

Performance of Superelastic Shape Memory Alloy  
Reinforced Concrete Elements Subjected to Monotonic and  
Cyclic Loading

by

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## ABSTRACT

The ability to adjust structural response to external loading and ensure structural safety and serviceability is a characteristic of Smart Systems. The key to achieving this is through the development and implementation of smart materials. An example of a smart material is a Shape Memory Alloy (SMA).

Reinforced concrete structures are designed to sustain severe damage and permanent displacement during strong earthquakes, while maintaining their integrity, and safeguarding against loss of life. The design philosophy of dissipating the energy of major earthquakes leads to significant strains in the steel reinforcement and, consequently, damage in the plastic hinge zones. Most of the steel strain is permanent, thus leading to large residual deformations that can render the structure unserviceable after the earthquake. Alternative reinforcing materials such as superelastic SMAs offer strain recovery upon unloading, which may result in improved post-earthquake recovery.

Shape Memory Alloys have the ability to dissipate energy through repeated cycling without significant degradation or permanent deformation. Superelastic SMAs possess stable hysteretic behavior over a certain range of temperature, where its shape is recoverable upon removal of load. Alternatively, Martensite SMAs also possess the ability to recover its shape through heating. Both types of SMA demonstrate promise in civil infrastructure applications, specifically in seismic-resistant design and retrofit of structures.

The primary objective of this research is to investigate experimentally the performance of concrete beams and shear walls reinforced with superelastic SMAs in plastic hinge regions. Furthermore, this research program involves complementary numerical studies



and the development of a proposed hysteretic constitutive model for superelastic SMAs applicable for nonlinear finite element analysis. The model considers the unique characteristics of the cyclic response of superelastic materials.

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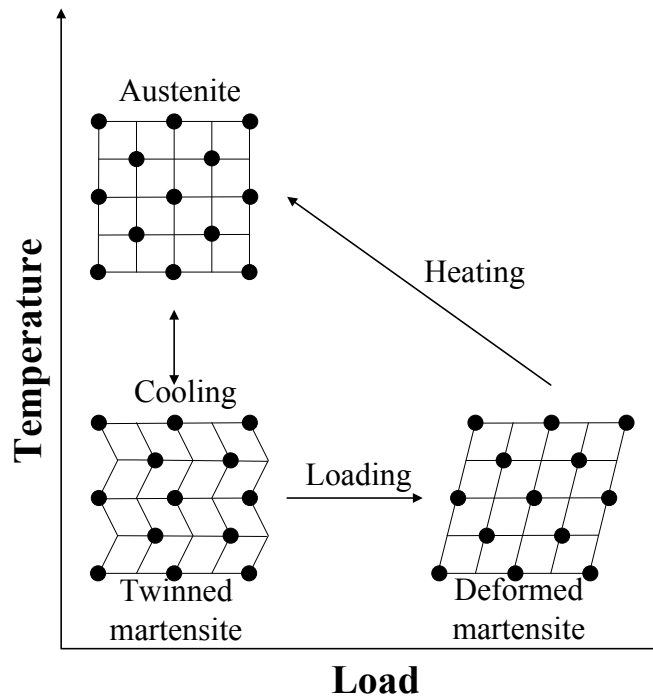
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## CHAPTER 1

### INTRODUCTION

#### 1.1 Background

During the last quarter of the 20<sup>th</sup> century, a new group of alloys known as Shape Memory Alloys (SMAs) evolved. They have been used in the medical, aerospace, automotive, and robotic industries. Recently, SMAs have been used in structures. Due to their unique properties, SMAs are classified as smart materials. In general, SMAs exhibit two distinct crystalline structures or phases: Martensite (M-phase) and Austenite (A-phase). These phases depend on two thermo-mechanical variables: the existing temperature of the composite, and the amount of mechanical loads or stresses applied (Figure 1.1).

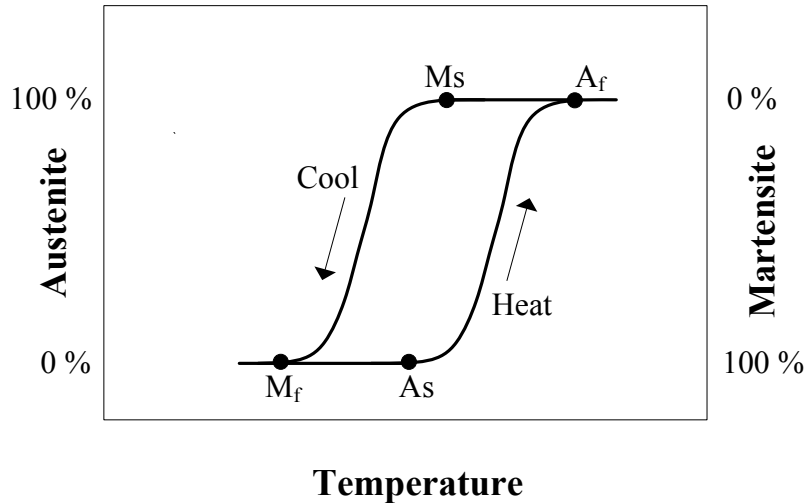


**Figure 1.1** SMA Phases at Different Temperatures and Loads

Martensite is the low temperature, easily deformed shape which has a parallelogram molecular structure. Upon deformation (loading), this phase takes on the second form as shown in Figure 1.1. Austenite, the stronger, body-centered cubic structure, occurs at higher temperatures. Both phases have the same size and shape on a macroscopic scale until the Martensite is deformed.

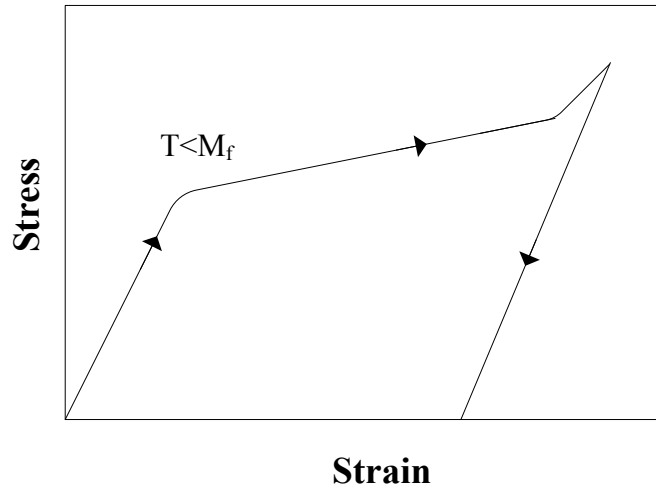
Shape Memory Alloys exhibit six unique properties: shape memory effect (SME) one-way memory, SME two-way memory, superelasticity, damping hysteresis, minimal fatigue, and absence of corrosion. The behavior of SMA exposed to a temperature cycle is shown in Figure 1.2. In the fully Austenitic phase, that is, the "high temperature" phase, the temperature of the SMA is greater than  $A_f$ . Upon cooling, the SMA reaches a temperature at which a phase transformation begins. This transformation is from a fully Austenitic phase to the Martensitic phase.  $M_s$  represents this temperature and marks the initiation of the Martensitic phase, which is the "low temperature" phase. Upon further cooling, the Martensite plates begin to increase until the SMA is in a fully Martensitic state at a temperature of  $M_f$ . If the SMA is heated, the material phase does not change, but the temperature increases. When the temperature reaches  $A_s$ , the deformed Martensite begins to rearrange its crystalline structure to the original configuration. Thus,  $A_s$  represents the temperature at which the "high temperature" phase starts. Upon a further increase in temperature, the re-arrangement continues and is complete at  $A_f$ , when the SMA is in a fully Austenitic state. Therefore, this cycle begins with the SMA in the Austenitic phase, transforming at a "lower temperature" to the fully Martensitic phase, and reverting, upon heating, to the original "high temperature" or parent phase. As

evident in the response to temperature, the SMA exhibits hysteresis when subjected to heating and cooling cycles.



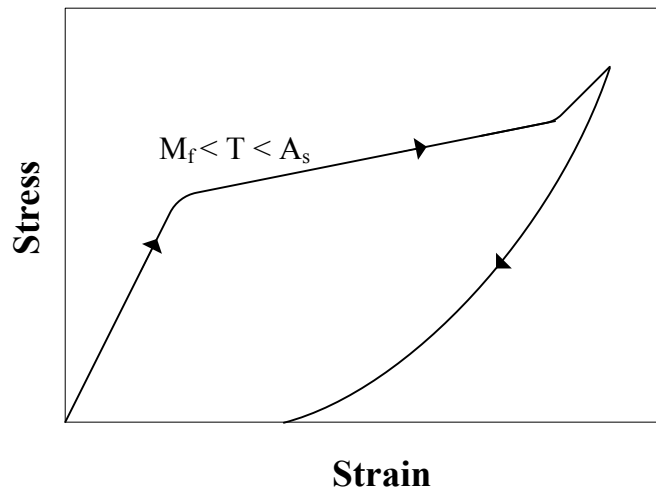
**Figure 1.2** Temperature Hysteresis

The transformation discussed above does not include mechanical stresses, and the cycle was entirely driven by temperature. The influence of stress on the behavior of SMA is dependent on the existing phase. When the SMA is in the Martensite phase, which occurs when the operating temperature of the material is below  $M_f$ , the alloy behaves ferro-elastically and can be easily manipulated (Figure 1.3). The Martensite SMA responds similarly to ordinary reinforcing steel with significant residual deformation.



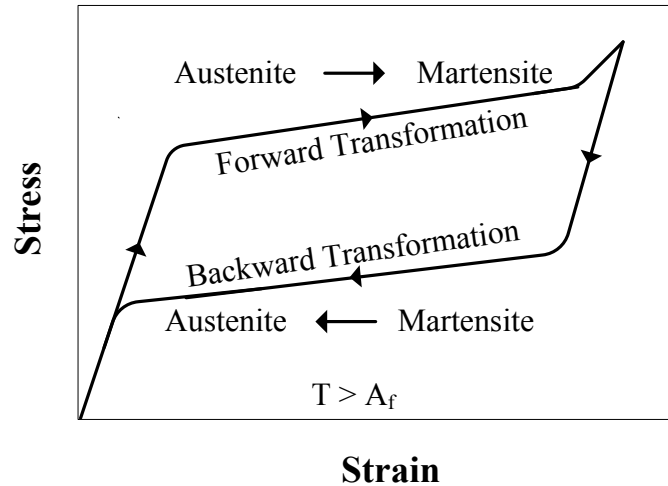
**Figure 1.3** Martensite Stress–Strain Relationship

In the transition stage between  $M_s$  and  $M_f$  (Figure 1.2), the alloy exists in both the Martensitic and Austenitic phases (Figure 1.4). Note that the alloy responds with reduced residual deformations relative to Figure 1.3 owing to the presence of Austenite. When the operating temperature of the alloy is increased and exceeds the Austenite finish temperature  $A_f$ , the response is altered to that shown in Figure 1.5.



**Figure 1.4** Transition Stage Stress–Strain Relationship





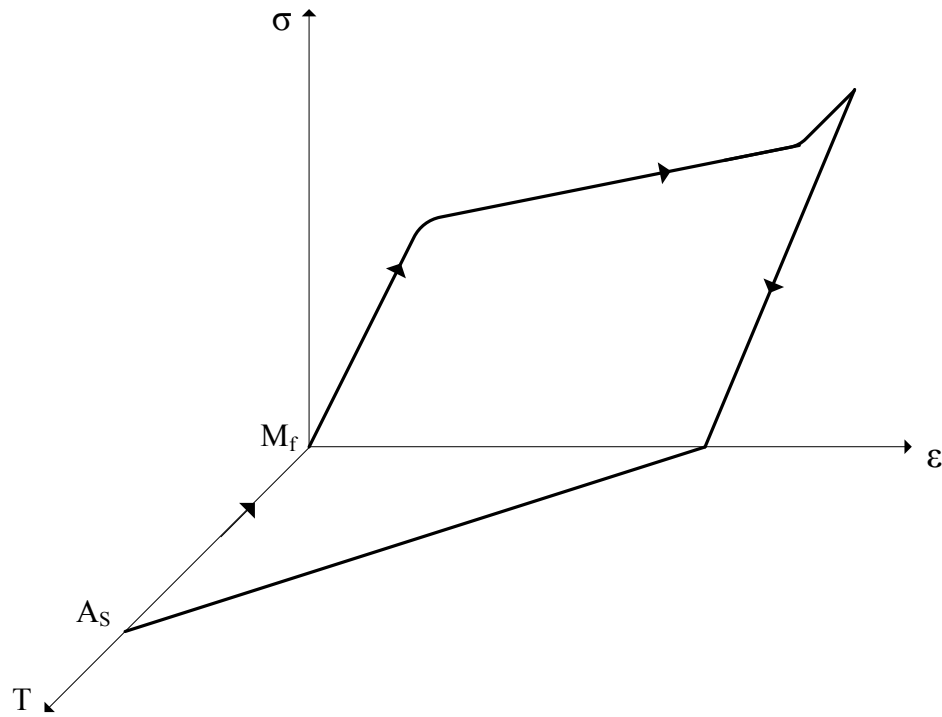
**Figure 1.5** Austenite Stress–Strain Relationship

This process demonstrates that when the material is in the fully Austenite phase, full recovery of the residual deformations is possible. One of the interesting characteristics of SMAs in the fully Austenite phase is pseudo-elasticity (Figure 1.5). Pseudo-elasticity refers to the superelastic characteristic of the SMAs. In this phase, the material transforms from Austenite to Martensite by increasing the external stress and without modifying the temperature. The alloy undergoes re-transformation to the Austenitic state by unloading the stress. The stress-strain curve for the fully Austenite phase shows a distinctive plateau and hysteresis. Shape Memory Alloys can reach strains up to 10% and still fully recover their deformation.

The critical temperatures ( $A_s$ ,  $A_f$ ,  $M_s$ , and  $M_f$ ) where the transformations occur can be chosen from a temperature range between  $-302$  degrees Fahrenheit ( $-150$  degrees Celsius) and  $392$  degrees Fahrenheit ( $200$  degrees Celsius). This depends on the alloy composition and micro-structural constitution; the latter is determined by the thermo-mechanical process. This process is applied during manufacturing of the SMA alloys, although different amounts of cold work and different heat treatments may be used for

different alloys and property requirements. Therefore, in order to optimize the physical and mechanical properties of an SMA product and achieve shape memory and/or superelasticity, the material is cold worked and heat treated.

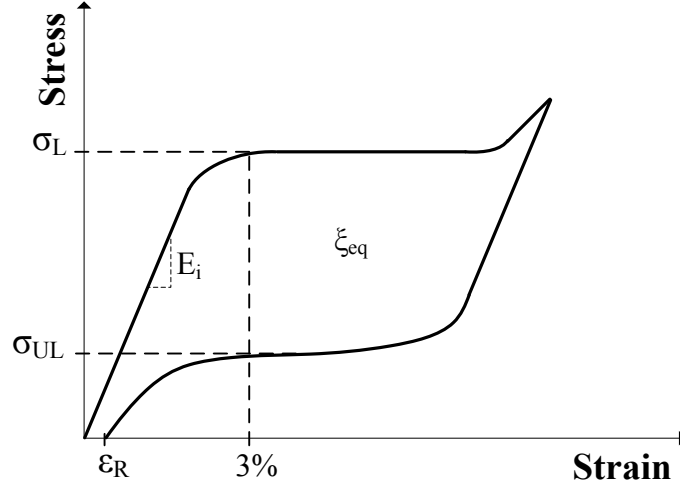
Through the Shape Memory Effect, the material has the ability to memorize a very specific configuration either in the Martensite or Austenite phases, which is known as one-way memory (Figure 1.6). In addition, SMAs are capable of “being trained” to memorize two different deformations in the two different phases. This is accomplished by deforming the alloy in the Martensitic phase and heating it into the Austenitic phase, where it regains its original size prior to deformation (Figure 1.6).



**Figure 1.6** Stress–Strain Temperature Curve for SME

Figure 1.7 provides the key properties which are important for superelastic SMAs when considered in structural applications: forward transformation stress,  $\sigma_L$ ; reverse

transformation stress,  $\sigma_{UL}$ ; initial elastic modulus,  $E_i$ ; residual strain,  $\epsilon_R$ ; and the equivalent viscous damping,  $\xi_{eq}$ . (McCormick et al. 2007).



**Figure 1.7** SMA Properties for Structural Applications (McCormick et al. 2007)

The forward transformation stress refers to the stress at which the Martensite phase transformation initiates. This value identifies the load at which an SMA undergoes a significant stiffness change and is used to determine the restoring force. The reverse transformation stress refers to the stress at which the reverse Martensite transformation occurs and determines the size of the hysteresis loop, which is important in modeling the cyclic behavior of the material. The initial elastic modulus is a measure of the stiffness of the structure which is significant in determining the system response during an earthquake. It is obtained from the slope of the stress-strain curve at the beginning of the loading cycle. The residual strain refers to the plastic deformation and provides a measure of the re-centering capability provided by an SMA element. The last property is the equivalent viscous damping which reflects the hysteretic behavior of SMAs and consequently the system dissipation capacity. These mechanical properties are defined in

the American Society for Testing and Materials (ASTM) Standards ASTM F2004 (2004), ASTM F2005 (2005), ASTM F2063 (2005) and ASTM F2082 (2006), which have been issued for SMA raw materials for use in medical devices.

In these standards, the forward and reverse transformation stresses correspond to a strain of 3.0% and 2.5% during the initial loading and unloading of the specimen, respectively; whereas, the dynamic method of ASTM E1876 (2009) was recommended for the measurement of modulus of elasticity following the same procedure for elastic materials.

Finally, SMAs possess high damping capacity, in both Austenitic and Martensitic phases, but with a slightly higher capacity in the Martensite phase. This is due to the stress-induced Martensitic transformation and re-orientation of Martensitic variants under the application of an external load. Damping, however, is affected by internal and external parameters such as temperature, frequency, time, strain amplitude, alloy content, grain size, and densities of the Martensite and Austenite phases. In general, the damping characteristics of SMAs are categorized into three categories: Martensite, Austenite, and Martensite-Austenite. The first category is a typical feature of Martensite SMAs: higher damping with respect to Austenite and the ability to recover residual strain by heating (Shape Memory Effect). The second category is a typical feature of Austenite SMAs: nominally zero residual strain when subjected up to 10% straining under arbitrary mechanical loading (pseudo-elastic effect) where the damping is less than that of the first category but with the benefit of full recovery. The third category contains common features of both phases: high damping capacity, and practical absence of fatigue and corrosion effects.

The unique properties of SMAs are attractive for use in earthquake-resistant structures. In particular, Nickel-Titanium (NiTi) alloys have been found to be the most promising SMA materials for seismic applications, specifically for their ability to recover large deformations and dissipate significant energy. Nickel-Titanium alloy (NiTi) is the most common type of superelastic (Austenite) SMA. It is an equiatomic compound of NiTi (55.9% Nickel and 44.1% Titanium), with a transformation temperature ( $A_s$  and  $A_f$ ) range between  $-198$  degrees Fahrenheit ( $-92$  degrees Celsius) and  $230$  degrees Fahrenheit ( $110$  degrees Celsius). It has a shape-memory strain limit of approximately 8%, and is corrosion resistant and thermally stable. The generic trade name for NiTi alloys is Nitinol: Ni for Nickel and Ti for Titanium, while (nol) refers to Naval Ordnance Laboratory where the shape memory effect was first discovered.

In summary, an SMA in the Austenite phase will fully recover residual deformations through unloading of mechanical stress, while heat is required to recover residual deformations when the alloy is predominantly in the Martensite phase. The Austenite finish temperature  $A_f$  is specified for the manufacturing process. Thus, the end-user has control over which phase is preferred, Austenite or Martensite.

## 1.2 Research Objectives

The primary objective of this research is to investigate the performance and recovery capacity of concrete beams and shear walls reinforced with superelastic SMAs (Austenite) in the plastic hinge area. The philosophy is to provide structural elements that can recover inelastic deformations, such that structures can remain serviceable after seismic events and limit the amount of repair. In addition, a new hysteretic constitutive model for SMA reinforcing bars is presented in consideration of the unique

characteristics of the cyclic response of superelastic materials. The material model is incorporated in the nonlinear finite element program VecTor2 (Vecchio 1989), which is based on the Modified Compression Field Theory (Vecchio and Collins 1986) and the Disturbed Stress Field Model (Vecchio 2000).

The objectives of this research program are as follows:

### **Reinforced Concrete Beams**

- Design and construct large-scale beams reinforced with conventional and large diameter SMA bars. Previous research in the literature focused either on small-scale beams reinforced with SMA wires or larger scale beams reinforced externally with small diameter SMA bars. The major contribution here is an understanding of the performance of large-scale beams reinforced with large diameter SMA bars. This provides the basis for understanding the behavior of full-scale structural members. Furthermore, this testing phase provides significant data to develop and corroborate the proposed hysteretic constitutive model for superelastic SMAs.
- Test the beams under different loading conditions (monotonic, cyclic, and reversed cyclic). Testing previously conducted considered either monotonic or cyclic; not necessarily in the same testing program. The objective is to understand the behavior under different loading conditions in a controlled and consistent test program.
- Load the beams to failure to verify the failure mode and to have a complete description of the specimen behavior. In previous research, SMA beams were

tested up to a certain percentage of strain in order to save the SMA materials for future work.

### **Reinforced Concrete Shear Walls**

- Design, construct and test large-scale ductile shear walls reinforced with conventional and SMA bars under reversed cyclic loading. To date, only two squat shear walls were reinforced with external Martensite and Austenite bars as replacement of internal reinforcement.
- Investigate the response of a shear wall reinforced with both SMA and conventional bars (hybrid effect) in the critical region. The wall is reinforced with SMA bars in the plastic hinge region at the boundaries, while all other regions are reinforced with conventional deformed steel. The re-centering ability of the SMA wall is monitored during the test to study the behavior of the SMA when combined with conventional reinforcement.
- Provide large-scale specimens to corroborate the proposed constitutive SMA model.
- Investigate the effectiveness of different coupling systems (mechanical connections between SMA bars and conventional deformed steel).

### **SMA Constitutive Hysteretic Modeling**

- Develop a new constitutive model to simulate the hysteretic behavior of superelastic SMAs. This model is incorporated in Program VecTor2 to be compatible with the Modified Compression Field Theory for the analysis of reinforced concrete structures.

- Define a realistic shape for the loading and unloading behavior of SMA which can be modeled as linear or nonlinear. The goal is to develop a simple, yet realistic model that can capture salient features of behavior including the forward transformation phase, reverse transformation phase, and damping.
- Identify the plastic offset strain upon unloading, which is a function of the maximum unloading strain. Available models in the literature assume full recovery for the SMA provided that the maximum strain is limited within a certain range. Current models do not consider the behavior of superelastic SMAs subjected to significant strains, and most models have implemented linear loading, reloading, and unloading responses.

### 1.3 Thesis Layout

Chapter 2 contains a comprehensive literature review of the material behavior of Shape Memory Alloys, analytical models, experimental studies and seismic applications using SMAs. A description of the experimental program is discussed in Chapter 3. Test results and discussion of the experimental results are detailed in Chapters 4 and 5, respectively. Chapter 6 contains details of the proposed hysteretic constitutive model for superelastic SMAs, and Chapter 7 presents finite element analysis of the reinforced concrete beams and shear walls tested in this research program. Verification of the proposed constitutive model using concrete specimens reinforced with superelastic SMA available in the literature is presented in Chapter 8. Finally, conclusions are provided in Chapter 9.



## CHAPTER 2

### LITERATURE REVIEW

#### 2.1 Introduction

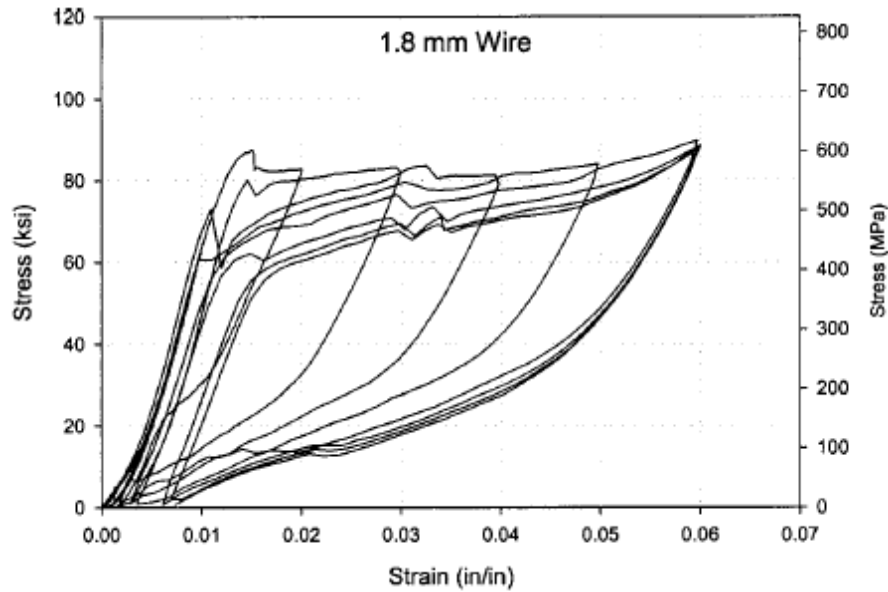
Limited research has focused on understanding the mechanisms of superelastic SMAs to recover deformations and decrease residual deformations in concrete members. Most research on the deformation recovery of SMAs has concentrated on transforming Martensite into the superelastic Austenite through the process of heating by means of an external heat source or current. This chapter contains a literature review pertaining to the use of SMAs for structural applications.

#### 2.2 Material Behavior of Shape Memory Alloys

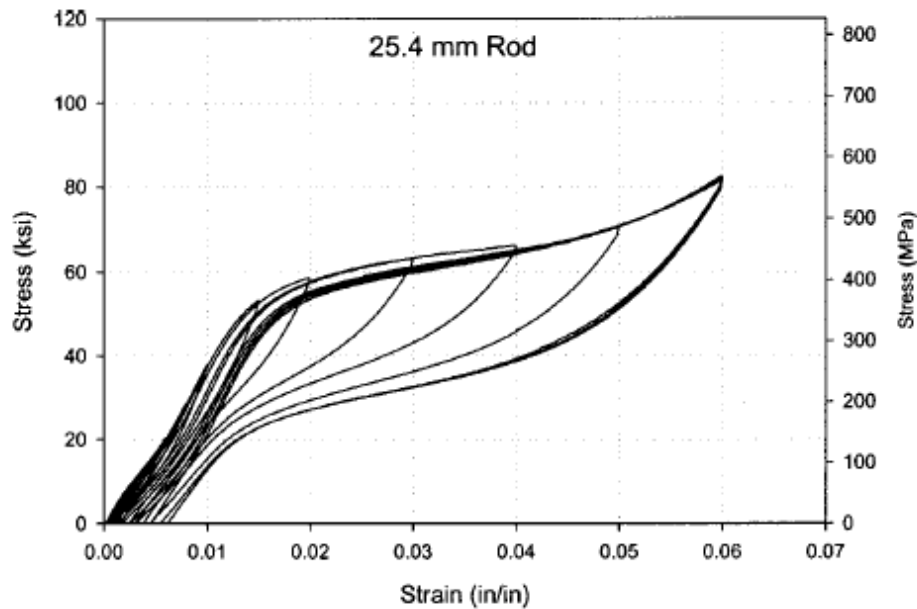
Material studies have focused on the cyclic properties of SMA wires and large diameter bars with respect to strain amplitude, loading frequency, number of cycles, and ambient temperature.

DesRoche et al. (2004) studied the performance of superelastic NiTi under cyclic loading in an effort to evaluate the use of NiTi in seismic applications. The cyclic properties of 1.8 mm diameter SMA wires were compared to that of 25.4 mm diameter bars to investigate the effect of bar size and loading history on the strength, damping and re-centering properties of the SMAs. The results showed that nearly ideal properties were obtained in the wire and bar form for the superelastic NiTi Shape Memory Alloy (Figures 2.1 and 2.2). However, the wire form of the SMA demonstrated higher strength and damping properties compared with the bars. The re-centering capabilities based on residual strains were not affected by section size. Overall, the damping potential of SMAs

in superelastic form was low for both wires and bars, typically less than 7% equivalent viscous damping. Cyclical strains greater than 6% lead to degradation in the damping and re-centering properties of the SMAs.

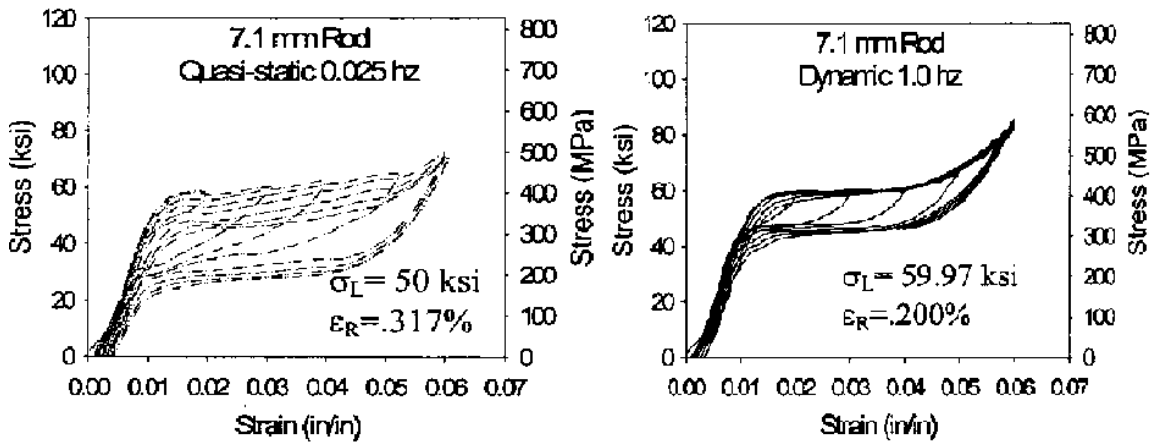


**Figure 2.1** Stress–Strain for 1.8 mm Diameter Nitinol Shape Memory Alloy Wire Subjected to Quasi-Static Cyclic Loading (DesRoche et al. 2004)



**Figure 2.2** Stress–Strain for 25.4 mm Diameter Nitinol Shape Memory Alloy Bar Subjected to Quasi-Static Cyclic Loading (DesRoche et al. 2004)

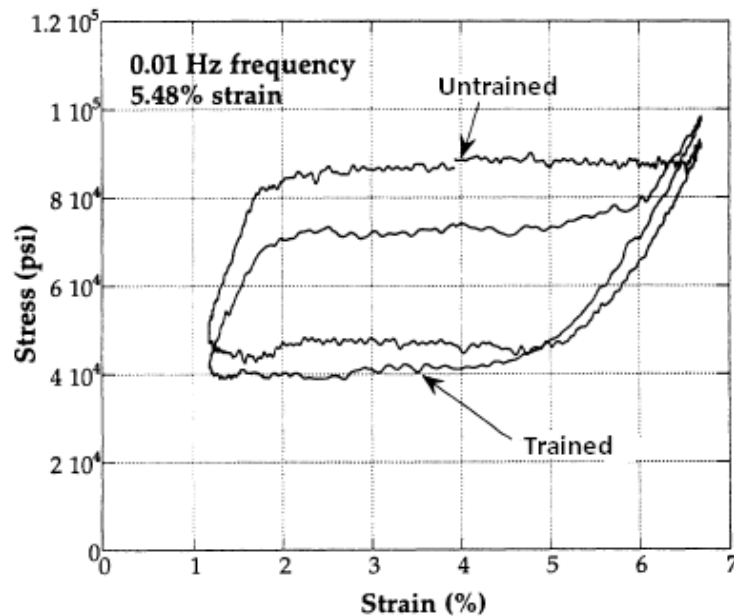
Strain rate effects were evaluated by subjecting the SMAs to loading rates representative of typical seismic loading. Results show that an increase in strain rate leads to an increase in both the loading (forward transformation) and unloading (reverse transformation) stresses, being more pronounced for the unloading stress. This produces an upward shift and a narrowing of the hysteresis loop as indicated in Figure 2.3. Increasing loading rates lead to decreases in the equivalent damping, with negligible effects on the re-centering properties of the SMAs.



**Figure 2.3** Stress–Strain for 7.1 mm Diameter Nitinol Shape Memory Alloy Bar Subjected to Quasi-Static and Dynamic Cyclic Loading (DesRoche et al. 2004)

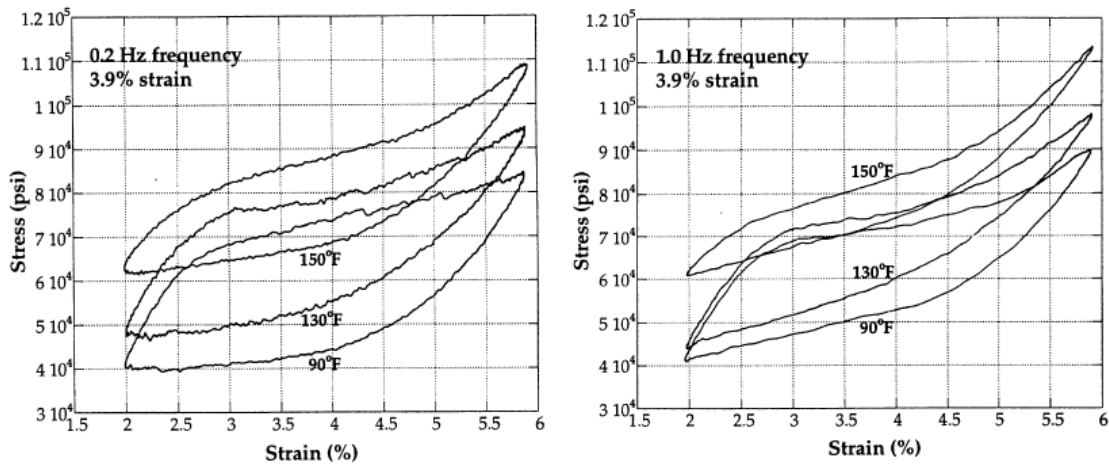
McCormick et al. (2007) investigated the behavior of large diameter SMA bars (31.75, 19.1 and 12.7 mm) under loadings typical of earthquakes and provided a link between the full-scale deformation response and the underlying material structure. The investigation comprised of cyclic tests on coupons and full-scale specimens. The results demonstrated that coupon specimens provided only limited information applicable to the full-scale behavior. Full-scale tests, using earthquake-type loading, showed similar behavior to the tensile cyclic tests suggesting that NiTi is applicable for structural applications.

Wolons et al. (1998) confirmed that a significant amount of mechanical cycling is required to stabilize SMA properties due to residual strains that appear in the early cycles. While each loading cycle induces residual strain, it is the first cycles that produce the largest residual strains with later cycles producing none. Figure 2.4 shows 5.48% strain hysteresis cycles for a trained and an untrained wire. It should be noted that the phase transformation plateau for the trained wire is lower than that for the untrained wire (the Martensitic forward transformation stress is reduced). Likewise, the stress required to produce the reverse transformation is also reduced. However, the reduction in the reverse transformation stress is less than the decrease in the forward transformation stress after cycling which resulted in smaller hysteresis loop. Since the hysteresis loop is smaller for a trained wire, the energy dissipation of a trained wire is lower. Therefore, the effect of training on energy dissipation should be considered for applications requiring damping.



**Figure 2.4** Hysteresis Response for Trained and Untrained SMA at Low Frequencies (Wolons et al. 1998)

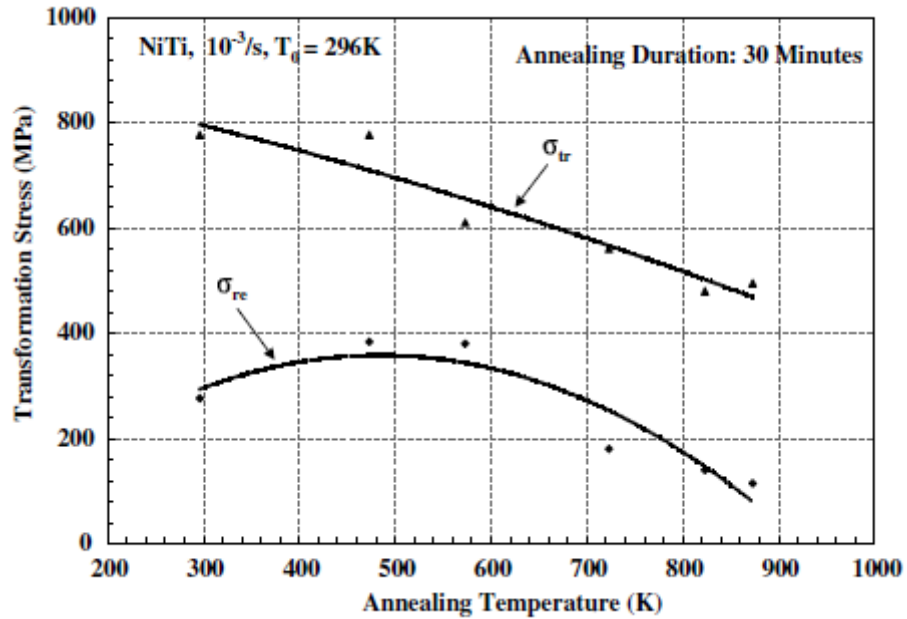
In the same research, the authors studied the effect of temperature and loading frequencies on the stress-strain relationship of NiTi SMAs as shown in Figure 2.5. It can be observed that the forward transformation from Austenite to stress-induced Martensite occurs at higher stress levels, shifting the hysteresis loop upwards. In addition, the hysteresis loop flattens out, reducing the energy dissipated as temperature increases. The authors suggested that the change in shape of the hysteresis loops can be associated with operation outside of the ideal pseudo-elastic temperature window which is usually no more than 60 to 80°C (108 to 144°F). Furthermore, the authors stated that at low loading frequencies (0.2Hz) the energy dissipated at 54°C (130°F) is 85% of that dissipated at 32°C (90°F). This ratio undergoes a rapid initial decrease before stabilizing at 60% for frequencies of 2Hz and above.



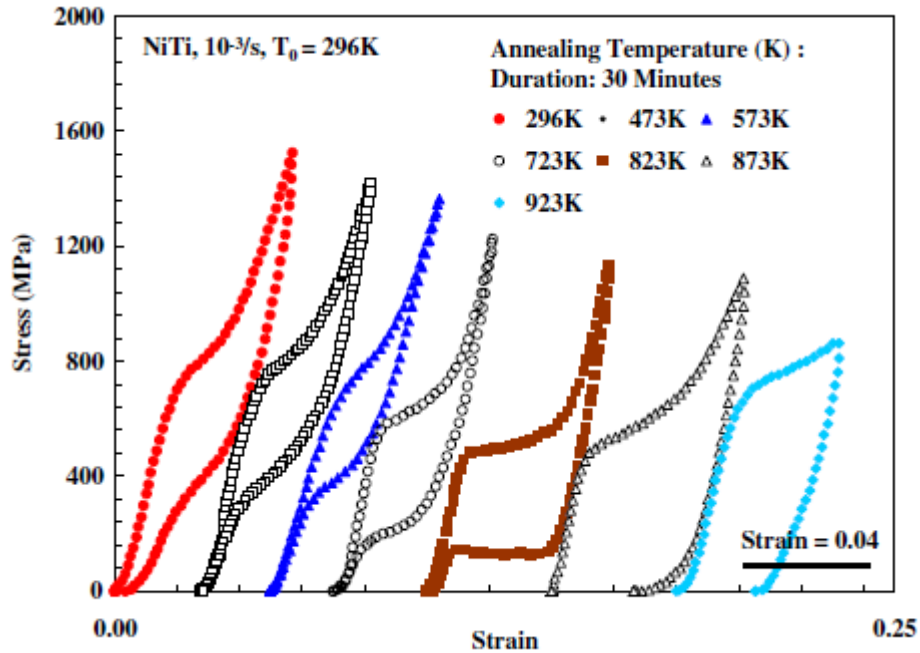
**Figure 2.5** Hysteresis Response at Different Frequencies (Wolons et al. 1998)

The effect of temperature and its duration on the superelastic behavior of SMAs was further investigated by Nemat-Naser and Gou (2006). Cylindrical specimens of 4.5 mm nominal diameter and 5 mm height were annealed (heat treated) at different temperatures and durations in a furnace with an accuracy of  $\pm 2^\circ\text{C}$ . The samples were then compressed

at a strain rate of  $10^{-3}/s$  at room temperature to a maximum strain of approximately 5.5% using an Instron servo-hydraulic testing machine. The experimental observations indicated that the annealing temperature and duration have a significant effect on the superelastic behavior of SMAs, specifically on the transformation stresses (Figure 2.6), residual strain (Figure 2.7), and dissipated energy (area of hysteretic loops in Figure 2.7). Decreasing the annealing temperature induces a greater residual strain and high transformation stress which leads to maximizing the dissipated energy. The researcher noted that the annealing duration has a weak effect on the superelastic behavior as compared with that of the annealing temperature. Finally, annealing temperature above 823 K ( $550^{\circ}\text{C}$ ) removes all the superelastic properties of the SMAs.



**Figure 2.6** Forward and Reverse Transformation Stress as a Function of Annealing Temperature (Nemat-Naser and Guo 2006)



**Figure 2.7** Stress–Strain Curves for Different Annealing Temperatures (Nemat-Naser and Guo 2006)

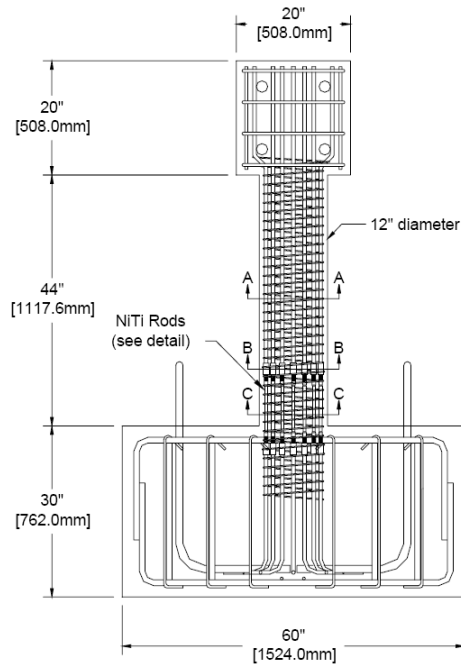
### 2.3 Shape Memory Alloy Applications

The unique properties of SMA under different loading and temperature conditions have made it attractive for various engineering applications.

#### 2.3.1 Reinforcement in Reinforced Concrete Structures

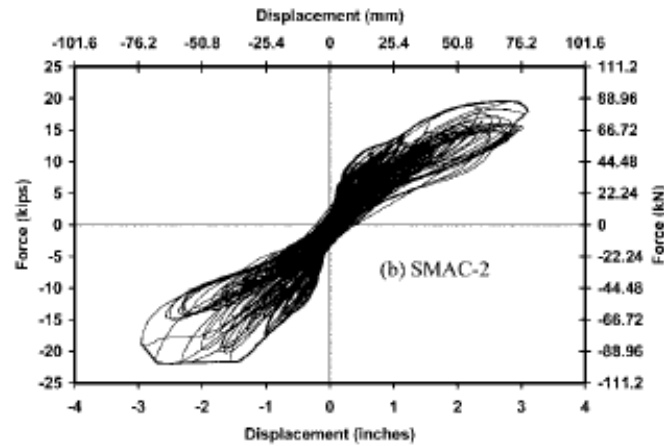
The contribution of SMAs to reduce the residual displacement in concrete columns reinforced with SMA bars in the plastic hinge area was investigated by Saiidi and Wang (2006). The authors evaluated the seismic performance and damage in a SMA-reinforced column repaired using engineered cementitious composites (ECC). A quarter-scale, spiral reinforced concrete column (Figure 2.8) with SMA longitudinal reinforcement in the plastic hinge area was tested on a shake table. After the column was damaged by simulated earthquakes, it was repaired using ECC in the plastic hinge area and then

retested. The tests demonstrated that SMA-reinforced columns were able to recover nearly all of the post-yield deformation (Figure 2.9) and the use of ECC reduced the concrete damage significantly, thus requiring minimal repair even after a strong earthquake. The authors proposed a new hysteresis model for SMA-reinforced members, which was able to satisfactorily predicted displacement histories.



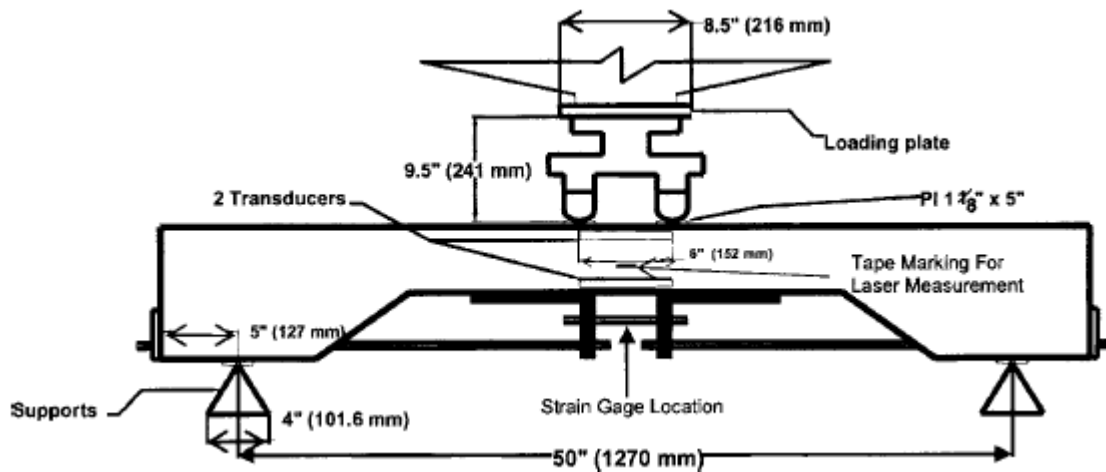
**Figure 2.8** Specimen Details of Columns Reinforced with SMAs (Saiidi and Wang 2006)





**Figure 2.9** Measured Cumulative Hysteresis Curves for Columns Reinforced with SMAs (Saïdi and Wang 2006)

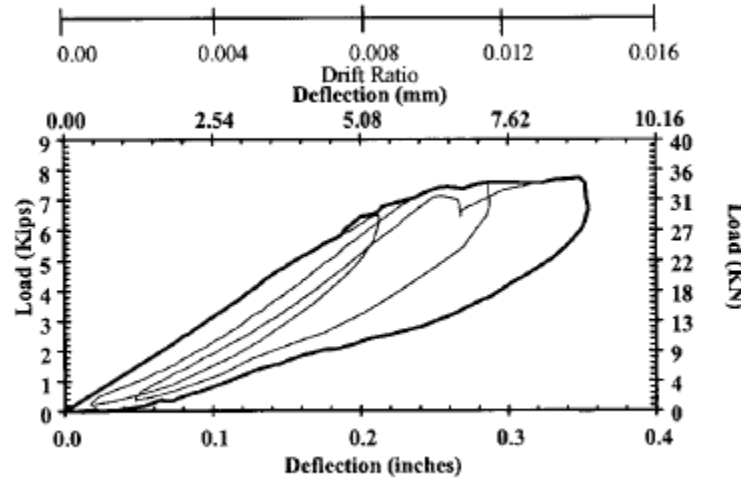
Ayoub et al. (2003) studied the superelastic behavior of Shape Memory Alloy (NiTi) reinforcement in small-scale concrete beams (Figure 2.10) that were tested under half-cycle loads.



**Figure 2.10** Beam Test Setup (Ayoub et al. 2003)

The experimental results (Figure 2.11) indicated that the average residual displacement in the NiTi reinforced beams was less than one-fifth of the corresponding residual

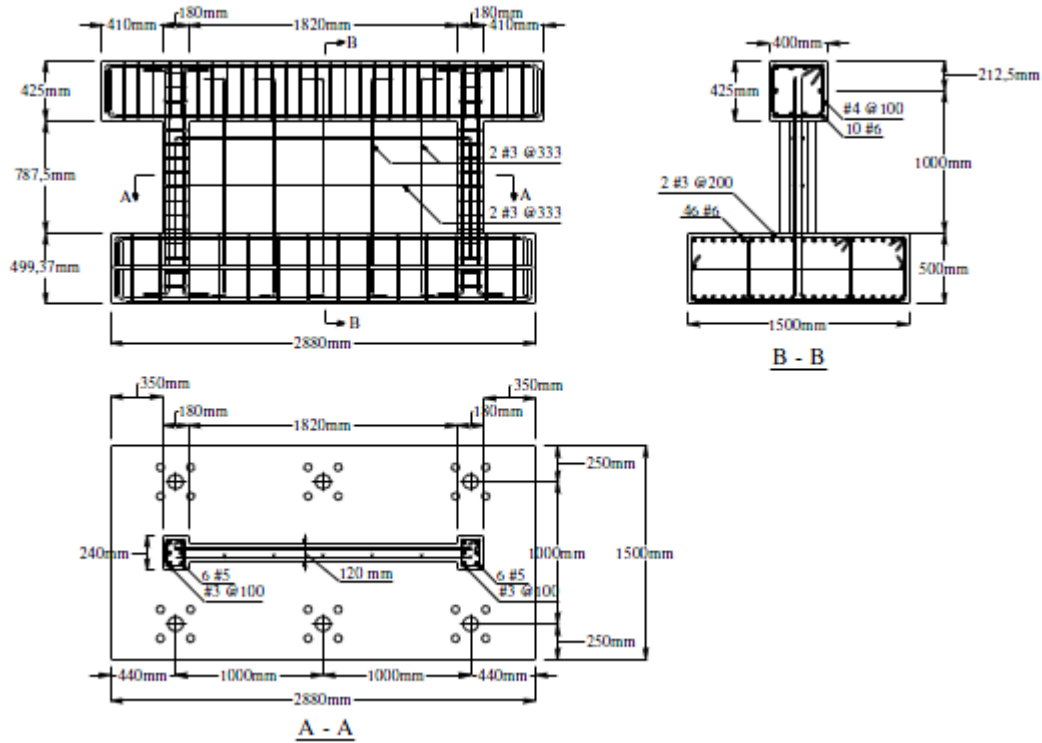
displacement in the companion steel reinforced beams. The stiffness of the NiTi reinforced beams, however, was lower than the steel reinforced beams.



**Figure 2.11** Load Versus Displacement Curves for Beams Reinforced with SMAs (Ayoub et al. 2003)

Alam et al. (2007) investigated the seismic behavior of beam-column joints reinforced with superelastic SMAs. Two large-scale beam-column joints were tested under reverse cyclic loading. The first joint was reinforced with regular steel bars, while the second joint was reinforced with superelastic SMA bars. Both joints were selected from a reinforced concrete building located in the high seismic region of western Canada, and designed and detailed according to the NBCC (2005) and CSA A23.3-04 (2004) recommendations. The behavior of the two specimens under reverse cyclic loading, their load-storey drift, moment-rotation envelope relationship, and energy dissipation ability were compared. The results demonstrated that the SMA-reinforced beam-column joint specimen was able to recover most of its post-yield deformation. Thus, it would require a minimum amount of repair after a strong earthquake.

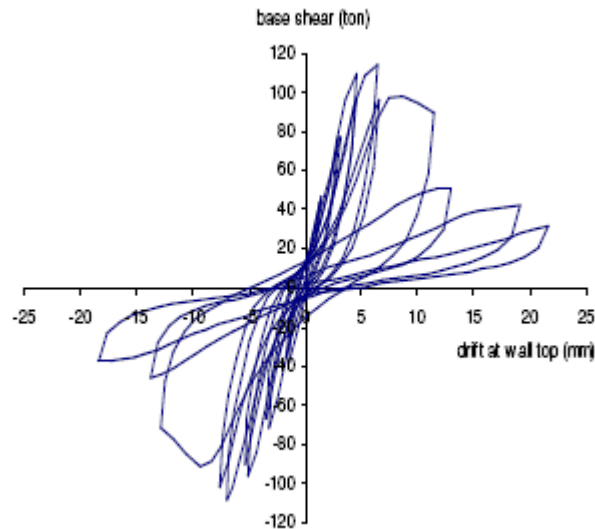
Effendy et al. (2006) investigated SMA bars acting as external structural bracing to increase ductility and energy dissipation capacity in low-rise shear walls. Three shear walls with dimensions of 1.0 m in height, 2.0 m in length, and 0.12 m in width were tested (Figure 2.12).



**Figure 2.12** Dimensions and Reinforcement Layout of Shear Wall Specimens (Effendy et al. 2006)

The first test specimen had only steel reinforcement at a ratio of 0.48% in each orthogonal direction. The second and third shear walls were reinforced with superelastic Austenite and Martensite SMA bars, respectively. Both walls had 0.24% steel and 0.24% SMA. The SMA bars were externally placed forming two diagonals at 25 degrees to the horizontal. The force-displacement hysteretic loops under reverse cyclic loading were recorded (Figure 2.13). The test results demonstrated that SMA bars increased the

maximum shear strength of the walls by about 16% in the Martensite shear wall and 26% in the Austenite shear wall. The shear wall with Martensite SMA bars experienced greater residual displacement relative to the shear wall with superelastic SMA bars. The Martensite reinforced shear wall failed due to weak boundary elements and weak interface between the boundary element and the foundation; therefore, the strength and ductility capacities of the wall were not realized. In addition, one of the superelastic SMA bars in Austenite shear wall buckled, decreasing the energy dissipation capacity of the wall.



**Figure 2.13** Force-Displacement Curve of Specimen with Superelastic SMA (Effendy et al. 2006)

Alam et al. (2008) studied numerically the behavior of SMA reinforced concrete bridge piers. Two quarter-scale spiral RC columns representing reinforced concrete bridge piers designed, constructed and tested previously using a shake table by Wang et al. (2004) were part of this study. Shape Memory Alloy bars were placed in the plastic hinge region and connected to steel bars with mechanical couplers. The performance of the bridge

piers demonstrated excellent re-centering capability when subjected to earthquake loading. It was observed that SMA reinforced concrete columns were superior to conventional RC columns in limiting relative column top displacement and residual displacements.

### 2.3.2 Prestressing Tendons

Pre-stretched Martensite SMA tendons can be placed in concrete forms during construction or after placing the concrete in post-tensioning ducts. In both techniques, a source of heat is required to induce post-tensioning. Jacking equipment is not necessary since an electrical source to heat-trigger the tendons is solely required.

Shape Memory Alloy composites were tested by Opara and Naaman (2000) to investigate their ability to improve reinforcing and prestressing of concrete structures. Shape Memory Alloys are able to deform and have their dimensions changed by heat, electricity, or radiation treatment. In the initial Martensitic phase, plastic deformation of SMAs is recoverable through a heat source, thus SMAs have the ability to return to their original shape. Such physical changes through phase changes are used in prestressing of concrete. Using plate-shaped concrete specimens, the authors investigated the concept of self-stressing composites with SMAs. Two specimens were plain mortar, five were steel wire reinforced, and two were SMA reinforced in which self-stressing was triggered using temperature. The results demonstrated that SMA wires in concrete behave as self-stressing composites capable of self-healing, where significant deformation beyond the first crack was fully recovered and cracks fully closed. The results also showed that deflection due to loads fully recovered after unloading. Additional benefits included less deflection and smaller strains. It was observed that when higher tensile stresses were

applied, cracking was delayed or avoided under service load conditions, and the capacity was increased at ultimate.

El-Tawil and Ortega-Rosales (2004) investigated the ability of SMA bars to provide post-tensioning without jacking equipment. Martensite Shape Memory Alloy tendons were first prestrained by prestretching. Then, mortar beams were cast around the prestretched SMA bars and a heat-trigger was used to induce a post-tensioning effect. The effect of prestraining levels on the mechanical properties was studied as well. Tensile tests of the SMA specimens showed that higher prestraining resulted in higher recovery stresses, while the results from the beam tests indicated that significant post-tensioning was achieved.

Arup and Nagret (1998) realized that the Shape Memory Effect in Nitinol alloy can be used as a method of inducing additional prestressing in concrete. Martensite Nitinol strands were elongated beyond their plastic limit and embedded into mortar beams in order to provide prestressing forces. The prestressing was induced by heating the strands, creating a Martensite to Austenite transformation that generated large shrinkage strains. It was noted that the SMA strands developed good bond to the surrounding concrete through mechanical interlocking where the strands transferred stresses to the concrete beam even beyond the yield stress of the SMA. The testing illustrated the ability of SMA to apply varying prestressing forces which could be used to increase the capacity of structures in service or to remedy prestress losses over time.

### 2.3.3 Restrainers

One of the major problems of bridges during earthquakes is their unseating due to excessive relative hinge opening and displacement. Limitations of existing unseating

prevention devices include small elastic strain range, limited ductility, and no re-centering capability. These limitations can be overcome by introducing SMA restrainers. Johnson et al. (2004) investigated superelastic Nitinol SMA as restrainers to retrofit bridges. A series of tests were conducted to determine the effect of SMA restrainers on the seismic performance of concrete box girder bridges. Steel and SMA restrainer cables were used and their performance in reducing hinge relative displacements was compared. The SMA restrainers showed promise as restraining devices to limit hinge displacement in bridges and the ability to dissipate energy. Under equivalent loading, the steel restrainers produced relative hinge displacement approximately three to four times that of the SMA restrainers. The hysteretic damping that was experienced under larger ground accelerations highlighted the material's ability to dissipate energy with small residual strains.

DesRoches and Delmont (2002) performed an analytical study on superelastic SMA bars as restrainers for use in multi-span bridges. A three span bridge supported on multi-column bents was used for the analytical model. The SMA restrainers were connected from the pier cap to the bottom flange of the beam in a manner similar to typical cable restrainers. Nonlinear analyses of a typical multi-span simply supported bridge were performed to assess the performance of the SMA and steel restrainer bars. The results showed that SMA restrainer bars were more effective than the steel type in limiting the relative displacement at the piers and abutments.

Bidaux et al. (1997) explored the potential of Nitinol Shape Memory Alloy rods as restrainers in multi-frame bridges. Nitinol Shape Memory Effect properties, superelasticity and damping characteristics were examined for the adequacy of Nitinol to

behave as steel restrainers. Initially, tests were performed to study the effect of bar size on the superelastic property of Nitinol. Two NiTi specimens with a diameter of 1.78 mm and 12.7 mm were tested under cyclic loading. The behavior of the wire (1.78 mm) showed the classic “flag-shape” hysteresis, whereas the rod (12.7 mm) showed a smaller hysteresis loop. This implies that the equivalent viscous damping in the rod was smaller than that of the wire. Furthermore, the residual strain in the rod was smaller than that of the wire. An analytical model of a bridge with 4 frames and 11 concrete box girders across the width, supported on single column bents, was used. The bridge was analyzed with three models: (1) without restrainers, (2) with steel cable restrainers and (3) with superelastic SMA restrainers. Restrainers were used to connect the end of each box girder to the beginning of the adjacent box girders. The analytical modeling consisted of a 2-dimensional nonlinear model of the bridge using DRAIN-2DX nonlinear analysis program. The results indicated that the maximum relative hinge displacement using SMA restrainers was reduced by 35% and 42% in comparison with conventional steel restrainers and no restrainers, respectively.

#### 2.3.4 Connectors

SMAs have been used in the Martensite phase to improve the performance of steel bolted joints. Ocel et al. (2004) investigated the effectiveness of partially restrained steel beam-column connections using SMAs in the Martensite phase. The full-scale connections consisted of four large diameter Nitinol tendons connecting beam flanges to column flanges. The connections exhibited high energy dissipation, large ductility capacity and no strength degradation after being subjected to load cycles up to 4% drift. As the SMAs were in the Martensite phase during testing, they were heated to recover the residual



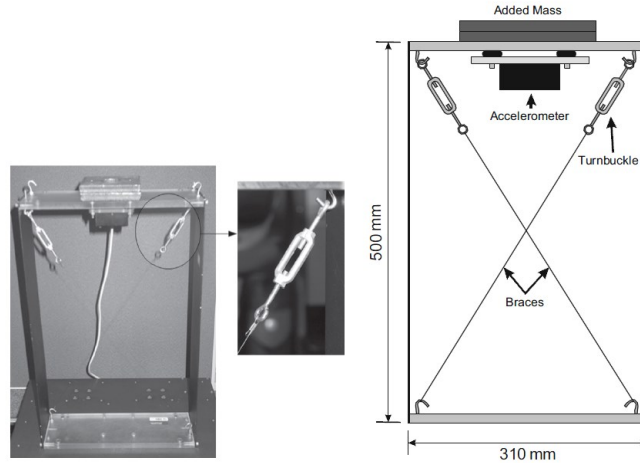
beam tip-displacements. The connections were then retested and displayed similar hysteretic behavior. These tests were performed under quasi-static loading and were then repeated under dynamic loading. The dynamic loading test results were similar to the quasi-static results except for a reduction in energy dissipation.

### 2.3.5 Bracing Members

Frame structures that are susceptible to lateral loading can be effectively retrofitted with SMA bracings. The SMA wire braces are installed diagonally in the frame structures. As the frame structures deform under excitation, SMA braces dissipate energy through stress-induced Martensite transformation (in the superelastic SMA case) or heat-induced Martensite reorientation (in the Martensite SMA case).

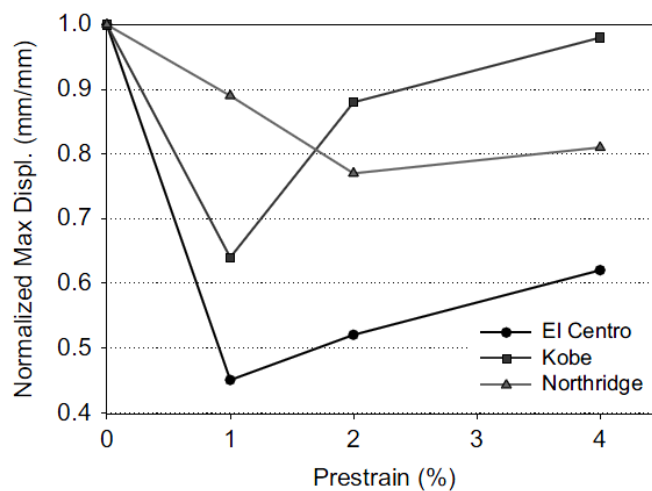
McCormick and DesRoches (2003) conducted an analytical evaluation on the effectiveness of using large diameter SMA bars in the superelastic Austenite phase as bracing members. In this study, a number of numerical analyses were performed considering six-storey concrete frame structures to evaluate the dynamic performance of different bracing systems: buckling-restrained steel braces versus superelastic SMA braces. A reduction in the inter-storey drift and column rotation of a RC frame when using the SE bracing compared to the performance of regular steel braces was observed. The average drift of traditional steel braced frames was decreased by approximately 20% when replaced with superelastic SMA braces.

Lafortune et al. (2007) conducted an experimental and analytical study on the effectiveness of using SMA braces to control the response of frame structures under seismic loading. Shake table tests were performed on a braced single bay steel frame (Figure 2.14).



**Figure 2.14** Experimental Test Setup and Structure (Lafortune et al. 2007)

The test variables were the bracing system and the prestraining level; the SMA and steel braces were used with different prestraining levels. Test results showed that the SMA bracing system provided a consistent reduction in the residual deformation due to the re-centering capability when compared to the steel braces. It was determined that an optimal prestraining level under different ground motion records is in the range of 1.0–1.5% (Figure 2.15).



**Figure 2.15** Normalized Maximum Displacement Obtained for Different Pre-strain Levels (Lafortune et al. 2007)

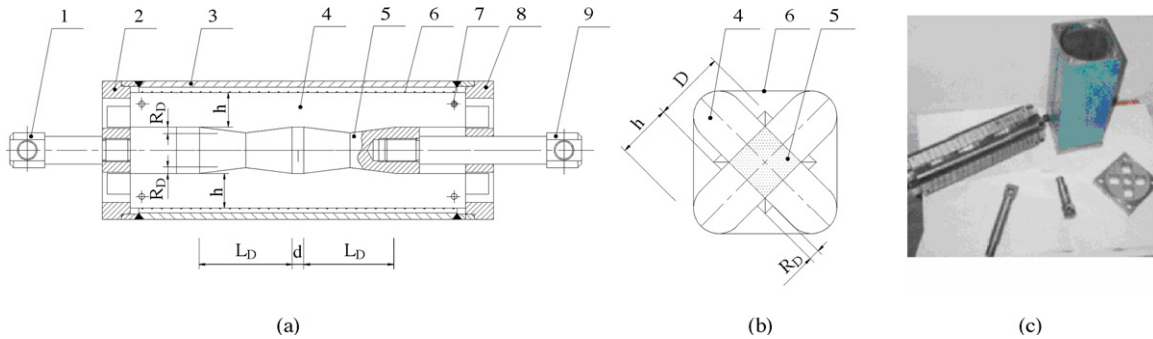
The frame tests were analyzed using the general-purpose finite element software ANSYS to simulate the shake table tests and to verify the applicability of the proposed multi-linear models for the SMA. The analysis showed that the displacement-time history was satisfactorily captured in the analysis. The recorded peak-displacement was 6% to 11% greater than the experimental results.

Cardone et al. (2004) carried out a shake-table test program on quarter-scale reinforced concrete frames designed for gravity loads. The performance of the bare frame, a frame with traditional steel bracing, and a frame braced with superelastic SMA placed inside concentric steel tubes were evaluated under uni-directional and bi-directional motions. It was found that for the retrofitted SMA braced frame, the natural frequency was higher than that of the steel braced frame due to the flexibility effect of dissipating components in the steel braced frame. The test results confirmed the feasibility of using SMA braces to retrofit concrete frames due to higher stiffness, lower weight, and re-centering capabilities.

#### 2.3.6 Dampers and Isolators

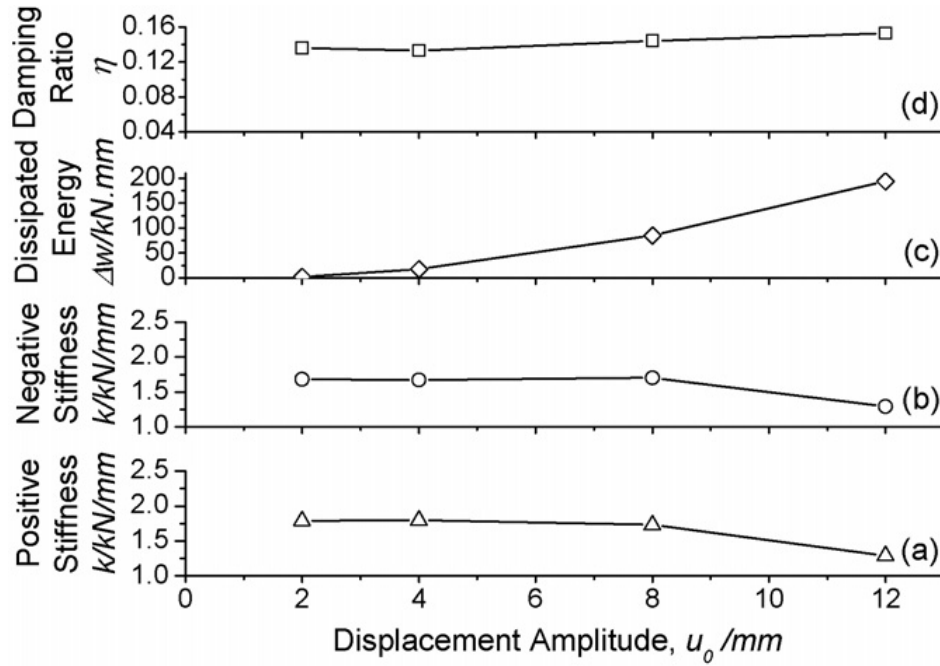
Both superelastic and Martensite SMAs can be used as damper elements for bridges. Clark et al. (1995) conducted experimental and analytical tests on the ability of SMAs to dissipate energy by using SMA dampers made by wrapping SMA wire between two cylindrical support posts. The tests demonstrated that SMA damping devices reduced accelerations and structural displacements by 1/3 to 1/2 over conventionally designed structural systems. In addition, the authors determined that SMAs exhibit excellent fatigue life. The device was also modeled in a six-storey steel frame and the results demonstrated that the device behaved well in reducing the displacement response of the

structure under earthquake-type excitations. Meanwhile, Casciati and Osman (2005) investigated SMAs in their Martensite phase as dampers. The authors utilized large diameter bars in the Martensite phase to dissipate energy through elasto-plastic deformations under strong dynamic action. The tests confirmed that Martensite phase SMAs exhibit large hysteretic cycles with suitable ductility and high energy dissipation. Zuo et al. (2007) investigated the influence of frequency and displacement amplitude of a damper (Figure 2.16) composed of SMA rods (1, 9), left and right covers (2, 8), a barrel (3), an external sliding block (4), an inner sliding block (5), SMA wires (6), and a locking stick (7). The damper was tested under different loading frequencies and displacement amplitudes.



**Figure 2.16** Structure of SMA Damper: (a) Structure; (b) Relation Between SMA Wires and the Inner and External Sliding Blocks; and (c) Photo (Zuo et al. 2007)

The results highlighted that, within the range of the experimental loading frequencies, the response of the SMA damper was more sensitive to the displacement amplitudes than loading frequencies. When the loading frequency increased from 0.02 Hz to 1.0 Hz, the changes in the positive and negative stiffness were 7.4% and 6.3%, respectively. Therefore, in the case of low frequency (less than 1.0 Hz), the effects of loading frequency can be ignored. Stiffness, dissipated energy, and damping ratio were more related to the displacement amplitudes as shown in Figure 2.17.



**Figure 2.17** Relationships Between Displacement Amplitudes and Parameters of SMA Damper (Zuo et al. 2007)

Isolators are a form of dampers that aim to isolate a structure from its foundation or base to limit the transfer of ground accelerations from the earthquake to the building. Significant energy dissipation is required to have an effective isolator. Dolce et al. (2007) performed a study comparing four types of isolator systems: rubber-based, steel-based, SMA-based, and SMA-steel hybrid. Shake table tests were conducted on identical 1:4 scale reinforced concrete frames, which were provided with different isolating systems. The authors noted that no optimal system exists, and the choice of an isolation system is dependent on numerous factors. The steel-based systems offer very good performance, but exhibit minimal re-centering capabilities. SMA-based isolators, on the other hand, reduce the dynamic demand while reducing permanent deformations.

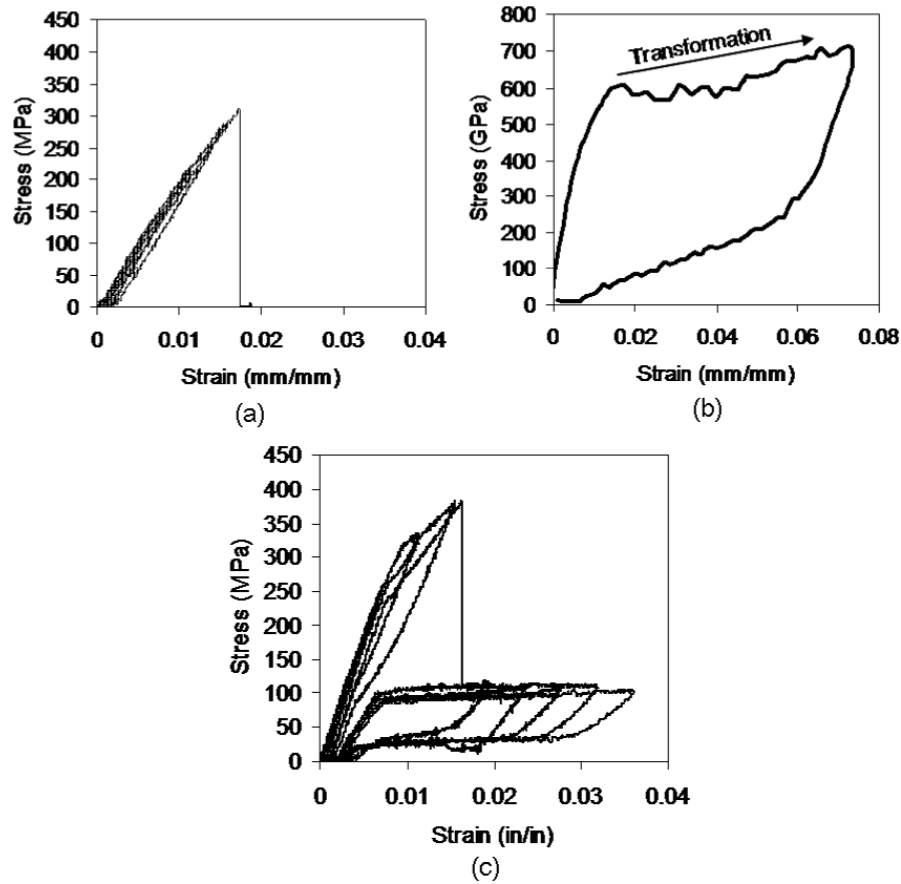
Wilde et al. (2000) proposed a variable isolation system for elevated highway bridges. Two isolation systems were numerically modeled and subjected to ground motions

simulating an earthquake: the proposed smart isolator utilizing SMAs, and a conventional laminated rubber bearing with a lead core. The analyses demonstrated that for a medium size earthquake, the SMA bars increased the damping capacity of the isolation device due to the Martensitic transformation of the alloy, while acting as a hysteretic damper and displacement-controlling device in case of the largest considered earthquake.

#### 2.4 SMA-FRP Hybrid Composite

Recently, SMAs have emerged as a retrofit material given the advantages it demonstrates relative to steel and FRP. SMAs are ductile with relatively high resistance to fire, while FRPs are brittle and vulnerable to fire. Moreover, SMAs are highly resistant to corrosion compared to steel. Therefore, using SMA along with steel or FRP can potentially enhance the performance of the structural element as well as its durability.

Wierschem (2009) conducted a number of tests on SMA–FRP composite coupons to determine their constitutive behavior regarding ductility, energy dissipation, and bond. The SMA–FRP coupons for this study were manufactured by running 0.5 mm diameter superelastic SMA wires in between layers of fiberglass cloth saturated with epoxy resin. The experimental results of cyclic tensile tests showed significant potential for SMA–FRP reinforcement to improve the ductility and damping of concrete structures while still maintaining its elastic characteristic (Figure 2.18).

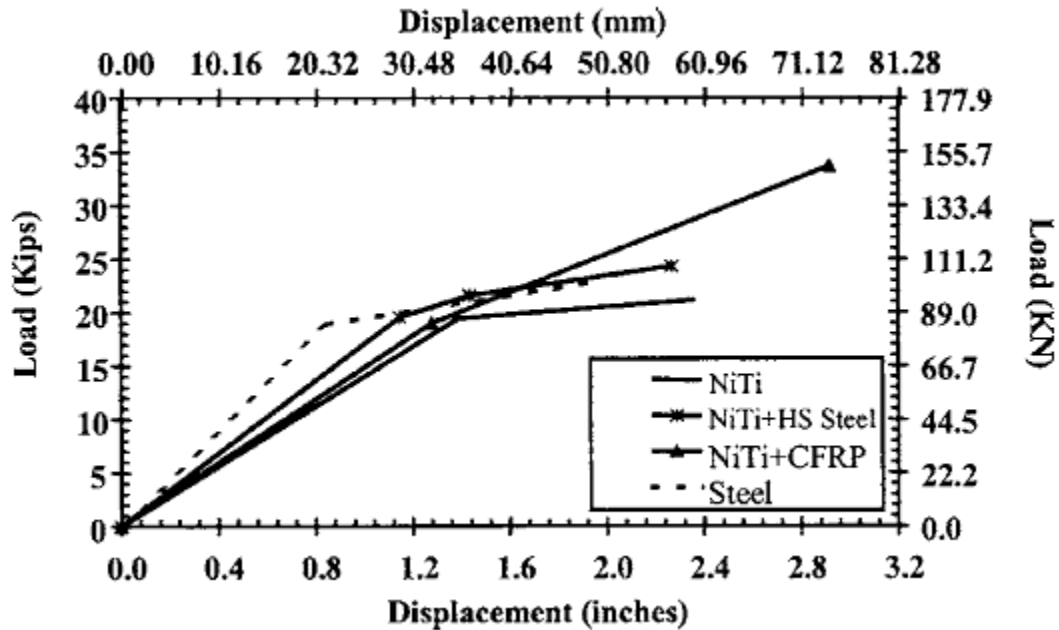


**Figure 2.18** Experimental Stress–Strain Curves: (a) GRFP Coupon; (b) SMA Wire; and (c) SMA–FRP Coupon (Wierschem 2009)

Wierschem stated that replacing 25% (by volume) of the fiberglass in a Glass-FRP (GFRP) composite with superelastic SMA wires would increase the amount of ductility and elastic energy dissipated by 64% and 160%, respectively. A simplified uniaxial stress–strain model of an SMA–FRP composite was developed. The model was used in a parametric study to investigate the effect of conventional fiber reinforcement, SMA volumetric percentage and level of SMA prestraining on the composite’s ductility and energy dissipation capabilities.

Four full-scale hybrid beams were analyzed by Saiidi et al. (2007) to study the effect of using hybrid longitudinal reinforcement on the stiffness, strength, and ductility of beams.

All the beams had the same span length (6096 mm) and cross section dimensions (305 mm x 456 mm), while the reinforcement in the plastic hinge region was the only variable. Superelastic SMA was used in the first model, a hybrid of SMA and high strength steel in the second, and a hybrid of SMA and carbon fiber-reinforced plastic in the third model. The fourth beam was conventionally reinforced and used as a benchmark. It was suggested that the design of hybrid systems using superelastic SMA with high strength steel or carbon FRP bars was the better option due to their relatively high stiffness (Figure 2.19).

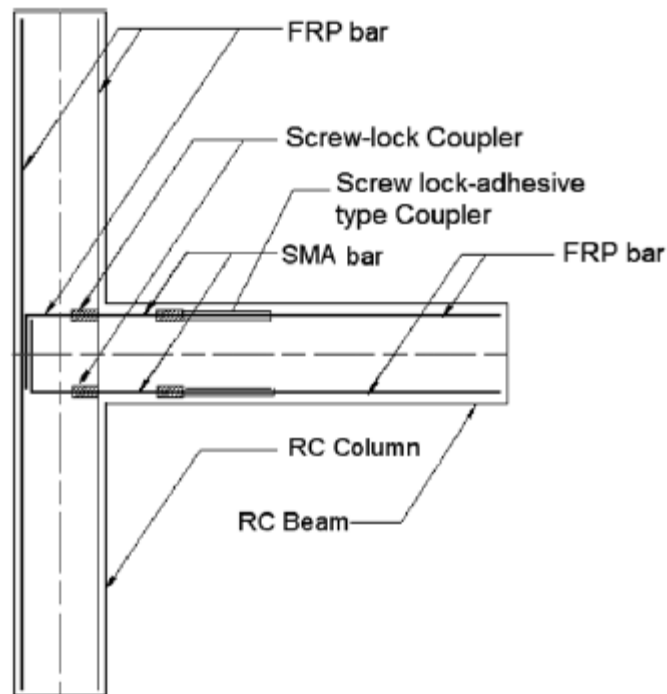


**Figure 2.19** Load–Displacement Relationship for Beams (Saiidi et al. 2007)

Nehdi et al. (2010) investigated a SMA-FRP hybrid system to develop corrosion free structural elements with the ability to recover large deformation upon unloading. A reinforced concrete beam-column joint was tested under reverse cyclic loading where superelastic SMA bars were used in the plastic hinge region and FRP in other regions.

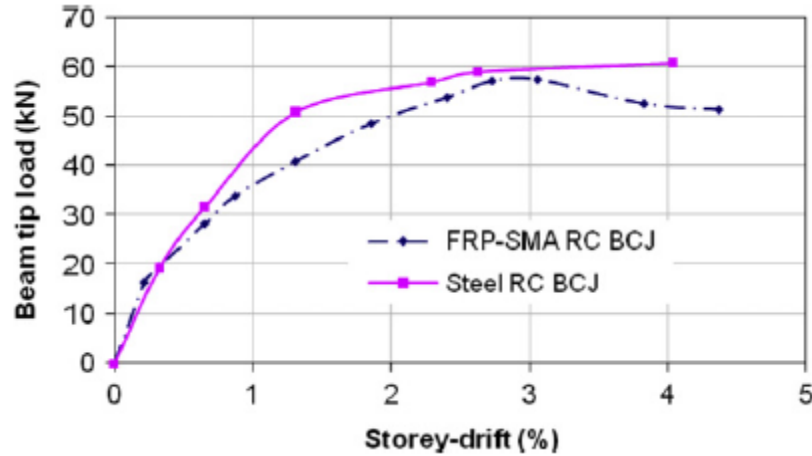


Splicing was replaced with regular single barrel screw-lock couplers to connect SMA with FRP bars as shown in Figure 2.20.

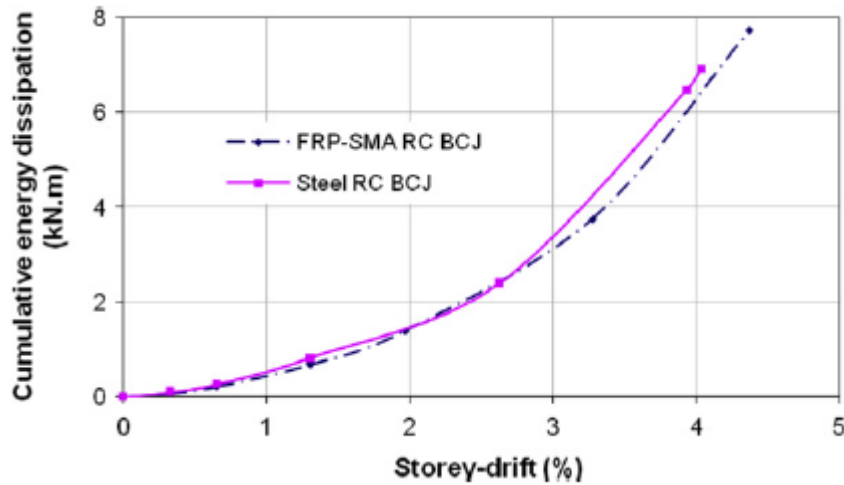


**Figure 2.20** Splice Details of the Test Specimen (Nehdi et al. 2010)

Test results were compared in terms of load-storey drift (Figure 2.21), moment-rotation and energy dissipation capacity (Figure 2.22) to those of a similar reinforced concrete beam-column joint specimen reinforced with conventional steel. Ultimate strength, maximum drift, and dissipated energy were comparable in both specimens. However, the ability of SMA in re-centering was not observed due to significant slippage of the FRP bar inside the couplers. This study highlighted the need to enhance the coupling system.



