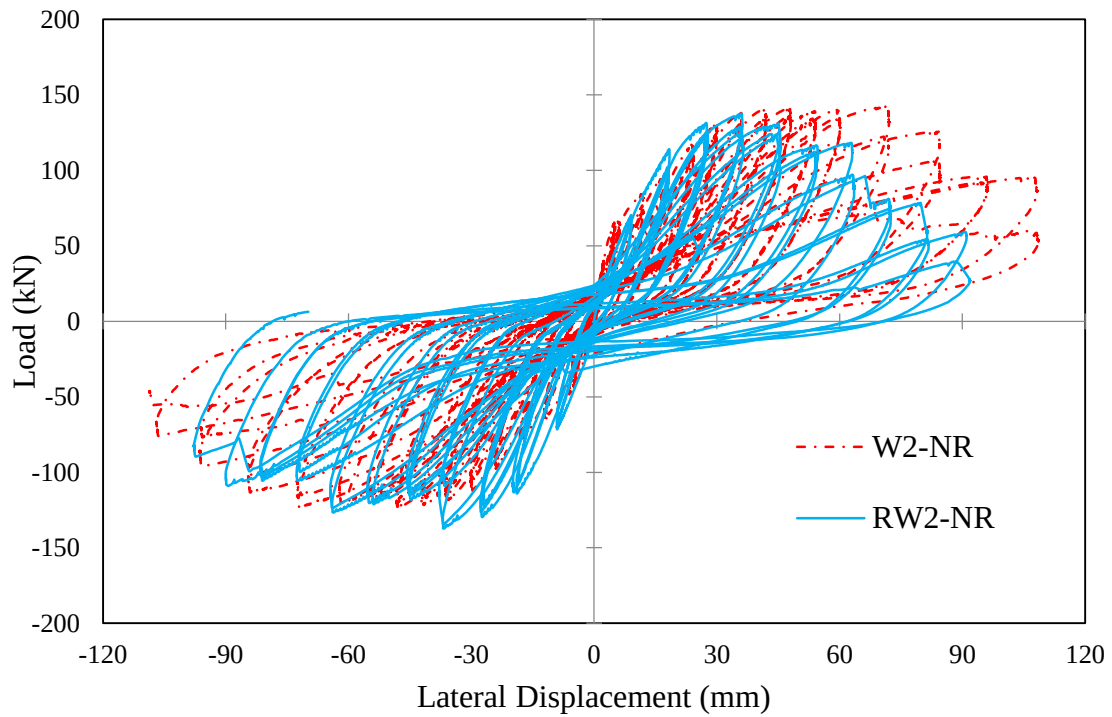


Figure 5.2: Hysteretic Load-Displacement Responses of Walls W2-NR and RW2-NR

Table 5.1: Load and Displacement/Drift Capacities

Wall	Yield			Peak		
	Load (kN)	Displacement (mm)	Drift (%)	Load (kN)	Displacement (mm)	Drift (%)
W1-SR	116	9.4	0.39	156	72	3.2
RW1-SR	123	14.5	0.6	143	65	2.71
W2-NR	112	26.4	1.1	133	72	3
RW2-NR	119	22	0.92	137.5	37	1.54

5.2.1 Lateral Strength

To further compare the results of the repaired and original walls, the load-drift envelope responses are provided in Figures 5.3 and 5.4 for the conventional steel-reinforced and hybrid SMA-reinforced walls, respectively. Note that for clarity, only the first repetition of loading in the positive direction is provided. Figure 5.3 illustrates that the repaired conventional wall (RW1-SR) response was similar to the original wall (W1-SR) up to 2.4% drift, albeit at a reduced strength and elastic stiffness. Thereafter, a sudden decrease in the lateral strength beyond 2.6% drift was experienced by the repaired wall due to rupturing of the longitudinal bars in the boundary region, while the original wall continued to experience a slight increase in strength up to 3.6% drift.

The pre-yielding behaviour of the repaired hybrid-SMA wall (RW2-NR) was very similar to the original hybrid-SMA wall (W2-NR). Beyond yielding, Wall W2-NR maintained the lateral load capacity to 3% drift. However, RW2-NR rapidly reached its peak lateral load capacity at 1.5% drift after yielding. Thereafter, RW2-NR experienced a near linear decrease in strength. The earlier onset of strength reduction in RW2-NR was attributed to fracturing of the SMA bars in the boundary region, whereas the SMA bars in W2-NR did not fracture until after 3% drift. Note that this was a consequence of the shortened SMA bars in RW2-NR and the presence of notches in the

SMA bars due to the connecting screws in the mechanical couplers. This deficiency was also present in W2-NR, but at end of the plastic hinge region.

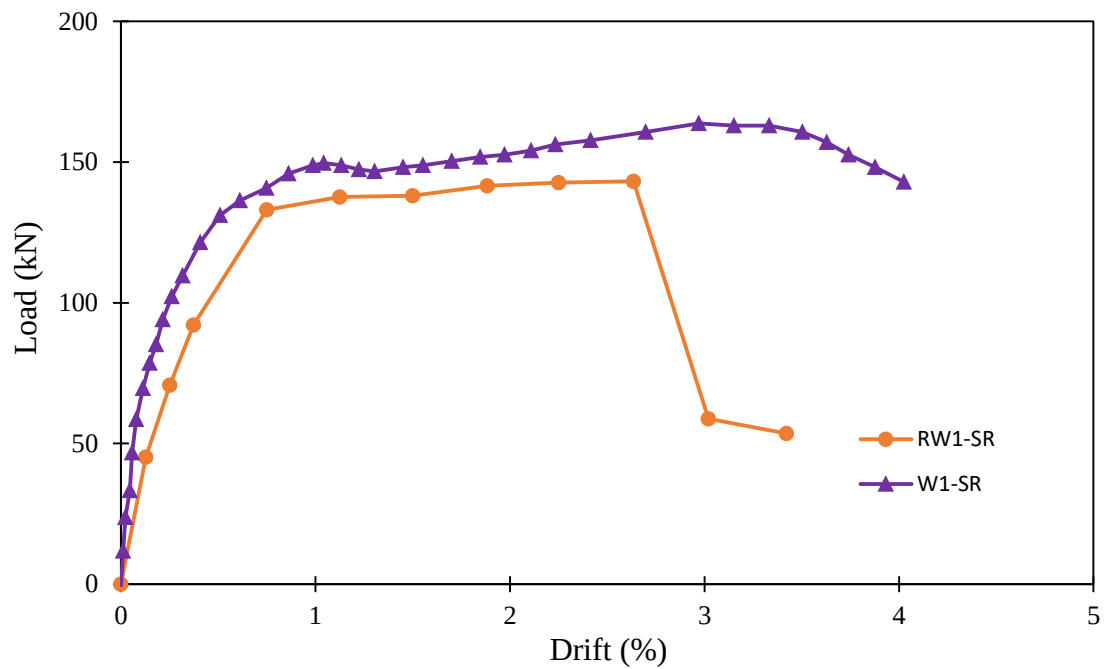


Figure 5.3: Lateral Load-Drift Envelope Responses of Walls RW1-SR and W1-SR

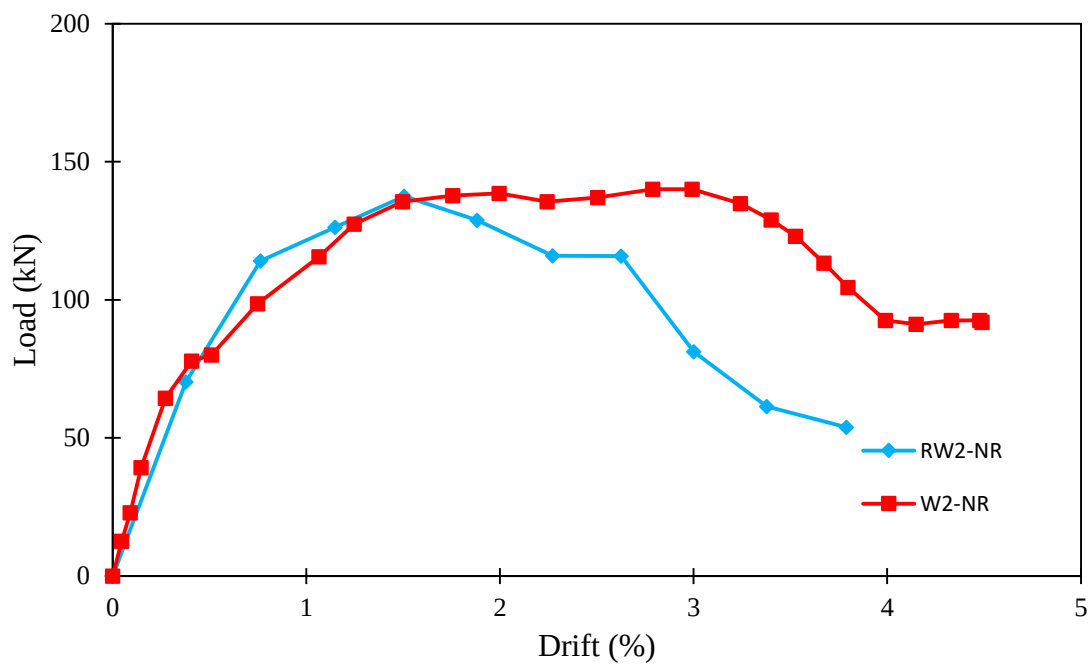


Figure 5.4: Envelope Load-Drift Response of Walls RW2-NR and W2-NR

5.2.2 Stiffness

Figure 5.5 compares the secant stiffness–displacement responses for the original and repaired walls. The secant stiffness is measured by calculating the linear slope to the lateral load and corresponding displacement for the first repetitions of loading in the positive direction. Abdulridha (2013) established an initial secant stiffness of 12.34 kN/mm and 4.24 kN/mm, respectively for the original conventional (W1-SR) and SMA (W2-NR) walls. The repaired walls provided initial stiffness of 11.73 kN/mm and 7.75 kN/mm, respectively for the repaired conventional (RW1-SR) and SMA (RW2-NR) walls. The yield stiffness of the original SMA wall was 35% of the yield stiffness of original steel-reinforced wall. After repairing, the yield stiffness of the SMA wall increased to 66% of the yield stiffness of repaired steel-reinforced wall. This was likely due to shortening of SMA bars in the boundary regions in RW2-NR.

The post-yield secant stiffness of the repaired SMA wall was approximately 30% smaller than the repaired conventional. Similar difference in the post-yield secant stiffness of the original walls was calculated by Alduilridha (2013). This was due to the yielding of the web vertical reinforcement prior to the yielding of the SMA bars in the boundary zone.

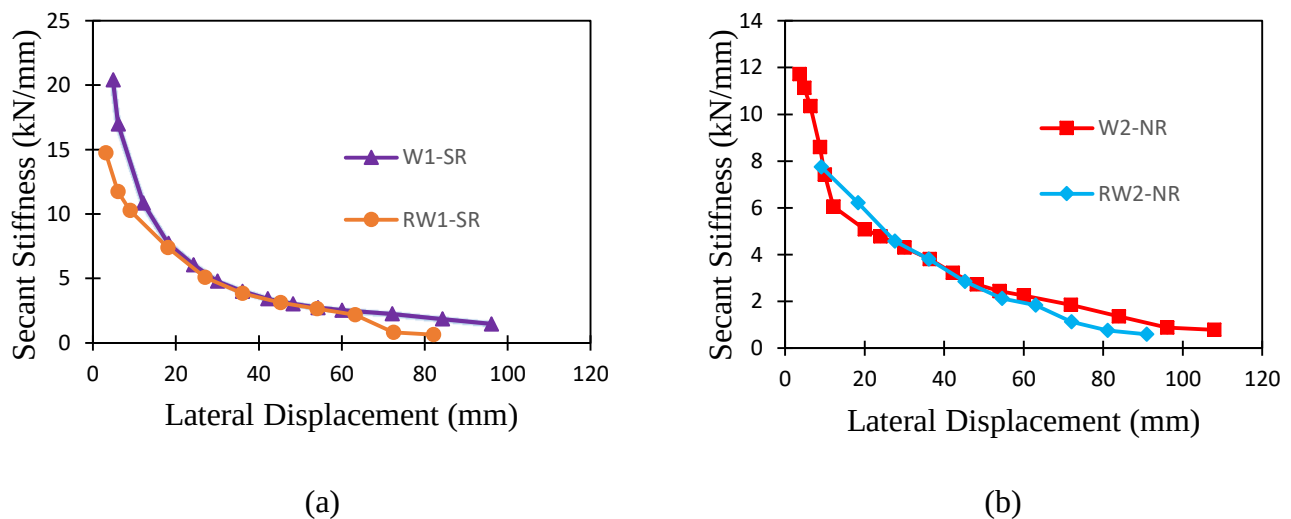


Figure 5.5: Secant Stiffness-Displacement Response: (a) Conventional walls; and (b) SMA walls

5.2.3 Recovery Capacity

Figure 5.6 illustrates the displacement recovery capacity of the original and repaired walls. The recovery was calculated as the ratio of the difference between the peak displacement and the residual displacement to the peak displacement for each displacement level (drift) for the positive direction of loading. The difference between the recovery capacities of the hybrid-SMA and conventional reinforced walls is attributed to the unloading responses of the SMA and conventional steel reinforcing bars (Abdulrida, 2013). The SMA reinforcement experiences reverse transformation during unloading, resulting in recovery of displacement. However, the conventional deformed steel reinforcement experiences a linear unloading response leading to large permanent displacement.

The recovery capacity response of Wall RW1-SR was very similar to Wall W1-SR to 1.26% drift. Thereafter, Wall W1-SR maintained an average recovering capacity of 52% to 2.5% drift, before further reductions in the recovery capacity. However, Wall RW1-SR continued to experience a linear decrease in recovery capacity beyond 1.26% drift. This phenomenon was probably a result of the greater unloading stiffness due to the high strength concrete in the plastic hinge region.

The original hybrid-SMA wall (W2-NR) maintained its recovery capacity of 90% to 100% throughout testing. From such recovery of displacements, it was evident that SMA bars were capable of overcoming the resistance from the conventional deformed steel in the web section of the wall. The repaired hybrid-SMA wall (RW2-NR) with shortened SMA bars also maintained significant recovery capacity of more than 80% up to 2% drift; thereafter a constant decrease in the recovery capacity is evident. This was a result of rupturing of SMA bars in the boundary region. From the results, it can be established that the re-centering capacity was not significantly affected by shortening of the SMA bars in the repaired wall, and deficiencies after 2% drift are attributed

to the couplers imposing a weakness in the SMA bars. Note, however, that a majority of buildings are designed to respond within the 2%-2.5% drift range.

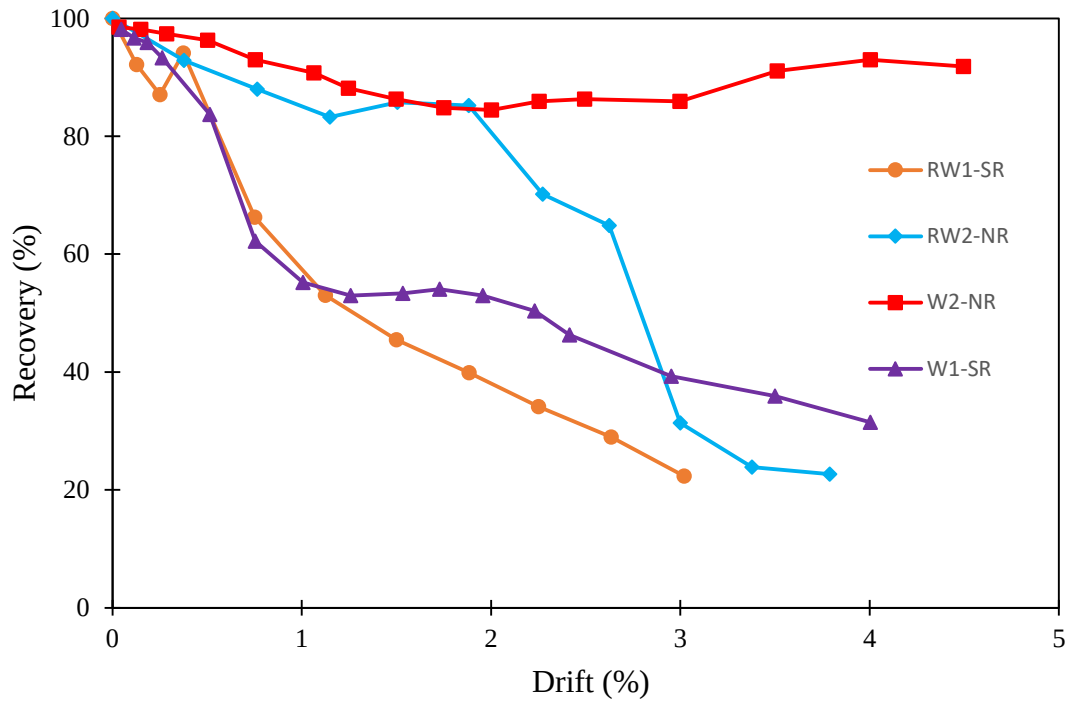


Figure 5.6: Recovery Capacity-Drift Behavior

5.2.4 Cyclic Degradation

Figure 5.7 illustrates the effects of load cycling on the degradation of lateral strength. In the original walls, the second repetition of loading resulted in an average post-yield strength degradation of approximately 6% and 3% in Walls W1-SR and W2-NR, respectively (Abdulridha, 2013). In the repaired walls, the average post-yield strength degradation was 8.7% and 4.6% in Walls RW1-SR and RW2-NR, respectively. The marginal increase in the degradation of strength at second repetition of loading of the repaired walls was due to the use of high strength concrete in the plastic hinge zone. The repetition of loading appears to have caused further damage in the repair concrete than in the original concrete. The significant drops in lateral strength observed in

Figure 5.7 were the result of rupturing of the longitudinal reinforcing bars in the boundary regions and were not accounted for in the calculation of the average lateral strength degradation.

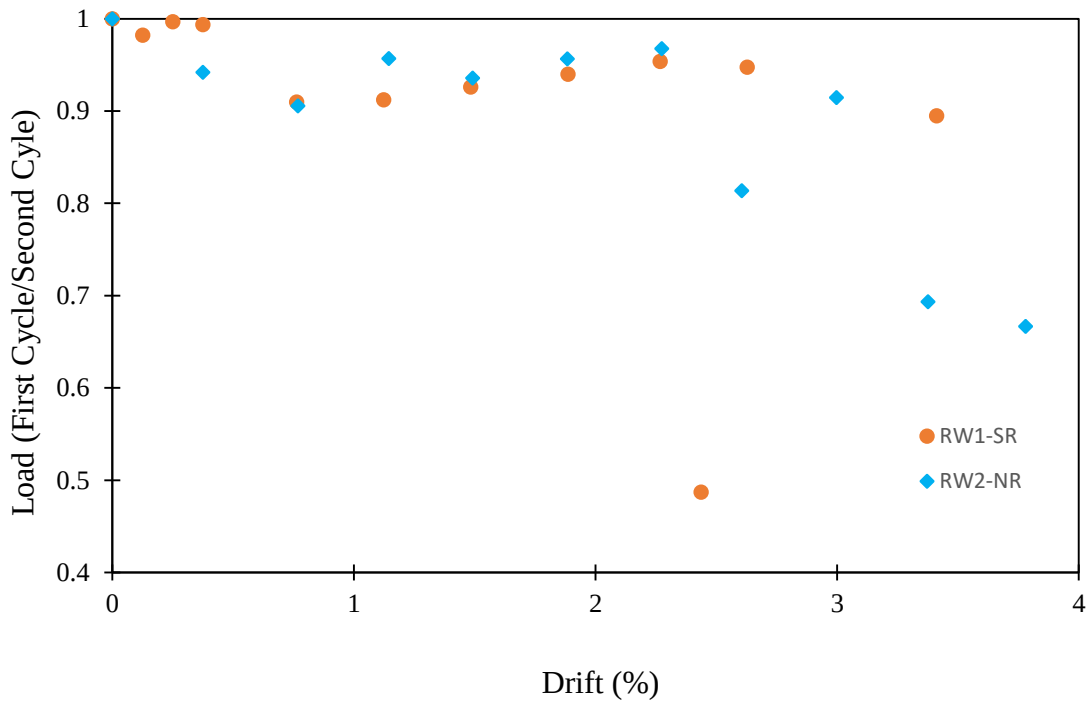


Figure 5.7: Cycling Loading Effect on Degradation of Lateral Strength

5.2.5 Strain Energy at Peak Displacements

The strain energy at 72 mm displacement (peak lateral load experienced by Wall W1-SR and W2-NR) was calculated for the original and repaired walls for the first repetition of loading (Figure 5.8). The original hybrid-SMA wall (W2-NR) dissipated 7750 Nm of energy which represented 60% of energy dissipated by the conventional steel-reinforced wall (W1-SR). The repaired hybrid-SMA wall (RW2-NR) dissipated 7950 Nm of energy, representing 57% of the energy dissipated by the repaired conventional steel-reinforced wall (RW1-SR) and 62% of the energy dissipated by the original steel-reinforced wall (W1-SR). At 72 mm displacement, RW2-NR dissipated 2.5% more energy than W2-NR. However, as illustrated in Figure 5.8, W2-NR had higher lateral strength than RW2-NR at this displacement level. The SMA reinforcing bars in the boundary zone

had ruptured in RW2-NR at 72 mm displacement affecting the strength capacity, reverse transformation stress, and the capacity of the wall to recover displacements, resulting in less pinching and increased energy dissipation.

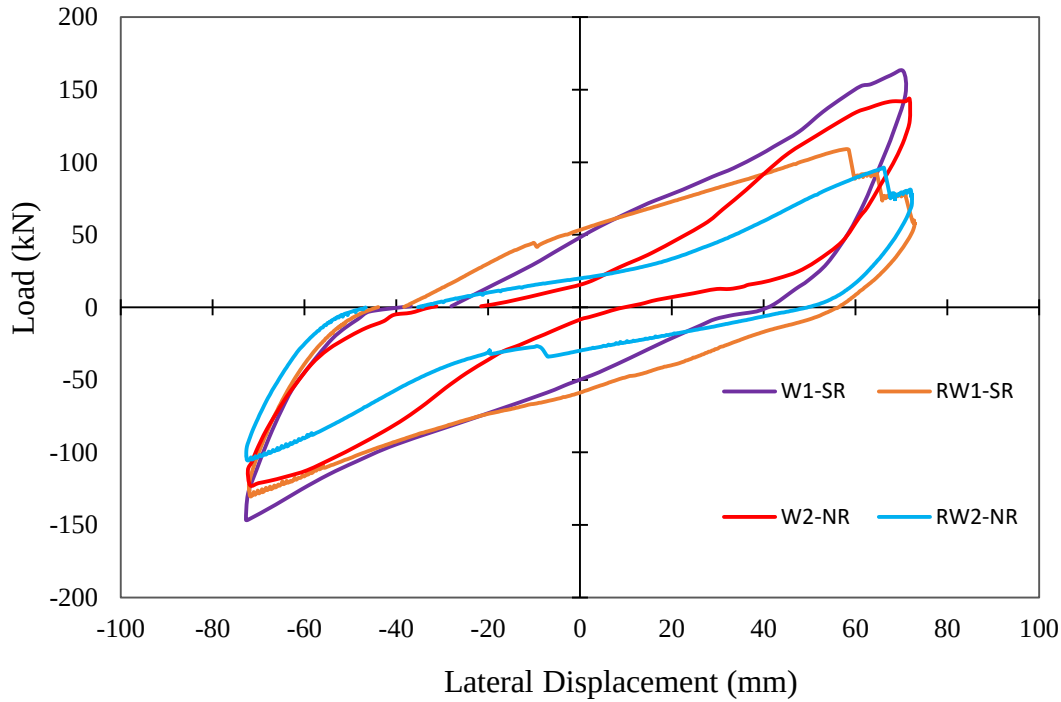


Figure 5.8: Hysteretic Response at Peak Displacement of 72 mm

The lateral load-displacement responses at the peak lateral load experienced by RW2-NR and RW1-SR corresponding to 37 mm and 63 mm, respectively, and the responses of their companion original walls are provided in Figure 5.9. At lateral displacement of 37 mm, the repaired hybrid-SMA wall (RW2-NR) dissipated 29% more energy than the original wall (W2-NR). This difference can be attributed to greater stiffness in the repaired wall during reloading leading to a wider hysteretic cycle. It is most probable that this was attributed to the stiffer repair concrete used in the plastic hinge region. This difference was also noted in the response of the conventional steel

reinforced walls at 63 mm displacement (Figure 5.9 (b)), where the repaired wall (RW1-SR) dissipated 31% more energy than original wall (W1-SR).