**Figure 2.21** Beam Tip Load Versus Storey Drift Envelope of Beam-Column Joint Specimens (Nehdi et al. 2010)



**Figure 2.22** Cumulative Energy Dissipation-Storey Drift Behavior of Beam-Column Joint Specimens (Nehdi et al. 2010)

## 2.5 Modeling of Shape Memory Alloys

The literature reveals significant interest on the specific micro- and macro-constitutive aspects of Shape Memory Alloys; however, lacking are complementary computational tools to support the design process of SMA devices. The development of an effective SMA constitutive model is a challenging task. This is mainly attributed to the description

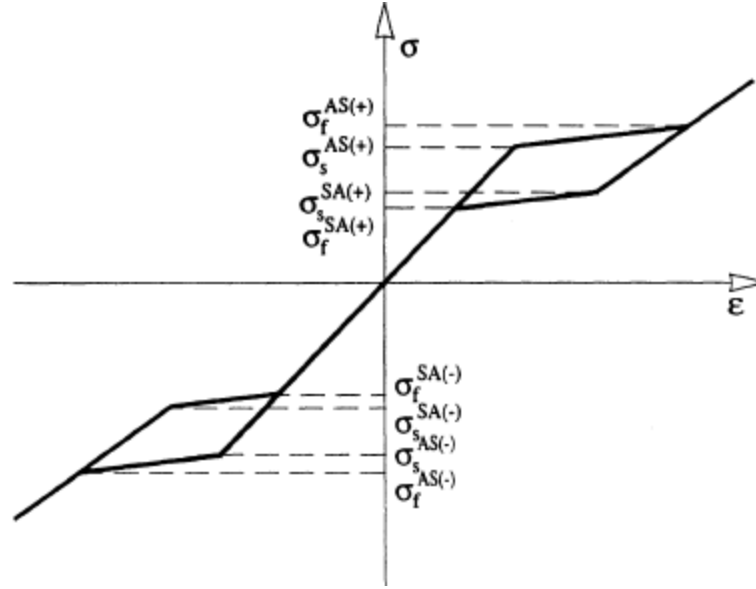
of phase transformations with no conventional theoretical framework, and the dependency of the material response on its phase. SMAs are usually modeled following either a phenomenological or a thermodynamics approach.

Thermodynamics-based models are built on the laws of thermodynamics and energy considerations. These models are much more complicated and computationally demanding than phenomenological models as they present a technique to derive precise three-dimensional constitutive laws. Phenomenological models, however, seem to be more appropriate for civil engineering applications involving SMA wires and bars since they can be easily incorporated in finite element programs.

#### 2.5.1 Phenomenological Modeling

SMA bars and wires are the most widely used shapes in civil engineering applications. Hence, one-dimensional phenomenological models are considered suitable. In phenomenological models, the parameters are defined experimentally. Several uniaxial phenomenological models have been proposed.

Aurichio and Sacco (1997) adopted the classical Euler-Bernoulli Beam theory in modeling one dimensional superelastic SMA elements. The SMA constitutive model (Figure 2.23) characterizes the material response in tension and compression as well as the different elastic properties between Austenite and Martensite.

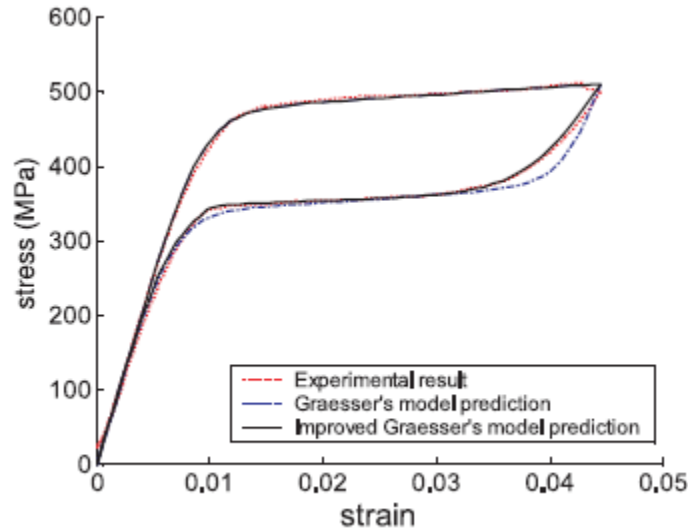


**Figure 2.23** One-Dimensional Superelastic Model of a Shape Memory Alloy (Aurichio and Sacco 1997)

The model assumes full recovery of strains and is a typical flagged-shape response. The loading, unloading, and reloading are captured by linear responses. Several numerical examples were investigated to determine the cross sectional response for a beam under pure bending. The unique behavior of SMA, where the forward and reverse transformation stresses are not the same in tension and compression, demonstrated different moment-curvature relationship from that of the conventional response.

Ren et al. (2000) proposed an improved Graesser's model (Graesser and Cozzarelli 1991) as shown in Figure 2.24 to describe the behavior of superelastic SMA wires under cyclic loading. Graesser's model has the advantages of a relatively simple formulation with consideration on the effect of the loading rate on hysteresis. However, Graesser's model uses identical parameters during both the loading and the unloading branches, which results in differences between predictions and experimental results. Ren improved this

deficiency in the model by using different parameters during the different loading branches.



**Figure 2.24** Comparison of Experimental and Predicted SMA Stress–Strain Curves (Ren et al. 2000)

The test specimens were 0.8 mm diameter NiTi SMA wires. Cyclic loading tests were implemented at different strain levels and various displacement rates. To simulate the hysteretic behavior of superelastic SMA, a full cycle was divided into three parts: the loading branch, the unloading branch before the completion of the reverse transformation, and the elastic unloading branch after the completion of the reverse transformation. Graesser's model and the improved version were utilized to simulate the hysteretic responses of the SMA wires for different peak strains and loading rates. The simulations demonstrated that the unloading behavior of superelastic SMA is better predicted by the improved Graesser's model than the original Graesser's model.

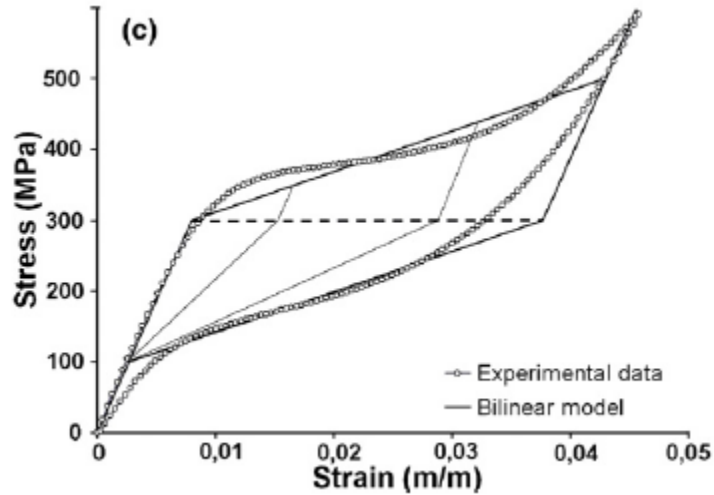
### 2.5.2 Thermodynamics-Based Modeling

Zake et al. (2003) investigated three models developed by Tanaka (1968), Liang (1990), and Brinson (1993), which described the behavior of SMAs. These models have been used widely for studying the static and dynamic performance of different composite material structures with embedded SMA components. The constitutive equations for these models assume that the total change in stress consists of three components: (1) the elastic component which is proportional to Young's modulus; (2) the transformational component proportional to the phase transformation; and (3) the thermal component. A simple experimental rig was designed and manufactured to study in detail the SMA mechanical properties. The three models predicted well the superelastic behavior at higher temperatures, above the Austenite finish temperature where SMAs remain in the fully Austenitic phase. However, at low temperatures, when the alloys are fully Martensitic, Brinson's model predicted the SMA behavior more accurately than the other models.

Ikeda et al. (2003) derived a constitutive model for SMAs under unidirectional loading. In this model, the strain consists of elastic, transformation and thermal components that are similar to the models studied by Zake et al. (2003). The partial hysteresis loops of a stress-strain-temperature relationship were modeled. Numerical results showed that the proposed model captured the observed experimental results accurately.

Torra et al. (2007) conducted an experimental program using SMA dampers along with modeling the thermo-mechanical behavior of SMAs. The SMA constitutive models (Figure 2.25) were developed and implemented in the finite element program ANSYS. The model is one-dimensional (1-D) bilinear with consideration of traction and

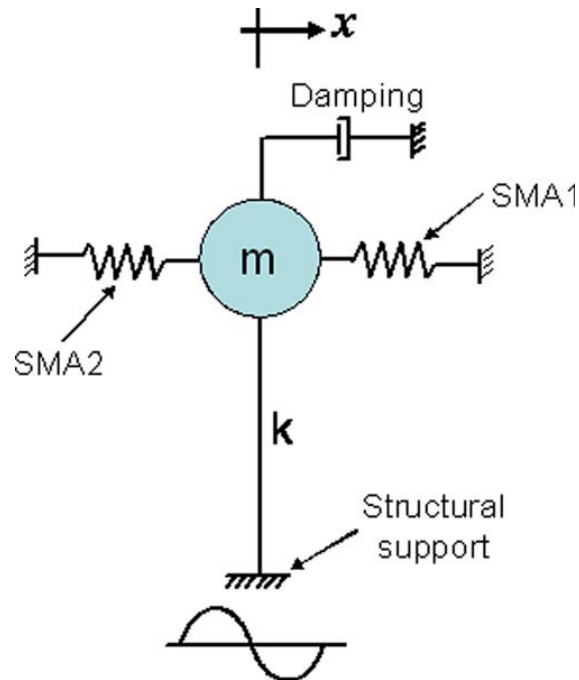
temperature effects. The experimental program included a complete damping solution for a residential house. The design and optimization methodology for the dampers and performance were studied.



**Figure 2.25** Thin Wire Bilinear Hysteresis Cyclic Models (Torra et al. 2007)

The simulations demonstrated that the SMA dampers were capable of reducing the maximum oscillation amplitude induced by the El Centro earthquake accelerations by a factor of 2, and 50% of the energy transmitted to the structure was dissipated. Therefore, the study showed that SMA dampers constitute a passive, unsupervised damping solution for residential homes.

Andrawes and DesRoches (2008) studied the effect of different SMA models on the performance of a single-degree-of-freedom system under seismic loading (Figure 2.26).

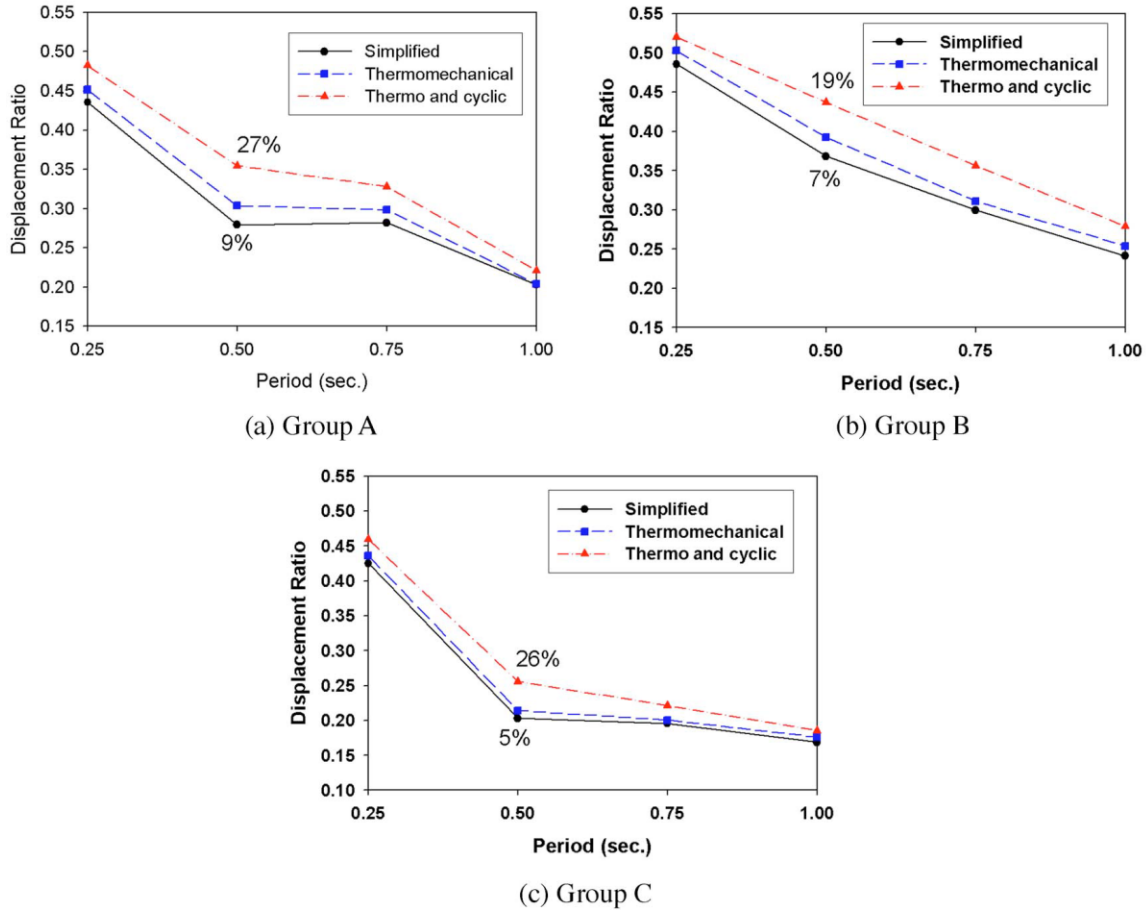


**Figure 2.26** SDOF Model to Study the Effect of SMA Models (Andrawes and DesRoches 2008)

Cables consisting of SMA wires were used to represent the SMA links. Three models were suggested: a simplified multi-linear model; whereas, the other two were thermo-mechanical models that were capable of capturing the effect of incomplete phase transformation cycles in SMAs. Moreover, one of these two models was also capable of capturing the strength degradation and residual deformation associated with cyclic loading.

Analysis results highlighted that the structural response using the simplified model was in good agreement with the more complex thermodynamic models where the difference in the peak displacement did not exceed 8% for the three ground motions studied (Figure 2.27).





**Figure 2.27** Normalized Response of SDOF Models Using Different SMA Models When Subjected to Earthquake Suites A, B, and C (Andrawes and DesRoches 2008)

When the cyclic loading effect was considered, the results indicated that the structural response regarding strength degradation and residual displacement is more sensitive to the cyclic loading effects of the SMAs than the natural period of the structure.

## 2.6 Summary

The literature has focused on the deformation recovery ability of SMAs based on transforming the Martensitic phase into the superelastic Austenitic phase. This occurs by increasing the temperature of the SMA through a heat application. Some researchers explored the possibility and efficiency of using SMAs as restrainers or connections.

Limited experimental work, however, has explored the use of larger diameter SMA bars in reinforced concrete structural elements.

The performance of superelastic SMAs as reinforcement in plastic hinge regions of reinforced concrete elements requires further investigation. Moreover, additional work is required to develop comprehensive material constitutive models capable of capturing the hysteretic response of SMAs, including the plastic deformation that accumulates upon unloading from large strains. Therefore, the research presented herein addresses the lack of information by investigating the performance of large-scale beams reinforced with large diameter SMA bars and shear walls reinforced with both SMA and conventional steel bars. The test specimens of this research program were loaded to failure under different loading conditions in the beams and under reversed cyclic loading in the shear walls. The test data was used to develop a plastic offset constitutive model for SMAs which was verified against the test specimens and data available in the literature.

## CHAPTER 3

### **EXPERIMENTAL PROGRAM**

#### 3.1 Introduction

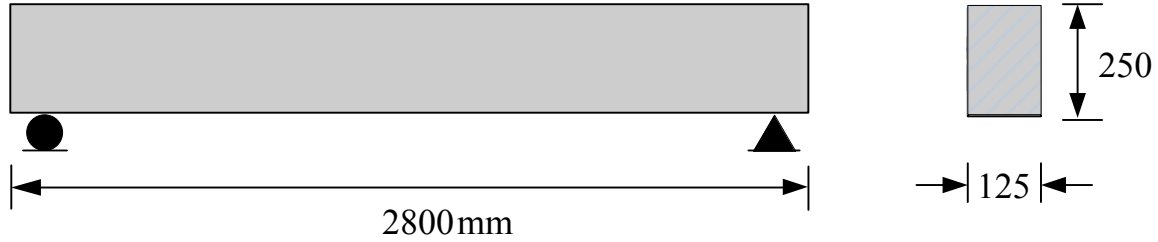
Superelastic Nitinol (NiTi) reinforcing bars present new challenges and opportunities for structural engineers. Their behavior with concrete is not entirely known and requires further research before being adopted in new construction or retrofit of existing structures. To understand the behavior of superelastic NiTi, tests involving reinforced concrete structural elements were conducted to observe the basic mechanical properties between NiTi reinforcing bars and concrete. The first phase of the experimental program involved testing flexure-dominant RC beams, while the second phase of the experimental program involved testing ductile RC shear walls. This chapter describes the experimental program conducted on concrete beam and shear wall specimens reinforced with SMA, including tests conducted to obtain the material properties.

#### 3.2 Test Specimens

##### 3.2.1 Beams

Seven simply supported reinforced concrete beams were constructed (Figure 3.1). The beams were 2800 mm long (2400 mm from centre to centre of supports), 125 mm wide and 250 mm deep. The test variables included loading method and the type of flexural reinforcement in the critical section (Table 3.1). Although the beams are simply supported and not necessarily typical of structural components in a building, the focus is the critical region at midspan, which is representative of a maximum moment at any

location in a structural member. Furthermore, these beams, with well-defined loading and support conditions, are used to corroborate the proposed hysteretic constitutive model.



**Figure 3.1** Dimensions of Typical Test Beams

**Table 3.1** Beam Test Variables

| Specimen      | Loading Method | Reinforcement Type | Longitudinal Reinforcement |                        |
|---------------|----------------|--------------------|----------------------------|------------------------|
|               |                |                    | Bottom                     | Top                    |
| <b>B1-SM</b>  | Monotonic      | Steel              | 2 bars, 10M                | 2 bars, $\phi=6.35$ mm |
| <b>B2-SC</b>  | Cyclic         | Steel              | 2 bars, 10M                | 2 bars, $\phi=6.35$ mm |
| <b>B3-SR</b>  | Reverse Cyclic | Steel              | 2 bars, 10M                | 2 bars, 10M            |
| <b>B4-NM</b>  | Monotonic      | NiTi               | 2 bars, $\phi=9.5$ mm      | 2 bars, $\phi=6.35$ mm |
| <b>B5-NC</b>  | Cyclic         | NiTi               | 2 bars, $\phi=9.5$ mm      | 2 bars, $\phi=6.35$ mm |
| <b>B6-NR</b>  | Reverse Cyclic | NiTi               | 2 bars, $\phi=9.5$ mm      | 2 bars, $\phi=9.5$ mm  |
| <b>B7-NCM</b> | Mono-Cyclic    | NiTi               | 2 bars, $\phi=9.5$ mm      | 2 bars, $\phi=9.5$ mm  |

The specimens were named B1-SM, B2-SC, B3-SR, B4-NM, B5-NC, B6-NR, and B7 NCM. The letter B denotes beam, and the letter S or N denotes steel (conventional deformed reinforcing steel) or Nitinol (SMA smooth reinforcing bar) used as longitudinal reinforcement in the critical section of the beam, respectively. The letters M, C, and R denote the loading method imposed during testing: monotonic, cyclic, and reverse cyclic, respectively. Beams B1-SM, B2-SC and B3-SR were the control specimens, while Beams B4-NM, B5-NC and B6-NR were the companion Nitinol reinforced specimens. An additional beam, B7-NCM, was constructed and tested to address some deficiencies experienced during testing of the SMA beams. Beam B7-NCM was first tested under

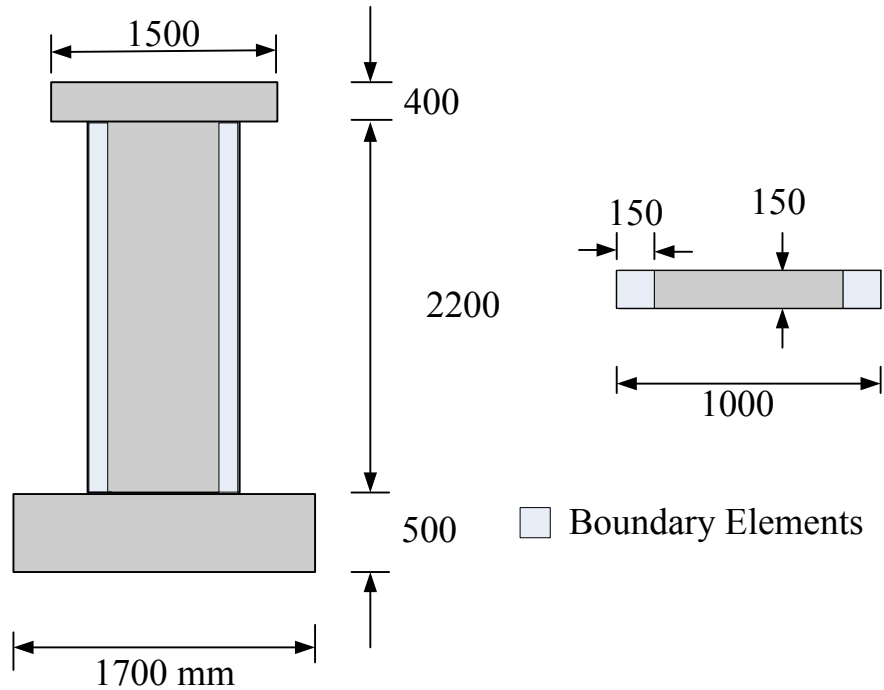
cyclic loading followed by monotonic loading in the opposite direction. The cyclic loading test was repeated due to problems encountered during testing of B5-NC. These problems were attributed to the coupling connection between the SMA bar and the 15M bar extending outside the critical region, which affected the strength, unloading response, and residual midspan displacements. The monotonic loading was repeated to better assess the response of an SMA reinforced beam controlled by failure in the critical section.

### 3.2.2 Shear Walls

Two ductile shear walls were tested under reverse cyclic lateral loading. The height to width ratio of walls was 2.2. Boundary elements were provided at the two ends of the shear walls. The specimens were named W1-SR and W2-NR. The letter W denotes shear wall, the letter S or N denotes steel or Nitinol, and the letter R denotes the loading method imposed during testing. Details of these walls are given in Table 3.2. The shear walls had a rectangular cross section with 150 mm thickness and 1000 mm length. The height of the walls was 2200 mm. The aspect ratio of 2.2 was selected to promote ductile flexural response. The walls were reinforced with two layers of orthogonal steel reinforcement in the web. Superelastic Nitinol bars replaced the longitudinal conventional reinforcement in the boundaries in Wall W2-NR, while steel reinforcement was used in Wall W1-SR as the primary flexural reinforcement. Each specimen was constructed with a heavily reinforced top beam, which distributed the applied lateral load uniformly along the top of the wall, and a heavily reinforced base foundation. The geometry and dimensions of the shear wall specimens are shown in Figure 3.2.

**Table 3.2** Shear Wall Test Variables

| Specimen     | Loading Method | Reinforcement Type | Vertical Reinforcement               |                        |
|--------------|----------------|--------------------|--------------------------------------|------------------------|
|              |                |                    | Web                                  | Boundaries             |
| <b>W1-SR</b> | Reverse Cyclic | Steel              | 10M @150 <sup>c</sup> / <sub>c</sub> | 4 bars, 10M            |
| <b>W2-NR</b> | Reverse Cyclic | NiTi               | 10M @150 <sup>c</sup> / <sub>c</sub> | 4 bars, $\phi=12.7$ mm |



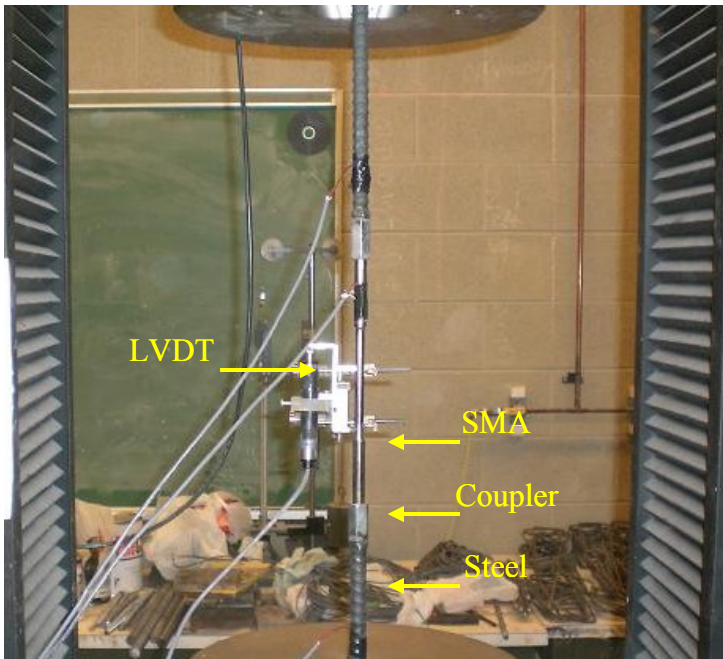
**Figure 3.2** General Dimensions of Shear Walls

### 3.3 Design of Specimens

#### 3.3.1 Design of Beams and Coupling System

The beams were simply supported and subjected to two central point loads. Specimens B1-SM, B2-SC, B4-NM, and B5-NC were designed as singly reinforced beams. Deformed reinforcing bars (10M) of 11.3 mm nominal diameter and 100 mm<sup>2</sup> cross-sectional area were used as longitudinal reinforcement in specimens B1-SM and B2-SC, while reshaped 12.7 mm diameter Nitinol rods were used in the plastic hinge region of B4-NM and B5-NC.

The Nitinol bars selected for this study were comparable in size to the deformed bars used in the control specimens. The two longitudinal bars placed at the bottom of the beams spanned the entire length in the conventional reinforced group, while the SMA bars were used over a length of 600 mm centered at the midspan of the beams to optimize the material given its high initial cost. The SMA bars were threaded at the ends and connected to 15M deformed reinforcing steel with threaded mechanical couplers. Carbonate bits were used to thread the ends of the SMA bars. The 15M bars, which have a nominal diameter of 16 mm and cross sectional area of 200 mm<sup>2</sup>, were used outside the critical section to prevent yielding of reinforcement outside this region. The standard threaded mechanical couplers were hexagon-shaped with an inner diameter of 12.6 mm, outer dimension of 22 mm, and approximate length of 50 mm, and provided a butt splice between the SMA and deformed bars (Figure 3.3).



(a)



(b)

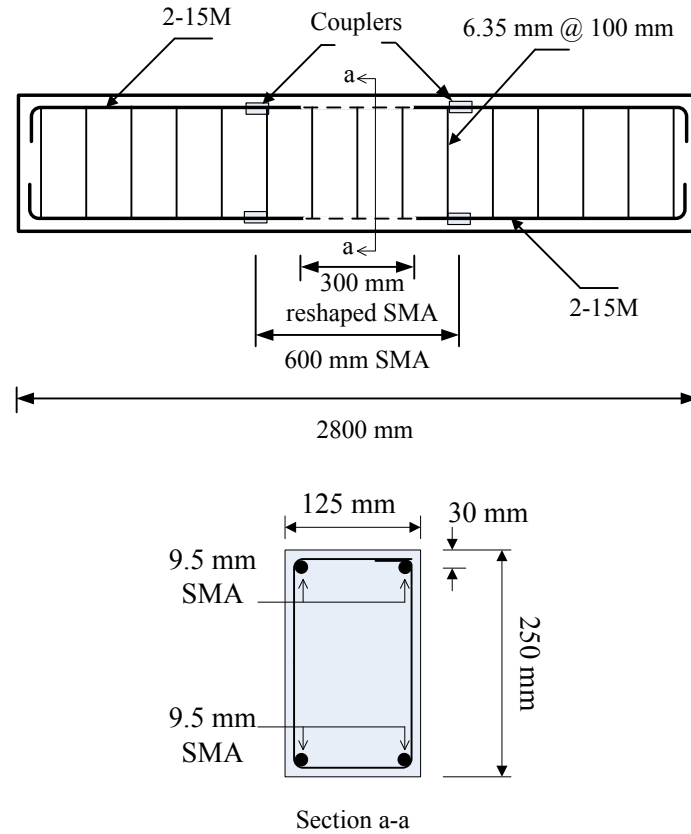


(c)

**Figure 3.3** Mechanical Couplers Used in Beams: (a) Test Setup; (b) Threaded Coupler; and (c) Threading and Reshaping of SMA Bar

In addition, the Nitinol bars were reshaped to a diameter of 9.5 mm, as shown in Figure 3.4, over a 300 mm length to promote yielding (forward transformation) and failure near the midspan and away from the threaded sections located at the ends of the bars. Two additional bars were placed at the top of the beams that were subjected to reverse cyclic loading (B3-SR and B6-NR) and in Beam B7-NCM. For B3-SR, this consisted of 10M bars for the full length of the beam; while for B6-NR and B7-NCM, Nitinol coupled with 15M deformed bars were used. For the singly-reinforced beams (B1-SM, B2-SC, B4-NM, and B5-NC), two smooth 6.35 mm diameter detailing bars were placed at the top of the beams for the sole purpose of supporting the transverse shear reinforcement. The shear reinforcement, consisting of 6.35 mm diameter closed smooth stirrups, was spaced at 100 mm along the entire length of the beams. A concrete clear cover of 20 mm was provided throughout. Design material details of the specimens are provided in Table 3.3, while design calculations for the SMA beams are presented in Appendix A.





**Figure 3.4** Details of SMA Bars and Couplers in Beam B6-NR

**Table 3.3** Design Material Properties of Test Beams

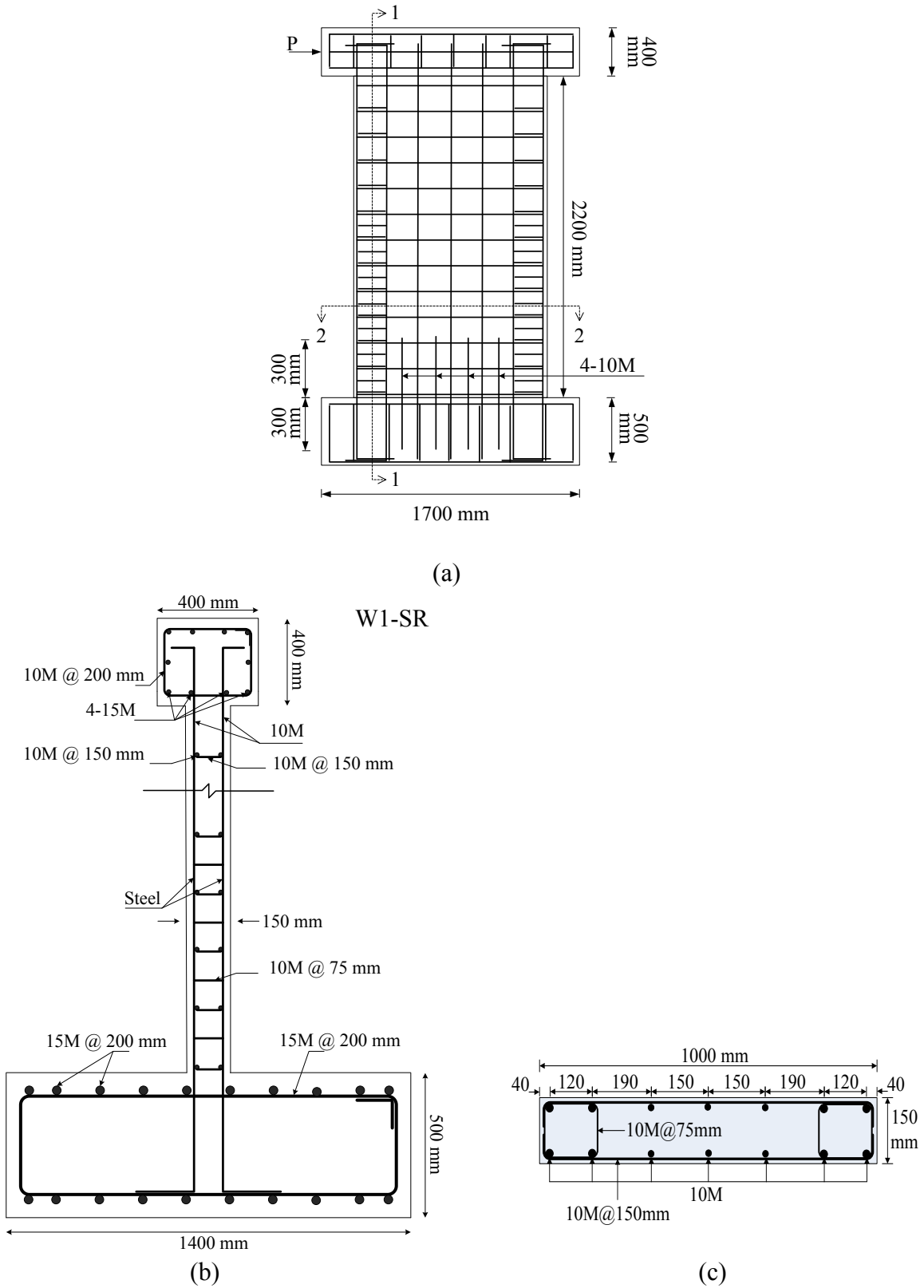
| Specimens     | Concrete        | Longitudinal Reinforcement |                |               |                | Transverse Reinforcement         |                |
|---------------|-----------------|----------------------------|----------------|---------------|----------------|----------------------------------|----------------|
|               | $f'_c$<br>(MPa) | Bottom                     |                | Top           |                | Stirrups                         | $f_y$<br>(MPa) |
|               | (Design)        | $\rho$<br>(%)              | $f_y$<br>(MPa) | $\rho$<br>(%) | $f_y$<br>(MPa) |                                  |                |
| <b>B1-SM</b>  | 30              | 0.71                       | 425            | 0.22          | 570            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B2-SC</b>  | 30              | 0.71                       | 425            | 0.22          | 570            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B3-SR</b>  | 30              | 0.71                       | 425            | 0.71          | 425            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B4-NM</b>  | 30              | 0.50                       | 415            | 0.22          | 570            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B5-NC</b>  | 30              | 0.50                       | 415            | 0.22          | 570            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B6-NR</b>  | 30              | 0.50                       | 415            | 0.50          | 415            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |
| <b>B7-NCM</b> | 30              | 0.50                       | 415            | 0.50          | 415            | $\phi=6.35\text{mm}@100^\circ_c$ | 570            |

### 3.3.2 Design of Shear Walls

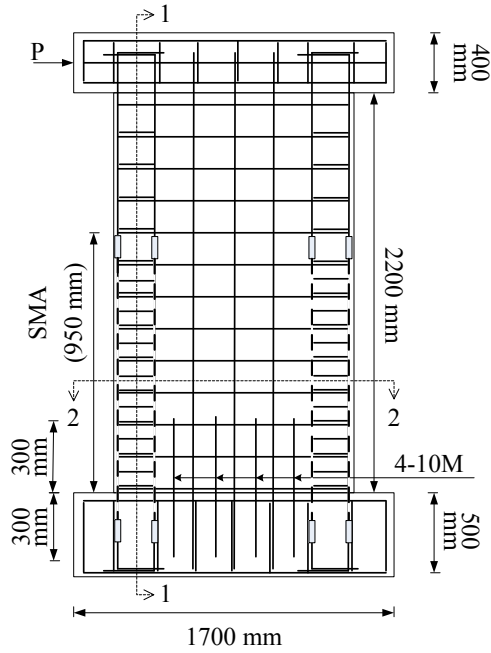
The shear wall specimens were designed to promote flexure behavior with sufficient shear strength to ensure that the probable moment capacity was attained. The length, height, and thickness of the shear walls were 1000 mm, 2200 mm, and 150 mm, respectively. Each wall specimen was built with a heavily reinforced top beam and bottom foundation. The end regions of the shear walls were provided with boundary elements. The top beam (400 mm x 400 mm x 1700 mm) was used to distribute the applied horizontal load uniformly across the top of the wall. The bottom foundation was designed to permit full fixity to the strong floor.

The walls were reinforced with two layers of orthogonal steel reinforcement in the web. The vertical steel reinforcement consisted of three pairs of 10M deformed bars spaced uniformly at 150 mm, whereas the horizontal steel reinforcement consisted of fifteen pairs of 10M bars spaced uniformly at 150 mm. As a result, each shear wall specimen had the same web vertical and horizontal steel reinforcement ratio of 0.88%. Walls of W1-SR (Figure 3.5) and W2-NR (Figure 3.6) were similar in terms of geometry and amount and arrangement of transverse reinforcement; however, they had different longitudinal reinforcement in the plastic hinge region at the boundaries: W2-NR utilized superelastic SMA; while W1-SR was reinforced with conventional steel. In specimen W2-NR, four SMA bars of 12.7 mm diameter replaced the corresponding 10M conventional steel in the boundary elements in W1-SR. Mechanical couplers were used in W2-NR to connect the SMA bars in the plastic hinge region with 15M deformed steel bars extending outside the plastic hinge region. The total length of the SMA bars was 1200 mm. The length from the top of the foundation into the wall was 950 mm, while the remaining 250 mm was

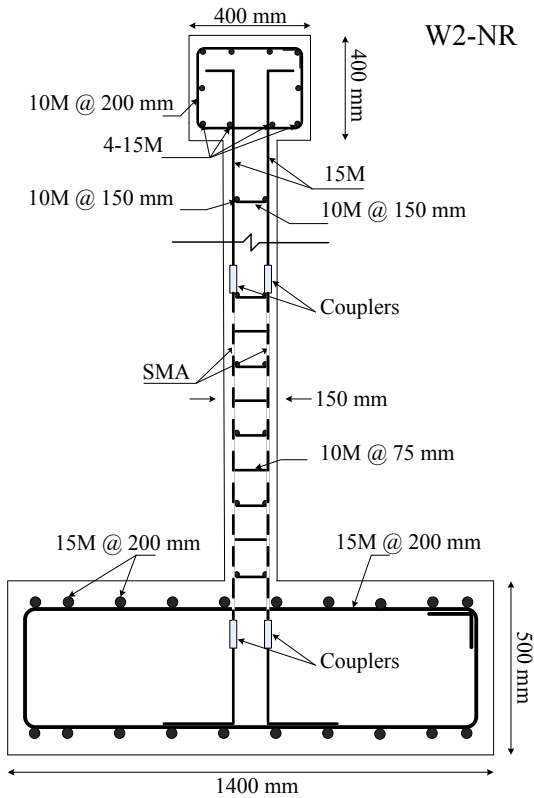
extended into the foundation. The length of the SMA bars into the wall was based on the result of finite element analysis (VecTor2), which predicted the plastic hinge length to be approximately 760 mm. To prevent buckling of the vertical principal reinforcement in the boundary zones of both walls, 10M closed steel ties were provided along the full height of the wall. The ties were spaced at 75 mm inside the plastic hinge region and 150 mm above the plastic hinge region. Furthermore, 4 pairs of 10M starter bars were placed in the web section of the wall to increase the shear sliding resistance.



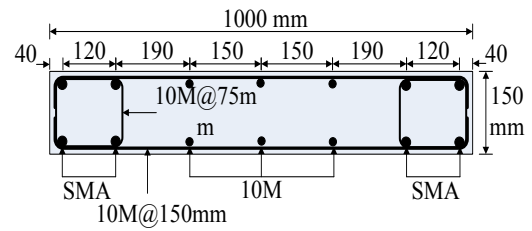
**Figure 3.5** Reinforcement Details for Shear Wall W1-SR: (a) Front View; (b) Section 1-1; and (c) Section 2-2



(a)



(b)

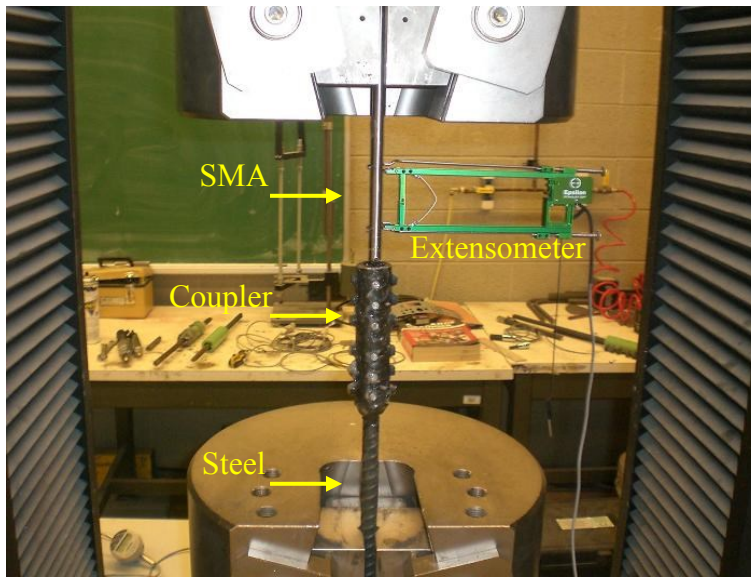


(c)

**Figure 3.6** Reinforcement Details for Shear Wall W2-NR: (a) Front View; (b) Section 1-1; and (c) Section 2-2

### 3.3.2.1 Reinforcement and Coupling System

Regular, single-barrel type screw lock couplers were used to connect the SMA bars and the deformed steel reinforcement in shear wall W2-NR to eliminate the reshaping of the SMA bars as required with threaded couplers. . The screw lock coupler is compatible with reinforcing bars and complies with ASTM A615 (2009). It consists of smooth-shaped steel sleeves with shear bolts and grip rails to mechanically splice the rebar. In these types of couplers, each end of the reinforcing bars is inserted into one end of the coupler. A pin is located in the middle of the coupler to separate the inserted bars. The bolts hold the bars in position by tightening them until their heads shear-off at a prescribed torque. Figure 3.7 illustrates the couplers used in Wall W2-NR.



(a)



(b)



(c)

**Figure 3.7** Mechanical Couplers Used in Shear Wall W2-NR: (a) Test Setup; (b) Screw Lock Coupler; and (c) Modified Coupler Used in Test Wall

The coupling system with the SMA bar inserted in one end and a 15M deformed steel bar inserted in the other end of the coupler was tested in a universal testing machine. To

minimize the relative slippage between the bars and the coupler, nine-10 mm diameter sharp end screws distributed in three rows were used to connect the SMA, while the other nine screws of the same type were used to connect the deformed steel. [Note that this type of coupler is typically manufactured with six sharp end screws.] Tests showed that modified arrangement resulted in slip of the SMA bar before reaching its yield capacity. Therefore, the entire length of the coupler was used to connect the SMA bar, while the 15M steel bar was welded to the end of the mechanical coupler. This arrangement allowed the SMA bar to be stressed to its yield capacity and attain its full superelastic strain range (10% strain). Design material details of the specimens are provided in Table 3.4, while design calculations for the SMA wall are presented in Appendix B.

**Table 3.4** Design Material Properties of Shear Walls

| Specimens    | Concrete                    | Vertical Reinforcement |                |               |                | Transverse Reinforcement |                |
|--------------|-----------------------------|------------------------|----------------|---------------|----------------|--------------------------|----------------|
|              | $f'_c$<br>(MPa)<br>(Design) | Web                    |                | Boundary      |                | $\rho$<br>(%)            | $f_y$<br>(MPa) |
|              |                             | $\rho$<br>(%)          | $f_y$<br>(MPa) | $\rho$<br>(%) | $f_y$<br>(MPa) |                          |                |
| <b>W1-SR</b> | 30                          | 0.88                   | 425            | 1.33          | 425            | 0.88                     | 425            |
| <b>W2-NR</b> | 30                          | 0.88                   | 425            | 1.68          | 380(NiTi)      | 0.88                     | 425            |

### 3.4 Material Properties

#### 3.4.1 Nitinol Rods

Superelastic Nitinol was used as the main flexural reinforcement in the test specimens. The dimensions and the shape of available NiTi products were investigated including Nitinol wires and bars (rods). The latter shape was the most appropriate for the specimens used in this study. The Nitinol rods used to reinforce the beam and shear wall specimens were cold drawn and thermally straightened to the superelastic condition. The specified

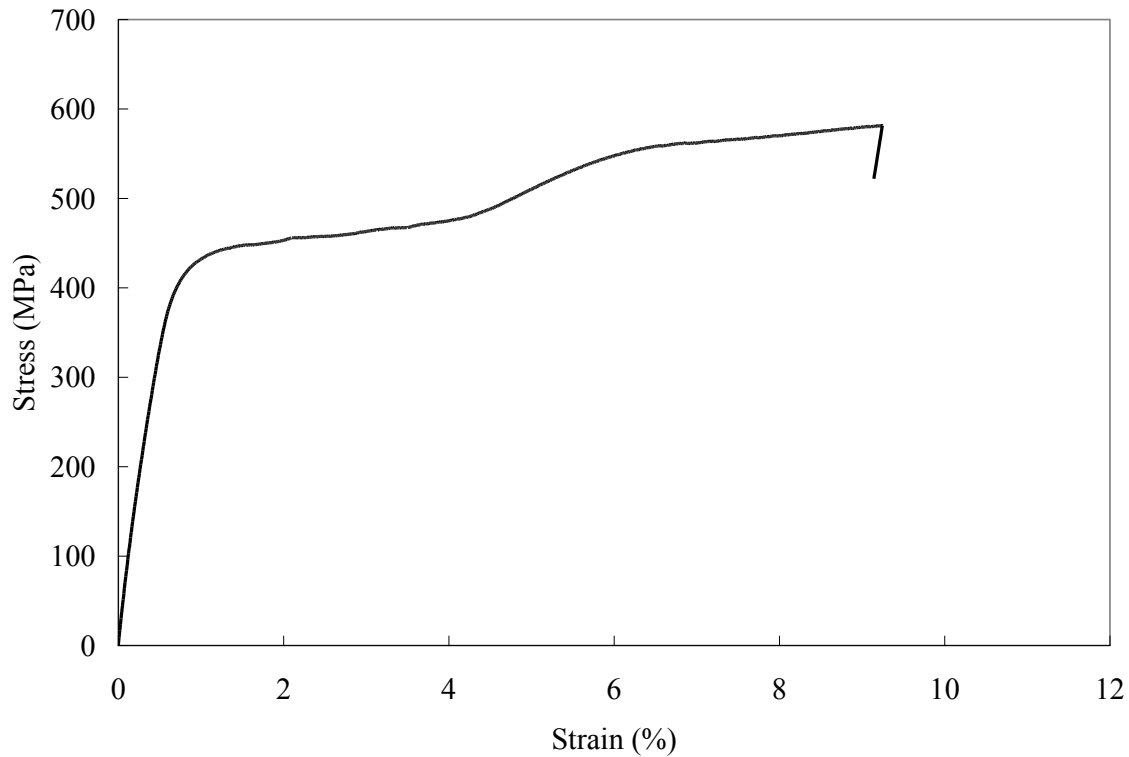
Austenite start,  $A_s$ , and finish,  $A_f$ , temperatures were  $-30$  and  $-10$  degrees Celsius, respectively. The NiTi rods were manufactured and supplied by SAES Smart Materials. Table 3.5 provides the mechanical and the chemical analysis of the material as provided by the manufacturer.

**Table 3.5** Properties of NiTi Provided by SAES

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

The results of monotonic and cyclic tensile coupon testing of the SMA bars with a comparison to the steel reinforcing bars is presented and discussed in Chapter 4. Figure 3.8 provides a typical monotonic response of a SMA bar used in the beam test specimens.





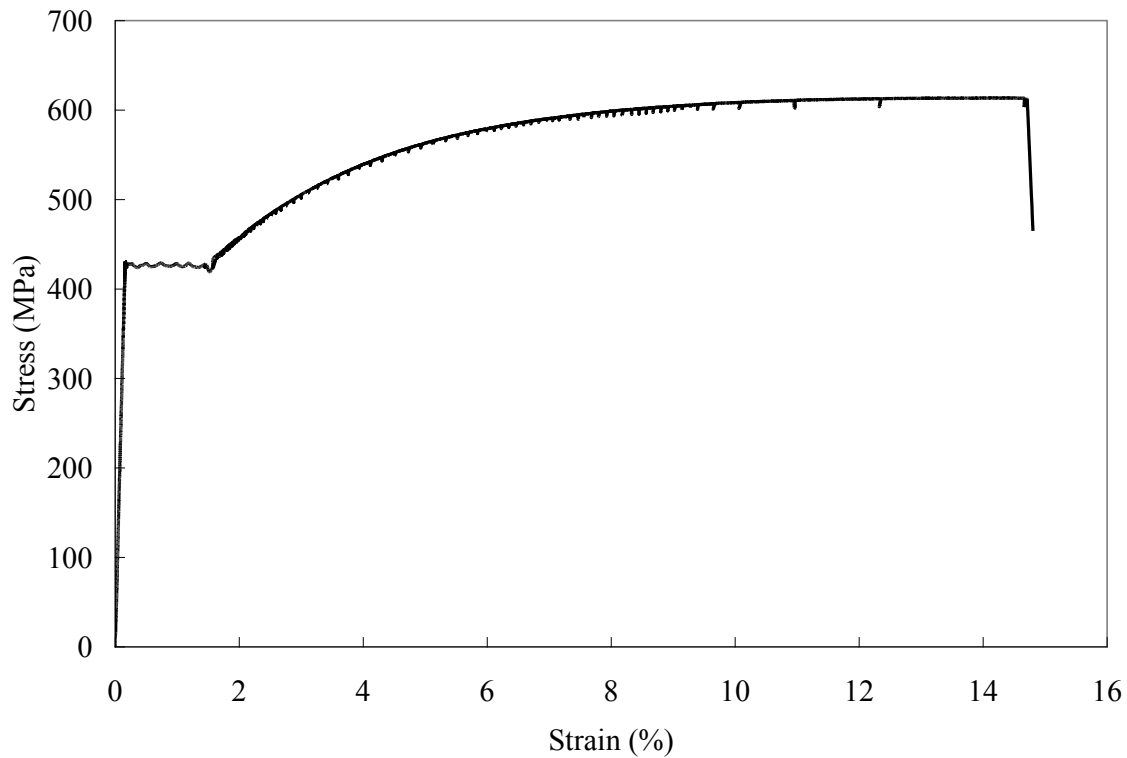
**Figure 3.8** Typical Strain-Stress Response of SMA Reinforcement in Beams

### 3.4.2 Steel Reinforcement

Different sizes of deformed and smooth reinforcing bars were used in the experimental program. Their designations, nominal diameters, and locations of usage are listed in Tables 3.1 and 3.2. Standard tensile coupon tests were performed using a 600 kN MTS testing machine. The strain in each test was measured using an extensometer attached to the bar. The average results of three coupon tests are listed in Table 3.6, and a typical 10M reinforcement stress-strain response is provided in Figure 3.9. Test results for other diameters (15M, 20M, and 6.35 mm diameter bar) are included in Appendix C.

**Table 3.6** Steel Properties

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

**Figure 3.9** Typical Strain-Stress Response of 10M Reinforcement

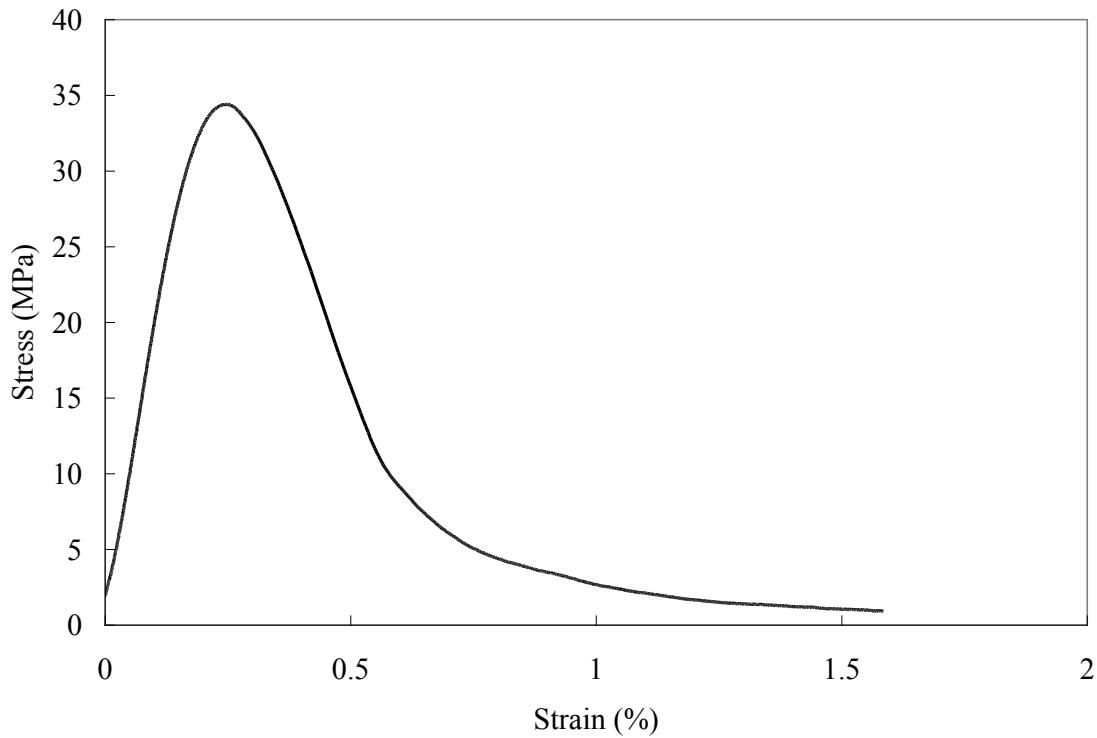
### 3.4.3 Concrete

The 28-day concrete cylinder compressive strength for the beams and the shear walls was specified to be 30 MPa with a maximum aggregate size of 10 mm and a slump of 100 mm. The conventional and SMA beams, and the conventional and SMA shear walls were cast on separate days. Twelve standard cylinders (100 mm x 200 mm) for each group

were prepared at the same time the concrete was poured in the beams and shear walls. Nine of the cylinders were distributed into three groups. The cylinders were subjected to lab curing conditions, which was the same curing condition as the specimens. Each group was used for 7-day, 14-day and 28-day compressive tests, while the three remaining cylinders were tested on the day of testing of the beams and walls. All cylinders were tested in accordance with ASTM C39 (1994). The loading was applied to correspond to a stress rate of 0.25 MPa/s on the cylinder. Table 3.7 summarizes the concrete test results for the beams and the shear walls, while a typical concrete stress-strain response for conventional beams is shown in Figure 3.10. Strain-stress responses of concrete used in the SMA beams and the shear walls are included in Appendix C.

**Table 3.7** Cylinder Compressive Strength of Beams and Shear Walls

| Designation               | Average Strength (MPa) |         |         |             | Strain at Peak Stress (%)<br>(Day of Test) | Tangent Modulus of Elasticity (MPa)<br>(Day of Test) |
|---------------------------|------------------------|---------|---------|-------------|--------------------------------------------|------------------------------------------------------|
|                           | 7 days                 | 14 days | 28 days | Day of Test |                                            |                                                      |
| <b>Conventional Beams</b> | 24.8                   | 29.4    | 32.3    | 34.6        | 0.25                                       | 28400                                                |
| <b>SMA Beams</b>          | 22.8                   | 27.1    | 31.1    | 32.7        | 0.26                                       | 26840                                                |
| <b>Conventional Wall</b>  | 20.7                   | 24.3    | 30.1    | 30.5        | 0.27                                       | 25100                                                |
| <b>SMA Wall</b>           | 21.1                   | 24.8    | 30.8    | 31.6        | 0.26                                       | 25980                                                |



**Figure 3.10** Typical Stress-Strain Response of Concrete in Conventional Beams

The average measured cylinder compressive strengths were 34.6 MPa and 32.7 MPa for the conventional and SMA beams, respectively. The corresponding strength for the conventional and SMA reinforced shear walls were 30.5 and 31.6 MPa, respectively. These compressive strengths were used for all calculations. The beams were cast in one stage, while the shear walls were constructed in two stages. First, the foundation formwork was built followed by placing the reinforcement for both the foundation and the wall. Concrete was then cast up to the foundation-wall interface and allowed to cure. Second, the wall and top beam formwork was constructed and the steel cage was placed in the top beam. Concrete was then cast for both the wall and top beam at the same time. This construction method resulted in only one cold joint, which was at the foundation-wall interface.

### 3.5 Specimen Construction

#### 3.5.1 Beams

The test beams were constructed in two phases, using similar procedures. In first phase, the conventional beams B1-SM, B2-SC, B3-SR were built. The construction commenced with the assembly of the steel forms previously fabricated in the workshop of the University of Ottawa. This formwork was constructed from 6 mm steel plates. Lateral support for the formwork was provided by steel rods bolted to the formwork and steel channel sections running along the entire length of the formwork. Strain gauges were attached to the longitudinal and transverse steel reinforcement at designated locations along the beam to capture the required strains during the test. A thin layer of silicone coating was applied throughout the gauge length and on the soldered connections to protect them against impact, moisture and damage during casting. After the strain gauges were applied to the individual reinforcing bars, the reinforcement cages were constructed outside of the formwork. The inside of the forms were brushed with oil to facilitate their removal after the casting and curing of the concrete. Plastic seats, positioned on the base of the formwork and tied to the stirrups, were used to maintain the cover around the beams. After the inside of the formwork was oiled, the reinforcement cages were placed inside. The end plates were then attached to the formwork and threaded steel rods were used to clamp the formwork across the width along the entire length to maintain a uniform beam width during the cast. Figure 3.11 shows the constructed reinforcement cages for the conventional beams and the formwork.



**Figure 3.11** Reinforcement Cages and Formwork for Beams

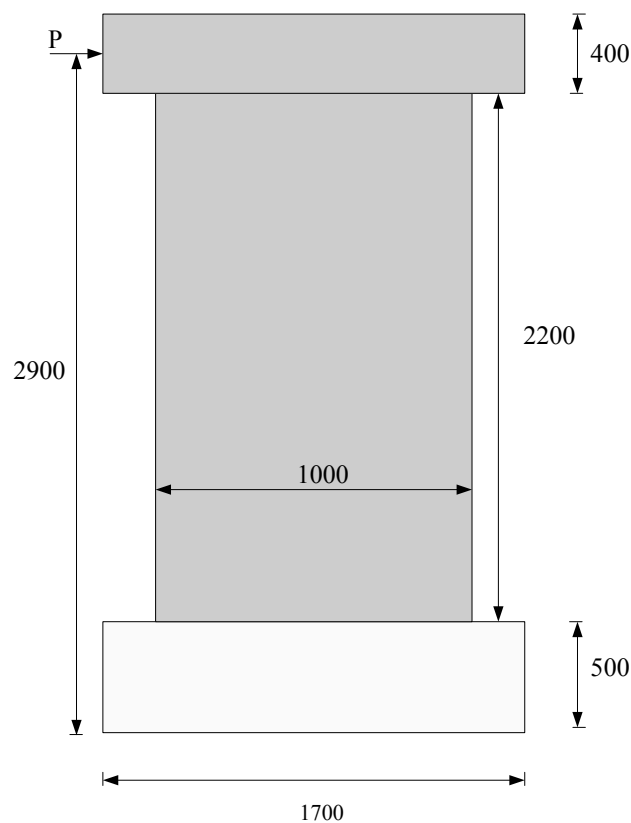
The concrete was prepared using the concrete mixer housed in the structures laboratory in accordance with the design specifications. The maximum aggregate size was 10 mm and the design slump was 100 mm. For each beam, the concrete was prepared in one mix, and vibrated using an electric vibrator and steel tamping rod. After finishing the concrete to a smooth level surface, the specimens were covered with plastic sheets. The concrete was cured at ambient temperature (15 - 30°C) and the surface was kept wet for at least 10 days. The forms were removed after three days and the curing was continued under ambient conditions.

In the second phase, SMA beams B4-NM, B4-NC, B6-NR, and B7-NCM were constructed using the same formwork used for the conventional beams. The formwork was first inspected for any defects from the previous usage. Prior to the assembly of the cages, strain gauges were attached to the steel and SMA reinforcement and were coated with a thin layer of silicone. Plastic seats were used to position the reinforcement cages within the formwork. The remaining steps in the assembly of the cages and casting of concrete were similar to the first phase.

Once the beam specimens were removed from the formwork, the surface of the concrete was white washed in order to trace the crack patterns during the tests.

### 3.5.2 Shear Walls

Construction of the test specimens was divided into two stages (Figure 3.12): (1) formwork construction, preparation and placement of the reinforcement, and concrete casting of the base foundation; and (2) building the wall and top beam.



All Dimensions are in mm

□ Stage 1    ■ Stage 2

**Figure 3.12** Construction Stages for Shear Walls

Formwork construction and the initial assembly involved cutting plywood sheets to the appropriate dimensions and assembling the formwork. Preparation and placement of

reinforcement involved cutting and bending longitudinal and transverse reinforcement to the specified dimension, installing strain gauges, and arranging the reinforcement.

The formwork for the specimens was constructed using 20 mm thick plywood sheets and 38 mm x 89 mm structural grade lumber. Three light coats of form oil were applied to the formwork prior to the placement of the concrete. The rectangular shaped footings were reinforced with 25M and 20M deformed bars as the main reinforcement in longitudinal and transverse directions, respectively. Four-80 mm diameter PVC pipes were placed in each reinforcing cage prior to casting and acted as hollow selves. Prior to testing, four-70 mm diameter high strength bolts were inserted into the selves to fasten the test specimens to the laboratory strong floor. Details of the foundation reinforcement and formwork are shown in Figure 3.13.



**Figure 3.13** Shear Wall Construction (Foundation)

After the reinforcing steel work was completed, the foundation was cast with a specified concrete mix supplied by a ready mix concrete plant. Once placing and vibrating of the concrete in the formwork was completed, the construction joint at the top of the



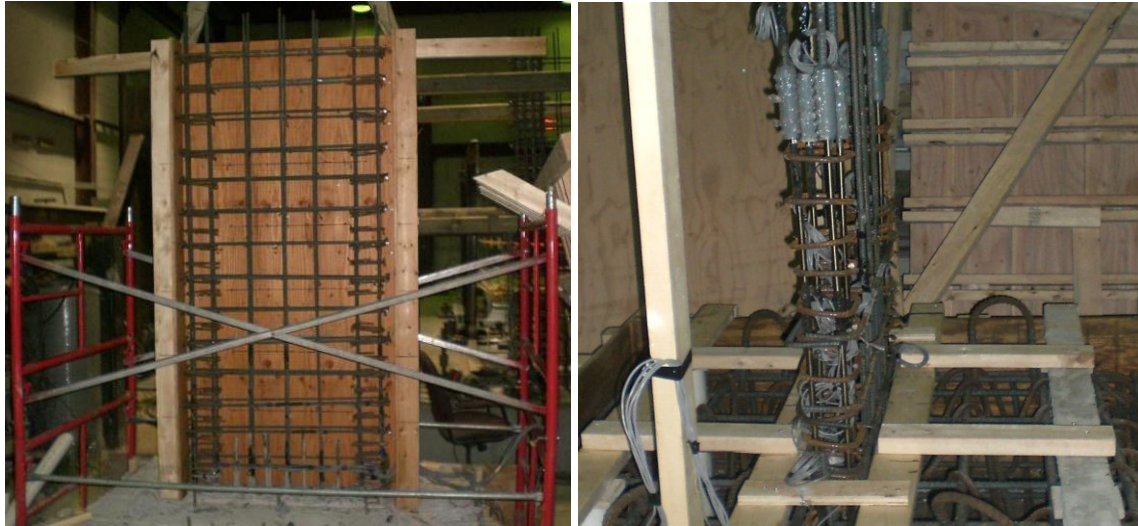
foundation was roughened. The specimen was then covered with polyethylene sheets and kept wet for one week.

The second stage in the construction of the shear wall specimens consisted of building and simultaneously casting the wall panel and top beam. The same construction process was repeated in this stage, with additional attention to support the sides of the formwork to provide stability during casting (Figure 3.14).



**Figure 3.14** Shear Wall Construction (Web Wall and Top Beam)

The reinforcement layout of W1-SR and W2-NR were similar as shown in Figure 3.15 with the exception of using SMA bars in place of deformed steel reinforcement in the boundaries of W2-NR.



**Figure 3.15** Shear Wall Construction (Wall Reinforcement)

10M deformed bars were used in both the longitudinal and transverse directions; while 10M deformed bars, in the form of closed stirrups, were used as anti-buckling bars for the principal longitudinal reinforcement in the boundary elements. To avoid any potential sliding in the wall at the construction joint, four-10M bars of 600 mm length were distributed in each layer, with 300 mm of the length extending into the wall above the foundation and the remaining embedded into the foundation. The extended length of the starter bars was less than the development length to minimize any contribution to the flexural capacity while improving the sliding shear capacity.

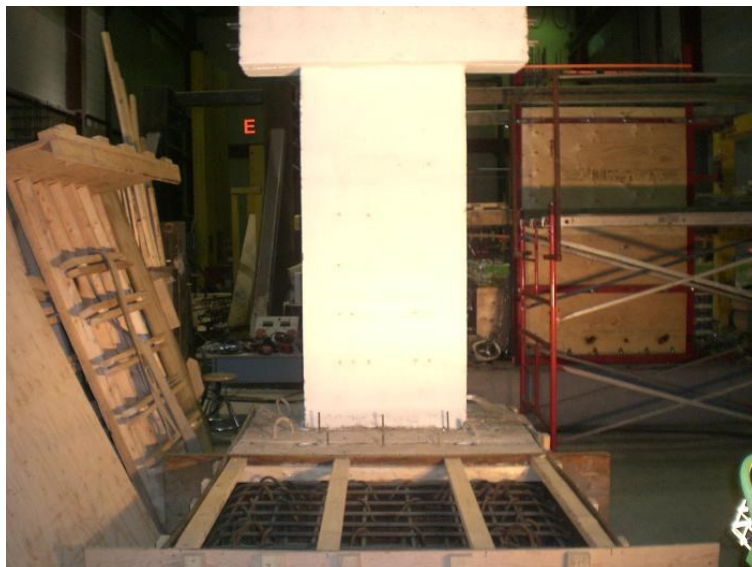
The spacing between the stirrups was 75 mm over the first 900 mm of the wall, thereafter the spacing was increased to 150 mm towards the top of the wall. All regular longitudinal and horizontal deformed steel bars had 90 degree hooks at both ends. The rectangular-shaped top beam was fabricated with 20M deformed bars for the main reinforcement and 10M deformed bars for stirrups. To prevent the formwork from bulging during the placement of the concrete, 6.3 mm diameter steel rods were inserted through the forms

and tightened at the ends. The formwork was removed after one week (Figure 3.16), and during the curing process, the wall was covered and sprayed with water.



**Figure 3.16** Shear Wall Construction (Removal of Formwork)

The wall was then washed, cleaned, and painted white for testing as shown in (Figure 3.17).



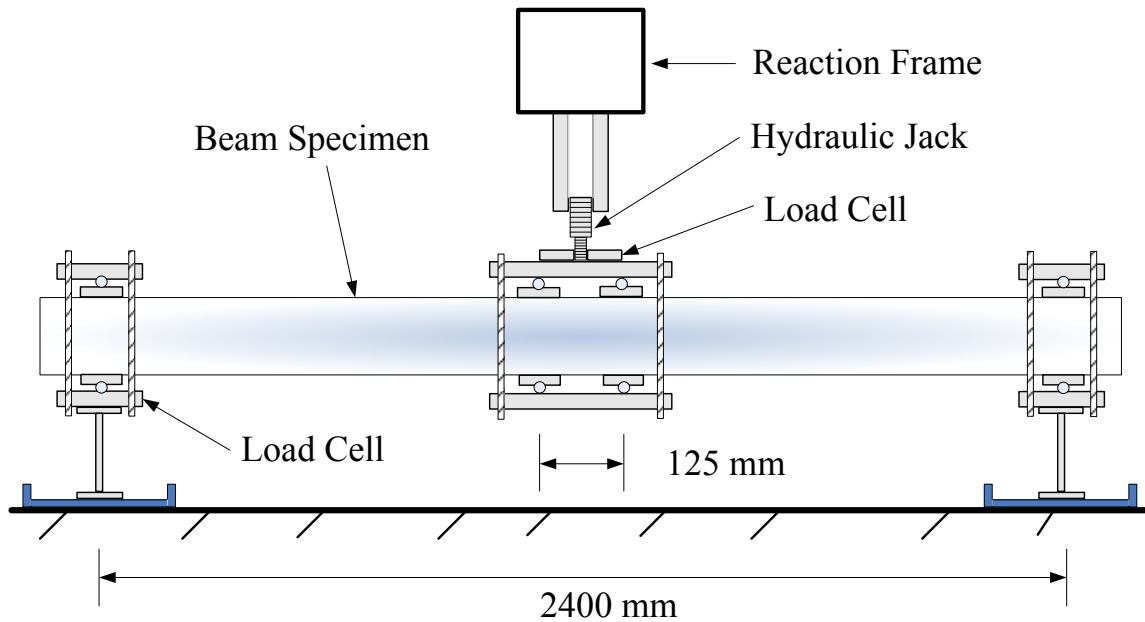
**Figure 3.17** Shear Wall Construction (Wall Painting)

The concrete supplied was in accordance with the design specifications. The maximum aggregate size was 20 mm and 10 mm for the foundation and the wall, respectively, while the slump was 100 mm for both components. Superplasticizer was used in the concrete to obtain good workability without increasing the water/cement ratio.

### 3.6 Test Setup

#### 3.6.1 Beams

The test setup involved building the testing rig, preparing the concrete surface, and assembling the instrumentation and testing apparatus. The testing rig, shown in Figure 3.18, was specifically built for beams and with the capability of applying different loading protocols (monotonic, cyclic, and reverse cyclic). Once the testing rig was in position, its base was fixed to the strong floor using eight anchor bolts.



**Figure 3.18** Test Setup

All beams were tested under two central loads, spaced 125 mm apart, to ensure the middle part of the beams was subjected to constant flexure. Steel plates, 75 mm long, 150 mm wide, and 25 mm thick were positioned under each load to distribute the imposed forces and to prevent local crushing of the concrete. The loading was applied by a hydraulic jack, mounted to a rigid steel assembly, in increments of 5 kN until failure for the monotonic tests, whereas displacement control was implemented for the cyclic tests. In all tests, the applied load was measured by three load cells: one mounted under the hydraulic jack, and the other two at the supports. PCB Piezotronics load cells with a maximum capacity of 225 kN in compression were used. Two digital protractor devices were attached to the beam at the supports to measure the rotation of the supports at different load stages during testing (Figure 3.19).



**Figure 3.19** Support Details



Deflection readings were continuously recorded at midspan of the beams using two Displacement Cable Transducers (DCTs) as shown in Figure 3.20.

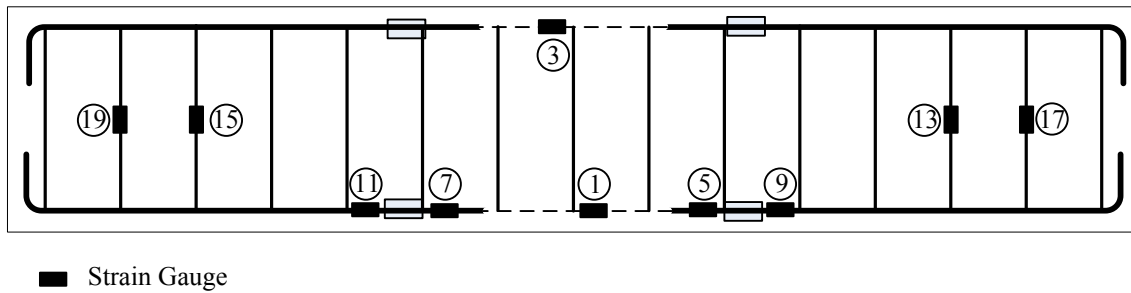


**Figure 3.20** Loading and Displacement Setup

The DCTs were installed under the beam on both sides of the loading plate. The base of each DCT was fixed to the strong floor while the top was in contact with the bottom surface of the beam.

Strain measurements in the longitudinal reinforcing steel were captured by general purpose (FLA-10) electrical resistance strain gauges with 10 mm length and 2.5 mm width. These strain gauges record strains in the range of 0.01 micro strains, with a maximum capacity of 5% strain. In the SMA beams, post-yield (YFLA-10) strain gauges were used. These strain gauges have the same gauge length and sensitivity of those that were used in the conventional beams, but have a maximum strain capacity of 15%. For all beams, strain gauges were attached to the longitudinal reinforcement in the maximum moment region with two strain gauges on the bottom reinforcement. Two additional

gauges were placed on the top reinforcement for B3-SR and B6-NR at mid-span. Additional strain gauges were attached on both sides of the couplers that were used in the SMA beams. Figure 3.21 shows the typical locations of the strain gauges on the longitudinal and transverse reinforcement of the SMA beam B6-NR. The odd numbers used in Figure 3.21 refers to strain gauges on the front side of the beam, while back-up strain gauges on the opposite side were assigned even numbers.



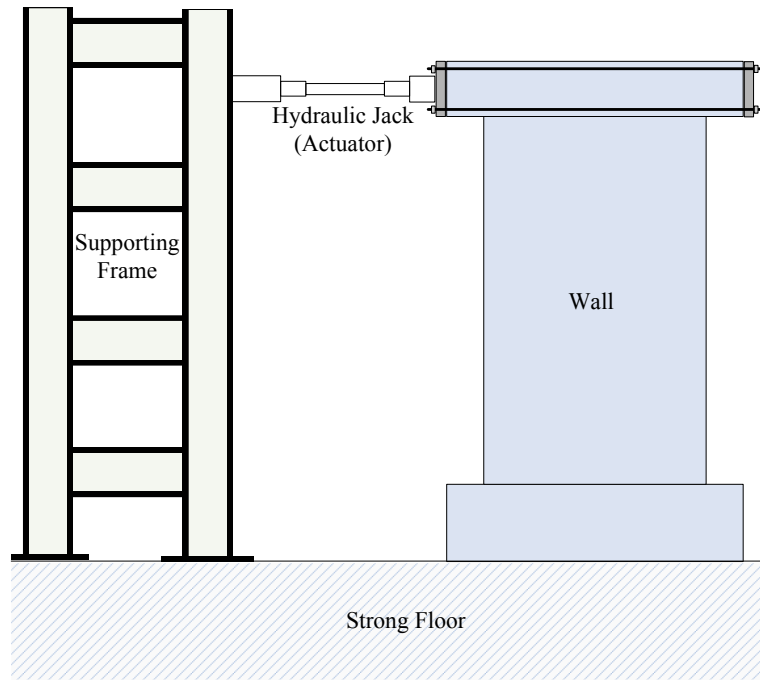
**Figure 3.21** Strain Gauge Locations in Beam B6-NR

Strain gauges on the transverse reinforcement were used to monitor the deformation resulting from shear stresses. Four strain gauges were attached to the steel stirrups: two located at a distance of 150 mm and two others at 250 mm from the supports. All strain gauges were attached to the bars with compatible adhesive and wired as a full Wheatstone bridge.

All load cells, displacement cable transducers, and strain gauges were connected to a data acquisition system. A portable computer attached to the data acquisition system was used to record readings at a constant time interval with one reading per second. The cracking patterns, crack widths, and crack spacing were recorded at each load stage.

### 3.6.2 Shear Walls

Forces, displacements and reinforcement strains of each shear wall were monitored and recorded during testing. The testing assembly consisted of a lateral loading system in which the horizontal loading was applied in displacement-controlled mode using a displacement controlled actuator positioned at the center of the top beam (Figure 3.22).



**Figure 3.22** Shear Wall Test Setup

The hydraulic actuator was mounted against a reaction frame which consisted of two I-section steel columns braced by I-section steel struts. The reaction frame was fixed to the strong floor of the laboratory with high strength bolts. The hydraulic actuator has a maximum load capacity of 1000 kN in tension and 1450 kN in compression with a maximum stroke capacity of 500 mm. At each end of the hydraulic actuator, swivel hinges allow rotation in both the vertical and horizontal planes. To eliminate out-of-plane displacements, which were observed during testing W1-SR, a lateral supporting frame (Figure 3.23) was designed for W2-NR to guide the top beam during testing. The lateral



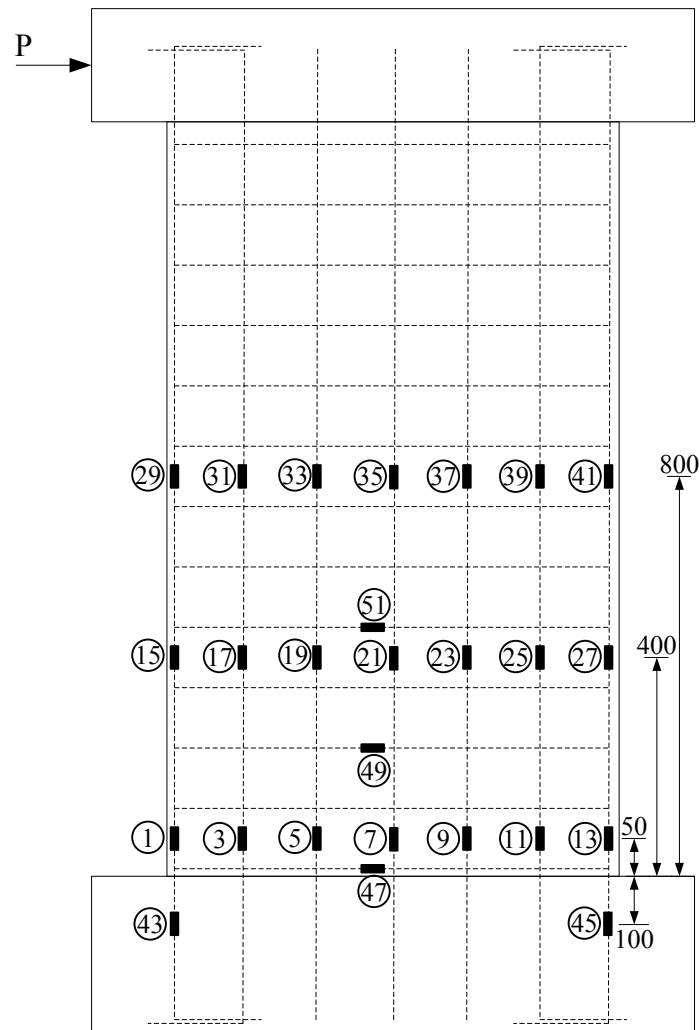
supporting frame was constructed from two-20 mm x 20 mm steel box section guides connected to a stiff steel truss through two adjustable bolts, which were able to control the gap between the side faces of the top beam and the steel guide. The supporting steel truss surrounded the top loading beam and was connected to a supporting frame that was fixed to the laboratory strong floor with four high strength bolts. The gap between the steel guides and the beam was filled with grease to minimize friction.



**Figure 3.23** Lateral Supporting Frame in Wall N2-NR

Each test specimen was instrumented for strain and displacement measurements. A data acquisition system periodically scanned and recorded the signals from the measurement sensors. The strains in the vertical and horizontal reinforcement were measured by electrical resistance strain gauges. Two types of strain gauges were used in the shear walls: general purpose (F-series) strain gauges for the deformed reinforcing steel, and post-yield (YF-series) strain gauges for the SMA bars. The strain gauges were bonded at

different locations on the vertical and horizontal reinforcement in the wall and base foundation. The layout and numbering of strain gauges are provided in Figure 3.24.



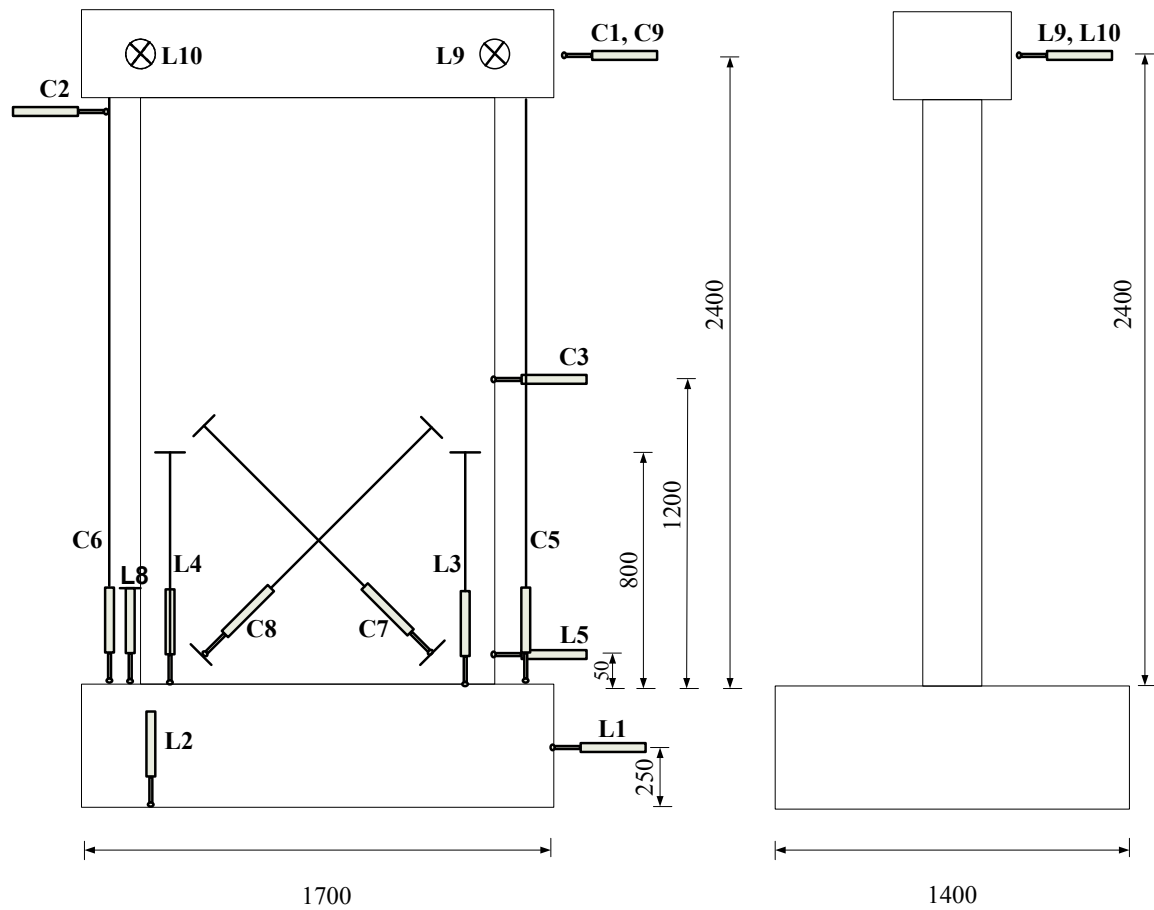
All Dimensions are in mm

**Figure 3.24** Location of Strain Gauges in Shear Walls

In the longitudinal reinforcement, the first strain gauges were located 50 mm from the base of the wall, while the other gauges were placed at 400 mm and 800 mm from the base to record the straining in the plastic hinge region. Three strain gauges on each face were placed on the transverse bars at different distances from the base foundation where

major shear cracking was expected. Four additional strain gauges were placed on the longitudinal bars 100 mm into the foundation to trace the penetration of yielding.