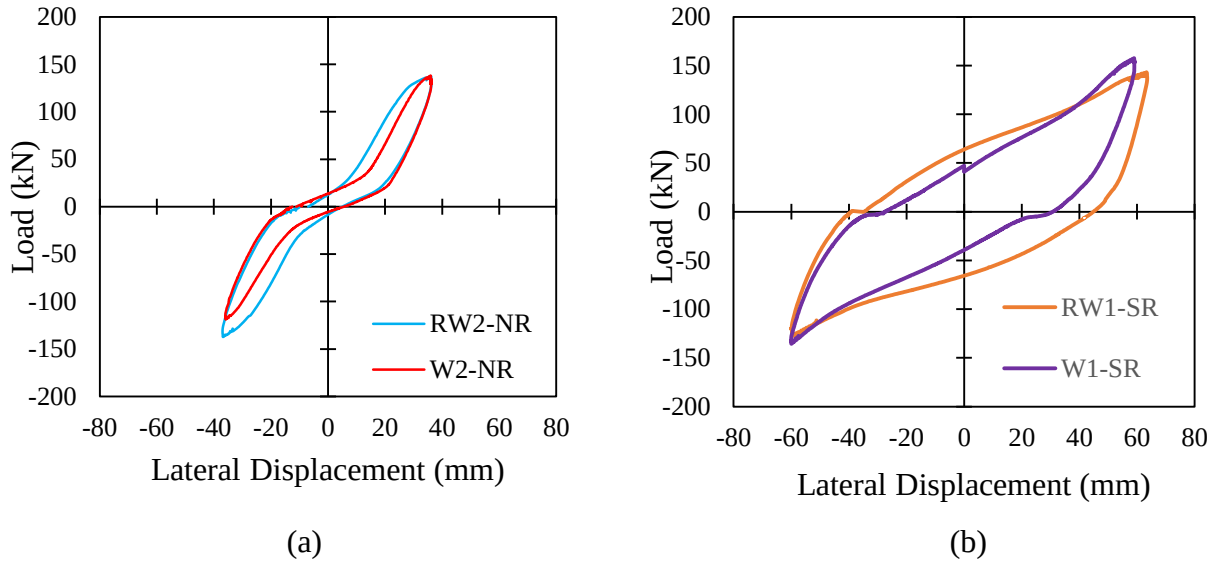


Figure 5.9: Hysteretic Response at Peak Displacement: (a) at 37 mm; and (b) at 63 mm

5.2.6 Energy Dissipation

Figure 5.10 provides the energy dissipation experienced by the walls during the first repetition of loading at each drift level. The conventional walls (W1-SR and RW1-SR) dissipated similar energy up to 1.5% drift, where the both walls dissipated 5792 Nm of energy. Beyond 1.5% drift, RW1-SR dissipated more energy than Wall W1-SR. At 2.5% drift Wall RW1-SR dissipated 1.44% more energy than W1-SR.

For the SMA walls, RW2-NR dissipated slightly more energy than W2-NR up to 3% drift. This is attributed to the increased stiffness in the plastic hinge region as a result of the higher strength concrete, which lead to stiffer reloading response in RW2-NR. At 3% drift both SMA walls dissipated similar energy of approximately 7700 Nm. Beyond 3% drift, however, Wall W2-NR

continued to experience an increase in energy dissipation, while Wall RW2-NR experienced a sudden reduction due to loss of strength at 3% drift. The largest difference in energy dissipation in the SMA occurred at 2% drift, where RW2-NR dissipated 1.44% more energy than W2-NR.

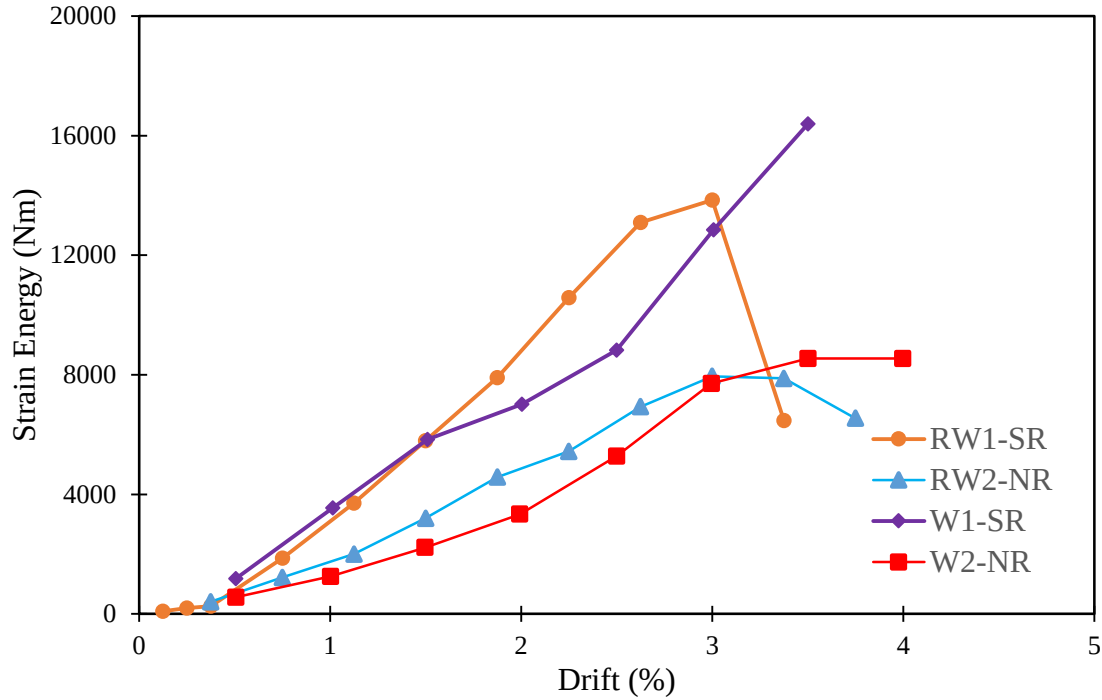


Figure 5.10: Energy Dissipation-Drift Response

5.2.7 Global Rotation

To assess the flexural response of the walls due to concrete cracking and yielding of the longitudinal reinforcement, the global rotation was established from the vertical displacements of the ends of the top loading beam relative to the base foundation. The hysteretic lateral load-rotation response of the top beam of RW1-SR and W1-SR are provided in Figures 5.11 and 5.12, respectively, while Figures 5.13 and 5.14 provide the global rotation of RW2-NR and W2-NR, respectively. The global rotation at yielding was 2.65×10^{-3} rad and 6.0×10^{-3} rad for RW1-SR and RW2-NR, respectively. The rotation at yielding of the original walls noted by Abdulridha

(2013) was 3.1×10^{-3} rad in W1-SR and 11.25×10^{-3} rad in W2-NR. The significant difference in rotation at yielding between RW2-NR and RW1-NR was due to shortening of the SMA bars in the boundary region of RW2-NR which resulted in stiffening of the wall and a reduction of vertical displacements.

At the peak lateral load capacity RW1-SR and RW2-NR experienced rotation of 22.9×10^{-3} rad and 9.8×10^{-3} rad, respectively. Therefore, the inelastic rotation of RW1-SR and RW2-NR were 20.25×10^{-3} rad and 3.8×10^{-3} rad, respectively. The inelastic rotation of the original walls was 24.63×10^{-3} rad for Wall W1-SR and 16.5×10^{-3} for Wall W2-NR. The difference of 12.7×10^{-3} in inelastic rotation between the original and repaired hybrid-SMA wall was due to Wall RW2-NR experiencing its peak lateral load at a significantly lower displacement level than W2-NR.

A subtle difference between the repaired walls (RW1-SR and RW2-NR) is the ratcheting in the response of RW1-SR in the positive direction of loading; whereas, RW2-NR provides a symmetrical response in both directions. It is probable that the re-centering effect of the SMA bars prevented ratcheting in Wall RW2-NR.

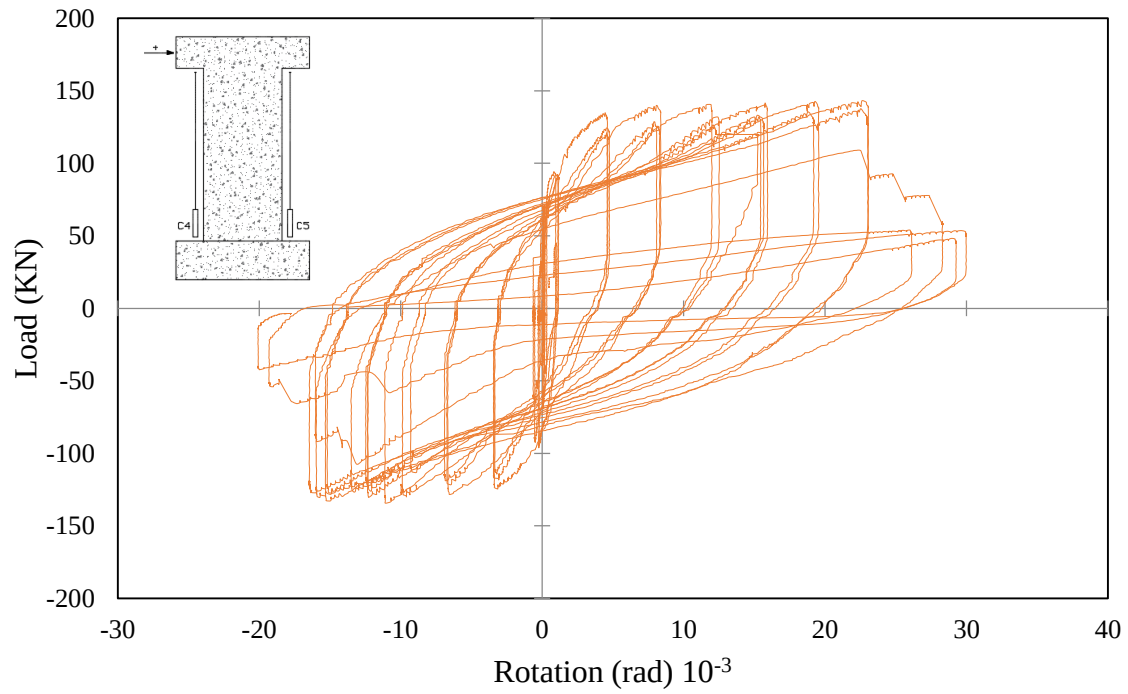


Figure 5.11: Global Rotation of Wall RW1-SR

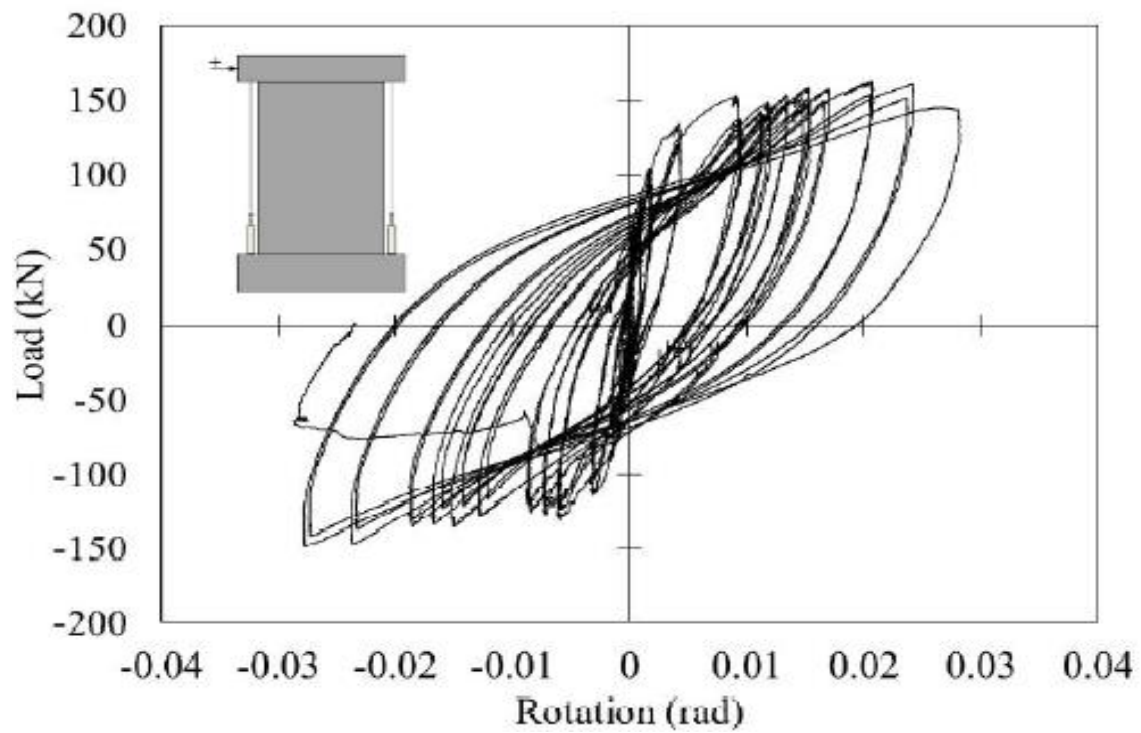


Figure 5.12: Global Rotation of Wall W1-SR (Abdulridha, 2013)

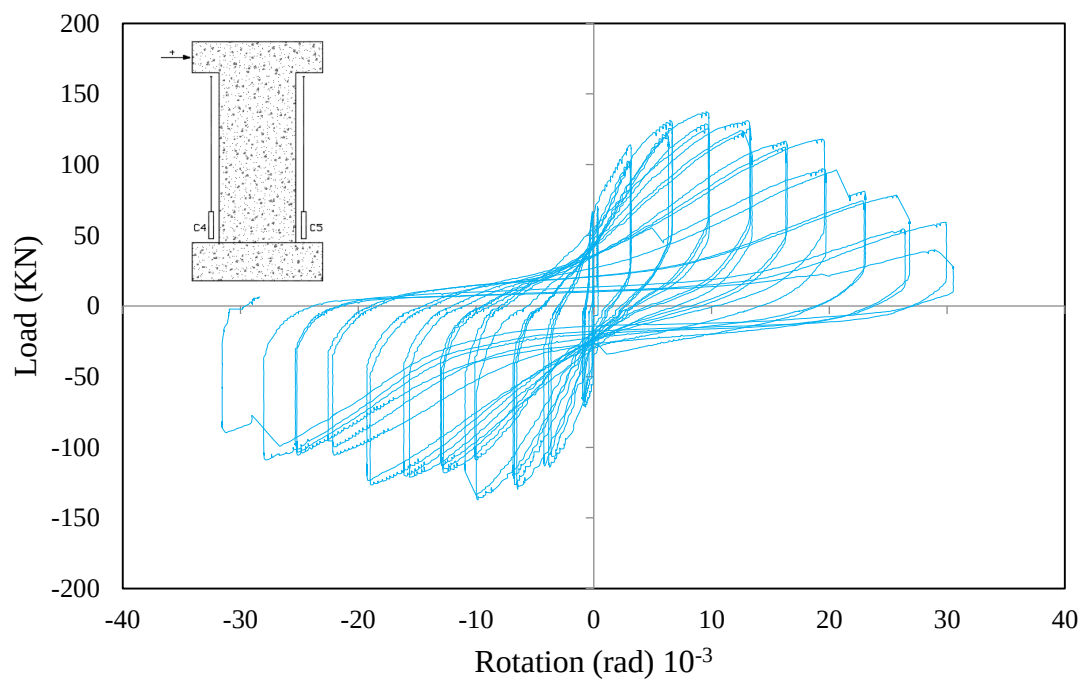


Figure 5.13: Global Rotation of Wall RW2-NR

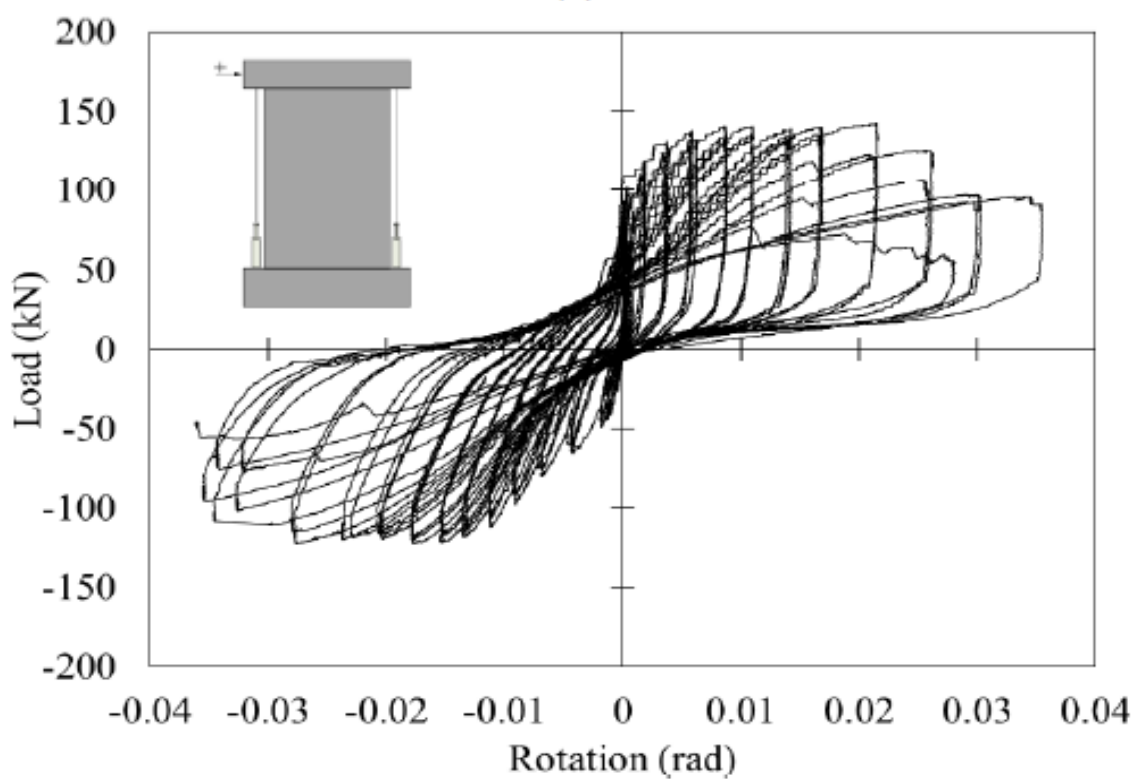


Figure 5.14: Global Rotation of Wall RW2-NR (Abdulridha, 2013)

5.2.8 Shear Strain

An average shear strain for the repaired walls was calculated using the approach suggested by Oosterle (1976), as stated in Equation (5.1):

$$\gamma_{avg} = (\delta_1 d_1 - \delta_2 d_2) / 2hl \quad (5.1)$$

Where γ_{avg} is the average shear strain, d_1 and d_2 are original lengths of the diagonal DCTs, δ_1 and δ_2 are the change in the length of the diagonal DCTs (C8 and C9), h is the height of the diagonal DCTs, and l is the length of the diagonal DCTs. For RW1-SR and RW2-NR, the parameters were the same as the original walls; d_1 and d_2 had lengths of 1265 mm, and h and l were 980 mm and 800 mm, respectively.

The hysteretic lateral load-shear strain response of Wall RW1-SR is plotted in Figure 5.15; while the response of the original wall (W1-SR) is provided in Figure 5.16. Up to yielding, the response of both conventional walls (before and after repair) was linear. Thereafter, both walls experienced non-linear response with additional diagonal cracking. At yielding, the average shear strain of RW1-SR was 0.02×10^{-3} rad, while the average strain of Wall W1-SR at yielding was 0.074×10^{-3} rad. The lower shear strain of Wall RW1-SR at yielding can be attributed to the high strength repair concrete used in the plastic hinge region of RW1-SR leading to a stiffer behavior compared to Wall W1-SR. At the peak lateral load, the shear strain of Wall RW1-SR was 1.19×10^{-3} rad, while Wall W1-SR experienced at the peak lateral load capacity a shear strain of 1.11×10^{-3} rad.