Linear Variable Differential Transducers (LVDTs) and Displacement Cable Transducers (DCTs) were placed at various locations on the wall to monitor lateral, vertical and out-of-plane displacements. The locations of the LVDTs and DCTs are shown in Figure 3.25, where the same layout was used for both specimens, with the exception of LVDTs 9 and 10 which were only used in W1-SR and DCT9 which was added to W2-NR.



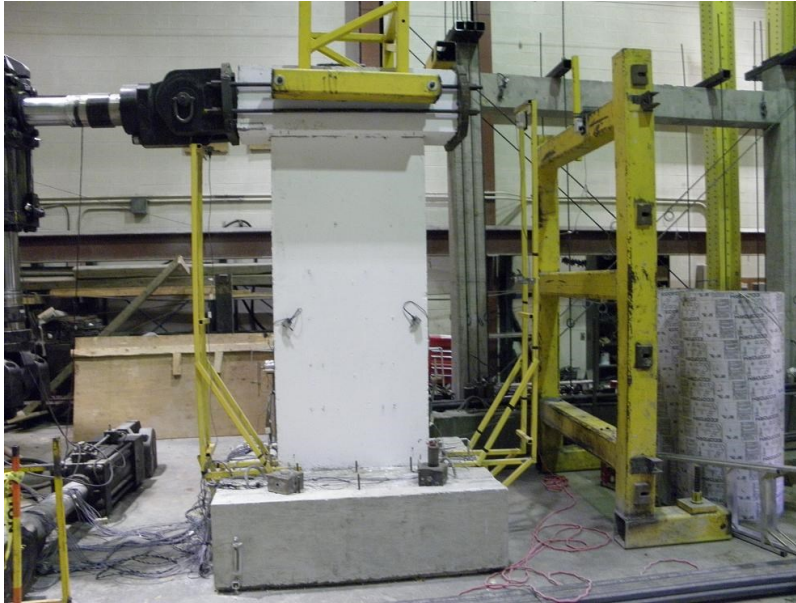
All Dimensions are in mm

**L = LVDT**

**C = DCT**

**Figure 3.25** Location of LVDT's and DCTs for Displacement Measurements

The lateral displacement of the wall was measured by four Displacement Cable Transducers (C1, C2, C3, and C9) supported by two light steel braced frames (Figure 3.26).



**Figure 3.26** Instrumentation Setup

These frames consisted of structural steel box sections and were mechanically fastened to the base foundation of the test specimens. Consequently, the wall displacements were measured relative to the foundation, thus the measurements were independent of any displacement of the foundation. Two DCTs (C1 and C2) were directly connected to the walls to measure the lateral displacement at the top of the wall from both ends. The lateral displacement of the wall on the left side (actuator end) was measured at a distance of 250 mm below the loading point to avoid interference with the loading system. DCT C3 was placed at a height of 1200 mm from the base of the wall to measure the lateral displacements at mid-height of the wall close to the plastic hinge region. To account for the effect of foundation uplift during the tests, an additional DCT (C9) was placed on a

stiff external steel frame (Figure 3.27) to measure the top lateral displacement relative to the lab strong floor.



**Figure 3.27** Location of DCTs for Top Beam Lateral Displacement

Two additional DCTs (C5 and C6) were placed at the base near the boundaries of the wall and connected to the top beam to measure the vertical displacement of wall relative to the base foundation. In addition to the vertical and horizontal displacement measurements, the specimens were instrumented to record the diagonal deformations in the plastic hinge region using a pair of DCTs (C7 and C8) attached at  $45^\circ$  near the base of the wall (Figure 3.25).

Six Linear Variable Differential Transducers (LVDTs) were directly connected to the wall specimens (Figure 3.25). The first two LVDTs (L1 and L2) were used to measure sliding and uplift of the foundation, respectively, while two other LVDTs (L3 and L4) were placed at a height of 800 mm at each end of the wall to measure the vertical displacement in the plastic hinge region. The in-plane slip at the base of the wall was

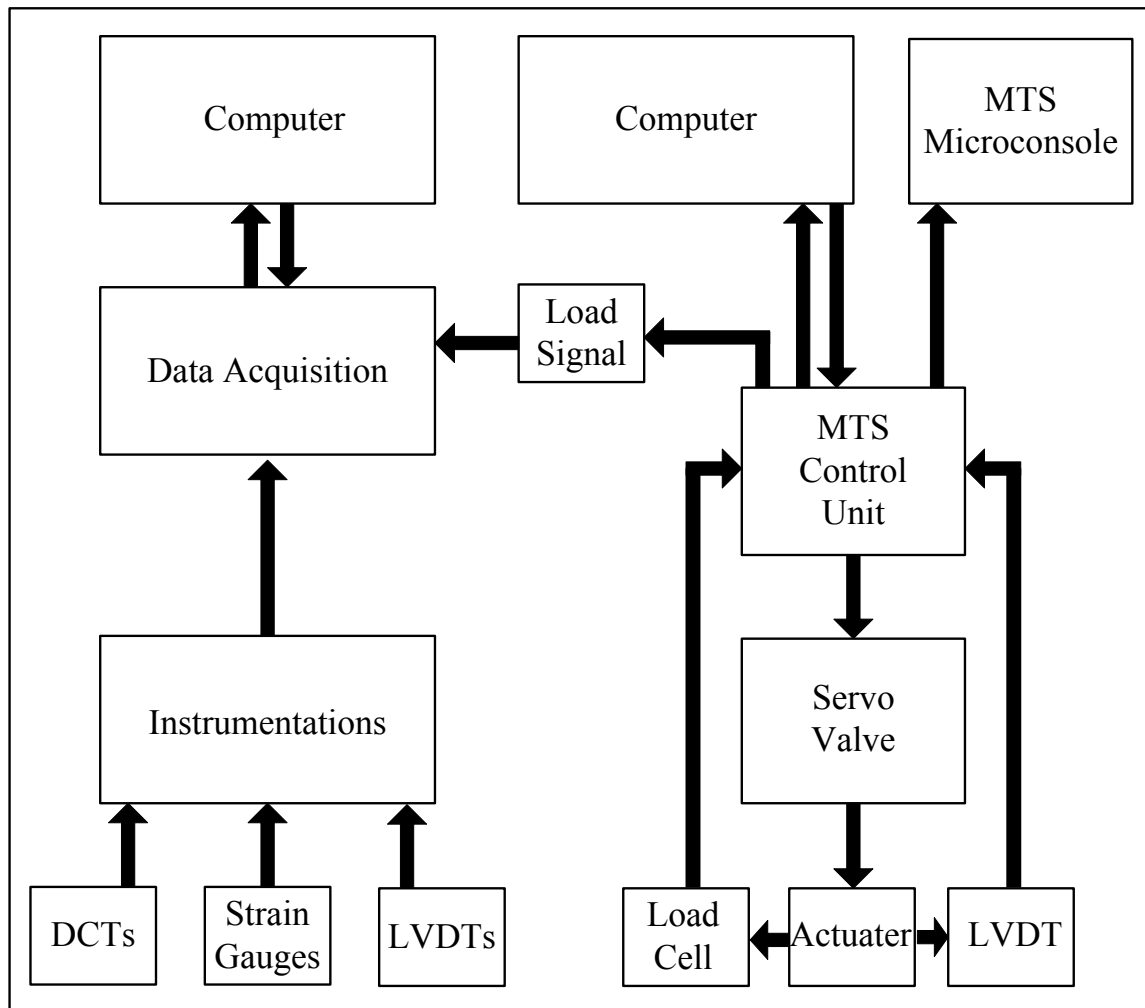
measured by one LVDT (L5) placed at one corner of the wall. The vertical anchorage slip at the base of the wall was monitored by LVDT L8 placed at a height of 200 mm from the top of the foundation (Figure 3.28).



**Figure 3.28** Location of LVDTs Near Wall Base

Two additional LVDTs (L9 and L10) were placed on the side of the top beam to record out-of-plane displacement of Wall W1-SR. The conventional wall W1-SR test was terminated at 2% drift due to significant out-of-plane displacements and to allow for repairing and retesting of the wall in a future research project. As a result of the out-of-plane displacements, the lateral bracing system previously described was used during the test of the second specimen W2-NR.

The readings from the instrumentation were recorded by a data acquisition system connected to a computer-controlled system. The data acquisition system consisted of Vishay scanner to record data and Vishay Strain Smart software to control the acquisition. A flowchart of the data acquisition system is shown in Figure 3.29.



**Figure 3.29** Flowchart of the Data Acquisition System

Each measurement sensor was connected to an individual channel control unit where the signals were conditioned and converted from analog to digital form. During the test, the signals from each channel were periodically scanned at a rate of 2 scans per second and recorded in the computer.

The data acquisition program converted the scanned instrumentation signals from voltages to units of strains, displacements or loads. The program also sent command signals to the Material Testing System (MTS). This feature allowed full control of the

hydraulic actuator. In addition, the program offered a real-time digital display and real-time plots of selected instrumentation readings.

### 3.7 Testing Program

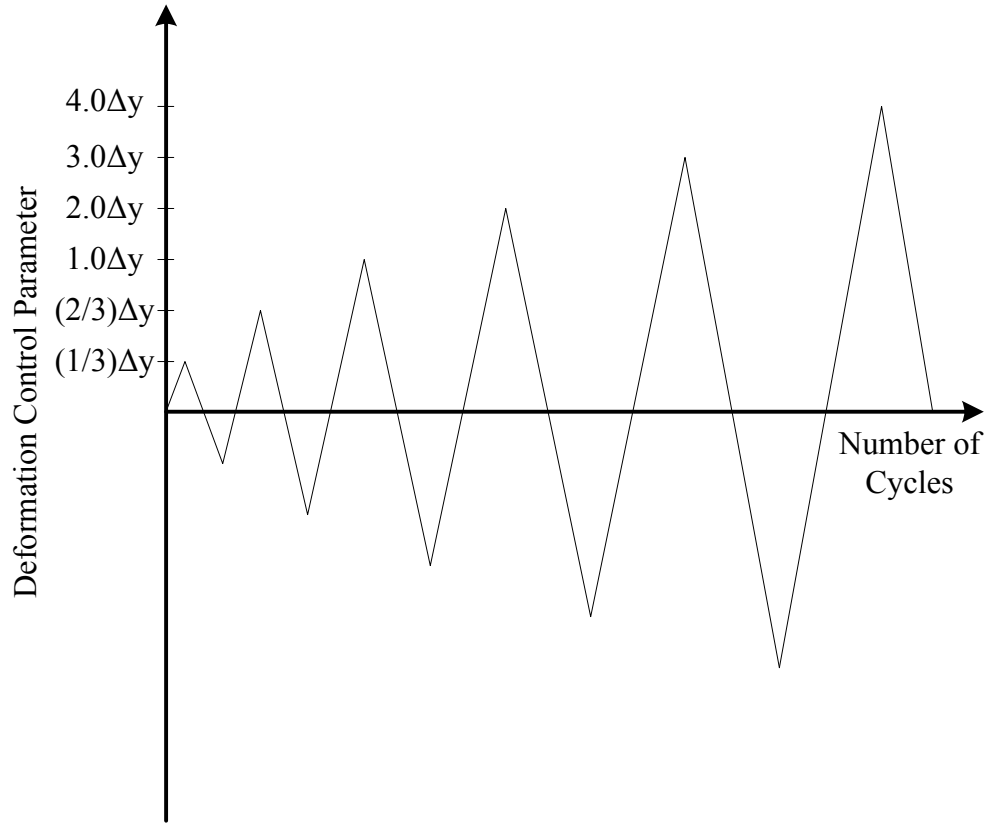
#### 3.7.1 Beams

All beam tests were performed at nearly the same ambient temperature at a quasi-static rate. Theoretical yield loads were calculated before the tests. During the monotonic test, and in order to keep the loading system stable, the beams were loaded first up to one-third of their calculated yield load and unloaded to a minimum load of 1 kN. The beams were then loaded slowly to the yield point, which was monitored by observing the load–displacement graph on the computer screen that was connected to the data acquisition system.

Based on The Guidelines for Cyclic Seismic Testing of Components of Steel Structures ATC-24 (1992), the yield load was established from results of the monotonic testing. The yield load corresponded to the ordinate prior to yielding from the experimental monotonic load-midspan displacement response. The estimated yield load was 21 kN and 28 kN for the SMA and conventional reinforced beams, respectively. The cyclic and reversed cyclic tests were controlled by displacements following ATC-24 with the exception of using one cycle instead of three as shown in Figure 3.30. The loading consisted of one cycle at  $\Delta y/3$ ,  $2\Delta y/3$ , and  $1.0\Delta y$ , where  $\Delta y$  is the yield displacement. Thereafter, subsequent cycles were multiples of  $\Delta y$  until failure or termination of testing. According to recommendations of ATC-24, the displacement at yield,  $\Delta y$ , is 1.33 times the displacement at first yield, which is defined as the displacement corresponding to 75% of the yield load  $P_y$ . The yield displacement,  $\Delta y$ , according to ATC-24 was



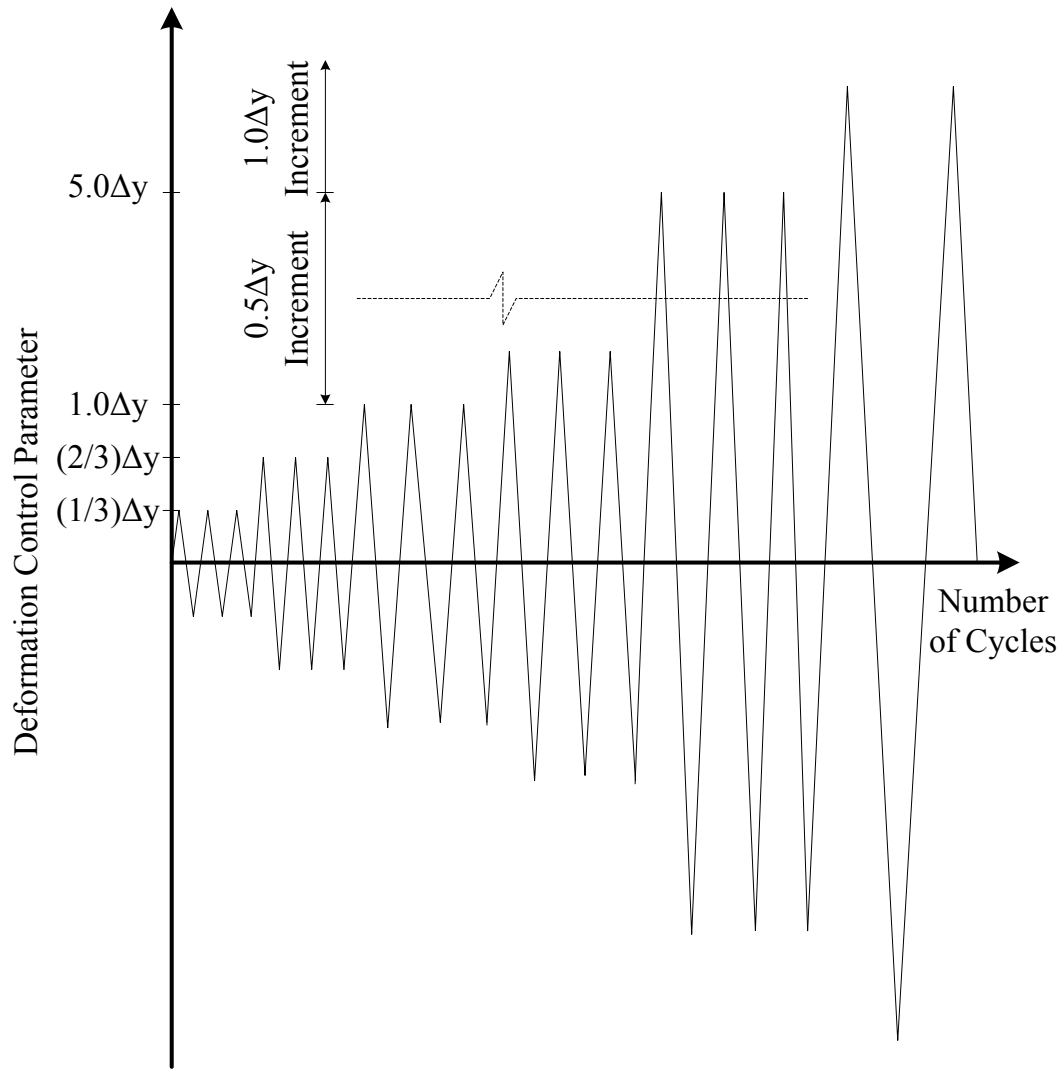
approximately 6.4 mm and 5.7 mm for the SMA and conventional reinforced beams, respectively.



**Figure 3.30** Reverse Cyclic Loading Sequence for Beam Tests

### 3.7.2 Shear Walls

The reinforced concrete shear wall test specimens were loaded in-plane according to a pre-determined quasi-static loading sequence. Loading was carried out in a sequence of reverse cycles at multiples of the yield displacement according to ATC-24 as shown in Figure 3.31.



**Figure 3.31** Loading Sequence for SMA Shear Wall Test

The walls were first loaded in three steps up to the yield load which was determined as described in Section 3.7.1. The yield load was determined from the theoretical monotonic lateral load-displacement response using the reinforced concrete sectional program Response 2000 (Bentz and Collins 2000). The first and the second loading stage aimed for  $\Delta y/3$  and  $2\Delta y/3$ , respectively. The yield load level was achieved during the third loading step. For each load step, three repetitions were imposed. The post-yield loading sequence continued to failure in displacement control. The deformation history consisted

of three cycles at each displacement level up to  $5\Delta y$  and two cycles thereafter. The load steps were increased by  $1\Delta y$  at each displacement level.

The predicted yield load was 98 kN and 84 kN for the conventional and the SMA shear walls, respectively. According to ATC-24, the corresponding yield displacements of the conventional and the SMA shear walls were approximately 6.2 mm and 12 mm, respectively. The SMA wall was loaded in increments of  $\Delta y/2$  up to  $5\Delta y$  in the post-yield range and by  $\Delta y$  thereafter.

## CHAPTER 4

### TEST RESULTS

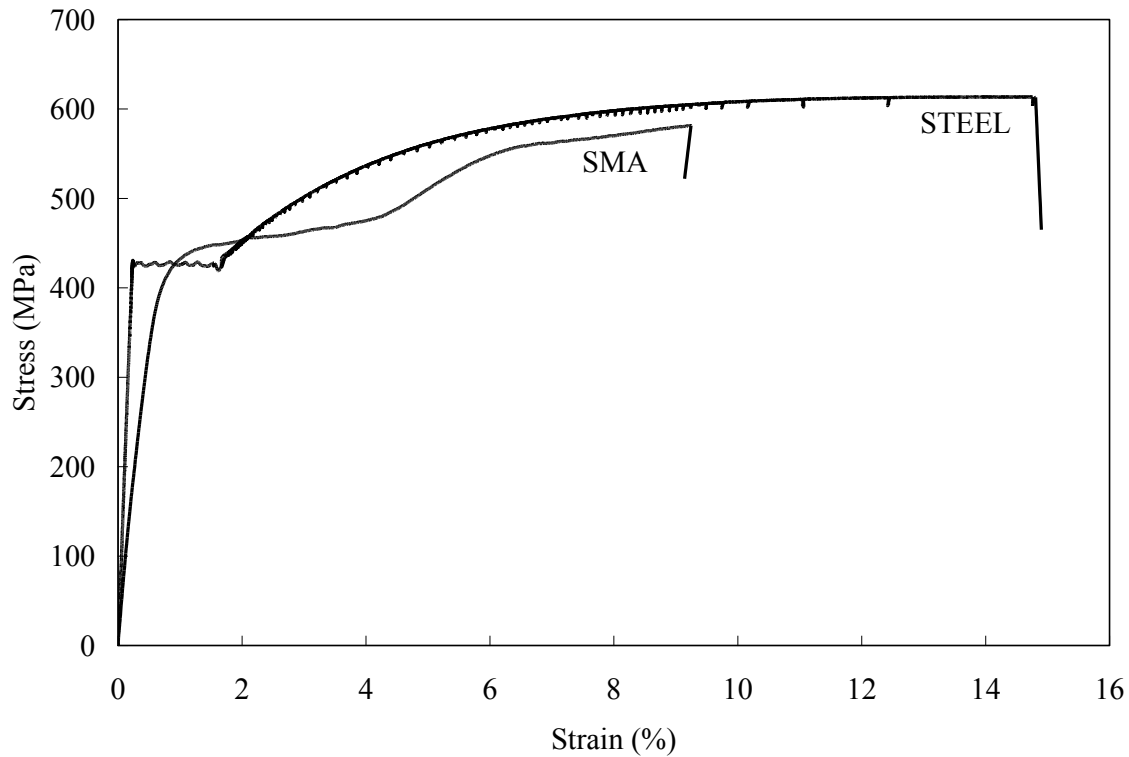
#### 4.1 Introduction

This chapter presents the experimental data and the observed behavior of the Nitinol (SMA) reinforcing bars, beam specimens, and shear wall specimens tested during the course of this research.

#### 4.2 SMA (Nitinol) Tests

Tensile tests were performed on a 12.7 mm diameter Nitinol bar to evaluate the mechanical properties of the material used for reinforcing the beams and shear walls. The Nitinol sample was in compliance with ASTM F2516-07 (Standard Test Method for Tension Testing of Nickel-Titanium Superelastic Materials 2007) and was tested under monotonic and cyclic loadings. Similar tests were conducted on conventional 10M steel samples for comparison. The sample specimens were 300 mm long, and the diameter of the Nitinol bars as used for beam tests, was reduced to 9.5 mm at mid-height to simulate the shape of the bars used in the beams at the critical section. Furthermore, the ends of the Nitinol bars were threaded and connected to conventional deformed 15M bars with threaded mechanical couplers to replicate the coupling system used in the beams. The diameter of the Nitinol bars used in the shear walls was 12.7 mm. Tension tests for these bars were conducted with the Nitinol bars connected to conventional deformed 15M steel using screw lock couplers as described in Section 3.3.2. To record the axial straining in the reinforcement, a linear variable differential transducer (LVDT) was mounted to the samples at mid-height for the beam SMA samples and an extensometer was used for the

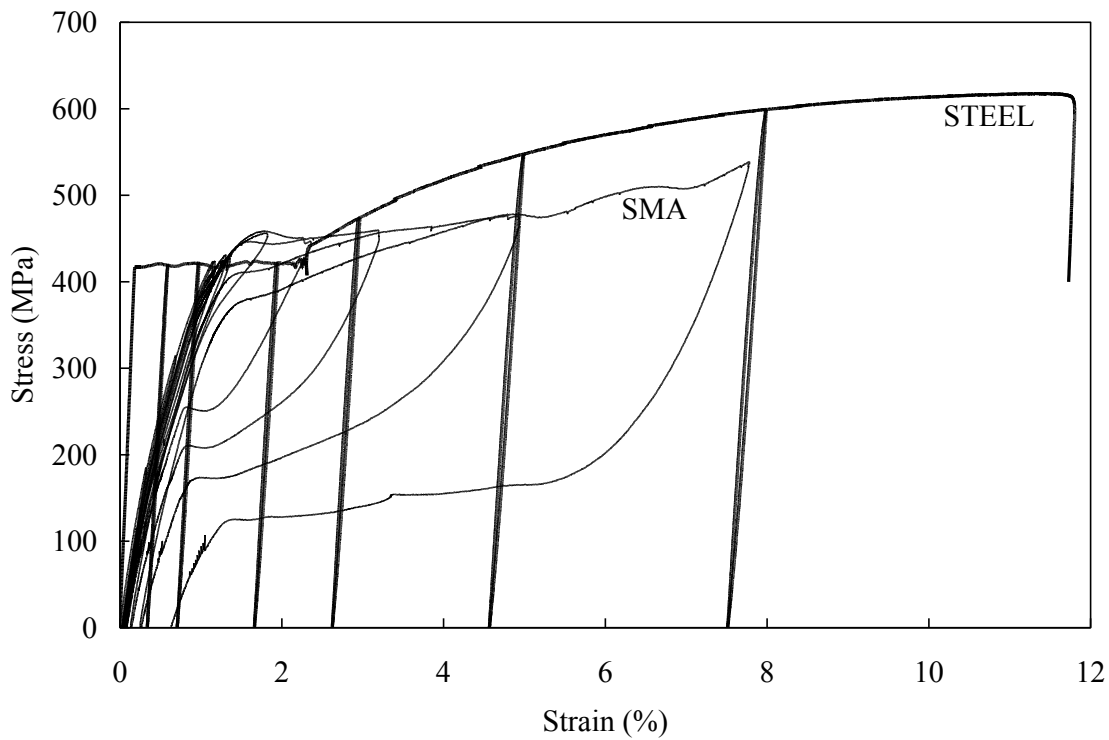
shear wall SMA samples for the same purpose. Testing was performed using a 600 kN Universal Testing Machine. The loading was imposed at a crosshead speed of approximately 2 mm/min. Figure 4.1 provides the monotonic stress-strain responses for the 10M conventional deformed bar and the 12.7 mm Nitinol reshaped bar used in the beam tests.



**Figure 4.1** Monotonic Stress-Strain Response of Reinforcement in Beams

The modulus of elasticity of the Nitinol bar was approximately 60 GPa, while the modulus of elasticity of the 10M deformed bar was approximately 205 GPa. Yielding of the SMA bar was identified as the initiation of the forward transformation phase. The Nitinol bar demonstrated a rounded loading curve and, thus, did not exhibit a well-defined yield plateau. Therefore, the yield point was based on a 0.2% offset, resulting in an approximate stress of 415 MPa. The 10M conventional steel yielded at 425 MPa and

the ultimate strength was 615 MPa. The bar failed by rupturing at mid-height near the LVDT location. The SMA bars reached an ultimate stresses of approximately 590 MPa at approximately 10% strain. The SMA bar failed earlier than expected at the threaded couplers. The cyclic response of the two reinforcing bar types is shown in Figure 4.2. The unloading and reloading curves of the Nitinol bar demonstrate significantly more nonlinearity in comparison to the 10M bar.



**Figure 4.2** Cyclic Stress-Strain Response of Reinforcement in Beams

The residual strains at the completion of unloading provide a clear differentiation between the Nitinol and conventional deformed bars. For the Nitinol bar, the residual strains were approximately 0.15% and 0.27% after loading the bar to approximately 3% and 4.8% strains, respectively. During the last loading cycle, the Nitinol bar was strained to 7.7%, and upon unloading, the residual strain was 0.65%, representing a 91% strain

recovery capacity. The conventional deformed 10M bar demonstrated significantly larger residual strains. During the last loading cycle to 8% strain, the 10M bar experienced a permanent strain of approximately 7.5%, representing an approximate strain recovery of 6%. The Nitinol cyclic response demonstrated that the loading plateaus (forward and reverse transformation stresses) gradually decrease while the residual strain increases for successive loading cycles due to the formation of stress-induced Martensite.

### 4.3 Beam Tests

#### 4.3.1 Cracking Characteristics

The behavior of the SMA and conventional (control) reinforced beams was investigated in terms of cracking patterns, maximum crack widths, and residual crack widths upon removal of load. Although cracking is generally a serviceability limit state, the following provides crack width characteristics for entire range of testing to demonstrate the effect of smooth SMA bars and the ability to recover crack opening.

For all beams, initial flexural cracking was observed, for the most part, in the critical (constant moment) region. Flexural cracking also surfaced outside the critical zone, and as the load increased, inclined shear cracks were noted near the supports. Additional shear cracks became evident with increased load, while the flexural cracking continued to propagate toward the loading points near midspan of the beams. In both the conventional and SMA beams, the first flexural crack was observed at approximately 4 kN of load. At this load level, B1-SM (conventional, monotonic) and B4-NM (Nitinol, monotonic) had crack widths of 0.1 mm and 0.15 mm, respectively, in the critical zone. With increased load, the flexural cracks in the SMA beam became wider and were spaced farther in comparison to the conventional beam. This was attributed to the smooth surface of the

Nitinol bars. The deformations along the conventional reinforcement aided in anchoring to the surrounding concrete promoting closer spaced cracks of smaller widths. Conversely, the smooth SMA bars exhibited reduced bonding resulting in fewer cracks of greater spacing and larger widths. For conventionally reinforced beams, large crack widths pose serviceability concerns, specifically the potential for corrosion. However, SMAs are resilient to corrosion, thus larger crack widths can be tolerated in SMA reinforced members. At yielding, the flexural cracks in B1-SM were 0.35 mm wide and spaced approximately 100 mm; while in B4-NM, the cracks were 2.0 mm wide and spaced at 140 mm. Prior to failure, and at a displacement ductility of approximately  $7\Delta_y$ , Beam B1-SM experienced a maximum crack width of 11 mm, while a crack width of 52 mm was recorded for Beam B4-NM.

The residual crack widths recorded under cyclic and reverse cyclic loading demonstrated the ability of the SMA beams to re-center. Maximum and residual crack widths, and average crack spacing were recorded as shown in Table 4.1.

**Table 4.1** Crack Width and Spacing in Beams

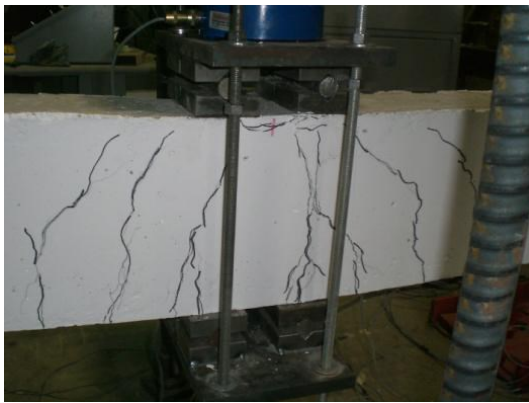
| Loading Cycle | Beam                      |                  |                           |                  |                           |                  |                           |                  |
|---------------|---------------------------|------------------|---------------------------|------------------|---------------------------|------------------|---------------------------|------------------|
|               | B2-SC                     |                  | B7-NCM                    |                  | B3-SR                     |                  | B6-NR                     |                  |
|               | $S_{cr} = 110 \text{ mm}$ |                  | $S_{cr} = 155 \text{ mm}$ |                  | $S_{cr} = 100 \text{ mm}$ |                  | $S_{cr} = 150 \text{ mm}$ |                  |
|               | $W_{cr}$<br>(mm)          | $R_{cr}$<br>(mm) | $W_{cr}$<br>(mm)          | $R_{cr}$<br>(mm) | $W_{cr}$<br>(mm)          | $R_{cr}$<br>(mm) | $W_{cr}$<br>(mm)          | $R_{cr}$<br>(mm) |
| $1\Delta_y$   | 0.3                       | 0.21             | 1.8                       | 0.28             | 0.28                      | 0.15             | 1.8                       | 0.15             |
| $2\Delta_y$   | 3.5                       | 2.5              | 6                         | 0.32             | 1.5                       | 1.25             | 5                         | 0.4              |
| $3\Delta_y$   | 4.5                       | 3                | 11                        | 1.1              | 2.5                       | 2                | 9                         | 0.8              |
| $4\Delta_y$   | 5.5                       | 4                | 16                        | 2                | 3.5                       | 2.75             | 12                        | 1                |
| $5\Delta_y$   | 6                         | 5                | 24                        | 2.4              | 5                         | 4.25             | 21                        | 2.5              |
| $6\Delta_y$   | 7                         | 5.5              | 34                        | 3                | 8                         | 6.4              | 30                        | 3                |
| $7\Delta_y$   | -                         | -                | -                         | -                | 9.5                       | 7.5              | 48                        | 5.5              |

$S_{cr}$  = Crack spacing;  $W_{cr}$  = Crack width;  $R_{cr}$  = Residual crack width



At a displacement ductility of  $1\Delta y$ , the crack width and spacing for B2-SC (conventional, cyclic loading) and B3-SR (conventional, reversed cyclic loading) were approximately 0.3 mm and 110 mm, and 0.28 mm and 100 mm, respectively. The corresponding residual crack widths and cracking opening recovery were 0.21 mm and 30%, and 0.15 mm and 46%, respectively for Beams B2-SC and B3-SR. At  $1\Delta y$ , the crack width and spacing for SMA Beams B7-NCM (Nitinol, cyclic loading) and B6-NR (Nitinol, reverse cyclic) were 1.8 mm and 155 mm, and 1.8 mm and 150 mm, respectively. Upon removal of load, the residual crack width for B7-NCM was 0.28 mm and for B6-NR was 0.15 mm. Thus, B7-NCM and B6-NR were capable of recovering approximately 84% and 92% of the maximum crack widths, respectively. Prior to failure and at a displacement ductility of  $6\Delta y$  for B2-SC and  $7\Delta y$  for B3-SR, the crack and residual widths were 7 mm and 5.5 mm, and 9.5 mm and 7.5 mm, respectively. The corresponding crack recovery was approximately 21% for both conventionally reinforced beams. Conversely, at  $6\Delta y$  for B7-NCM and  $7\Delta y$  for B6-NR, maximum crack widths and resulting residual crack widths of 34 mm and 3 mm, and 48 mm and 5.5 mm, respectively, were noted during testing. The crack recovery capacities for B7-NCM and B6-NR were approximately 91% and 89%, respectively. The SMA reinforced beams demonstrated a significant enhancement in crack recovery capacity in comparison to the conventionally reinforced beams. The average recovery capacity for the SMA beams was 90% throughout testing, whereas the conventional beams recovered, on average, 25% of the maximum noted crack openings. Figure 4.3 illustrates the condition of the beams at the end of testing. Note that Figure 4.3 (d) is a photo of Beam B7-NCM at the end of the cyclic loading

phase. Cracking characteristics of the conventional and SMA beams are included in Appendix E.



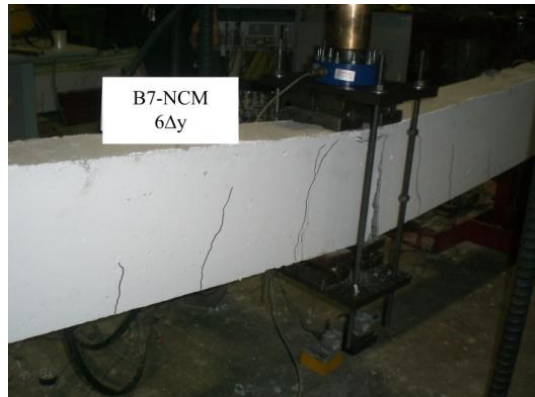
(a)



(b)



(c)



(d)



(e)

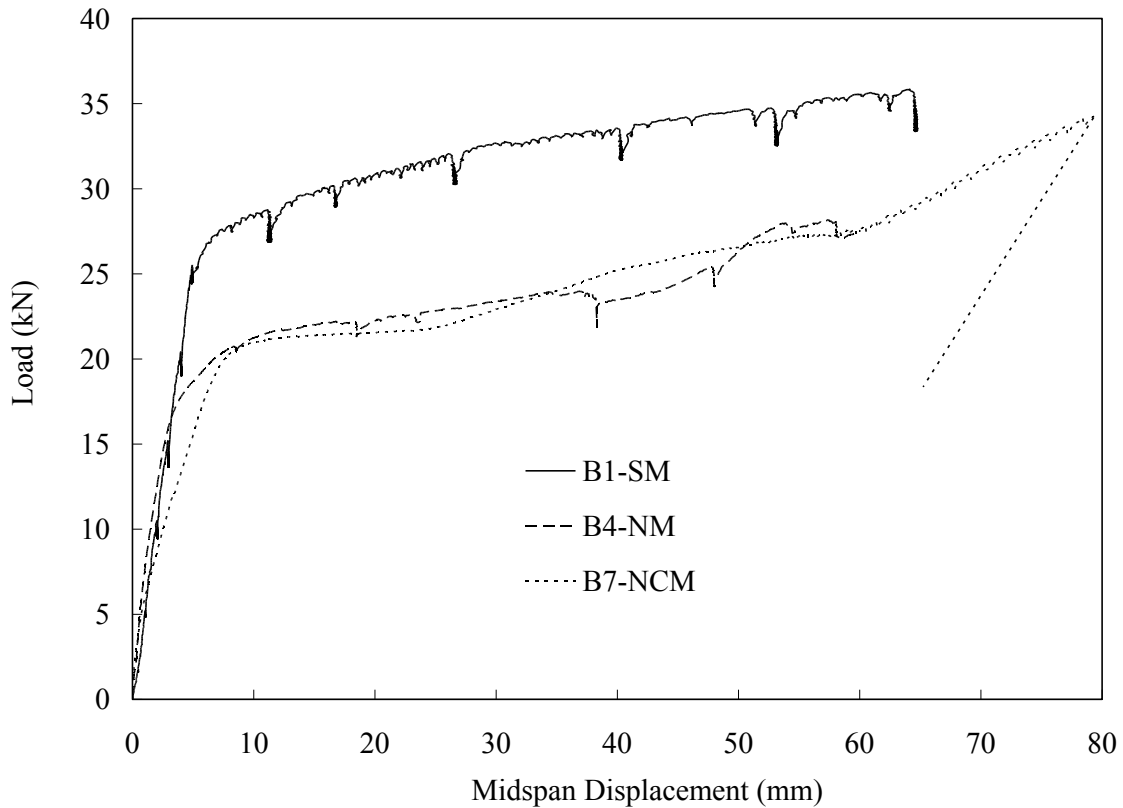


(f)

**Figure 4.3** Failure Condition of Test Beams: (a) B1-SM; (b) B4-NM; (c) B2-SC; (d) B7NCM; (e) B3-SR; and (f) B6-NR

#### 4.3.2 Load-Displacement Response

The measured load–displacement results for all seven beams were studied to determine the effects of Nitinol (NiTi) compared to conventional steel. Four factors were of particular interest: the initial stiffness, ultimate load, ultimate displacement and the residual displacement of the specimens. Figure 4.4 shows the experimental load-midspan displacement response of Beams B1-SM, B4-NM, and B7-NCM. The responses display characteristics of the stress-strain behaviors of the reinforcement used in the critical region, which was expected for ductile dominated response.



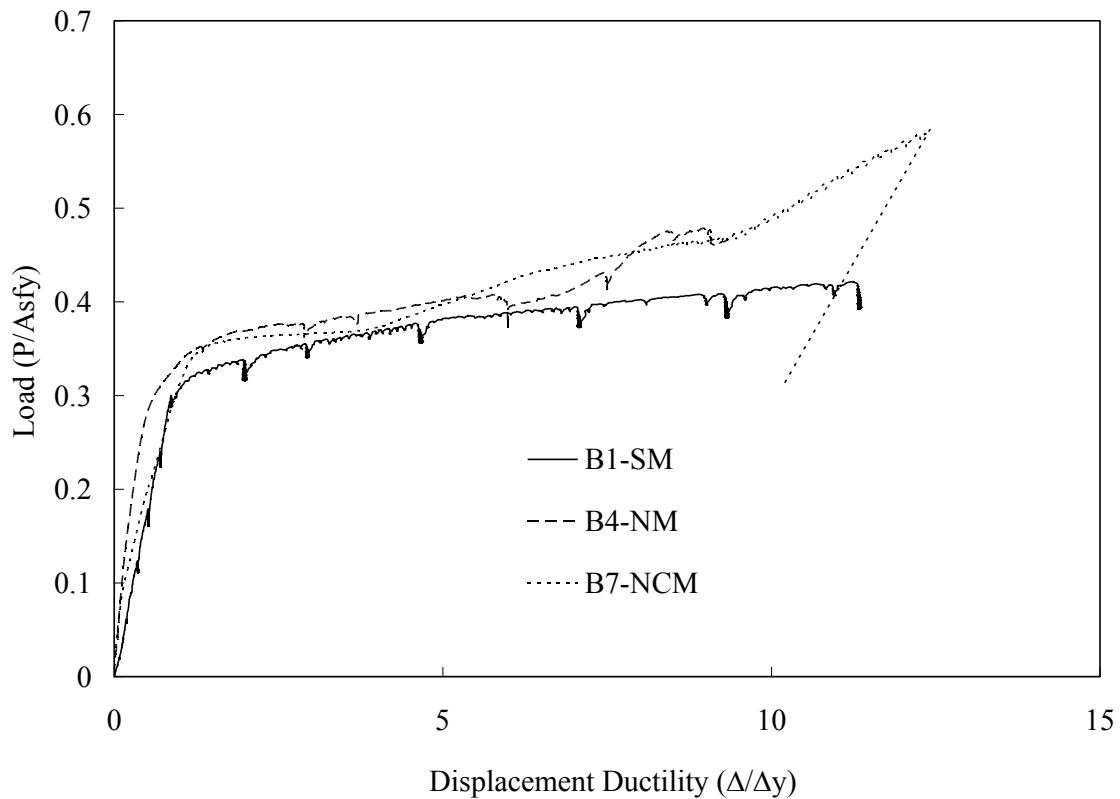
**Figure 4.4** Monotonic Load-Displacement Responses

The conventional beam (B1-SM) behaved with a well-defined elastic loading branch followed by a yielding response with post-yield stiffness. The beam failed at

approximately 65 mm, corresponding to 36 kN of load, by concrete crushing in the flexural compression zone after significant yielding of the longitudinal reinforcement. SMA Beam B4-NM had a similar elastic stiffness as B1-SM up to approximately 17 kN. Beyond this load level, there was softening in the response as the Nitinol bar experienced yielding (forward transformation). The post-yielding stiffness was comparable to B1-SM. Beam B4-NM failed at approximately 60 mm, corresponding to 28 kN due to rupturing of the Nitinol bar at the threaded section adjacent to the mechanical coupler. Concrete crushing in the flexural compression zone and a large flexural crack in the critical section were visible at failure. Beam B7-NCM was first subjected to cyclic loading and then monotonic loading in the opposite direction. The intent was to determine the strength and ductility capacities under monotonic loading of an SMA beam controlled by failure in the critical section. The initial stiffness was less than both B1-SM and B4-NM mostly due to damage that was experienced by the beam during the cyclic loading phase. The post-yielding stiffness was similar to the other beams; however, Beam B7-NCM experienced a hardening response at approximately 60 mm, resulting in a significant increase in load capacity. The beam failed at approximately 80 mm and 34 kN of load by rupturing of the Nitinol bar in the critical section coinciding with concrete crushing in the flexural compression zone. The load-midspan displacement responses appear to suggest that the conventional beam sustained higher yield and ultimate strength capacities; however, this was influenced by the higher tensile force capacity of the longitudinal reinforcement in the conventional reinforced beam. A more appropriate comparison of the strength capacity of the beams was investigated taking into account the difference in the area of longitudinal reinforcement in the critical section and the corresponding yield stress. The

SMA bar had a cross sectional area of approximately  $71 \text{ mm}^2$  at the critical section and corresponding yield strength of 415 MPa, whereas the conventional reinforcement had a cross section of  $100 \text{ mm}^2$  and yield strength of 425 MPa.

The load-displacement responses were normalized in Figure 4.5 with respect to the tensile yield force ( $A_s f_y$ ) of the flexural reinforcement in the critical region. Furthermore, the midspan displacement was replaced by the displacement ductility to reflect differences in yield displacement.



**Figure 4.5** Normalized Monotonic Load-Displacement Ductility Response

Figure 4.5 demonstrates that the normalized yield load was similar for all the beams. Beyond yielding, and up to a displacement ductility of 10, the SMA beams experienced slightly larger normalized load capacity. At the end of testing, Nitinol Beam B7-NCM

experienced larger normalized load capacity than the other beams. The normalized responses also indicate that the SMA beams have similar ductility capacity as the conventional reinforced beams. Note that B4-NM failed at the threaded section of the Nitinol bar; therefore, the beam experienced less strength and ductility than expected. Table 4.2 provides the yield and ultimate load capacities, and corresponding displacement ductility capacities for all beam tests.

**Table 4.2** Load and Displacement Ductility Capacities

| Beam                      | Loads       |            |             |            | Displacements |
|---------------------------|-------------|------------|-------------|------------|---------------|
|                           | Yield       |            | Ultimate    |            | Ductility     |
|                           | Actual (kN) | Normalized | Actual (kN) | Normalized |               |
| <b>B1-SM</b>              | 28          | 0.33       | 36          | 0.42       | 11.3          |
| <b>B2-SC</b>              | 28          | 0.33       | 35          | 0.41       | 8.2           |
| <b>B3-SR</b>              | 27          | 0.32       | 35          | 0.41       | 9.4           |
| <b>B4-NM</b>              | 21          | 0.36       | 28          | 0.48       | 9.3           |
| <b>B6-NR</b>              | 20          | 0.34       | 23          | 0.39       | 9.5           |
| <b>B7-NCM<sub>1</sub></b> | 21          | 0.34       | 24          | 0.41       | 6.4           |
| <b>B7-NCM<sub>2</sub></b> | 21          | 0.36       | 34          | 0.58       | 12.4          |

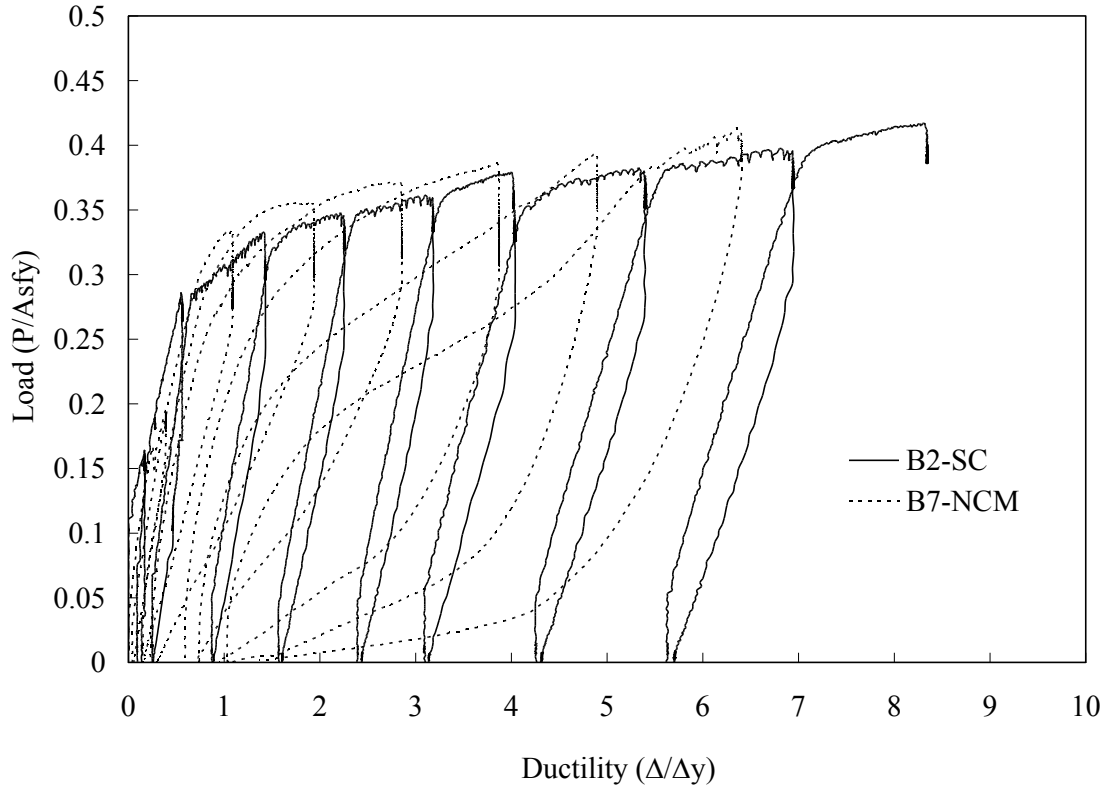
<sub>1</sub>Results of the cyclic load test. This beam was not tested to failure.

<sub>2</sub>Results of the monotonic test

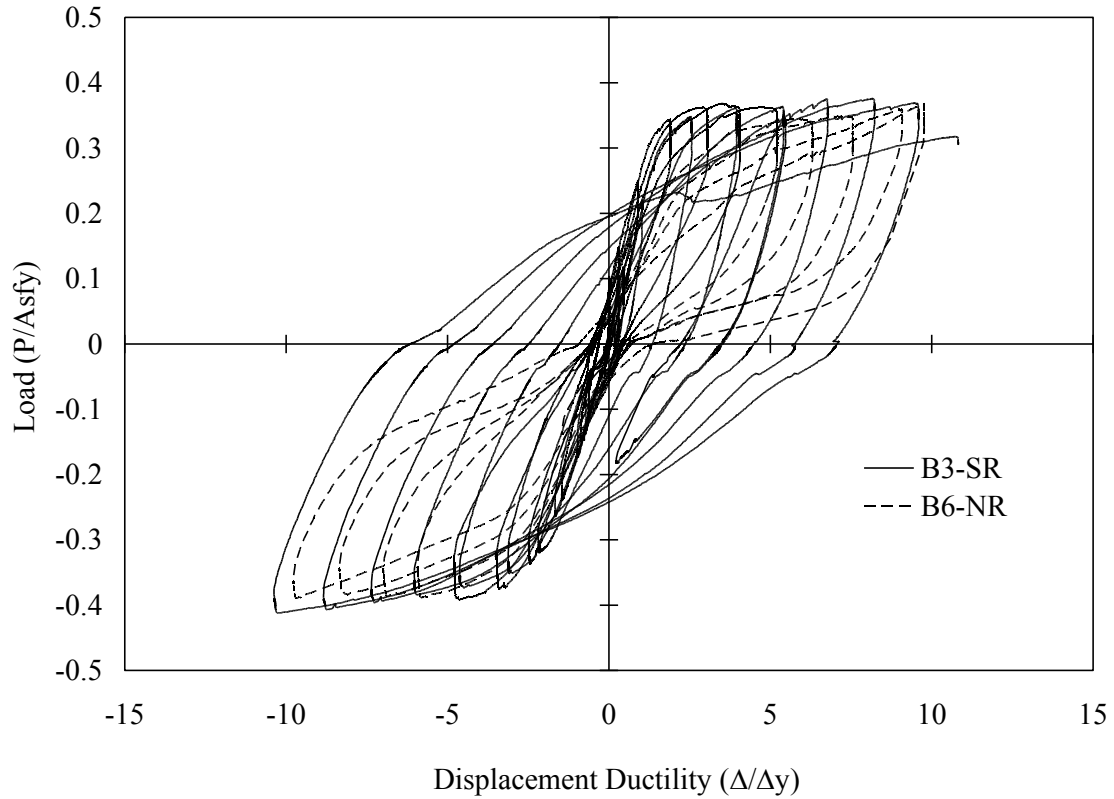
The normalized load-displacement ductility responses of the Nitinol and conventional reinforced beams under cyclic and reversed cyclic loading are shown in Figures 4.6 and 4.7, respectively.

The results indicate that the SMA and conventional reinforced beams experienced comparable normalized yield load, normalized ultimate load, and ductility. Testing of Beam B7-NCM was terminated prior to failure to permit monotonic loading in the opposite direction; therefore, the ultimate ductility was not realized. The unloading curves demonstrated stiffness similar to initial loading stiffness; however, the SMA beams (B7-NMC and B6-NR) experienced a rapid recovery of displacement when the

Nitinol bars reached the reverse transformation stress. Therefore, the pinching evident in the responses was a result of the strain recovery capacity. The reloading curves of the SMA beams are significantly more nonlinear.



**Figure 4.6** Normalized Load-Displacement Ductility Response under Cyclic Loading



**Figure 4.7** Normalized Load-Displacement Ductility Response under Reverse Cyclic Loading

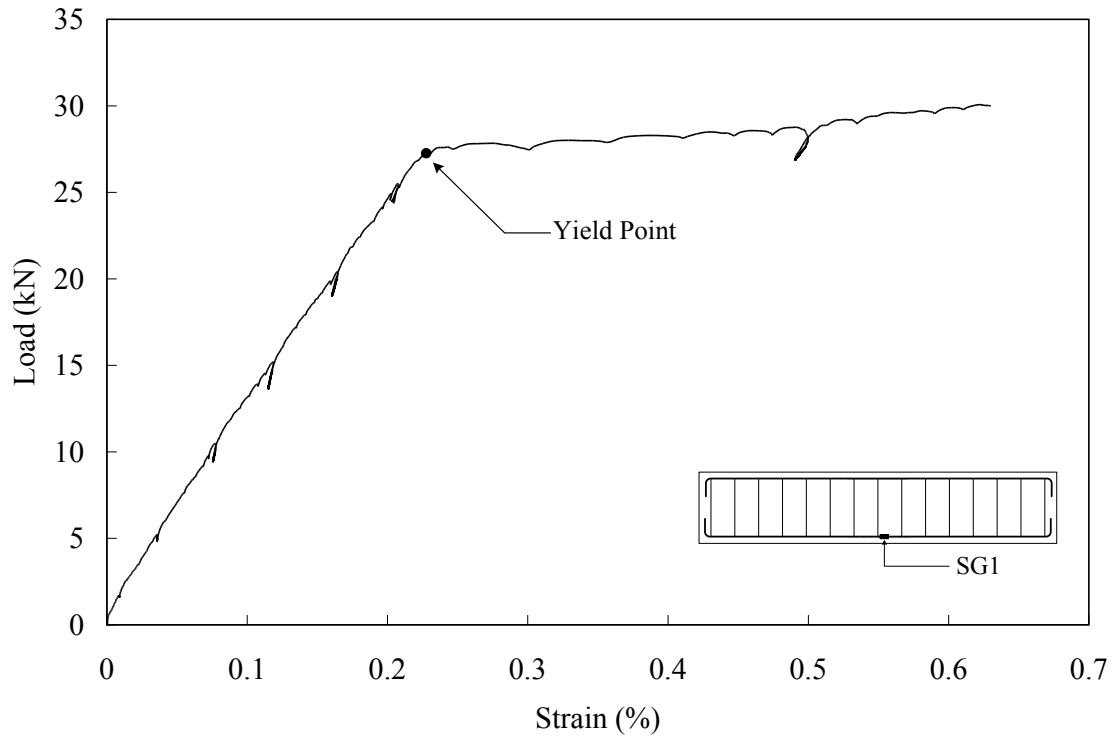
Beams B2-SC and B3-SR failed in flexure by concrete crushing in the flexural compression zone, preceded by significant yielding of the longitudinal reinforcement. Beam B7-NCM did not experience failure as testing was terminated prior; however, a major crack was visible in the critical section. Beam B6-NR failed due to rupturing of the Nitinol bar at the transition zone where the bar diameter changed from 12.5 mm to 9.5 mm. Concrete crushing near the load application was also visible at failure. Rupturing of the SMA bar outside the critical section caused a reduction of load-carrying capacity and ultimate ductility. Although failure occurred at the transition zone, the normalized load-displacement ductility response under reverse cyclic loading illustrates that the SMA



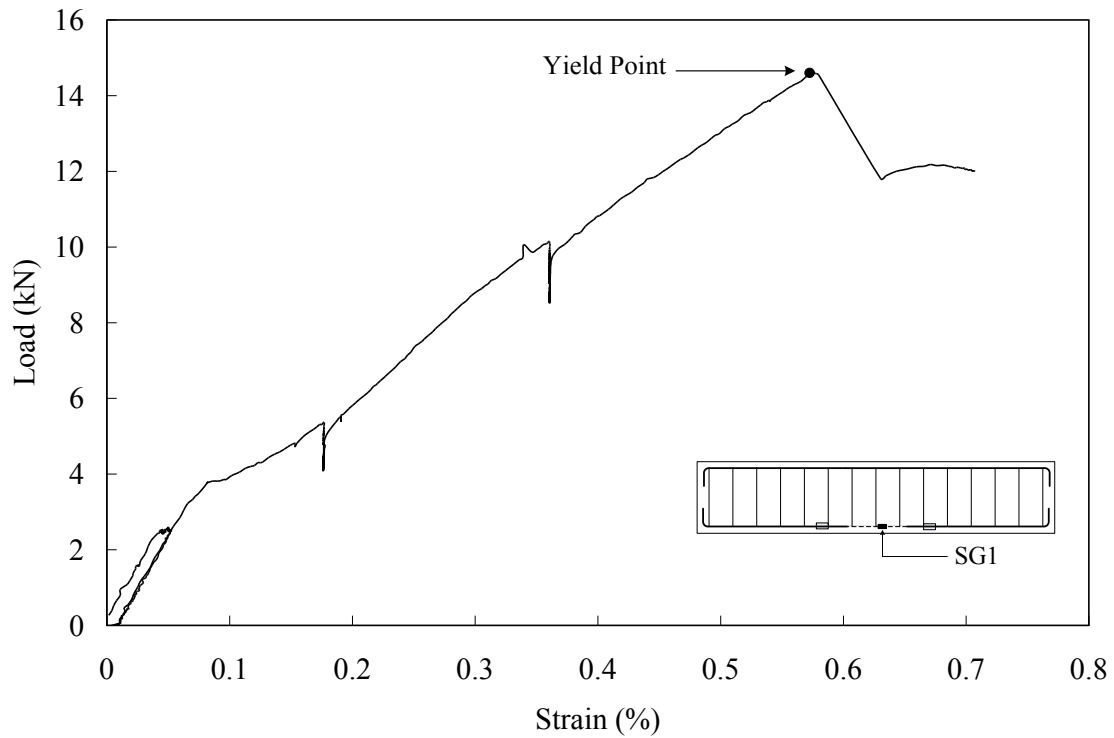
beam was capable of experiencing similar displacement ductility as the conventional reinforced beam.

#### 4.3.3 Reinforcement Strains

Reinforcement strains were measured in the longitudinal and transverse reinforcing bars. Precautions were taken in attaching the strain gauges on the SMA rods. However, some of the strain gauges in the SMA beams did not function after yielding. Figures 4.8 and 4.9 show the measured strains of Strain Gauge 1 (SG1) located on the main longitudinal reinforcement in the plastic hinge region (midspan of beams) in the conventional (B1-SM) and SMA (B4-NM) reinforced beams, respectively.



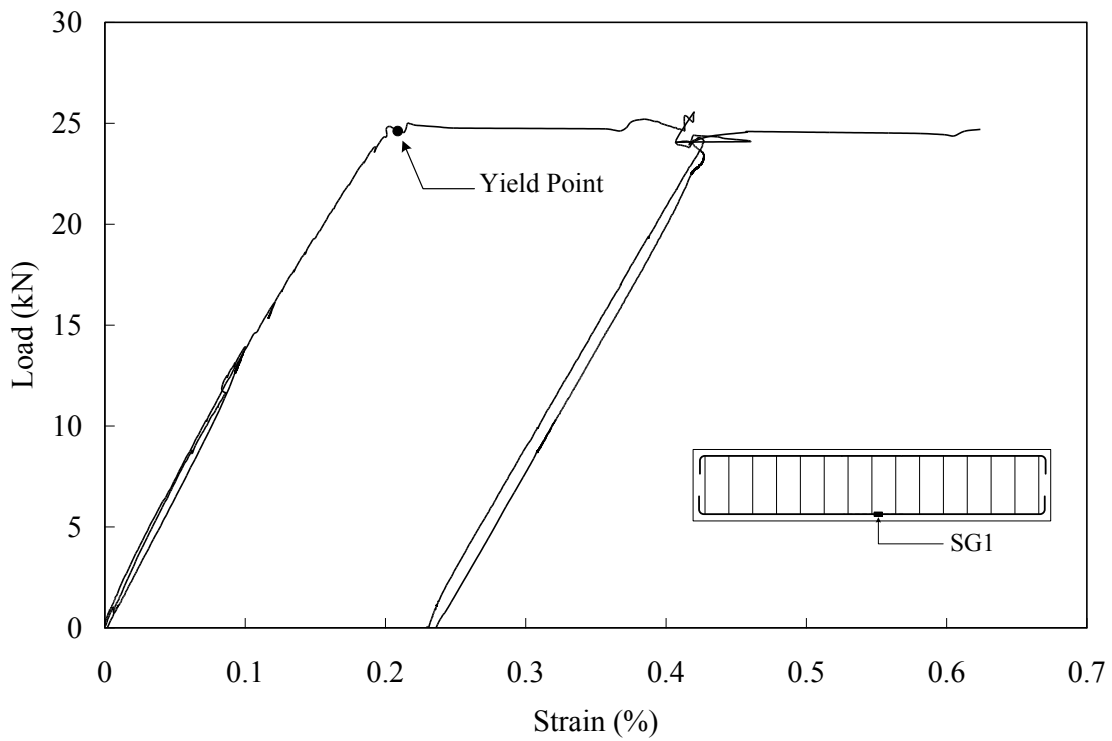
**Figure 4.8** Load-Strain Response of SG1 in B1-SM



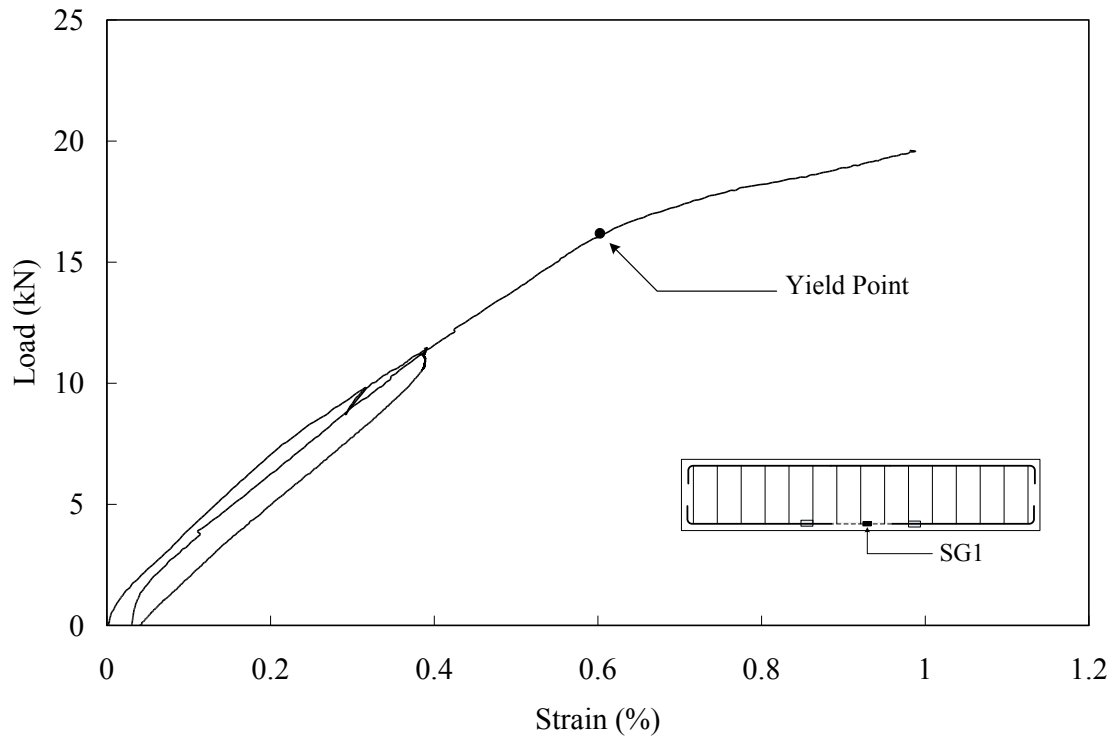
**Figure 4.9** Load-Strain Response of SG1 in B4-NM

The responses demonstrate the lower modulus of elasticity for the SMA reinforcement compared to the steel reinforcement, resulting in higher strains at midspan when first crack initiated. This led to a major crack in the plastic hinge region for B4-NM. At first crack, the measured strain in B1-SM was 0.04%, while the corresponding strain for specimen B4-NM was 0.09%. In both specimens, strain gauges were capable of capturing the strain up to 0.6%; about three times the yield strain of the conventional steel in B1-SM and approximately the yielding point of the SMA in B4-NM. At this strain level, the applied load was 27 kN in B1-SM while it was only 15 kN in B4-NM. This difference is related to the lower yielding stress of the SMA and smaller reinforcement ratio used in B4-NM.

The conventional and SMA beams tested under cyclic loading illustrated that specimen B2-SC experienced significant residual straining beyond yielding, whereas B5-NC experienced negligible residual strains. However, strains in the conventional and SMA bars could not be recorded to the end of the testing due to damage of the gauges. After unloading from 0.4% strain, the residual strain in specimen B2-SC was 0.22%, whereas the residual strain in specimen B5-NC was 0.04% (Figures 4.10 and 4.11). Therefore, B2-SC experienced a residual strain of 5.5 times that of B5-NC. The recovery of strains in B5-NC is due to the re-centering behavior of the superelastic reinforcement upon unloading.

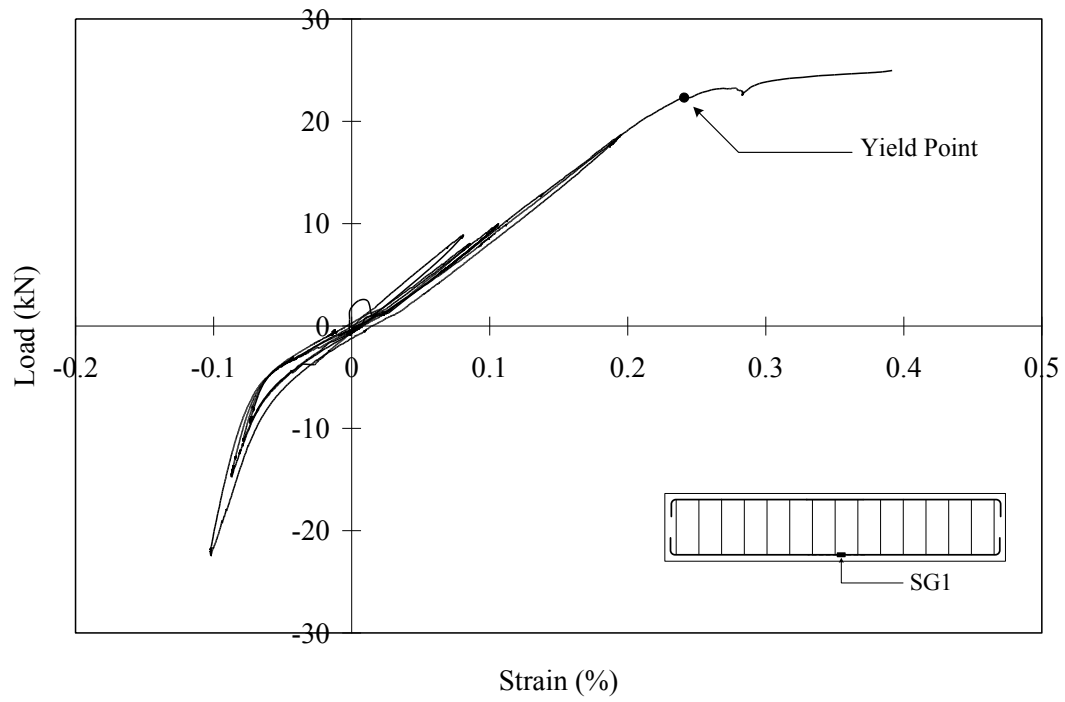


**Figure 4.10** Load-Strain Response of SG1 in B2-SC

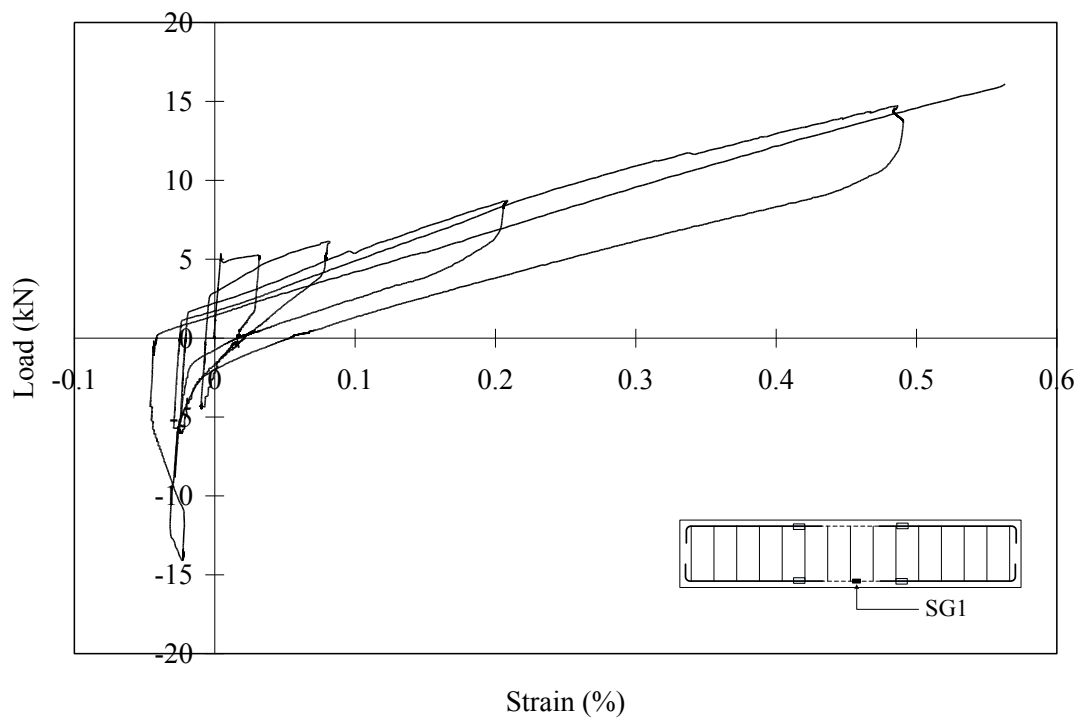


**Figure 4.11** Load-Strain Response of SG1 in B5-NC

Figures 4.12 and 4.13 provide the load-strain results under reverse cyclic loading for Beams B3-SR and B6-NR, respectively. The results confirmed the re-centering capability of the superelastic reinforcement upon unloading. After experiencing a strain of 0.5%, specimen B6-NR recovered 90% of the strain upon unloading as shown in Figure 4.13. In the conventional beam, B3-SR, no strain was recorded at that level to provide a comparison as shown in Figure 4.12. For this beam, the unloading was within the elastic range; therefore, marginal residual straining is evident.



**Figure 4.12** Load-Strain Response of SG1 in B3-SR



**Figure 4.13** Load-Strain Response of SG1 in B6-NR

The responses of other strain gauges are provided in Appendix E.

#### 4.4 Shear Wall Tests

##### 4.4.1 Cracking Characteristics

The distribution and pattern of flexural and shear cracks in both the conventional (W1-SR) and SMA (W2-NR) shear walls were monitored and recorded. In both walls, flexural cracks initiated near the wall base, and as the load increased, inclined shear cracks developed from the initial flexural cracks. At the same time, the flexural cracks became wider and propagated into the wall. The first flexural cracks surfaced during the first load stage in W1-SR and W2-NR and were perpendicular to the longitudinal reinforcing bars. The lateral load was approximately 48 kN which corresponded to a displacement of 2.4 mm (0.1% of drift). The first cracks initiated slightly above the top of the starter bars: two cracks were observed in W1-SR, one at 250 mm and the other at 380 mm from the base, while only one crack was observed in W2-NR at a height of 350 mm from the base (Figure 4.14).



**Figure 4.14** First Flexural Cracking in Shear Wall Specimens: (a) W1-SR; and (b) W2-NR

The crack widths at this load level were 0.15 mm and 0.85 mm in W1-SR and W2-NR respectively. The first diagonal shear cracks were noticed at the end of the second load stage to 4.8 mm of displacement (0.2% drift) in both W1-SR and W2-NR.

As the loading progressed, additional flexural and shear cracks formed and existing flexural cracks extended in length. The flexural cracks in the SMA wall became wider and spaced farther in comparison with the conventional wall. Moreover, the flexural crack width was observed to be wider in the boundary region than the interior region of the SMA wall (Figure 4.15). This was partly due to the smoothness of the SMA bars as discussed in Section 4.3.1.



**Figure 4.15** Flexural Crack in SMA Shear Wall

At yielding, the number of flexural and shear cracks increased along with an increase in width. The maximum flexural crack width in W1-SR and W2-NR were 0.6 mm and 2.5 mm, respectively. More shear cracks were observed in the middle region of the web of W1-SR than W2-NR. During the post-yield loading sequence up to a displacement level of 30 mm (1.25% drift), the crack pattern experienced significant change with additional flexural and shear cracks surfacing (Figure 4.16). At this displacement level, the maximum flexural crack width near the base of the Wall W2-NR was 9.5 mm which was significantly wider than the maximum flexural crack of 2.2 mm observed in Wall W1-SR.



**Figure 4.16** Crack Pattern of Test Walls at 1.25% Drift: (a) W1-SR; and (b) W2-NR

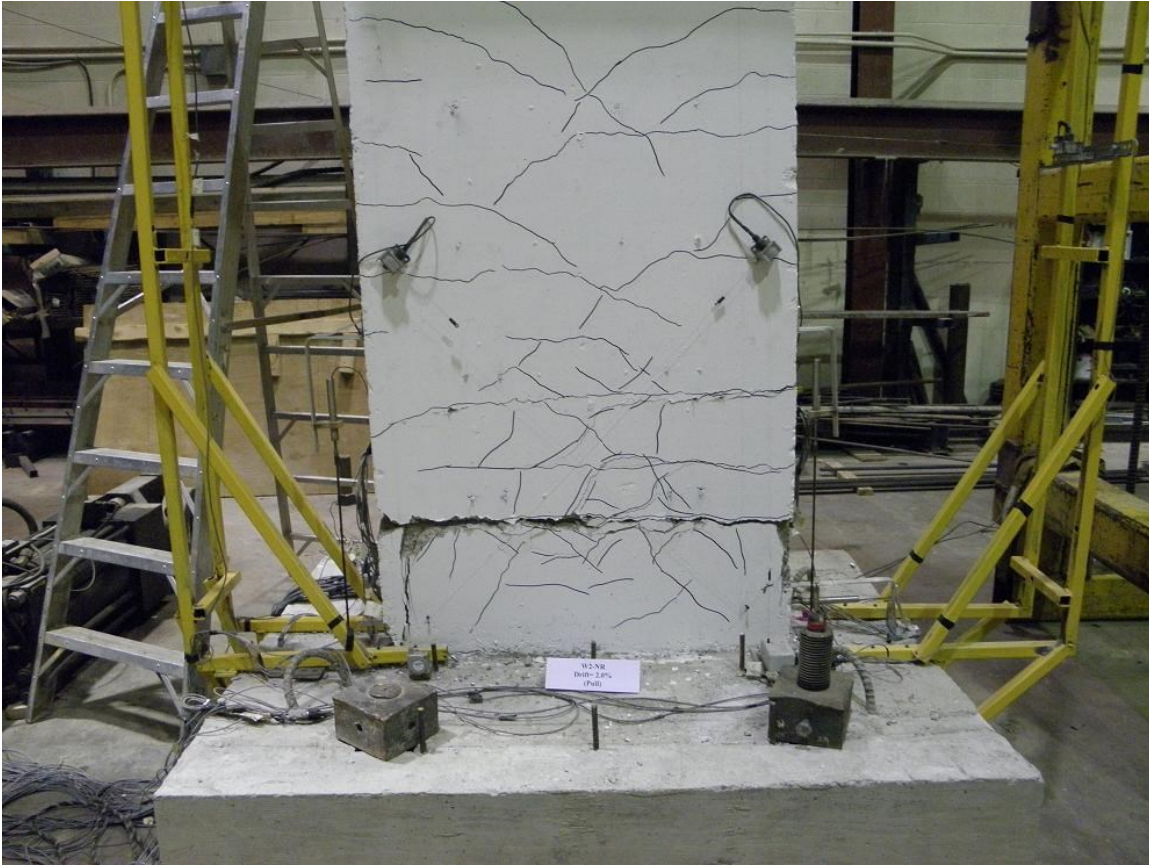
Beyond 30 mm of displacement, additional flexural cracking was limited to the extension of previously formed cracks, and new shear cracks continued to appear. At 36 mm of lateral displacement (1.5% drift), spalling of the concrete at the boundaries was observed near the base of W1-SR and W2-NR as shown in Figure 4.17.





(a) (b)  
**Figure 4.17** Crack Pattern of Test Walls at 1.5% Drift: (a) W1-SR; and (b) W2-NR

Loading of the conventional wall was terminated at 36 mm of displacement, corresponding to 1.5% drift, while the SMA wall was tested to failure as explained in Section 3.6.6. At a displacement level of 48 mm (2% drift), the flexural crack located at a height of 380 mm from the base of the SMA wall extended over the entire length of the wall. This major crack controlled the response of the wall up to failure. At this location, concrete crushing accompanied with SMA bar buckling was noted prior to failure (Figure 4.18). At failure, the wall experienced sliding along this crack. Note that this crack surfaced adjacent to the cut-off location of the starter bars.



**Figure 4.18** Crack Pattern of SMA Shear Wall at Failure

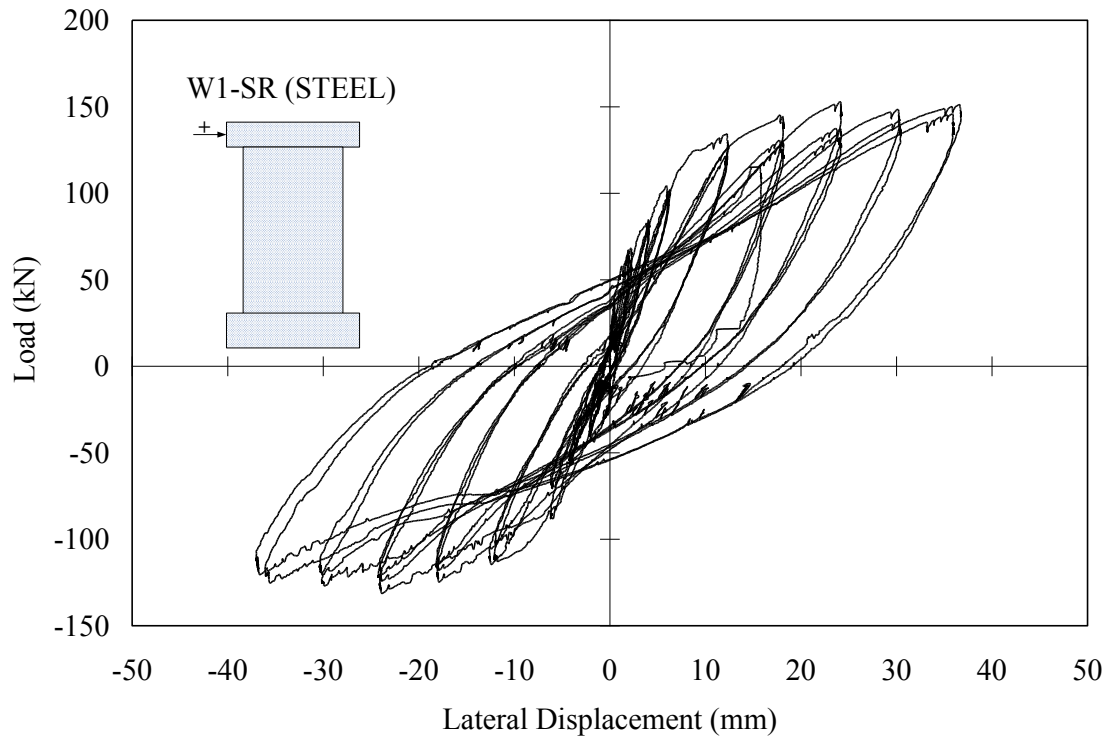
The recorded residual crack widths demonstrated the superior capacity of the SMA wall to recover inelastic displacements. At yielding, the maximum flexural crack widths, near the base of the wall, were approximately 0.6 mm and 2.5 mm for W1-SR and W2-NR, respectively. During the unloading phase, Wall W2-NR recovered most of the crack opening; the residual crack width and crack opening recovery were 0.45 mm and 25% for Wall W1-SR and 0.35 mm and 0.86% for Wall W2-NR. At a displacement level of 36 mm, the maximum and resulting residual crack widths of 3.0 mm and 2.4 mm, and 12.5 mm and 1.2 mm were noted in W1-SR and W2-NR, respectively. The average recovery capacity for the SMA wall was 88% throughout testing, whereas the average recovery

capacity for the conventional wall was 24%. Cracking characteristics of the conventional and SMA shear walls are included in Appendix E.