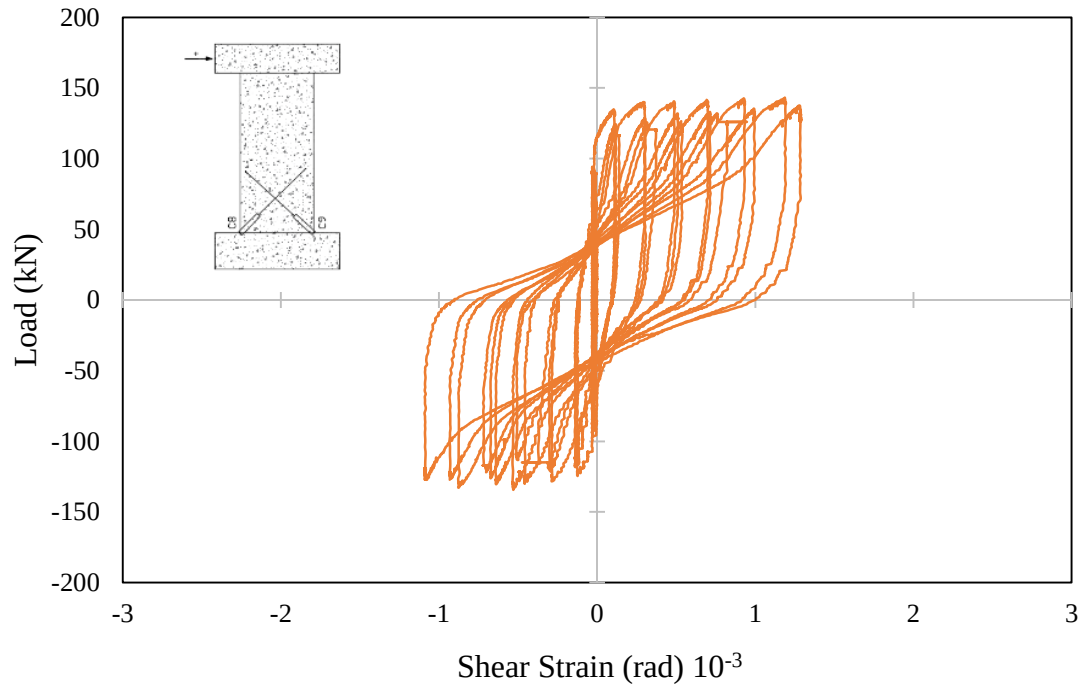


Figure 5.15: Load-Shear Strain Response of Wall RW1-SR

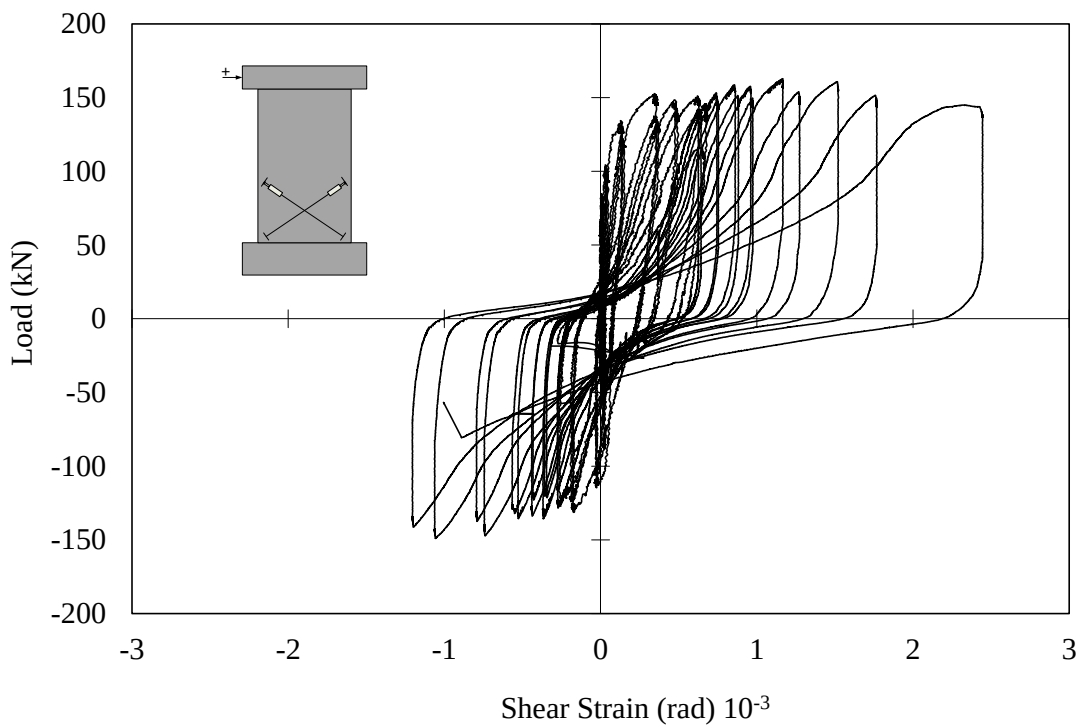


Figure 5.16: Load-Shear Strain Response of Wall W1-SR (Abdulridha,2013))

The lateral load-shear strain response for Wall RW2-NR is plotted in Figure 5.17, while the response for W2-NR is provided in Figure 5.18. The response of both walls prior to yielding was the same as the response of conventional walls. At yielding, the shear strain of Wall RW2-NR was 0.05×10^{-3} rad, and for Wall W2-NR the shear strain was 0.35×10^{-3} . As discussed for Wall RW1-SR, the shear strain at yielding of Wall RW2-NR was significantly lower than Wall W2-NR due to the use of high strength repair concrete in the plastic hinge region. At the peak lateral load, the shear strain in the repaired SMA wall was 0.225×10^{-3} rad, while the original SMA wall experienced a shear strain of 1.1×10^{-3} . Note, the full shear strain response of Wall RW2-NR was not captured due to early pull-out of DCT C8.

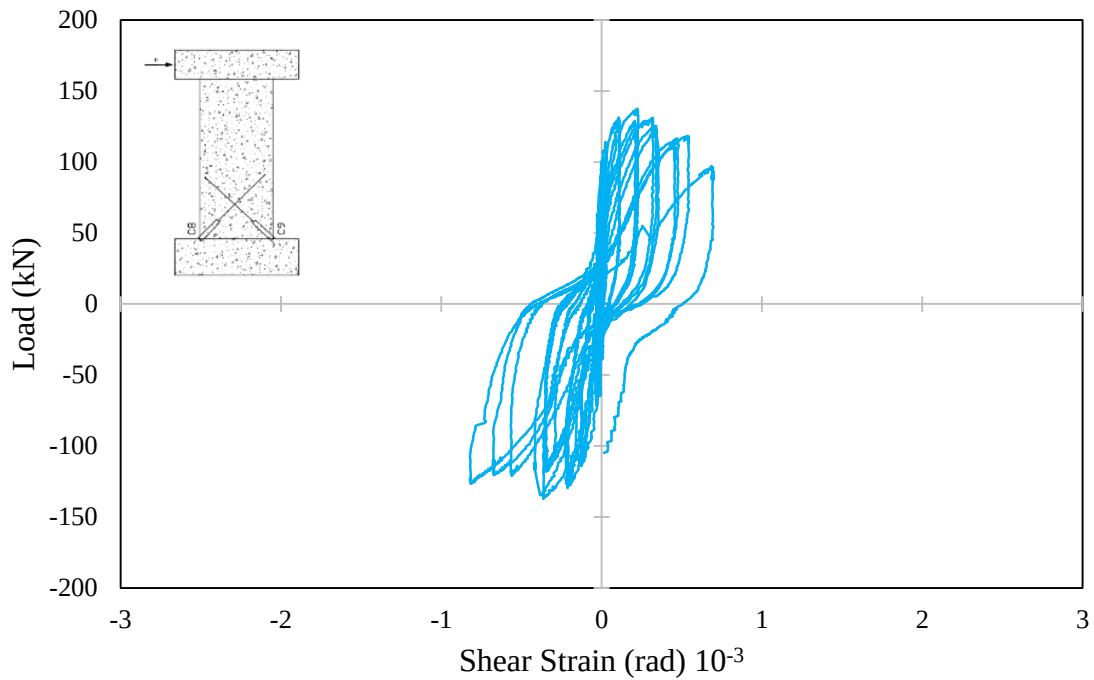


Figure 5.17: Load-Shear Strain Response of Wall RW2-NR

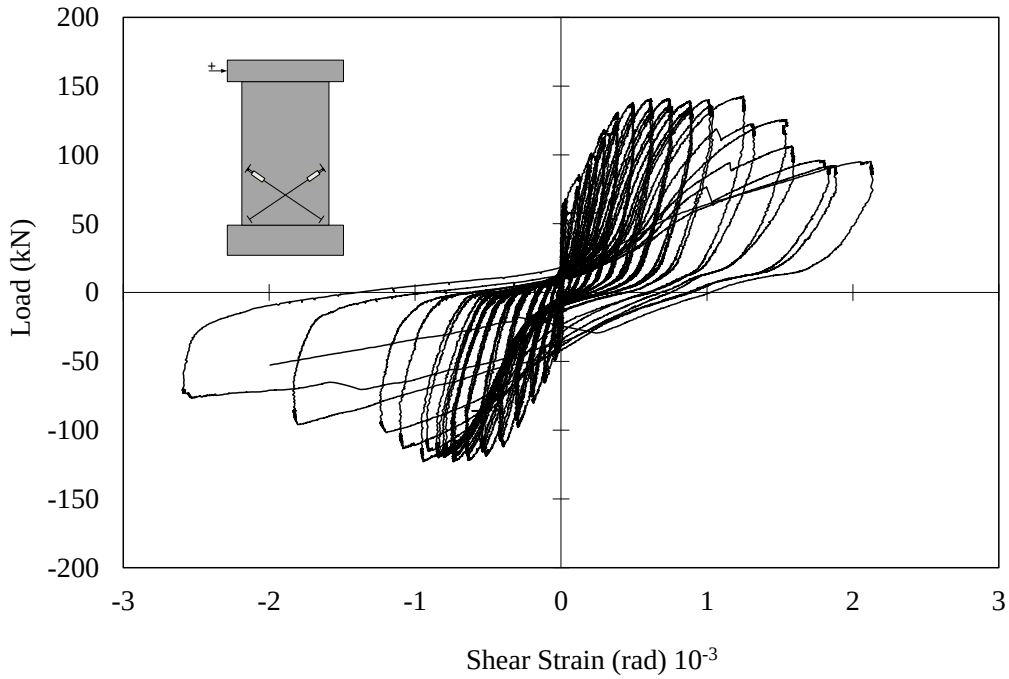


Figure 5.18: Load-Shear Strain Response of Wall W2-NR (Abdulridha, 2013)

5.3 Failure Modes of Each Wall

In the original SMA wall, the length of longitudinal SMA bars in the end boundary regions extended 950 mm from the base foundation into the wall. During repairing, the same SMA bars were shortened such that they extended 424 mm from the base into the wall. After testing Wall RW2-NR, the concrete in the region where the SMA bars connected with the mechanical couplers was removed to examine the state of the shortened SMA bars and to compare to the condition of the SMA bars from the original wall at failure (Figure 5.19).

In Wall RW2-NR, the SMA bars fractured just below the six-inch mechanical coupler as illustrated in Figure 5.19. This was a result of the screws from the coupler introducing a zone of weakness in the SMA bars at this location. In addition, the vertical reinforcing bars in the web fractured and buckled at the same height, adjacent to the mechanical coupler where the SMA bars fractured.

Wall RW2-NR with shortened SMA bars maintained sufficient recovery capacity to 2% drift; beyond this drift level the SMA bars lost significant recovery capacity due to fracturing of the SMA bars. For cases of moderate lateral cyclic loading, shortened SMA bars are adequate. However, to ensure the SMA bars do not fracture in cases of extreme lateral cyclic loading, the mechanical coupling should be provided beyond the plastic hinge region. Otherwise, an improved coupling system should be devised.



(a)



(b)

(c)

(d)

Figure 5.19: Condition of Reinforcement in RW2-NR: (a) Damaged Repaired Zone; (b) Fractured SMA Bar at Left Boundary; (c) Fractured Reinforcement at Web; and (d) Fractured SMA Bar at Right Boundary

Similarly, in Wall W2-NR one SMA bar fractured adjacent to the mechanical coupler due to the same weakness introduced by the mechanical couplers. In addition, partial lengths of the SMA bars in W2-NR had experienced local buckling at the failure location of the wall. In the web, three

vertical bars fractured, while the others experienced buckling. This damage was concentrated along the major crack adjacent to the termination of the starter bars. The fractured and buckled reinforcement at the boundary and web is shown in Figure 5.20.



(a)



(b)



(c)



(d)

Figure 5.20: Condition of Reinforcement in W2-NR: (a) Exposed Reinforcement at Plastic Hinge Zone; (b) Fractured SMA Bar at left Boundary; (c) Fractured and Buckled Reinforcement at Web; and (d) Buckled SMA Bar at Right Boundary

Figure 5.21 shows damage sustained by Wall RW1-SR after testing. The main damage was concentrated just above the bottom couplers where the reinforcement in both the boundary and web fractured. The presence of the couplers resulted in higher stiffness at the base of the wall, thus shifting the failure location just above the coupler. Similarly, this phenomenon was also observed in Wall W1-SR where the presence of starter bars caused high stiffness resulting in the failure location shifting just above the starter bars.



(a)



(b)

(c)

(d)

Figure 5.21: Condition of Reinforcement in RW1-SR: (a) Damaged Repaired Zone; (b) Reinforcement in Left Boundary Zone; (c) Fractured Reinforcement in Web; and (d) Fractured Reinforcement in Right Boundary Zone

The longitudinal reinforcement in the boundary zones of Wall W1-SR experienced both fracture and buckling (Figure 5.22). The left boundary zone experienced significantly more buckling than the right zone, while at the right boundary zone all the longitudinal reinforcement fractured.

However, in the web only the buckling of longitudinal reinforcement was evident. All the damage in the boundary zones and web was concentrated adjacent to the termination of the starter bars.



(a)



(b)



(c)



(d)

Figure 5.22: Condition of Reinforcement in W1-SR: (a) Exposed Reinforcement at Plastic Hinge Zone; (b) Buckled Reinforcement in Boundary Zone; (c) Buckled Reinforcement in Web; and (d) Fractured Reinforcement in Boundary Zone

CHAPTER 6

RELIABILITY ANALYSIS OF DUCTILE SHEAR WALL

6.1 Introduction

In order to conduct the reliability analysis of the ductile hybrid-SMA shear wall, in regard to failure due to the fracture of reinforcement and crushing of concrete, extensive amount of experimental data and its probabilistic interpretation are required for completely describing the behavior of these walls in both stages: before and after repairing. The concept of structural reliability applied to hybrid structural elements is complex especially when the experimental data is used for determining the descriptors of the random variables involved in the failure functions. Developing such investigation would be the focus of an entire research project. Also, due to the lack of sufficient probabilistic data, the current chapter will provide only a framework of the estimation procedure for the reliability indexes and probabilities of failure associated with the hybrid-SMA shear walls tested.

6.2 Ductile Shear Walls

Reinforced concrete shear walls are the main components in a structure that resist lateral forces due to wind and earthquake. Depending on the height to width ratio, a shear wall can be characterised as a slender wall, squat wall or combination of both slender and squat walls. Shear walls, having the height to width ratio greater than 2.0, are known as slender wall. For these types of walls, the flexure strength governs the design, while shear is negligible. On the other hand, for the squat walls, the shear strength is the main design criteria, and flexure is negligible. In the current research, all the wall specimens tested were slender walls, therefore the flexure strength governs the design. During the application of the lateral loading, after a few cycles, horizontal

cracks developed at the lower hinging region of the wall. Beyond yielding point, the fracture strength of the wall is restricted by the fracture of the main reinforcement at the wall edges, caused by the alternation of tension and compression of the reinforcement, induced by the cyclic fatigue. Therefore, in this study, the reliability of the shear walls is analyzed for flexural behaviour.

6.3 Approach to Reliability Analysis

6.3.1 Constitutive Laws

To conduct the structural reliability analysis for the tested shear walls, as a first step, developing the limit state function, which describes the failure mechanism of the wall, is required. For flexure members, the resistance function can be estimated through sectional analysis. To conduct sectional analysis for shear walls in this research study, the following constitutive laws for concrete and reinforcement were applied: the nonlinear model suggested by Hognestad (1955) for concrete (Figure 6.1), and the stress-strain model of SMA bars suggested by Abdulridha (2013), as illustrated in Figure 6.2. For the regular reinforcement the conventional model was used (Figure 6.3).

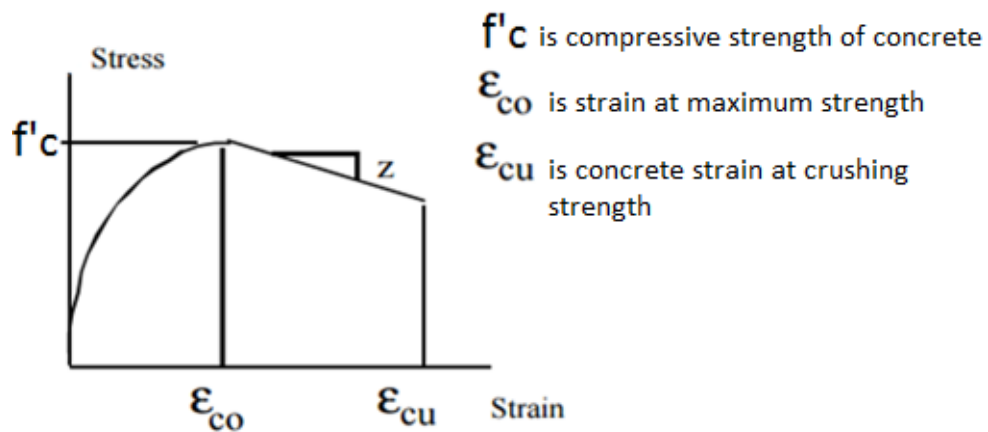


Figure 6.1: Stress- Strain Response of Concrete

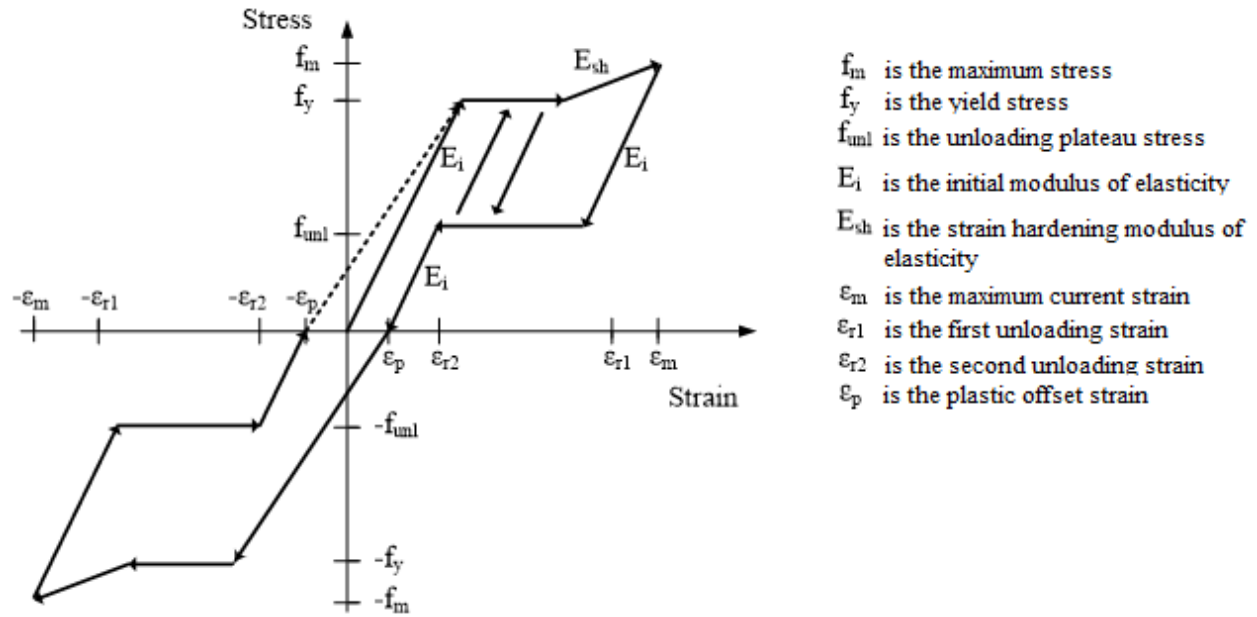


Figure 6. 2: Stress- Strain Response of SMA from Right Boundary (Abdulridha, 2013)

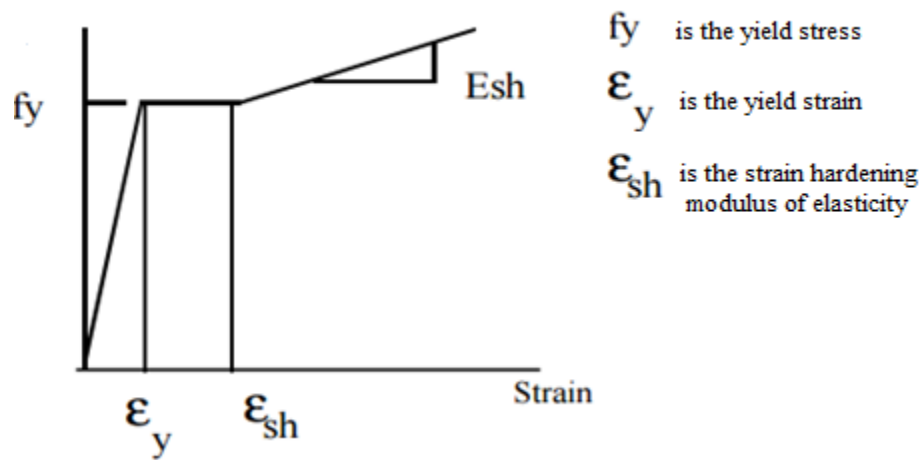


Figure 6.3: Typical Stress- Strain Response of Steel Reinforcement

6.3.2 Rectangular Stress Block Method

6.3.2.1 Stress Block Method for Normal Strength Concrete

After the stress-strain models used for the current investigation were defined, all possible flexural failures modes were analyzed. The sectional analysis was conducted by defining the depth of the neutral axis for the defined flexural failure, and the software Response 2000 (Bentz and Collins, 2004) was used to find the neutral depth “ c ”. The strain developed in the reinforcement bars was determined from the strain compatibility condition, assuming perfect bond between concrete and reinforcement bars. For SMA bars this assumption should be altered, due to the smooth surface of the SMA bars; however for this reliability study, it is assumed a perfect bond between the SMA bars and the concrete material. This allows the use of the ultimate stress block method (see Figure 6.4) defined in the Canadian design code (CSA A23.3-04) for computing the compressive force of concrete at ultimate stage. The ultimate stress block parameters are provided in Equations (6.1) and (6.2). For a better illustration of the SMA bar-concrete interaction, an extensive study is required in order to develop a stress block method for a hybrid SMA section, which would take into account the weak bond between concrete and SMA bars.

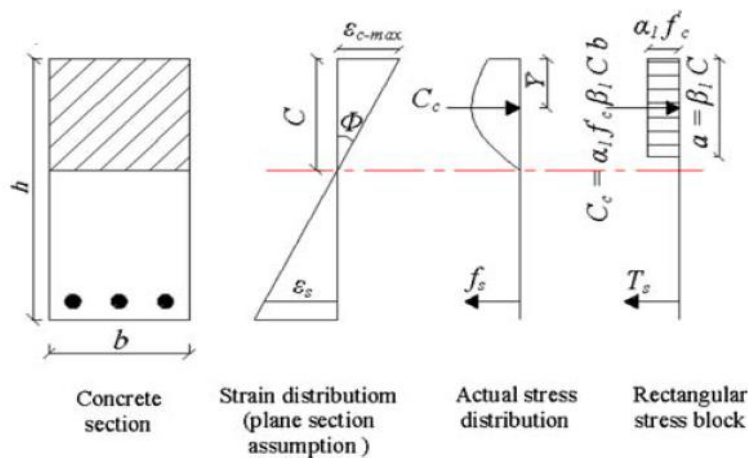


Figure 6.4: Ultimate Stress Block Method (Elbahy, 2009)

$$\alpha_1 = 0.85 - 0.0015 * f'c \geq 0.67 \quad (6.1)$$

$$\beta_1 = 0.97 - 0.0025 * f'c \geq 0.67 \quad (6.2)$$

where:

c is the depth of the neutral axis;

a is the depth of the stress block;

f_s is the current stress of steel;

f'_c is the compressive strength;

ϵ_{c-max} is the rupture strain of concrete;

Φ is the curvature.