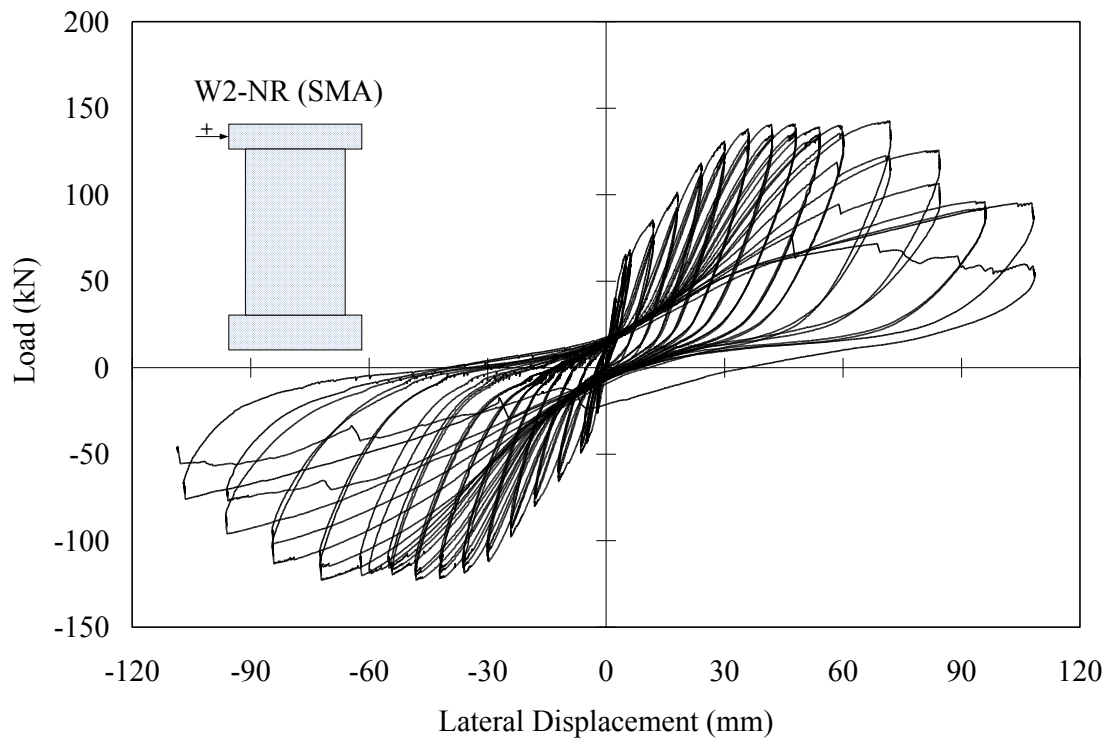
#### 4.4.2 Load-Displacement Response

Linear Variable Differential Transducers (LVDTs) and Displacement Cable Transducers (DCTs) were mounted at various locations on the walls to monitor lateral, vertical and out-of-plane displacements. Wall base slip and opening in addition to the foundation uplift and sliding were also traced.

The lateral displacement of the top beam was used to determine the hysteresis response of the conventional (W1-SR) and SMA walls (W2-NR) to study the initial stiffness, yield and ultimate load, and ultimate and residual displacements. The hysteretic response of W1-SR and W2-NR are shown in Figures 4.18 and 4.19, respectively.



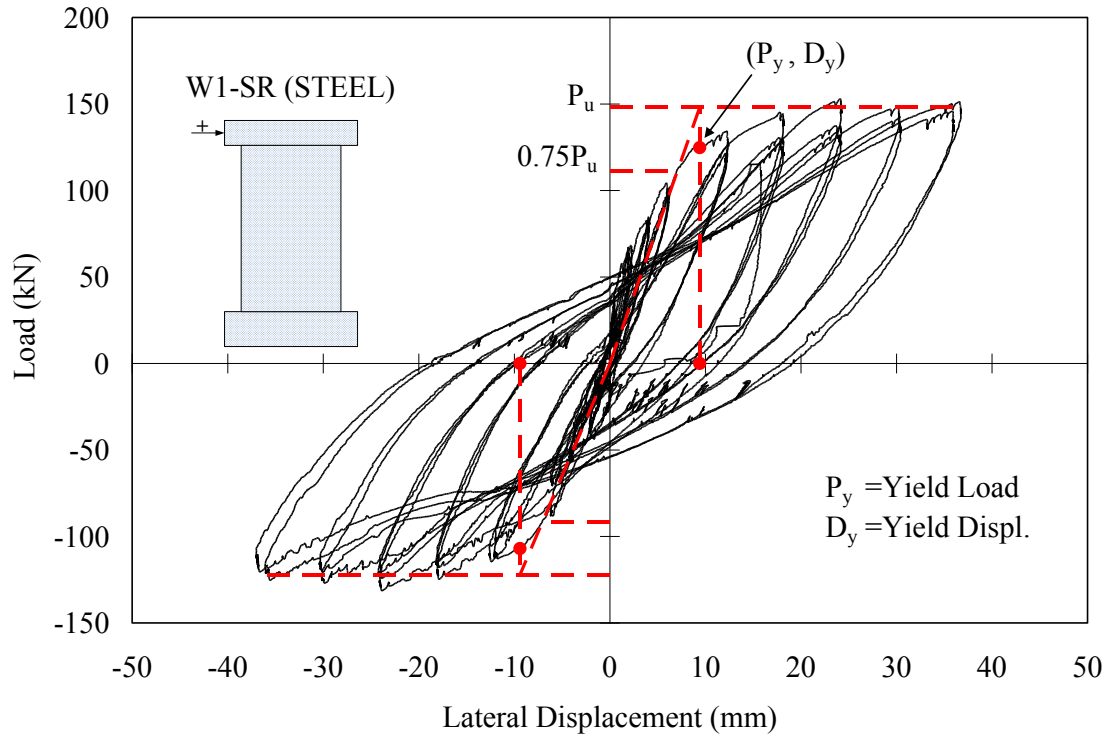
**Figure 4.19** Load-Displacement Response of Wall W1-SR



**Figure 4.20** Load-Displacement Response of Wall W2-NR

The hysteretic loops of the first two loading stages were similar for both walls and within the linear elastic range, illustrating the elastic behavior of the specimens prior to yielding. The stiffness of the walls degraded during the second and third repetitions of loading due to the cyclic loading effect. Beyond the first two loading stages, there was more softening in the response of W2-NR than W1-SR. This was due to the lower modulus of elasticity of SMA in comparison to the deformed steel. Furthermore, yielding of the web deformed steel bars occurred prior to yielding (forward transformation) of the SMA bars in W2-NR. The yield displacement of the conventional wall (W1-SR) was 9.4 mm (0.39% drift) corresponding to 116 kN of load, while the SMA wall (W2-NR) started yielding at 26.4 mm of displacement (1.1% drift) corresponding to 112 kN of load. The yield displacement 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 average yield displacements calculated from both directions of loading was adopted as shown in Figure 4.21.



**Figure 4.21** Calculation of Yield Displacement and Yield Load in W1-SR

The post-yielding stiffness of both walls was comparable and both specimens performed in a similar ductile manner. Testing of Wall W1-SR was terminated, prior to failure, after being subjected to a displacement level of 36 mm. At this point, a significant portion of the load-deformation response had been attained. The maximum load capacity sustained by the wall was 149 kN, corresponding to a displacement of 24 mm. Throughout testing of Wall W1-SR, the load capacity of subsequent loading stages was at least 95% of the maximum load capacity.



Wall W2-NR was tested to failure. The maximum load capacity of 140 kN was recorded at a corresponding displacement of 42 mm. The peak load of subsequent loading stages (from 48 mm to 72 mm) was at least 96% of the maximum load capacity. In the post-peak range, it was evident that significant plastic hinging occurred near the base of the wall. The wall maintained the lateral load capacity up to the second repetition of loading to a displacement level of 72 mm. At this point, the wall suffered a sudden degradation in the lateral load strength capacity (about 14%) due to rupturing of one of the longitudinal deformed steel bars adjacent to the SMA bars as shown in Figure 4.22. This bar was located in the first row of deformed bars in the web section of the wall.



**Figure 4.22** SMA Shear Wall Condition at a Displacement Level of 72 mm

Three additional drops in load capacity are evident in the positive direction of the load-displacement response: during the second repetition of loading to a displacement level of 84 mm; during the first repetition of loading to a displacement level of 94 mm; and during the last loading cycle. The drops at 84 mm and 94 mm were also associated with rupturing of the web vertical reinforcement. These fractures were observed along the major crack above the starter bars. Due to the closely spaced tie reinforcement in the boundary elements, the wall sustained the load carrying capacity until a displacement level of 108 mm at which point the wall failed. Concrete crushing and buckling of the SMA bars at the right edge were visible as shown in Figure 4.23.



**Figure 4.23** SMA Shear Wall Condition at Displacement Level of 108 mm (Failure)

At the end of testing, and after concrete was removed from the lower part of the wall, it became evident that one of the SMA bars had also fractured at the location of the mechanical coupler; a distance of approximately 900 mm above the base foundation. This phenomenon was responsible for the last drop in lateral load capacity. In the negative direction of loading, there was a drop in the lateral load capacity during the second repetition of loading to a displacement level of 94 mm, which was caused by fracturing of a web vertical reinforcing bar.

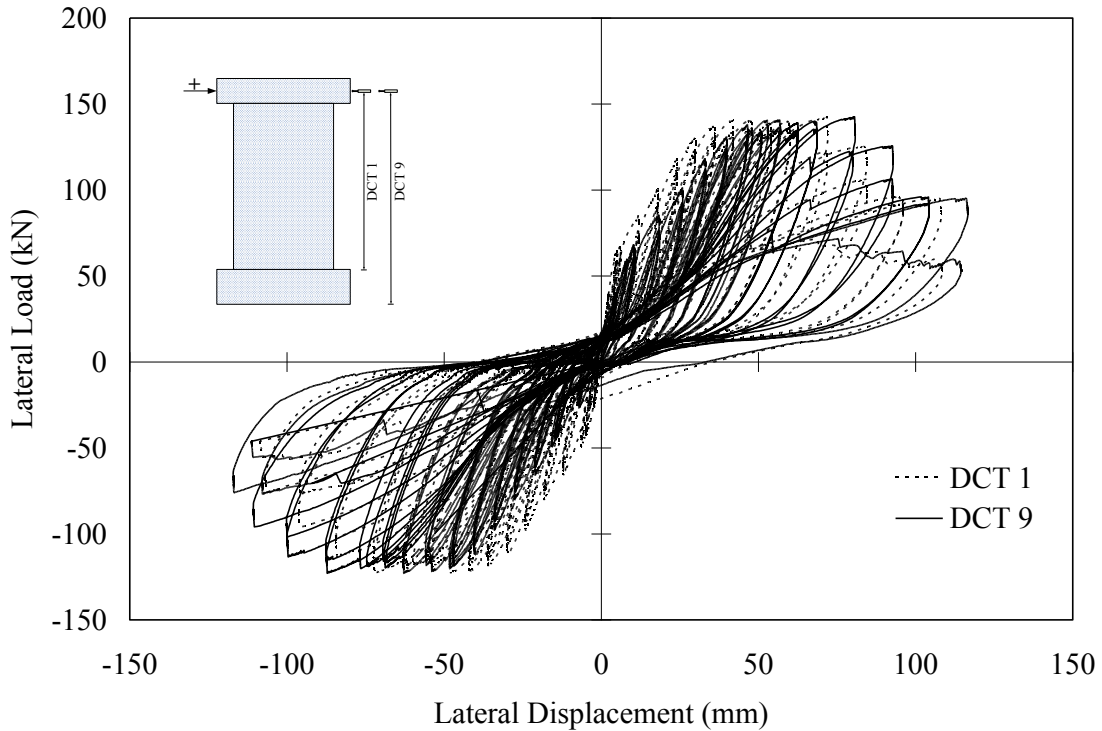
Both Wall W1-SR and Wall W2-NR responded with unloading stiffness similar to the initial loading stiffness for a significant part of the response. Beyond 60 mm of lateral displacement, the unloading stiffness in the SMA wall experienced degradation relative to the initial stiffness. Note that the longitudinal reinforcement in the web fractured beyond the 60 mm displacement level. The SMA wall experienced a rapid recovery of displacements when the SMA bars reached the reverse transformation stress. As a result, more pinching is evident in the response of Wall W2-NR than that of Wall W1-SR. The reloading curves of the SMA wall are significantly more nonlinear.

The ability of the SMA wall to recover displacements due to its strain recovery capacity was evident throughout the test. At a displacement level of 18 mm, the residual displacements of Wall W1-SR and Wall W2-NR were 7.2 mm and 0.48 mm, respectively. Furthermore, at 36 mm of lateral displacement, Wall W1-SR was capable of recovering only 18 mm, while Wall W2-NR recovered 30 mm upon unloading.

As discussed in Section 3.6.2, the maximum lateral displacement of the top beam was measured relative to the wall base and relative to the laboratory strong floor using two DCTs to track the possible rotation of the foundation during the test. The responses of

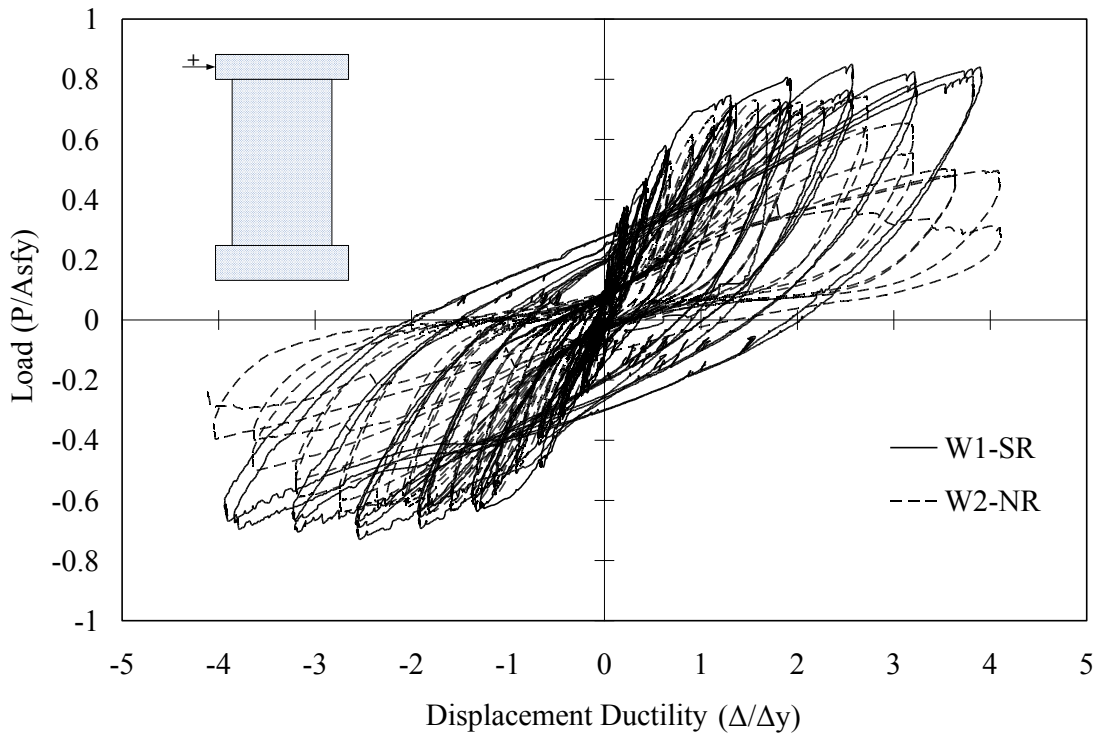


both DCTs, as shown in Figure 4.24, are similar, which indicate that the foundation experienced negligible uplift or rotation during the test.



**Figure 4.24** Load-Displacement Response of Wall W2-NR Relative to the Base (DCT1) and the Laboratory Strong Floor (DCT9)

To provide a better comparison of the strength capacity of the walls, the load-displacement responses were normalized to account for the difference in the area of longitudinal SMA reinforcement in the boundary region and the corresponding yield stress. Figure 4.25 shows the normalized response of both walls, where the midspan displacement is replaced by the displacement ductility to reflect differences in yield displacement.



**Figure 4.25** Normalized Load-Displacement Ductility Response of Shear Walls W1-SR and W2-NR

The normalized load-displacement ductility responses of the Nitinol and conventional walls indicate that the SMA reinforced wall W2-NR has lower initial stiffness than the conventional wall W1-SR; however, both walls experienced comparable normalized yield load and normalized ultimate load.

Table 4.3 provides the yield and maximum load capacities for the SMA and conventional walls. Testing of Wall W1-SR was terminated prior to failure to permit future testing; therefore, the ultimate ductility was not realized. Note that the ductility was based on the ultimate displacement which was defined to be the displacement at which the load capacity of the wall dropped below 80% of the maximum load capacity or the displacement corresponding to fracturing of the reinforcement (Park 1980). The ultimate

displacements calculated from both directions of loading were used to determine the average ductility.

**Table 4.3** Load and Displacement Ductility Capacities

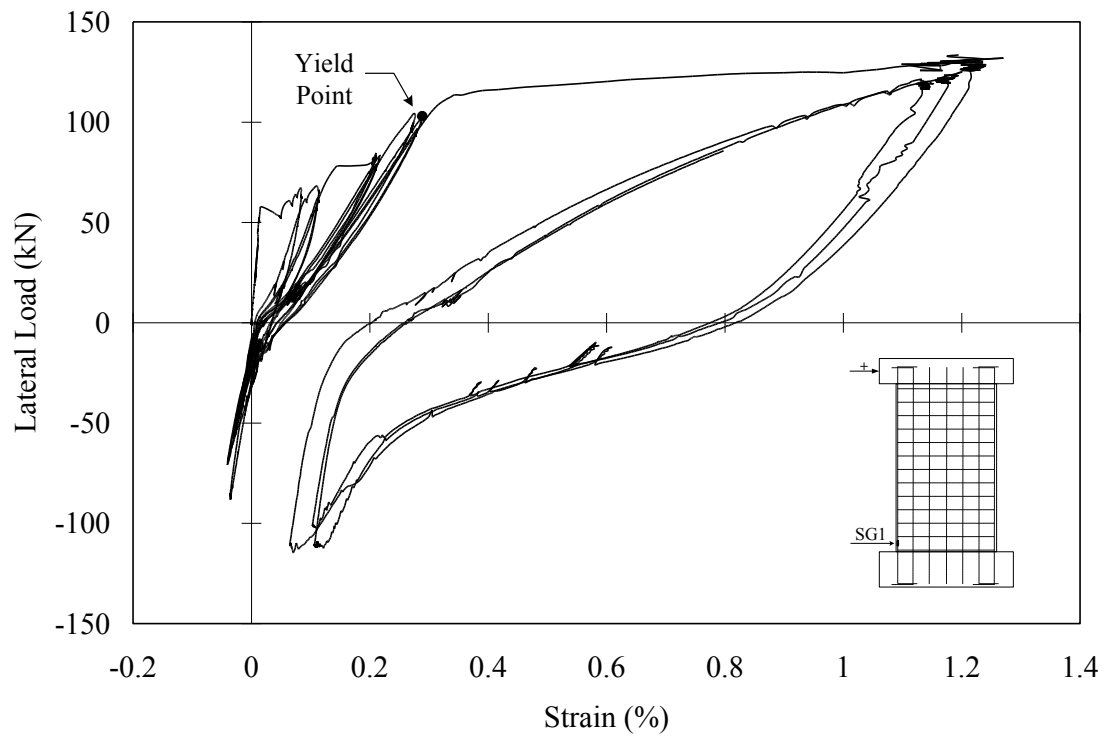
| Wall         | Yield Load  |            | Peak Load   |            | Ductility |
|--------------|-------------|------------|-------------|------------|-----------|
|              | Actual (kN) | Normalized | Actual (kN) | Normalized |           |
| <b>W1-SR</b> | 116         | 0.69       | 149         | 0.82       | >4        |
| <b>W2-NR</b> | 112         | 0.62       | 140         | 0.73       | 2.76      |

The load-displacement responses recorded by the other DCTs and LVDTs are available in Appendix D. The vertical displacement at the wall boundaries, foundation base slip, and associated phenomenological wall behaviors including wall and foundation rotation, twisting of the top beam and plastic hinge development are discussed in Chapter 5.

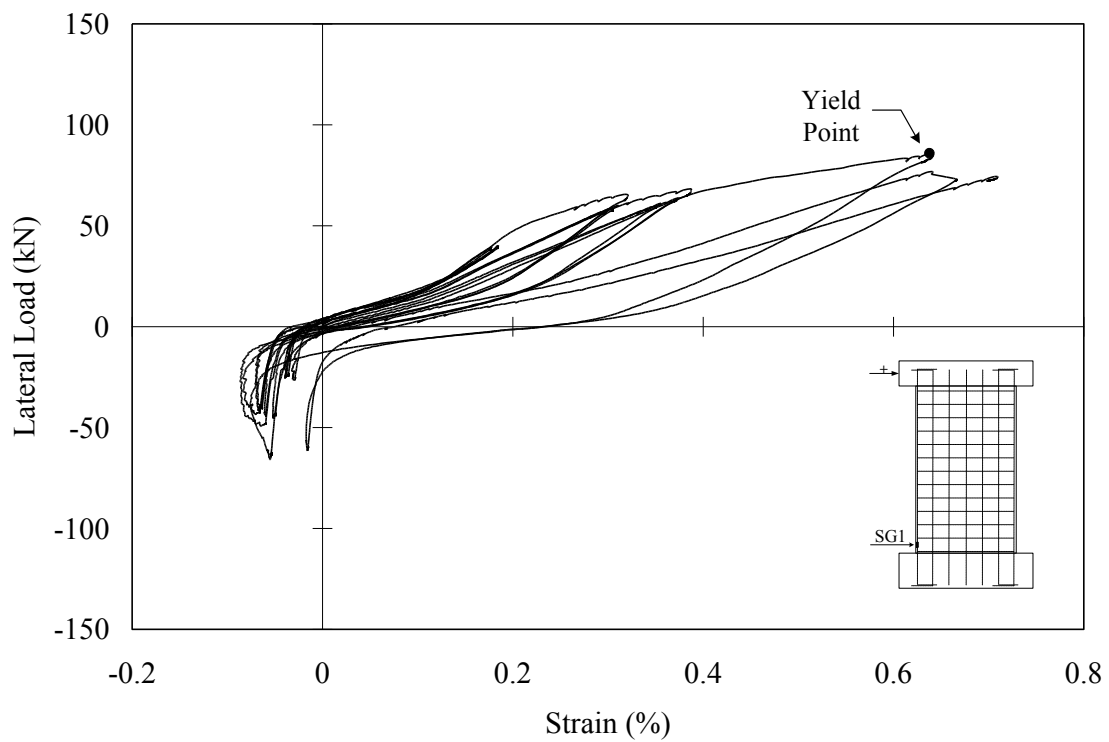
#### 4.4.3 Reinforcement Strains

Strain gauges were attached to the reinforcing bars at different locations, as shown in Figure 3.21, to measure the strain in the longitudinal and transverse reinforcement. The strain gauges located on the longitudinal reinforcement near the base of the wall failed after the specimen experienced large displacements, while other gauges were capable of recording to the end of the test. From the recorded data, load-strain responses were constructed to study the behavioral features such as initial elastic stiffness, yielding of the rebar, hysteretic behavior, dissipated energy, and permanent straining after each displacement cycle.

Figures 4.26 and 4.27 show the measured strains of Strain Gauge SG1 located on the main boundary longitudinal reinforcement in the plastic hinge region (near the base of the wall) in the conventional wall W1-SR and SMA wall W2-NR, respectively.



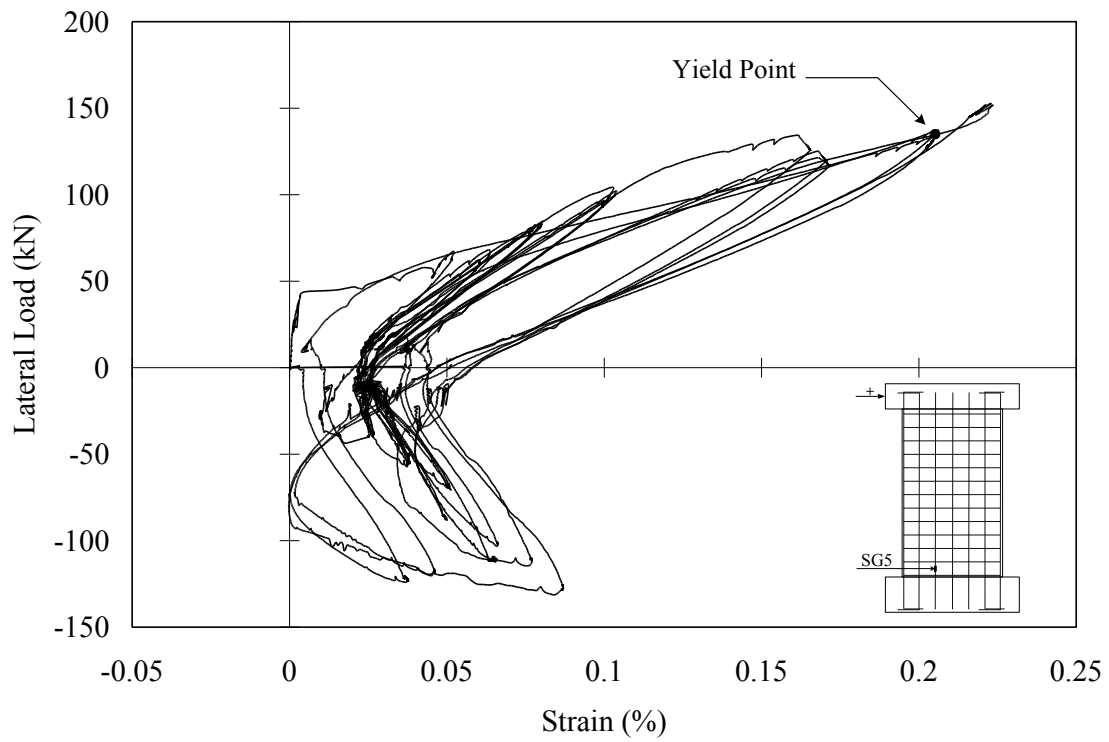
**Figure 4.26** Load-Strain Response of Strain Gauge SG1 in Wall W1-SR



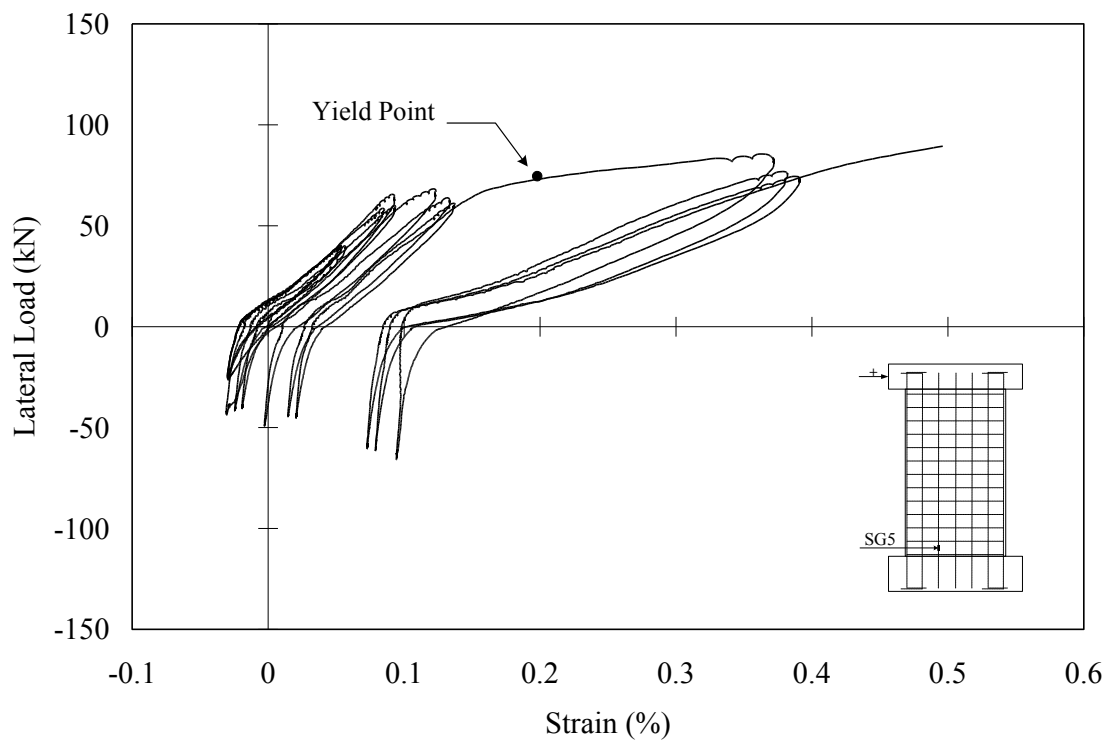
**Figure 4.27** Load-Strain Response of Strain Gauge SG1 in Wall W2-NR

The first crack surfaced in both walls at relatively the same applied load. At this point, the maximum measured strains in the boundary longitudinal reinforcement near the base were 0.05% and 0.18% in W1-SR and W2-NR, respectively. A sudden jump was evident in strain during the next loading cycle due to this flexural cracking and the transfer of load from the concrete to the reinforcement. The test data demonstrated the low stiffness of SMA in W2-NR, which lead to higher yield strain; about three times the corresponding yield strain of the steel in W1-SR. At this strain level, the applied load was 98 kN in W1-SR while it was only 84 kN in W2-NR. Note that, although Wall W1-SR was subjected to a lower maximum drift than Wall W2-NR, the strains were higher in Wall W1-SR.

The straining of the web longitudinal steel bars (strain gauge SG5) outside the boundary region of Wall W2-NR indicated that the steel bars yielded (near the base of the wall) at a load of 64 kN and prior to yielding of the adjacent SMA bars. This behavior supports the response of the wall where the web longitudinal reinforcement fractured, while the SMA bars remained intact. The web bars reached their strain capacity along the predominant flexural crack, while the SMA bars had reserve strain capacity. For Wall W1-SR, the web longitudinal bars near the boundary region yielded at a load of 128 kN, but after the longitudinal bars in the boundary region yielded. The responses for W1-SR and W2-NR are shown in Figures 4.28 and 4.29, respectively.

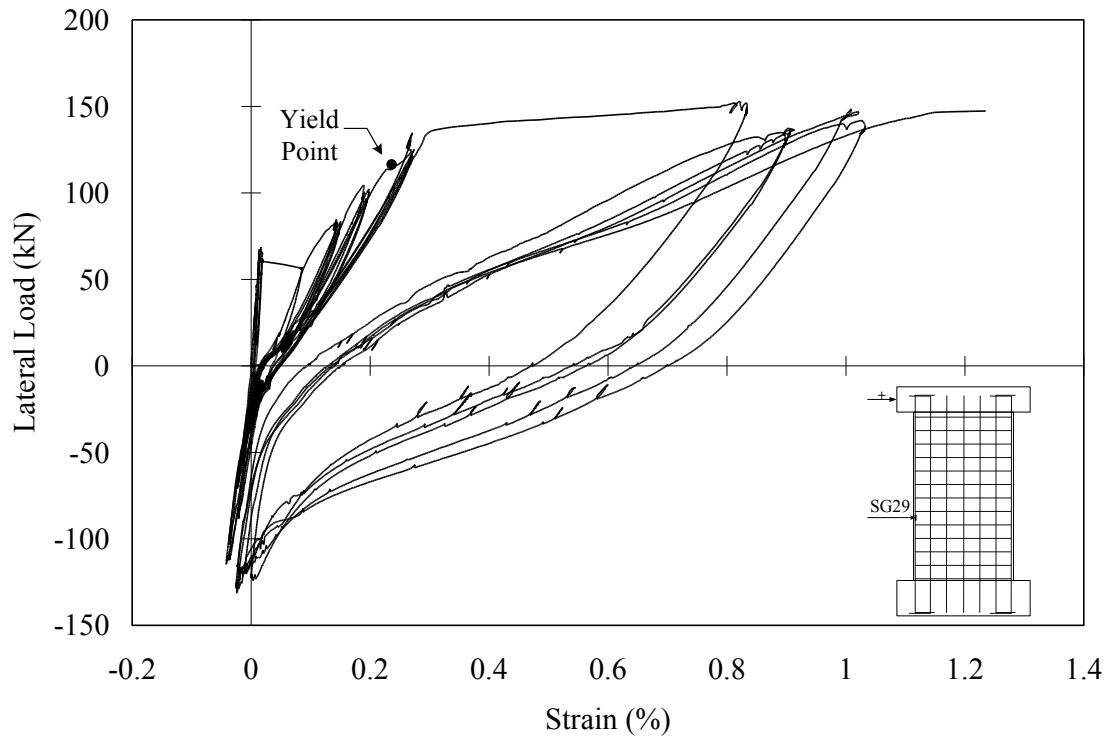


**Figure 4.28** Load-Strain Response of Strain Gauge SG5 in Wall W1-SR

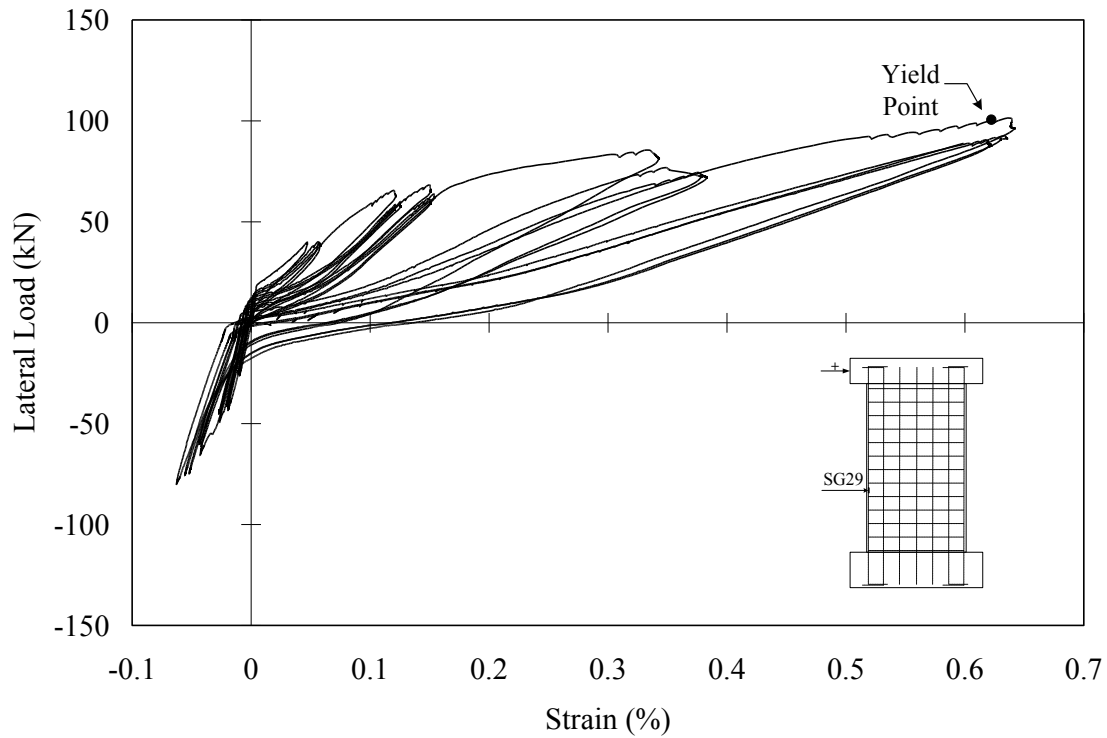


**Figure 4.29** Load-Strain Response of Strain Gauge SG5 in Wall W2-NR

With increased loading, tensile yielding spread along the longitudinal reinforcement in the boundaries. Figure 4.30 shows the strains recorded by strain gauge SG29, which was located on the exterior longitudinal bar at a height of 800 mm from the wall base of W1-SR. Yielding at this location initiated at a load level of 118 kN. In W2-NR, the same strain gauge recorded yielding in the longitudinal SMA bar at a load level of 102 kN (Figure 4.31). These results demonstrate that yielding (plastic hinging) spread to a height of at least 800 mm from the base of the wall for both walls.



**Figure 4.30** Load-Strain Response of Strain Gauge SG29 in Wall W1-SR



**Figure 4.31** Load-Strain Response of Strain Gauge SG29 in Wall W2-NR

The re-centering capability of the superelastic reinforcement upon unloading was evident in the response of strain gauge SG29 in W2-NR. After unloading from 0.3% strain, the residual strain in W1-SR was 0.05%, whereas the residual strain in W2-NR was 0.02% (Figures 4.30 and 4.31). At the highest recorded strain of 0.8% in W1-SR, the recovered strain was only 0.37%, while the SMA wall W2-NR recovered approximately 84% of the maximum imposed strain of 0.66%.



## CHAPTER 5

### DISCUSSION OF EXPERIMENTAL RESULTS

#### 5.1 Introduction

This chapter discusses the experimental results with regards to the contribution of superelastic Nitinol SMAs to the overall behavior of the beams and shear walls tested as part of this research study. The stiffness, strength, ductility, recovery, and failure mode of SMA reinforced beams and shear walls are discussed relative to the conventional reinforced specimens. In addition, wall rotation, plastic hinge development, base slip and rotation, and top beam twisting of the shear walls are discussed.

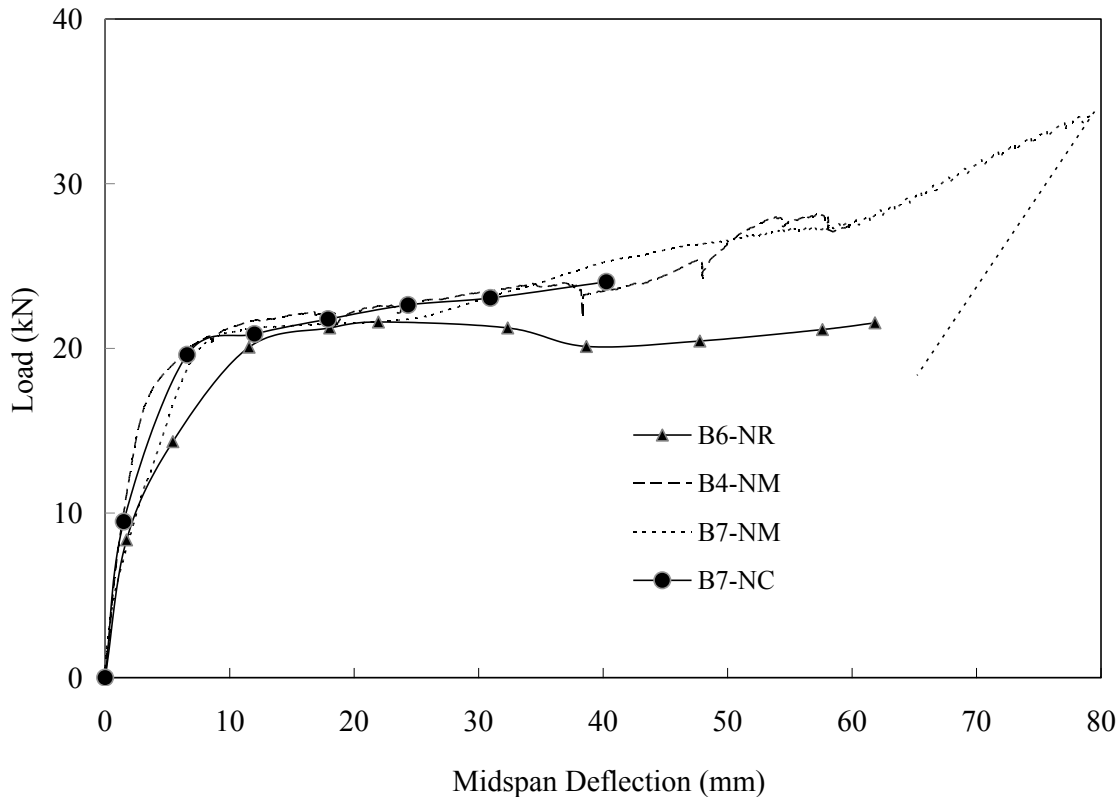
#### 5.2 Beam Tests

##### 5.2.1 Stiffness and Strength

Both SMA and conventional beams were tested using the same loading protocol where multiples of  $\Delta y$  were imposed. Differences in the reinforcement ratio and yield strength prevent direct comparison, thus the responses were normalized to reflect differences in reinforcement area and yield strength. The initial pre-yielding stiffness was lower in the SMA specimens. This was due to the lower modulus of elasticity in the SMA bars. Consequently, higher yield deflections were experienced in the SMA specimens. For NiTi, the modulus of elasticity can range from 1/6 to 1/2 of the modulus of steel reinforcement. The ratio of the modulus of elasticity of the SMA bars used in this study to typical reinforcing steel was approximately 1/4. The lower reinforcement modulus led to a lower cracked stiffness for the SMA beams. The post-yield response and stiffness are

similar. However, the SMA beams experienced larger normalized load capacity at higher ductility levels (refer to Figure 4.5).

The SMA beams were subjected to monotonic, cyclic, and reverse cyclic loading. Figure 5.1 provides the envelope load-midspan displacement responses for the SMA beam tests. The envelopes were developed from the peak loads and corresponding displacements for each load stage of the load-displacement response for the cyclic and reverse cyclic tests.



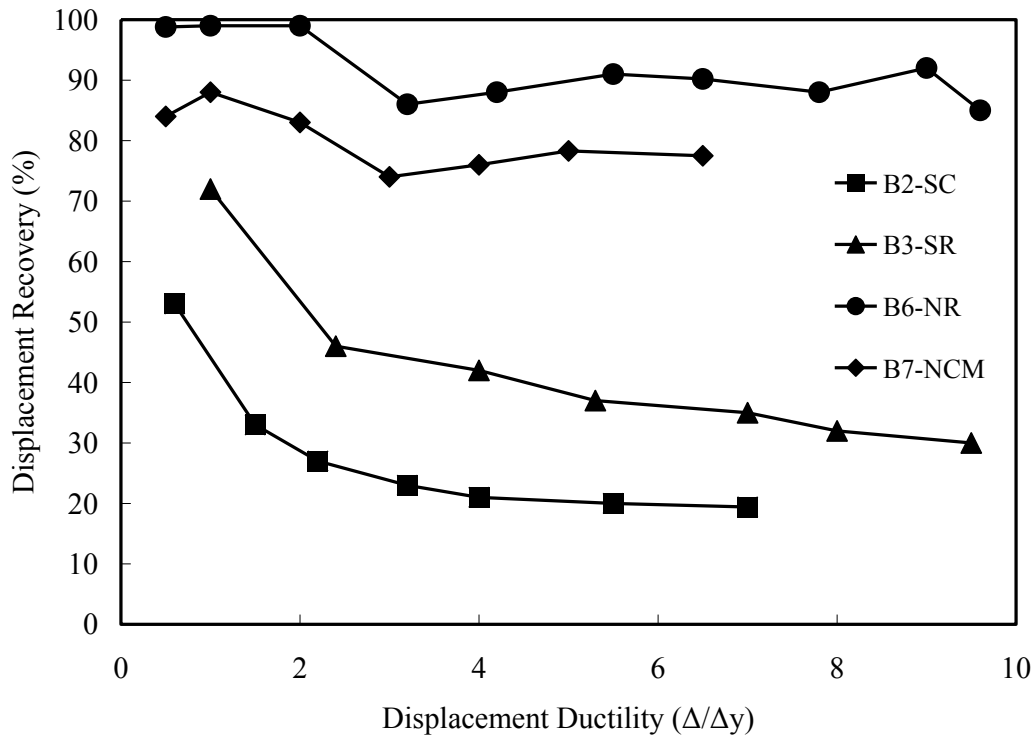
**Figure 5.1** Envelope Load-Displacement Responses of SMA Beam Tests

Up to 10 kN of load, the initial stiffness was similar for all beams. Beyond 10 kN of load, the cyclic loads caused a reduction in the stiffness of the beams. The post-yielding response of B4-NM and monotonic and cyclic responses of B7-NCM were similar. [Note that the monotonic and cyclic responses of B7-NCM, in Figure 5.1, are denoted B7-NM

and B7-NC, respectively.] The stiffness of B6-NR was significantly reduced beyond 10 kN, and a significant degradation in strength was also recorded beyond 30 mm of midspan displacement. The former can be partly attributed to the damage caused by the reversing loads. The strength degradation was a result of the Nitinol bar fracturing outside the critical section. This caused a reduction in strength capacity and displacement ductility. In contrast, Beam B7-NCM, under monotonic loading, experienced the highest displacement and strength capacities as a result of the Nitinol bar fracturing in the critical section. The envelope responses suggest that threaded bar ends and the location of the transition zones (change in bar diameter) can be problematic. Although the coupling was located outside the critical region, future investigations should consider the extent of plastic hinging involving smooth Nitinol bars to ensure that couplers are used outside this region. Improved behavior of the SMA beams would be possible with adjustments to the location of the coupling system and transition zone.

#### 5.2.2 Ductility and Recovery

The normalized load-displacement ductility responses (Figures 4.6 and 4.7), and the recorded residual crack widths (Table 4.1) demonstrated the recovery characteristics of the SMA beams. The capability of SMA reinforced beams to recover displacement is further demonstrated in Figure 5.2, which provides the midspan displacement recovery capacities for the beams under cyclic and reversed cyclic loading.



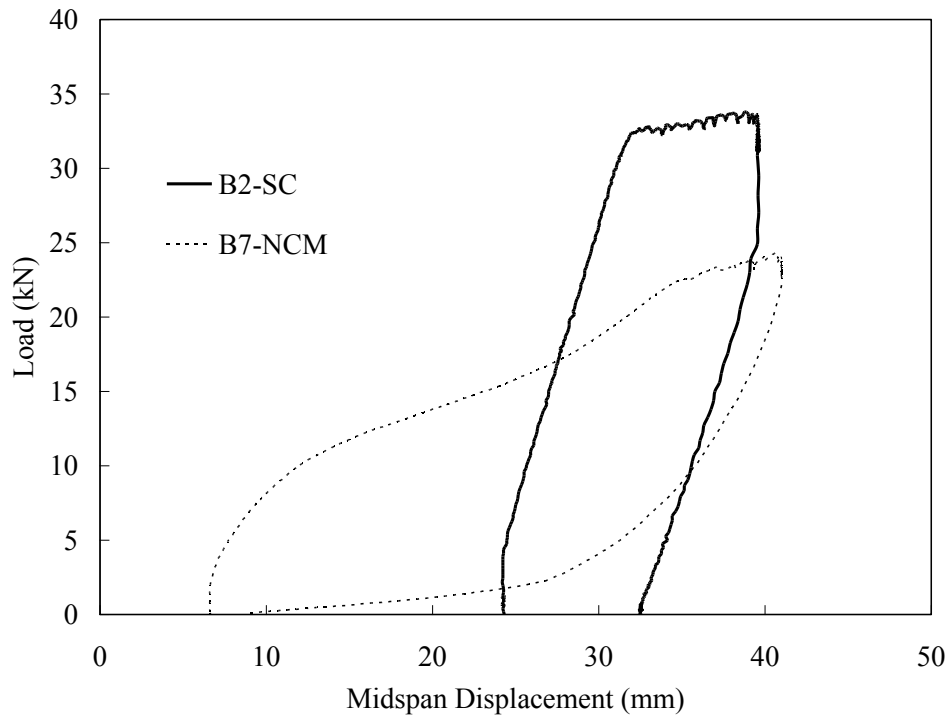
**Figure 5.2** Recovery Capacity-Displacement Ductility Behavior

The SMA beam subjected to reverse cyclic loading demonstrated higher recovery capacity relative to the cyclically loaded SMA beam. The conventional reinforced beams demonstrated similar behavior. This difference was associated with a reduction of stiffness in the members due to damage experienced in the concrete under load reversals, which resulted in increased pinching relative to the cyclically loaded beams. The conventional reinforced beams experienced a significant reduction in the capacity to recover inelastic deformation in comparison to the SMA reinforced beams. This was a direct result of the response of the deformed reinforcement used in the conventional beams; the unloading stiffness was linear and approximately equal to the initial elastic modulus, thus resulting in significant residual deformations. Conversely, the reverse transformation that occurs in Nitinol bars during unloading triggers significant recovery

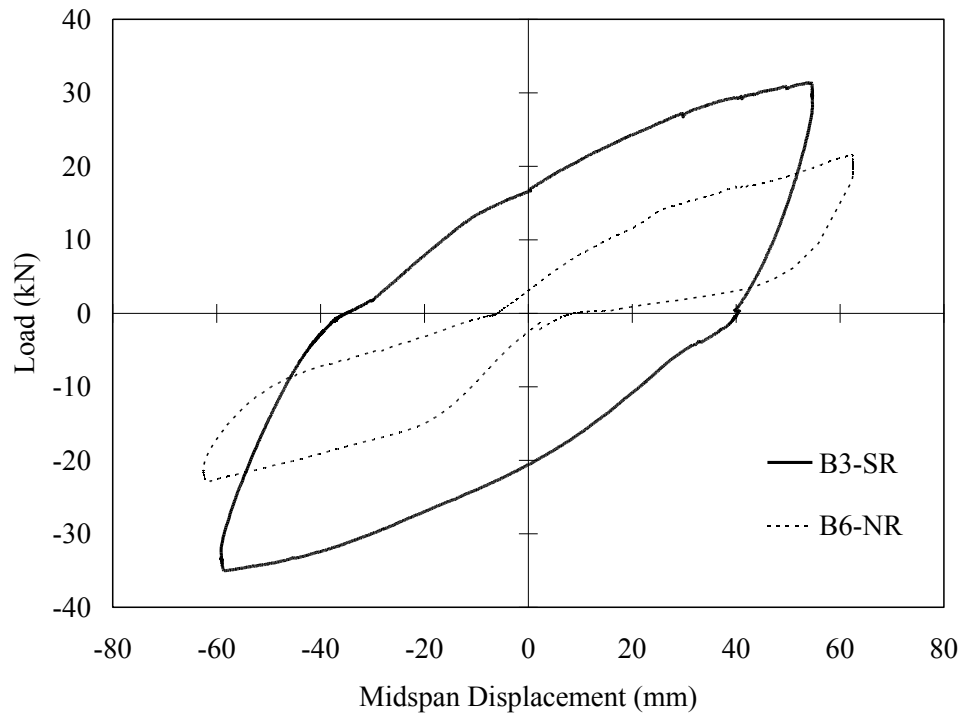
of deformations. Under reverse cyclic loading, and at a displacement ductility of approximately 9.5, the SMA beam recovered 85% of the midspan displacement experienced during loading, while the conventional reinforced beam only achieved a recovery of 26%. Under cyclic loading, the SMA and conventional reinforced beams, recovered 80% and 17%, respectively, at a displacement ductility of 6.5. Figure 5.2 highlights the progressive reduction in the re-centering capability of the conventional reinforced beams; a consequence of the linear unloading behavior of the reinforcement, which resulted in accumulation of permanent strains. The SMA beams, on the other hand, maintained a recovery capacity of approximately 75% to 100% for the ductility demands imposed during testing.

### 5.2.3 Strain Energy

Energy dissipation capacity is a salient feature of structural response to dynamic loading. Figures 5.3 and 5.4 provide the load-displacement responses of the beams subjected to cyclic and reverse cyclic loading, respectively, at ultimate displacement.

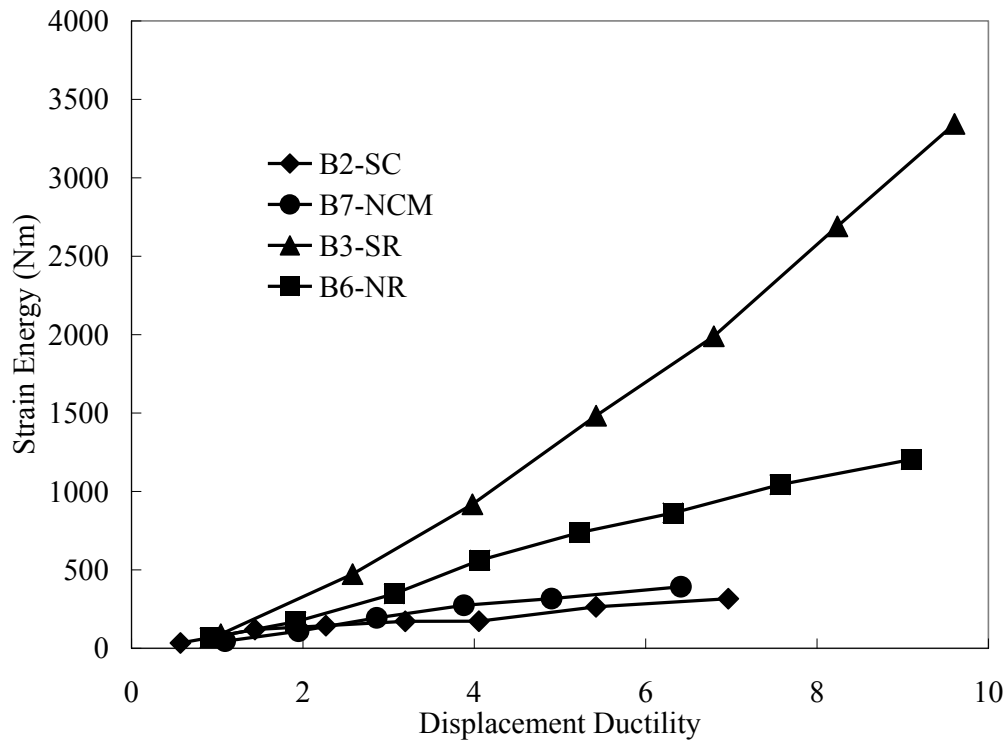


**Figure 5.3** Hysteretic Response at Ultimate Displacement of Beams under Cyclic Loading



**Figure 5.4** Hysteretic Response at Ultimate Displacement of Beams under Reverse Cyclic Loading

The SMA beam was capable of dissipating energy (area under the load-displacement response) comparable to the conventional beam under cyclic loading, while the conventional beam experienced higher energy dissipation under reverse cyclic loading. The reduced energy dissipation in the reverse cyclic SMA beam was a result of the reverse transformation during unloading, which caused pinching as the member re-centered. To a lesser extent, the tensile force capacities of the SMA and conventional reinforcing bars contributed to differences in the energy dissipation. Furthermore, B6-NR failed by rupturing of the reinforcement at the transition zone, which resulted in reductions in strength and energy dissipation capacities. Figure 5.5 provides the energy dissipation experienced by the beams subjected to cyclic and reverse cyclic loading at each displacement ductility level.



**Figure 5.5** Energy Dissipation-Displacement Ductility Responses

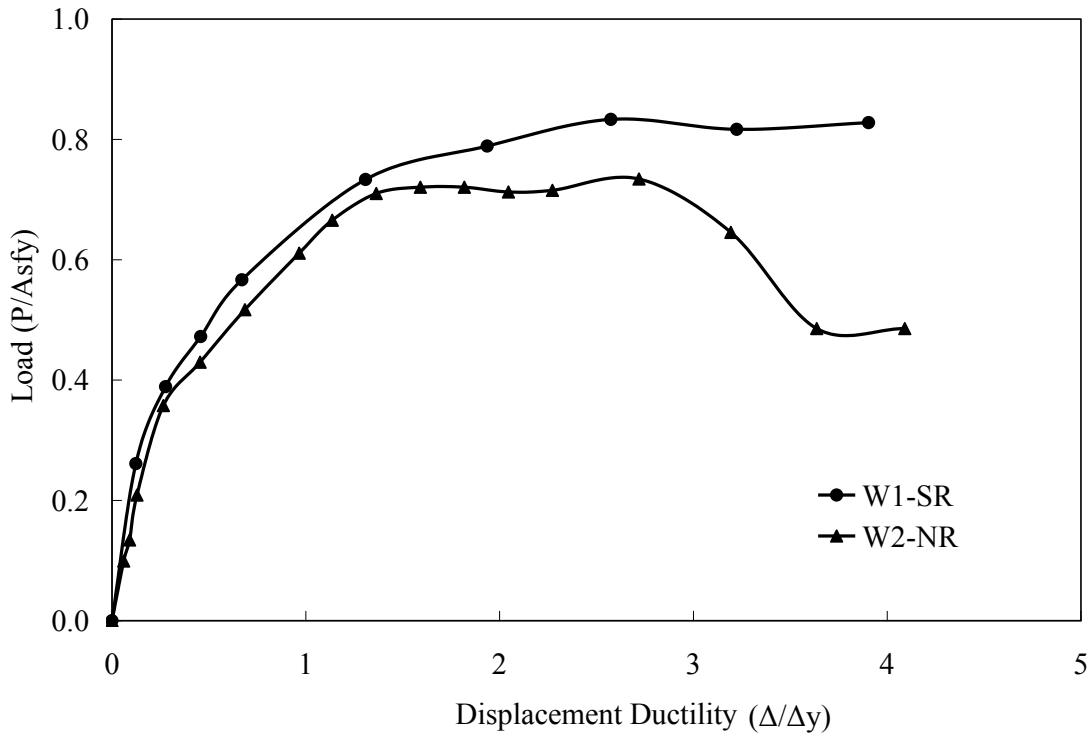
The energy was calculated from the area under the actual load versus midspan displacement response. In general, the SMA beam subjected to cyclic loading dissipated energy comparable to the conventional reinforced beam; whereas, the conventional reinforced beam dissipated significantly more energy than the SMA beam under reverse cyclic loading. At the ultimate displacement ductility under reverse cyclic loading, 3345 Nm and 1205 Nm of energy were dissipated by the conventional and SMA reinforced beams, respectively. Thus, the SMA beam dissipated approximately 36% of the energy dissipated by the conventional beam. Accounting for differences in yield displacement (ductility) and tensile yield force of the reinforcing bars, the SMA beam dissipated 46% of the energy dissipated by the conventional beam. If consideration is given to the location of failure in B6-NR at the transition zone, the difference in energy dissipation would further be reduced. The ultimate ductility for the beams under reverse cyclic loading was approximately 9.5. Referring to Figure 4.5, the difference in SMA to conventional beam strength capacity was 1.165 at a displacement ductility of 9.5. This is based on the response provided by Beam B7-NCM, which failed by rupturing of the Nitinol bar in the critical section and is the best representation of the capacity of the SMA reinforced beams. Therefore, the energy dissipated by the SMA beam under reversed cyclic loading could reasonably be 54% of that dissipated by the conventional beam. This assumes that the monotonic response is identical to the envelope of the reverse cyclic response if the SMA reinforced beam is controlled by failure in the critical section.



## 5.3 Shear Wall Tests

### 5.3.1 Stiffness and Strength

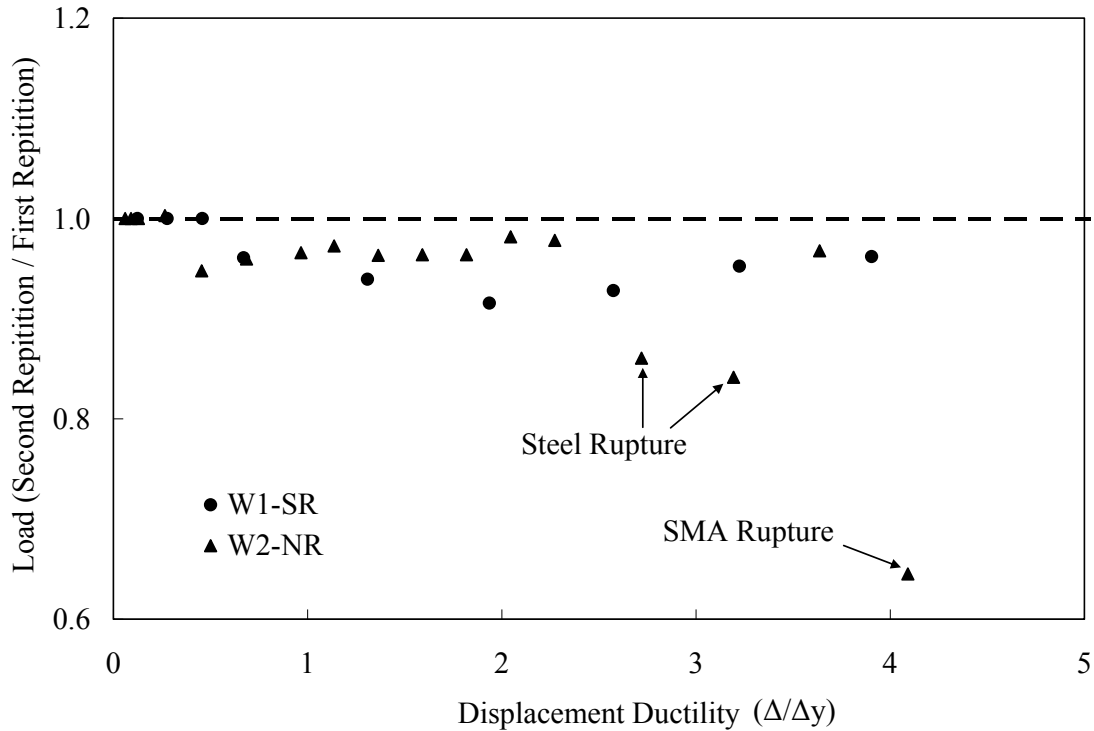
The hysteretic responses of the two wall specimens, conventional reinforced Wall W1-SR and SMA reinforced Wall W2-NR, were compared, specifically the initial stiffness, stiffness degradation, yield strength, and ultimate strength to evaluate the effectiveness of replacing the conventional steel by SMA bars in the boundary zones. The hysteretic responses of W1-SR and W2-NR were first normalized (Figure 4.24) to account for differences in the reinforcement ratio and yield strength. Then, the envelope responses for both walls were traced; peak load and corresponding displacement of the first cycle for each displacement level (Figure 5.6). The responses indicate that the SMA specimen had a lower initial pre-yielding stiffness and, consequently, higher yield deflection. Initial stiffness of the SMA wall was approximately 80% of the conventional wall, which reflects the lower modulus of elasticity of the SMA bars. Both walls exhibited ductile responses. However, the conventional wall experienced larger normalized yield and maximum load capacity. W2-NR was able to attain approximately 94% of the yield load and 93% of the maximum load resisted by W1-SR.



**Figure 5.6** Envelope Load-Displacement Ductility Responses of Wall Tests

In addition to the lower post-cracking stiffness, the SMA wall demonstrated a reduced stiffness relative to the conventional wall in the post-yielding region up to the peak load. This can be attributed to the condition of the deformed steel in the web of both walls at the yield load. In W2-NR, the web steel reinforcement yielded before the SMA bars, which reduced the wall stiffness at this stage. Degradation of strength in W2-NR initiated at a displacement ductility of 2.76 due to rupturing of one of the longitudinal deformed steel bars in the first row in the web. This resulted in approximately 14% drop in strength of W2-NR. At a displacement ductility of 3.25, a further 17% of strength degradation was recorded due to rupturing of another deformed steel bar in the second row in the web. To study the effect of cycling on the degradation in strength, the envelope response of the

second repetition of loading was assessed against the first repetition as shown in Figure 5.7.



**Figure 5.7** Effect of Cycling on Degradation of Strength in Wall Tests

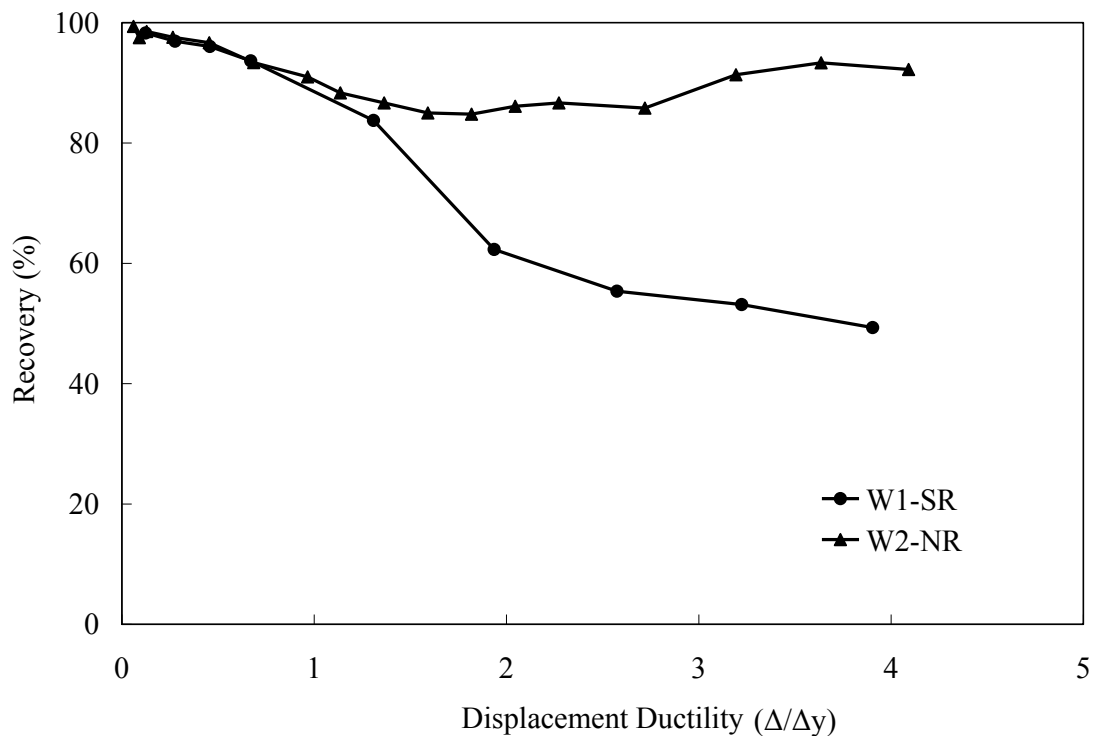
The results indicate that the second repetition of loading caused a marginal post-yielding degradation in lateral load capacity of approximately 6% and 3% in W1-SR and W2-NR, respectively, up to a ductility level of 2. The substantial drop in strength in W2-NR at higher ductility levels was mainly due to the rupturing of conventional deformed steel in the web and SMA bar in the boundary zone as previously discussed.

### 5.3.2 Ductility and Recovery

The normalized load-displacement ductility responses (Figure 4.24) demonstrated the ability of both specimens to sustain large inelastic deformations without suffering a

significant loss in load carrying capacity. At peak load, the displacement ductility of W1-SR and W2-NR was 2.55 and 1.6, respectively. Wall W1-SR sustained a displacement ductility of 4, while Wall W2-NR experienced a ductility of 2.76 without degradation of strength capacity. Note that the degradation in the SMA wall was due to rupturing of the vertical reinforcement in the web, while the main vertical reinforcement (SMA bars) remained intact up to a displacement ductility of 4.15.

The recovery capacity under cyclic loading, which is one of the main motivations for using SMA bars, is further demonstrated in Figure 5.8. The differences in the unloading response between SMA and conventional reinforcement due to the reverse transformation in the SMA bars, as shown in Figure 4.24, triggered the significant recovery of displacements in the SMA wall.



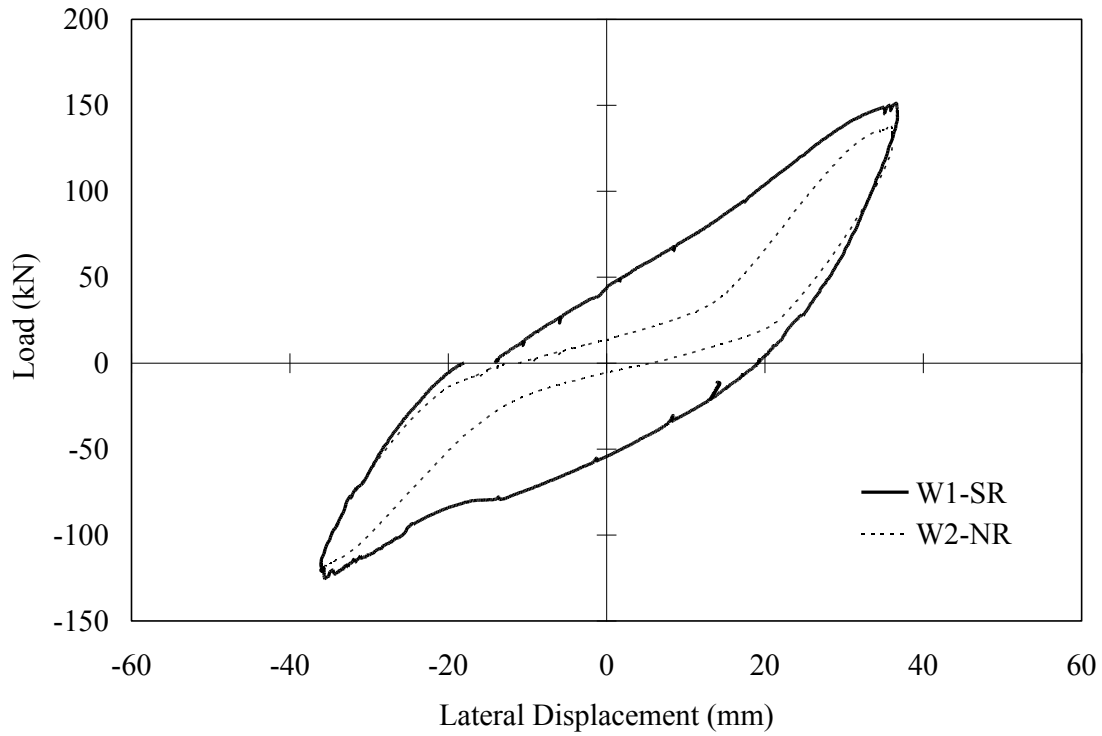
**Figure 5.8** Displacement Recovery Capacity-Displacement Ductility Behavior

At a displacement ductility of approximately 4, the SMA wall recovered 91% of the lateral displacement experienced during loading, while the conventional wall only achieved a recovery of 49%. The recovery capacity of the SMA wall throughout testing ranged from approximately 85% to 98%. Conversely, the linear unloading behavior of the conventional reinforcement led to a progressive reduction in the re-centering capability of the conventional wall.

The performance of the hybrid reinforcement layout across the length of the wall at the base (SMA bars in the boundaries and conventional steel in the web) and its effect in shear walls is one of the main objectives of this study. It is evident from Figure 5.8 that the SMA in W2-NR was able to overcome the resistance of the conventional steel to re-centering. Although the conventional steel in W2-NR yielded prior to the SMA bars, the wall still achieved significant recovery in comparison with W1-SR. Moreover, when the SMA wall was subjected to displacement ductility ranging from 2.7 to 4, the recovery capacity increased from 85% to 93%. This was a direct result of the deformed steel rupturing in the web, which enabled the SMA to restore the wall more effectively.

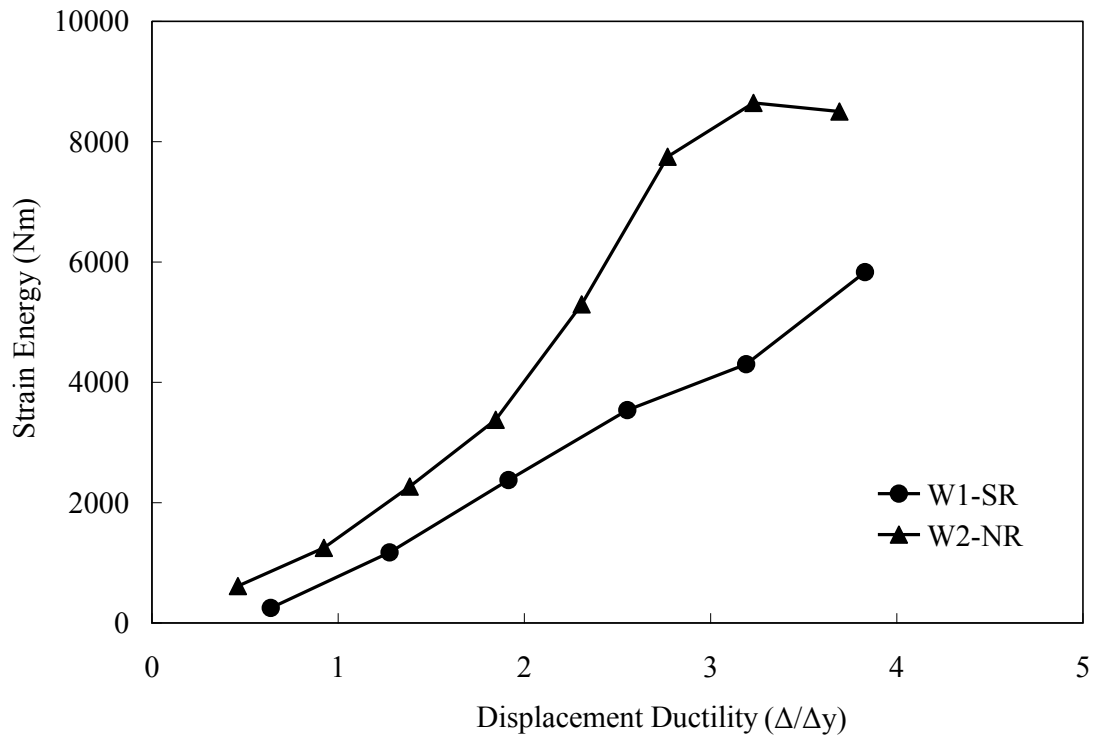
### 5.3.3 Strain Energy

Figure 5.9 presents the load-displacement response of the conventional wall at the end of the test and the corresponding response of the SMA wall at the same displacement level.



**Figure 5.9** Hysteretic Response at Ultimate Displacement of W1-SR

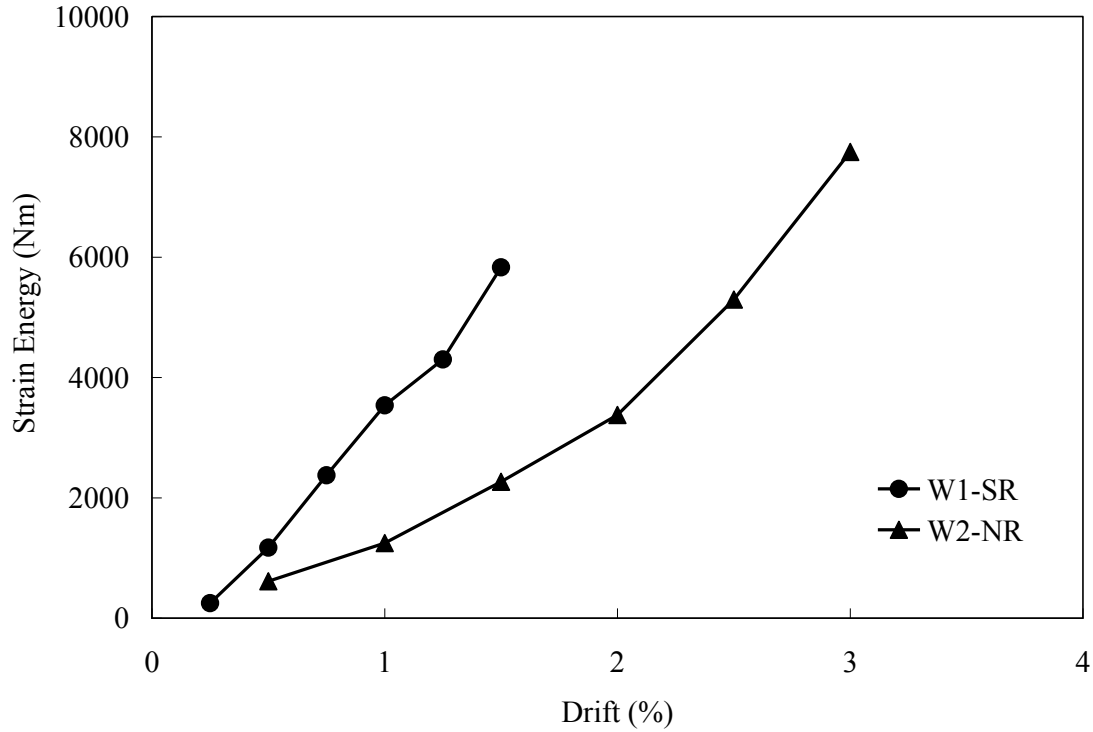
At the ultimate displacement of W1-SR (36 mm), the SMA wall was capable of dissipating 2309 Nm which represented 40% of the energy dissipated by the conventional wall. This was a consequence of the reverse transformation during unloading, which caused pinching in the response of the SMA wall. This difference in energy dissipation capacity can be slightly reduced by accounting for differences in the yield displacement and the tensile force capacities of the SMA and conventional reinforcing bars. Figure 5.10 provides the energy dissipation experienced by the conventional and SMA walls during the first repetition of loading at each displacement ductility level.



**Figure 5.10** Energy Dissipation-Displacement Ductility Responses

In general, the SMA wall dissipated strain energy higher than the conventional wall with an average margin of 620 Nm up to displacement ductility of 2. Thereafter, there was an obvious increase in the energy dissipated by the SMA wall compared to the conventional wall. At the maximum displacement ductility in W1-SR, 5831 Nm and 8500 Nm of energy were dissipated by the conventional and SMA walls, respectively, an increase of 45% for the SMA wall. The latter is, in part, related to the difference in yield displacement between the conventional and SMA walls and the noticeable plastic deformation and reverse transformation at higher displacement ductility in W2-NR, which resulted in less pinching and larger hysteretic loops (Figure 4.20). Note that the comparison was based on the ductility level. However, based on comparable drift levels,

(Figure 5.11) the conventional wall dissipated significantly more energy than the SMA wall.



**Figure 5.11** Energy Dissipation-Drift Responses

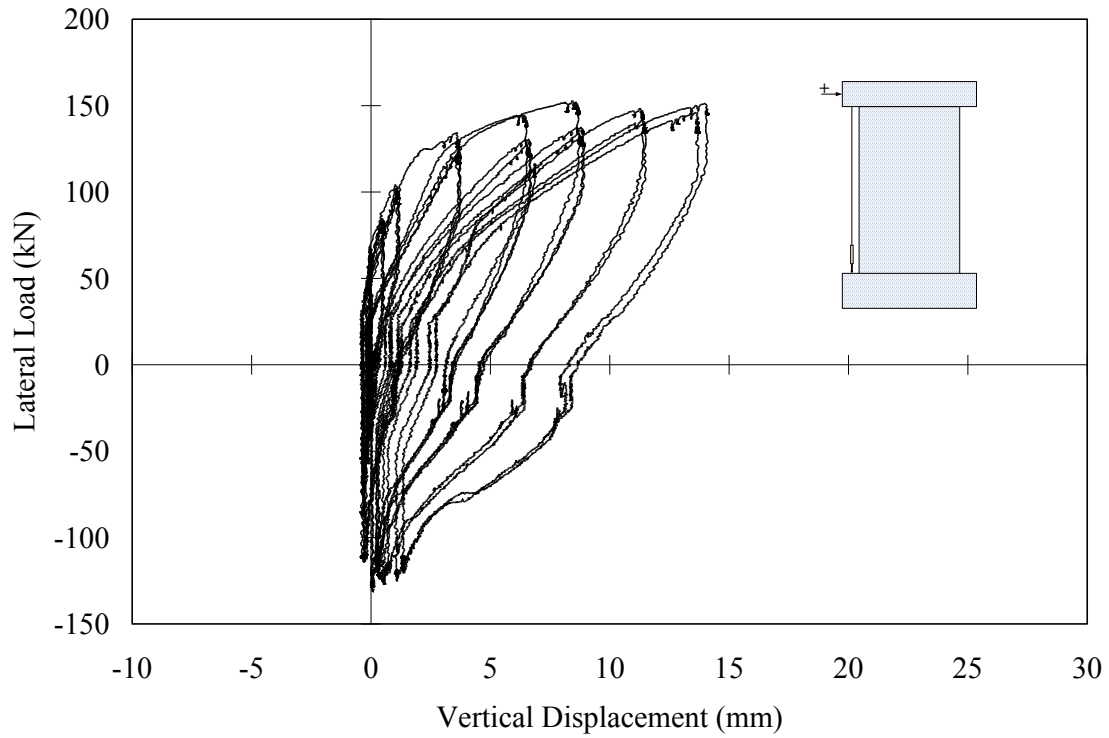
At lateral drift of 1.5%, 5831 and 2268 Nm of energy was dissipated by the conventional and SMA walls, respectively. Thus, the SMA wall dissipated approximately 39% of the energy dissipated by the conventional wall.

#### 5.3.4 Vertical Displacement of Walls

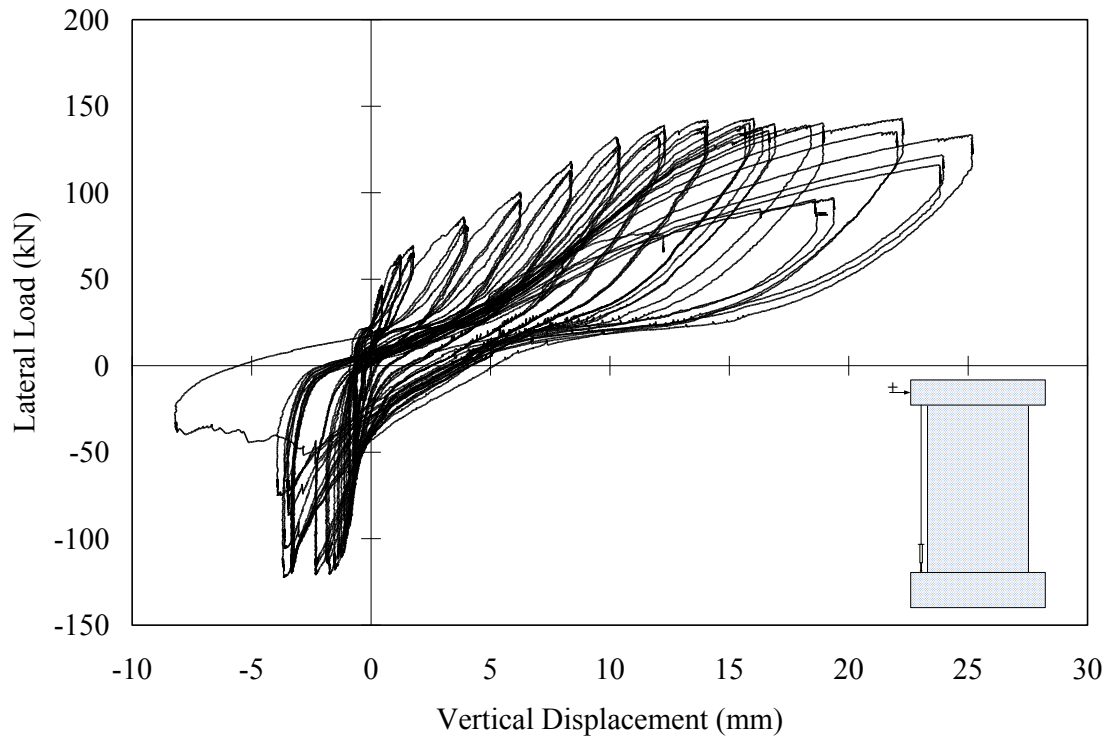
Each shear wall specimen was equipped with two LVDTs at a height of 800 mm from the base of the wall boundaries (L3 and L4) to monitor the vertical displacement of the plastic hinge region and two DCTs (C5 and C6) to monitor the vertical displacement of the top beam (Figure 3.22). These displacements resulted from concrete cracking and yielding of the longitudinal reinforcement in the wall. The hysteretic response of the



vertical displacements of the top beam for W1-SR and W2-NR are shown in Figure 5.12 and 5.13, respectively.