6.3.2.2 Rectangular Stress Block for High-Strength Concrete (HSC)

Self-consolidating high strength concrete was used in the repaired regions for both tested Walls RW1-SR and RW2-NR due to the tight formwork. According to Ozbakkalglu (2004) the stress block specified in building codes were developed in the 1950s, based on the test data obtained from normal-strength concrete columns, where the strength usually remained below 40 MPa. Therefore, a rectangular stress block established for normal strength concretes may not be applicable to HSC. Ozbakkalglu (2004) suggested the stress block parameter stated in Equations (6.3) and (6.4) for HSC.

$$\alpha_1 = 0.85 - 0.0014 (f'c - 30) \geq 0.72 \quad (6.3)$$

$$\beta_1 = 0.85 - 0.00200 (f'c - 30) \geq 0.67 \quad (6.4)$$

6.3.3 Limit State Function

The limit state function was developed by establishing the moment resistance function ($M_r(x)$) with the help of sectional analysis (Figure 6.5), where x was considered a random variable. Once the neutral axis depth is defined, the moment is estimated at the depth of the neutral axis, which yields Equation (6.3). $M_r(x)$ is defined as the compression force from the concrete times the moment arm to the neutral axis (Equation (6.4)) and reinforcement tensile/compressive force at different reinforcement layers times the lever arm to the neutral axis (Equation (6.5)). The depths to reinforcement layers are stated in Table 6.1.

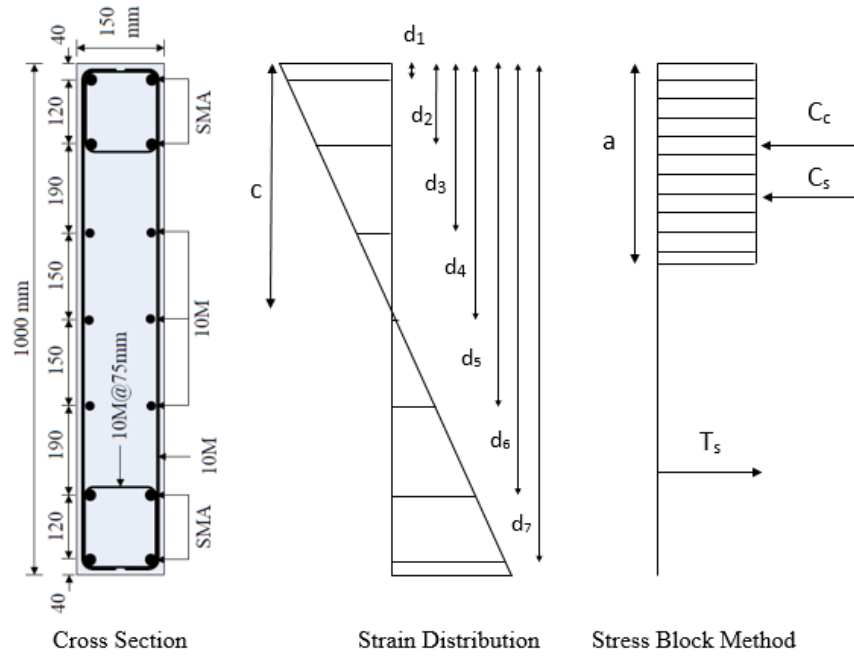


Figure 6.5: Sectional Analysis of Shear Walls (Modified from Abdulridha, 2013)

$$M_r(x) = C_c * (c - a/2) + C_{s1} * (c - d_1) + T_{s2} * (d_2 - c) \dots + T_{sn}(d_n - c) \quad (6.3)$$

$$C_c * (c - a/2) = (\alpha * \beta * b * c * f'_c) * (c - a/2) \quad (6.4)$$

$$T_{sn} * (d_n - c) = (f_s * 2 * A * 0.001) * (d_n - c) \quad (6.5)$$

where:

C_c is the compressive force of the concrete;

C_s is the compressive force in the reinforcement;

n is the respected layer of reinforcement;

T_s is the tensile force at the reinforcement;

d is the depth of the reinforcement layer;

A is the area of the reinforcement.

Table 6.1: Depth to Reinorcement layer

Reinforcement Layer	Depth (mm)
d_1	40
d_2	140
d_3	320
d_4	500
d_5	680
d_6	860
d_7	900

Thus, the $Mr(x)$ function was developed for the limit state function stated in Equation (6.6) which is used as the failure function to conduct reliability analysis. The reliability index was determined for different values of the design moment (Mu) using the software RT (Haukaas and Mahsuli, 2013) and the First Order Reliability Method (FORM).

$$g(x) = Mr(x) - Mu \quad (6.6)$$

6.3.4 Random Variables

One of the most important stages of the reliability analysis is defining the probability distribution function (PDF) and the main descriptors, such as the mean, the standard variation and the

coefficient of variation, for the materials and loads used for the structural element investigated, the shear wall in the current study. The characteristics of the probability distribution function used for the reliability analysis of the tested shear walls, as represented in Table 6.2, were based on the studies performed by Ellingwood. (1980) and Mirza and MacGregor (1982).

The material properties used as random variables in this study are the compressive concrete strength (f'_c), steel yield strength (f_y), SMA yield strength ($f_{y \text{ SMA}}$) and area of reinforcement (A). In order to establish probabilistic characteristics of SMAs, an extensive amount of experimental data would be required, and such study is not available in the literature. Therefore, for this research the probabilistic characteristics of SMAs are considered the same as for the reinforcing steel stated in Table 6.2

Table 6.2: Probability Functions and Main Descriptors (Ellingwood , 1980 and Mirza and MacGregor 1982)

Load type	Mean/nominal	Covariance	Probability function
Compressive concrete strength (f'_c)	1.16	0.14	Normal
Steel yield strength (f_y)	1.115	0.064	Lognormal
SMA yield strength ($f_{y \text{ SMA}}$)	1.115	0.064	Lognormal
Area of reinforcement (A)	1.01	0.04	Shifted Lognormal

6.4 Reliability Analysis of Hybrid-SMA Shear Walls Before and After Repair

The material properties of the tested shear walls W1-SR, W2-NR, RW1-SR and RW2-NR employed in the current reliability analysis are detailed in Table 6.3. Once the material properties and their statistical characteristics (Table 6.2) were defined in the software RT (Haukaas and Mahsuli, 2013), the reliability was conducted using the First order reliability method (FORM).

Table 6.3: Material Properties of the Tested Shear Walls

Material Properties	W1-SR	W2-NR	RW1-SR	RW2-NR
A_s (mm ²)	100	100	100	100
A_{SMA} (mm ²)	-	123	-	123
f'_c (MPa)	30.5	31.6	76	81
F_{yRebar} (MPa)	425	425	480	480
F_{ySMA} (MPa)	-	380	-	305
E_{Rebar} (MPa)	200,000	200,000	200,000	200,000
E_{SMA} (MPa)	-	38 000	-	27 000

Figure 6.3 illustrates the reliability index (β) obtained for all the tested walls (before and after repair), considering the limit state function introduced above (Equation 6.3), while Figure 6.4 provides the probability of failure (P_f) of the shear walls (before and after repair) corresponding to the same limit state function.

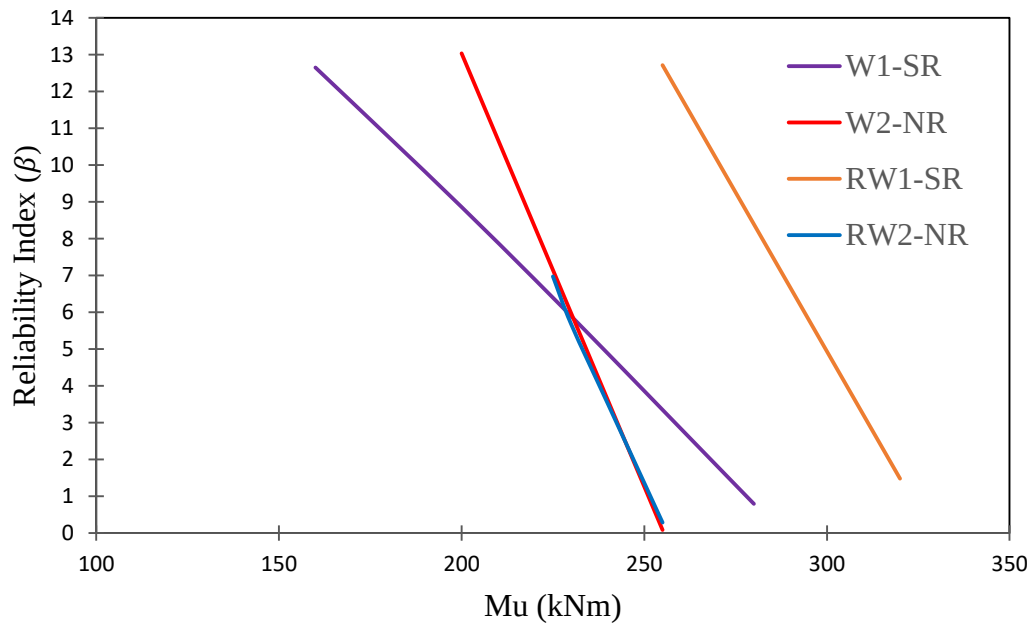


Figure 6. 6: Reliability Index for Design Moment M_u

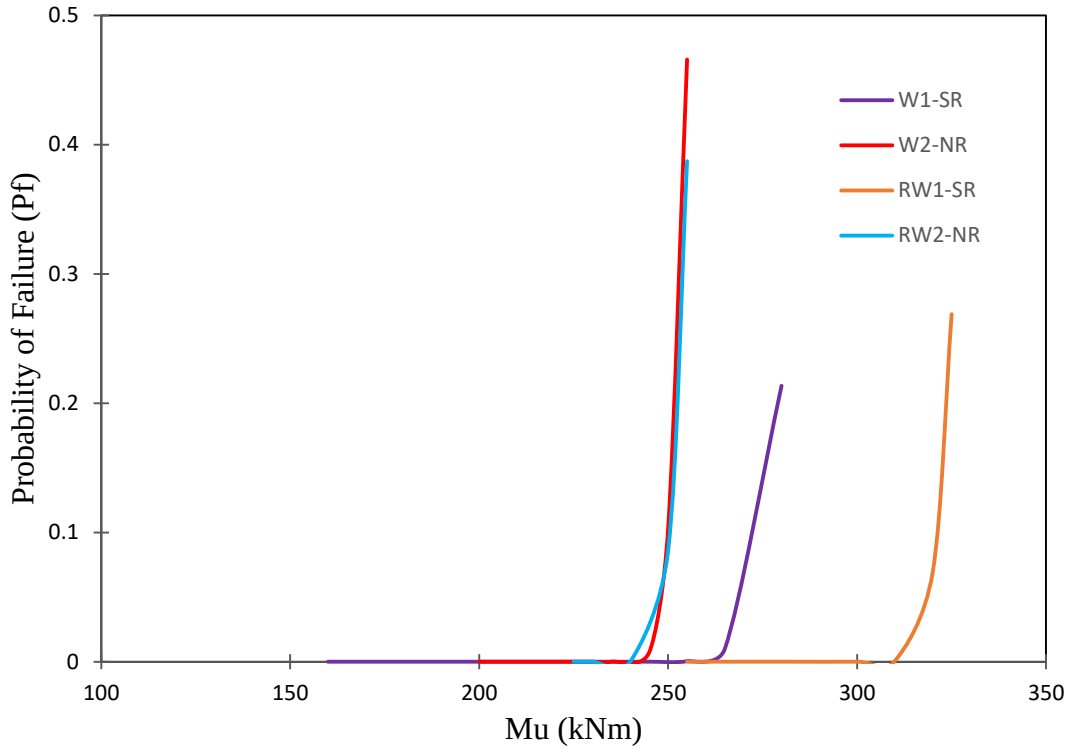


Figure 6.7: Probability of Failure for Design Moment (Mu)

The reliability index varies linearly with the design moment (Mu). As the design moment decreases, the β values increases; in other words, the shear walls designed for lower values of design moment would have a higher reliability index and a lower probability of failure.

The main goal of the designers is to design structures with a target reliability index, which yields the most economical and safe design. The target reliability index can vary depending on the location and type of structure. A structure situated in a high seismic zone would be designed for higher β value. In the case where serious damage and high number of casualties can result due to the failure of massive structures, such as dams and high-rise buildings, the target probability of failure should be even smaller. According to a study by MacGregor (1976), the target reliability index of $\beta = 3.5$ can be used for a ductile structure designed for standard consequences of failure. For consequences of severe failure, or if the failure occurs in a brittle manner, $\beta = 4$ can be used.

For the ductile shear walls tested in this research, the design moments for $\beta = 3.5$ were 254 kNm for Walls W1-SR, 240 kNm for W2-NR, 309 kNm for RW1-SR and 240 kNm for RW2-NR. The values of μ for the walls W2-NR and RW2-NR were similar, but it should be noted that the sectional analysis used to derive the general failure function does not incorporate the coupling connection, the length of SMA bars and the fatigue of the wall segment situated above the repaired section, caused by the initial test. To incorporate these factors that can influence the failure function, a system reliability procedure should be implemented that include combinations of all the failure modes and conditional probabilities of failures. This requires an entire different research project and a detailed interpretation of extensive experimental data, which are beyond the objective of the current study.

Also for $\beta = 3.5$, the design moment for the Wall RW1-SR was greater than that of the wall W1-SR, due to the use of high strength concrete for the later. In the case of the repaired SMA wall, the resistance was governed by the lower yielding and the modulus of elasticity of the SMA bars.

6.5. Statistical Characteristics from Experimental Data

This section provides a frame work for converting the experimental data into probability distributions, in order to conduct reliability analysis for further failure mechanisms. The experimental displacement data at yield point and the ultimate recorded displacement from the test were converted into random variables characterized by probability distribution functions.

However, in order to conduct such analysis, extensive testing of shear walls is required to calculate statistical properties of yielding and ultimate displacement. Therefore, only the framework of the probabilistic investigation is proposed herein. To commence with such analysis from the currently performed tests, an interval of the displacement data near the yielding and the ultimate points for each cycle were identified from the experimental data, considering the assumption that testing of

other shear walls would register yielding and ultimate displacements within the same interval. The experimental yielding and the ultimate displacements of the Walls RW1-SR and RW2-NR are illustrated in Figures 6.8 and 6.9, respectively.

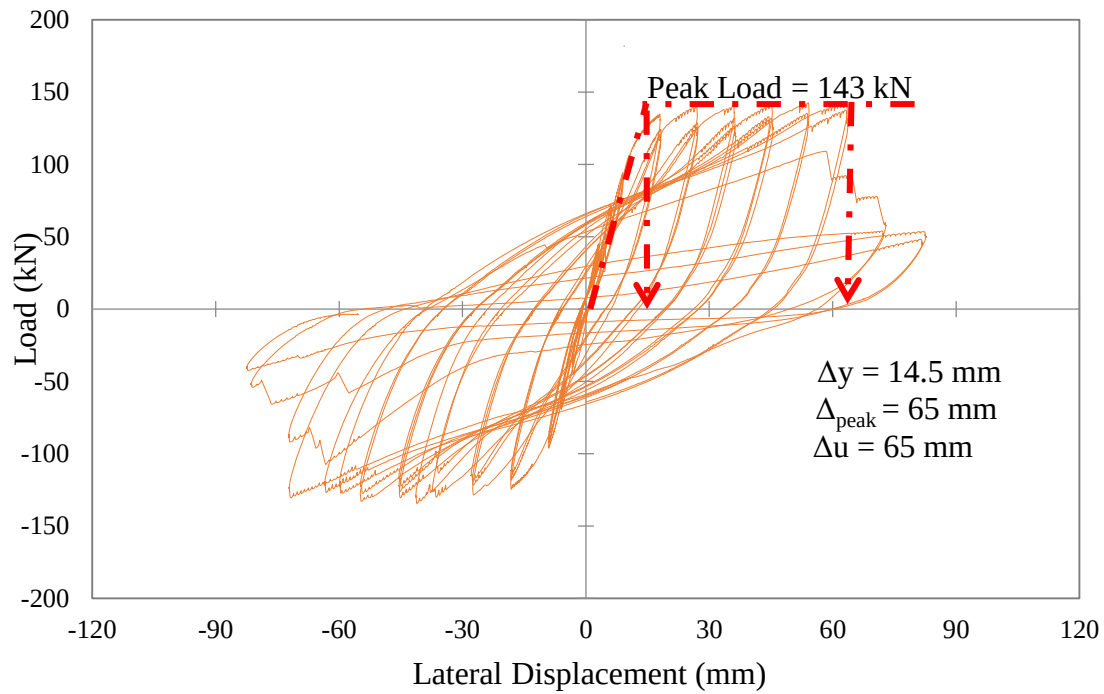


Figure 6.8: Load-Displacement Response of Wall RW1-SR

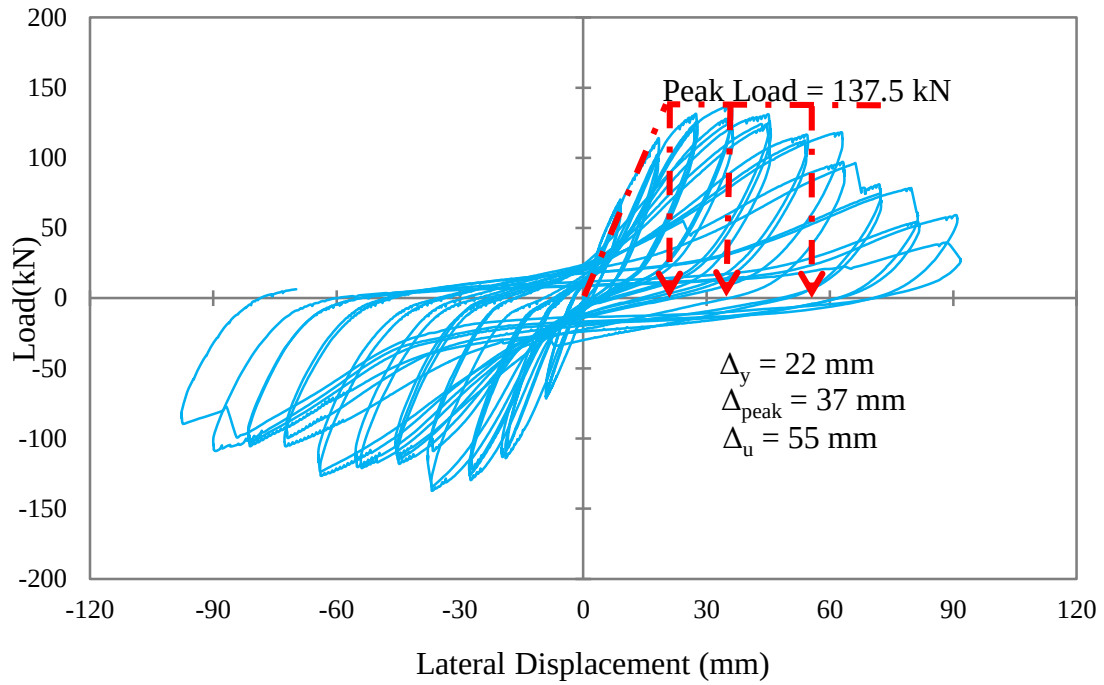
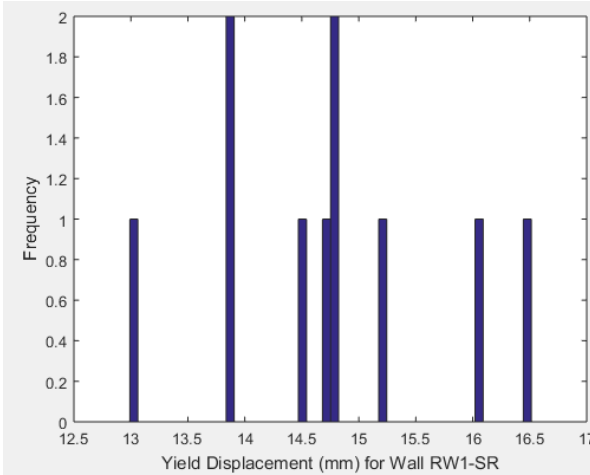
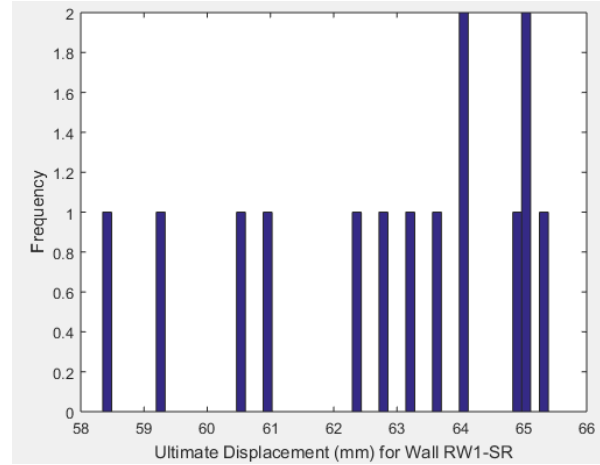


Figure 6.9: Load-Displacement Response of Wall RW2-NR

The selected experimental displacement data was first converted into an histogram representation for better illustration of the data used for the probabilistic procedure. For the Walls RW1-SR and RW2-NR, the histogram for the yield and ultimate displacements were provided in Figures 6.10 and 6.11, respectively.

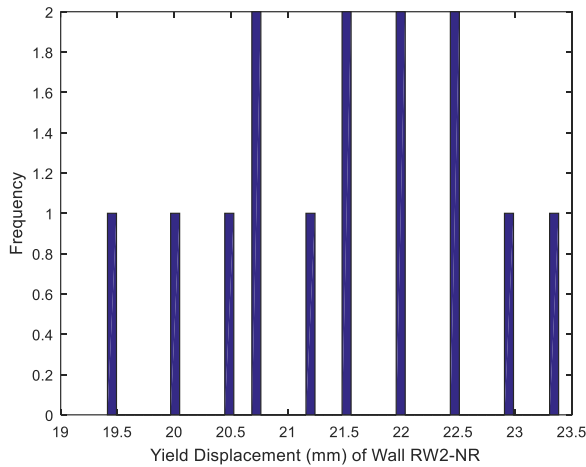


(a)

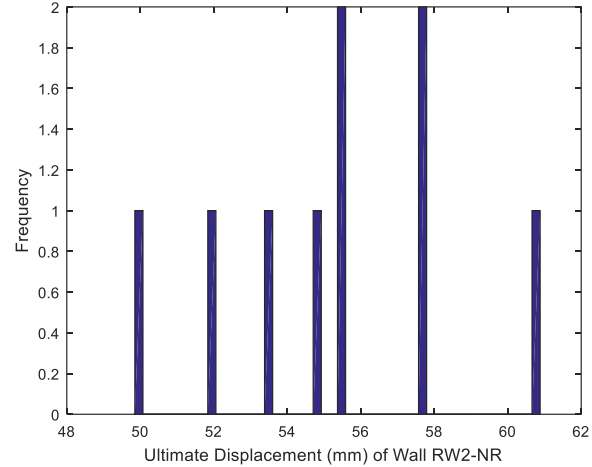


(b)

Figure 6.10: Wall RW1-SR Displacement in Histogram (a) Yielding; and (b) Ultimate



(a)



(b)

Figure 6.11: Wall RW2-NR Displacement in Histogram (a) Yielding; and (b) Ultimate

The module “dfittol” in Matlab software was used for identifying the best fitting probability distribution function (PDF) describing the displacement data obtained from the experimental records of the RW2-NR and RW1-SR tested walls. Several PDF types were considered, from which only Normal, LogNormal and Weibull distributions were a better fit for the displacement

data. Figure 6.12 provides fittings for the distribution of displacement for the Wall RW1-SR and Figure 6.13 illustrated the PDF fitting for the wall RW2-NR. The statistical properties for the displacements distributions of the Walls RW1-SR and RW2-NR are mentioned in Tables 6.4 and 6.5, respectively.