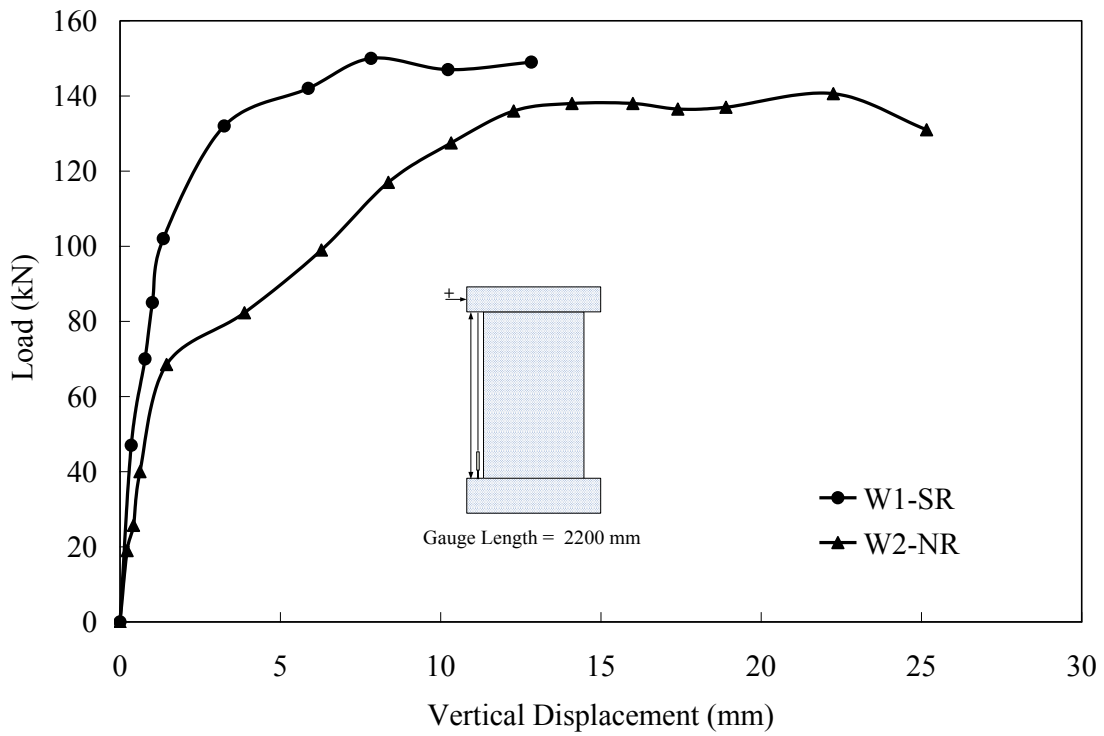


**Figure 5.12** Load-Vertical Displacement Response of Wall W1-SR



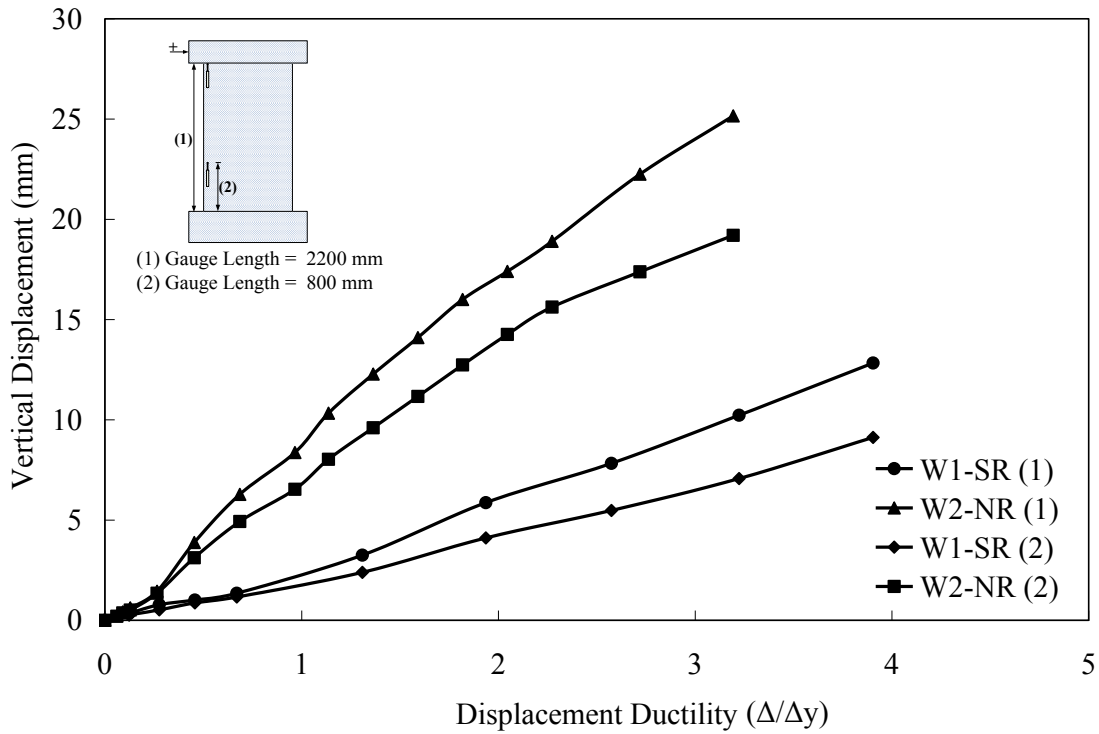
**Figure 5.13** Load-Vertical Displacement Response of Wall W2-NR

The envelope of the vertical displacement of the top beam of specimens W1-SR and W2-NR are presented in Figure 5.14. The vertical response of both walls follows the same trend as the lateral displacement response in terms of the elastic, yielding, and post-yielding behavior.



**Figure 5.14** Envelope Load-Vertical Displacement Responses for Top Beam of Wall Specimens

To study the distribution of vertical displacement along the height of the wall, the vertical displacements at the top loading beam and at a height of 800 mm versus lateral displacement ductility for both specimens are plotted in Figure 5.15.

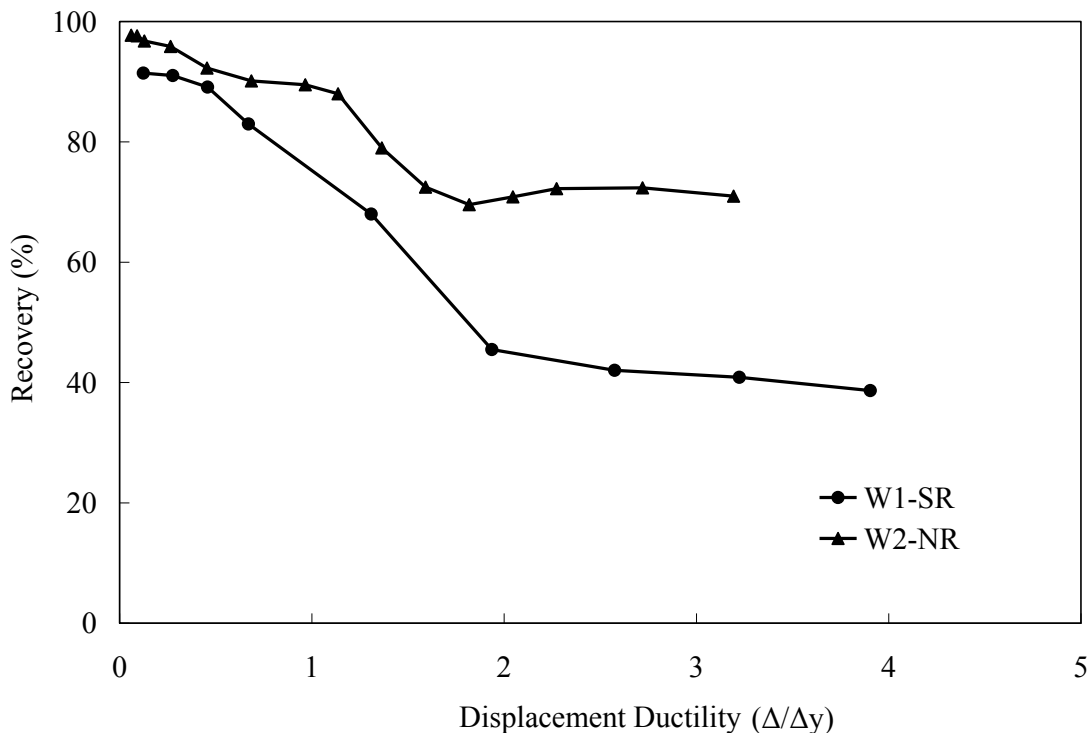


**Figure 5.15** Envelope Vertical Displacement-Ductility Responses for W1-SR and W2-NR at Different Heights

At yielding, vertical displacements of 2.21 mm and 1.77 mm were measured in W1-SR at the top beam and at 800 mm height, respectively. The corresponding displacements in W2-NR were 8.37 mm and 6.54 mm. It is apparent that based on similar lateral displacement ductility levels, larger vertical displacements were measured in the SMA wall due to the higher yield strain of the SMA bar and the wide single predominant crack that surfaced near the base of the wall. Moreover, the vertical displacements were not proportional along the height of the wall since the majority of concrete cracking occurred in the lower part (plastic hinge region) of the wall. This can be confirmed by comparing the post-yield results where the vertical displacement at 800 mm height was about 69% and 80% of the displacements at the top beam in W1-SR and W2-NR, respectively. The

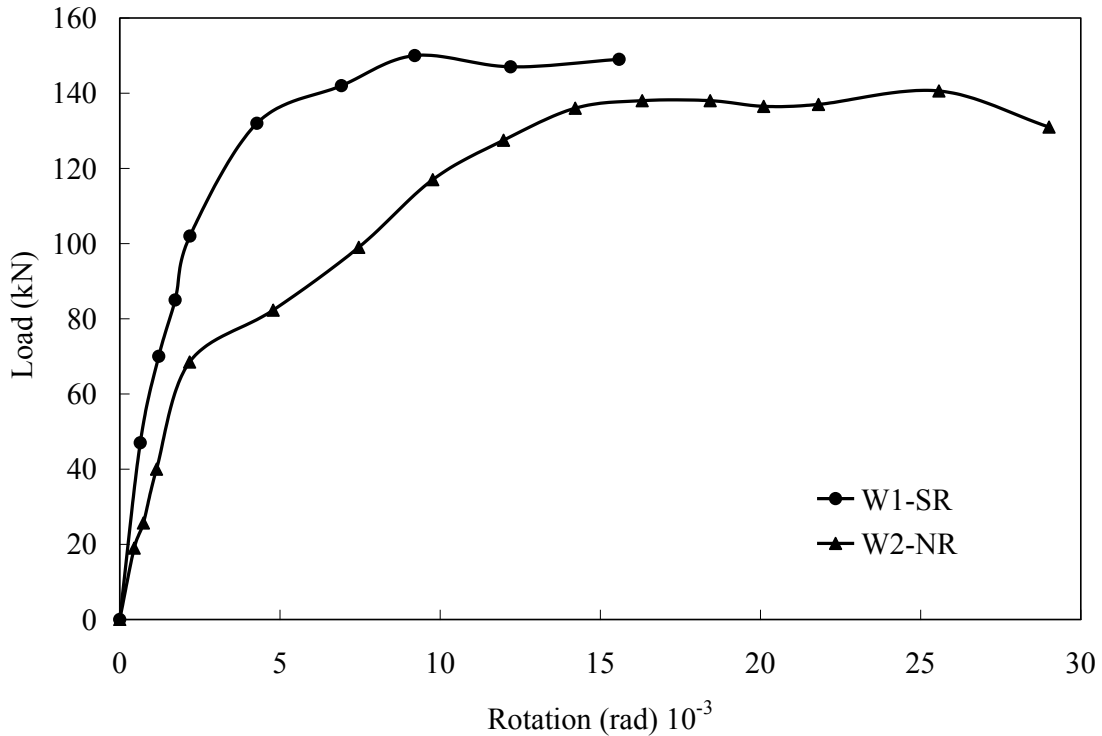
larger displacements near the base for the SMA wall indicate that the inelastic rotation is more localized near the base of the wall due to single predominant crack that controlled response. However, for the conventional reinforced wall, a smaller fraction is confined to the base of the wall.

The responses of DCTs (C5 and C6) and LVDTs (L3 and L4) in W1-SR and W2-NR demonstrated the re-centering capability of the SMA reinforcement at the boundaries. When both walls were pushed to a displacement ductility of 3.2 and then unloaded, specimen W2-NR recovered 71% of the maximum vertical displacement (25.16 mm) and was able to close most of the cracks while W1-SR managed only to recover 41% of the maximum vertical displacements (10.23 mm) as shown in Figure 5.16. The residual vertical displacement at this ductility level was 6.11 mm and 7.28 mm in W1-SR and W2-NR, respectively.



**Figure 5.16** Vertical Displacement Recovery Capacity-Displacement Ductility Behavior

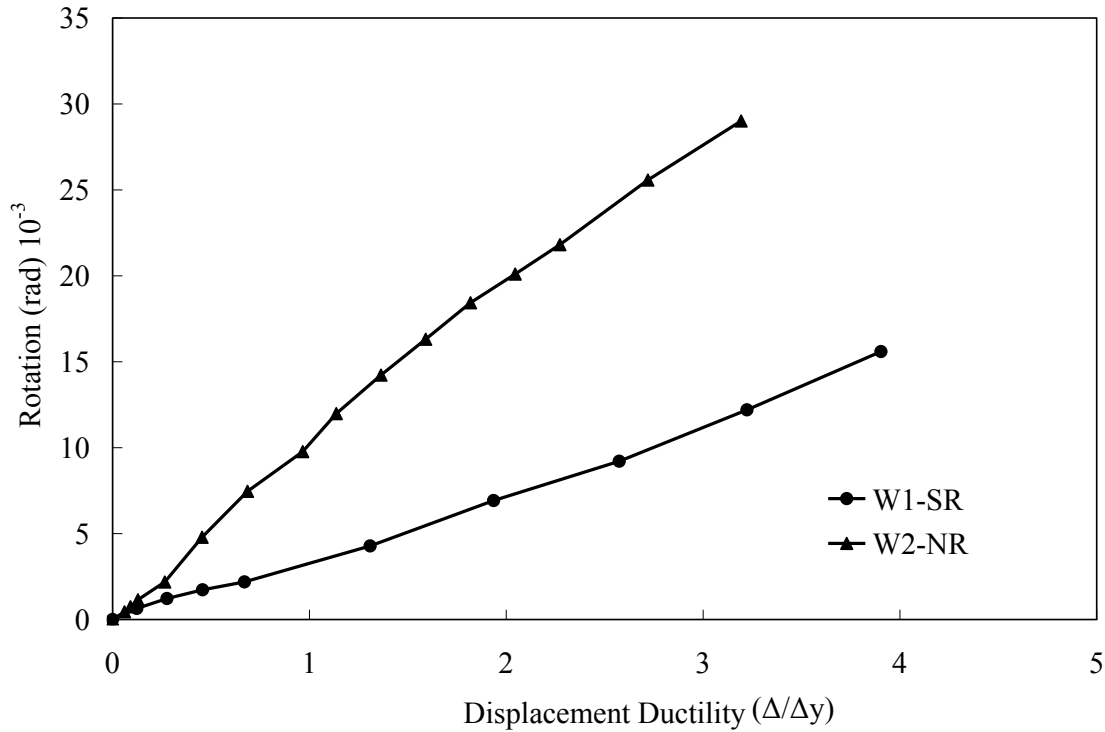
The vertical displacements were used to calculate the rotation of the wall due to the effects of flexural. The envelope load–rotation relationship of the top beam is presented in Figure 5.17 for W1-SR and W2-NR. It is evident that the rotation of the top beam in both walls increased rapidly after yielding but at a higher rate in W2-NR than W1-SR. The rotation at yielding was  $3.85 \times 10^{-3}$  radians in W1-SR and  $8.82 \times 10^{-3}$  radians in W2-NR, while at peak load W1-SR and W2-NR experienced  $9.21 \times 10^{-3}$  radians and  $16.31 \times 10^{-3}$  radians, respectively.



**Figure 5.17** Envelope Load-Rotation Responses of Wall Tests

The rotational capacity of the SMA and conventional shear walls is further demonstrated in Figure 5.18, which provides the envelope rotation–displacement ductility relationship of the top beam. Beyond yielding, the rotation in W2-NR has an average of 2.7 times the

corresponding rotation in W1-SR. It is apparent that the higher yield strain of the SMA bar resulted in larger rotations in W2-NR when compared with W1-SR.



**Figure 5.18** Rotational Capacity-Displacement Ductility Behavior

### 5.3.5 Plastic Hinge

Section 4.4.3 detailed the development of tensile yielding in the longitudinal reinforcement at the boundaries of W1-SR and W2-NR. The strain gauges in the boundaries demonstrated that plasticity spread over a length of at least 800 mm at the peak load in W1-SR. The same yield length developed in W2-NR; however, it occurred prior to the peak load. This can be attributed to the low bond stress between SMA bars and the concrete due to the smooth surface.

Table 5.1 provides a comparison of the observed and predicted plastic hinge lengths for W1-SR and W2-NR. The predictions were calculated using three methods: sectional analysis method (RESPONSE 2000), an analytical model by Paulay and Priestly (1993), and code provisions (CSA A23.3-04).

The calculation of plastic hinge length ( $l_p$ ) in RESPONSE 2000 is based on strain compatibility between the steel and the concrete in the wall section. Paulay and Priestly suggested the following expression for the plastic hinge length in ductile shear walls:

$$l_p = 0.2 l_w + 0.044 h_w$$

where  $l_w$  and  $h_w$  are the total length and height of the wall, respectively. The Canadian Standards Association Standard A23.3-04 states that the value of the plastic hinge length varies from  $0.5l_w$  to  $1.0l_w$ . The standard recommends a plastic hinge length of  $0.5l_w$  to estimate capacity and  $l_w$  to determine the demand.

**Table 5.1** Experimental and Theoretical Plastic Hinge Length

| Specimen | Plastic Hinge Length, $l_p$ (mm) |                      |              |              |
|----------|----------------------------------|----------------------|--------------|--------------|
|          | Theoretical                      |                      |              | Experimental |
|          | RESPONSE 2000                    | Paulay and Priestley | CSA A23.3-04 |              |
| W1-SR    | 680                              | 305                  | 500-1000     | >800         |
| W2-NR    | 365                              | 305                  | 500-1000     | >800         |

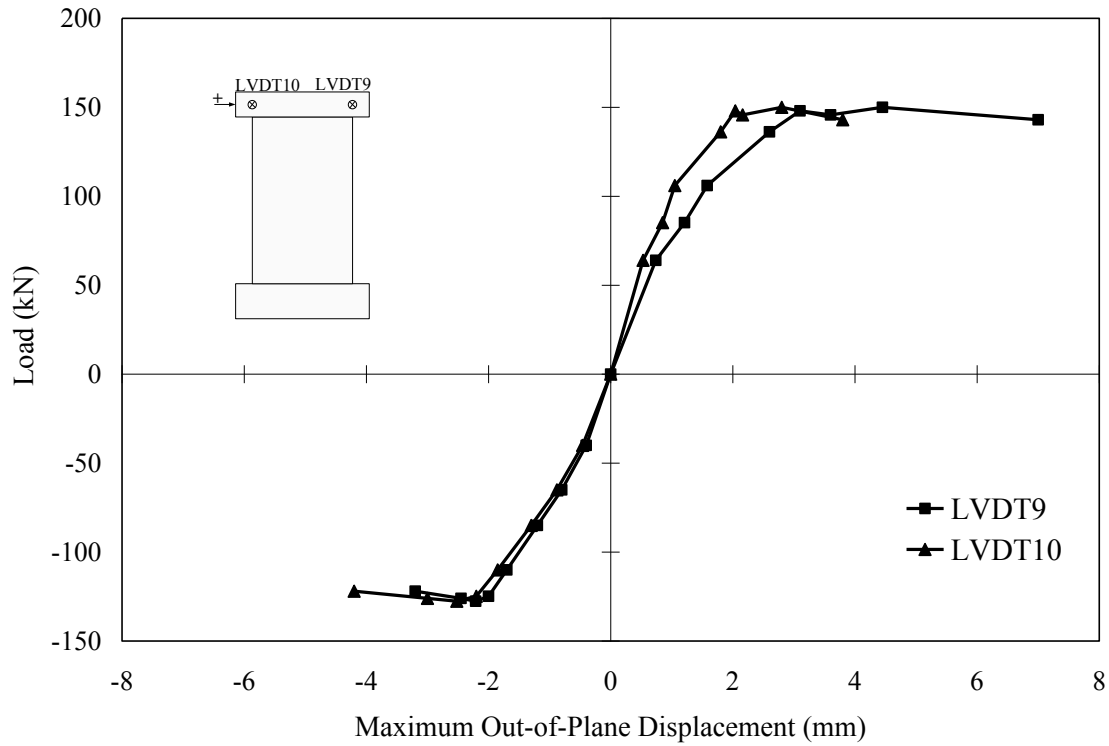
The sectional analysis method using Program Response 2000 predicted a higher plastic hinge length for Wall W1-SR than the corresponding lower bound length predicted with A23.3-04 and Paulay and Priestly. However, the plastic hinge length calculated with the sectional analysis method was less than the length determined from the strain gauge data obtained during testing and the upper bound of A23.3-04. In the SMA Wall W2-NR, the experimental plastic hinge developed over a length that exceeded all predictions, except



for the upper bound length used in A23.3-04. Therefore, for both walls, the plastic hinge approaches the upper bound ( $1.0 l_w$ ) prescribed in CSA A23.3-04. Further research and parametric studies are required to set new criteria for the plastic hinge length for SMA reinforcement in shear walls.

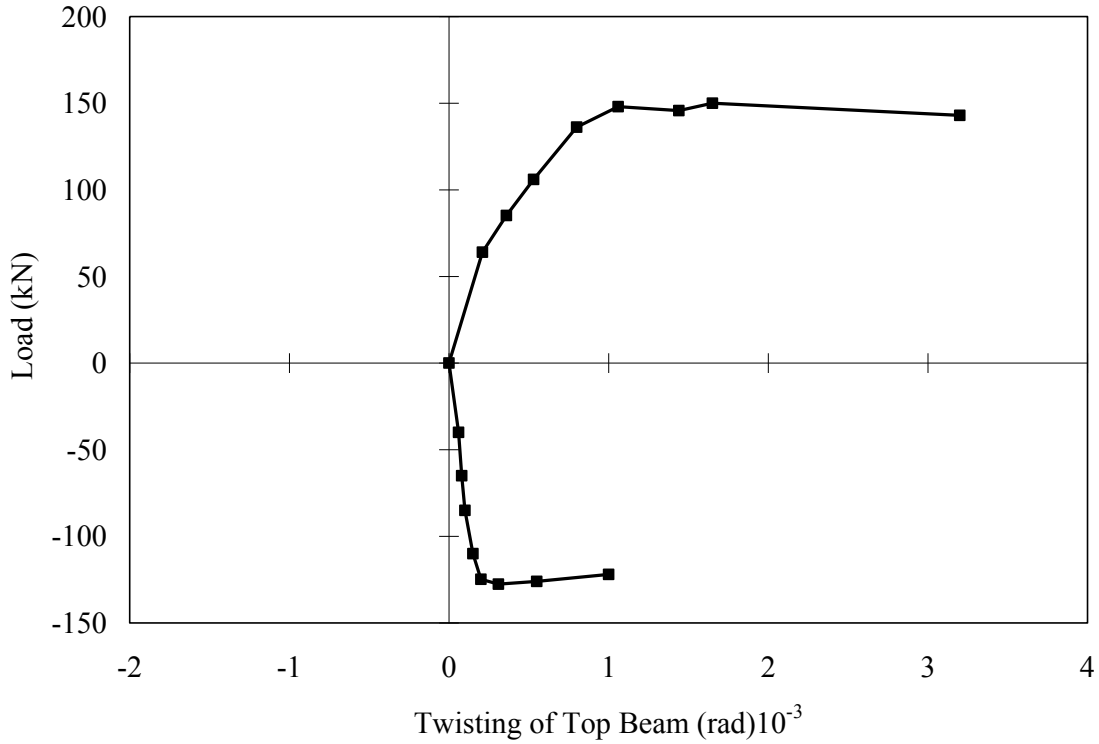
#### 5.3.6 Twisting of the Top Beam

Two LVDTs (L9 and L10) (Figure 3.22) were used to record out-of-plane displacements of W1-SR, which was used to calculate twisting of the wall. Both LVDTs were placed on the same side of the top beam and separated by a distance of 1000 mm. The hysteretic responses of LVDT9 and LVDT10 are presented in Appendix D. The envelope responses at each loading level for both LVDTs are plotted in Figure 5.19. At yielding, an out-of-plane displacement of 1.58 mm and 1.05 mm were recorded at a lateral load of 116 kN, while notable out-of-plane displacements of 6.8 mm and 3.8 mm were measured during the final loading stage in LVDT9 and LVDT10 respectively. These results indicated the need for a lateral support system, which was implemented during testing of the SMA wall. Note that for a significant range of testing, the out-of-plane displacements were similar for both LVDTs. This is an indication of out-of-plane bending.



**Figure 5.19** Envelope Load-Out-of-Plane Displacement Response of W1-SR

Twisting of the top beam was calculated from the difference in displacement readings between LVDT9 and LVDT10. Figure 5.20 shows the envelope of the hysteretic response of the twisting behavior of specimen W1-SR. The out-of-plane displacements and twisting of the wall were insignificant during the test up to a displacement level of 30 mm (displacement ductility of 3.2). At this displacement level, the wall began to experience relatively large out-of-plane twisting.

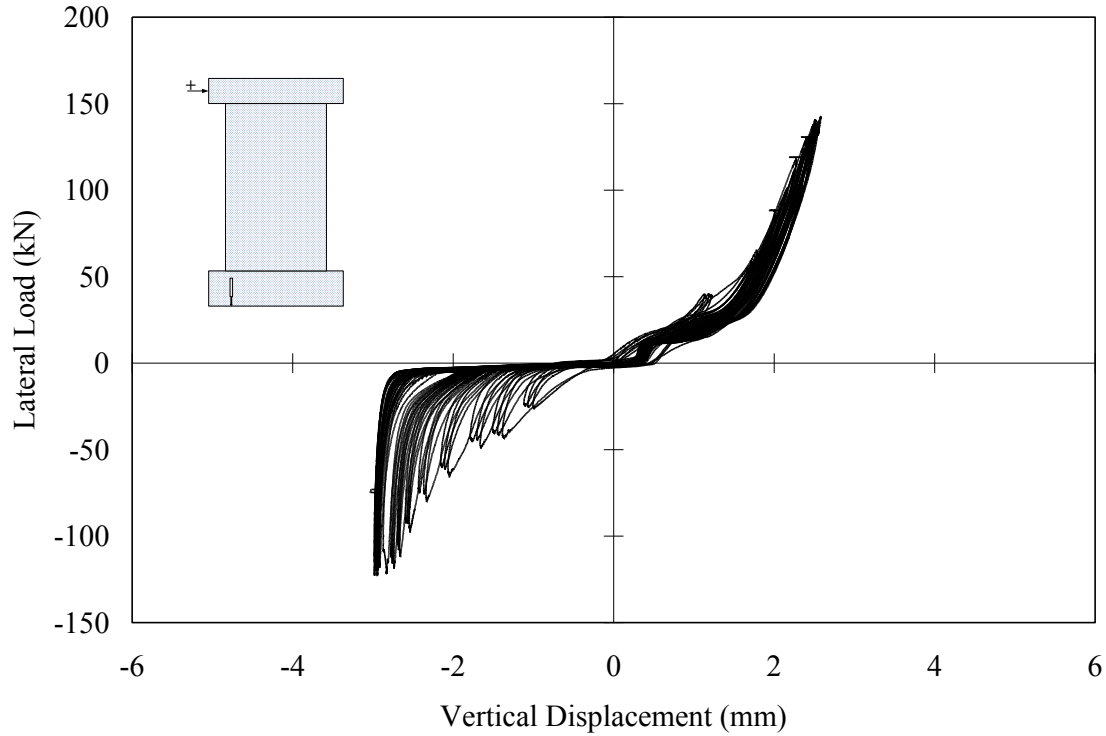


**Figure 5.20** Envelope Load-Twisting Response of W1-SR

### 5.3.7 Foundation Base Rotation and Slip

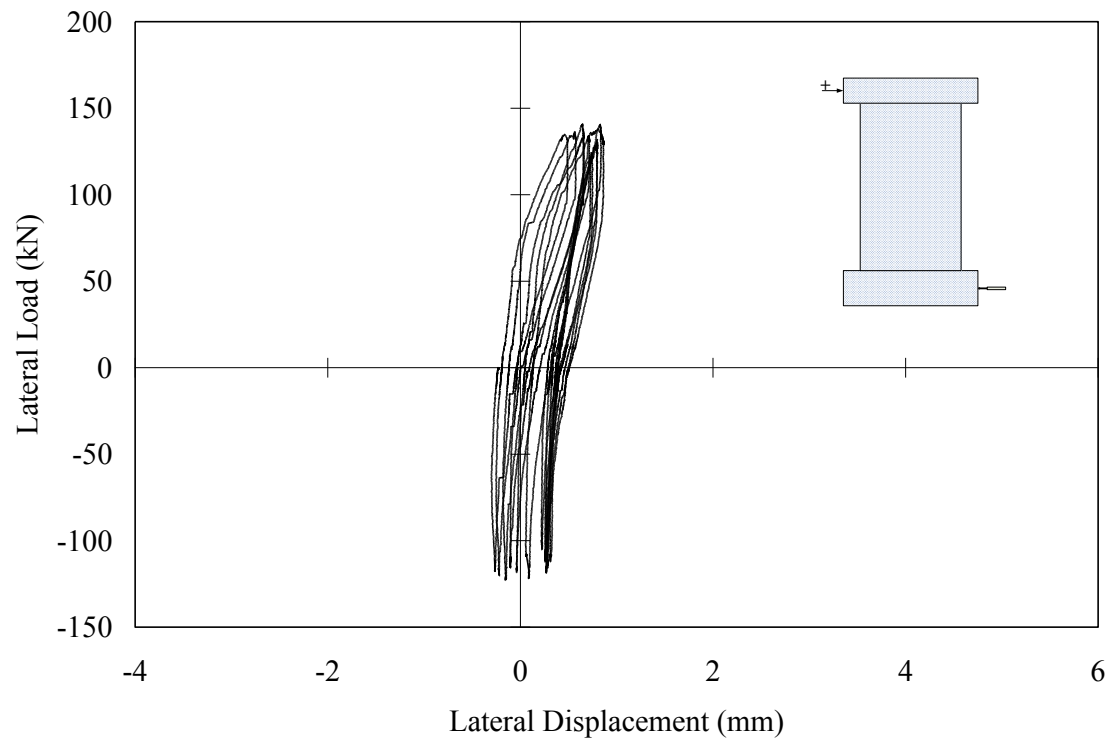
The test specimens were fastened to the laboratory strong floor by means of four-70 mm diameter high strength bolts to resist sliding and rocking of the foundation during testing. Due to the absence of externally applied axial load, except the self-weight of the test specimens, the base of the foundation was subjected to tensile stresses (uplift forces) during testing. To monitor uplift and rocking of the foundation, LVDT2 (Figure 3.22) was attached to the side of the foundation at a height of 100 mm from the strong floor. Figure 5.21 provides the response of LVDT2 for specimen W2-NR. The vertical displacements were negligible (less than 3 mm) and, therefore, had marginal effect on the general response of the wall. For W1-SR, the maximum vertical displacement was 1.3

mm which confirms that foundation uplift was negligible. The base vertical displacement hysteretic responses of W1-SR is presented in Appendix D



**Figure 5.21** Load-Vertical Displacement Response of Foundation in W2-NR

LVDT1 was attached at a height of 250 mm from the floor to measure the horizontal displacement of the base relative to the strong lab floor. As shown in Figure 5.22, the measured base slip in W2-NR was negligible (less than 1 mm) and, therefore, had no effect on the general lateral displacement response of the wall. For W1-SR, a similar response (Appendix D) was observed with a maximum base slip of 2.2 mm. This confirms the results obtained from DCT1 and DCT9 (Figures 4.23), which measured the hysteretic lateral response of the wall relative to the base and the lab floor, respectively.



**Figure 5.22** Load-Horizontal Displacement Response of Foundation in W2-NR

## CHAPTER 6

### SMA CONSTITUTIVE MODELING

#### 6.1 Introduction

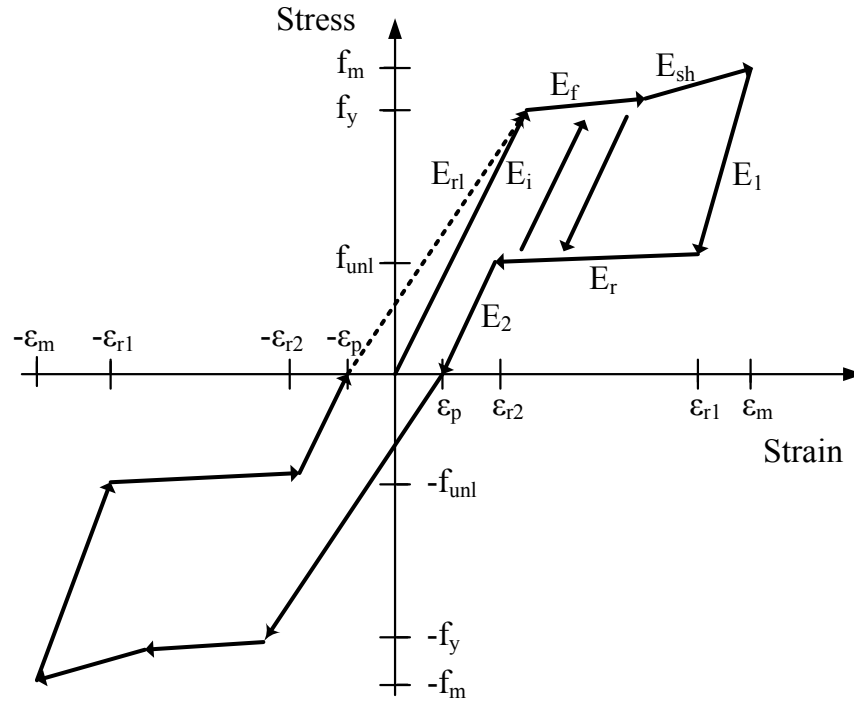
In this chapter, a complementary analytical program consisting of developing hysteretic constitutive models for superelastic SMA bars is presented. The proposed model considers the initial linear elastic loading curve, followed by yielding and strain hardening. In addition, consideration is given to unloading and reloading curves such that the energy dissipation of a superelastic SMA bar is satisfactorily captured. The proposed model was implemented in Program VecTor2 (Wang and Vecchio 2002), a nonlinear two-dimensional finite element program applicable for membrane structures. Program VecTor2 uses a smeared, rotating-crack formulation based on the Modified Compression Field Theory (Vecchio and Collins 1986) and the Disturbed Stress Field Model (Vecchio 2000). The program algorithm is based on a secant stiffness formulation using a total-load iterative procedure. The program was previously updated to simulate the hysteretic response of reinforced concrete members (Vecchio 2000; Palermo and Vecchio 2003, 2004 and 2007). VecTor2 was used to conduct analyses of the beams and shear walls reinforced with SMA bars of this experimental testing program.

#### 6.2 Superelastic Nitinol SMA Reinforcement Model

Several hysteretic models, as detailed in Chapter 2, were previously developed for superelastic SMA bars. These models assumed full strain recovery assuming no accumulation of residual strains. The proposed model provides improvements to other models including explicit consideration of permanent strains (Figure 4.2); the degradation

of the lower plateau stress (reverse transformation), which results from the recovery of strains; and a trilinear unloading response.

The proposed model is shown in Figure 6.1, which is a multi-segment hysteretic model consisting of linear branches to capture the hysteretic behavior of SMAs (Figure 4.2).



**Figure 6.1** SMA Plastic Offset Constitutive Hysteretic Model

### 6.2.1 Model Formulation

The formulation of the SMA hysteretic model is based on calculating stresses from a set of rules linked to the backbone curve, which is assumed to be represented by the monotonic response. These rules define the backbone curve, unloading and reloading responses, and partial unloading/reloading behavior.

### 6.2.2 Backbone Response

The backbone response is assumed to be trilinear. The initial response is linear elastic followed by yielding (forward transformation phase) and strain hardening, which defines the stress-induced elastic response of Martensite. The backbone envelope response is described by the following general formulation:

$$f_s = f_{s-i} + E(\varepsilon_s - \varepsilon_{s-i}) \quad (6.1)$$

$f_s$  is the stress at the current strain of  $\varepsilon_s$ , and  $f_{s-i}$  and  $\varepsilon_{s-i}$  are the stress and strain from the previous load step, respectively.  $E$  is the modulus of elasticity that represents the initial  $E_i$ , forward transformation  $E_f$  or strain hardening  $E_{sh}$  moduli depending on the imposed strain level.

The domain for each modulus is defined by the strain at the beginning and at the end of the associated loading path as described by the following formulations:

$$f_s = f_{s-i} + E_i (\varepsilon_s - \varepsilon_{s-i}) \quad 0 < \varepsilon_s < \varepsilon_y \quad (6.2)$$

$$f_s = f_{s-i} + E_f (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_y < \varepsilon_s < \varepsilon_{sh} \quad (6.3)$$

$$f_s = f_{s-i} + E_{sh} (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_{sh} < \varepsilon_s < \varepsilon_m, \quad \varepsilon_m < \varepsilon_u \quad (6.4)$$

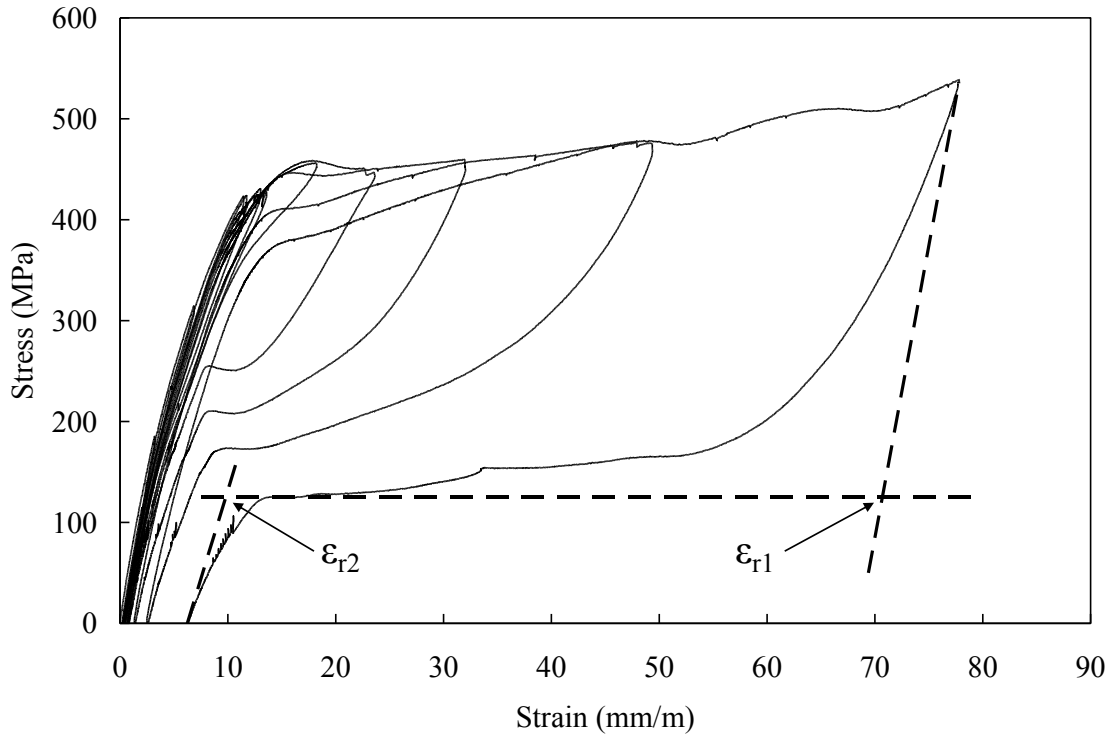
$\varepsilon_m$  is the maximum current strain,  $\varepsilon_y$  and  $\varepsilon_u$  are the yield and ultimate strains, respectively, and  $\varepsilon_{sh}$  is the strain at the onset of strain hardening. From coupon testing of the SMA bar or specifications provided by the manufacturer,  $E_i$ ,  $E_f$ ,  $E_{sh}$ ,  $\varepsilon_y$ ,  $\varepsilon_u$ , and  $\varepsilon_{sh}$  can be determined. The same formulations are assumed applicable for SMA reinforcement in tension and compression.

### 6.2.3 Unloading/Reloading Curves

The unloading response is defined by a trilinear response starting from the maximum current strain  $\varepsilon_m$  to the plastic offset strain  $\varepsilon_p$ . In addition to the plastic strain  $\varepsilon_p$ , the first



$\epsilon_{r1}$  and the second  $\epsilon_{r2}$  unloading strains are required to define the unloading domain. It is evident from the experimental results that the plastic strain can be extracted easily while judgment is needed to determine the values of the first  $\epsilon_{r1}$  and the second  $\epsilon_{r2}$  unloading strains, respectively. The first and the second unloading strains can be reasonably estimated from the intersection points of the flat plateau of the unloading branch (reverse transformation) with tangents drawn from the current maximum and plastic strains with slope equal to the initial stiffness as shown in Figure 6.2.



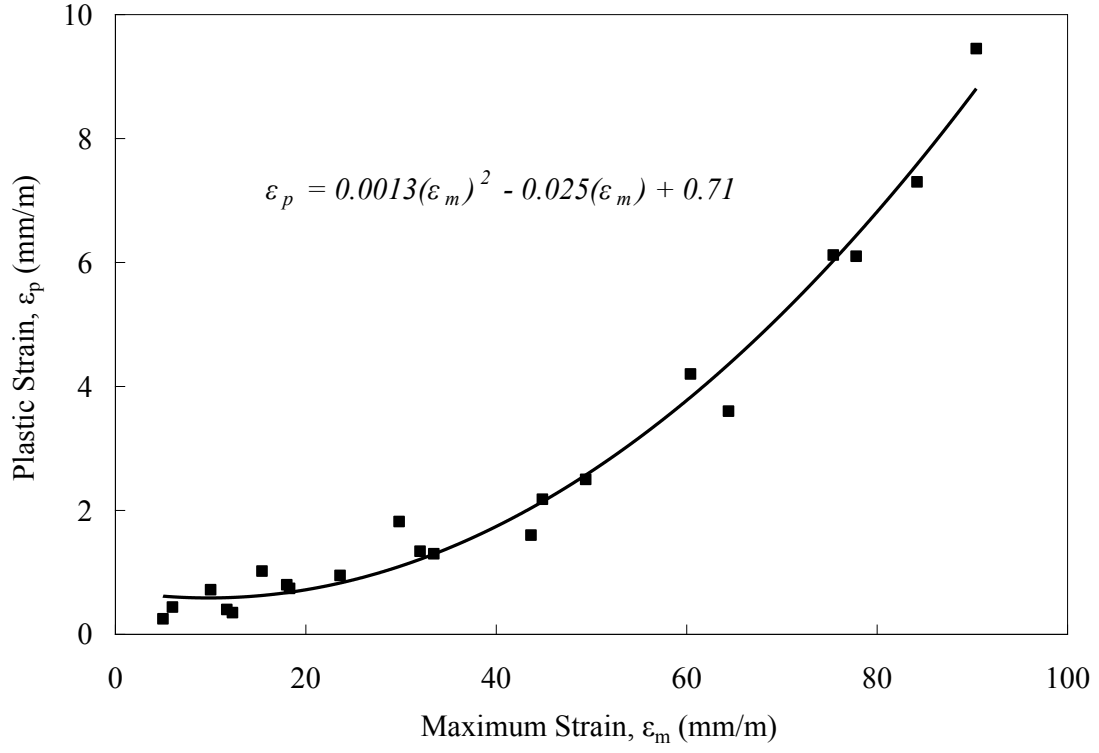
**Figure 6.2** Modeling of the SMA Cyclic Test

The unloading and plastic strains are dependent on the current maximum strain in the history of loading. Figures 6.3, 6.4 and 6.5 illustrate the strong correlation between these parameters and the maximum strain. Thus, numerical expressions were derived for  $\epsilon_p$ ,  $\epsilon_{r1}$ , and  $\epsilon_{r2}$  as a function of the current maximum strain  $\epsilon_m$ :

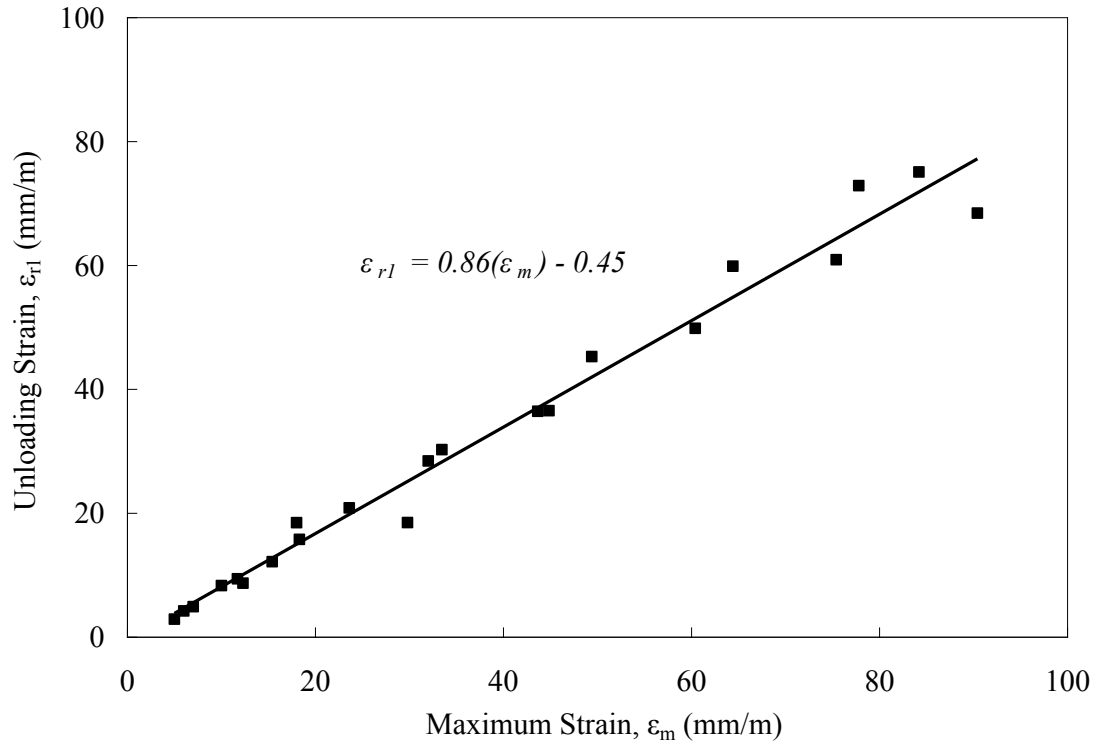
$$\varepsilon_p = 0.0013(\varepsilon_m)^2 - 0.025(\varepsilon_m) + 0.71 \quad (6.5)$$

$$\varepsilon_{r1} = 0.86(\varepsilon_m) - 0.45 \quad (6.6)$$

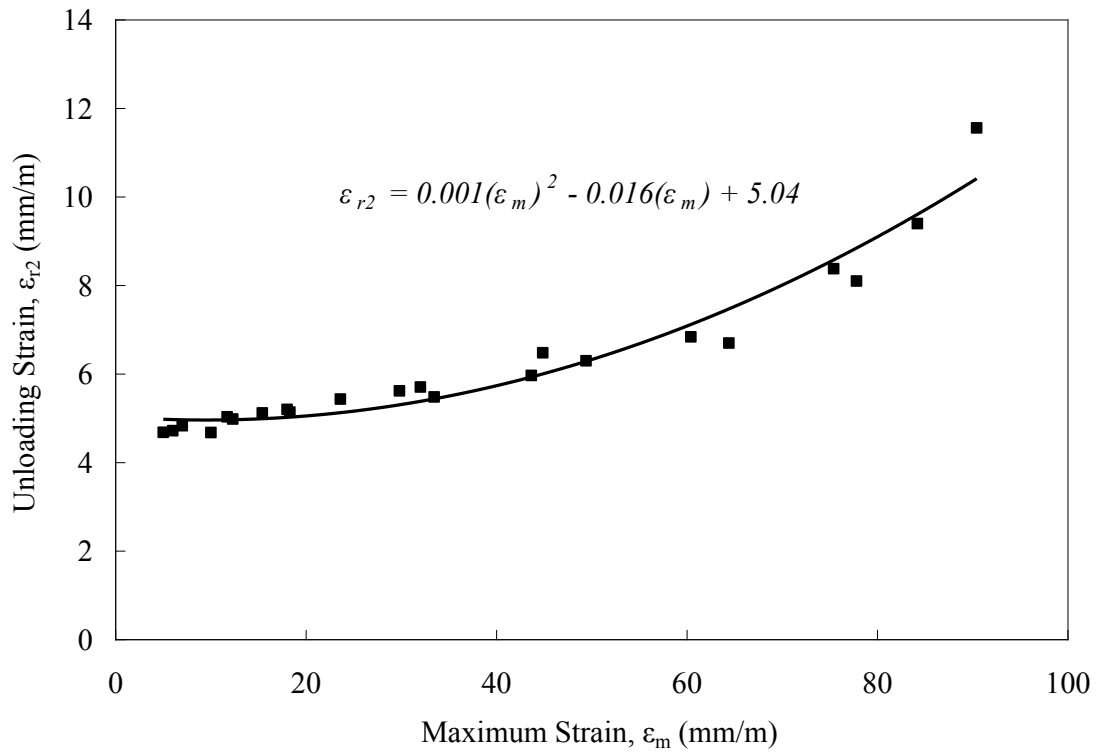
$$\varepsilon_{r2} = 0.001(\varepsilon_m)^2 - 0.016(\varepsilon_m) + 5.04 \quad (6.7)$$



**Figure 6.3** Plastic Strain Relationship for SMA Bar



**Figure 6.4** First Unloading Strain Relationship for SMA Bar



**Figure 6.5** Second Unloading Strain Relationship for SMA Bar

Accordingly, the unloading response can be determined using the relationships derived for  $\varepsilon_p$ ,  $\varepsilon_{r1}$ , and  $\varepsilon_{r2}$  in addition to the moduli  $E_1$ ,  $E_2$ , and  $E_r$  which are determined experimentally or provided by the manufacturer:

$$f_s = f_{s-i} + E_1 (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_{r1} < \varepsilon_s < \varepsilon_m \quad (6.7)$$

$$f_s = f_{s-i} + E_r (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_{r2} < \varepsilon_s < \varepsilon_{r1} \quad (6.8)$$

$$f_s = f_{s-i} + E_2 (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_p < \varepsilon_s < \varepsilon_{r2} \quad (6.9)$$

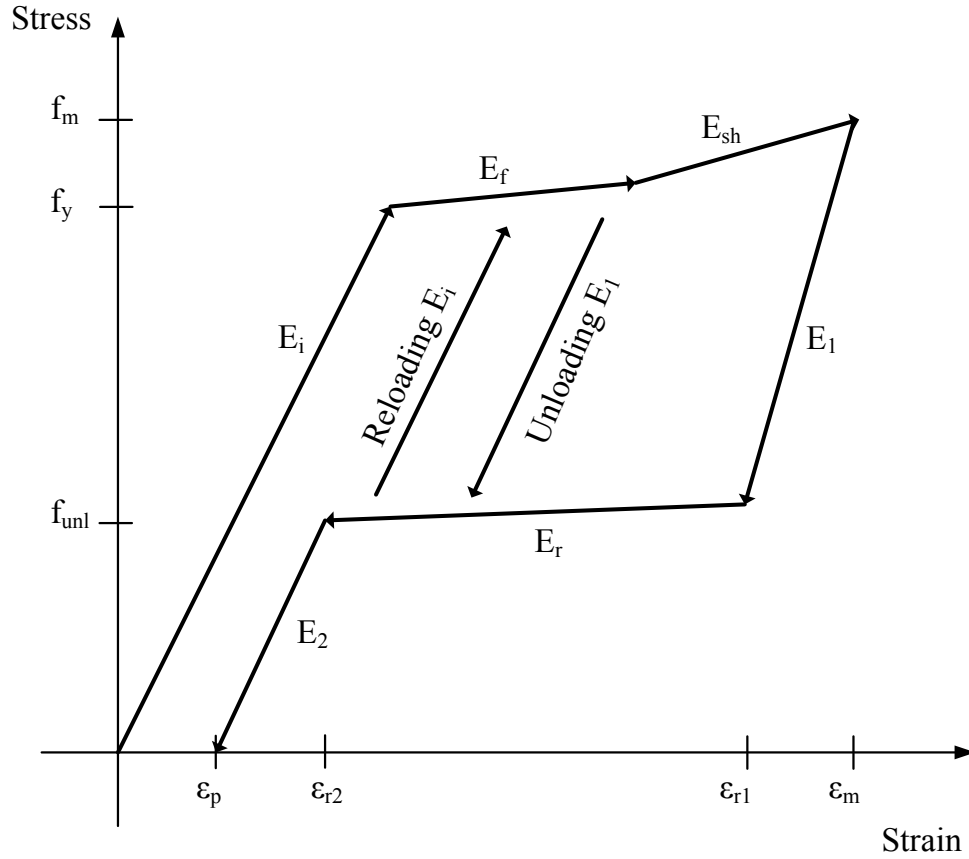
$E_1$  and  $E_2$  are the first and second unloading moduli, respectively, while  $E_r$  refers to the reverse transformation modulus.

The reloading response after one complete cycle is linear originating from the plastic offset strain  $\varepsilon_p$  to the yield strain  $\varepsilon_y$ . Thereafter, the response follows the backbone envelope curve. The required parameters to define the reloading response, as shown in Figure 6.1, are the plastic and yield strains and yield stress. Equation 6.1 is still applicable for the reloading response noting that  $E_{rl}$  is calculated from the plastic strain after each full loading cycle:

$$f_s = f_{s-i} + E_{rl} (\varepsilon_s - \varepsilon_{s-i}) \quad \varepsilon_p < \varepsilon_s < \varepsilon_y \quad (6.9)$$

#### 6.2.4 Partial Unloading/Reloading Response

Since test data are not available to model partial unloading and reloading responses of superelastic SMA bars, a linear model is adopted using the general loading expression of Equation 6.1. Following the same rules applied in the loading and unloading responses, the modulus  $E$  is assumed equal to the first unloading modulus  $E_1$  during unloading and  $E_i$  during reloading as illustrated in Figure 6.6.



**Figure 6.6** Partial Unloading/Reloading Response

### 6.2.5 Model Modifications

The general rules for the hysteretic SMA model were modified to simplify the implementation into Program VecTor2.

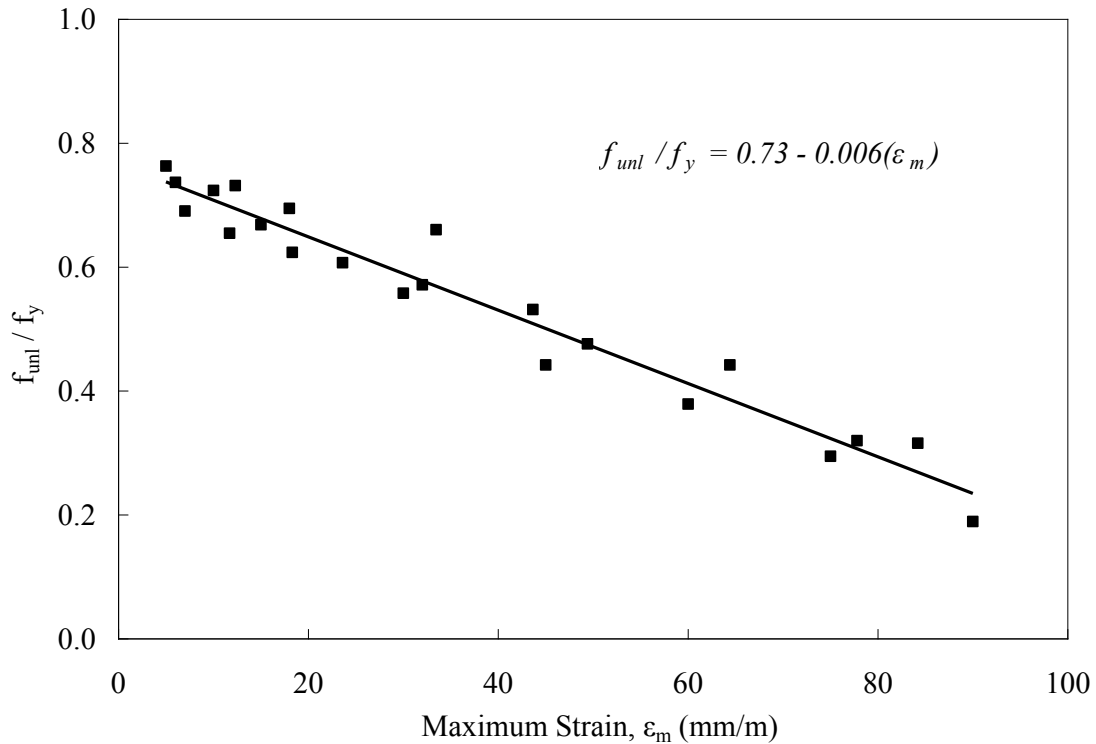
Coupon test results of the SMA indicated that the forward and backward transformation stiffness are low and range from 4 GPa to 22 GPa depending on the maximum strain level. Hence, the transformation loading and unloading path can be reasonably assumed flat resulting in zero values for the transformation stiffness  $E_f$  and  $E_r$ , respectively. This assumption simplifies the general and the partial unloading/reloading responses in the SMA model.

The second modification for the SMA model includes the approximation of the unloading stiffness  $E_1$  and  $E_2$  to be equal to the initial stiffness  $E_i$ . This is extended to include the partial unloading/reloading stiffness.

As a result of these adjustments, Equations 6.2 and 6.4 are still applicable to describe the loading branch, while equation 6.3 is rewritten as follows:

$$f_s = f_y \quad \varepsilon_y < \varepsilon_s < \varepsilon_{sh} \quad (6.10)$$

In addition, the reverse transformation stress can be expressed as a function of the current maximum strain as shown in Figure 6.7.

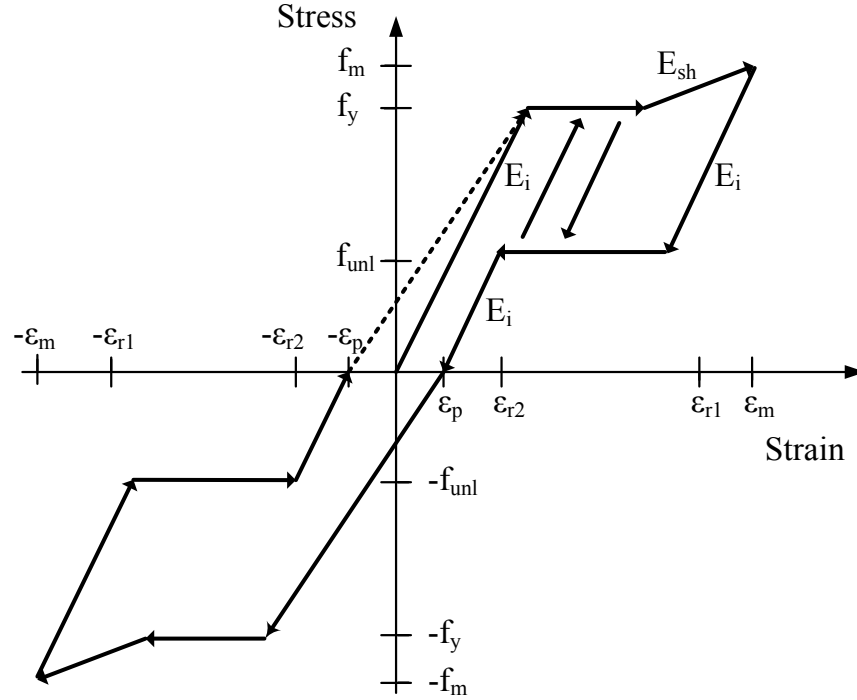


**Figure 6.7** Unloading Stress Relationship for SMA bar

$$f_{unl}/f_y = 0.73 - 0.006(\varepsilon_m) \quad 0.25 f_y < f_{unl} < 0.75 f_y \quad (6.11)$$

$f_{unl}$  is the unloading plateau stress that defines the reverse transformation from Martensite back to Austenite. Note that  $f_{unl}$  is normalized with respect to the yield stress to eliminate

the influence of stress on the formulation. The upper and the lower bounds for  $f_{unl}$  are introduced for stability without affecting the actual SMA behavior. The modified hysteretic SMA model is shown in Figure 6.8 where only  $\epsilon_p$ ,  $\epsilon_{r1}$ , and  $f_{unl}$  are required in addition to the properties to define the backbone curve.



**Figure 6.8** Modified SMA Hysteretic Constitutive Model

In the modified model, the unloading branches can be described by the following formulations:

$$f_s = f_{s-i} + E_i (\epsilon_s - \epsilon_{s-i}) \quad \epsilon_{r1} < \epsilon_s < \epsilon_m, \quad \epsilon_p < \epsilon_s < \epsilon_{r2} \quad (6.12)$$

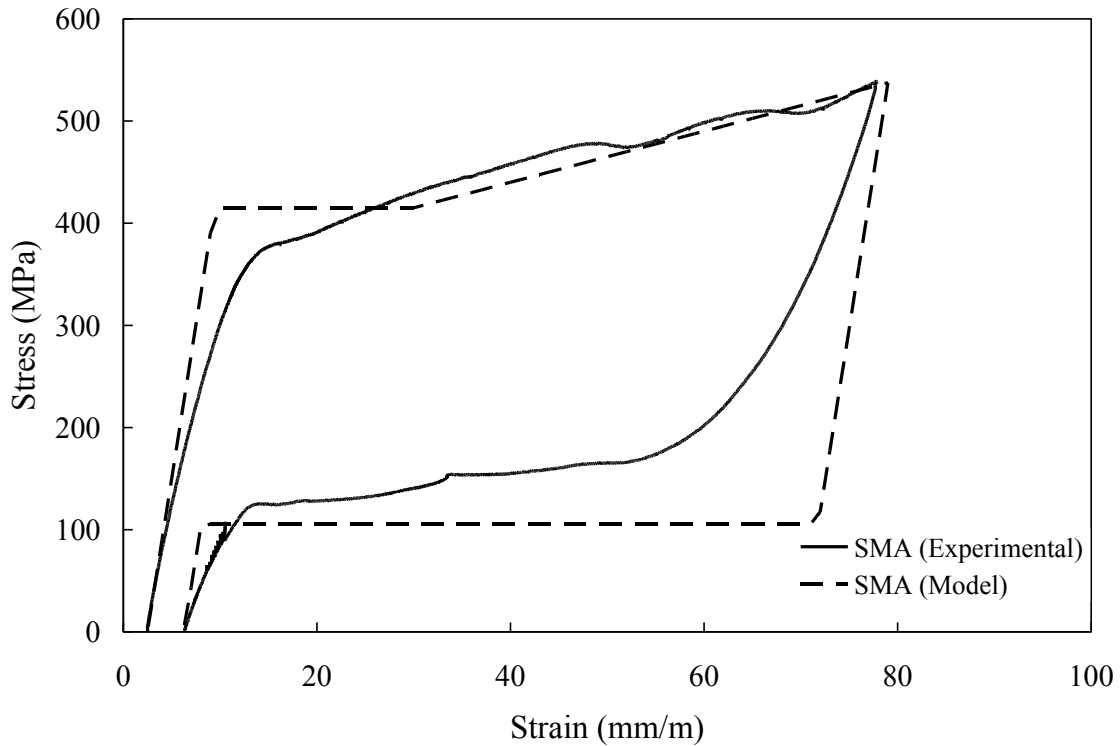
$$f_s = f_{unl} \quad \epsilon_{r2} < \epsilon_s < \epsilon_{r1} \quad (6.13)$$

The first unloading strain  $\epsilon_{r1}$  is defined as the strain corresponding to the intersection between initial unloading as per Equation 6.12 and the unloading plateau stress. The

unloading plateau stress  $f_{unl}$  is identified as the stress corresponding to zero stiffness along the unloading path. The second unloading strain,  $\epsilon_{r2}$ , is implicitly determined in Program VecTor 2 from the intersection of the unloading plateau stress and the unloading to plastic offset.

### 6.3 Verification of the SMA Constitutive Model

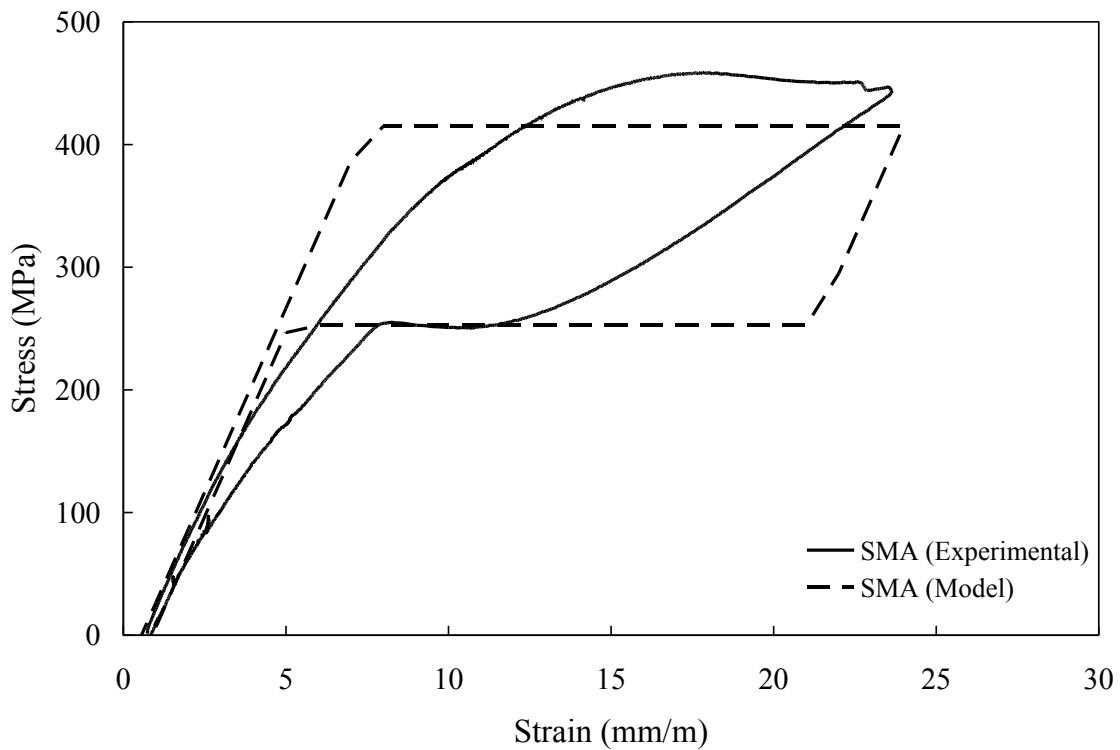
The proposed preliminary model is utilized to simulate the hysteretic response of superelastic SMA bars under cyclic loading. The experimental results obtained from the cyclic coupon test of a SMA bar (Figure 4.2) are compared with the model at various strain levels. Figure 6.9 provides a comparison of the model predictions and the experimental response of SMA after being subjected to a strain level of 78 mm/m.



**Figure 6.9** SMA Hysteretic Response at 78 mm/m strain



Figure 6.9 demonstrates the satisfactory simulation of the hysteretic response predicted by the proposed model in comparison to the experimental data. It is evident that the model can reasonably reflect the characteristics of the mechanical behavior of the SMA, specifically the reverse transformation (unloading) phase and the plastic offset strain. Note that the energy dissipated is slightly overestimated as a result of the trilinear unloading incorporated in the model. Compared to the response at an imposed strain level of 24 mm/m (Figure 6.10), the SMA model reasonably predicts the hysteretic response at both stages, but more accurately at higher level of strains, specifically the post-yield plateau and the unloading response.



**Figure 6.10** SMA Hysteretic Response at 24 mm/m strain

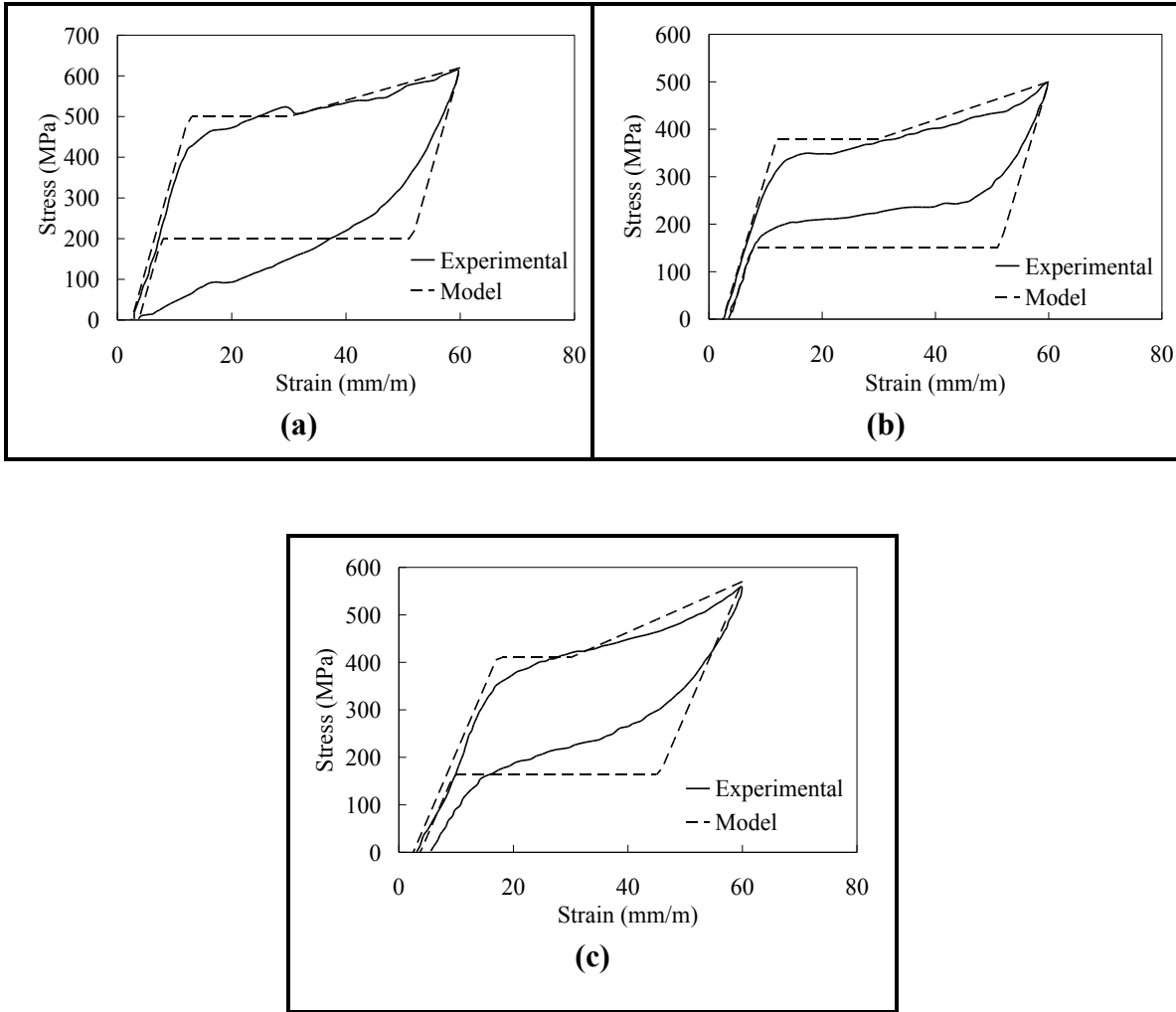
To better assess the proposed model, hysteretic responses of superelastic SMA wires and bars tested by other researchers (DesRoches et al. 2003) were examined. These SMA

specimens have different material properties than those used in this research program as shown in Table 6.1. However, a similar loading protocol for cyclic tests was followed.

**Table 6.1** Material Properties for SMA Wires and Bars

| <b>Specimen</b> | <b>Diameter<br/>(mm)</b> | <b>Modulus of<br/>Elasticity<br/>(MPa)</b> | <b>Yield Stress<br/>(MPa)</b> | <b>Maximum<br/>Strain<br/>(mm/m)</b> | <b>Maximum<br/>Stress<br/>(MPa)</b> |
|-----------------|--------------------------|--------------------------------------------|-------------------------------|--------------------------------------|-------------------------------------|
| <b>W1</b>       | 1.8                      | 45000                                      | 510                           | 60                                   | 620                                 |
| <b>W2</b>       | 7.1                      | 38000                                      | 380                           | 60                                   | 500                                 |
| <b>D1</b>       | 25.4                     | 28000                                      | 410                           | 60                                   | 570                                 |

Figure 6.11 shows the experimental and the model responses for imposed strains of 60 mm/m. The predicted hysteretic response satisfactorily simulated the experimental response of the specimens.

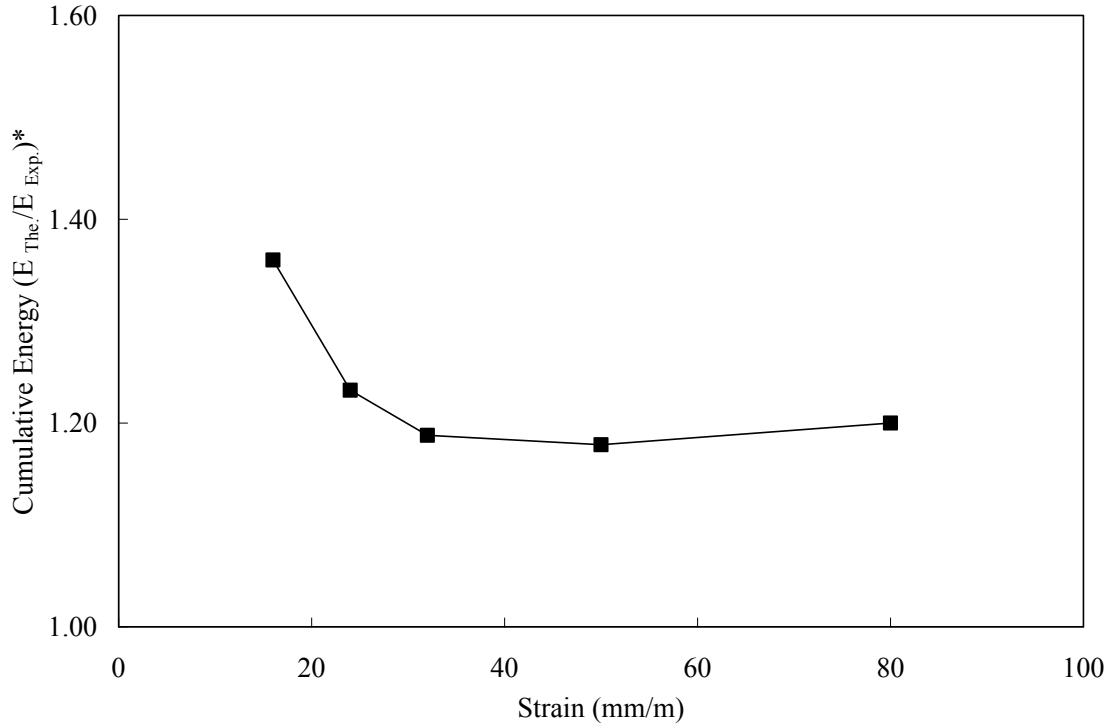


**Figure 6.11** Model and Experimental Response of SMA Reinforcement: (a) W1; (b) W2; and (c) D1

The residual strains upon unloading were accurately predicted in W1 and W2, while it was underestimated in D1. Alternatively, the unloading response, specifically the unloading plateau stress, was better simulated for W2 and D1 than W1.

To provide an additional quantitative evaluation between the proposed model and experimental results of SMA coupon testing in this research, the calculated cumulative absorbed strain energy (based on the area under the stress-strain response) for the

proposed model relative to the experimental results at various strain amplitudes are compared in Figure 6.12.



\*  $E_{The}$  = Model Cumulative Dissipated Energy  
 $E_{Exp}$  = Experimental Cumulative Dissipated Energy

**Figure 6.12** Cumulative Calculated Absorbed Energies

The results indicate that the absorbed energy predicted by the proposed model is approximately 1.18 (at 50 mm/m strain) to 1.36 (at 16 mm/m strain) times the corresponding energy obtained from the experimental data. It can be concluded that, although the predicted first unloading responses are shifted from the corresponding experimental response at early stages, the proposed model estimates reasonably the total absorbed energy and provides better predictions at higher strain levels.

In general, the proposed model consisting of a series of linear segments provided reasonable simulations demonstrating its applicability to model superelastic SMA bars. Improved results can be achieved with a refined constitutive model that includes nonlinear unloading and reloading curves. Furthermore, with additional coupon testing of SMA bars, the plastic offset strain  $\varepsilon_p$ , the first unloading strain  $\varepsilon_{rl}$  and the unloading plateau stress  $f_{unl}$  formulations can be refined.

## CHAPTER 7

### **FINITE ELEMENT ANALYSIS**

#### 7.1 Introduction

To augment the experimental work, numerical modeling was performed on the beam and shear wall test specimens using VecTor2, a nonlinear finite element (FE) software package. The modeling procedures and numerical results are discussed in this chapter. Specifically, the subsections contain descriptions of the modeling techniques and a comprehensive comparison between the numerical and experimental responses.

#### 7.2 Finite Element Modeling

Program VecTor2, a nonlinear finite element program was used to perform the numerical analyses. The program has been in development since 1990; the original version was named TRIX. This development has coincided with experimental tests to corroborate VecTor2 to predict the load-deformation response of a variety of reinforced concrete structures exhibiting well-distributed cracking when subjected arbitrary loading conditions.

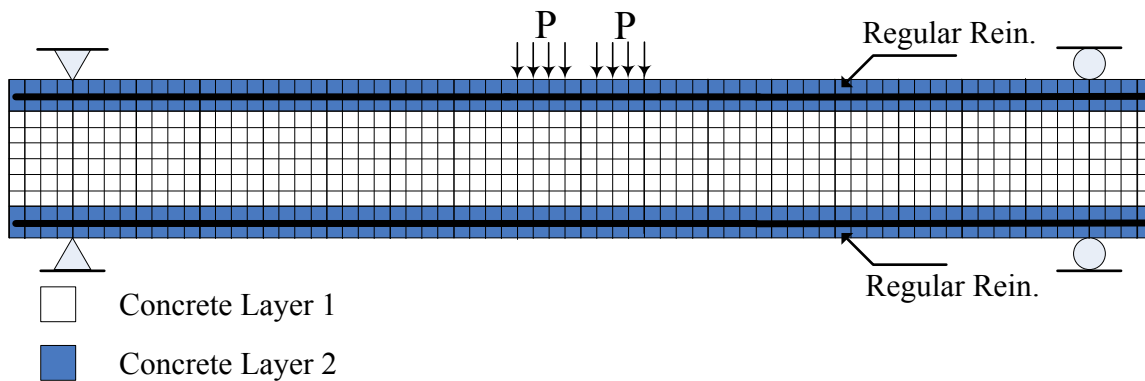
The theoretical bases of VecTor2 are the Modified Compression Field Theory (MCFT) (Vecchio et al. 1986) and the Disturbed Stress Field Model (DSFM) (Vecchio 2000) – analytical models for predicting the response of reinforced concrete elements subject to in-plane normal and shear stresses. VecTor2 models cracked concrete as an orthotropic material with smeared, rotating cracks. The program utilizes an incremental total load, iterative secant stiffness algorithm to produce an efficient and robust nonlinear solution. Originally, VecTor2 employed the constitutive relationships of the MCFT. Subsequent

developments have incorporated alternative constitutive models for a variety of second-order effects including compression softening, tension stiffening, tension softening, and tension splitting. In addition, the capabilities of VecTor2 have been augmented to model concrete expansion and confinement, cyclic loading and hysteretic response, construction and loading chronology for repair and retrofit applications, bond slip, crack shear-slip deformations, reinforcement dowel action, reinforcement buckling, and crack allocation processes. More recently, dynamic (earthquake, impact and blast loading) capabilities have been incorporated. Generally, the finite element model constructed for VecTor2 analysis consists of a fine mesh with simple elements. This methodology has advantages of computational efficiency and numerical stability. It is also well suited to reinforced concrete structures, which require a relatively fine mesh to model reinforcement detailing and crack patterns. The element library includes a three-node constant strain triangle element, a four-node plane stress rectangular element and a four-node quadrilateral element for modeling concrete with smeared reinforcement; a two-node truss-bar for modeling discrete reinforcement; and a two-node link and a four node contact element for modeling bond-slip between reinforcement and concrete. In addition to VecTor2, FormWorks (Wang et al. 2002) and Augustus (Wang et al. 2002) were used during the pre-processing and post-processing stages of the analyses, respectively. FormWorks is a pre-processor software designed to provide a user-friendly interface for generating the input files needed by VecTor2, and visualizing the geometry and checking of the finite element model. Augustus provides graphical post-processing capabilities for the visualization of stresses, strains, cracks, and other behavioral features predicted with VecTor2.

## 7.3 Modeling of Experimental Specimens

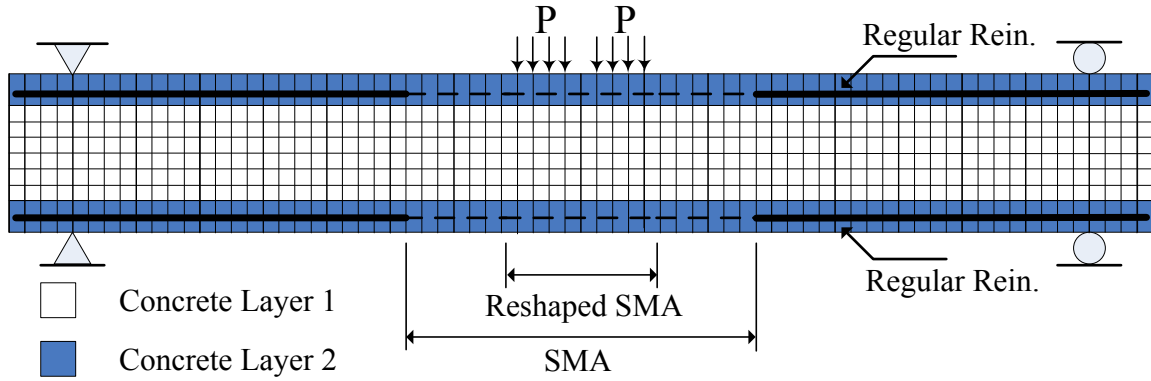
### 7.3.1 Beam Specimens

Analyses were conducted on the conventional and SMA reinforced beams. The beams were modeled with rectangular plane stress concrete elements. The transverse shear reinforcement was considered smeared within the concrete elements, and the longitudinal flexural reinforcement was modeled with discrete truss bar elements. A total of 1008 rectangular elements were used for the concrete and 224 truss elements to model the top and bottom longitudinal reinforcement. The truss bars were assumed perfectly bonded to the surrounding concrete for modeling conventional steel reinforcement while they were partially bonded for modeling the SMA given the smooth surface of the bars leading to lower bond stresses. The simulated loading followed the same protocol as used during testing. The finite element meshes developed in VecTor2 for modeling the conventionally reinforced beam (B3-SR) and SMA reinforced beam (B6-NR) are shown in Figures 7.1 and 7.2, respectively. Modeling details are presented in the subsequent subsections.



**Figure 7.1** Finite Element Mesh for Conventional Beam B3-SR





**Figure 7.2** Finite Element Mesh for SMA Beam B6-NR

#### 7.3.1.1 Concrete Elements

Plane stress rectangular elements were used to model both reinforced and plain concrete. The dimensions of the elements were 25 mm x 25 mm. The sizes of the concrete elements were chosen to coincide with the centrelines of the longitudinal steel reinforcement, while maintaining an aspect ratio of 1 for the rectangular elements. Uniformly distributed transverse reinforcement was modeled as smeared reinforcement within the concrete elements.

Two different concrete types were used due to varying reinforcement ratios. Concrete Type 1 (Concrete Layer 1) was assigned to the concrete elements between the top and bottom longitudinal reinforcement and contained smeared reinforcement in the vertical direction to account for shear reinforcement. The two concrete layers adjacent to the longitudinal reinforcement were assigned to Concrete Type 2 (Concrete Layer 2). These elements contained smeared reinforcement in the vertical direction due to the stirrups and smeared reinforcement in the out-of-plane direction due to the out-of-plane legs of the stirrups. The latter explicitly accounts for the confining effect near the edges of the beams

due to the closed stirrups. The locations of each material type are shown in Figures 7.1 and 7.2.

#### 7.3.1.2 Reinforcement Elements

The longitudinal conventional steel reinforcement and SMA bars were modeled using discrete truss elements. The truss elements are two-node elements with uniform cross-sectional area. Each node has two degrees of freedom (X and Y), giving each element a total of four degrees of freedom. The length of each truss element was 25 mm to coincide with the nodes of the concrete rectangular elements. Recall that the nodal geometry was governed by the concrete element geometry. A total of 224 truss elements were used. Figures 7.1 and 7.2 illustrate the steel and SMA modeling layout in the conventional and SMA beams, respectively. In the conventional beams, the longitudinal reinforcement was designed to reach the full yield capacity by providing adequate development lengths and hooks. Although the hooks were not explicitly modeled, they were simulated by assuming a perfect bond between the reinforcement with the concrete. Conversely, the smoothness of the SMA bars in the superelastic beams dictated modeling the bond condition for the entire length of the SMA bar, while the couplers, which connected the SMA with the conventional steel (Figure 3.4), were represented by a truss bar with the same length, material properties, and cross-sectional steel area as the couplers. The conventional deformed reinforcement and the couplers in SMA beams were assumed perfectly bonded to the surrounding concrete, while the bond between the SMA bars and the concrete was modeled with bond-slip elements which serve as deformable interfaces between concrete elements and discrete reinforcement elements. Prior to slippage, bond-slip elements are defined by paired nodes having the same coordinates. A concrete

element is attached to one of the nodes, and a discrete reinforcement element is attached to the other node.

Two different reinforcement types were used to account for varying reinforcement areas and material properties in the conventional beams. Reinforcement Type 1 was assigned to the bottom longitudinal rebar, while Type 2 was allocated for the top longitudinal steel. In the SMA beams, Types 1 and 2 refer to the bottom and top conventional reinforcement, respectively, while Types 3 and 4 were assigned to the SMA bars and reshaped SMA bars, respectively.

#### 7.3.1.3 Support Conditions

In the experimental setup, the concrete beams were simply supported at the ends with the freedom to move horizontally. Both ends were free to rotate but prevented from moving vertically. The base of the supports was attached to the laboratory strong floor through four bolts. The supports were modeled with vertical and horizontal restraints (represented by pins) at one end of the beam, while the other end was provided with a vertical restraint (represented by rollers) only, as shown in Figures 7.1 and 7.2. The restraints were provided at one node only, which was sufficient to simulate the behavior of the supports during the analysis. No local concrete crushing as a result of the support simulation was predicted in the analyses.

#### 7.3.1.4 Loading Conditions and Procedures

In the experimental setup, two vertical loads, spaced 125 mm, were applied uniformly at the midspan of the beams using two loading plates. In the finite element model, four nodes were assigned to represent each applied load at the top of the beam (Figures 7.1

and 7.2). The centroid of the load imposed for analysis was consistent with that imposed during testing. The distributed load was intended to represent the spreading of the load due to the loading plate and to prevent local crushing of the concrete. The analysis was conducted with displacement control in increments of 1 mm to allow the analysis to capture the post-peak behavior.

#### 7.3.1.5 Concrete, Reinforcement and Bond Constitutive Models

The constitutive material models for the concrete, reinforcement and bond behavior are summarized in Table 7.1. The constitutive models with asterisks indicate that models other than the default options in VecTor2 were selected. The FormWorks (Wang et al. 2002) manual provides detailed descriptions of each model.

**Table 7.1** Material and Behavioral Models

| <b>Material Property</b>          | <b>Material Model</b>          |
|-----------------------------------|--------------------------------|
| Concrete Compression Base Curve   | Popovics (NSC)*                |
| Compression Post-Peak             | Base Curve*                    |
| Concrete Compression Softening    | Vecchio 1992-A                 |
| Concrete Tension Stiffening       | Modified Bentz                 |
| Concrete Tension Softening        | Linear                         |
| Concrete Tension Splitting        | Not Considered                 |
| Concrete Confinement Strength     | Kupfer / Richart               |
| Concrete Dilatation               | Variable - Kupfer              |
| Concrete Cracking Criterion       | Mohr-Coulomb (Stress)          |
| Concrete Crack Width              | Agg/5 Max Crack Width          |
| Concrete Hysteresis               | Non-Linear with Plastic Offset |
| Slip Distortion                   | Vecchio-Lai                    |
| Reinforcement Hysteretic Response | Seckin Model (Bauschinger)     |
| Reinforcement Dowel Action        | Tassios Model (Crack Slip)     |
| Reinforcement Buckling            | Asatsu Model                   |
| Concrete Bond                     | Eligehausen Model              |

Typically, the default models are chosen unless an alternative model better captures the behavioral response (Palermo and Vecchio 2007, and Cortes and Palermo 2012). For all analyses conducted, the default options were selected, except the concrete compression base curve model and the corresponding compression post-peak model. Pre-peak and post-peak responses of Popovics (Popovics 1973) normal strength model (Popovics NSC) were chosen given the design strength of the concrete used in the beams (30 MPa). These models have successfully been used for the prediction of the response of reinforced concrete structures with concrete strengths ranging from 25 MPa to 45 MPa..

Secondary effects of concrete were modeled with the default models provided in VecTor2 (Table 7.1). Compression softening, which accounts for the reduction in compressive strength and stiffness due to co-existing transverse tensile straining was modeled with Vecchio's 1992-A model. Tension stiffening was modeled by the modified Bentz model, which determines the tensile stresses that exist in concrete between cracks due to the bond action between the reinforcement and the concrete. The post-cracking tensile stresses present in plain concrete, known as tension softening, was modeled with a linear descending response after cracking. The lateral expansion of concrete caused by internal micro cracking, which increases as the compressive stresses increase, was modeled with a variable Poisson's ratio based on the work of Kupfer. In addition, the Kupfer/Richard confined strength model accounted for the strength enhancement of concrete in compression due to tri-axial stress conditions. The cracking strength of concrete which can differ from the concrete tensile strength due to co-existing transverse compressive stresses was considered with the Mohr-Coulomb stress model. Consideration was also given to the width of cracks, which can cause a reduction in the

compressive strength when crack widths exceed a specified limit; a limit of 20% of the aggregate size was used. The crack shear-slip deformations, which represent the main compatibility feature of the DSFM was evaluated with the Vecchio-Lai model.

For steel reinforcement, the Seckin Model was selected to capture the hysteretic response of the reinforcement. The model includes a linear elastic region followed by a yield plateau and strain hardening. The unloading and reloading response includes the Bauschinger effect. Dowel action of the reinforcement, which arises from crack slip occurring transversely to the axes of the reinforcement, was considered using the Tassios crack-slip model. The Asatsu model was used to capture buckling of internal reinforcement which requires the use of bond-link elements between the truss bar elements and the concrete elements.

Bond between conventional reinforcement and concrete was not modeled; perfect bond conditions were assumed. For the SMA beam, the models used in the analysis were the same as those used for the conventional beams, except for explicit modeling of the bond between the SMA and surrounding concrete, and the reinforcement hysteretic response for the SMA bars. The concrete bond was modeled by the Eligehausen model, which defines the bond relationship between concrete elements and discrete reinforcement elements as a series of bond stresses and bond slips. Within VecTor2, the bond stresses and corresponding slips to model smooth embedded bars is available. The hysteretic simplified model proposed in this research program as described in Section 6.2 was chosen to model the SMA hysteretic response.

VecTor2 requires certain material properties for concrete, reinforcement and bond to be defined. For concrete, only the compressive strength is necessary; the program internally

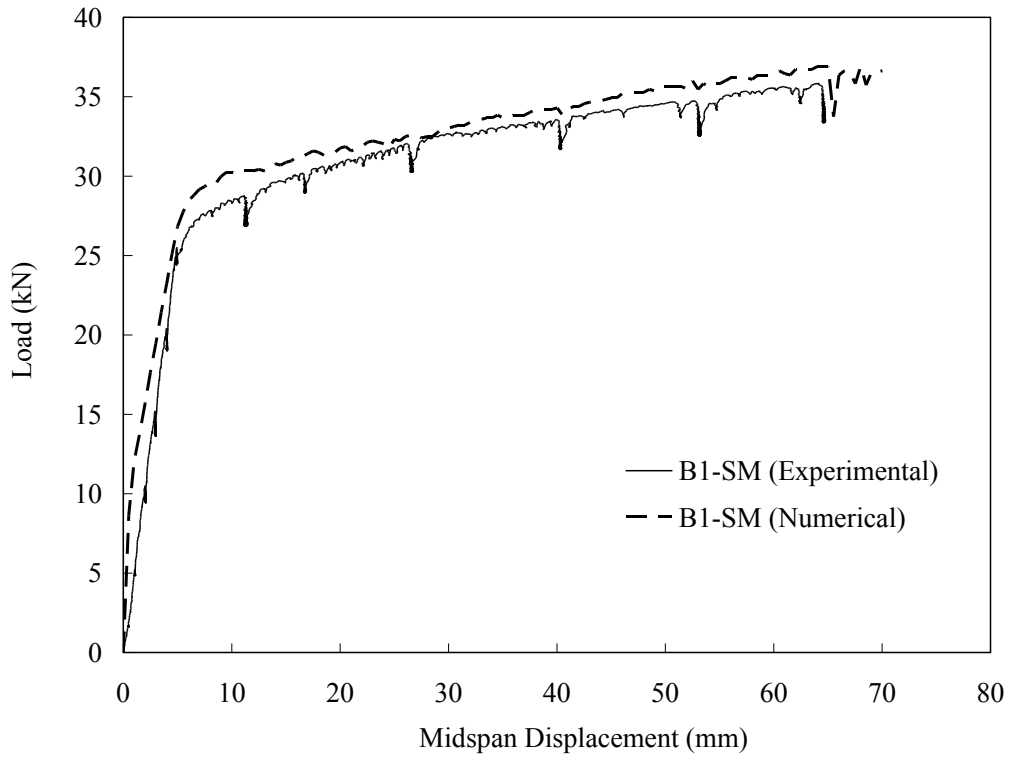
evaluates other concrete properties, such as tensile strength, initial elastic modulus, poison's ratio, and thermal coefficient of expansion. All reinforcement properties must be defined; no default values are available except the thermal coefficient of expansion for the reinforcement. Note that for reinforcement, material properties are readily available for the types of reinforcement typically modeled. Bond properties require the compressive strength of concrete, bar diameter and distribution of the reinforcement.

### 7.3.2 Analysis Results of Beams

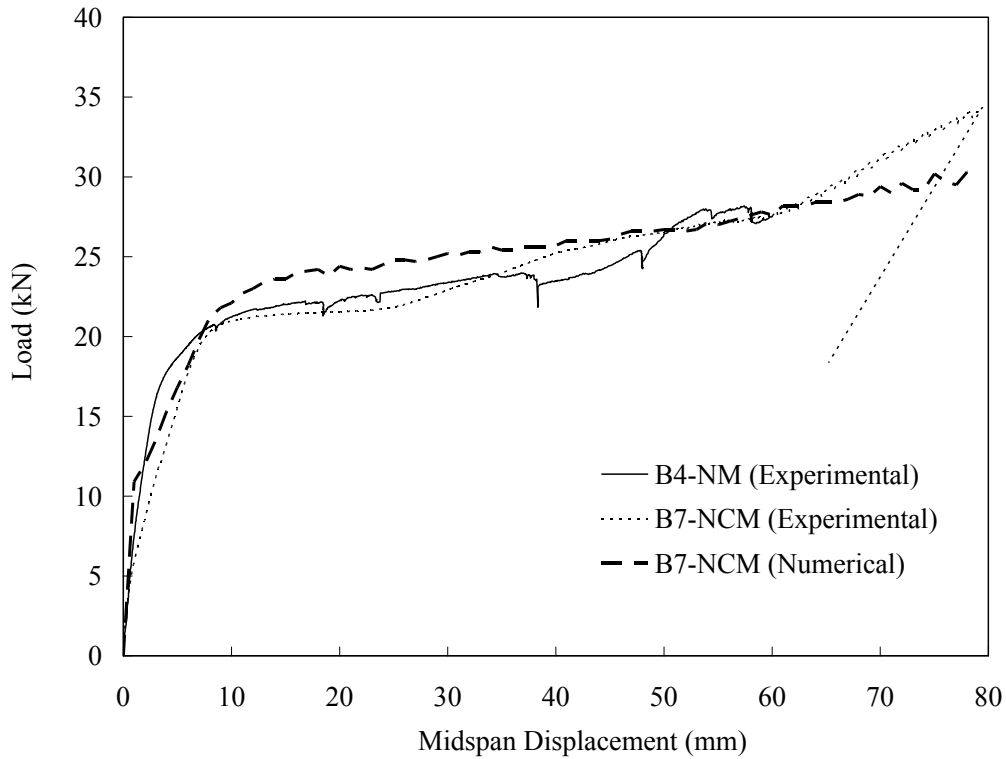
The ultimate strength capacity, ductility, plastic deformation, crack pattern and energy dissipation for the conventional and SMA reinforced beams were numerically investigated and compared to the experimental results in this section.

#### 7.3.2.1 Load–Displacement Response

The numerical and experimental load-displacement responses under monotonic loading for conventional Beam B1-SM and SMA Beams B4-NM and B7-NCM are shown in Figures 7.3 and 7.4, respectively.



**Figure 7.3** Load-Displacement Responses for Conventional Beam B1-SM

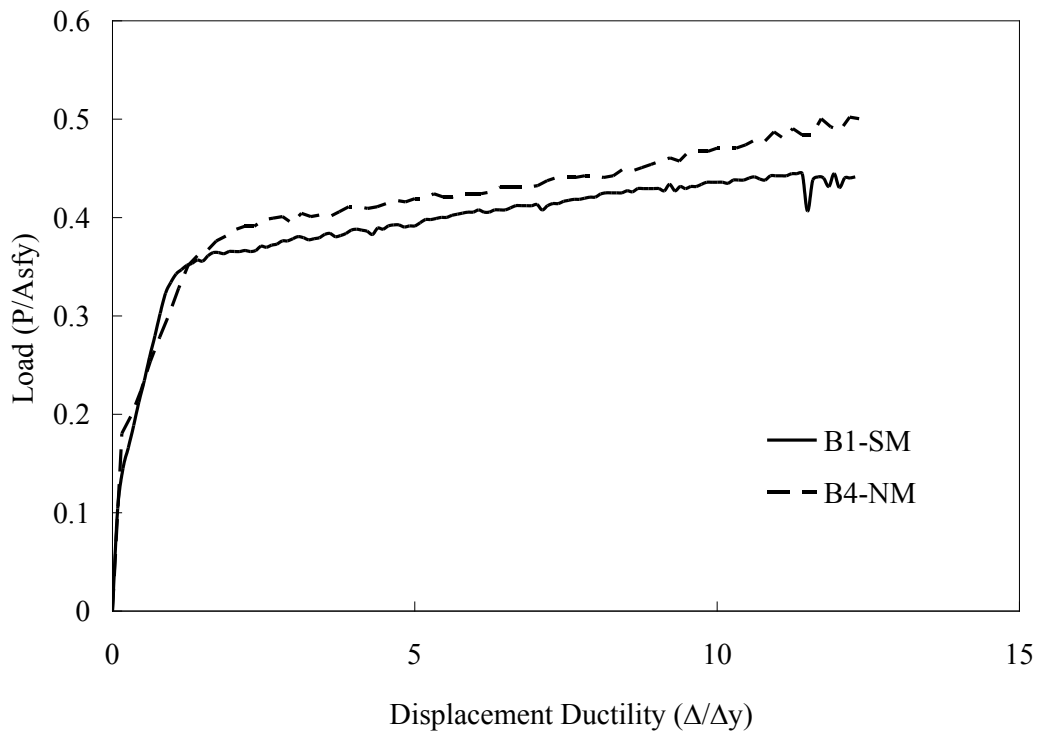


**Figure 7.4** Load-Displacement Responses for SMA Beams B4-NM and B7-NCM



The analyses satisfactorily captured the behavior observed during testing including strength and ductility capacities. In general, the FE model predicted higher initial stiffness than the experimental results in both the conventional and SMA beams. At yield displacement  $\Delta_y$ , the predicted load was 6% greater than the experimental load in the conventional beam B1-SM, while it was 4% and 3% greater than the experimental load in the SMA beams B4-NM and B7-NCM, respectively.

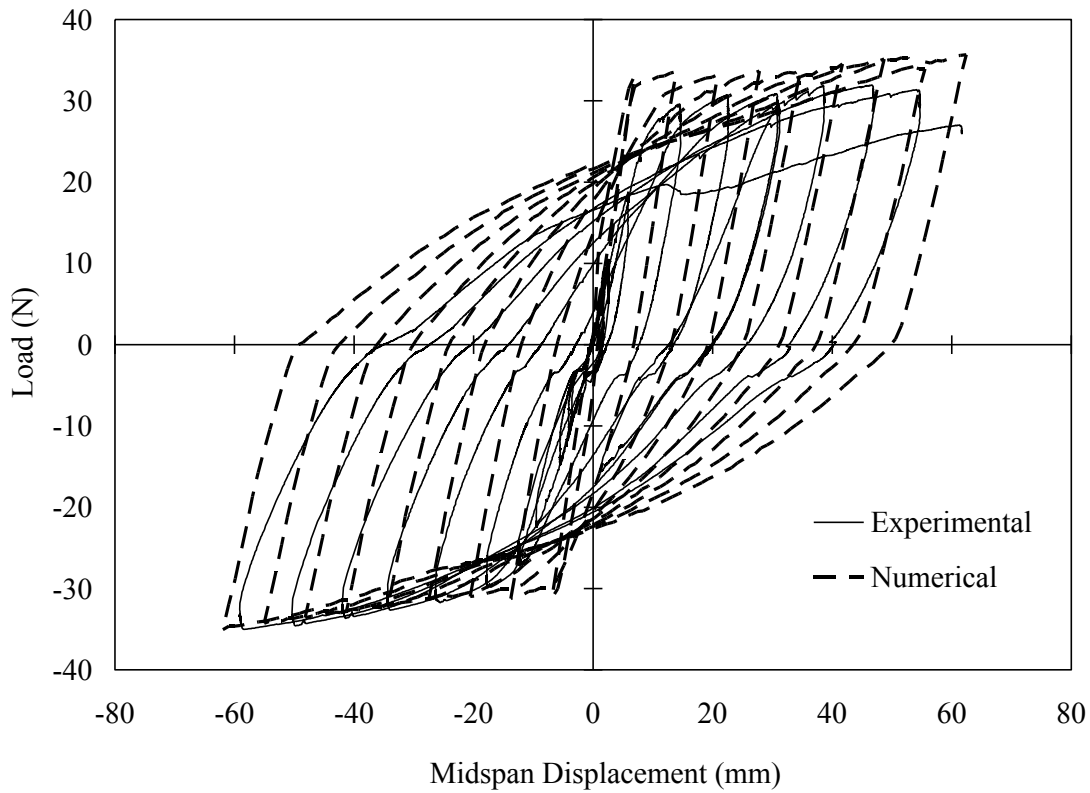
From the normalized numerical responses (Figure 7.5), it is evident that the initial pre-yielding stiffness is lower in the SMA beam. This was due to the lower modulus of elasticity in the SMA. Consequently, higher yield deflections were experienced in the SMA reinforced beams. At yield, the midspan deflection in the SMA beam was approximately 22% higher than the corresponding conventional beam.



**Figure 7.5** Numerical Normalized Load-Displacement Responses for Monotonic Tests

The post-yield responses are similar. The ultimate load of the conventional and SMA beams are similar due to the load normalization and similar concrete properties. When compared with the experimental normalized results (Figure 4.5), the numerical responses demonstrate slightly larger normalized load capacity in both SMA and conventional beams while similar ductility capacity was achieved.

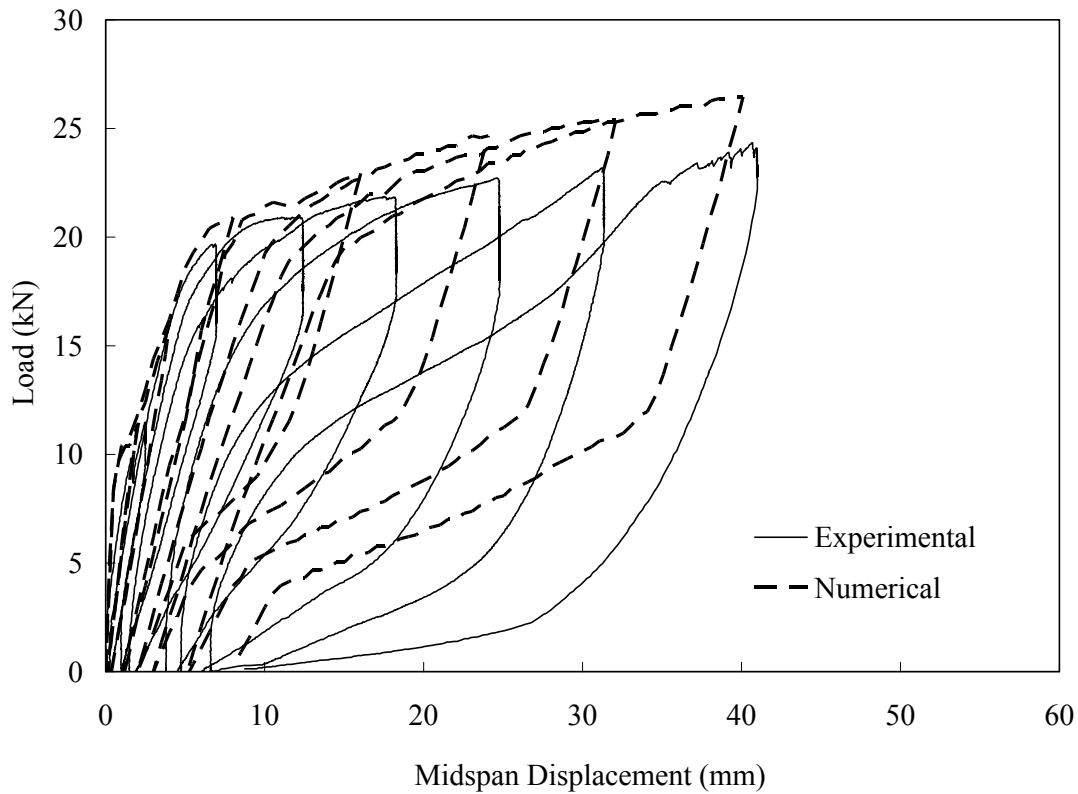
Figure 7.6 shows the conventional reinforced beam under reverse cyclic loading. The analysis predicted slightly greater strength and higher initial stiffness than the experimental results. The unloading and reloading curves were well simulated. The predicted yield strength of 29 kN was 7% greater than the experimental load; whereas, the analysis predicted an ultimate load of 36 kN, which was 3% higher than the experimental. Both beams failed by concrete crushing after significant yielding of the longitudinal reinforcement but at slightly different displacements. The maximum predicted displacement was 63 mm, while the ultimate displacement recorded during testing was 56 mm at failure. Permanent displacements were satisfactory captured during the analysis; however, more pinching was observed in the experimental hysteretic response.



**Figure 7.6** Load-Displacement Responses for Conventional Beam B3-SR

Discrepancies noted in the numerically predicted response can be improved by a more refined finite element mesh, a parametric study to select more appropriate material models, and more accurate modeling of the loading and support conditions. In general, though, the numerical response of the conventional reinforced beam demonstrates the applicability of the finite element model and VecTor2 adopted for this study.

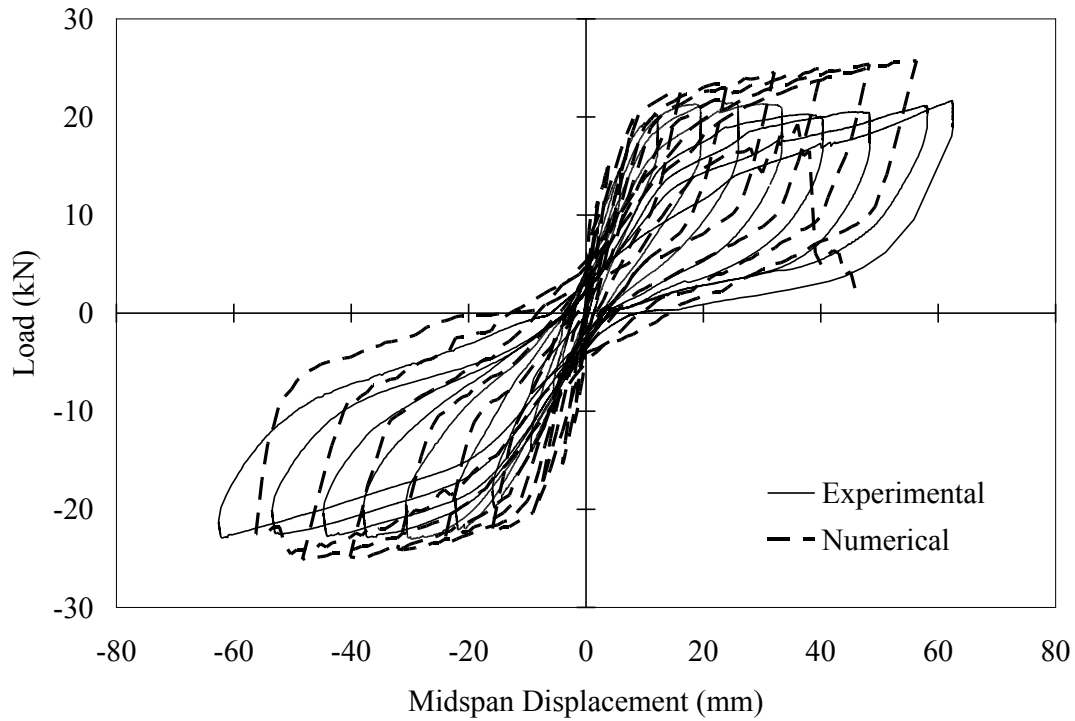
The calculated response of the SMA reinforced beams indicates that the analysis reasonably predicted the behavior observed during testing. Figure 7.7 shows the experimental and numerical responses for Beam B7-NCM. The predicted yield and ultimate strengths of 20.4 kN and 26.5 kN, respectively, were only 7% and 9% greater than that experimentally recorded.



**Figure 7.7** Load-Displacement Responses for SMA Beam B7-NCM

The residual displacements upon unloading were accurately predicted. The unloading curves were well simulated at early stages of loading. With increased loading, discrepancies are evident. The initial unloading response was better simulated than the unloading plateau stress. The analysis was terminated at approximately 40 mm of midspan displacement and was not pushed to failure, which was consistent with testing.

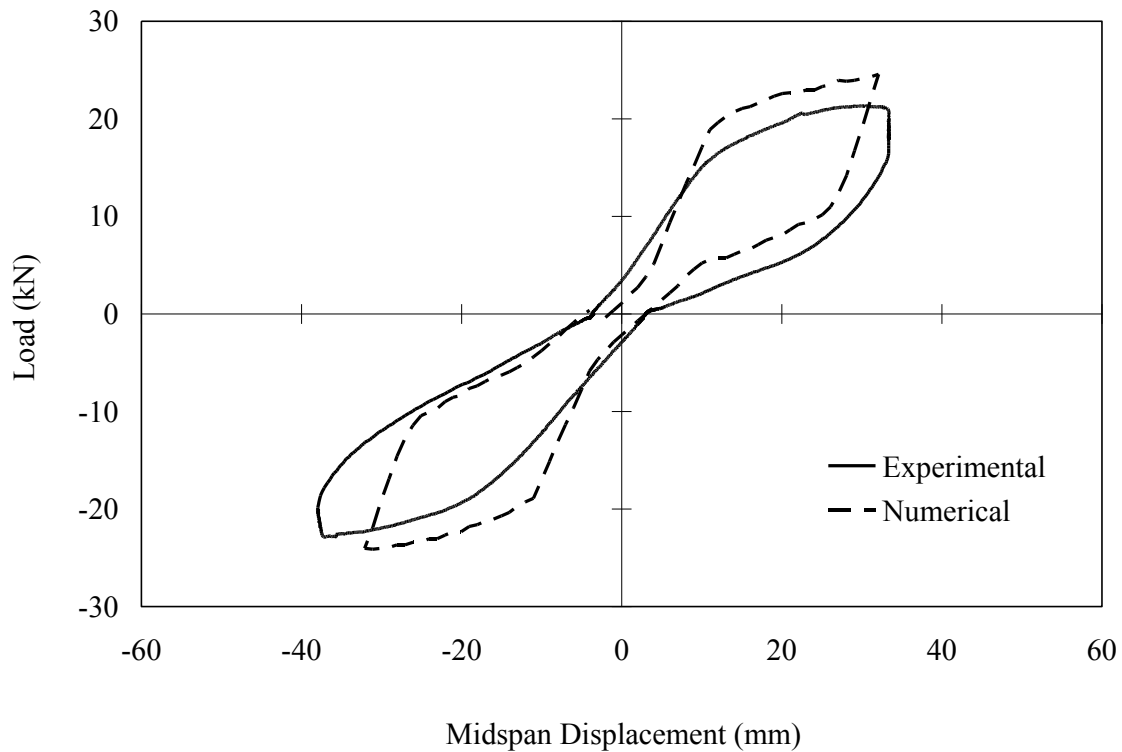
Figure 7.8 provides the experimental and numerical responses of Beam B6-NR. The hysteretic behavior was well simulated, including peak strength, ductility, residual displacements, and unloading response.



**Figure 7.8** Load-Displacement Responses for SMA Beam B6-NR

The predicted yield strength of 18 kN was 9% greater than experimentally recorded; whereas, the analysis predicted an ultimate load of 25.7 kN, which was 20% higher than experimentally observed. The discrepancy in the ultimate load capacity can be attributed to the fact that the SMA beam experienced a major crack outside the critical region during the test which resulted in a reduction in strength.

To further assess the proposed SMA model, the numerical and experimental responses corresponding to the displacement at the recorded peak strength for Beam B6-NR are shown in Figure 7.9.



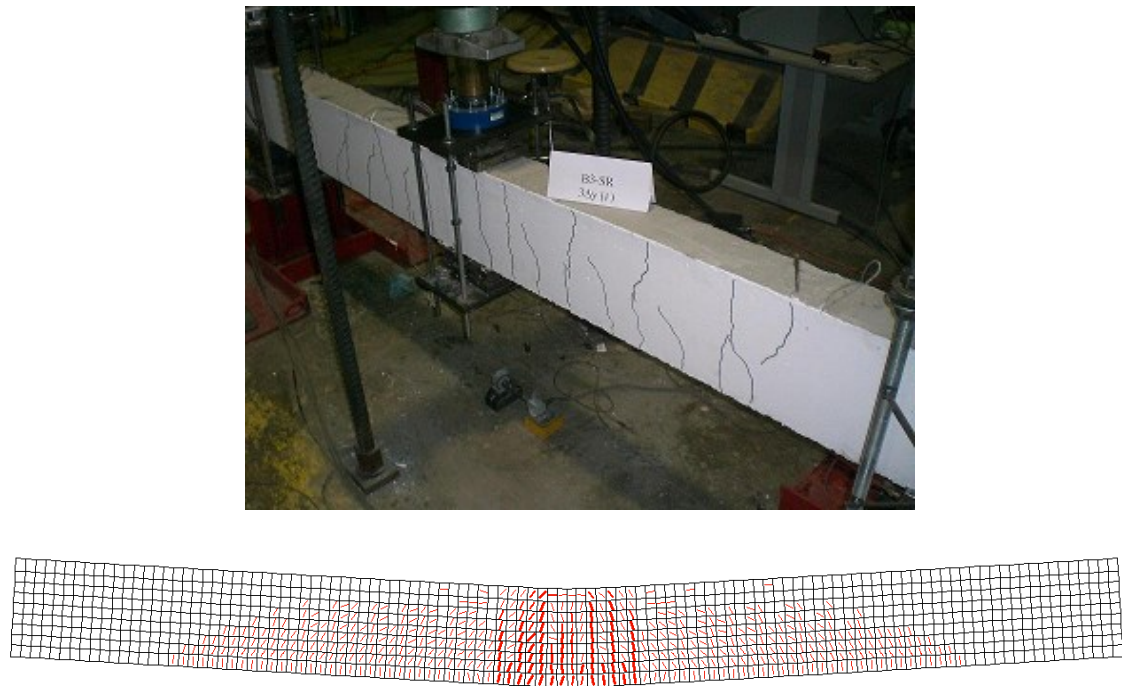
**Figure 7.9** Load-Displacement Responses at Peak Strength for SMA Beam B6-NR

The numerical analysis satisfactorily simulated the response of Beam B6-NR at peak load. The analysis predicted a load of 24.5 kN, while the corresponding strength recorded during testing was 21.6 kN. The unloading response was reasonably simulated and the predicted plastic displacement was accurately captured even though the SMA Model is based on a tri-linear unloading behavior.

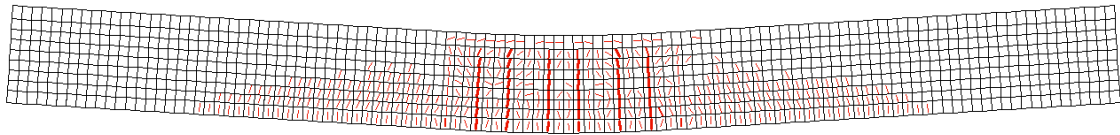
#### 7.3.2.2 Crack Patterns and Failure Modes

The numerical models were also assessed by comparing the cracking patterns with those observed during testing. Generally, the finite element results provided good predictions of the sequence and locations of the first flexural and shear cracks. For comparison purposes, the first numerical crack was defined by a crack width of 0.10 mm, which

corresponds to the visible crack width during testing. Experimentally, flexural cracking at midspan was observed first followed by inclined shear cracks near the supports. Numerically, the sequence of cracking was similar to that observed in the conventional and SMA beams, except B4-NM where local effects from the presence of the couplers governed the location of critical damage. This resulted in the first flexural crack initiating at the coupler location where failure was observed. The numerical and experimental crack patterns for the conventional Beam B3-SR and SMA Beam B6-NR are shown in Figures 7.10 and 7.11, respectively.



**Figure 7.10** Cracking Patterns in Conventional Beam B3-SR



**Figure 7.11** Cracking Patterns in SMA Beam B6-NR

The numerical results predicted closely spaced cracks in the conventional reinforced beam and significantly farther spaced cracks in the SMA reinforced beam. These trends were consistent with observations noted during testing.

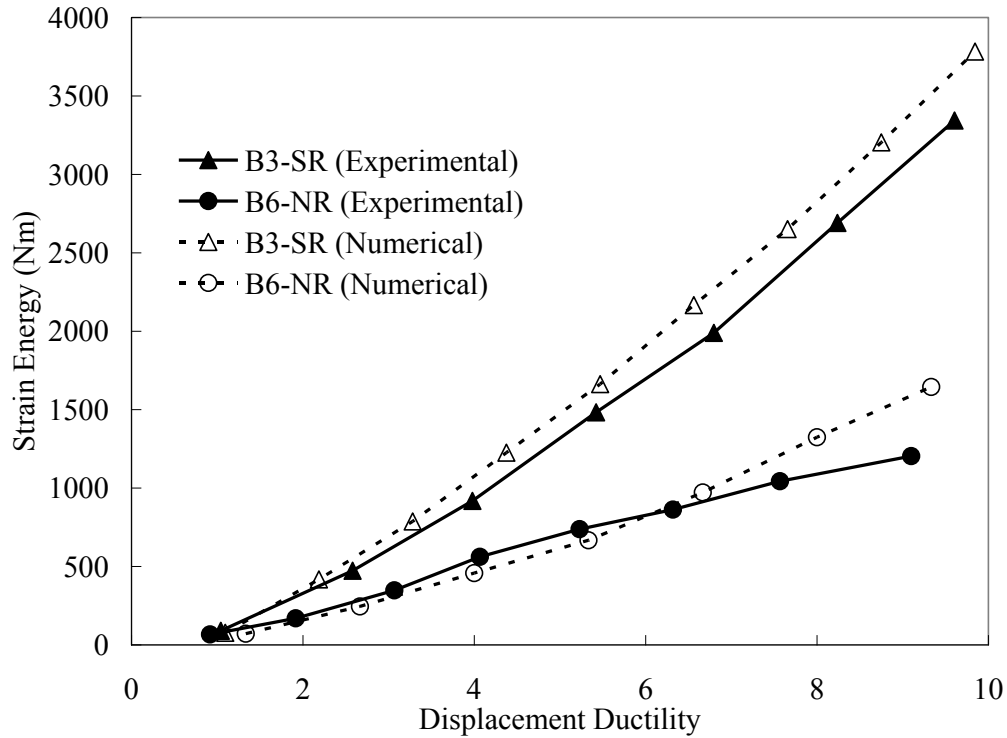
All beams were designed as flexural members that experience significant reinforcement yielding prior to concrete crushing. This type of failure was predicted in the finite element analysis for all conventional and SMA beams. Experimentally, the beams had similar failure modes except B1-NM and B6-NR. Beam B1-NM experienced failure by rupturing of the SMA at the coupler, and Beam B6-NR experienced failure by rupturing of the SMA at the transition point where the SMA changed diameter.

#### 7.3.2.3 Dissipated Energy

The dissipated energy was calculated from the force-displacement hysteresis curves for the reverse cyclic analysis of B3-SR and B6-NR and compared with the experimental

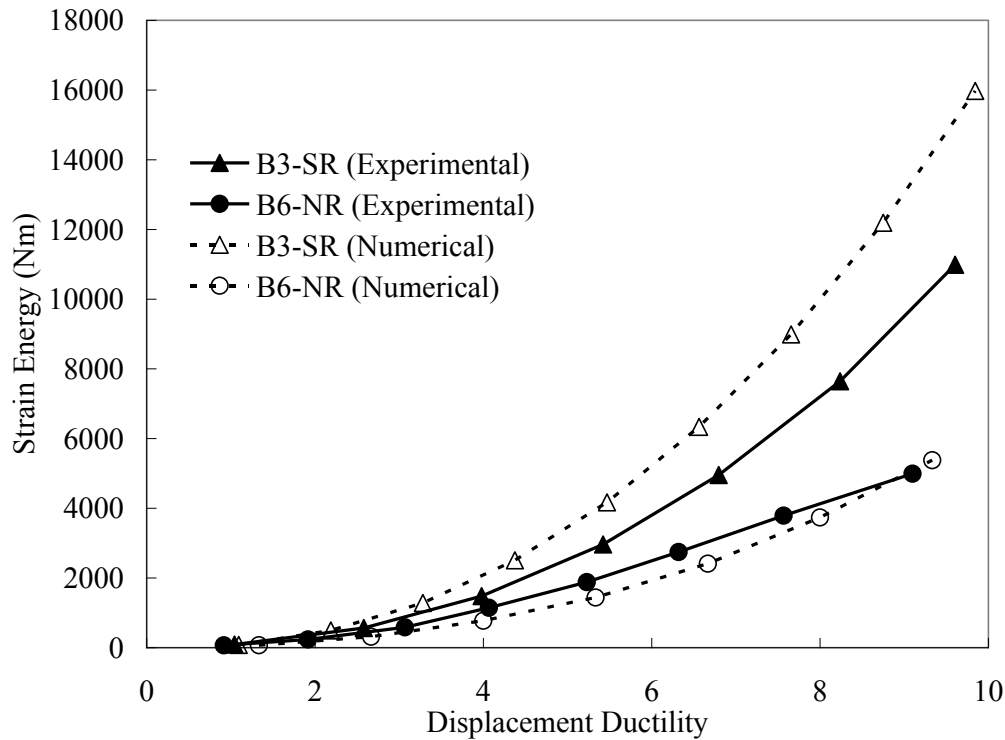


results. The energy dissipation was calculated for both specimens at each displacement ductility level as shown in Figure 7.12.



**Figure 7.12** Energy Dissipation-Displacement Ductility Responses

In both specimens, B3-SR and B6-NR, the numerical and experimental results for dissipated energies were in close agreement, which demonstrates the adequacy of the finite element program and the proposed hysteretic constitutive SMA model to capture the behavior of the specimens. The cumulative hysteretic energy dissipation was also satisfactorily captured by the numerical model as shown in Figure 7.13.



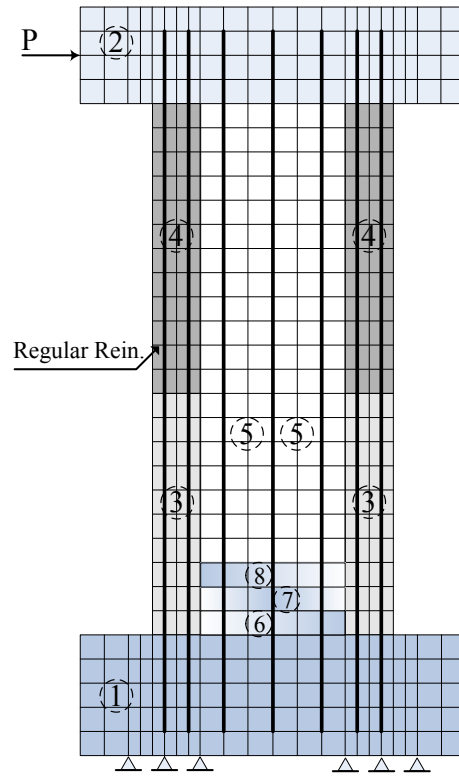
**Figure 7.13** Cumulative Energy Dissipation-Displacement Ductility Responses

At the ultimate ductility, the predicted dissipated energy in B6-NR was 36% greater than the experimental results. However, this difference was reduced to only 7% for the cumulative energy dissipation. In the conventional beam B3-SR, at the ultimate ductility, the predicted strain energy and cumulate strain energy were higher than the experimental by 13% and 42%, respectively.

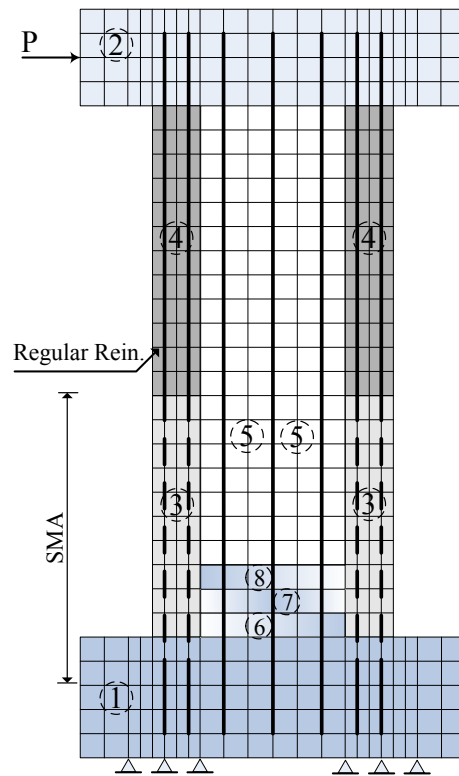
### 7.3.3 Shear Wall Specimens

The conventional and SMA reinforced concrete walls were discretized into finite number of elements in order to perform the numerical analysis. The finite element mesh of the shear walls consisted of different layers of concrete and steel elements to simulate the wall specimens. Plane stress elements were used to model the concrete layers. The

transverse reinforcement was considered smeared within the concrete elements, while the longitudinal reinforcement in the web and the boundary zones were modeled with truss bar elements. A total of 506 rectangular plane stress elements were used to model the concrete and 203 truss elements were used to model the longitudinal reinforcement in the web and boundary zones. Truss elements representing the conventional steel reinforcement were assumed perfectly bonded to the surrounding concrete, while they were assumed partially bonded for the SMA reinforcement. The loaded imposed in the numerical models followed the same protocol as used during testing and in accordance with the recommendations of ATC-24. The numerical models developed in VecTor2 for the conventional and SMA shear walls are shown in Figures 7.14 and 7.15, respectively. Modeling details are presented in the subsequent subsections.



**Figure 7.14** Finite Element Mesh for Conventional Wall W1-SR



**Figure 7.15** Finite Element Mesh for SMA Wall W2-NR

#### 7.3.3.1 Concrete Elements

The finite element mesh for both the conventional shear wall (Wall W1-SR) and the SMA shear wall (Wall W2-NR) consisted of 100 mm x 100 mm rectangular plane stress elements. An aspect ratio of 1 was maintained for all concrete zones except the boundary zones, which were discretized with 50 mm x 100 mm elements. Meshing of the foundation and top loading beam of the shear walls conformed to the mesh of the walls. The mesh was divided into eight concrete zones to account for the variation in concrete thicknesses and reinforcement ratios as shown in Figures 7.14 and 7.15. The first and the second zones correspond to the heavily reinforced foundation and top beam respectively; the third and the fourth zones correspond to the boundary elements of the wall; and the fifth zone corresponds to the web portion of the wall. The remaining three zones were used to model the extension of the starter bars into the web.

Vertical, transverse, and out-of-plane reinforcement were smeared in concrete zones 1 and 2. Concrete zones 3 and 4 contained smeared reinforcement in the transverse and out-of-plane direction. The latter reinforcement accounts for the confinement effect caused by the closed ties in the boundaries. Note that zone 3 contains more closely spaced ties and transverse bars. Concrete zone 5 has smeared reinforcement in the horizontal direction to represent the uniformly distributed transverse reinforcement. Concrete zones 6, 7, and 8 have the same properties as zone 5 with the addition of varying reinforcement ratios in the vertical direction to account for the starter bars which were modeled to have variable ratios in each zone proportional to their development lengths.

#### 7.3.3.2 Reinforcement Elements

The longitudinal steel reinforcement and SMA bars were modeled using discrete, two-node truss bar elements. The screw lock couplers were represented by a truss bar with the same length, material properties, and cross-sectional area as the couplers.

Figures 7.14 and 7.15 illustrate the steel and SMA modeling layout in the conventional and SMA walls, respectively. Truss elements of 100 mm in length were used to model the longitudinal reinforcement to coincide with the nodes of the rectangular concrete elements. The conventional steel bars and the couplers were assumed perfectly bonded to the surrounding concrete. Partially bonded truss bars were used for the SMA due to the smoothness of the bar surface. Bond-slip elements were used at the interface of the SMA truss elements and the rectangular concrete elements to properly model the bond characteristics.

In the conventional wall W1-SR, a single type of reinforcement was required to define the longitudinal reinforcement, whereas four different reinforcement types were used with varying reinforcement areas and material properties in the SMA wall W2-NR. Reinforcement Type 1 was assigned to SMA bars, while reinforcement Type 2 and 3 were assigned to the connecting couplers and the steel reinforcement on either side of the couplers, respectively. Reinforcement Type 4 was allocated for the longitudinal steel in the web portion of the wall.

Available constitutive models in VecTor2 for concrete, conventional and SMA reinforcement, and bonding interface were selected for the analysis of the walls as detailed in Section 7.3.1.5.

#### 7.3.3.3 Support Conditions

The base of the shear wall specimens were connected to the lab strong floor by means of four high strength bolts. To model this connection, six nodes around each bolt at the bottom of the foundation were restrained in the vertical and horizontal directions to simulate the fixed support condition of the shear walls. All other base nodes outside the bolt region were considered free to move.

#### 7.3.3.4 Loading Conditions and Procedures

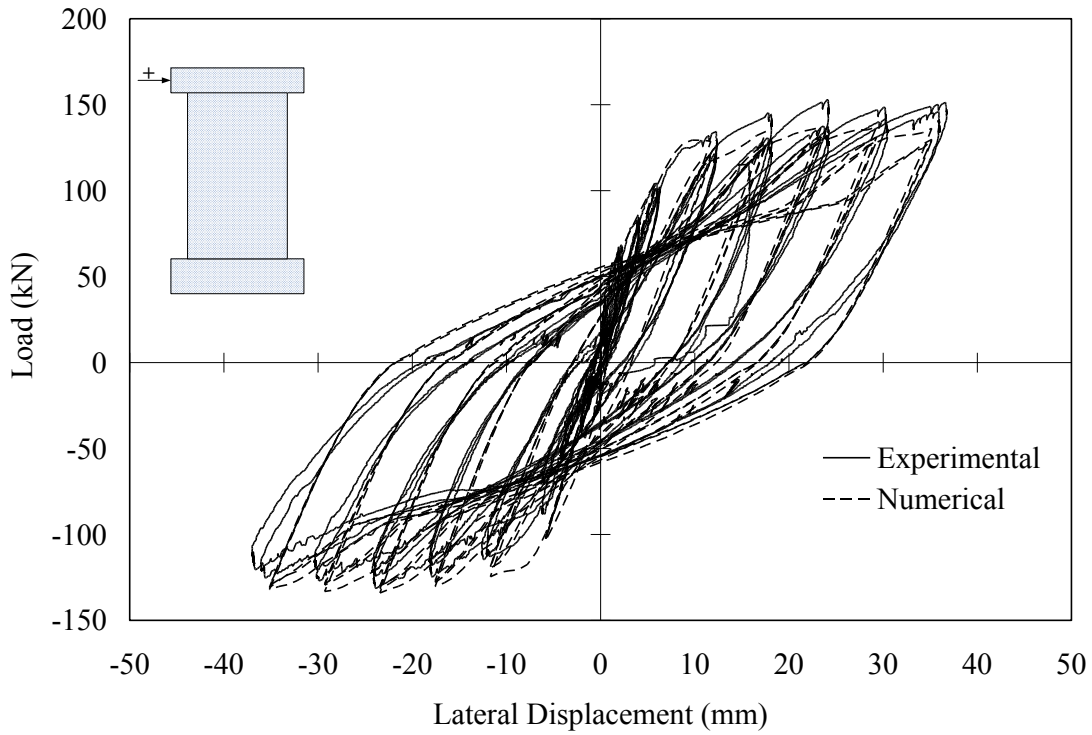
The shear wall specimens were tested in displacement-controlled mode using a displacement controlled actuator positioned at the centerline of the top beam. In the finite element model, the loading consisted of incremental reverse cyclic lateral displacements applied at the center line of the top beam to simulate the experimental loading position. The loading started with displacement increments of 2 mm up to 6 mm followed by displacement increments of 6 mm up to 60 mm and finally by displacement increments of 12 mm up to failure. Three repetitions were imposed at each displacement level up to 60 mm and two repetitions thereafter. Self weight of the wall was considered in the analysis. The analysis of the SMA wall W2-NR was performed to failure, while the conventional wall W1-SR was analyzed up to a displacement level of 36 mm to simulate the actual test.

#### 7.3.4 Analysis Results of Shear Walls

The ultimate lateral strength capacity, ductility, plastic displacement, crack pattern and energy dissipation for the conventional and SMA reinforced shear walls were numerically investigated and compared to the experimental results in this section.

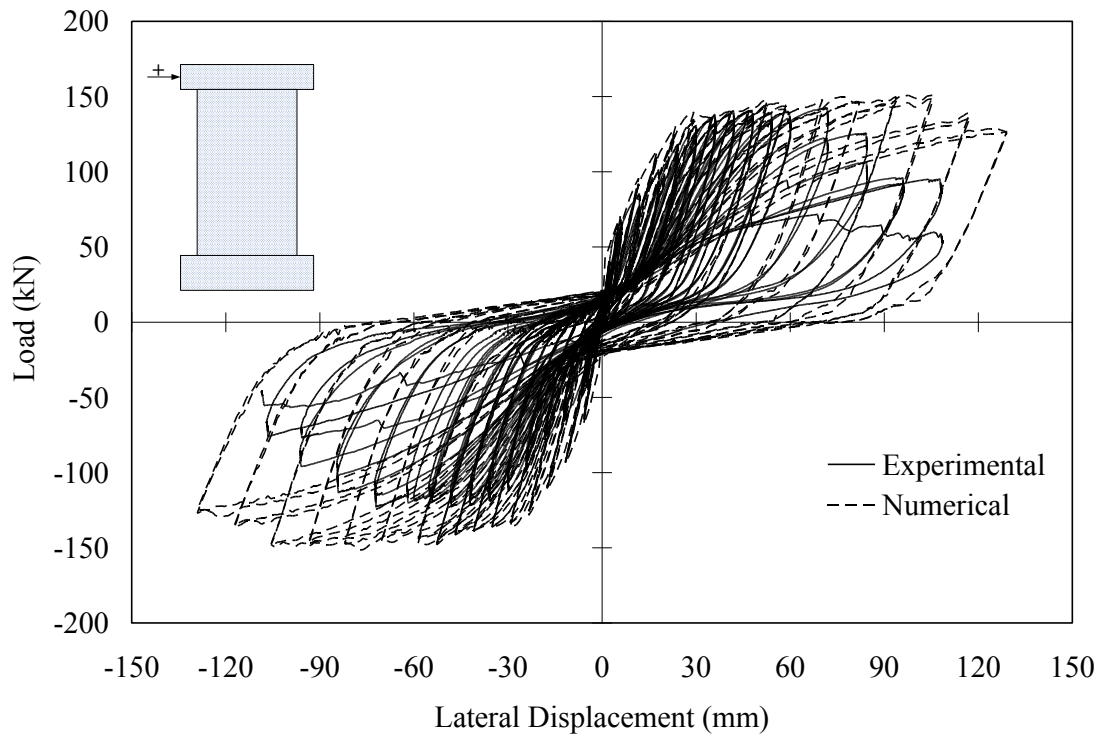
#### 7.3.4.1 Load–Displacement Response

The numerical and experimental load-displacement responses under reverse cyclic loading for conventional and SMA test specimens W1-SR and W2-NR are shown in Figures 7.16 and 7.17, respectively.



**Figure 7.16** Load-Displacement Responses for Conventional Wall W1-SR



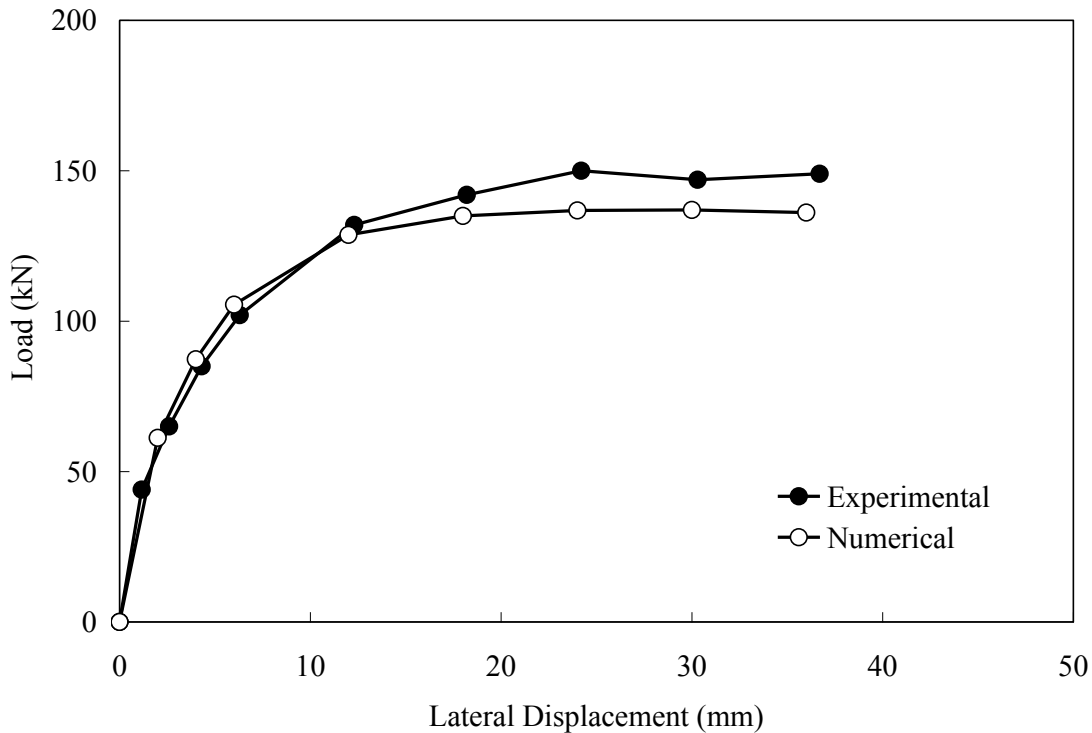


**Figure 7.17** Load-Displacement Responses for SMA Wall W2-NR

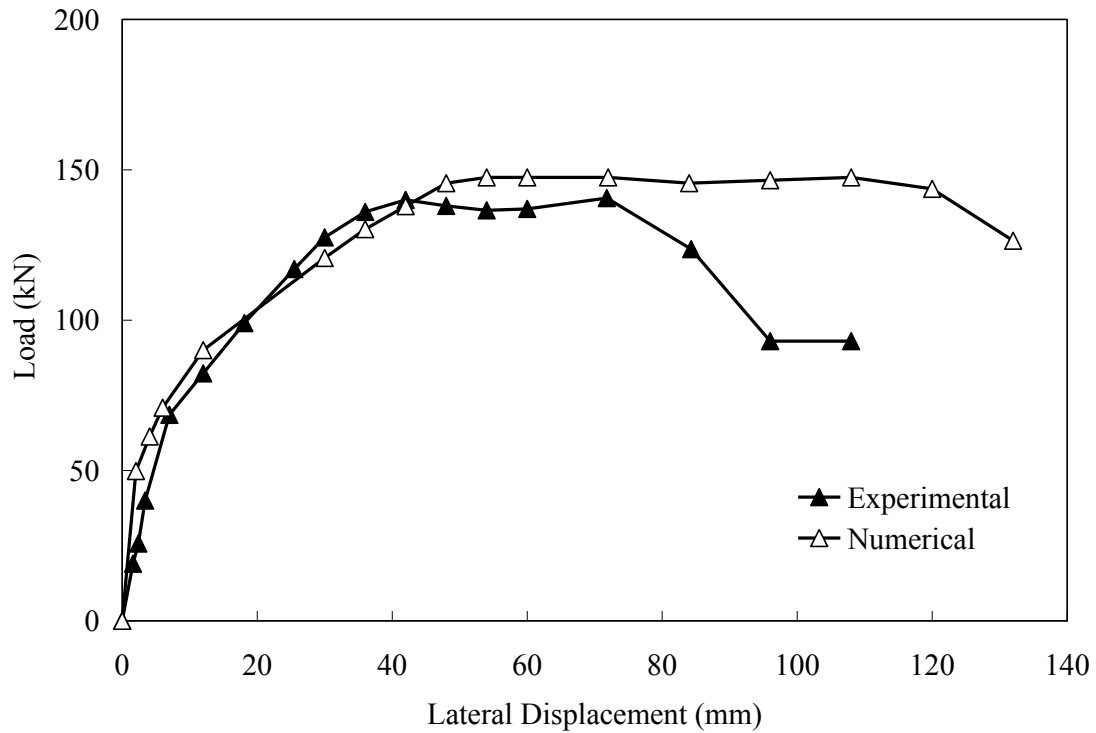
The predicted and recorded hysteretic responses were in good agreement in terms of yield and maximum strength, residual displacement, pre-yielding and post-yielding stiffness, and overall hysteretic behavior. The unloading and reloading curves were well simulated. The analysis satisfactorily captured the permanent displacements; however, the experimental responses of both test specimens demonstrated more pinching. For Wall W2-NR, this difference can be attributed to rupturing of the vertical reinforcement in the web at the end of testing which was not captured by the analysis. Another noticeable discrepancy between the calculated and observed behaviors was the degree of degradation in the reloading stiffness of the repeated cycles at each displacement level where the degradation was slightly underestimated in the analyses.

The predicted response of the SMA wall was characterized by a slightly higher displacement recovery than observed up to a displacement level of 54 mm. Thereafter, the difference became more evident. During the test, the conventional steel in the web of the SMA wall began to rupture at a displacement level of 72 mm, which resulted in a reduction in strength and permanent displacements. The numerical analysis did not capture fracturing of the web vertical reinforcement. Therefore, the analysis predicted a higher ultimate load capacity and less displacement recovery at higher displacement levels..

The numerical and experimental envelope load-displacement responses for walls W1-SR and W2-NR are plotted in Figures 7.18 and 7.19, respectively.



**Figure 7.18** Envelope Load-Displacement Responses for W1-SR



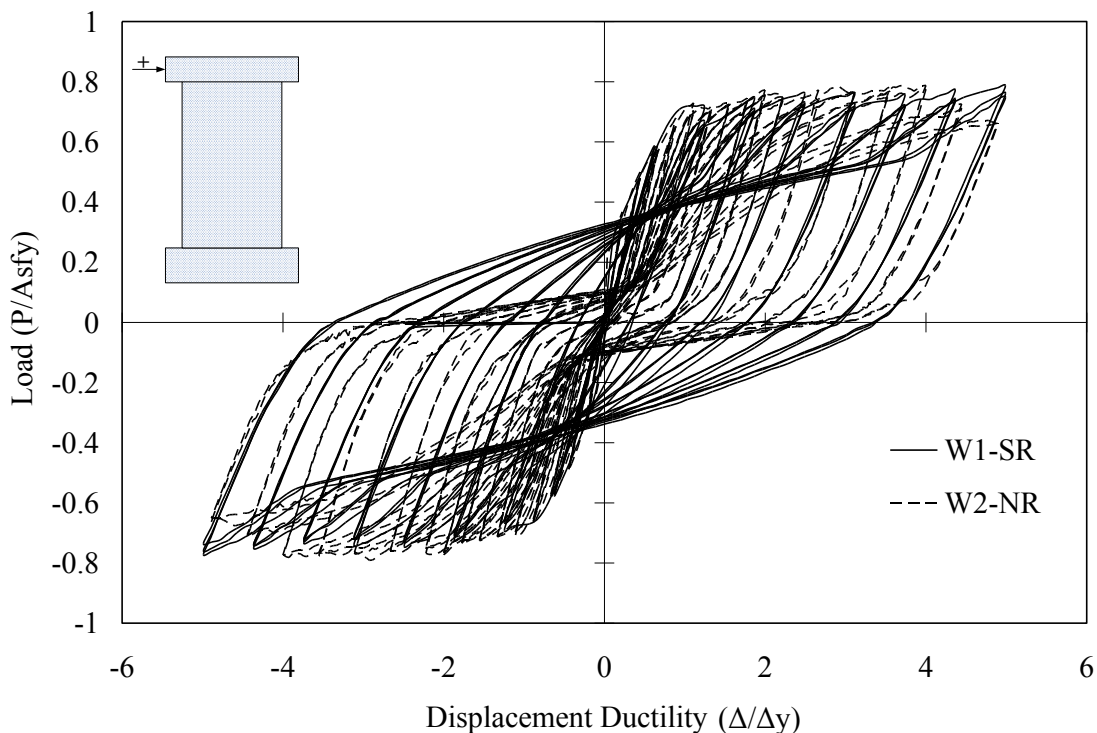
**Figure 7.19** Envelope Load-Displacement Responses for W2-NR

The predicted initial pre-yielding stiffness was approximately 5% greater than that recorded for the conventional wall W1-SR. This difference, however, was slightly higher for the SMA wall W2-NR; the initial pre-yielding stiffness was approximately 12% greater than that recorded. At yield displacement  $\Delta_y$ , the predicted load was only 4% and 9% greater than the experimental load for the conventional and SMA walls, respectively. For the SMA wall, the predicted peak strength of 148 kN was 6% greater than the experimental load, whereas the predicted ultimate load of 126 kN was only 3% greater than the experimental load. Note that the ultimate load was defined to be the point at which the load capacity of the wall dropped below 80% of the maximum load capacity or fracture of the reinforcement (Park 1980). The maximum predicted displacement in W2-NR was 132 mm, while the maximum displacement recorded at failure was 108 mm; an

increase of 22% for the analysis. For the conventional wall, the predicted peak strength was 137 kN which was 8% less than the recorded peak strength.

Failure of the SMA wall during the test was initiated by rupturing of the steel in the web followed by concrete crushing in the boundary zones and buckling of the SMA bars at a displacement level of 108 mm. In addition, shear sliding was evident along the main crack that extended along the length of the wall. The predicted failure involved crushing of concrete in the boundary zone with significant yielding of the SMA and conventional steel reinforcement during the second cycle to 132 mm of displacement. The higher load carrying capacity of the wall predicted by the numerical model was a result of not capturing fracture of the longitudinal reinforcement in the web.

To further demonstrate differences between the conventional and the SMA walls, the numerical responses of both walls were normalized as shown in Figure 7.20.

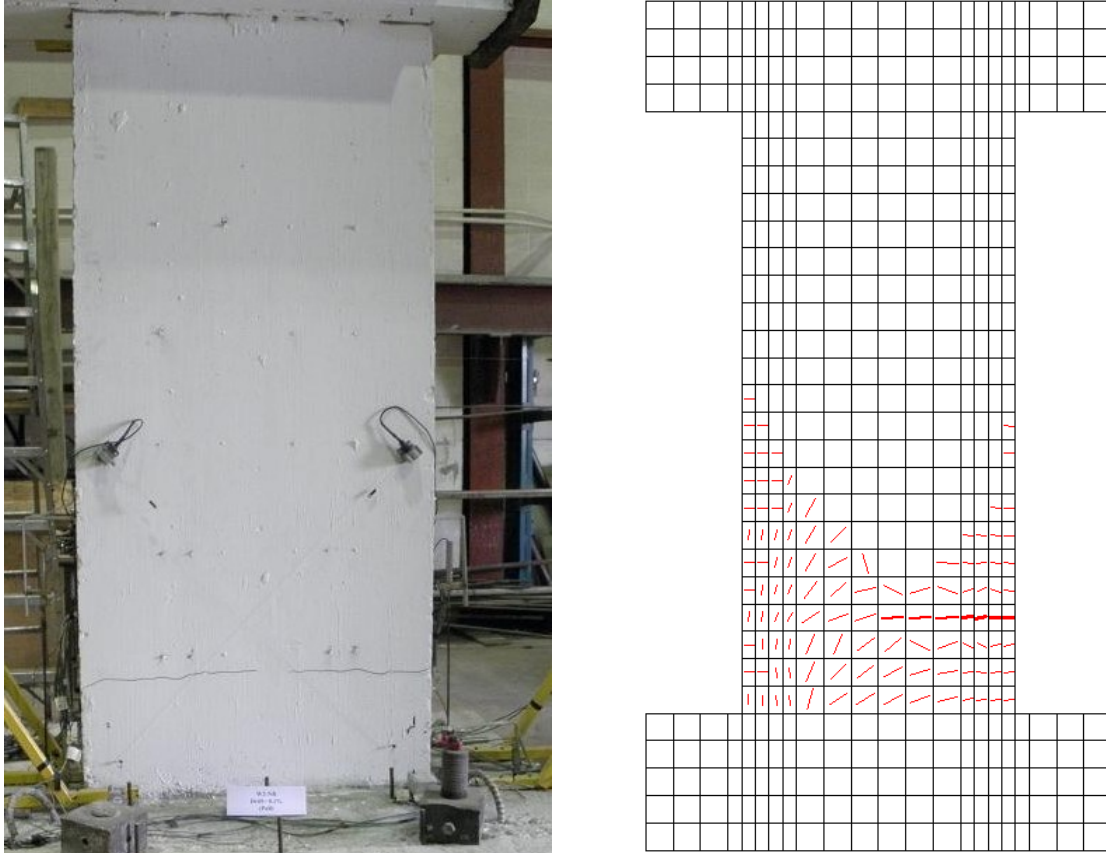


**Figure 7.20** Normalized Numerical Load-Displacement Responses

It is evident that the lower modulus of elasticity in the SMA resulted in lower initial pre-yielding stiffness in the SMA wall and, consequently, higher yield displacement. However, for the most part, both walls have comparable post-yielding stiffness. The ultimate load and displacement ductility of the conventional and SMA walls are similar due to the load normalization and similar concrete properties. The response of both walls demonstrates the high recovery capability of the SMA wall compared to the regular steel wall. When compared with the experimental normalized responses (Figure 4.25), it is evident that the numerical pre-yielding stiffness is higher in the conventional and SMA walls compared to the experimental responses. At yield, the predicted normalized yield load is approximately 5% and 9% higher than the experimental normalized results in W1-SR and W2-NR, respectively. The post-yielding stiffness is comparable in both walls with a slight increase in the predicted normalized peak load in W1-SR.

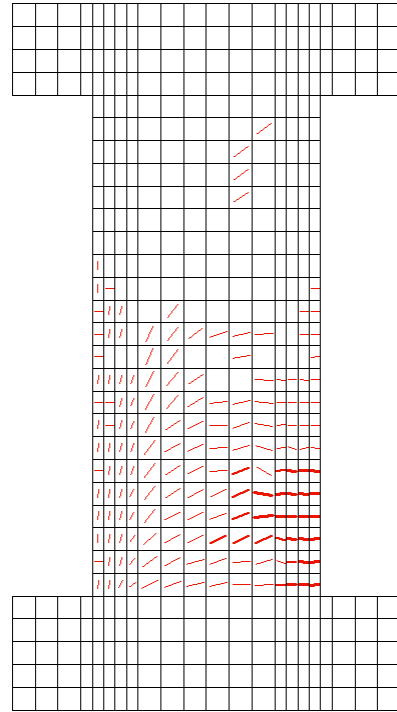
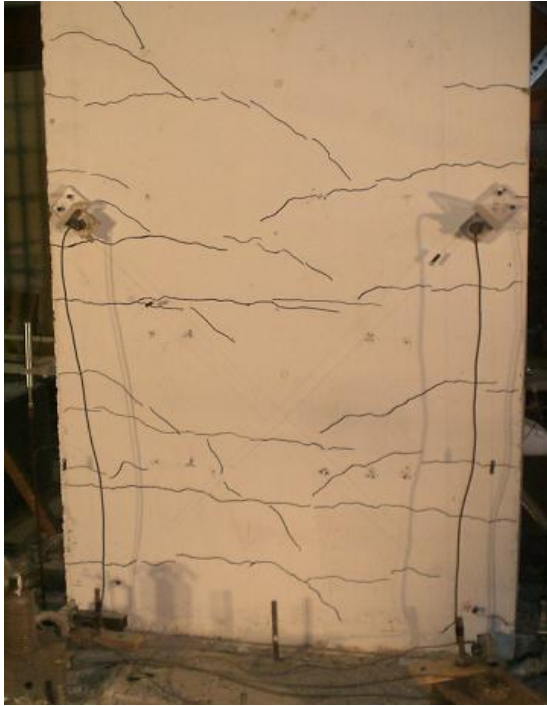
#### 7.3.4.2 Crack Patterns and Failure Modes

A comparison between the numerical and observed crack distribution and failure mode further demonstrates the ability of the constitutive hysteretic model and VecTor2 to simulate the wall behavior. During testing, the formation of the first visible flexural crack became evident at approximately 48 kN of lateral load which corresponded to 2.4 mm of displacement in both walls. The analysis predicted similar cracking that initiated perpendicular to the longitudinal reinforcing bars at approximately the same lateral load. These cracks appeared near the base of the wall at height of approximately 300 mm which is consistent with the observed cracks during testing as shown in Figure 7.21.

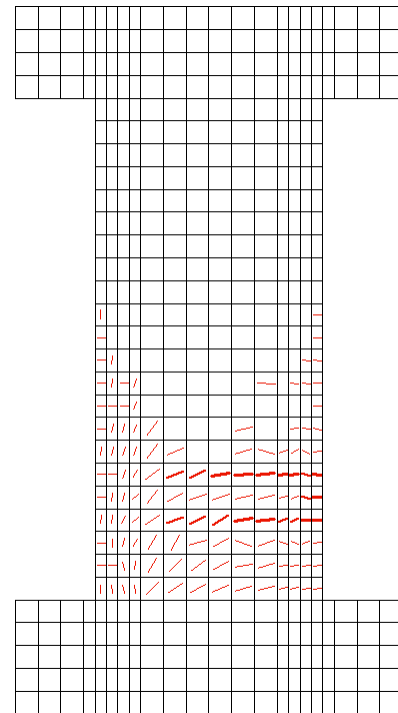
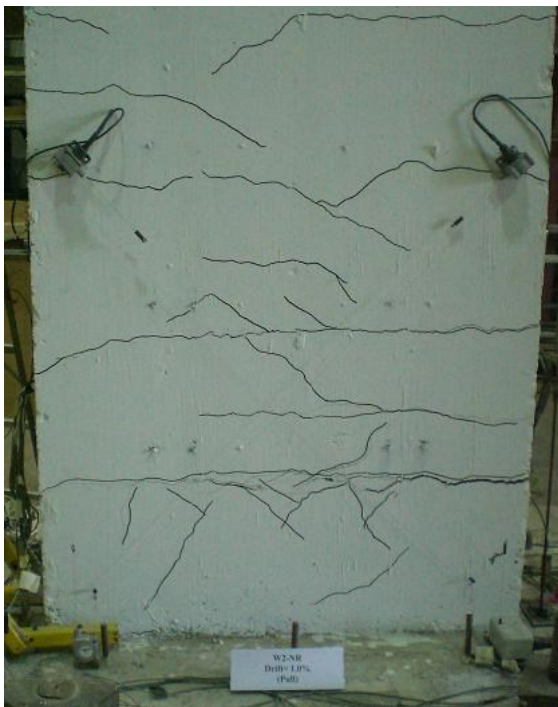


**Figure 7.21** Cracking Pattern at Displacement of 2.4 mm in W2-NR

The numerical and experimental crack patterns for the conventional and SMA walls W1-SR and W2-NR are shown in Figures 7.22 and 7.23, respectively. Generally, the finite element results provided good predictions of the sequence and locations of the flexural and shear cracks.

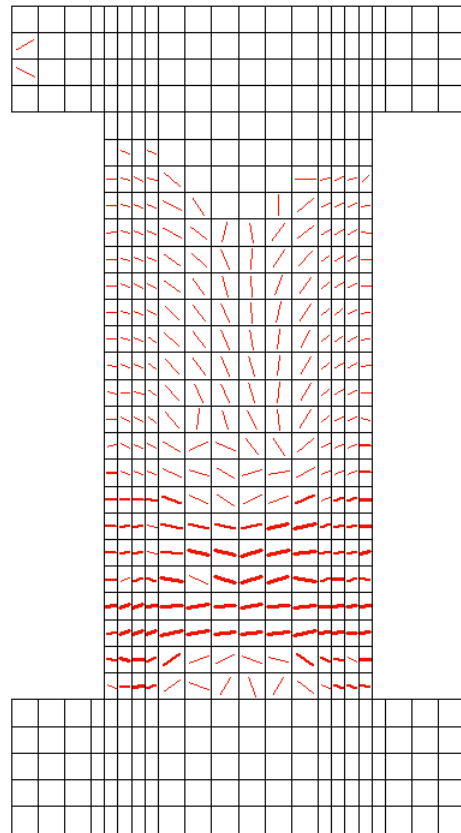


**Figure 7.22** Cracking Pattern in Conventional Wall W1-SR



**Figure 7.23** Cracking Pattern in SMA Wall W2-NR

The observed closely spaced cracking pattern in W1-SR and farther spaced cracks in W2-NR were well predicted by the analyses. However, the predicted crack width was larger than the experimental. At yielding, the numerical maximum flexural crack width was 0.7 mm and 3.4 mm in W1-SR and W2-NR which is 16% and 36% higher than the recorded results, respectively. During the test, the first major flexural crack extended over the full length of the wall near the wall base for both walls W1-SR and W2-NR. This crack further developed and defined the failure location for the SMA wall. Similar behavior was captured by the analysis, where the SMA wall experienced significant yielding prior to concrete crushing at this location as shown in Figure 7.24.

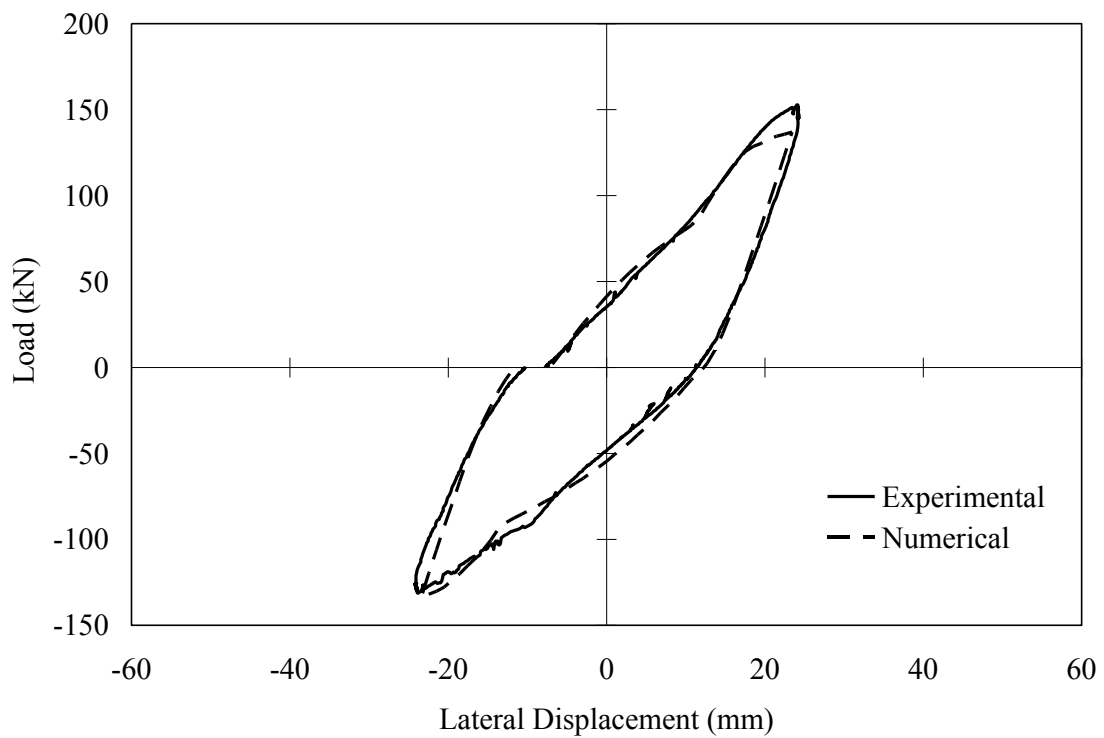


**Figure 7.24** Cracking Pattern at Failure in W2-NR

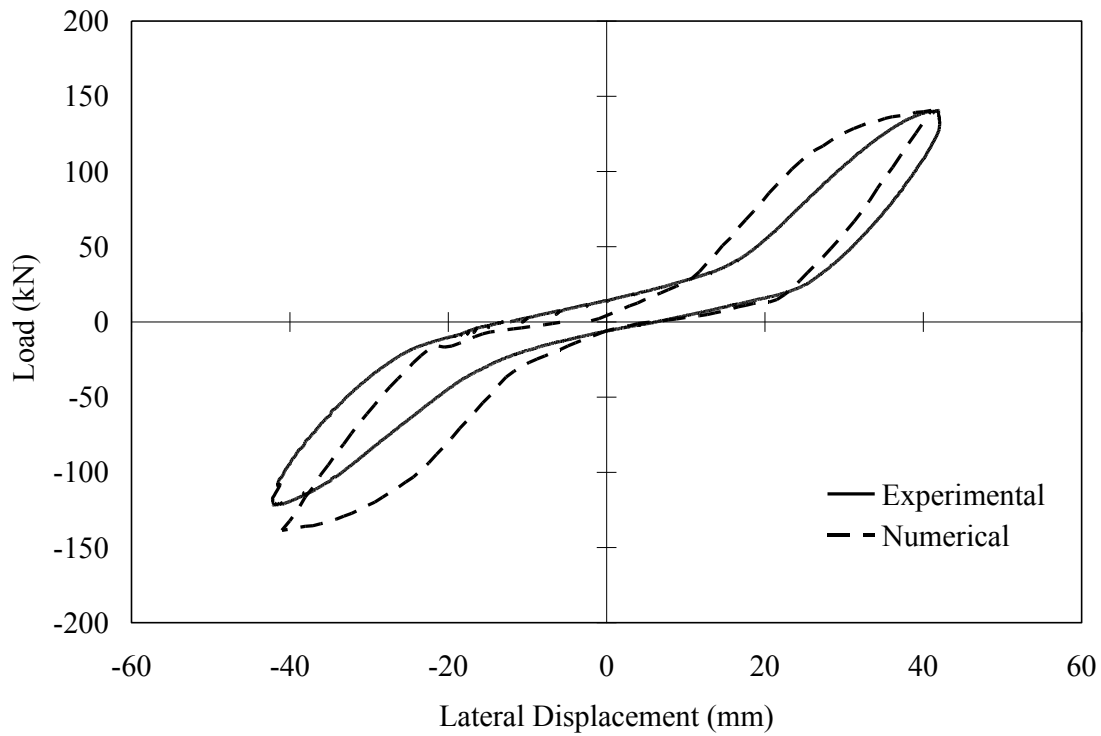


#### 7.3.4.3 Dissipated Energy

Energy dissipation capacity is significant in the design of buildings for earthquake resistance where the structure dissipates energy to sustain the earthquake demands. Figures 7.25 and 7.26 are plots of the numerical and experimental load-displacement hysteresis curves at peak load for the conventional and SMA walls W1-SR and W2-NR, respectively.