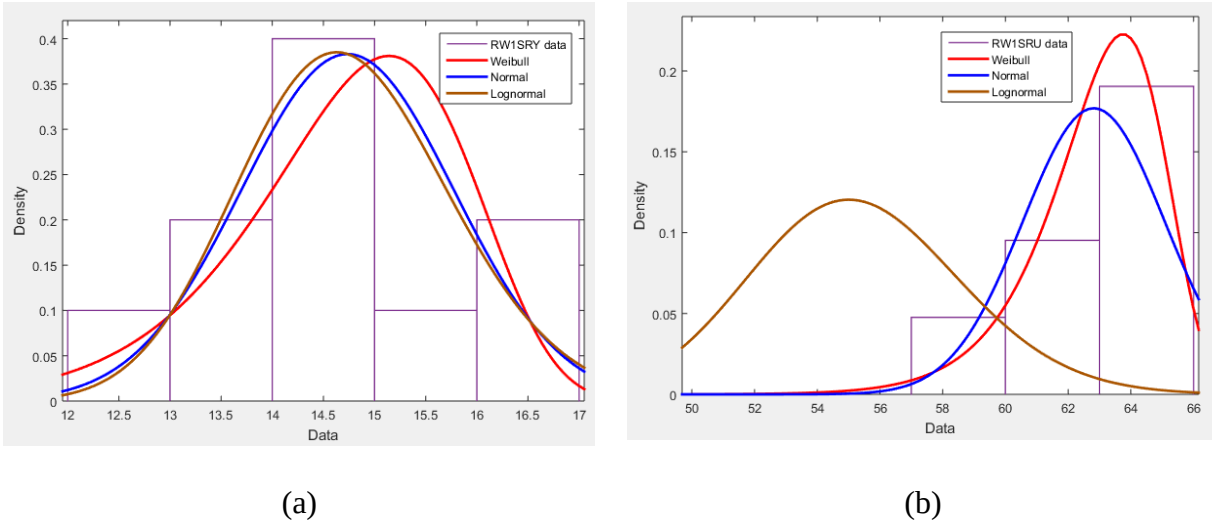


Figure 6.12: Wall RW1-SR Probability Distribution Function (a) Yielding Displacement; and (b) Ultimate Displacement

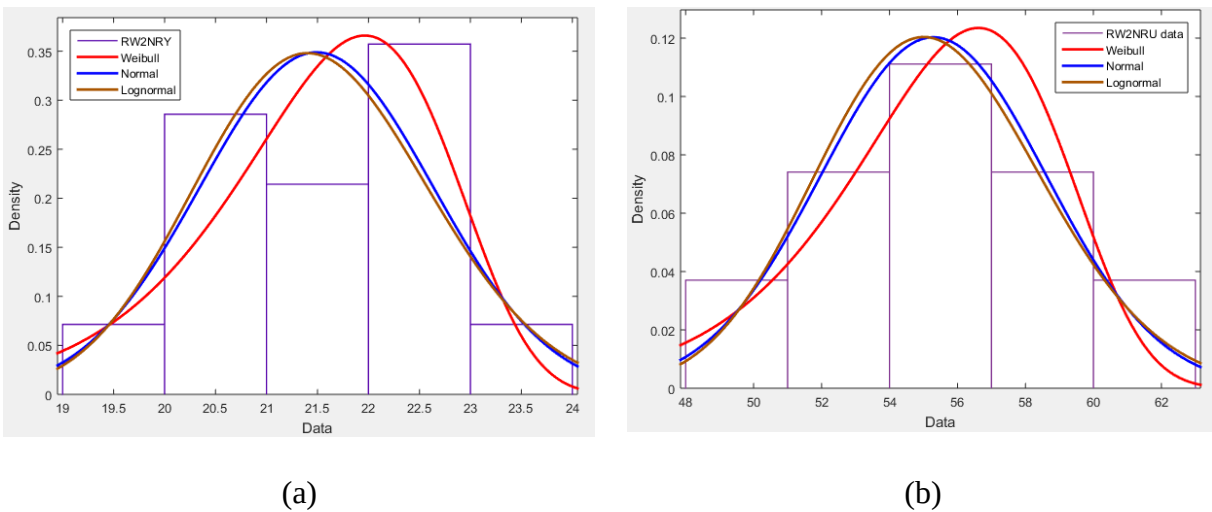


Figure 6.13: Wall RW2-NR Probability Distribution Functions (a) Yielding Displacement; and (b) Ultimate Displacement

Table 6.4: Wall RW1-SR Statistical Properties for Distributions

	Yielding Displacement		Ultimate Displacement	
Main descriptors	Mean	COV	Mean	COV
Experimental data	14.736	1.084	62.8165	5.0843
Weibull	14.704	1.322	62.879	4.2054
Normal	14.736	1.084	62.8165	5.0843
LogNormal	14.740	1.0867	55.305	11.0817

Table 6.5: Wall RW2-NR Statistical Properties for Distributions

	Yielding Displacement		Ultimate Displacement	
Main descriptors	Mean	COV	Mean	COV
Experimental data	21.49	1.3080	55.29	10.9971
Weibull	21.47	1.489	55.2016	12.8832
Normal	21.49	1.308	55.29	10.9971
LogNormal	21.494	1.324	55.305	11.0871

6.6. Summary

For a reliability analysis to provide a good estimation of the reliability indexes β and the probabilities of failures, in regard to a certain failure mechanism, random variables employed in the defined limit state function should be identified and probabilistically described. For shear walls under the effect of lateral loads, besides the material properties, for which some probabilistic

characterization is already available in the literature, more investigations are required for reflecting the structural behavior of the shear walls. Thus experimentally obtained displacements, strains and stresses should be converted into random variables and the recorded data should be assigned a PDF, after performing data fitting analysis. For each type of wall, extensive experimental data is required for completing the probabilistic interpretation of the random variables defined. In the current experiment only a limited set of tests were performed, because the main objective of this research projects was to investigate the stability of repaired SMA reinforced walls to lateral loading. A general framework for conducting reliability analysis was proposed however, and the probabilistic interpretation of the yield and ultimate displacements obtained from the tests was conducted for exemplifications. These random variables can be further employed for performing reliability analysis in regard to other failure or performance verifications such as wall ductility verification and drift level.

CHAPTER 7

CONCLUSIONS AND FUTUREWORK

7.1 Conclusions

Structures subjected to a major earthquake undergo large displacements to dissipate the energy of the earthquake, such that the structural materials yield and respond in the inelastic range. Consequently, extreme earthquakes could cause structural instability due to the large imposed displacements, which may cause the structure to lose its load carrying capacity and collapse.

Specifically, research was conducted by Abdulridha (2013) on hybrid-reinforced shear walls, with shape memory alloy as the principal reinforcement in the boundary regions and traditional deformed reinforcement in the web section of the wall. A control shear wall with only regular deformed steel reinforcement was also tested to provide a comparison to the SMA shear wall.

It was concluded that SMA is an excellent alloy for controlling large displacements when used in the critical regions in a shear wall. The self-centering phenomenon was also evident even with the use of regular deformed steel reinforcement in the web region. However, in the control wall, after yielding, increases in residual displacements were evident.

In the current study, the shear walls tested by Abdulridha (2013) were repaired and re-tested to ensure that the structural behavior and capacities that were present in the original walls could be satisfactorily restored after minimum repairs.

7.2 General Conclusions

The main objective of this research was to repair the plastic hinge region of previously tested conventionally and hybrid-SMA reinforced slender walls and to study their seismic performance. The focus was to illustrate that a structure with SMA-reinforced shear wall, which experiences a seismic event, can be fully serviceable after minimum repairs. In addition, reliability analysis was

proposed using a first order reliability method (FORM) to determine the values of the appropriate reliability indexes for hybrid-SMA reinforced shear walls before and after repairing.

In order to achieve the main target of this research project, the following project goals were fulfilled:

- Investigation of different types of mechanical connections for reinforcement.
- Investigation of the effect of shortened SMA lengths on the deformation recovery capacity of the shear wall.
- Investigation of the effects of using high strength self-consolidating concrete to repair the plastic hinge region.
- Comparison of the performance of shear walls before and after repairing.
- Sectional analysis for the reliability of shear walls.

7.2.1 Behaviour of Repaired Shear Walls

Several main conclusions can be drawn based on the experimental work performed during the current research project, as follows:

- Spalling of the concrete was evident in the repaired wall (RW2-NR) at a low drift level adjacent to the location of the mechanical couplers connecting the SMA bars to conventional deformed reinforcement. This was attributed to reduced concrete cover at the location of the couplers and bursting stresses that developed at the head of the coupler.
- In Wall RW2-NR, cracking was mostly concentrated in the web region, while cracking in Wall RW1-SR spread throughout the repaired region.
- For both repaired walls, no new cracks surfaced in the region above the plastic hinge zone.

- Beyond yielding, the ability of the SMA bars to recover the wall displacements was evident, up to the rupture of the SMA bars. In the Wall RW1-SR an increase in residual displacements was noticed beyond yielding.
- It can be established that the re-centering capacity of the wall was not significantly affected by shortening the SMA reinforcing bars in the repaired walls, and deficiencies were evident after 2% drift, which were attributed to the couplers imposing a weakness in the SMA bars. It should be noted, however, that a majority of buildings are designed to respond to earthquake within the 2%-2.5% drift range.
- Larger vertical displacements were recorded in Wall RW2-NR when compared to Wall RW1-SR. This was due to the smooth surface of SMA bars.
- Compared to Wall RW1-SR, greater pinching was evident in Wall RW2-NR, due to the self-centering phenomenon.
- The web reinforcement in Wall RW2-NR yielded prior to yielding of SMA bars. For Wall W1-SR, yielding of the boundary and web reinforcement occurred at approximately the same load.
- The web reinforcement in RW2-NR yielded at a lower lateral load than the web reinforcement in RW1-SR, due to the lower modulus of elasticity of the SMA bars.
- Yielding of the SMA bars in Wall RW2-NR did not spread beyond the six-inch coupler length.
- The yield stiffness of the repaired SMA wall was 66% of the yield stiffness of the repaired steel-reinforced wall. This was likely due to shortening of the SMA bars in the boundary regions of the wall RW2-NR.
- The post-yield secant stiffness of RW2-NR was 30% smaller than RW1-SR.

- A subtle difference between RW1-SR and RW2-NR is the ratcheting in the lateral load-rotation response of RW1-SR in the positive direction of loading; whereas, RW2-NR provided a symmetrical response in both directions.

7.2.2 Comparison of Repaired Shear Walls with Original Walls

A comparison between the repaired walls tested in the current research and the original walls previously tested led to the following conclusions:

- Tensile testing of the bare SMA bars demonstrated that they maintained sufficient strength and recovery capacities after testing the original walls. As a result, the same SMA bars were used in the repaired walls as well.
- The displacement recovery capacity in RW2-NR was reduced in comparison with the W2-NR.
- The repaired walls experienced slightly less pinching and wider hysteretic curves compared to original walls.
- Wall RW1-SR experienced an increase in strength of up to 2.6% drift. Wall W1-SR experienced slight increase in strength up to 3.6% drift.
- The pre-yielding behaviour of Walls RW2-NR and W2-NR was very similar. Beyond yielding, RW2-NR rapidly reached its peak load capacity at 1.5% drift. The peak load capacity of W2-NR was experienced at 3% drift.
- The repaired SMA wall experienced higher stiffness at yield than the original SMA wall. This increase was attributed to the significantly higher strength of the repair concrete used in the plastic hinge region.
- The repaired SMA wall exhibited more than 80% recovery of displacements for up to the 2% drift level. Thereafter, the wall lost its recovery capacity due to fracturing of the SMA

bars. The original SMA wall (W2-NR) maintained its recovery capacity of 90% to 100% throughout testing.

- The repaired SMA wall failed due to rupturing of the SMA bars adjacent to the mechanical couplers, whereas, the original wall failed due to rupturing of the deformed vertical reinforcement in the web.
- The conventional walls (before and after repair) dissipated approximately the same energy for up to 1.5% drift. Beyond 1.5% drift, RW1-SR dissipated more energy than Wall W1-SR.
- RW2-NR dissipated slightly more energy than W2-NR for up to 3% drift. This is attributed to the increased stiffness in the plastic hinge region, as a result of the higher strength concrete used for the repair.
- Beyond 3% drift, Wall W2-NR continued to experience an increase in energy dissipation, while Wall RW2-NR experienced a sudden reduction, due to loss of strength at 3% drift.
- The shear strain at yielding of the repaired walls (RW1-SR and RW2-NR) was significantly lower than the shear strain at yielding of the original walls (W1-SR and W2-NR).
- At failure of Wall RW2-NR, the SMA bars fractured just below the six-inch mechanical coupler. The reinforcing bars in the web buckled and fractured at the same level, adjacent to the mechanical coupler. At failure of Wall W2-NR, one SMA bar fractured adjacent to the mechanical coupler, other SMA bars buckled locally at the failure region of the wall. In the web, three vertical bars fractured, while the other experienced buckling. The failure location of the Wall W2-NR was adjacent to the termination of the starter bars.
- At failure of Wall RW1-SR, the main damage was concentrated just above the bottom couplers where the reinforcement in both boundary and web fractured. At failure of Wall

W1-SR, the longitudinal reinforcement in the boundary zones experienced both fracture and buckling. The left boundary zone experienced significantly more buckling than the right zone, while in the right boundary zone all the longitudinal reinforcement fractured. However, in the web only buckling of longitudinal reinforcement was noticed.

7.2.3 Reliability Analysis

To determine the probability of failures and the reliability indexes for the tested shear walls, a limit state function describing the resisting moment M_r was defined. Several random variables were incorporated, based on the data reported in the literature. However, due to lack of experimental data complete reliability analysis was not conducted, and only the framework was provided. According to the literature, the reliability index of 3.5 provides an economical design for ductile members in normal consequences of failure. At this reliability index, design moments of Walls RW2-NR and W2-NR were similar due to the design being governed by the low modulus of elasticity of the SMA bars. As for the conventional walls, the high strength concrete in Wall RW1-SR resulted in a greater design moment than Wall W1-SR. A framework for conducting reliability analysis in regard to other failure and performance mechanisms, such as ductility verification and drift, was also proposed by incorporating a probability distribution function (PDF) for experimental yield and ultimate displacements described as random variables. To complete such analysis, extensive experimental data is required which was beyond the objective of the current research project.

7.3 Recommendations for Future Research

Based on this current study and lessons learned, the following are areas that require additional research:

1. Extensive testing is required to develop safety factors for SMA reinforcement and develop appropriate design procedures.
2. More testing of SMA walls are required with focus on connectors between the SMA bars and deformed steel reinforcement, which will not affect the response of the walls.
3. Numerical analysis needs to be conducted in order to find the optimum length for SMA bars for the structure to be economical.
4. Behaviour of hybrid-SMA shear walls repaired using other materials/techniques such as externally bonded FRP sheets needs investigation.
5. Study is required for repairing hybrid-shear walls of different configurations.
6. Investigation is required to study the effect of vertical load on hybrid-SMA shear walls during reverse cyclic testing.

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APPENDICES

Appendix A: Condition of Repaired Shear Walls at End of Each Cycle

A.1 Wall RW1-SR (Steel Wall)



Figure A.1.1: State of Wall RW1-SR Before Testing



Figure A.1.2: Crack Pattern of Wall RW1-SR at 0.125% Drift (3 mm Displacement)



Figure A.1.3: Crack Pattern of Wall RW1-SR at 0.25% Drift (6 mm Displacement)

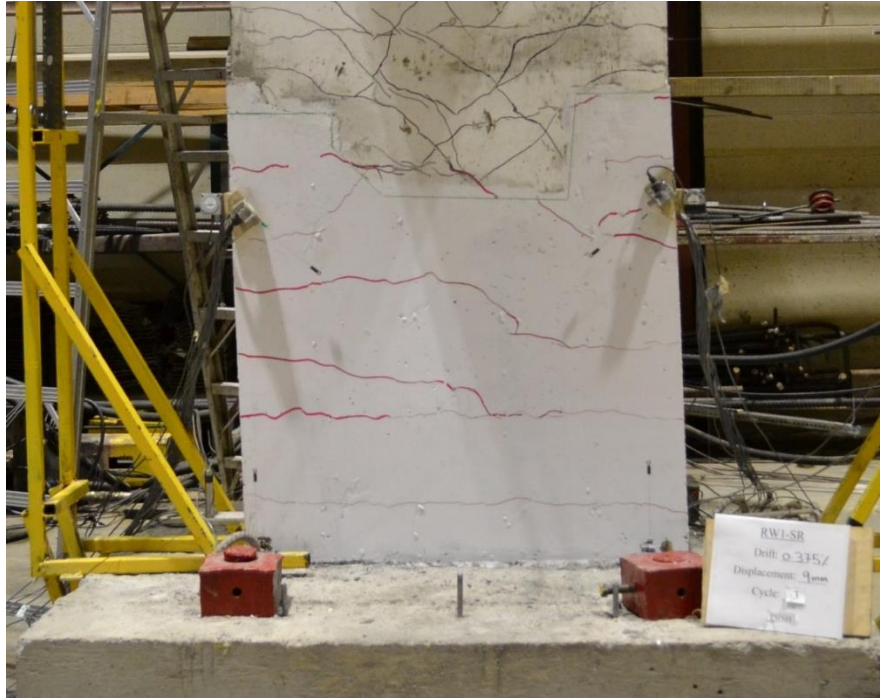


Figure A.1.4: Crack Pattern of Wall RW1-SR at 0.375% Drift (9 mm Displacement)



Figure A.1.5: Crack Pattern of Wall RW1-SR at 0.75% Drift (18 mm Displacement)



Figure A.1.6: Crack Pattern of Wall RW1-SR at 1.125% Drift (27 mm Displacement)



Figure A.1.7: Crack Pattern of Wall RW1-SR at 1.5% Drift (36 mm Displacement)



Figure A.1.8: Crack Pattern of Wall RW1-SR at 1.875% Drift (45 mm Displacement)

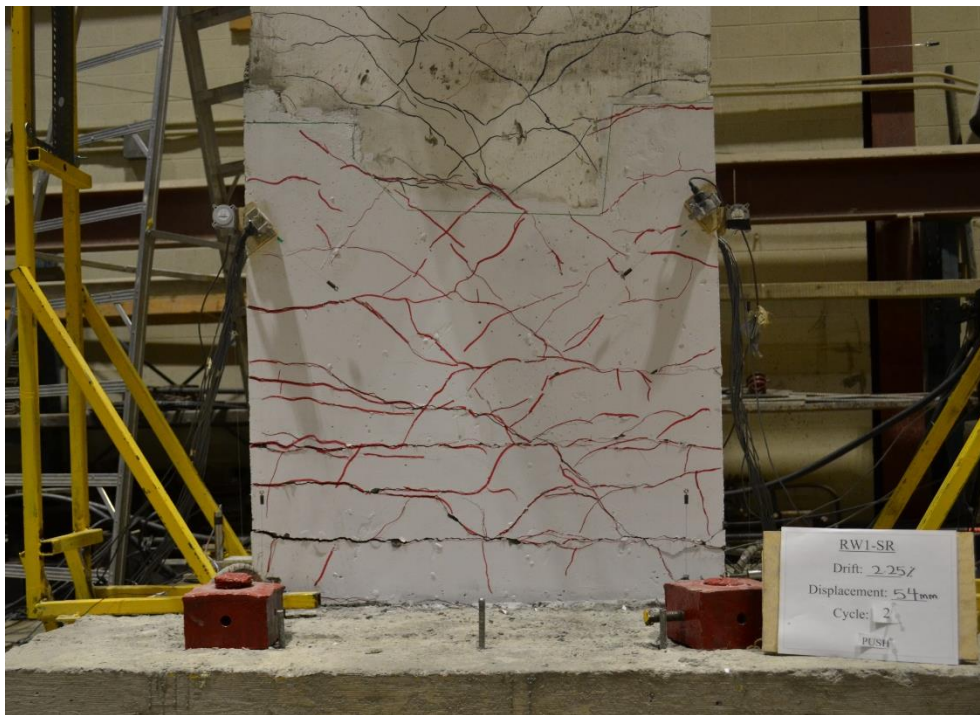


Figure A.1.9: Crack Pattern of Wall RW1-SR at 2.25% Drift (54 mm Displacement)



Figure A.1.10: Crack Pattern of Wall RW1-SR at 2.625% Drift (63 mm Displacement)

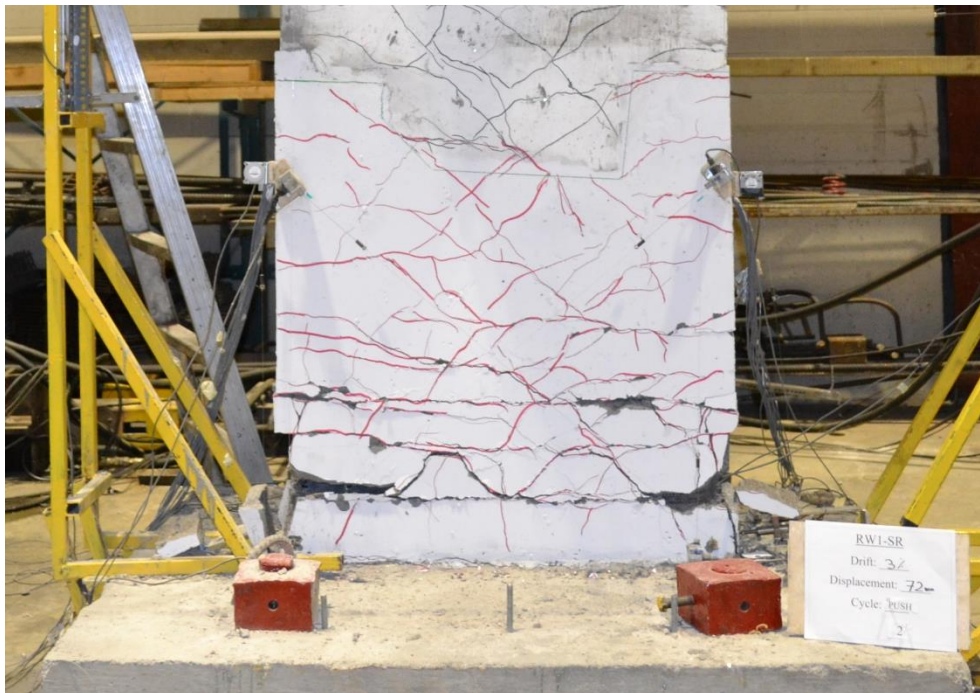


Figure A.1.11: Crack Pattern of Wall RW1-SR at 3% Drift (72 mm Displacement)



A.1.12: Crack Pattern of Wall RW1-SR at 3.375% Drift (81 mm Displacement)



A.1.13: Crack Pattern of Wall RW1-SR at Failure

A.2 Wall RW2-NR (SMA Wall)



Figure A.2.1: State of Wall RW2-NR Before Testing

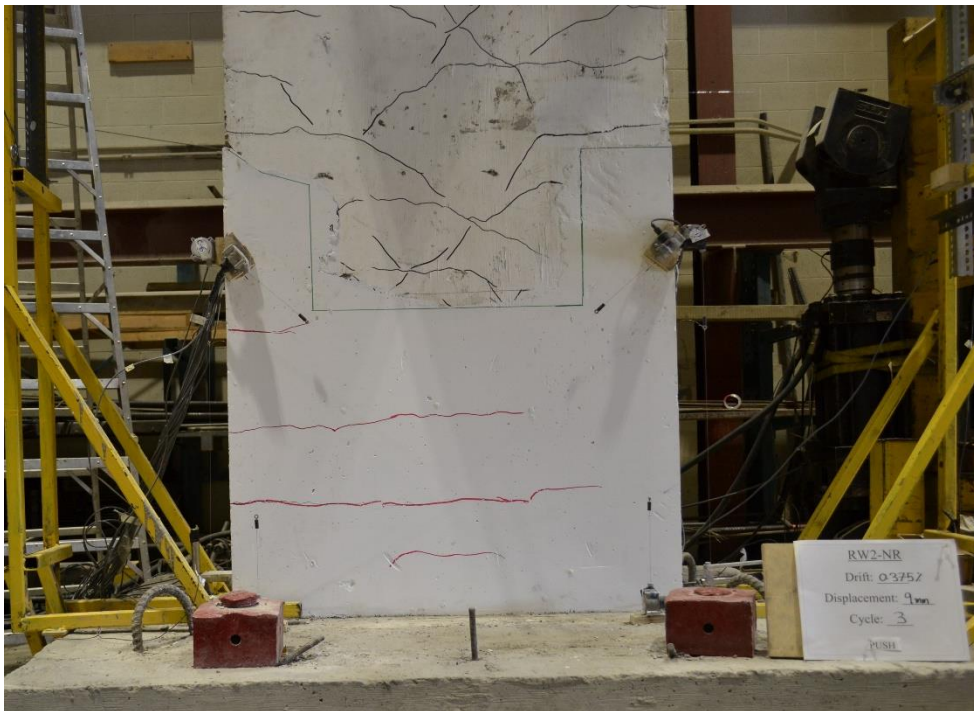


Figure A.2.2: Crack Pattern of Wall RW2-NR at 0.375% Drift (9 mm Displacement)



Figure A.2.3: Crack Pattern of Wall RW2-NR at 0.75% Drift (18 mm Displacement)



Figure A.2.4: Crack Pattern of Wall RW2-NR at 1.125% Drift (27 mm Displacement)



Figure A.2.5: Crack Pattern of Wall RW2-NR at 1.5% Drift (36 mm Displacement)



Figure A.2.6: Crack Pattern of Wall RW2-SR at 1.875% Drift (45 mm Displacement)



Figure A.2.7: Crack Pattern of Wall RW2-NR at 2.25% Drift (54 mm Displacement)

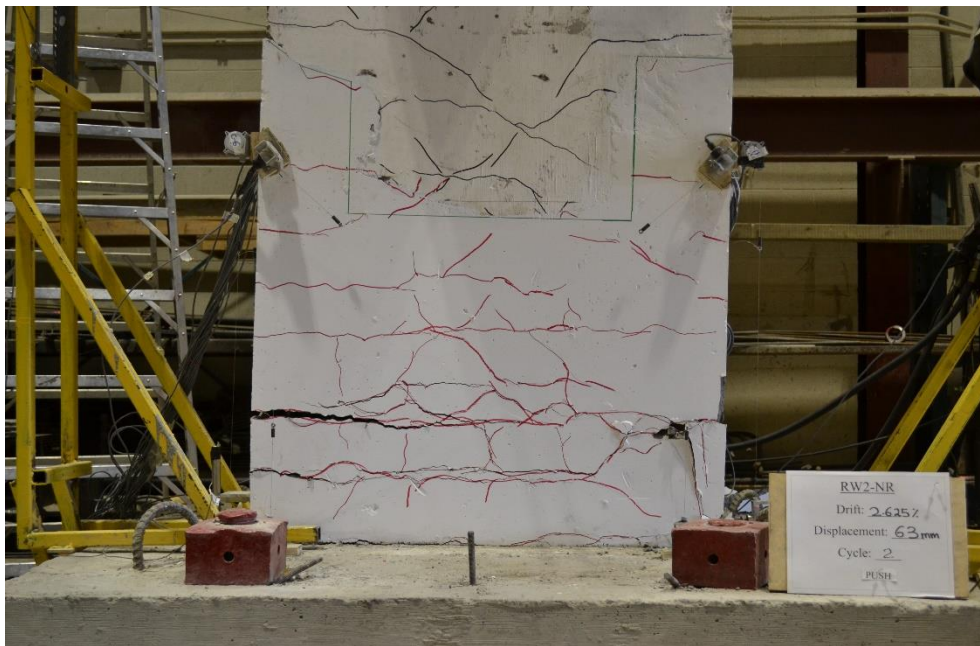


Figure A.2.8: Crack Pattern of Wall RW2-NR at 2.625% Drift (63 mm Displacement)



Figure A.2.9: Crack Pattern of Wall RW2-NR at 3% Drift (72 mm Displacement)



Figure A.2.10: Crack Pattern of Wall RW2-NR at 3.375% Drift (81 mm Displacement)



Figure A.2.11: Crack Pattern of Wall RW2-NR at 3.75% Drift (90 mm Displacement)