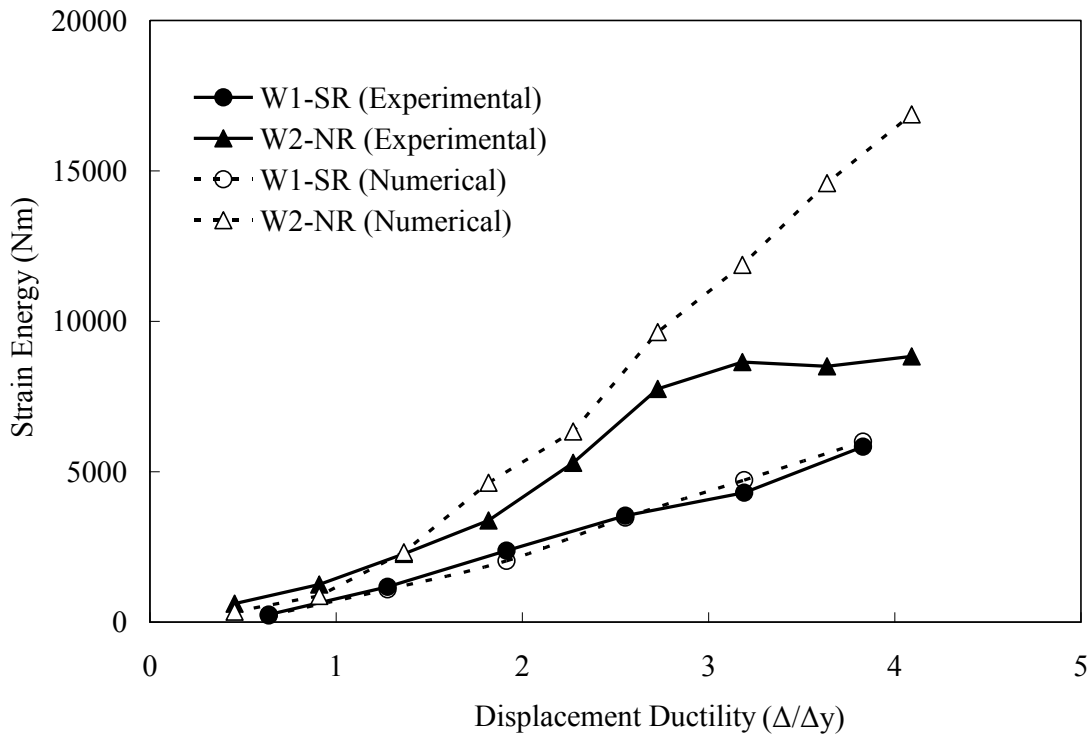
**Figure 7.25** Load-Displacement Hysteresis Response at Peak Load for Wall W1-SR



**Figure 7.26** Load-Displacement Hysteresis Response at Peak Load for Wall W2-NR

The energy dissipation captured with the analysis at the peak load was 3473 Nm for W1-SR, which represents 98% of the corresponding experimental energy dissipation. For Wall W2-NR, the analysis predicted 3425 Nm of energy dissipation, which is 16% greater than the experimental energy dissipation. This demonstrates the adequacy of the finite element program to capture the behavior of the specimens. To further assess the proposed constitutive hysteretic model, the energy dissipated in the first repetition at each cycle was calculated for both specimens at each displacement ductility level as shown in Figure 7.27. The energy dissipated for W1-SR was adequately predicted at all ductility levels, whereas for W2-NR an increasing difference was noted for displacement ductility greater than 2.76. The discrepancy in the SMA wall is attributed to the strength degradation and high recovery observed at higher ductility levels during the test.

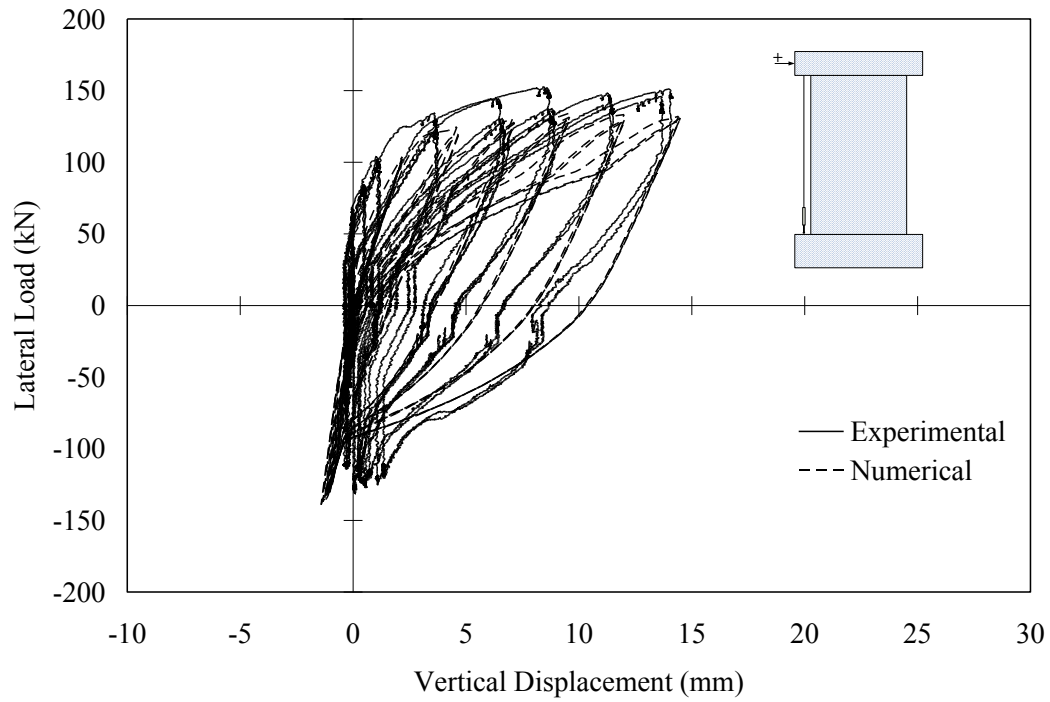
Improved results would be obtained with a parametric study of the finite element model to capture fracture of the longitudinal reinforcement in the web. In addition, refinements to the constitutive hysteretic model, including nonlinear unloading and reloading responses of the SMA bars would provide improvements.



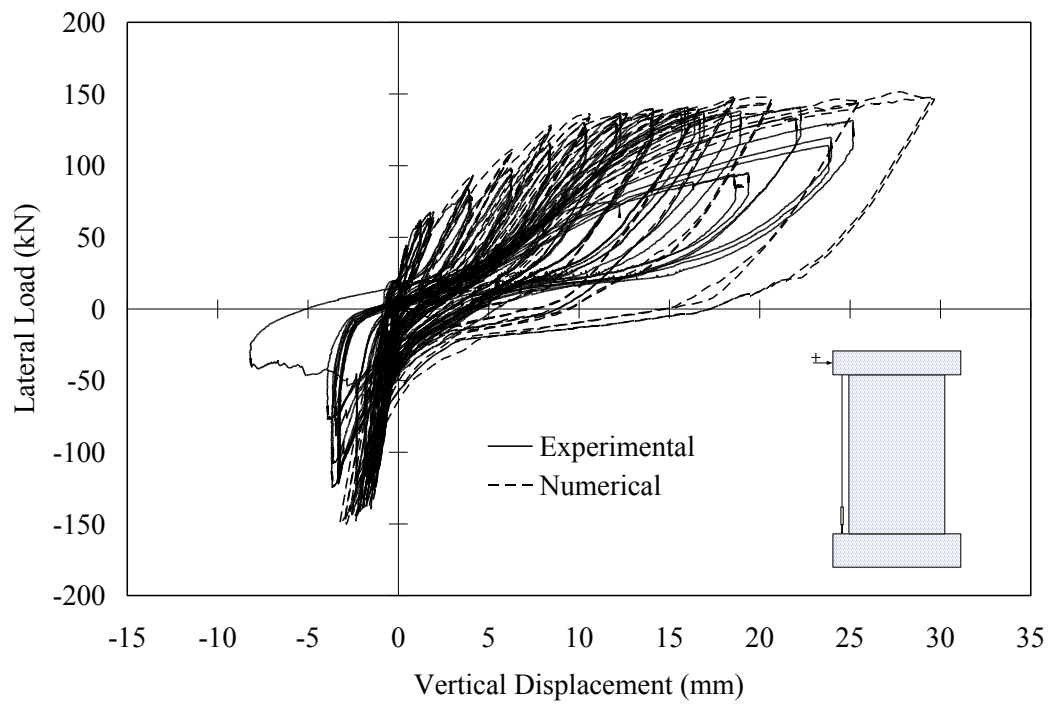
**Figure 7.27** Energy Dissipation-Displacement Ductility Responses

#### 7.3.4.4 Wall Rotation and Vertical Displacement

The vertical displacements at the top of the walls observed during testing and predicted for W1-SR and W2-NR are shown in Figures 7.28 and 7.29, respectively.

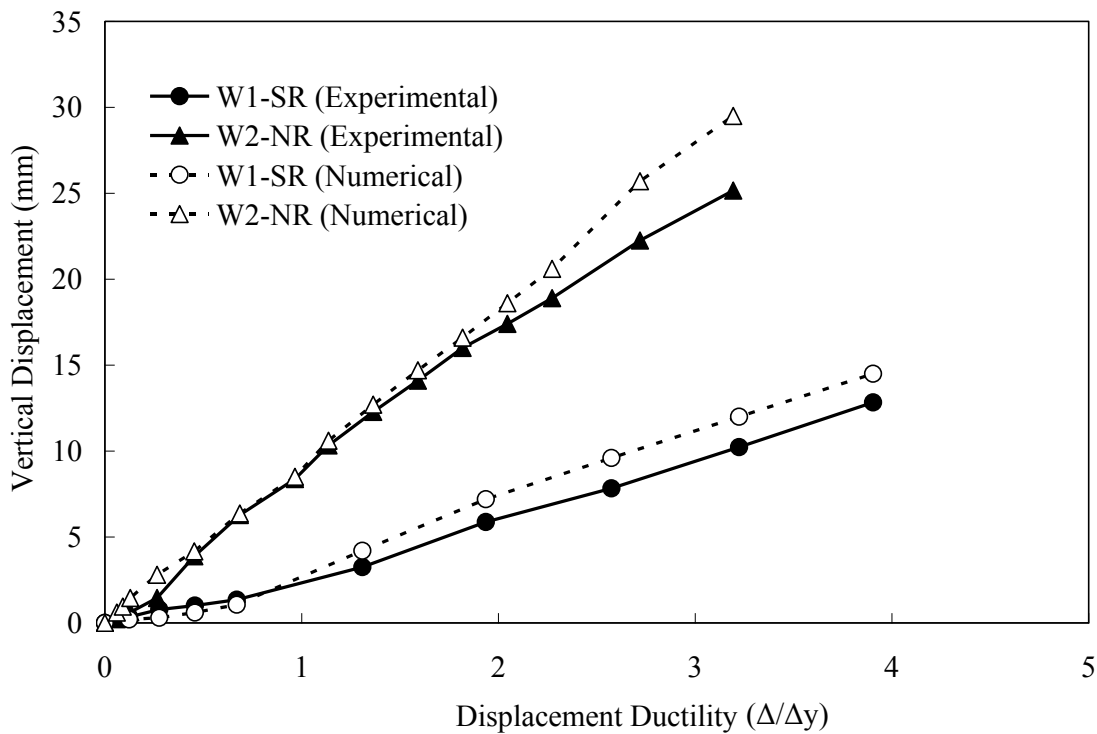


**Figure 7.28** Load-Vertical Displacement Responses for Conventional Wall W1-SR



**Figure 7.29** Load-Vertical Displacement Responses for SMA Wall W2-NR

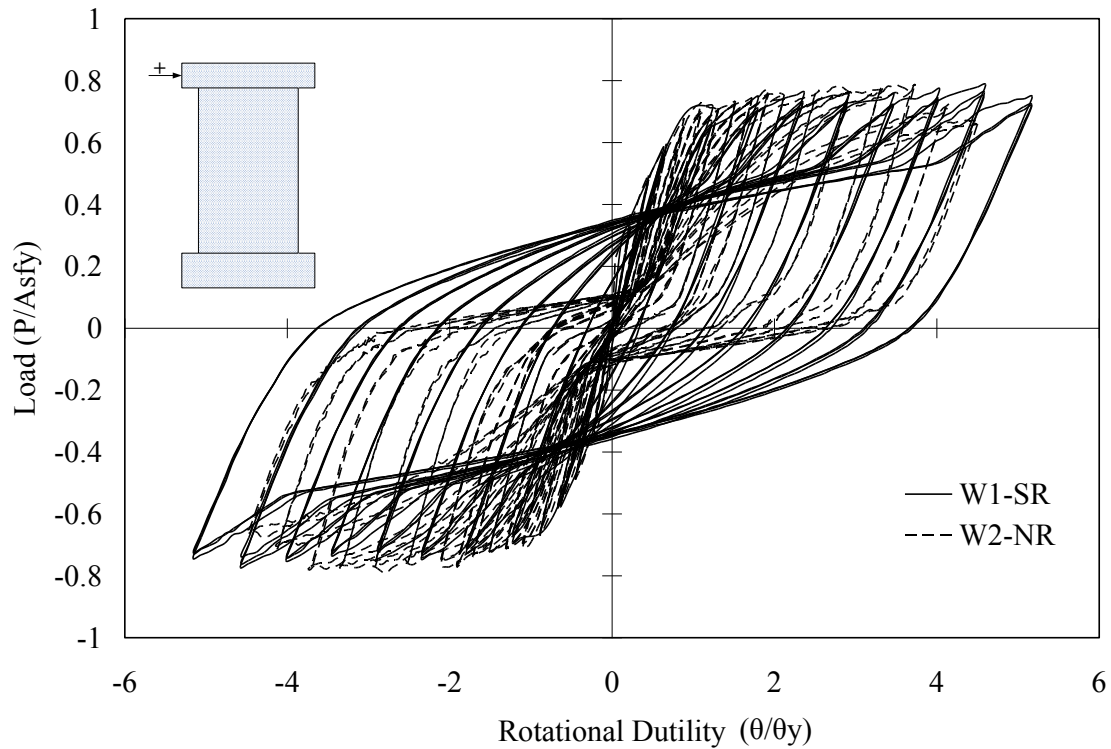
The numerical results for both walls comply in general with the recorded test data. Prior to yielding, the recorded vertical displacement was adequately captured by the analysis. At yield, the reported vertical displacement was 7% greater than the predicted values for the SMA walls and 4% less for the conventional wall. As the wall was pushed into the inelastic range, the analysis predicted higher vertical displacements in both walls, and the difference increased with loading as shown in Figure 7.30.



**Figure 7.30** Vertical Displacement-Displacement Ductility Responses

The discrepancy between the numerical and experimental results can be attributed to the effect of concrete cracking; the predicted crack widths were larger than the recorded crack widths. The vertical displacements can be employed to predict the wall rotation for W1-SR and W2-NR. When comparing the numerical normalized responses (Figure 7.31), it is evident that the lower stiffness of the SMA wall resulted in higher pre-yield

rotations. At yield, the top wall rotation in the SMA wall was about 16% higher than the corresponding conventional wall. The post-yield response demonstrated similar behavior in both walls.



**Figure 7.31** Numerical Normalized Load-Rotational Ductility Responses

The numerical response of W1-SR and W2-NR was generally similar to the experimental response (Figure 5.17) with the exception of higher rotational capacity in the analysis of W2-NR.

In general, the proposed constitutive hysteretic SMA model consisting of a series of linear segments provided reasonable simulations demonstrating the applicability of the model and VecTor2 to simulate SMA reinforced members. Improved results for the SMA beams and shear wall can be achieved with a refined constitutive model that includes nonlinear unloading and reloading. In addition, a reloading response that aims for the

previous maximum unloading strain would improve the hysteretic response. Furthermore, with additional coupon testing of SMA bars, the plastic offset strain  $\varepsilon_p$ , the first unloading strain  $\varepsilon_{rl}$  and the unloading plateau stress  $f_{unl}$  formulations can be updated.

## CHAPTER 8

### **FINITE ELEMENT MODELING OF SMA REINFORCED CONCRETE STRUCTURES**

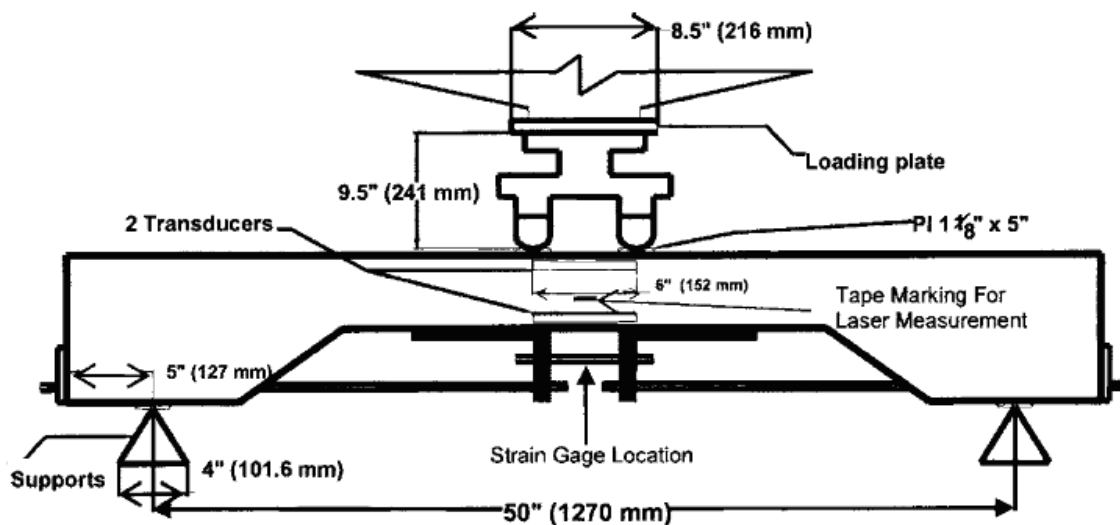
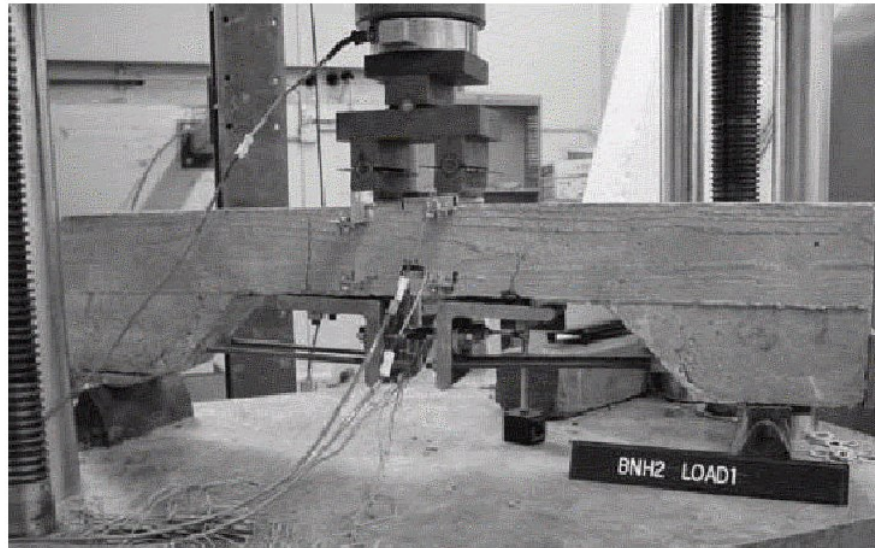
#### 8.1 Introduction

Numerical analyses of SMA reinforced concrete elements documented in the literature were performed to assess the applicability of the proposed constitutive hysteretic model for SMAs developed in this study (Chapter 6). Two ductile, simply supported beams (Ayoub et al. 2003); one beam-column joint (Alam et al. 2007); and one squat shear wall (Effendy et al. 2006) were analyzed using the finite element program VecTor2 and the proposed SMA constitutive relationship. All the required information to conduct the analyses, including specimen material properties and load history were based on available data in the literature; however, some assumptions were necessary where information was insufficient.

#### 8.2 Application to Beams

Ayoub et al. (2003) conducted a pilot study to investigate the behavior of simply supported beams reinforced with superelastic NiTi SMA bars at midspan. Eight small-scale beams were tested under cyclic loading. The beams were 1270 mm long, 250 mm wide and 305 mm deep. The height of the beam was reduced at midspan, as shown in Figure 8.1, such that the external SMA longitudinal reinforcement was used only in the plastic hinge region.





**Figure 8.1** Details of Beam Specimen (Ayoub et al. 2003)

The main test parameters studied by Ayoub et al. were the type and percentage of longitudinal reinforcement. Table 8.1 lists the material properties of the test specimens, including the reinforcement ratios. Furthermore, the effect of bar diameter on the hysteretic response was also investigated by using two diameters of SMA bars: 9.35 mm

and 12.7 mm. The lengths of the SMAs were 254 mm and were used as external reinforcement, which allowed the bars to be reused.

**Table 8.1** Beam Test Variables

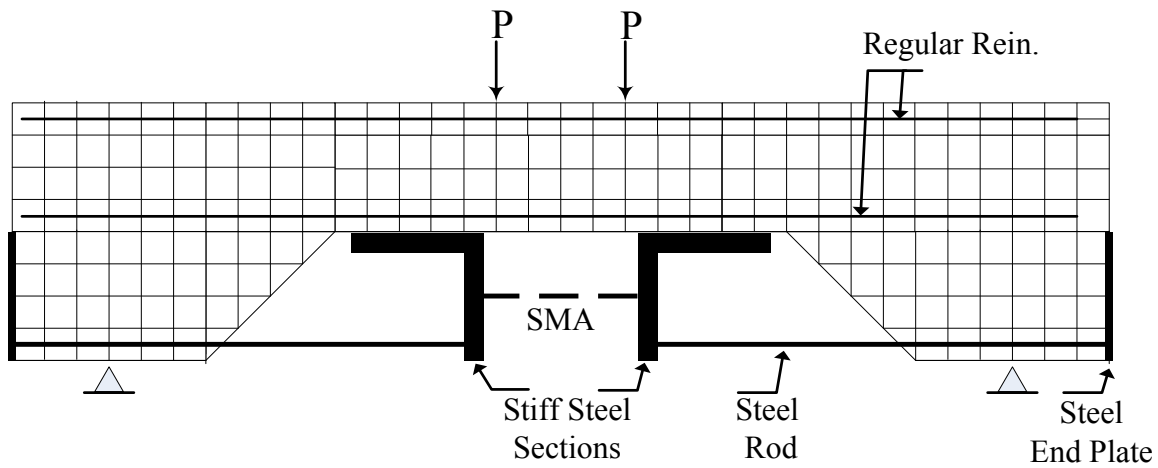
| Specimen    | Reinforcement Type | Longitudinal Reinforcement |                         |
|-------------|--------------------|----------------------------|-------------------------|
|             |                    | Midspan Reinforcement      | Reinforcement Ratio (%) |
| <b>BNL1</b> | NiTi               | 1 bar, $\phi=6.4$ mm       | 0.12                    |
| <b>BNL2</b> | NiTi               | 2 bars, $\phi=6.4$ mm      | 0.24                    |
| <b>BNH1</b> | NiTi               | 1 bar, $\phi=9.5$ mm       | 0.26                    |
| <b>BNH2</b> | NiTi               | 2 bars, $\phi=9.5$ mm      | 0.52                    |
| <b>BSL1</b> | Steel              | 1 bar, $\phi=9.5$ mm       | 0.26                    |
| <b>BSL2</b> | Steel              | 2 bars, $\phi=9.5$ mm      | 0.52                    |
| <b>BSH1</b> | Steel              | 1 bar, $\phi=12.7$ mm      | 0.47                    |
| <b>BSH2</b> | Steel              | 2 bars, $\phi=12.7$ mm     | 0.94                    |

Two relatively stiff angle sections were used to connect the reinforcement to minimize the deformations during the test. All the test beams were reinforced with 9.35 mm diameter conventional steel in the longitudinal and transverse directions to provide minimum steel ratio according to code requirements. Two longitudinal bars placed at the top of the beams spanned the entire length, while the bottom bars were cut at midspan of the beams to ensure that the strength was provided only by the external SMA bars. All beams were tested under two central loads, spaced 150 mm apart.

Testing of all beams was performed by applying quasi-static load up to the yield strength and the corresponding yield displacement was recorded. Thereafter, the test specimens were cycled in multiple increments of yield displacement up to displacement ductility of two. The experimental results indicate that the conventional and SMA beams responded with comparable yield and peak strength.

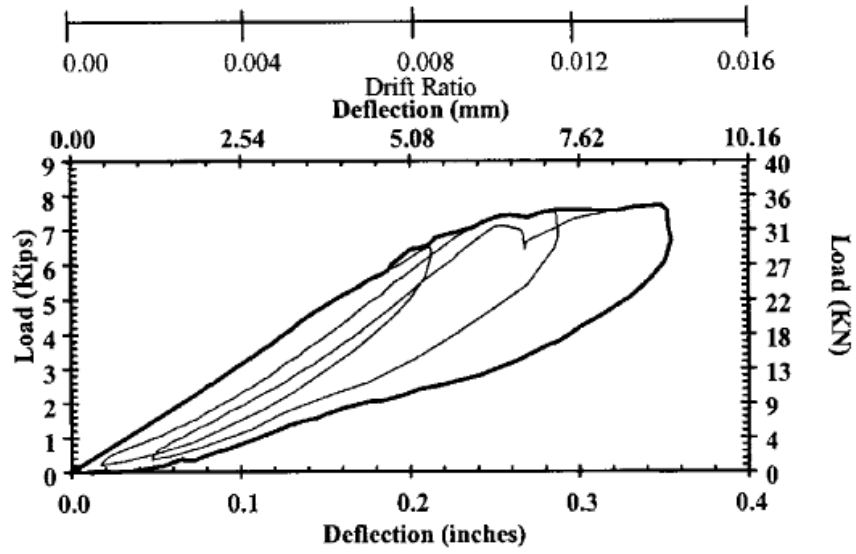
In the finite element analysis, the beams were modeled with only one homogeneous concrete layer. Transverse reinforcement was smeared in the concrete layers, whereas,

the embedded and external longitudinal reinforcement were modeled as discrete truss bars. The numerical model, as shown in Figure 8.2, consisted of 548 rectangular and triangular plane stress elements to represent the concrete layers and 155 truss bar elements to represent the longitudinal reinforcement. Note that Program VecTor2 only includes models for reinforcing bars in its library. Therefore, to model the steel angle sections, a new concrete layer with high stiffness was used to model the steel sections. The loading followed the same protocol that was used during testing.

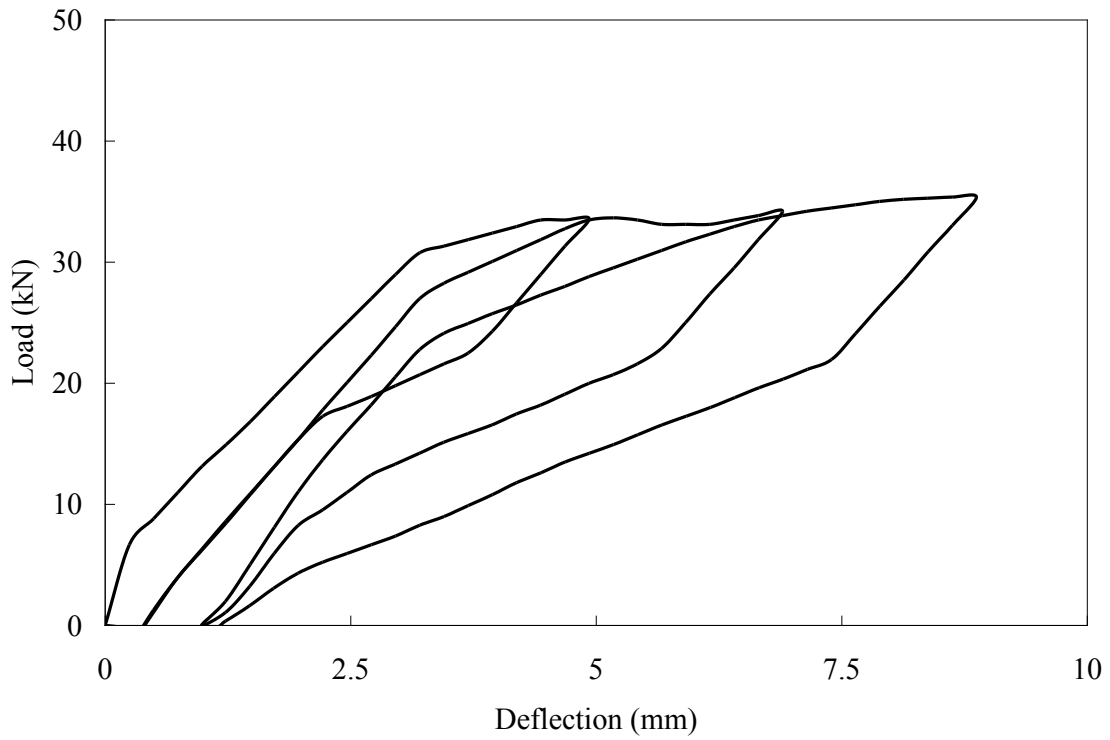


**Figure 8.2** Finite Element Mesh for SMA Beams

The predicted response of the SMA beams BNH1 and BNH2, as shown in Figures 8.3 and 8.4, respectively, was close to the experimental response in terms of stiffness, strength, ductility, and displacement recovery.

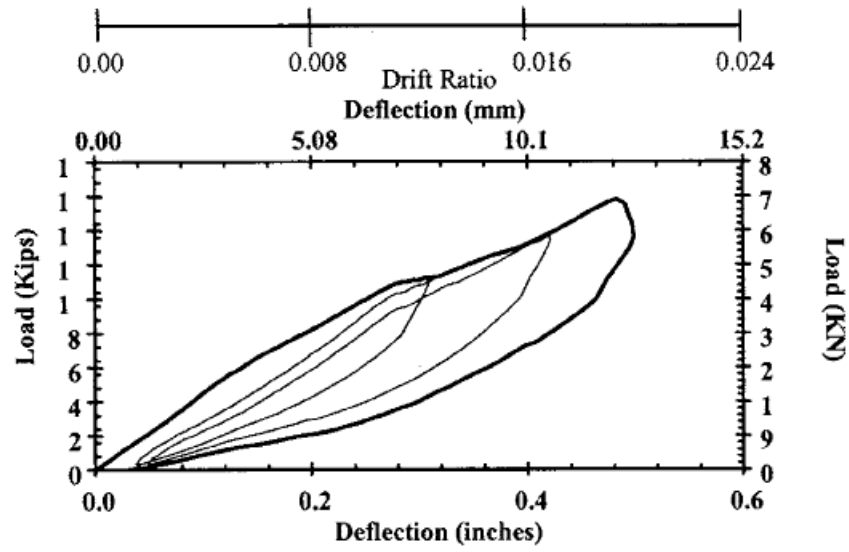


(a)

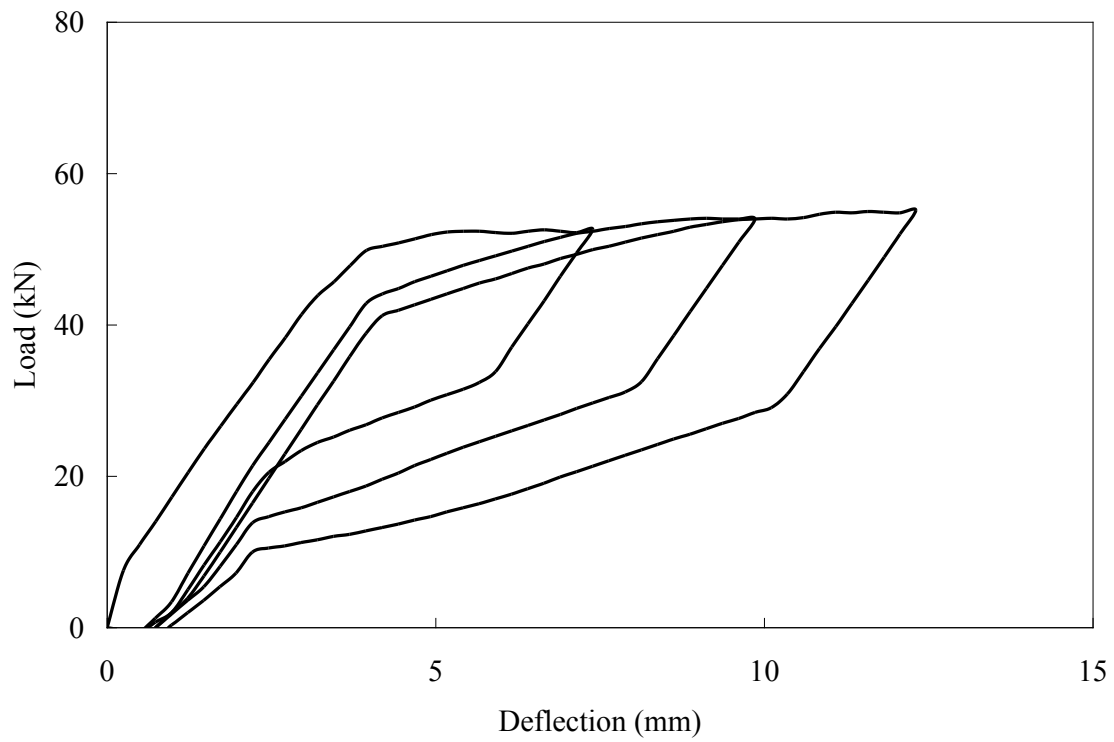


(b)

**Figure 8.3** Load-Displacement Response for Beam BNH1: (a) Experimental (Ayoub et al. 2003); and (b) Numerical



(a)



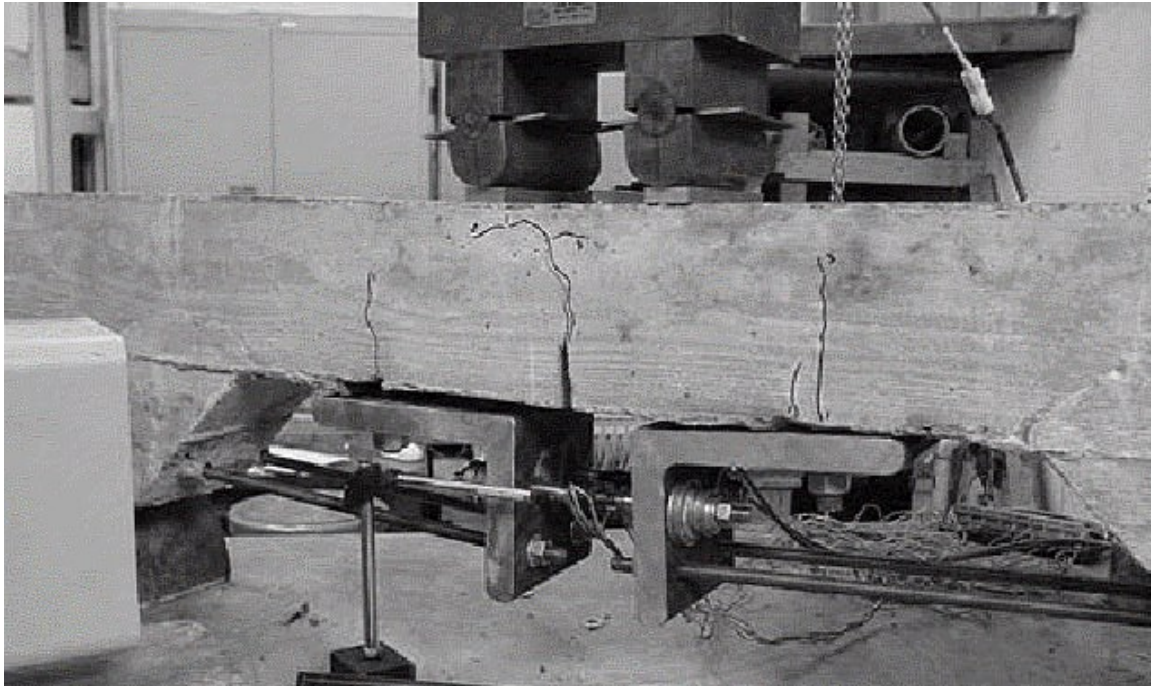
(b)

**Figure 8.4** Load-Displacement Response for Beam BNH2: (a) Experimental (Ayoub et al. 2003); and (b) Numerical

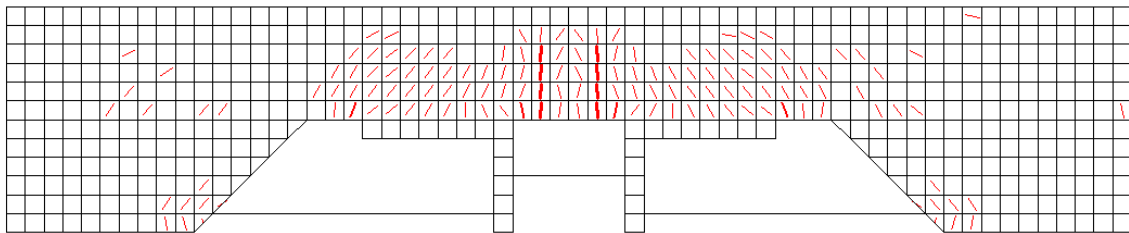
The analyses predicted a slightly higher initial stiffness than that observed for Beam BHN1. The initial stiffness, however, was clearly overestimated for Beam BHN2. This can be attributed to the modeling of the steel angles. The model did not permit rotation, whereas the steel angles rotated and separated from the beam during the test as shown in Figure 8.5. Since these beams were externally reinforced, properties of the reinforcement controlled the response and the contribution of the concrete was secondary to the overall behavior. The predicted yield strength of Beam BNH1 and Beam BNH2 were 6% and 4% higher than the corresponding experimental results. The numerical post-yield stiffness of BNH1 was similar to the experimental response. On the other hand, BNH2 experienced early strain hardening during the test which was not captured in the analysis and resulted in higher peak strength during testing. The numerical maximum strength of BNH2 was approximately 58 kN, which underestimated the observed strength by approximately 17%. This difference was only 5% for BNH1.

The unloading and reloading responses were satisfactory simulated in the analysis and minor differences were observed between the predicted and measured residual displacements. For the first loading cycle, which corresponded to a vertical displacement of 5 mm and 7.5 mm in BNH1 and BNH2, respectively, the recorded plastic displacements were approximately 10% greater than the predicted residual displacement in both beams; whereas at maximum drift, the difference increased to 14% for Beam BNH1 and 18% for Beam BNH2.

The numerical and experimental crack patterns for the SMA beam BNH2 are shown in Figure 8.5.



(a)



(b)

**Figure 8.5** Cracking Patterns for SMA Beam BNH2: (a) Experimental (Ayoub et al. 2003); and (b) Numerical

During the test, one major flexural crack surfaced at midspan and extended to the top surface of the beam with two secondary cracks forming outside the SMA region. Note that the two secondary cracks seem to have originated at the location of the bolts that were used to connect the steel angles to the beam. The analysis, however, indicated two major cracks directly beneath the location of the two externally applied loads. It is

probable that restricting the rotation of the steel plates in the analysis resulted in two cracks rather than one as was observed during the test.

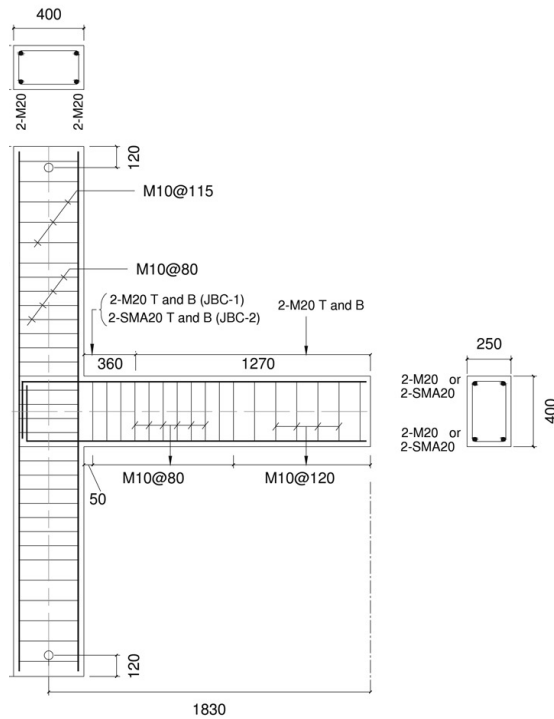
### 8.3 Application to Beam-Column Joints

The seismic behavior of a beam-column joint reinforced with superelastic SMAs was studied by Alam et al. (2007). Two large-scale, beam-column joints representative of an exterior beam-column from an eight story building located in the high seismic region of western Canada were designed and detailed according to the NBCC 2005 (NBCC 2005) and CSA A23.3-04 (CSA 2004) recommendations. The test specimens were scaled down by a factor of 1/4 from the full-scale prototype. Both test specimens were identical in geometry, material properties, and longitudinal and transverse steel ratios with the exception of using superelastic SMA bars in the plastic hinge region of one of the specimens instead of conventional reinforcing steel bars. The test variables are listed in Table 8.2, while Figure 8.6 provides the dimensions and reinforcement layout of the specimens.

**Table 8.2** Frame Test Variables

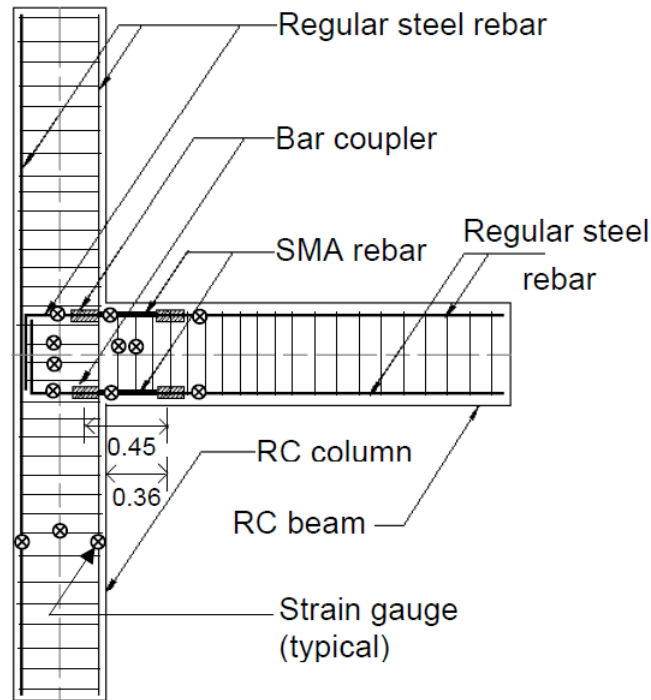
| Specimen | Loading Method | Reinforcement Type | Joint Longitudinal Reinforcement |             |
|----------|----------------|--------------------|----------------------------------|-------------|
|          |                |                    | Top                              | Bottom      |
| JBC-1    | Reverse Cyclic | Steel              | 2 bars, 20M                      | 2 bars, 20M |
| JBC-2    | Reverse Cyclic | NiTi               | 2 bars, 20M                      | 2 bars, 20M |





**Figure 8.6** Details of Beam-Column Joint Specimen (Alam et al. 2007)

The beam of the test specimen was 1630 mm long, 250 mm wide and 400 mm deep. The column of the test specimen had the same cross-section dimensions as the beam but with a total length of 3000 mm. Two-20M longitudinal bars were placed at the top and at the bottom of the beam and column. The reinforcement spanned the entire length of the beam and column. Closed 10M rectangular ties were used in the column and in the beam at different spacing to prevent premature shear failure. As shown in Figure 8.7, screw-lock mechanical couplers were used in the SMA specimen to connect 20 mm diameter longitudinal SMA and the regular steel bars extending outside the plastic hinge region. The total length of SMA bars was 450 mm, which was measured from centre to centre of the couplers.



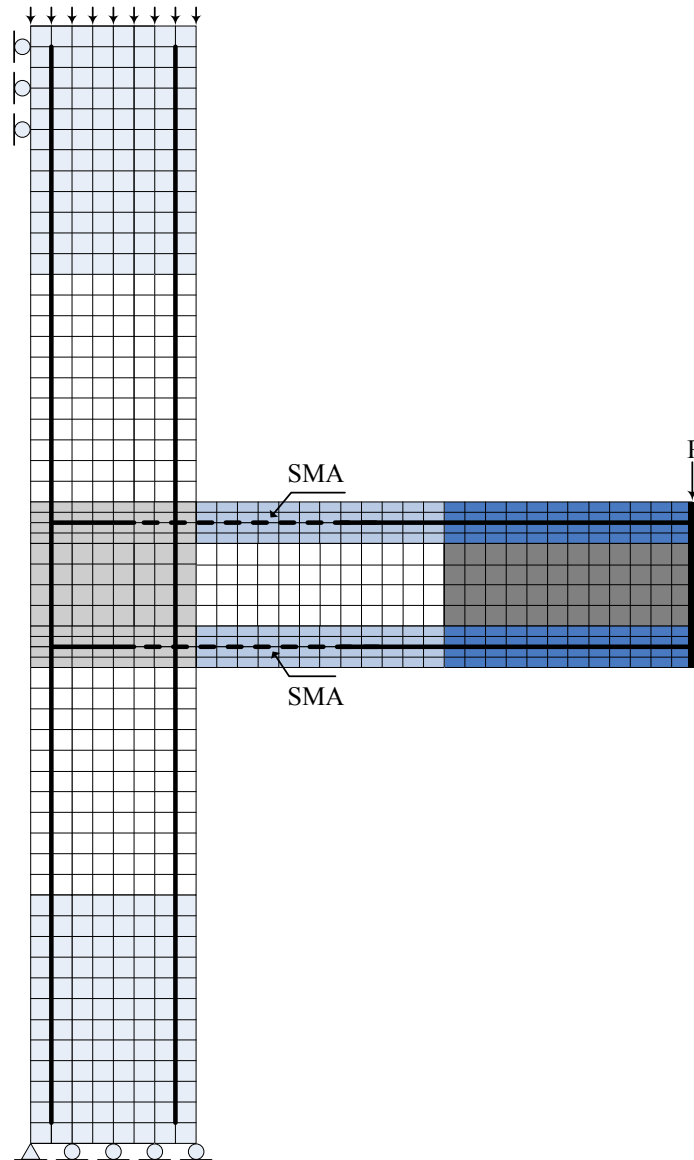
**Figure 8.7** Detail of SMA Region in Beam-Column Joint JBC-2 (Alam et al. 2007)

During the test, the top and the bottom of the column were prevented from horizontal and vertical displacements but allowed to freely rotate to represent a hinge support. Loading consisted of constant axial load (350 kN) applied at the top of the column and incremental reverse cyclic vertical displacements at the tip of beam. The loading protocol included a load-controlled phase to yielding followed by a displacement-controlled phase to failure. The displacement-controlled phase consisted of incremental multiplies of the yield displacement. At each displacement level two repetitions were applied. The test was terminated at a story drift of 7.9%.

Test results showed that the conventional and SMA specimens had similar yield and peak load carrying capacity; however, the corresponding yield displacement was approximately 50% larger for the SMA specimen. Test results demonstrated the ability of the SMA reinforced beam-column joint specimen to recover most of its post-yield

displacement. Thus, it would require a minimum amount of repair after a strong earthquake.

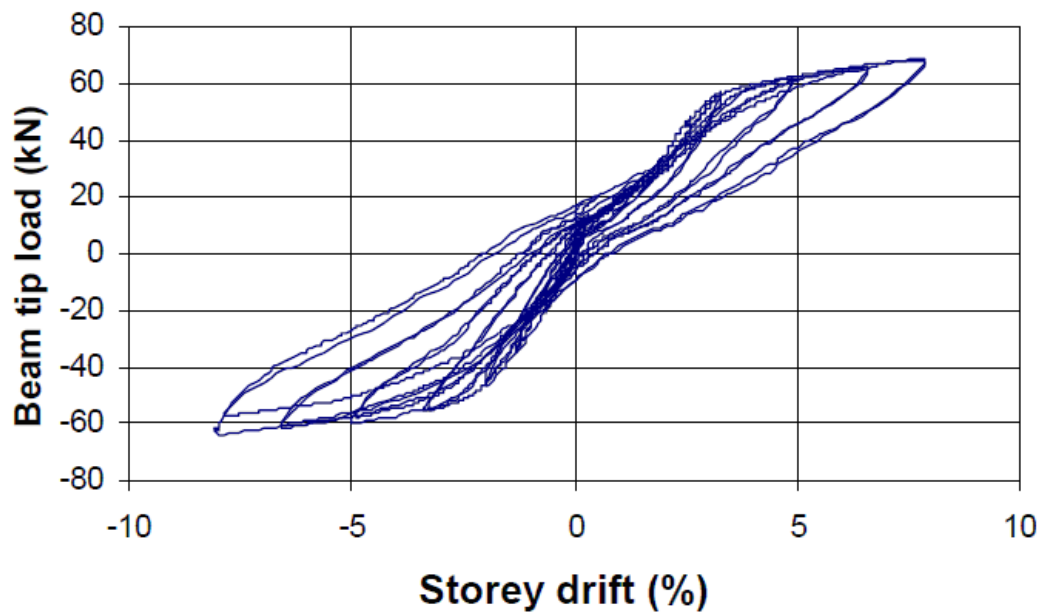
Finite element modeling of the SMA reinforced beam-column joint specimen JCB2 (Figure 8.8) consisted of seven concrete layers. The first three layers represented the column, while the last four layers represented the beam.



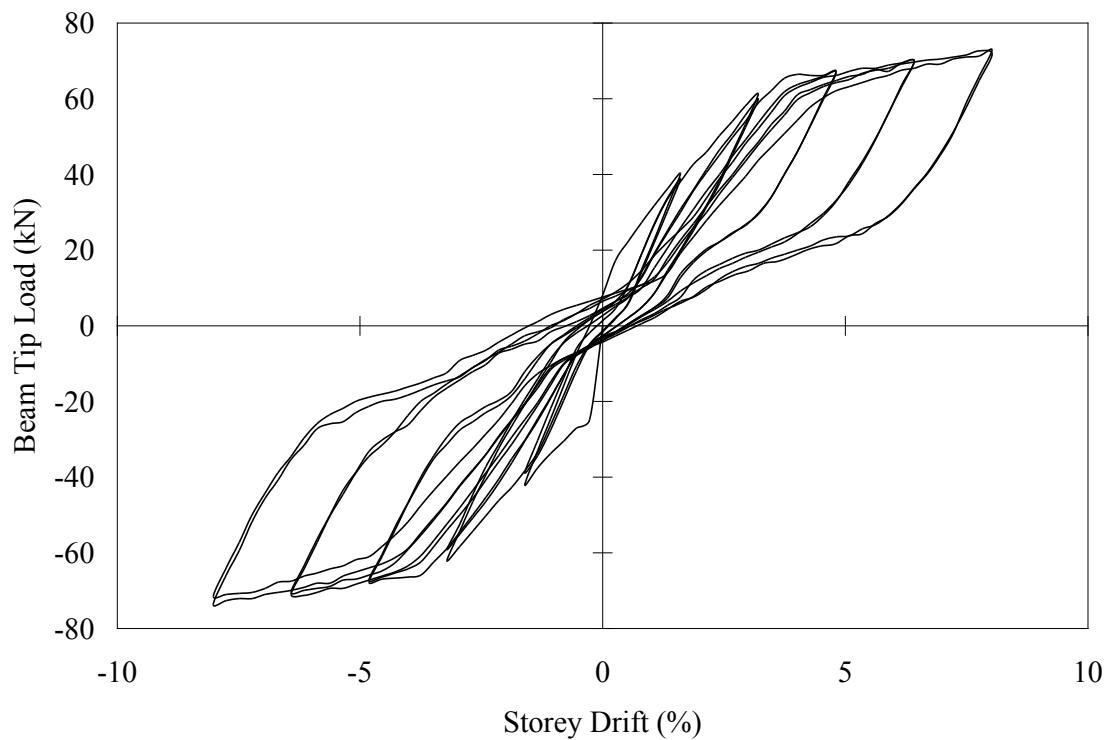
**Figure 8.8** Finite Element Mesh for SMA Beam-Column Joint JCB2

Geometric and concrete properties were similar for all the layers, whereas the steel (transverse) properties, which were smeared in the concrete, were different. The main longitudinal reinforcement at the top and the bottom of the beam and column was modeled with truss bars. The mesh used for modeling the specimen consisted of 1008 plane stress rectangular elements and 232 truss bar elements. In addition, 24 link elements were added to the mesh to model the bond between the SMA bars and the surrounding concrete. Vertical and horizontal constraints were distributed along the base of the column to simulate the support condition of the test specimen. The initial axial loading was modeled by equivalent distributed vertical forces placed at the top of the column.

The experimental and predicted results for the SMA specimen as illustrated in Figures 8.9 and 8.10, respectively, demonstrates the ability of the SMA constitutive hysteretic model to simulate the experimental response in terms of stiffness, strength, ductility, and drift recovery.



**Figure 8.9** Experimental Load-Story Drift Response for Beam-Column Joint JBC-2 (Alam et al. 2007)



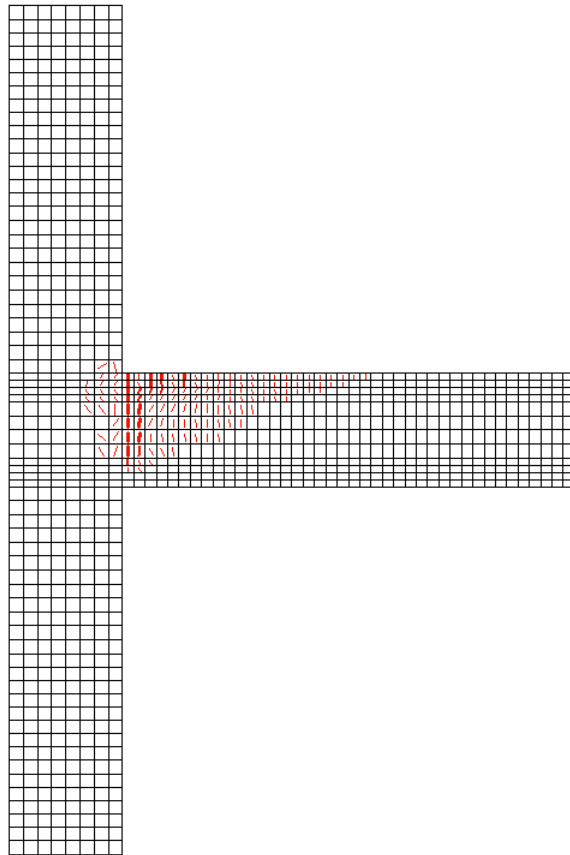
**Figure 8.10** Numerical Load-Story Drift Response for SMA Beam-Column Joint JCB2

The pre-yielding stiffness was overestimated in the analysis resulting in analytical-to-experimental yield strength ratio of 1.14. The post-yield stiffness was comparable and the predicted peak load at the end of the test was 72 KN which was only 6% greater than the experimental results. The pinching response was evident in the analysis owing to the ability of SMA to recover most of the inelastic drift. However, the plastic displacements were underestimated by the analysis. At drift ratio of 7.9% (end of the test) the predicted and experimental residual displacements were 6.1 mm and 9.8 mm, respectively. The experimental unloading response was linear with an unloading stiffness lower than the initial stiffness and the reverse transformation was not obvious as in the analysis. This is partly due to the slippage of the SMA bars inside the couplers that were, in turn, connected to the conventional steel outside the plastic hinge region.

During the test, the first flexural crack (Figure 8.11) initiated at 160 mm away from the column face. While loading progressed, small flexural cracks formed along the beam. In the analysis, the general crack pattern was similar to the experimental results, but the first flexural crack was closer to the column face. In both the experiment and the analysis, no additional major cracks surfaced other than the first flexural crack, which increased in width and depth in subsequent load cycles.



(a)



(b)

**Figure 8.11** Crack Patterns in SMA Beam-Column Joint JCB2: (a) Experimental (Alam et al. 2007); and (b) Numerical

#### 8.4 Application to Shear Walls

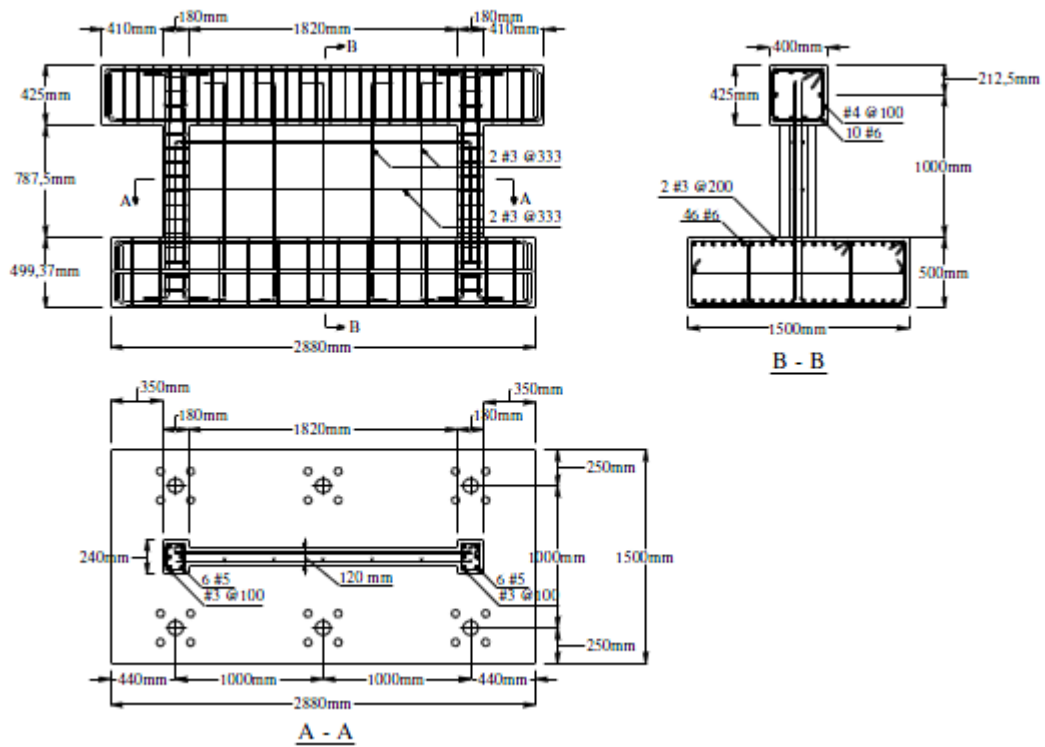
Effendy et al. (2006) tested three low-rise barbell-shaped shear walls under reverse cyclic loading. The main parameter in the study was the type of SMA bars, namely superelastic Austenite and Martensite. Superelastic SMA bars were placed externally in one of the walls in the diagonal directions acting as structural bracing to increase ductility and energy dissipation capacity. The dimensions of the web walls in all specimens were 787.5 mm in height, 2180 mm in width (including boundary elements), and 120 mm in thickness. The boundary elements were 180 mm square. Furthermore, the walls were built integrally with a heavily reinforced foundation to serve as a fixed support and a stiff top beam for the purpose of distributing the loading to the wall during the test.

Table 8.3 lists material properties and reinforcement ratios, and Figure 8.12 provides the dimensions and reinforcement layout of the SMA reinforced specimens.

**Table 8.3** Shear Wall Test Variables

| Specimen    | Steel Ratio (%) |          | SMA Bars                              |
|-------------|-----------------|----------|---------------------------------------|
|             | Horizontal      | Vertical |                                       |
| <b>SMAC</b> | 0.48            | 0.48     | N/A                                   |
| <b>SMAS</b> | 0.24            | 0.24     | 4 bars, $\phi=12.7$ mm (Superelastic) |
| <b>SMAM</b> | 0.24            | 0.24     | 4 bars, $\phi=12.7$ mm (Martensite)   |



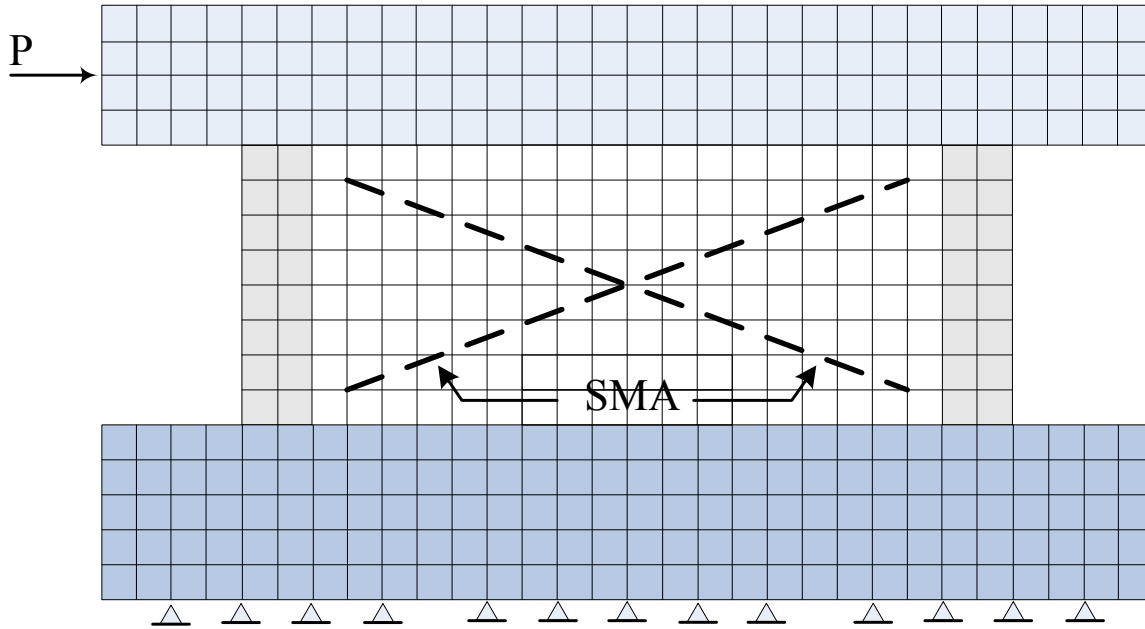


**Figure 8.12** Details of SMA Shear Wall Specimen (Effendy et al. 2006)

The internal steel ratio in both perpendicular directions of the control specimen (SMAC) was 0.48%, while it was 0.24% in the specimens externally reinforced with Austenite (SMAS) or Martensite (SMAM) SMA bars. The SMA-reinforced specimens were provided with an additional 0.24% of SMA reinforcement to be consistent with the 0.48% reinforcement ratio used for the control specimen. Loading of the SMA test specimens consisted of two repetitions of incremental reverse cyclic lateral displacements at the top beam. The wall was initially displaced by 1 mm followed by a displacement of 2.5 mm. Thereafter, increments of 2.5 mm were imposed to failure. No axial load was applied on the top beam.

The experimental response of the conventional and superelastic SMA specimens was typical of shear-dominant behavior. The effect of the SMA in limiting the residual displacement was evident; however, the energy dissipation capacity was minimal in comparison to ductile walls.

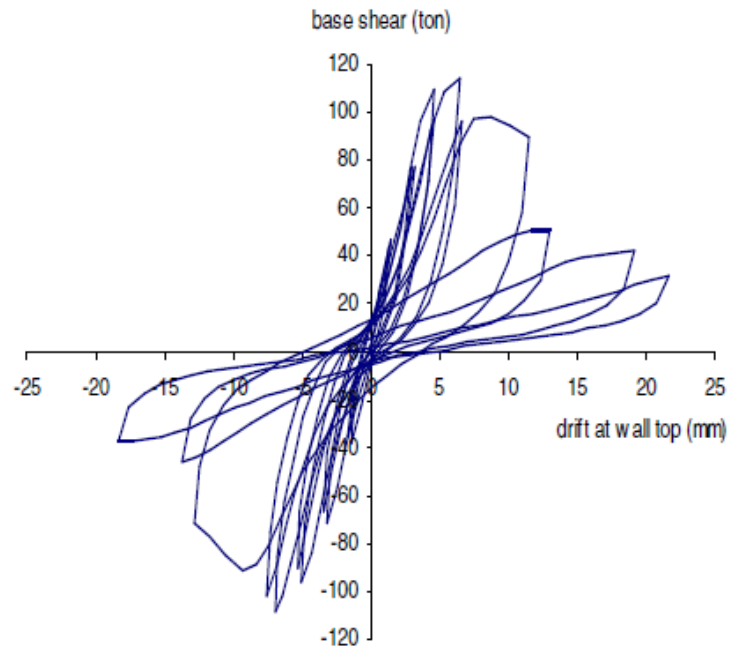
The geometry and material properties of the wall were modeled with four homogeneous concrete layers with smeared reinforcement (Figure 8.13).



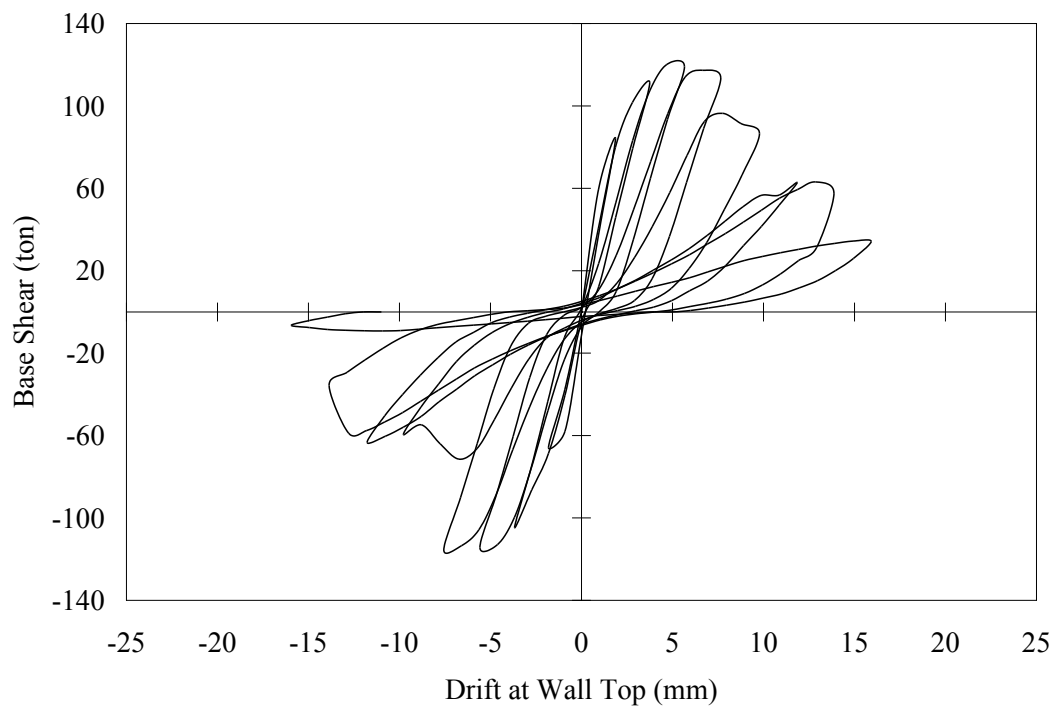
**Figure 8.13** Finite Element Mesh for SMA Shear Wall

The first and the second zones were assigned to the foundation and the top beam, respectively. The third zone corresponded to the web wall and the fourth zone to the barbell ends of the wall. Plane stress rectangular elements were used to define the mesh for the concrete layers. The diagonal SMA bars were modeled with truss bar elements as shown in Figure 8.13. These truss elements were perfectly connected to the concrete only at the base and top of the wall to simulate the unbonded condition of the external SMA bars. The web wall mesh consisted of 22 rectangular elements horizontally and 8 rectangular elements vertically. A total of 446 rectangular elements were used for the concrete, and two truss elements were used to model the diagonal SMA bars. The simulated loading followed the same protocol as used during testing.

There was good correlation between the reported and the numerical hysteretic responses as displayed in Figures 8.14 and 8.15, respectively.



**Figure 8.14** Experimental Load-Displacement Response for Wall SMAS (Effendy et al. 2006)

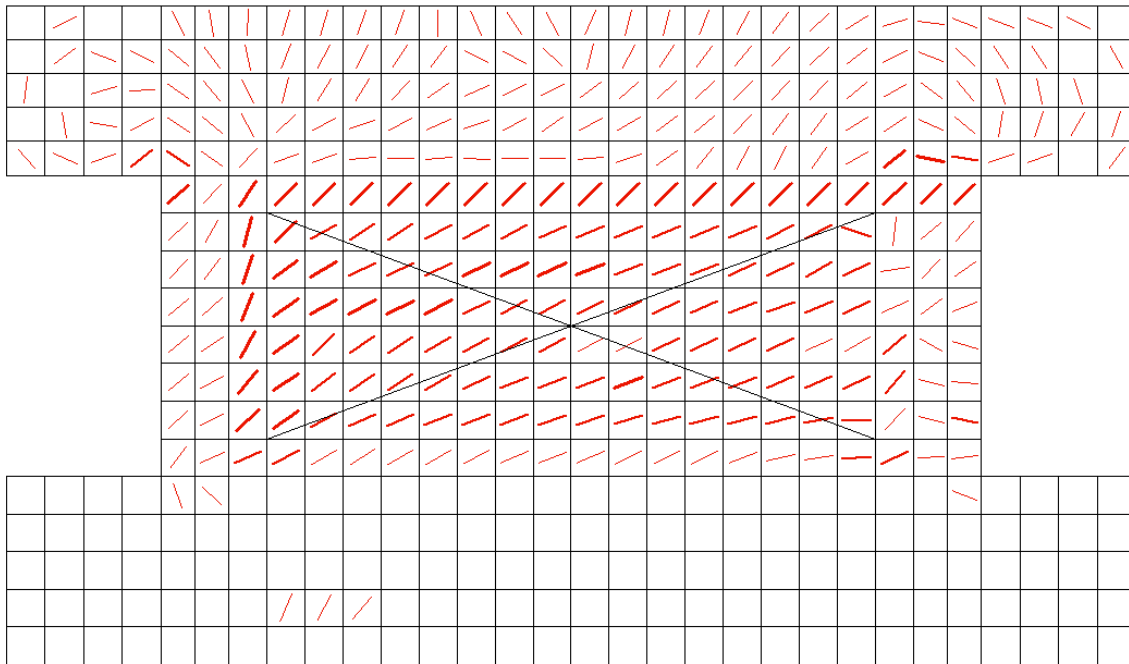


**Figure 8.15** Numerical Load-Displacement Response for Wall SMAS

The predicted initial stiffness, yield strength and peak strength were higher than the corresponding experimental values by 21%, 16% and 6%, respectively. Post-peak degradation was captured by the analysis and was similar to the experimental response. The ability of the SMA to recover the lateral displacement was evident in the analysis with slightly higher pinching than that recorded during the test. The predicted ultimate ductility closely matched the test. Buckling of the SMA, which was noted during testing, was not predicted in the analysis since it was not explicitly modeled; however, both the numerical and observed failures involved severe crushing of the concrete in the web as shown in Figure 8.16.



(a)



(b)

**Figure 8.16** Crack Patterns in SMA Shear Wall SMAS: (a) Experimental (Effendy et al. 2006); and (b) Numerical

## CHAPTER 9

### CONCLUSIONS AND RECOMMENDATIONS

#### 9.1 Conclusions

The ability of superelastic Shape Memory Alloys, specifically Nitinol, to recover displacements and dissipate energy in flexural beams and a shear wall was experimentally and numerically verified. In general, the results demonstrated that Nitinol is an excellent alloy for controlling residual displacements when used as longitudinal reinforcement in concrete beams and in critical flexural regions in shear walls. In addition, a constitutive plastic offset hysteretic model for superelastic SMA compatible with a compression-field approach was proposed. The following conclusions are based on the experimental and numerical findings.

##### **Reinforced Concrete Beams**

1. Nitinol was superior to the conventional deformed steel at limiting residual displacements in the concrete beams.
2. The stiffness of SMA reinforced beams was lower than those of the steel reinforced beams. Comparisons made between the SMA and steel reinforced specimens demonstrated that the stiffness was reduced by approximately 40%.
3. Nitinol reinforced beams experienced higher normalized yield and ultimate loads in comparison to the conventional reinforced beams, provided the Nitinol bars did not rupture at the threaded ends near the mechanical couplers or at the transition zone.
4. The SMA beams sustained comparable displacement ductility to the conventional reinforced beams.

5. The SMA beam, under cyclic loading, dissipated energy comparable to the conventional reinforced beam. However, under reverse cyclic loading, the SMA reinforced beam dissipated approximately 54% of the energy dissipated by the conventional reinforced beam.
6. The crack widths were larger and spaced farther in the Nitinol reinforced beams due to the smooth bar surface. However, Nitinol bars are more resilient to corrosion and, thus, Nitinol reinforced members can tolerate larger crack widths. Furthermore, the SMA reinforced beams were effective at limiting the residual crack widths after removal of load.
7. Threaded couplers used to connect Nitinol bars with deformed bars should be located a sufficient distance from the critical region to avoid failure of the Nitinol bar at the location of the coupler.
8. The beam tests demonstrated that using only Nitinol as longitudinal reinforcement reduced the stiffness of the beams, whereas yield and ultimate loads were not affected.

### **Reinforced Concrete Shear Walls**

1. Hybrid systems consisting of Nitinol longitudinal reinforcement combined with steel reinforcement in the plastic hinge region were investigated in large-scale shear wall tests.
2. The normalized load-displacement ductility responses of the Nitinol and conventional reinforced walls indicated that W2-NR (Nitinol-Hybrid) had lower initial stiffness than W1-SR (conventional); however, both walls experienced comparable normalized yield and ultimate loads.



3. The normalized load-displacement ductility responses demonstrated the ability of both specimens to sustain large inelastic deformations without suffering a significant loss in load carrying capacity.
4. The SMA wall demonstrated the ability to recover significant inelastic displacements in spite of the presence of conventional longitudinal reinforcement in the web portion of the wall. The average recovery was approximately 85% throughout testing.
5. It was evident from test results that the SMA in W2-NR was able to overcome the resistance of the conventional steel to re-centering. Although the conventional steel in W2-NR yielded prior to the SMA bars, the wall still achieved significant recovery in comparison to W1-SR.
6. The strain gauges in the boundaries demonstrated that plasticity spread over a length of at least 800 mm at the peak load in W1-SR. The same yield length developed in W2-NR; however, it occurred prior to the peak load.
7. Based on ductility, the SMA wall dissipated strain energy higher than the conventional wall. At maximum displacement ductility in W1-SR, 5831 Nm and 8500 Nm of energy were dissipated by the conventional and SMA walls, respectively, an increase of 45% for the SMA wall. On the other hand, the conventional wall dissipated significantly more energy than the SMA wall when the comparison was based on lateral drift.
8. The vertical displacements were not proportional along the height of the wall as a result significant cracking occurring in the lower part (plastic hinge region) of the wall. In the post-yield regime, the vertical displacements recorded at 800 mm above

the base foundation was about 69% and 80% of the total vertical displacement capture at the top beam in W1-SR and W2-NR, respectively.

9. The higher yield strain of the SMA bar resulted in larger rotations in W2-NR when compared with W1-SR. The rotation in W2-NR had an average of 2.7 times the corresponding rotation in W1-SR at different displacement ductility levels.
10. Yielding and rupturing of the web vertical steel reinforcement prior to the SMA bars in the boundary zones indicates that code requirements for the design of reinforced concrete structures need to be revised for hybrid concrete sections reinforced with steel and SMA bars. Limiting maximum strains would be required in the SMA reinforcement to prevent rupture of the web reinforcement.
11. The use of mechanical couplers in the SMA wall was an excellent alternative to the threaded couplers that were used in the beams. The use of mechanical couplers eliminated the need for threading and reshaping the SMA bars.

### **SMA Constitutive Hysteretic Modeling**

1. The proposed constitutive plastic offset SMA hysteretic model consisting of a trilinear backbone envelope and linear segments to simulate unloading and reloading was capable of predicting the force-displacement response, energy dissipation, and residual displacements of the SMA reinforced beams and shear wall of this study.
2. The proposed model provides improvements to other models including explicit consideration of permanent strains; the degradation of the lower plateau stress (reverse transformation), which results from the recovery of strains; and a trilinear unloading response.

3. The predicted and recorded hysteretic responses were in good agreement in terms of yield and maximum strength, residual displacements, pre-yielding and post-yielding stiffness, and overall hysteretic behavior.
4. The analyses satisfactorily captured the permanent displacements; however, more pinching was noted in the experimental responses of test specimens. Another noticeable discrepancy between the calculated and observed behaviors was the degree of degradation in the reloading stiffness of repeated cycles at each displacement level where the degradation was slightly underestimated in the analyses.
5. The numerical and experimental dissipated energies in the beams and shear wall were in close agreement, which demonstrates the adequacy of the finite element program and the proposed hysteretic constitutive SMA model to capture the behavior of the specimens. In SMA wall W2-NR, an increasing difference was noted for displacement ductility greater than 2.7. The discrepancy is attributed to the strength degradation and significant recovery observed at higher ductility levels during the test associated with fracturing of the web vertical reinforcement, which was not captured in the analysis.
6. The proposed constitutive model provided reasonable simulations of various types of concrete structural elements reinforced with superelastic SMA demonstrating the applicability of the model in conjunction with VecTor2 to predict the response of SMA reinforced concrete members.
7. Future refinements in the model, including nonlinear unloading would provide improvements in the simulation.

## **General Conclusions**

1. The ability of SMA to limit residual displacements and dissipate energy leads to the possibility of using SMA bars to strengthen reinforced concrete structures designed for seismic zones.
2. The ability of SMA to recover significant inelastic displacements permits repairing of damaged structural elements by replacing the damaged concrete only.
3. Traditional methods for the analysis and design of reinforced concrete structures can be used for concrete sections reinforced with SMA bars.
4. Given the high initial cost of Nitinol, the implementation of the material in concrete structures should be limited to critical regions. The efficient use of Nitinol can substantially mitigate earthquake risk and reduce the associated cost after a major earthquake.

In summary, the capacity to recover inelastic displacements, exhibit yielding (forward transformation) and strain hardening, while sustaining large displacement ductility are structural characteristics that make superelastic SMA a potential alternative reinforcement in seismic design. However, the high initial cost may limit where the material is used. Thus, optimization is important in the design process for SMA reinforced members.

## **9.2 Recommendations**

1. Additional experimental data of SMA coupon tests is required to refine the proposed constitutive SMA model, including nonlinear unloading and reloading rules in addition to the plastic deformation.

2. Additional testing of concrete structures of different shapes and functions (columns, slabs, squat walls, etc...) are required to investigate the applicability of using superelastic SMA as reinforcement in different structures.
3. Further studies are required to investigate the potential use of superelastic shape memory alloys for repair and/or retrofitting of reinforced concrete structures.