Figure A.2.142: Crack Pattern of Wall RW2-NR at Failure

Appendix B: Displacement Responses in Walls

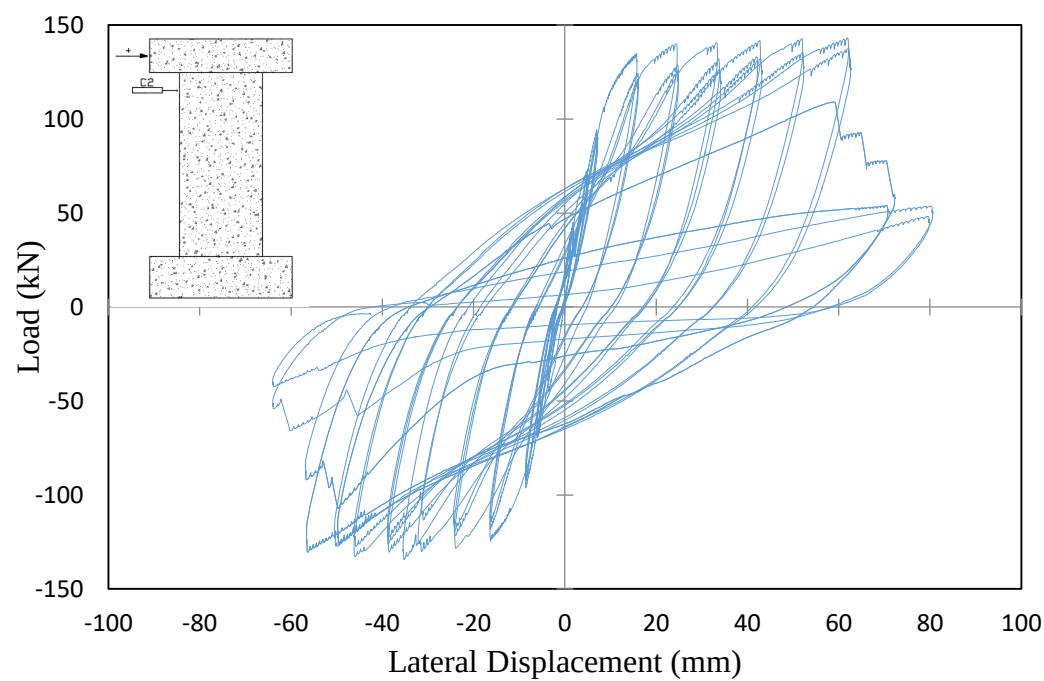


Figure B.1: DCT C2 Response in Wall RW1-SR

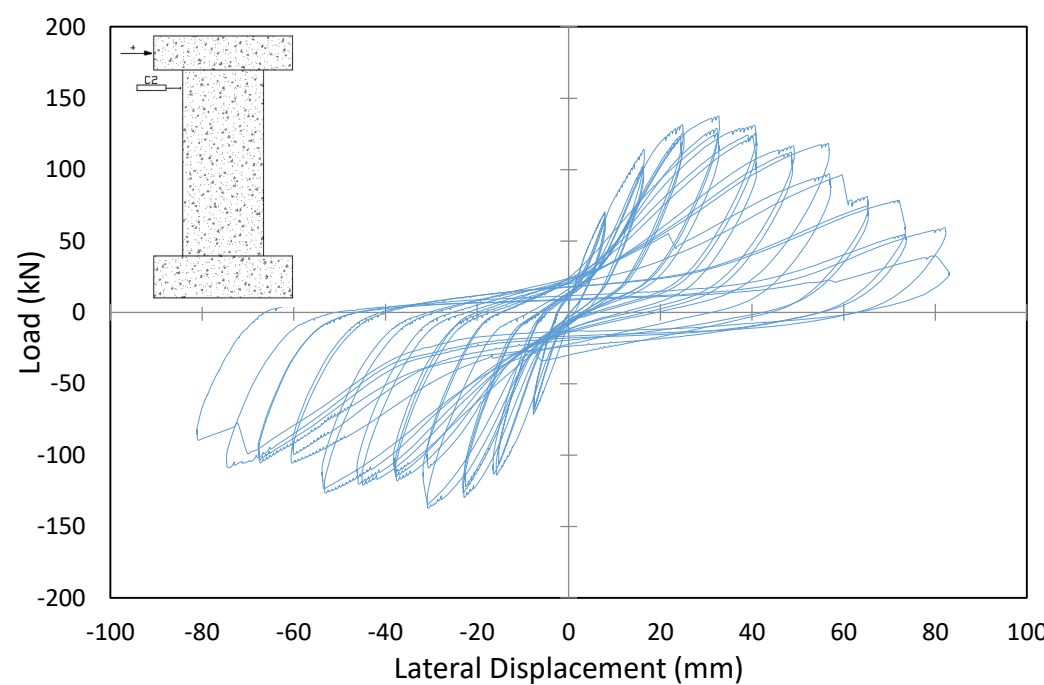


Figure B.2: DCT C2 Response in Wall RW2-NR

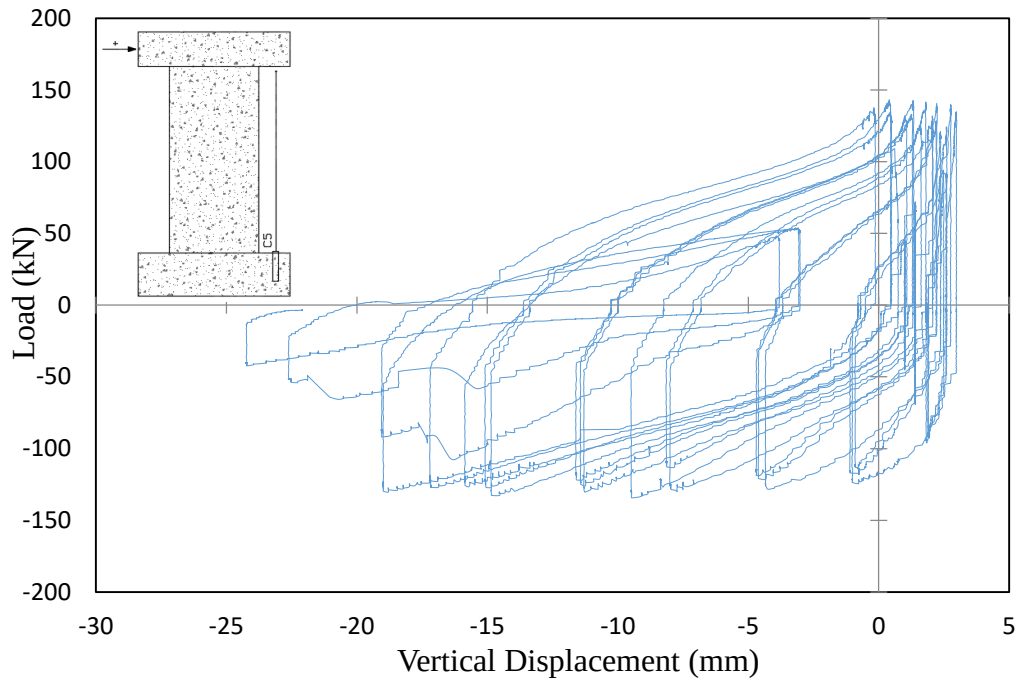


Figure B.3: DCT C5 Response in Wall RW1-SR

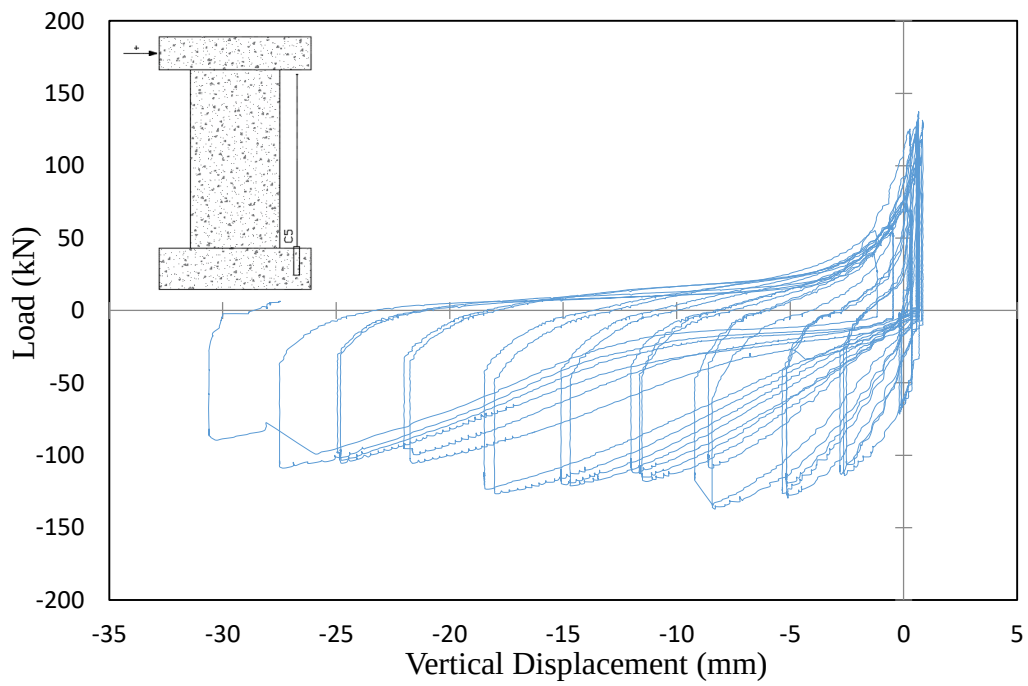


Figure B.4: DCT C5 Response in Wall RW2-NR

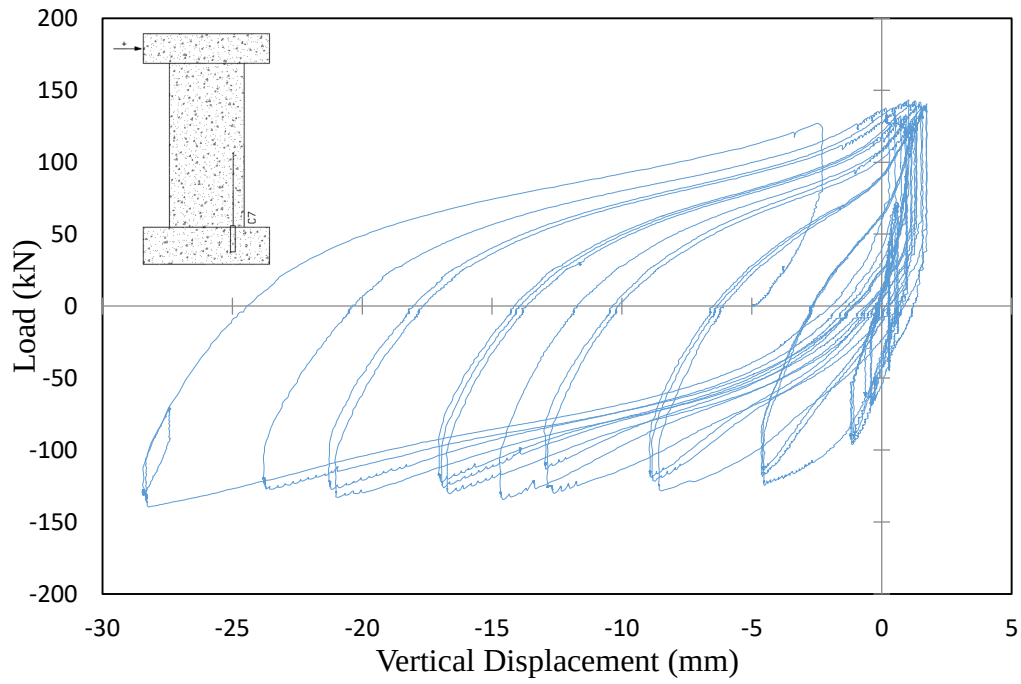


Figure B.5: DCT C7 Response in Wall RW1-SR

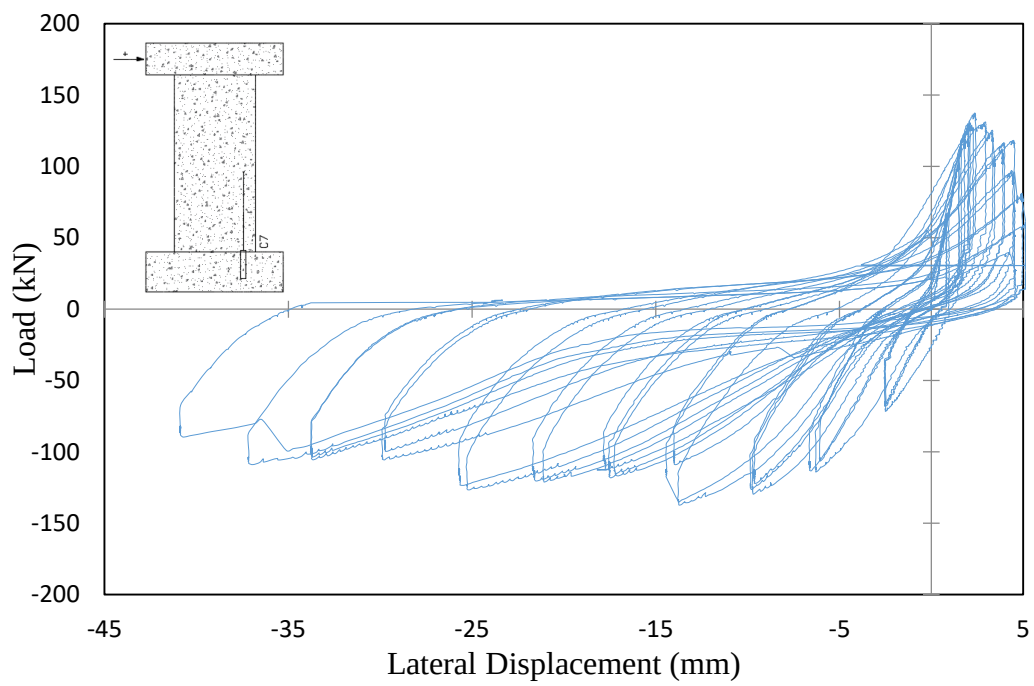


Figure B.6: DCT C7 Response in Wall RW2-NR

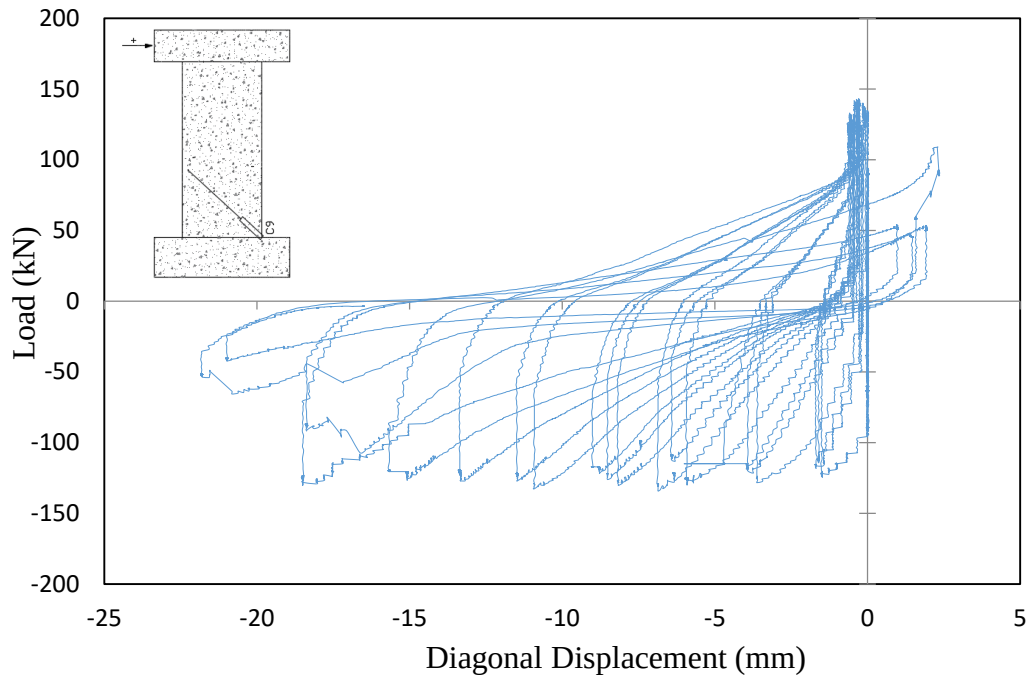


Figure B.7: DCT C9 Response in Wall RW1-SR

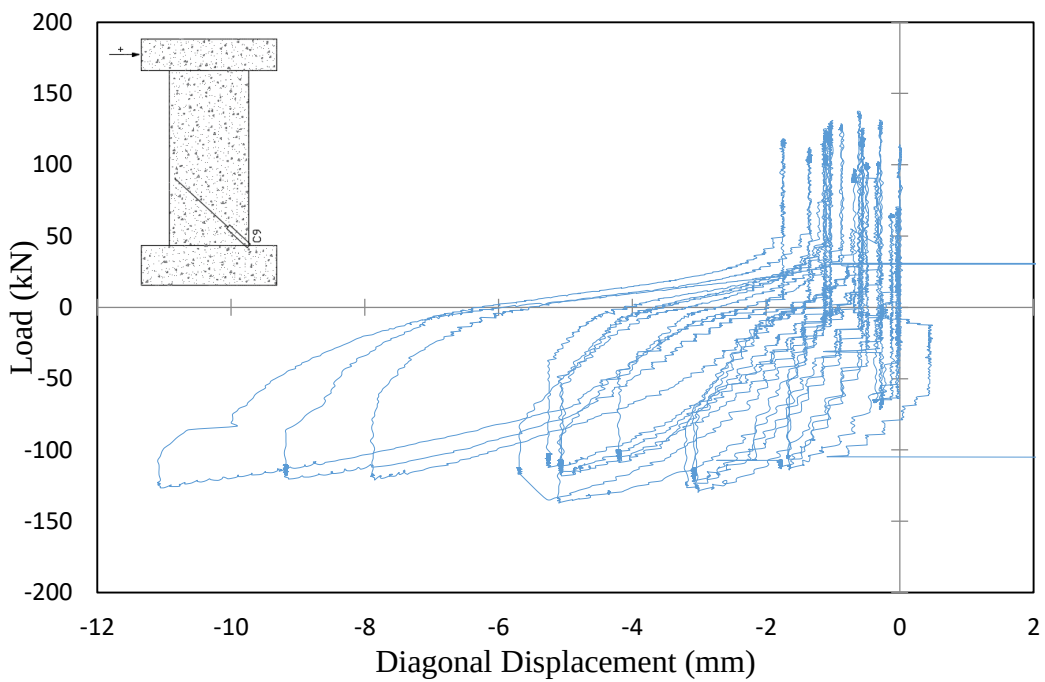


Figure B.8: DCT C9 Response in Wall RW2-NR

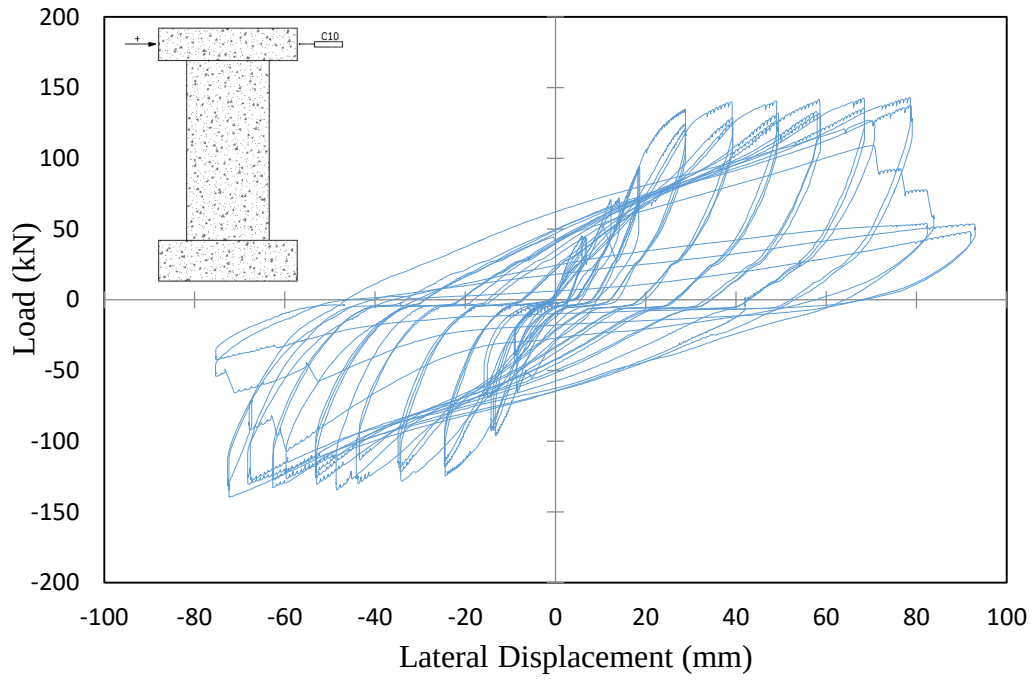


Figure B.9: DCT C10 Response in Wall RW1-SR

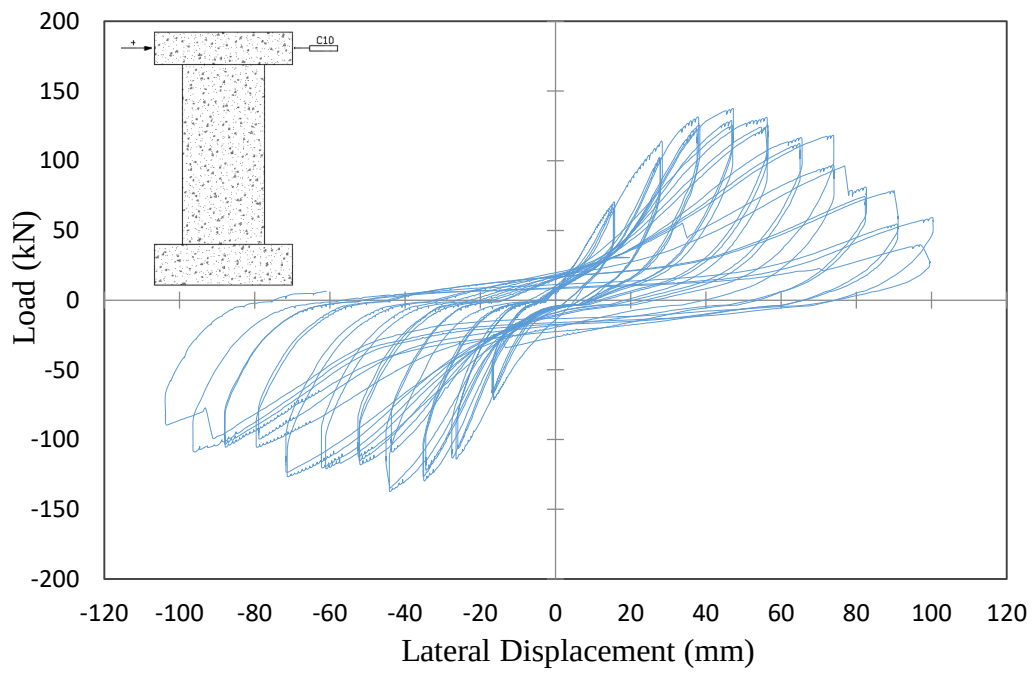


Figure B.10: DCT C10 Response in Wall RW2-NR

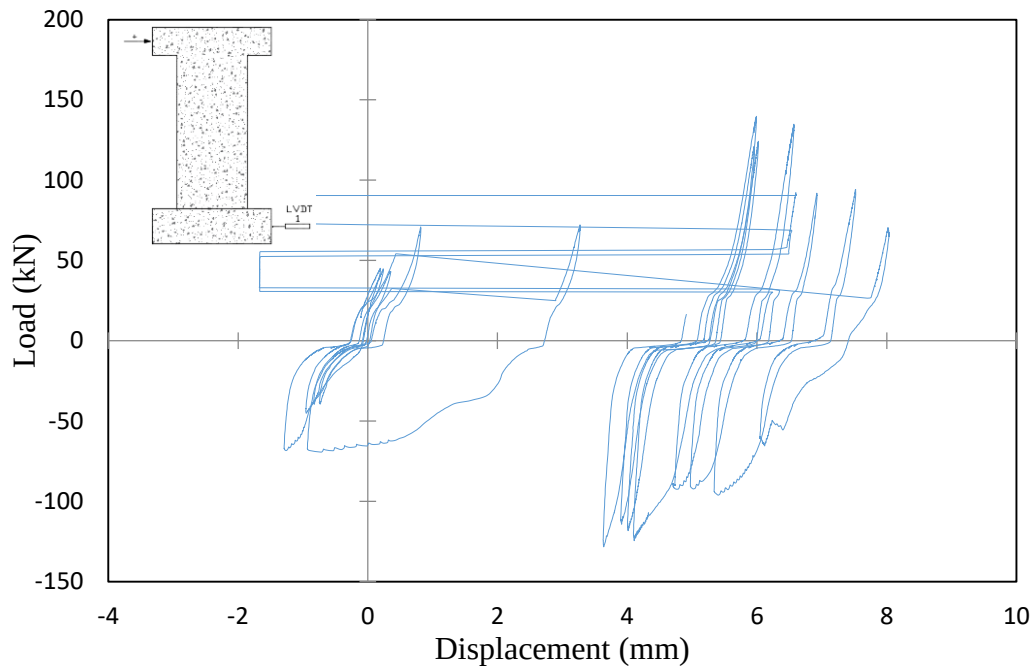


Figure B.11: LVDT 1 Response in Wall RW1-SR

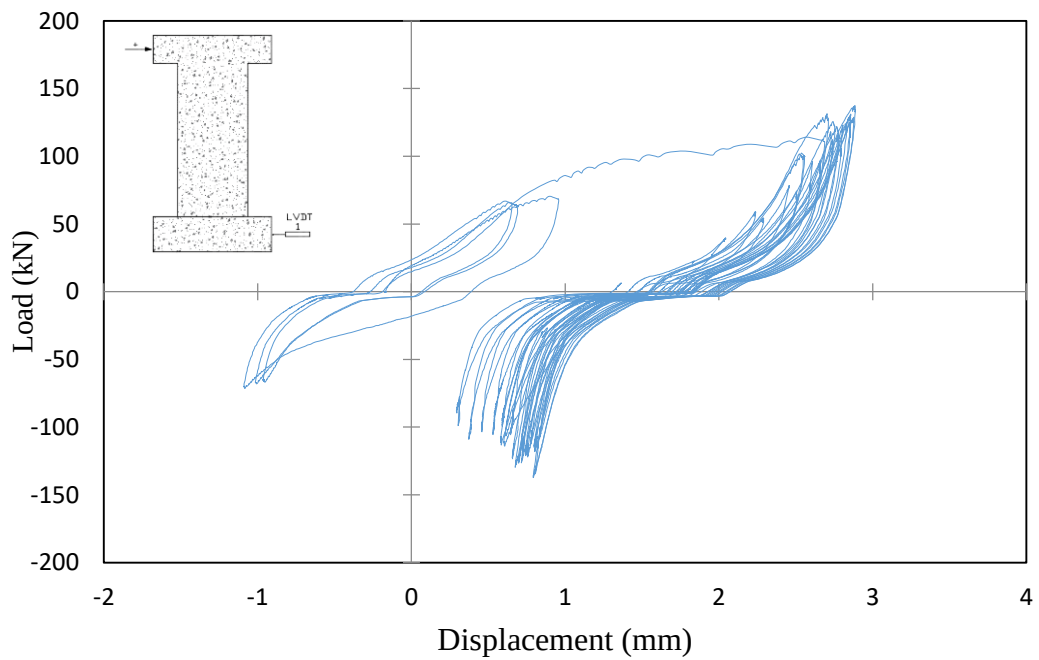


Figure B.12: LVDT 1 Response in Wall RW2-NR

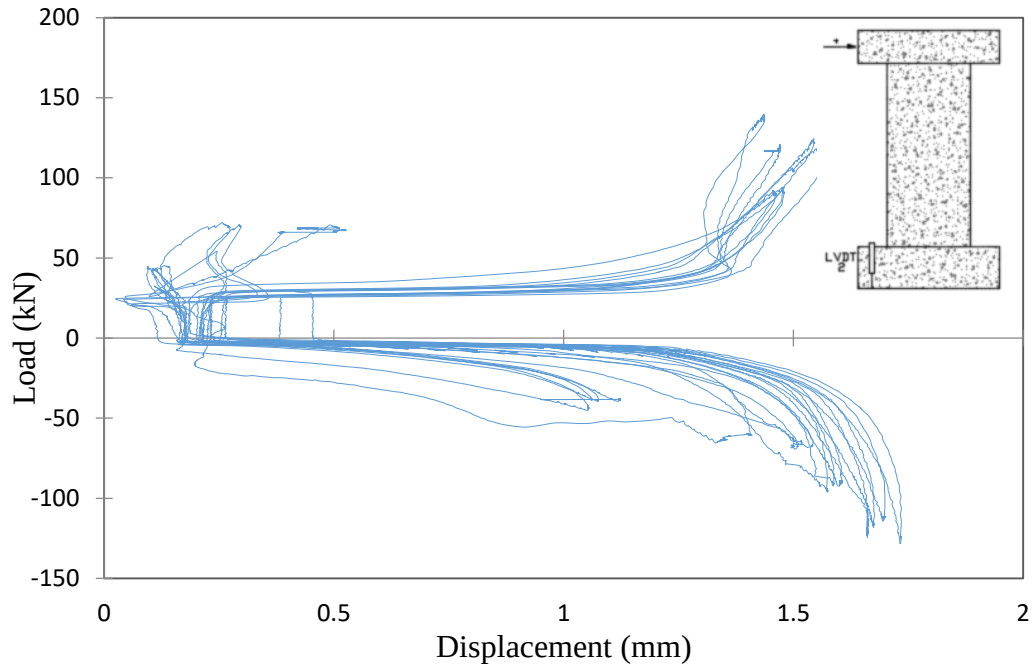


Figure B.13: LVDT 2 Response in Wall RW1-SR

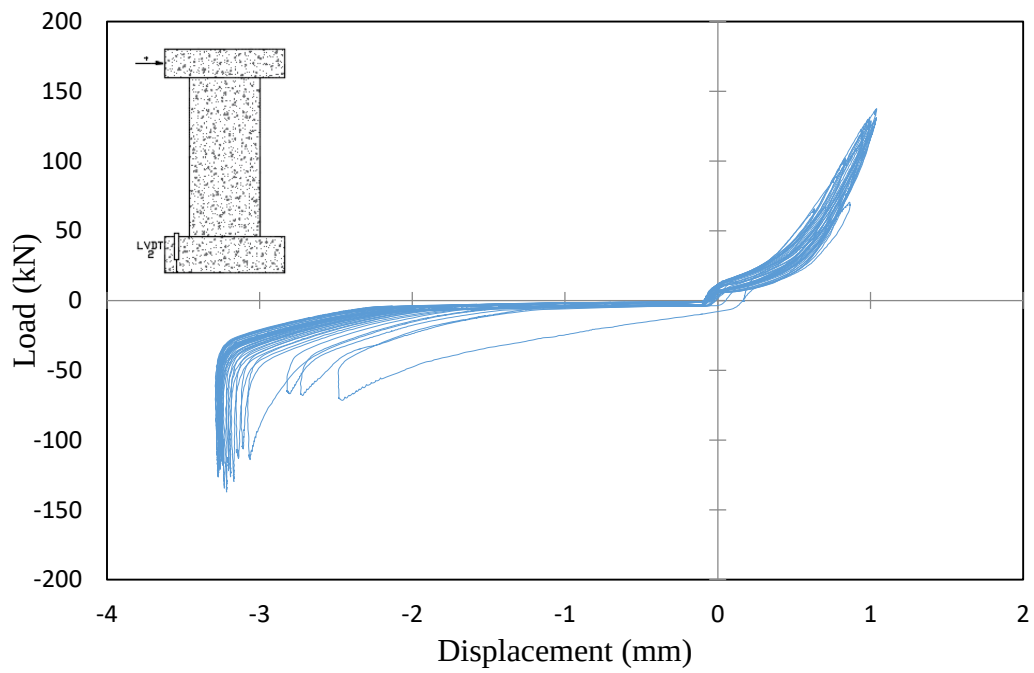


Figure B.14: LVDT 2 Response in Wall RW2-NR

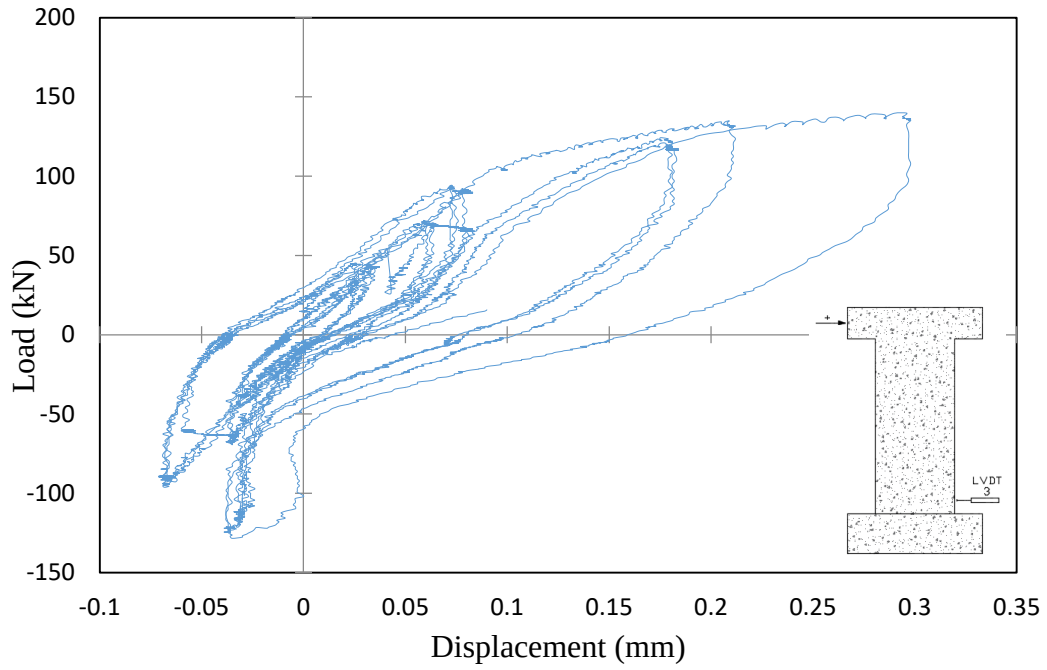


Figure B.15: LVDT 3 Response in Wall RW1-SR

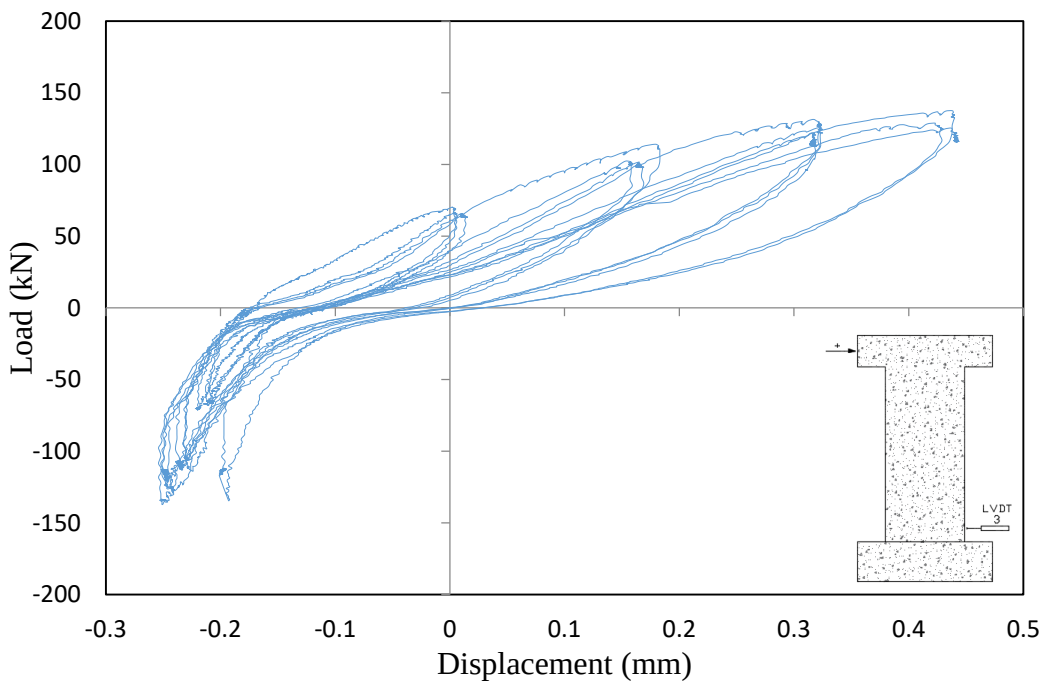


Figure B.16: LVDT 3 Response in Wall RW2-NR

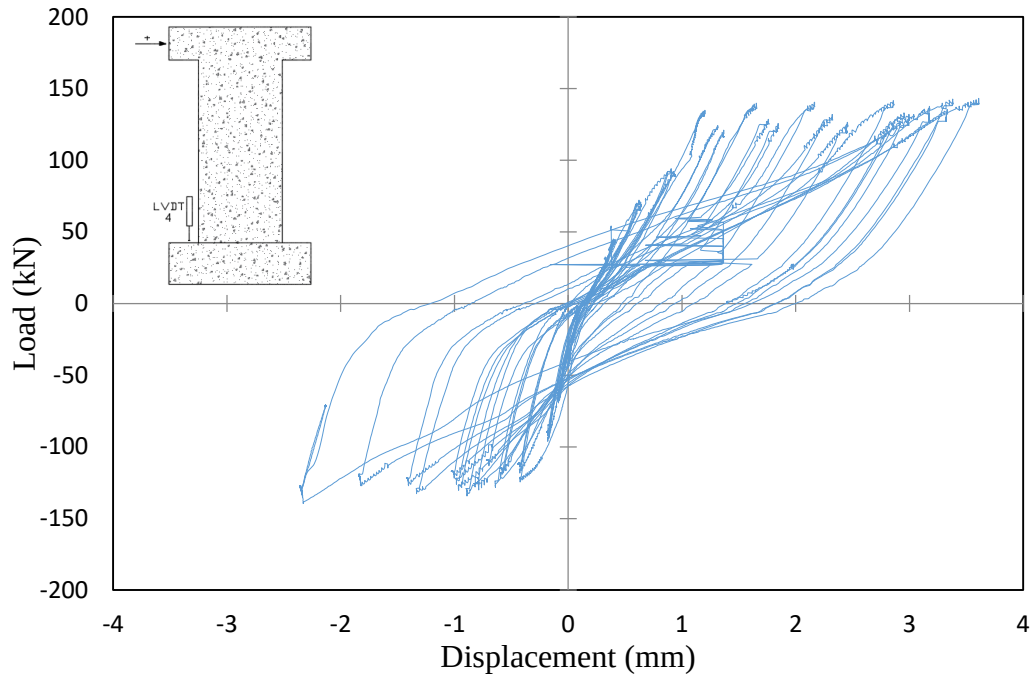


Figure B.17: LVDT 4 Response in Wall RW1-SR

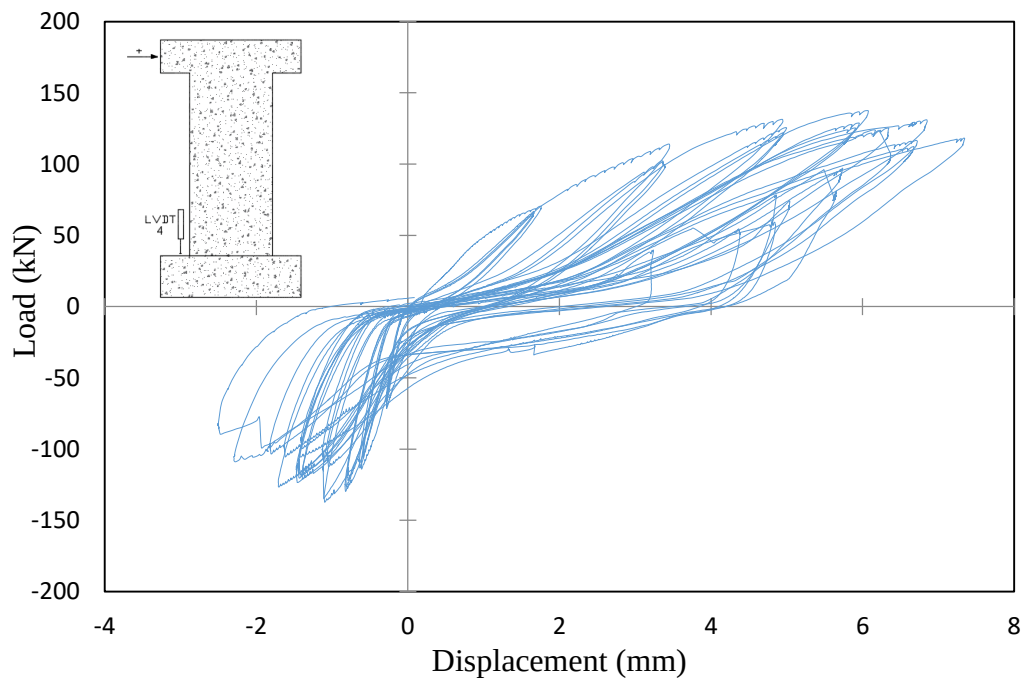


Figure B.18: LVDT 4 Response in Wall RW2-NR

Appendix C: Reinforcement Strain Responses

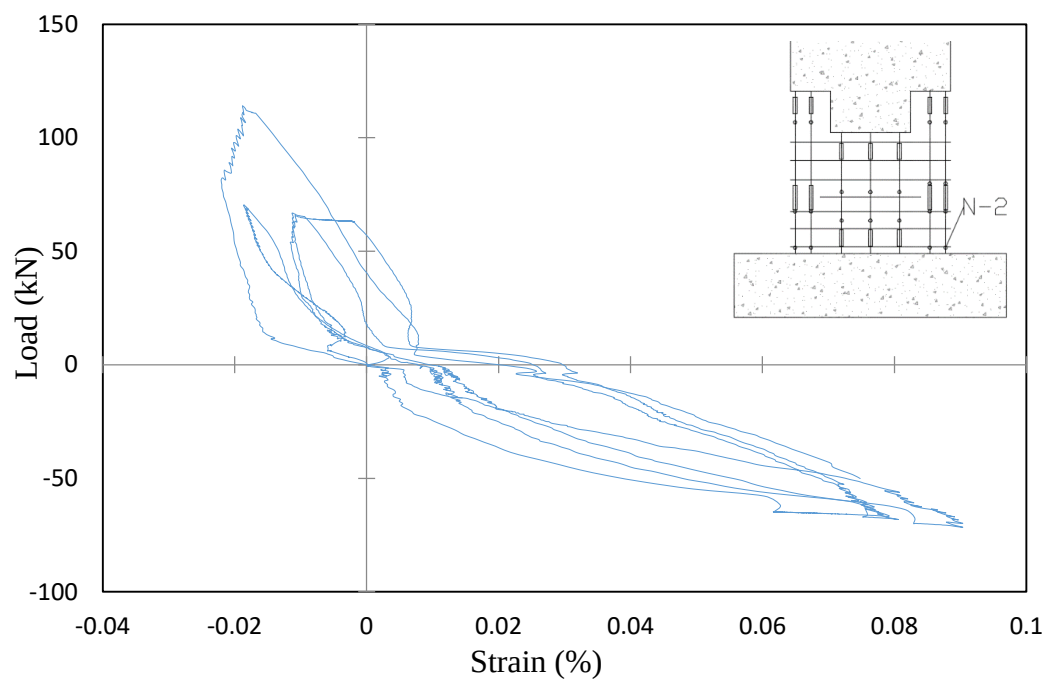


Figure C.1: Lateral Load-Strain Response of Gauge N-2 in Wall RW2-NR

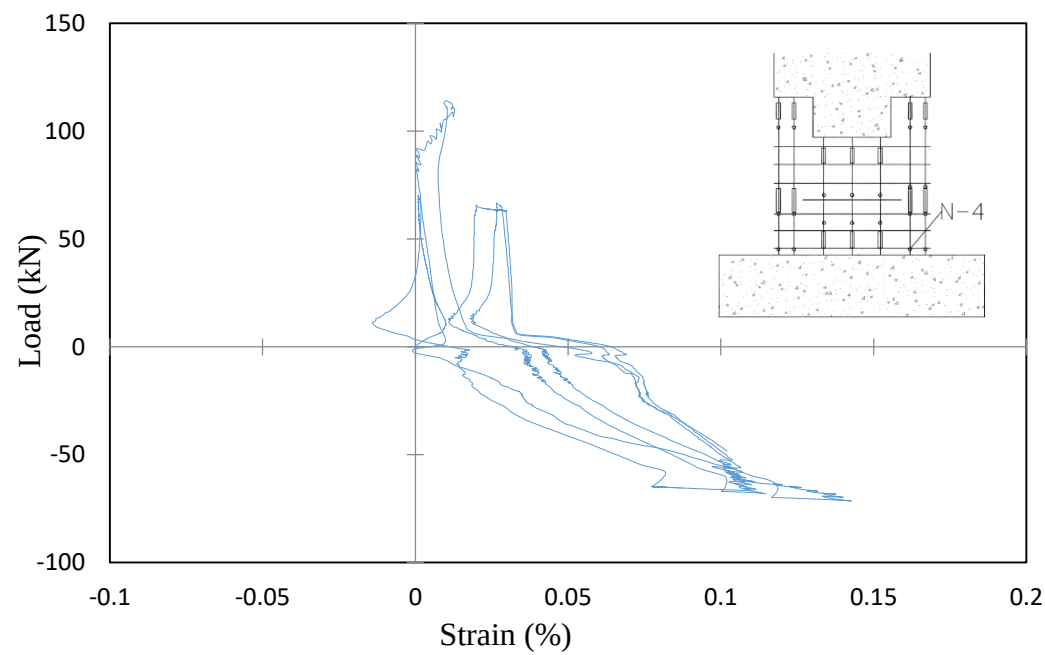


Figure C.2: Lateral Load-Strain Response of Gauge N-4 in Wall RW2-NR

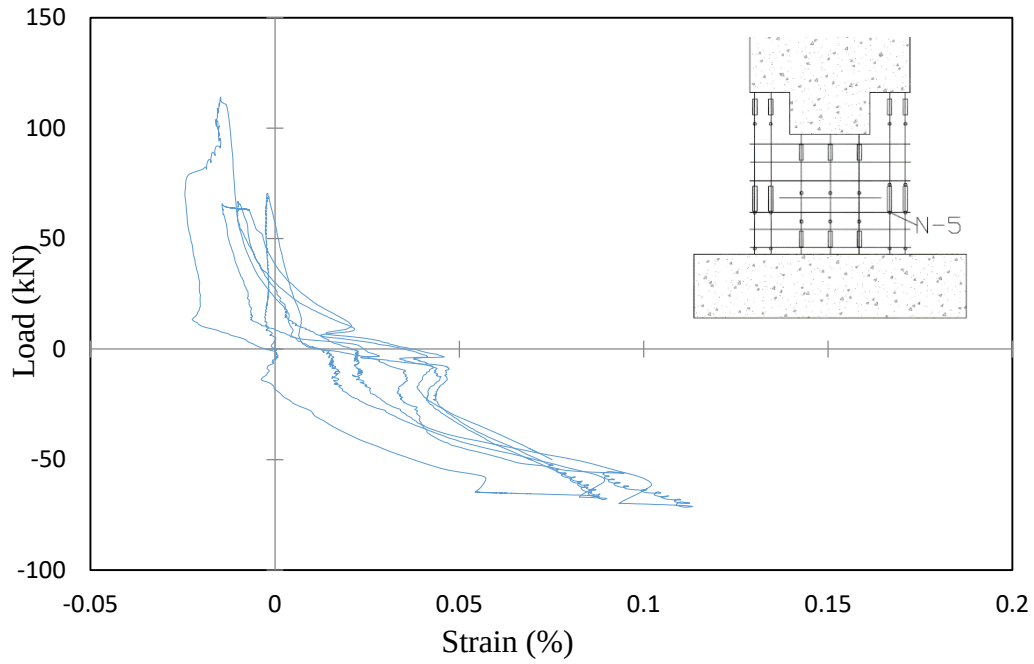


Figure C.3: Lateral Load-Strain Response of Gauge N-5 in Wall RW2-NR

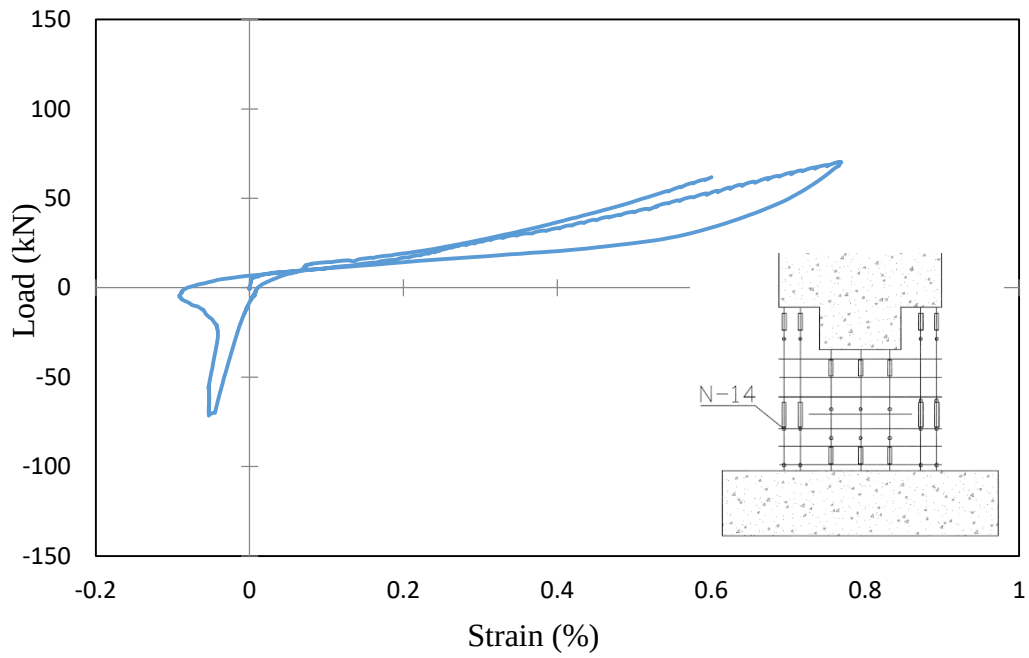


Figure C.4: Lateral Load-Strain Response of Gauge N-14 in Wall RW2-NR

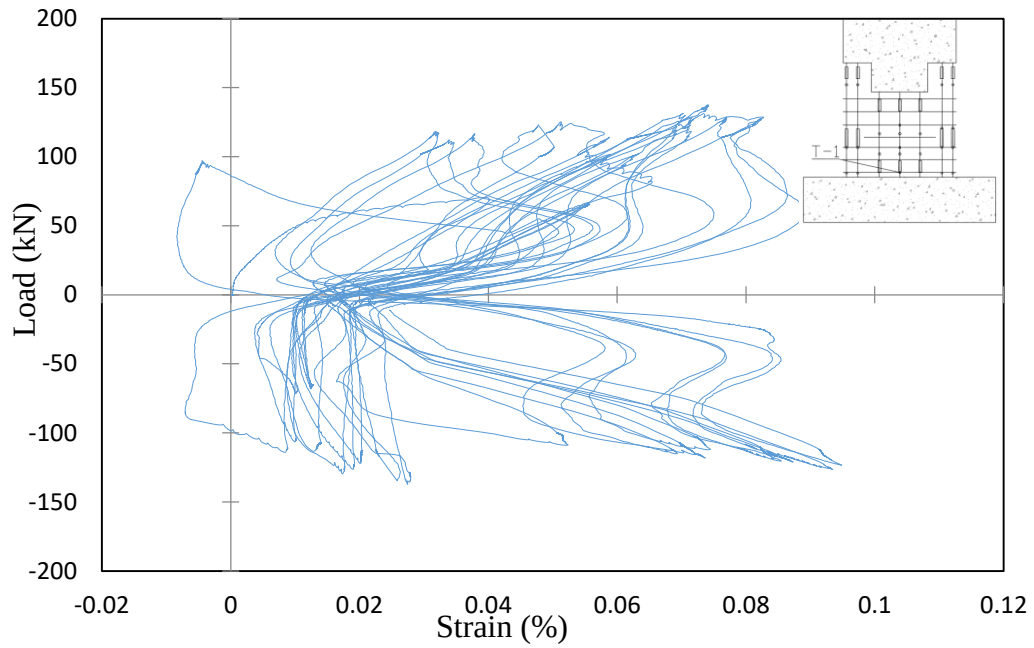


Figure C.5: Lateral Load-Strain Response of Gauge T-1 in Wall RW2-NR

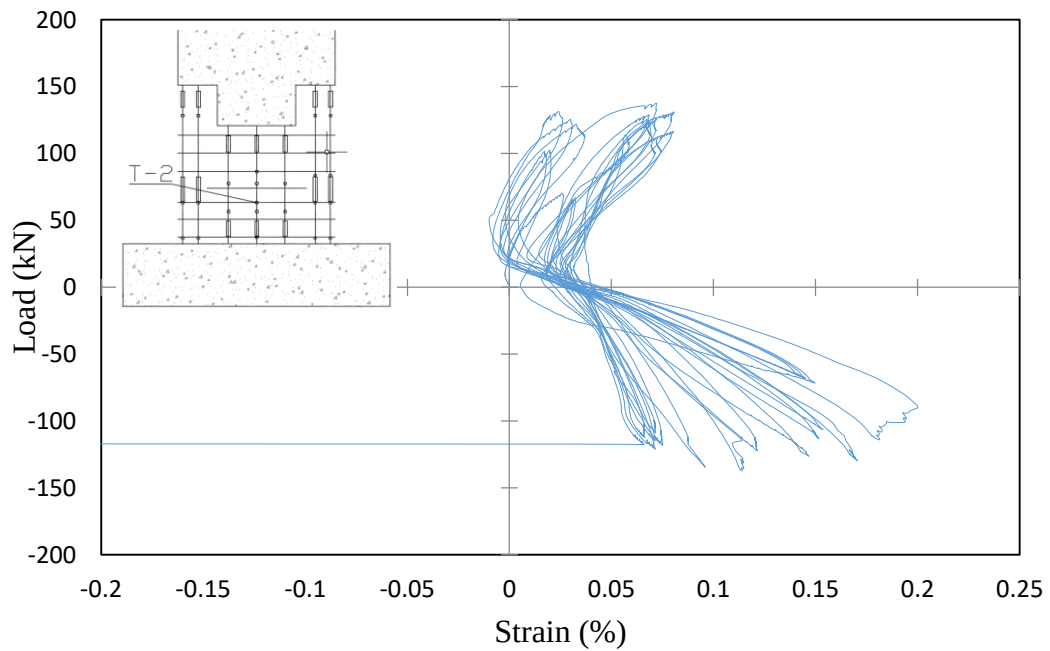


Figure C.6: Lateral Load-Strain Response of Gauge T-2 in Wall RW2-NR

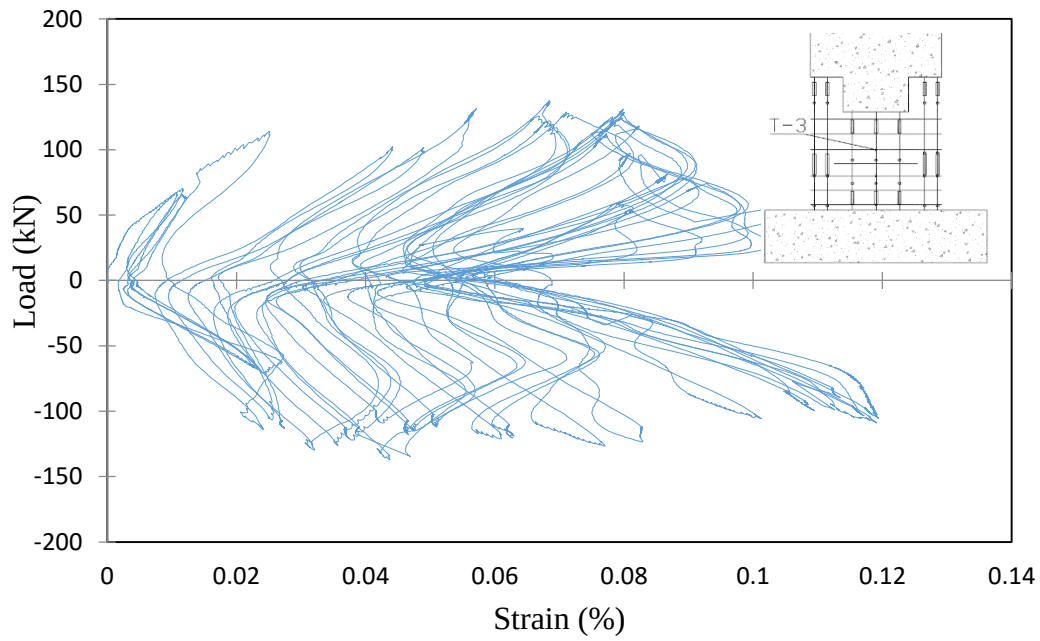


Figure C.7: Lateral Load-Strain Response of Gauge T-3 in Wall RW2-NR

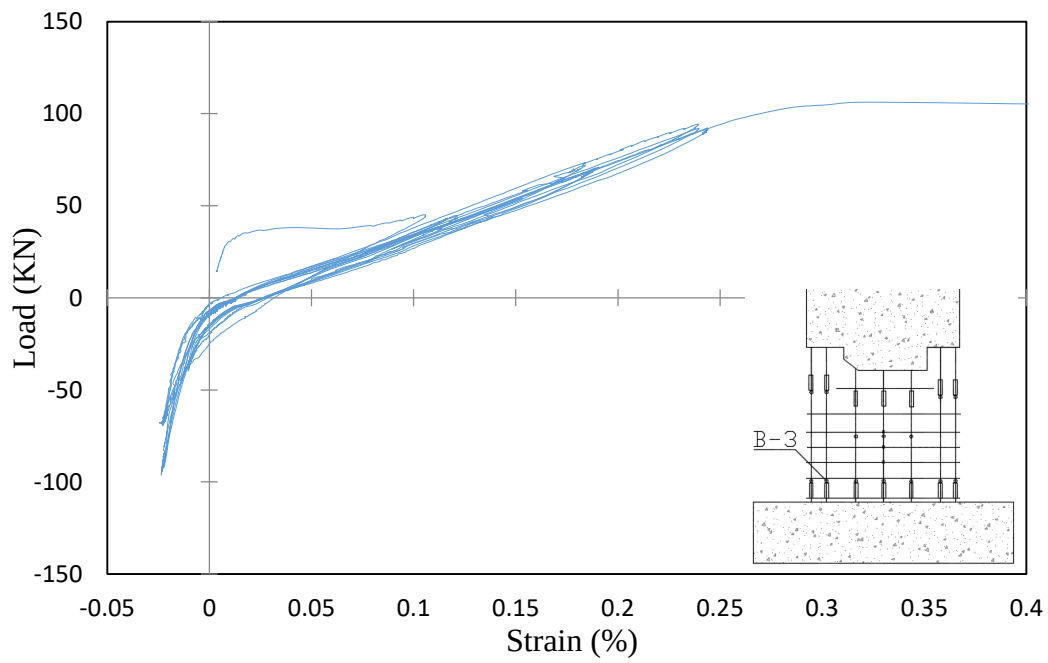


Figure C.8: Lateral Load-Strain Response of Gauge B-3 in Wall RW1-SR

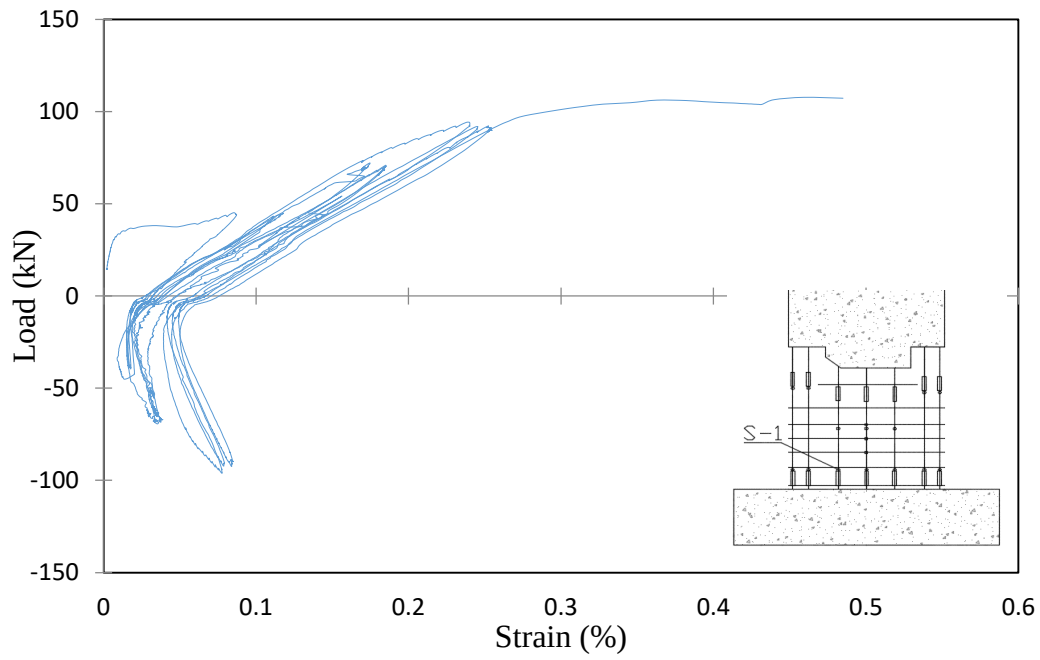


Figure C.9: Lateral Load-Strain Response of Gauge S-1 in Wall RW1-SR

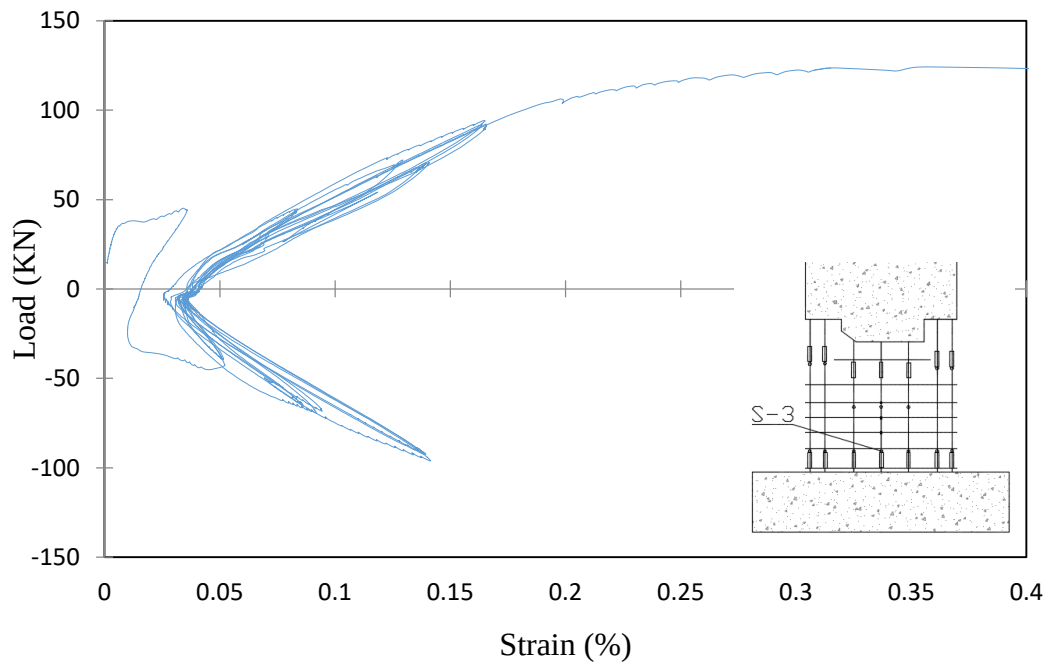


Figure C.10: Lateral Load-Strain Response of Gauge S-3 in Wall RW1-SR