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## APPENDICES

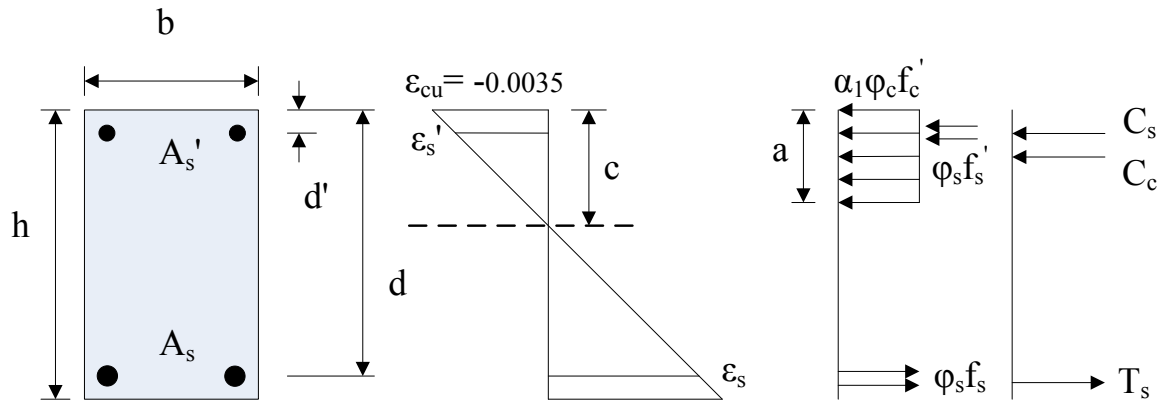
### Appendix A

#### Design of Test Beams

##### A.1 Design of SMA Reinforced Test Beams

##### A.1.1 Calculation of Moment Resistance for SMA Beams B4-NM and B5-NC According to CSA A23.3-04

Perfect bond is assumed between the concrete and SMA bars



Concrete:  $f_c' = 30 \text{ MPa}$

Steel:  $f_y = 570 \text{ MPa}$ ,  $E = 203 \text{ GPa}$

SMA:  $f_y = 415 \text{ MPa}$ ,  $E = 60 \text{ GPa}$

$b = 125 \text{ mm}$ ,  $h = 250 \text{ mm}$ ,  $\text{cover} = 20 \text{ mm}$

$$d = 250 - (20 + 6.35 + \frac{9.5}{2}) = 218.9 \text{ mm}$$

$$d' = 20 + 6.35 + \frac{6.35}{2} = 29.5 \text{ mm}$$

$$\alpha_1 = 0.805$$

$$\beta_1 = 0.895$$

$$T_s = A_s \phi_s f_y$$

$$T_s = 2(71)(1)(415) = 58.9 \text{ kN}$$

$$C_s' = A_s'(\phi_s f_y - \alpha_1 \phi_c f_c')$$

$$C_s' = 2(32)\{(1)(570) - 0.805(1)(30)\} = 34.9 \text{ kN}$$

$$a = \frac{T_s - C_s'}{\alpha_1 \phi_c f_c' b} = 7.95 \text{ mm}$$

$$\rightarrow c = \frac{a}{\beta_1} = 8.88 \text{ mm}$$

$$\rightarrow c < d'$$

Upper steel not yielded

Use strain compatibility

$$c = 25 \text{ mm}, T_s = 58.9 \text{ kN}, C_s' = 18.1 \text{ kN}, C_c = -67.5 \text{ kN}, a = 22.4 \text{ mm}$$

$$\rightarrow M = 12.46 \text{ kN.m}$$

$$M = \frac{P(l - 0.125)}{2}$$

$$\rightarrow P = \frac{4(12.46)}{2.4 - 0.125}$$

$$P = 21.9 \text{ kN}$$

Check for yielding

$$\frac{c}{d} \leq \frac{\epsilon_c}{\epsilon_c + \epsilon_y} = \frac{0.0035}{0.0035 + 415/60000} = 0.336$$

$$\frac{c}{d} = \frac{25}{218.9} = 0.114 < 0.336 \text{ OK}$$

Check for  $A_{s \min}$

$$A_{s \min} = \frac{0.2 \sqrt{f_c'}}{f_y} b h = 82 \text{ mm}^2 < A_s(142) \text{ OK}$$

Check for Shear

$$V_f = \frac{P}{2} = 10.95 \text{ kN}$$

$$V_r = V_c + V_s$$

Use stirrups 6.35 mm dia. @ 100mm

$$V_c = 0.2\phi_c\lambda\beta\sqrt{f'_c} b_w d$$

$$V_c = 0.2(1)(1)(0.18)\sqrt{30}(125)(218.9)$$

$$V_c = 5.4 \text{ kN}$$

$$V_s = \frac{\phi_s A_v f_y d_v \cot \theta}{s}$$

$$\theta = 35$$

$$d_v = \min \text{ of } (0.9d) \text{ or } (0.72h)$$

$$d_v = 196.2 \text{ mm}$$

$$\rightarrow V_s = 102 \text{ kN}$$

$$\rightarrow V_c + V_s > V_f$$

A.1.2 Calculation of Moment Resistance for SMA Beam B6-NR According to CSA

A23.3-04

$$\rightarrow M_r = 12.82 \text{ kN.m}$$

$$\rightarrow P = 22.54 \text{ kN}$$



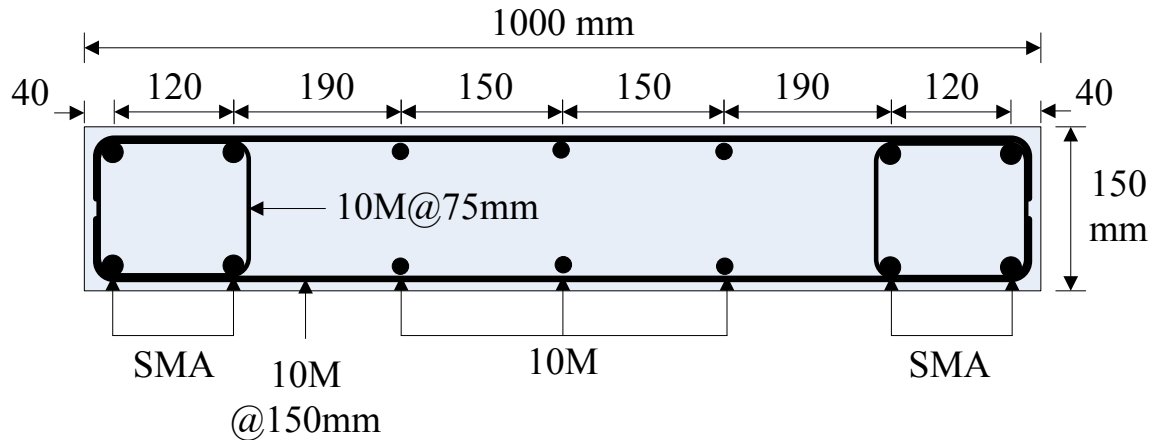
## Appendix B

### Design of Test Shear Walls

#### B.1 Design of SMA Shear Wall (W2-NR)

The design of the wall was according to the requirements of CSA A23.3-04

Perfect bond is assumed between the concrete and SMA bars



Concrete:  $f'_c = 30 \text{ MPa}$

Steel:  $f_y = 425 \text{ MPa}$ ,  $E = 202 \text{ MPa}$

SMA:  $f_y = 380 \text{ MPa}$ ,  $E = 36.5 \text{ GPa}$

The wall was designed as a ductile wall with force modification factor  $R_d$  of 3.5. The length and the height of the wall were 1000 mm and 2200 mm, respectively.

$$\frac{h}{l_w} = 2.2 \rightarrow \text{Ductile wall}$$

Dimension Limitations

CL 21.6.3

For stability:

$$\frac{l_w}{t} \leq 10$$

$$\text{Use } t = 150 \text{ mm} \rightarrow \frac{l_w}{t} = 6.7$$

$$l_w = 1000 \text{ mm}, t = 150 \text{ mm}, \text{ cover} = 20 \text{ mm}$$

Concrete cover of 20 mm will satisfy the requirements of CL A12.6.2

Use 10M for longitudinal and transverse reinforcement

Use 12.7 mm diameter SMA bars as concentrated reinforcement at the boundaries

Concentrated Reinforcement Requirements:

Four SMA bars in two layers were provided at each end of the wall CL 21.6.6.1

Bar diameter CL 21.6.4.4

$$d_b \leq \frac{b_w}{10}$$

$$d_b = 12.7, \frac{b_w}{10} = 15$$

Minimum steel area CL 21.6.4.3 and CL 21.6.6.4

$$A_s = 4(126) = 504 \text{ mm}^2$$

$$\text{Steel ratio } \rho = \frac{504}{150(200)} = 0.0168 \leq 0.06$$

$$A_s = 504, 0.0015(b_w)(l_w) = 0.0015(150)(1000) = 225$$

$$\rightarrow A_s > 0.0015(b_w)(l_w)$$

Hoop Reinforcement Requirement in Plastic Hinge Region CL 21.6.6.9

Use 10M for providing ties in all regions

In plastic hinge region the tie spacing shall not exceed the minimum of

$$6d_b = 6(12.7) = 76.2 \text{ mm}$$

$$24d_h = 24(11.3) = 271.2 \text{ mm}$$

$$\frac{1}{2} b_w = 0.5(150) = 75 \text{ mm}$$

→ Use Ties placed at at spacing 75mm in plastic hinge region

In regions other than plastic hinge the tie spacing shall not exceed the minimum of

$$16d_b = 16(12.7) = 203.2 \text{ mm}$$

$$48d_h = 48(11.3) = 542.4 \text{ mm}$$

Least dimension of the member = 150mm

→ Use Ties placed at at spacing 150 mm in other than plastic hinge region

Distributed Reinforcement CL 21.6.5.1, CL 21.6.5.2, and CL 21.6.5.3

Two layers of vertical and horizontal reinforcement were provided using 10M spaced at 150mm in vertical and horizontal directions in all regions.

$$\rho_{\text{transverse}} = \frac{200}{(150)(150)} = 0.88\% > 0.25\%$$

spaced at 150mm < 300mm (plastic hinge)

< 450mm (other regions)

Moment and Shear Capacity

Probable Moment  $M_{pw}$

$$\phi_s = \phi_t = 1, \quad f_s = 1.25 f_y$$

Using strain compatibility  $\rightarrow M_{pw} = 312 \text{ kN.m}$

Shear Design in Plastic hinge region

$$V_d = \frac{M_{pw}}{2.4} = \frac{312}{2.4} = 130 \text{ kN.m}$$

Max. Shear Capacity  $V_{f \max} = 0.1 \phi_s f_c' b_w d_v$

$$\rightarrow V_{f \max} = 368 \text{ kN}$$

The spacing of the horizontal distributed steel in the plastic hinge region

$$s \leq \frac{\phi_s A_y f_y d_v \cot \theta}{V_d}$$

$$s \leq 264 \text{ mm}$$

$\rightarrow$  Using 10M spaced at 150mm is adequate

Ductility

The wall was designed to have ductility-force modification factor ( $R_d$ ) of 3.5

Using strain compatibility:

$$M_y = 244 \text{ kN.m}, \quad V_y = 101 \text{ kN}, \quad c = 172 \text{ mm}$$

$$M_u = 301 \text{ kN.m}, \quad V_u = 125 \text{ kN}, \quad c = 149 \text{ mm}$$

$$\phi_y = \frac{\epsilon_{cc}}{c} = \frac{0.0014}{172} = 8.1(10)^{-6}$$

$$\phi_u = \frac{\epsilon_{cu}}{c} = \frac{0.0035}{149} = 23.5(10)^{-6}$$

$$\rightarrow \Delta_y = 13.25 \text{ mm}, \quad \Delta_u = 24.9$$

$$\frac{\Delta_u}{\Delta_y} = 1.8 < R_d(3.5)$$

For ductility of 3.5,  $\epsilon_{cu}$  has to be greater than 0.0035 but less than or equal to 0.014 and the compression region should be confined as a column according to CL 21.6.7.4

$$R_d = \frac{\Delta_y + \Delta_i}{\Delta_y} = 1 + \frac{\Delta_i}{\Delta_y}$$

$$R_d = 3.5, \quad \Delta_y = 13.25 \text{ mm}$$

$$\rightarrow \Delta_i = 33.125, \quad \phi_i = 26.4(10)^{-6}, \quad \phi_u = 34.5(10)^{-6}$$

$$\rightarrow \epsilon_{cc} = 34.5(10)^{-6} * (149) = 0.0051 < 0.014$$

Transverse reinforcement is to be determined and distributed according to CL 21.6.7.4

$$A_{sh} = 0.2 K_n K_p \frac{A_g}{A_{ch}} \frac{f'_c}{f_{yh}} s h_c$$

$$K_n = \frac{n_l}{n_l - 2} = 2$$

$$K_p = (0.1 + 30 \varepsilon_u) = 0.253$$

$$A_g = 200 \times 150 = 30(10)^{-3}$$

$$A_{ch} = 160 \times 110 = 17.6(10)^{-3}$$

$$h_c = 160$$

$$\text{For } s = 75 \text{ mm} \rightarrow A_{sh} = 148 < 200$$

$$A_{sh} \text{ is provided over a distance not less than } \frac{c(\varepsilon_{cc} - 0.0035)}{\varepsilon_{cc}}$$

$$\frac{c(\varepsilon_{cc} - 0.0035)}{\varepsilon_{cc}} = 65 \text{ mm} < 180$$

Stability

Wall dimension satisfies the requirements of CL 21.6.3

$$c = 142 < 4b_w$$

$$< 0.3l_w$$

$$\text{boundary elements: } (b_w) \times \left\{ \frac{c}{2} \text{ (at least)} \right\}$$

→ 150 x 200 is enough

Shear requirements in plastic hinge region

Sliding shear

CL 21.6.9.4

$$N = 0 \text{ kN}, C = 0.5, \mu = 1$$

$$\sigma = \rho_v f_y + \frac{N}{A_g}$$

$$\sigma = 0.0088(425) + 0 = 3.74 \text{ MPa}$$

$$v_r = \phi_c (c + \mu \sigma)$$

$$v_r = (1)(0.5 + (1)(3.74)) = 4.25 \text{ MPa}$$

$$V_r = v_r A_v = 4.25(1000)(150)$$

$$V_r = 637 \text{ kN} > V_d(130)$$

No sliding is expected

## Appendix C

### Steel and Concrete Properties

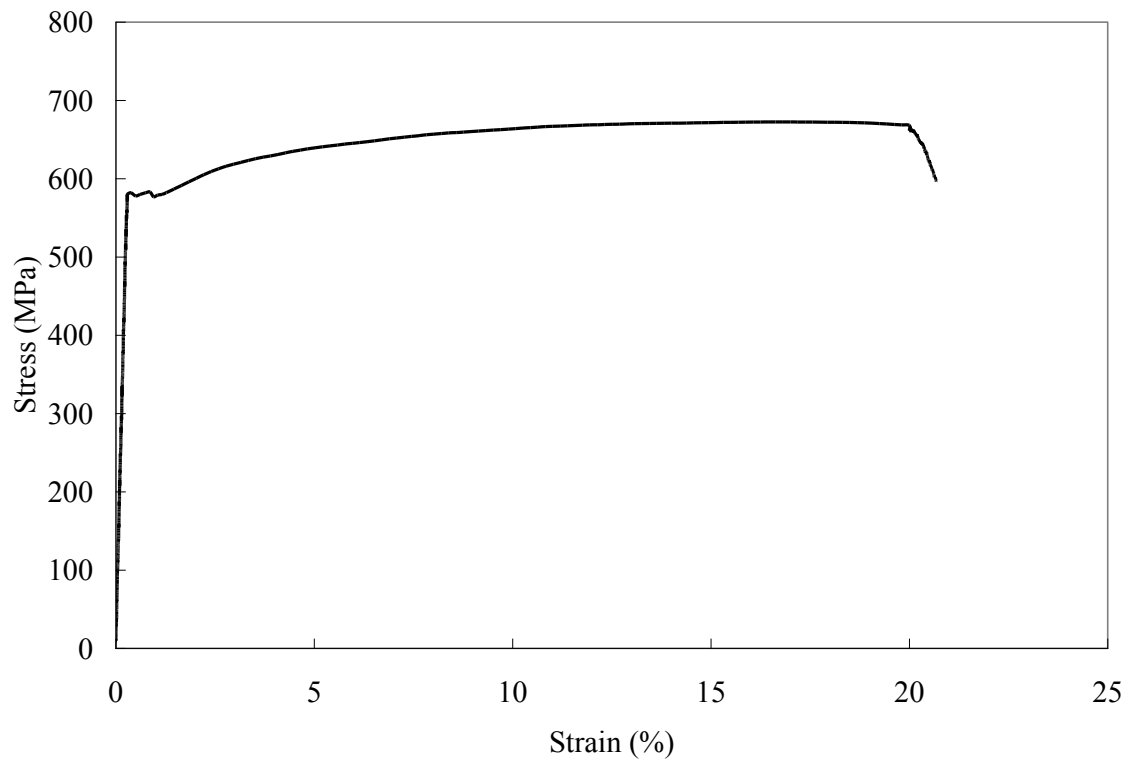


Figure C.1 Typical Strain-Stress Response of 6.35 mm Reinforcement

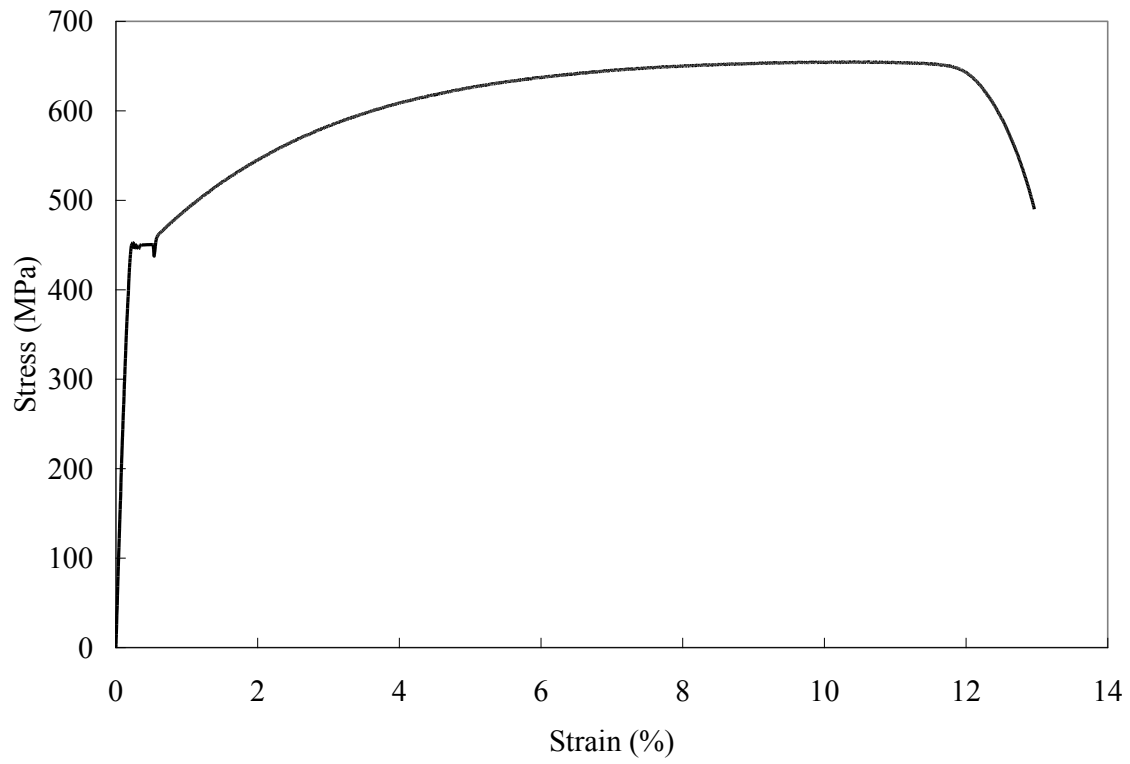


Figure C.2 Typical Strain-Stress Response of 15M Reinforcement

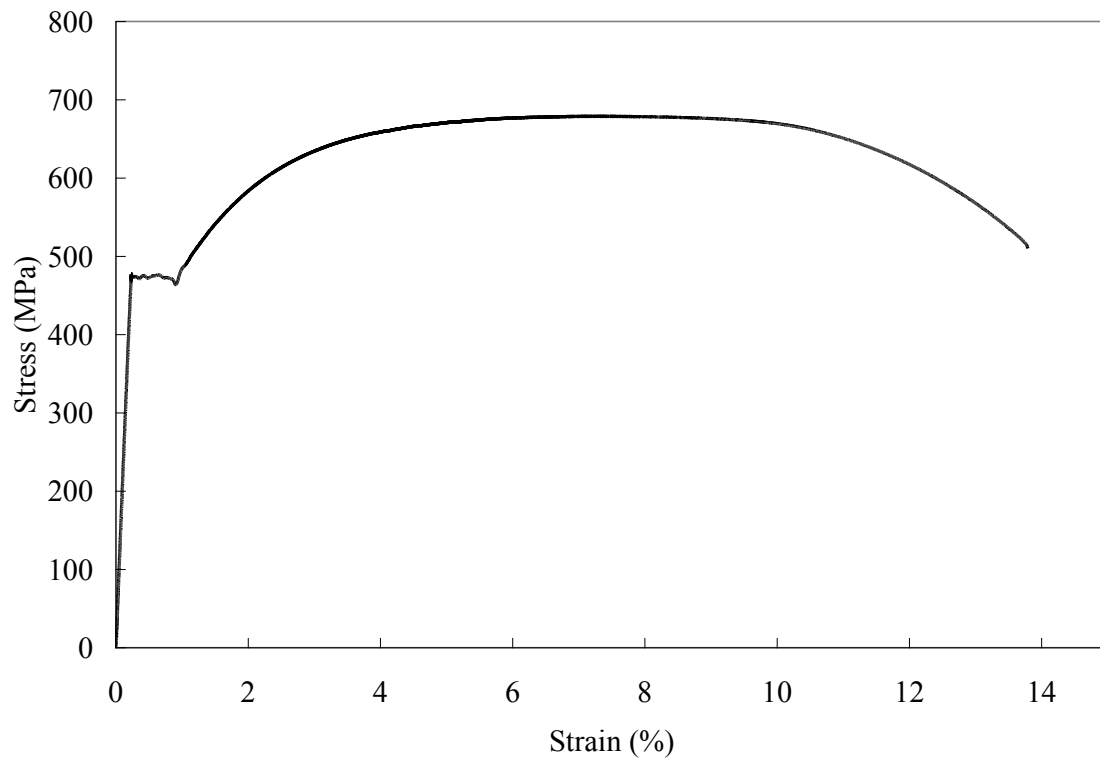


Figure C.3 Typical Strain-Stress Response of 20M Reinforcement

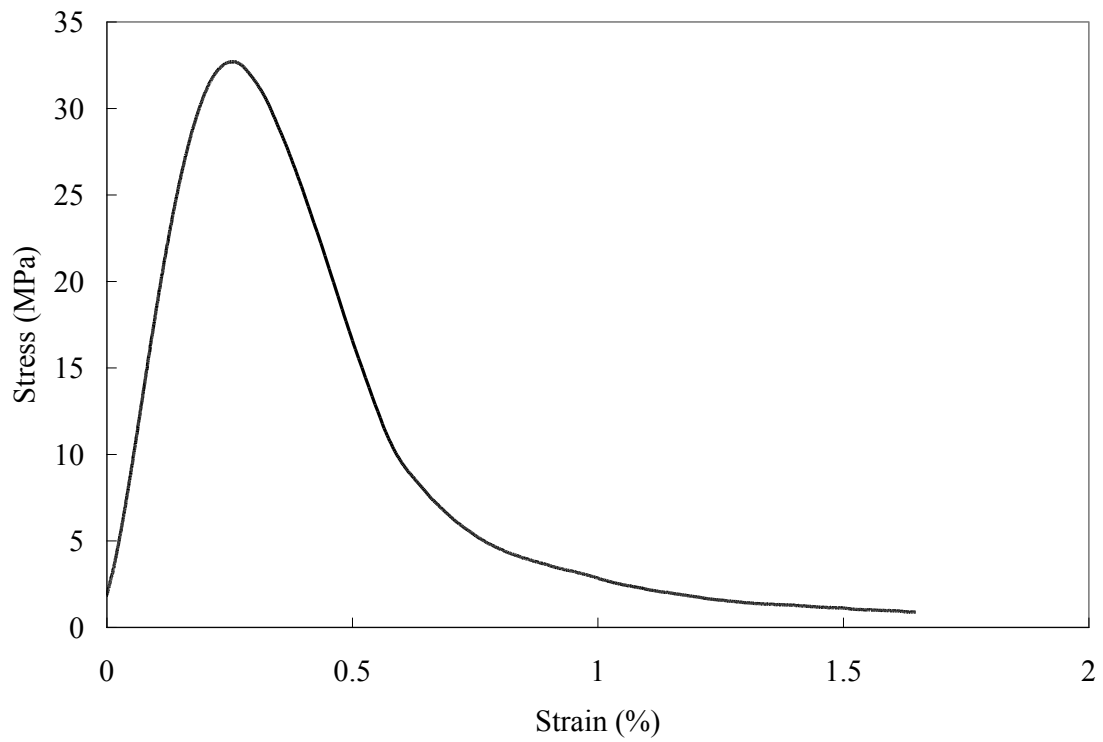


Figure C.4 Typical Stress-Strain Response of Concrete Used in SMA Beams

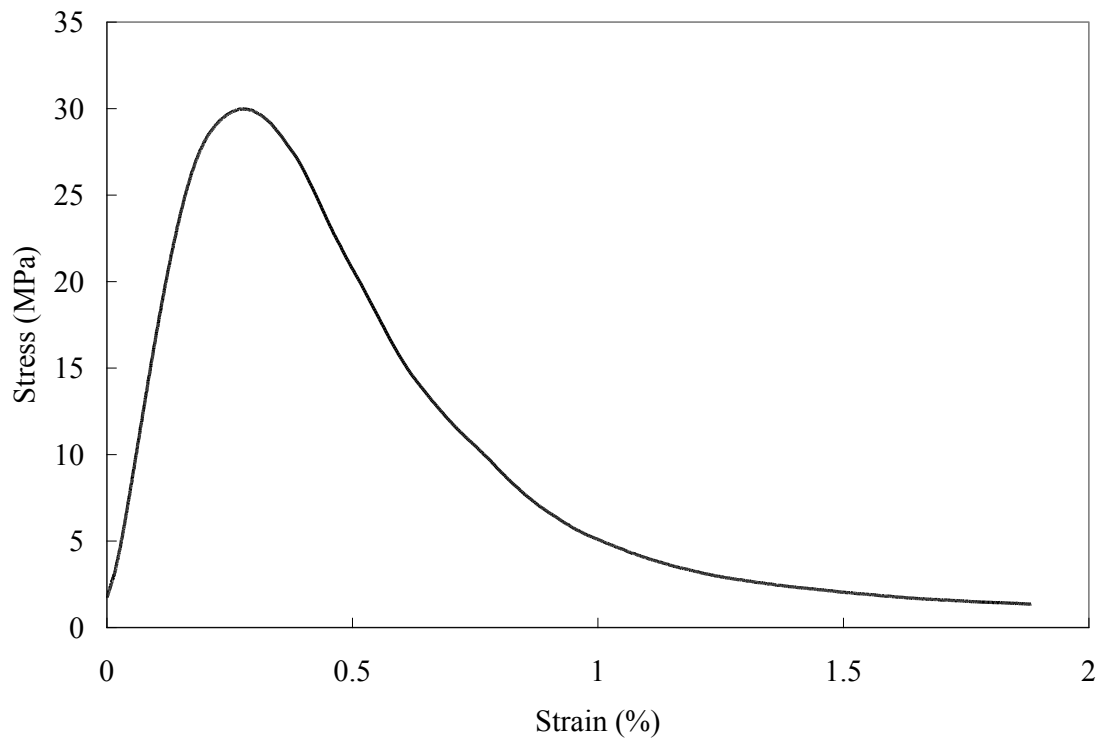


Figure C.5 Typical Stress-Strain Response of Concrete Used in Regular Shear Wall

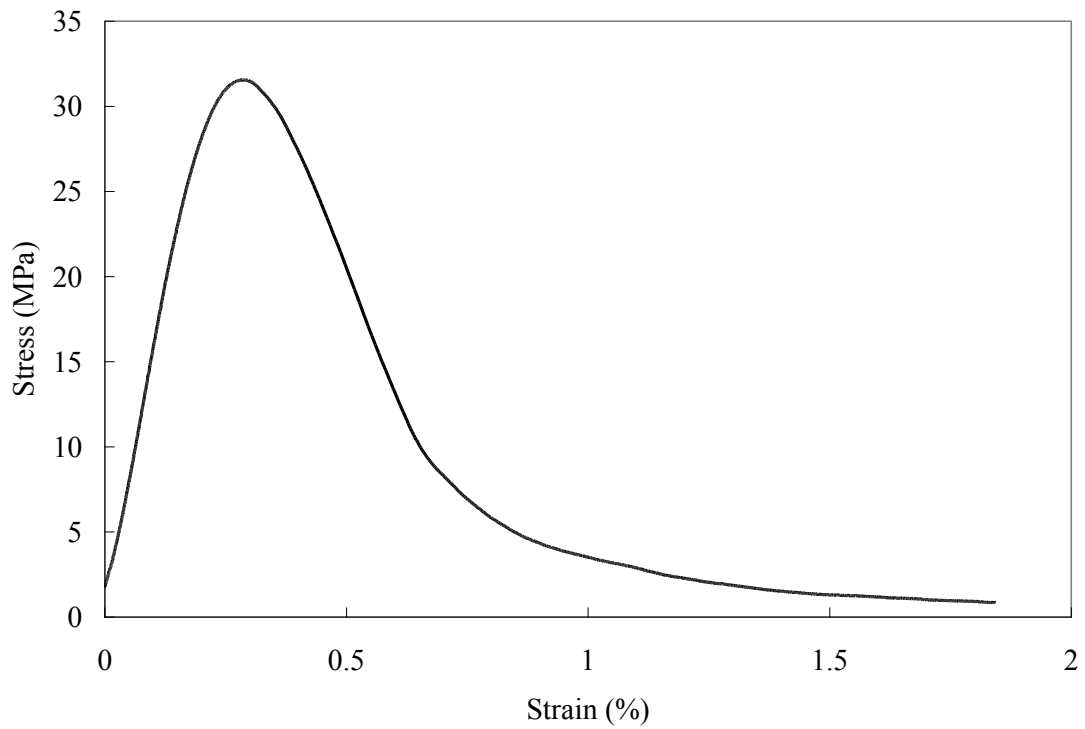


Figure C.6 Typical Stress-Strain Response of Concrete Used in SMA Shear Wall

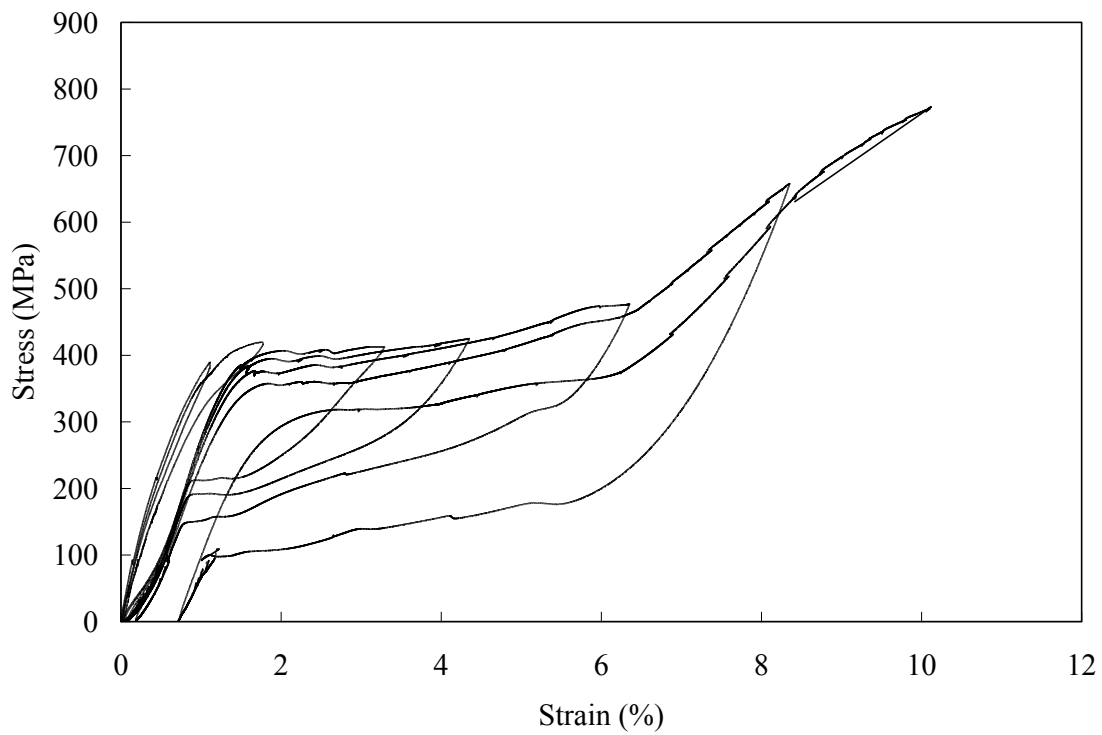


Figure C.7 Typical Stress-Strain Response of SMA Reinforcement Used in SMA Shear Wall



## Appendix D

### Displacement Cable Transducer and Linear Variable Differential Transducer Responses in Shear Walls

#### D.1 Response of Displacement Cable Transducers (DCTs) in Wall W1-SR

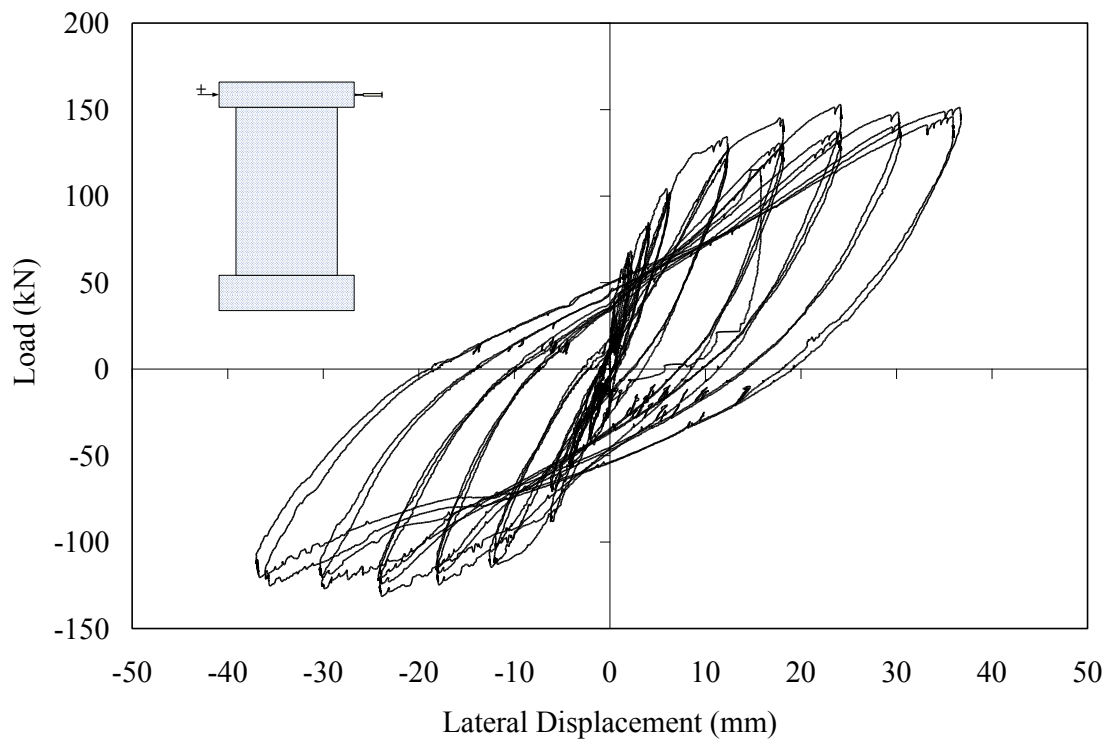


Figure D.1 DCT1 Response in W1-SR

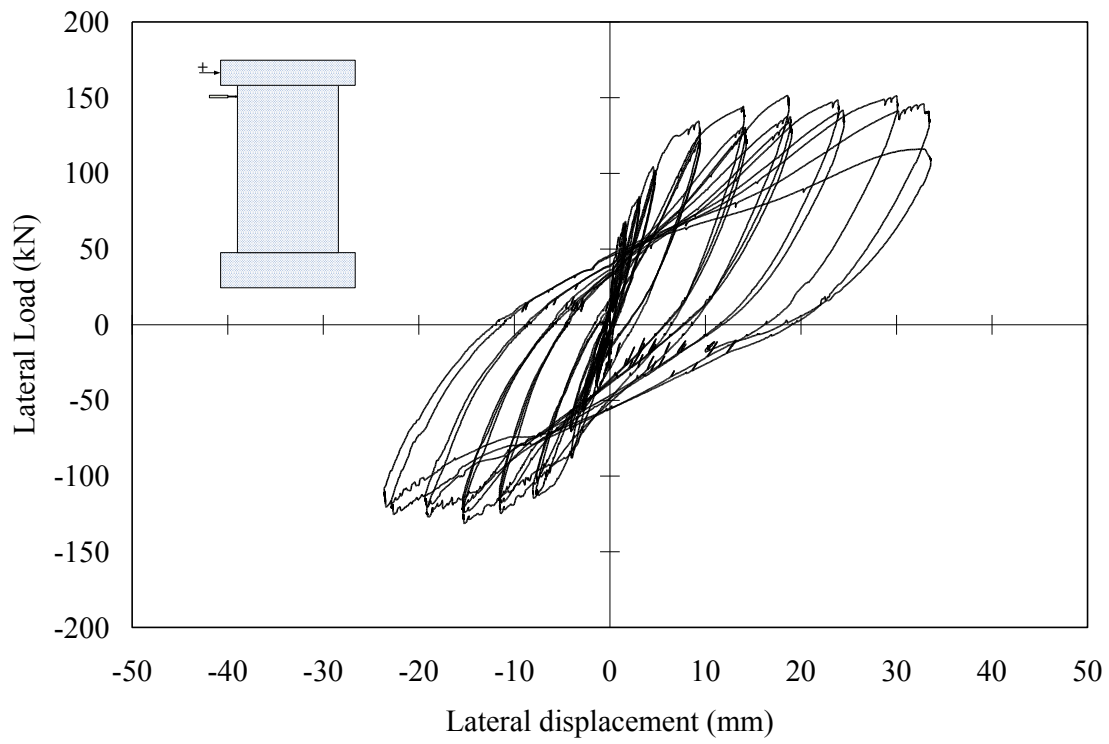


Figure D.2 DCT2 Response in W1-SR

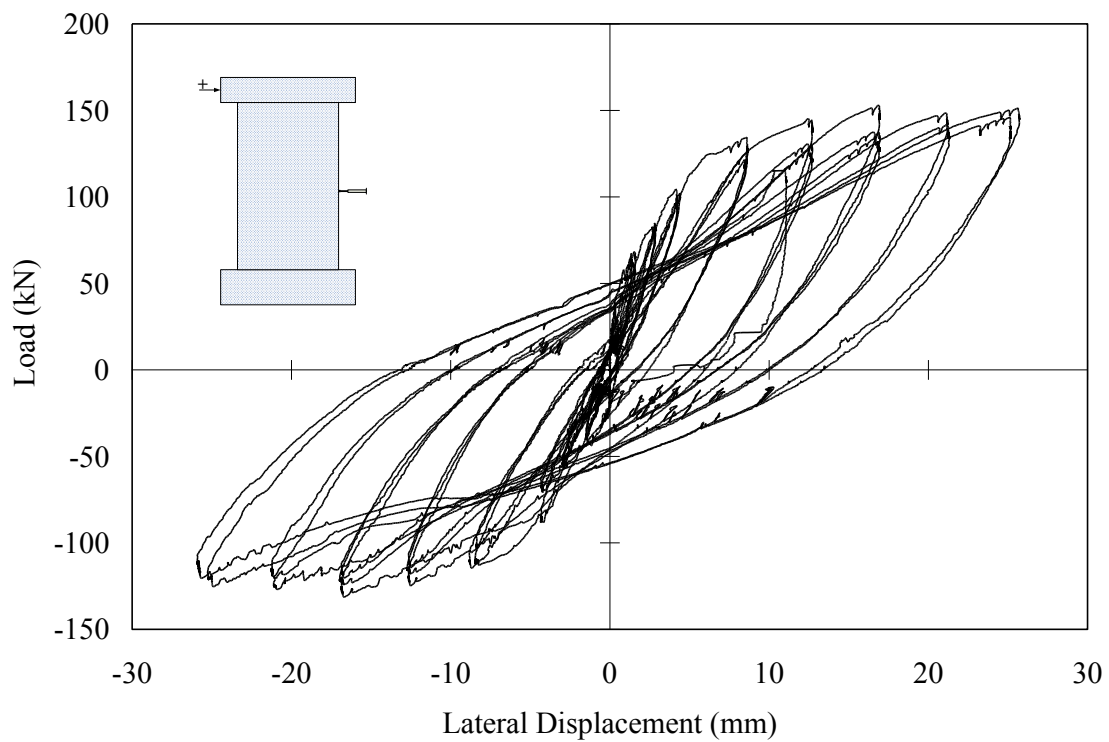


Figure D.3 DCT3 Response in W1-SR

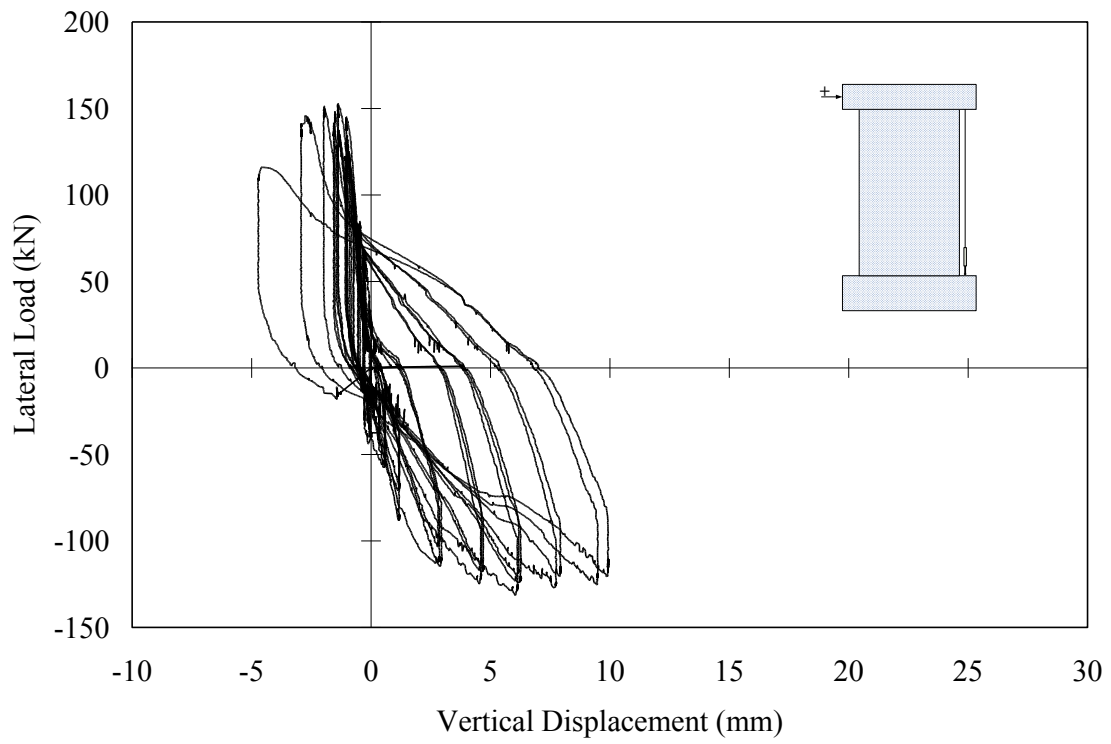


Figure D.4 DCT5 Response in W1-SR

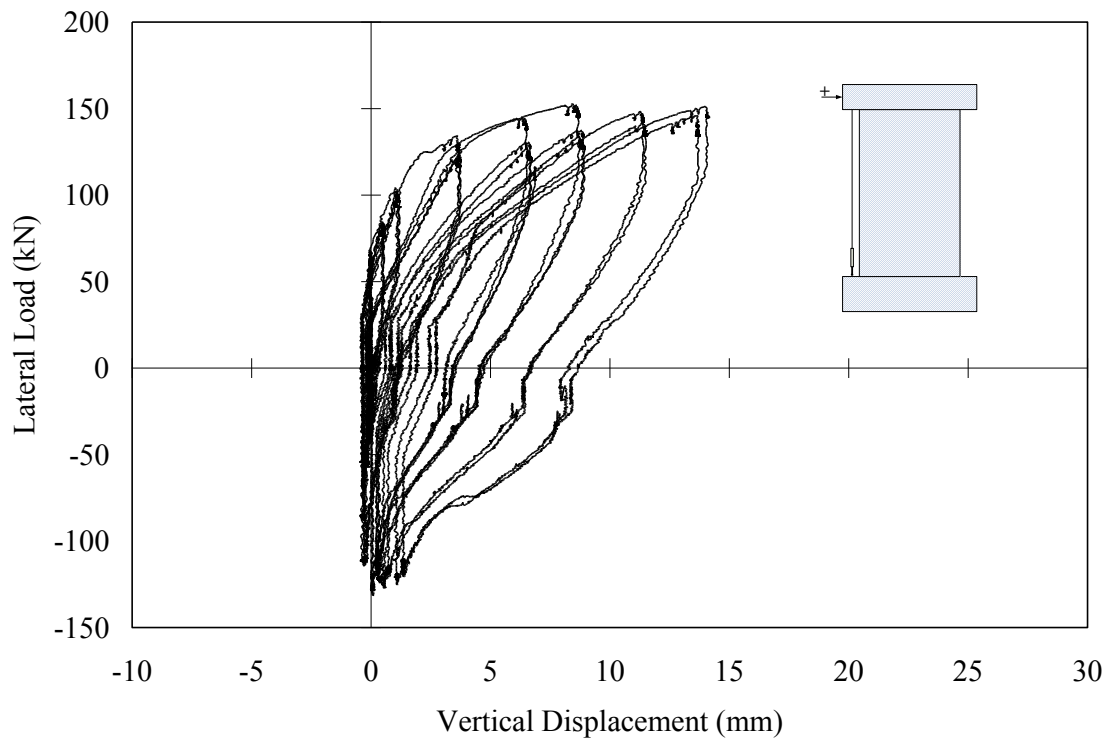


Figure D.5 DCT6 Response in W1-SR

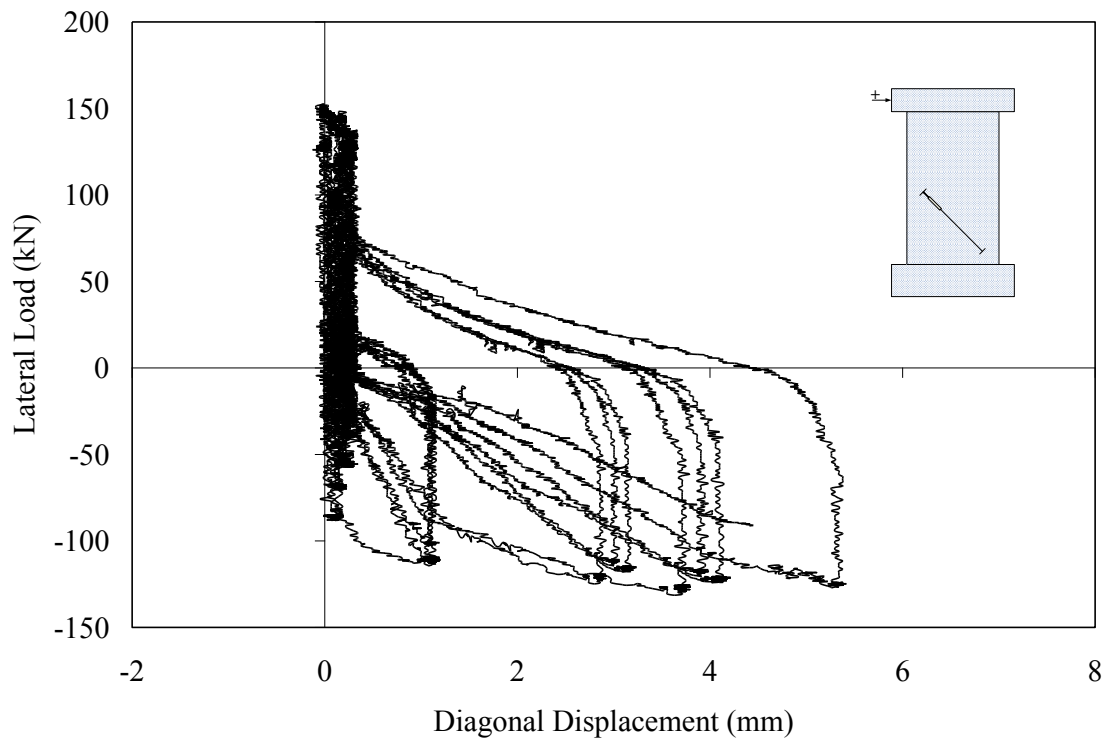


Figure D.6 DCT7 Response in W1-SR

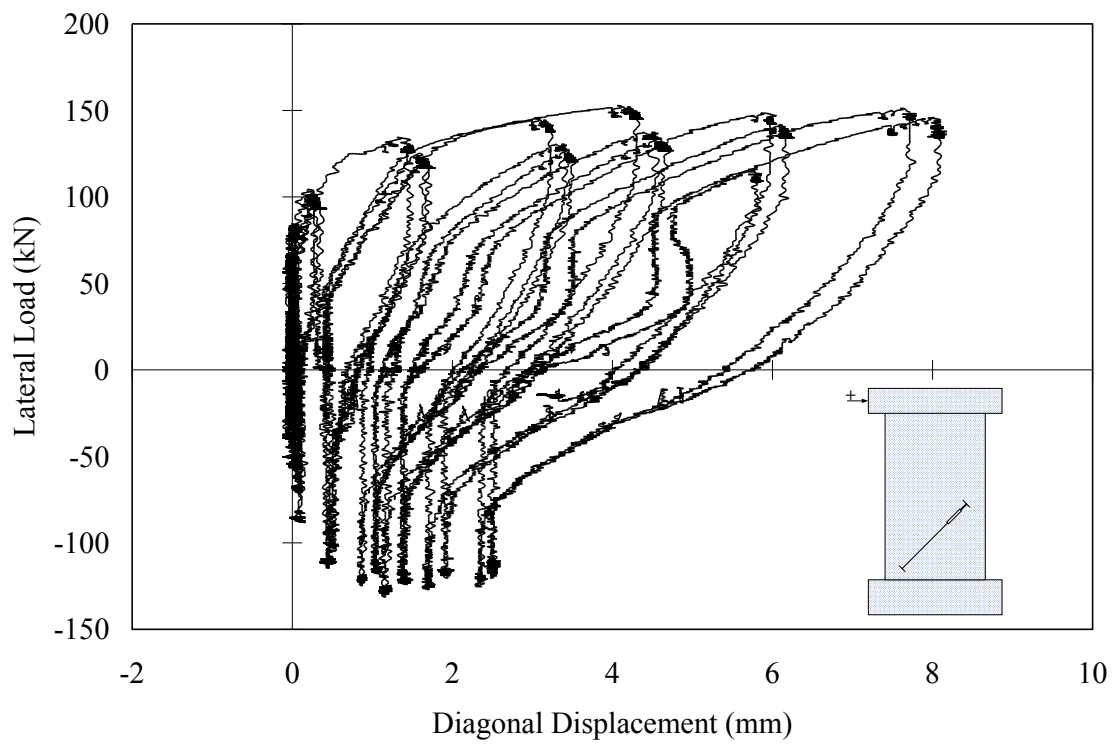


Figure D.7 DCT8 Response in W1-SR

## D.2 Response of Linear Variable Differential Transducers (LVDTs) in Wall W1-SR

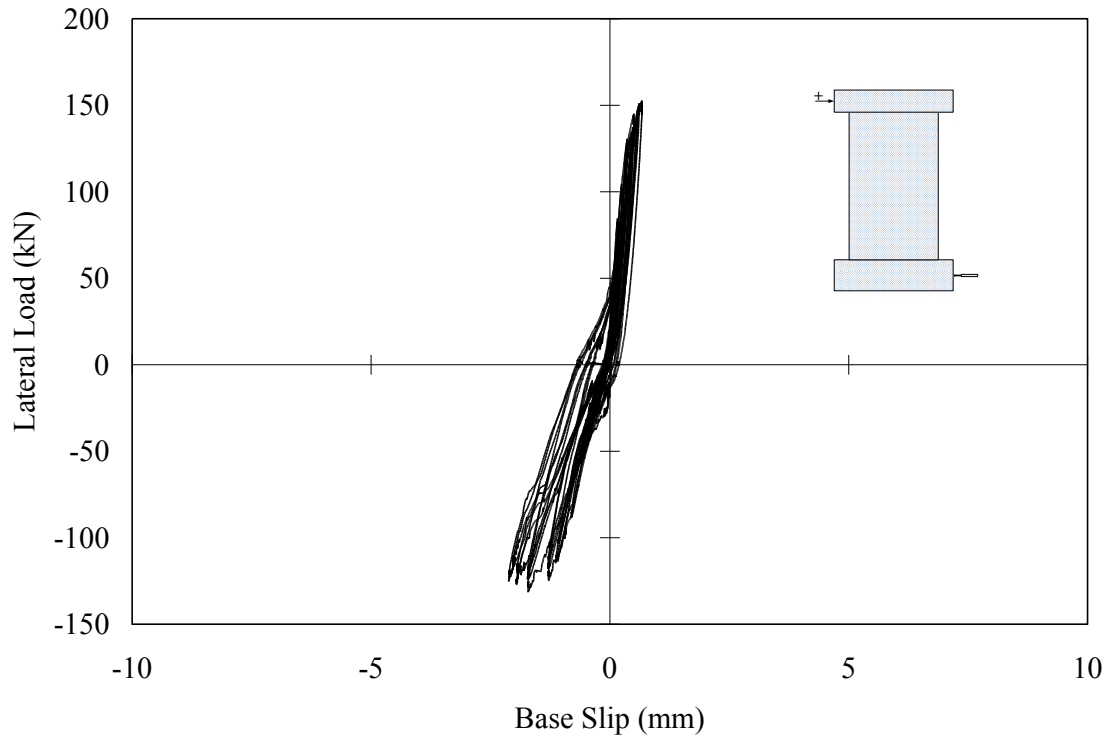


Figure D.8 LVDT 1 Response in W1-SR

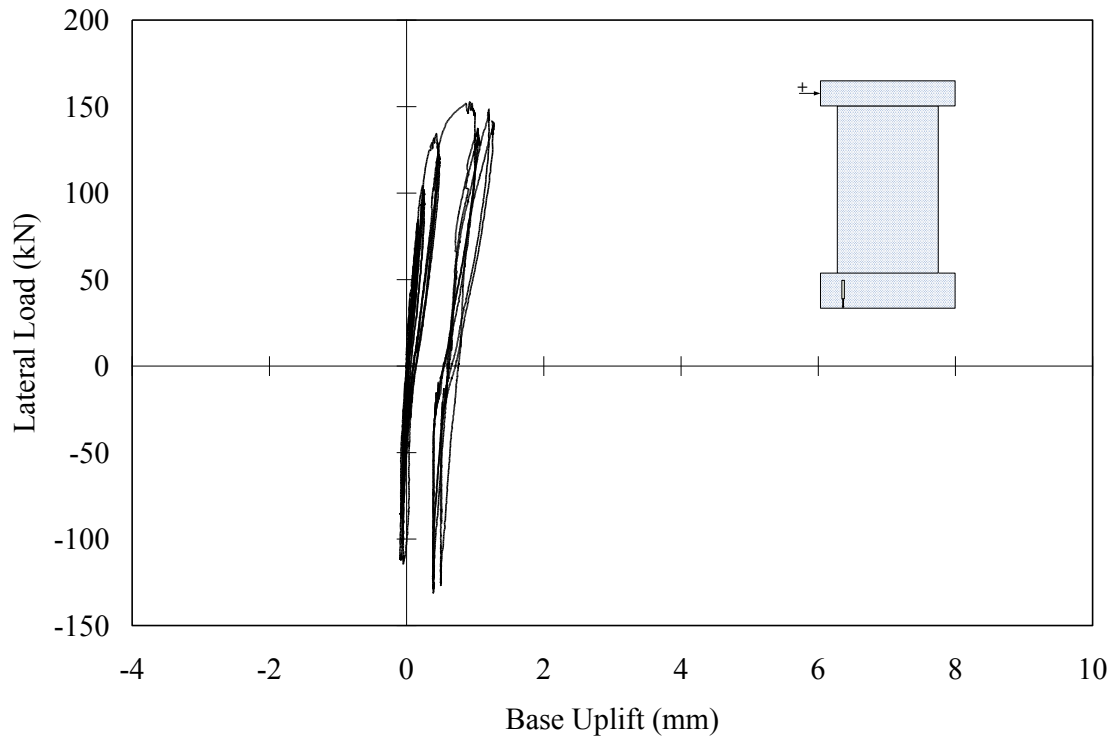


Figure D.9 LVDT2 Response in W1-SR

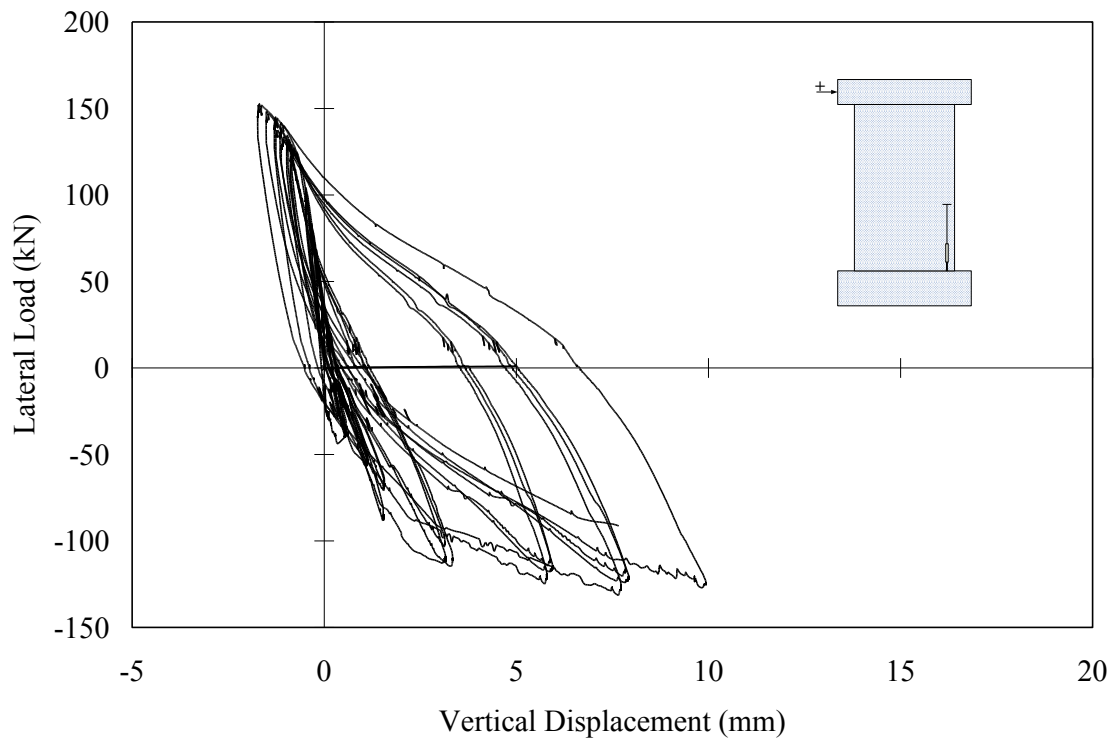


Figure D.10 LVDT3 Response in W1-SR

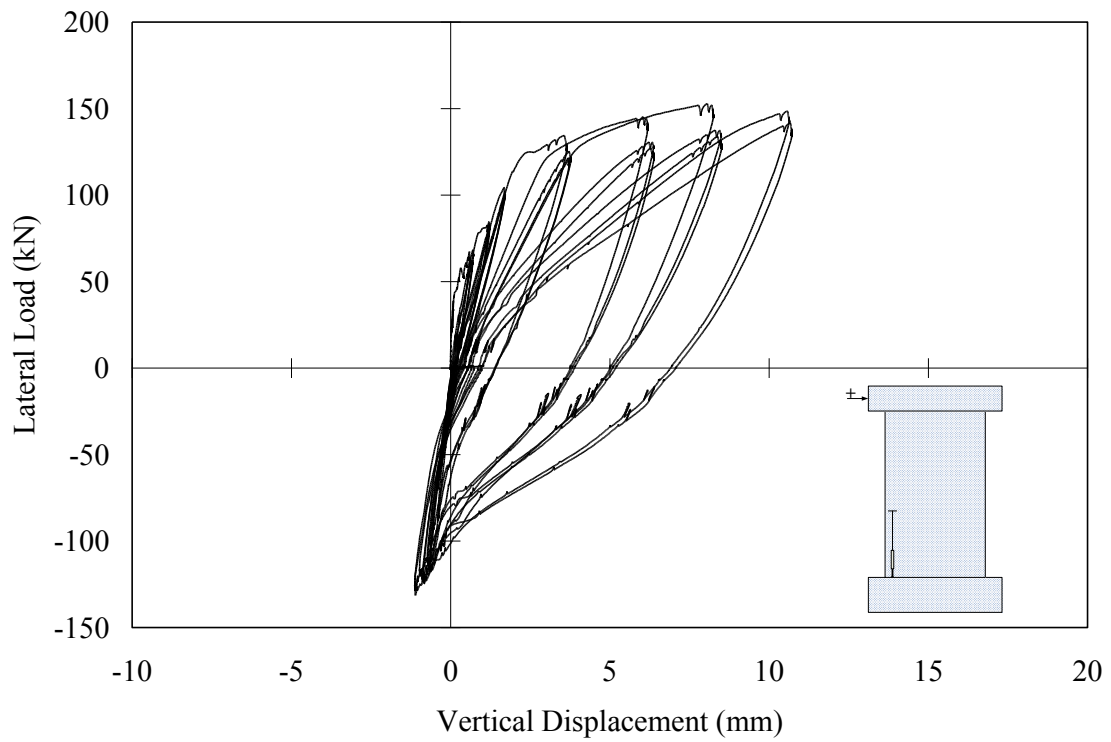


Figure D.11 LVDT4 Response in W1-SR

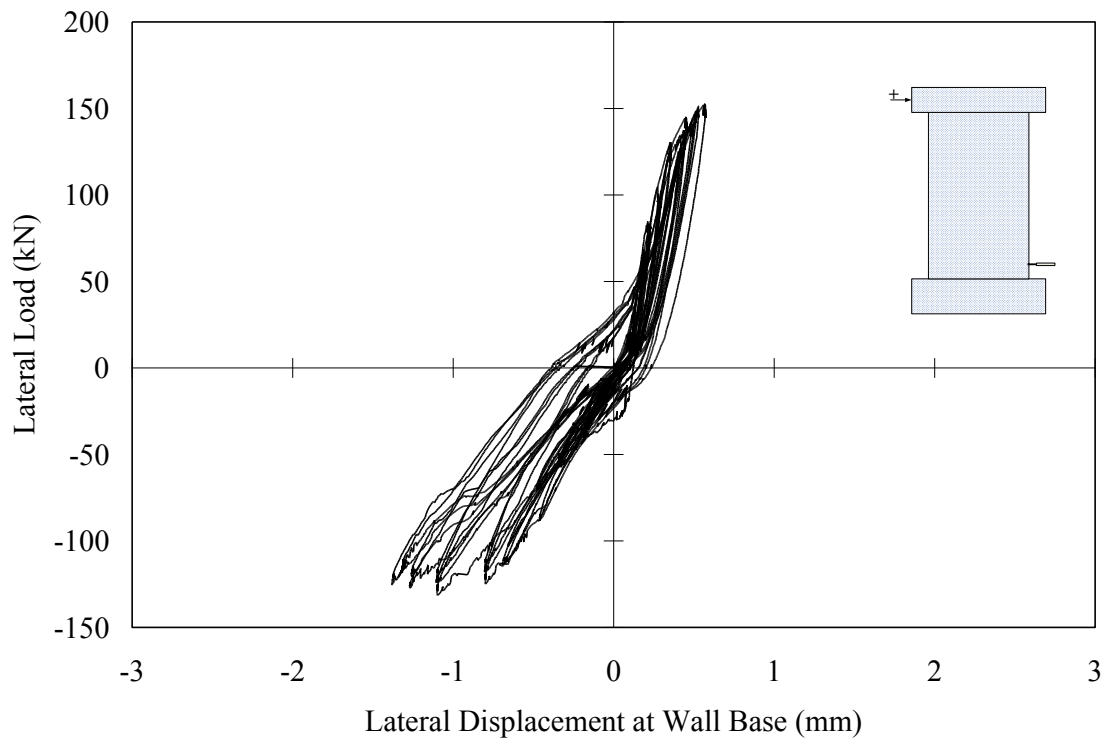


Figure D.12 LVDT5 Response in W1-SR

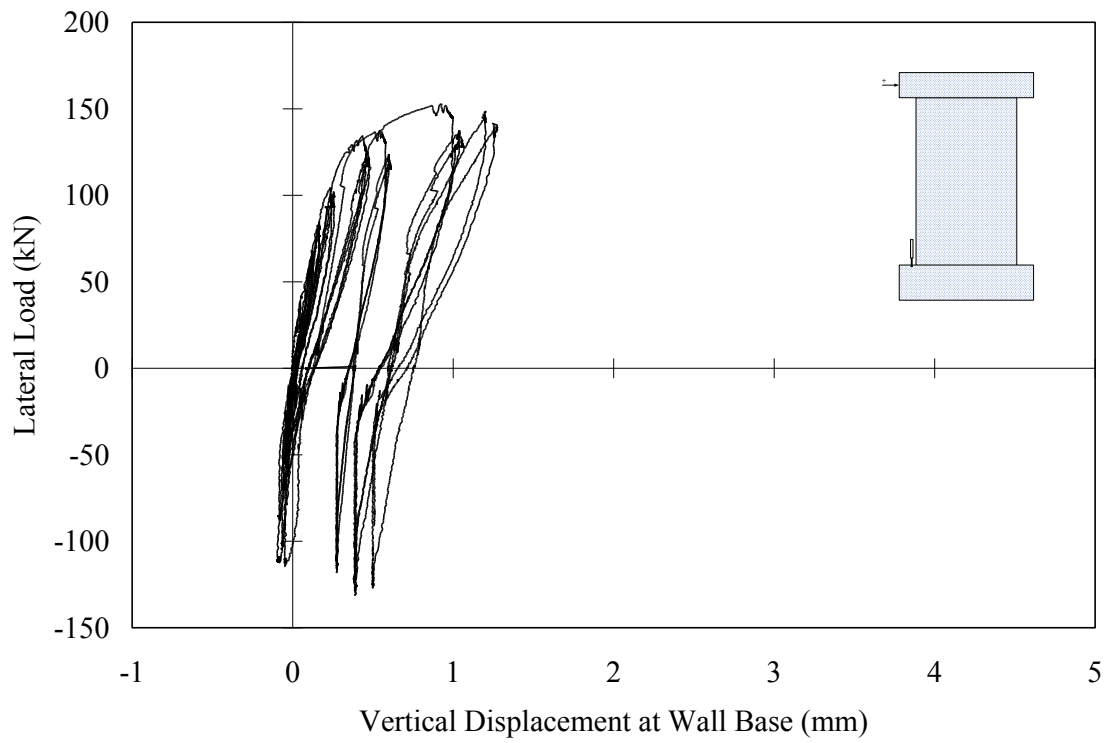


Figure D.13 LVDT8 Response in W1-SR

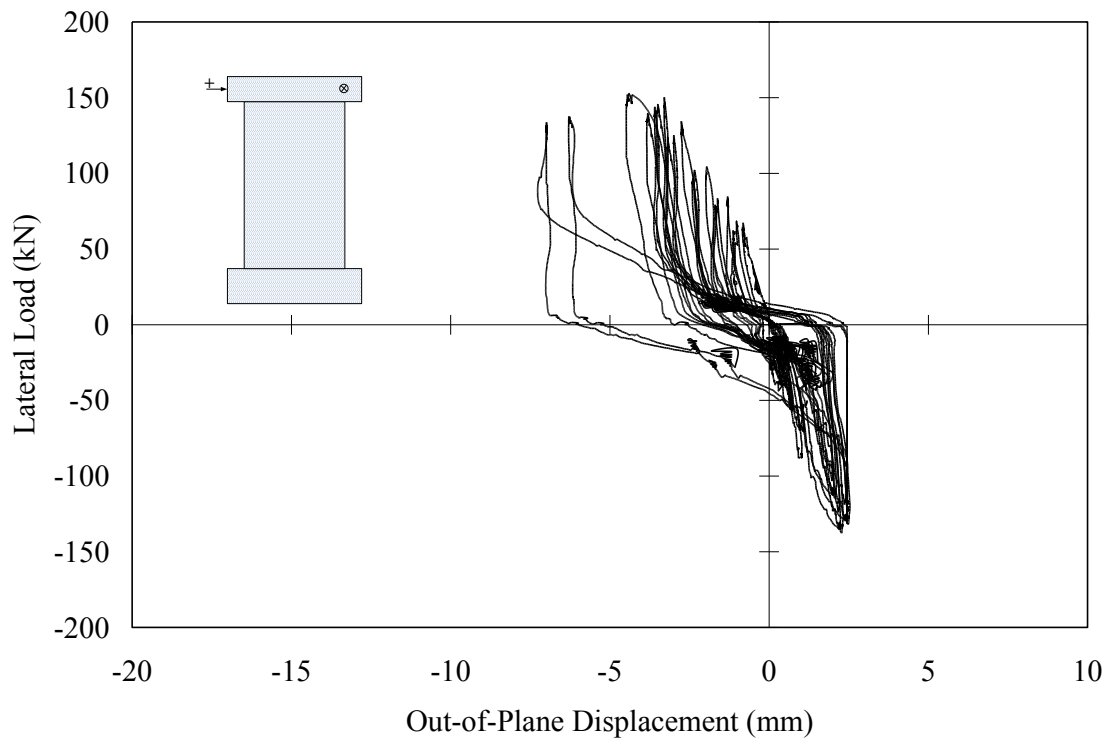


Figure D.14 LVDT9 Response in W1-SR

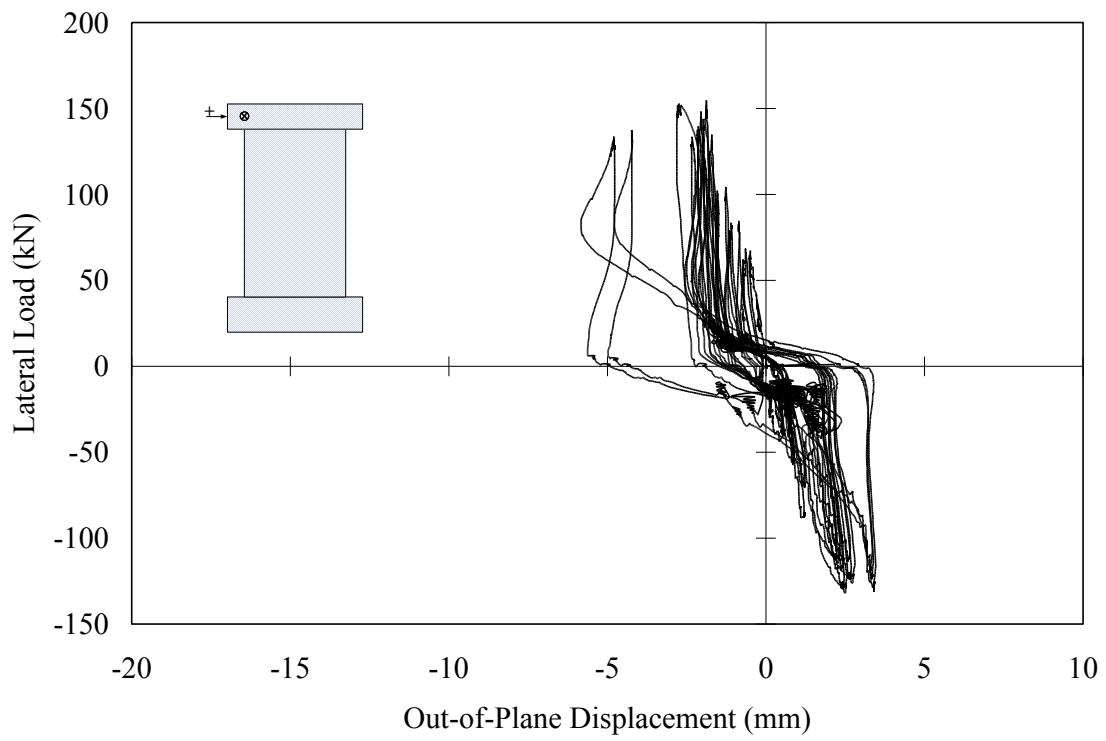


Figure D.15 LVDT10 Response in W1-SR



D.3 Displacement Cable Transducer (DCTs) Responses in Wall W2-NR

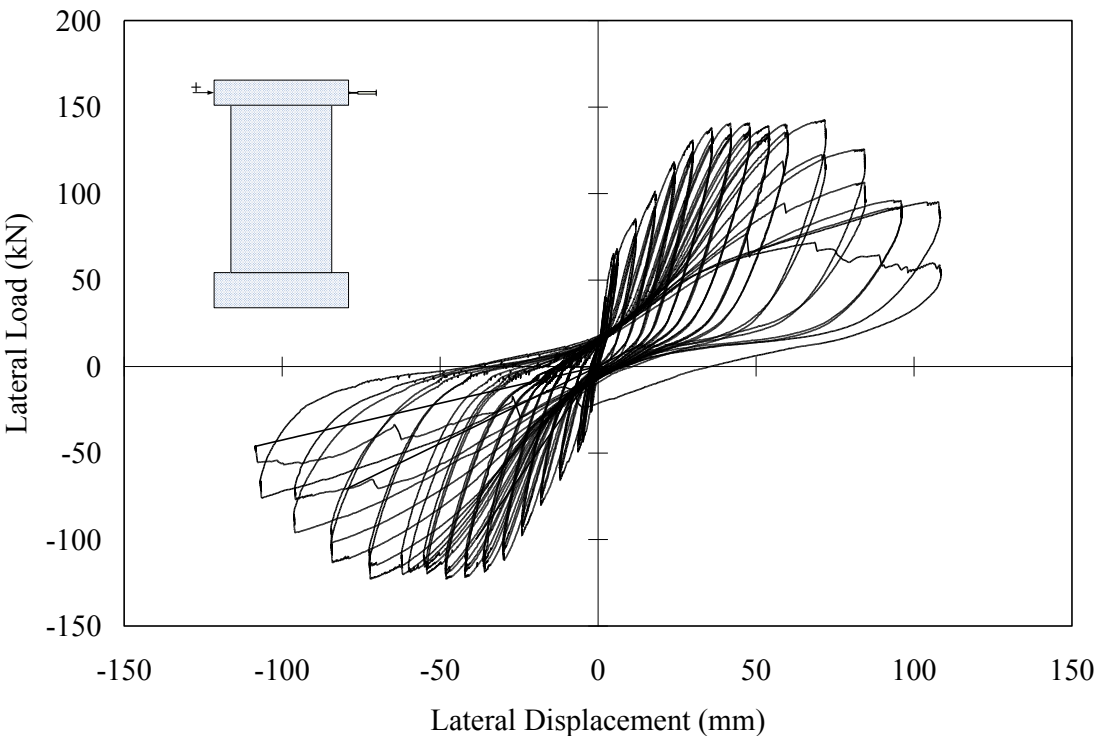


Figure D.16 DCT1 Response in W2-NR

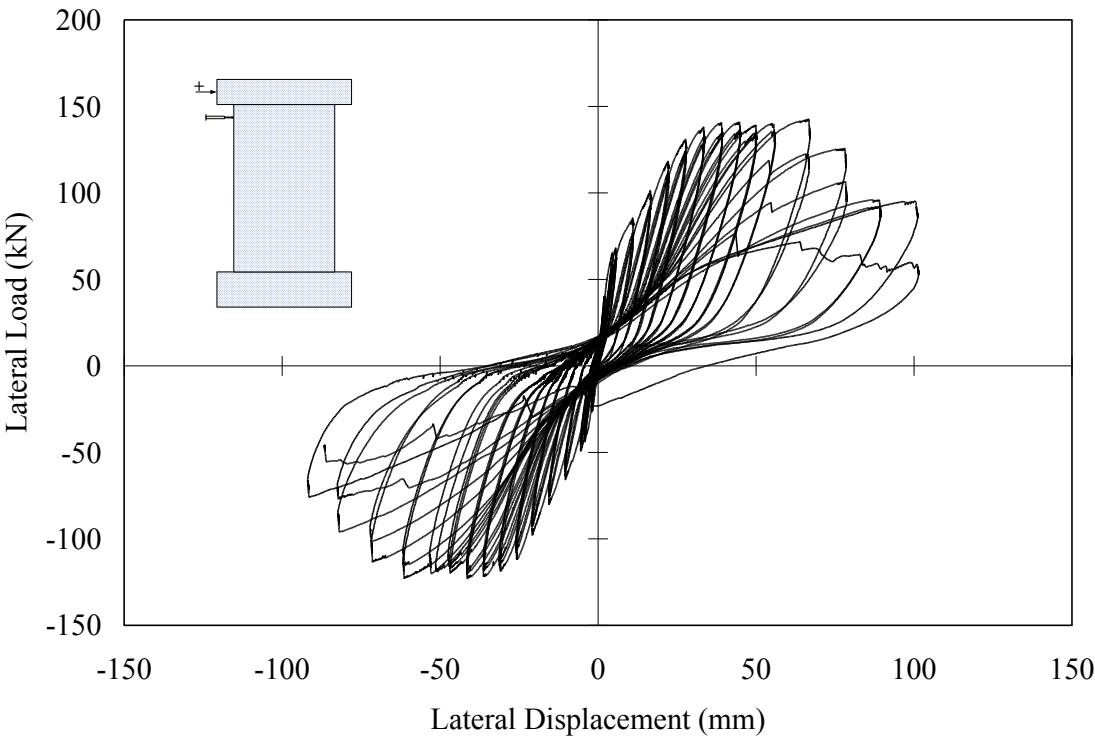


Figure D.17 DCT2 Response in W2-NR

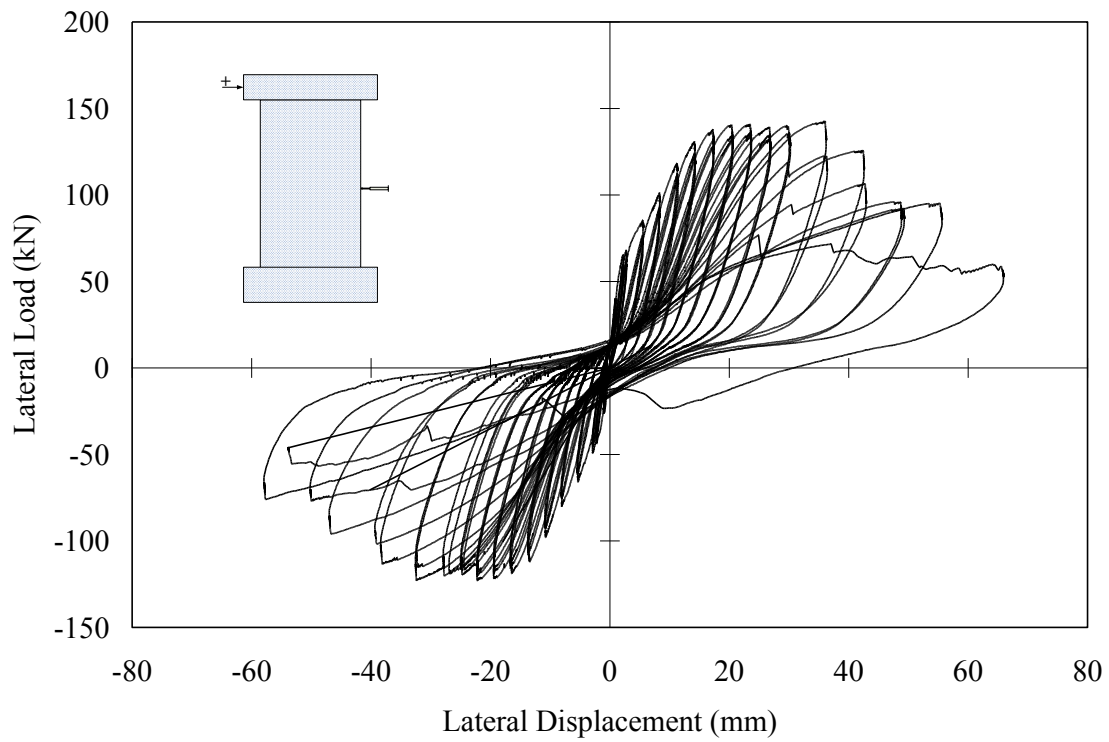


Figure D.18 DCT3 Response in W2-NR

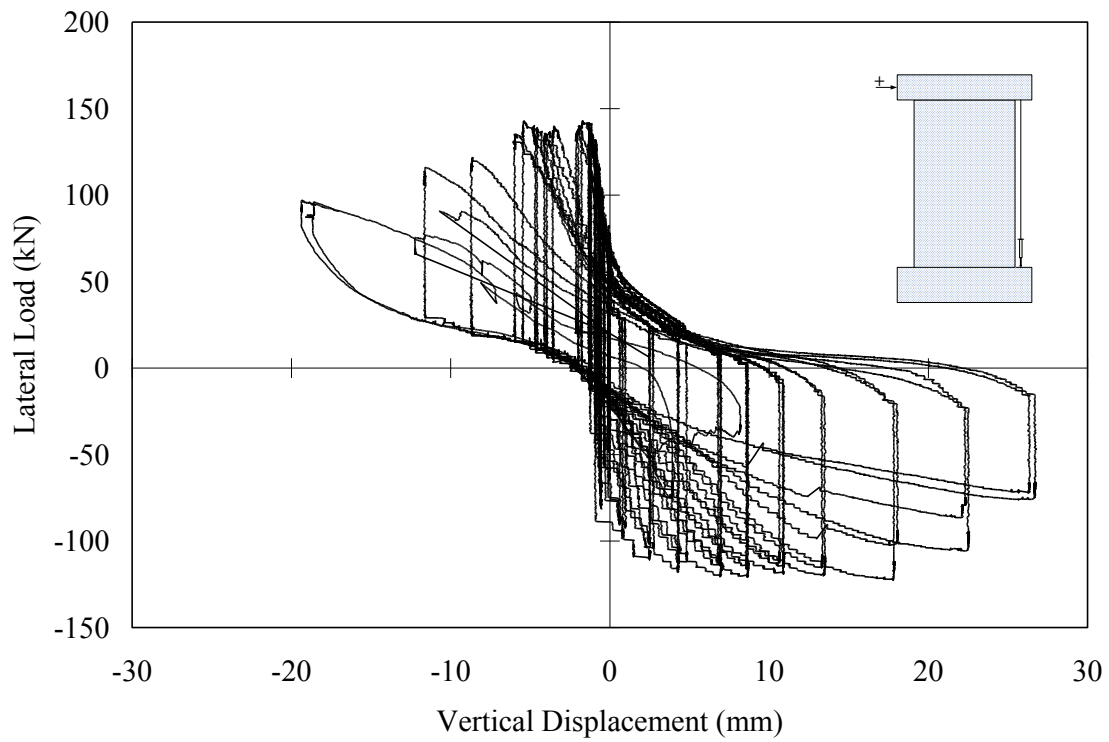


Figure D.19 DCT5 Response in W2-NR

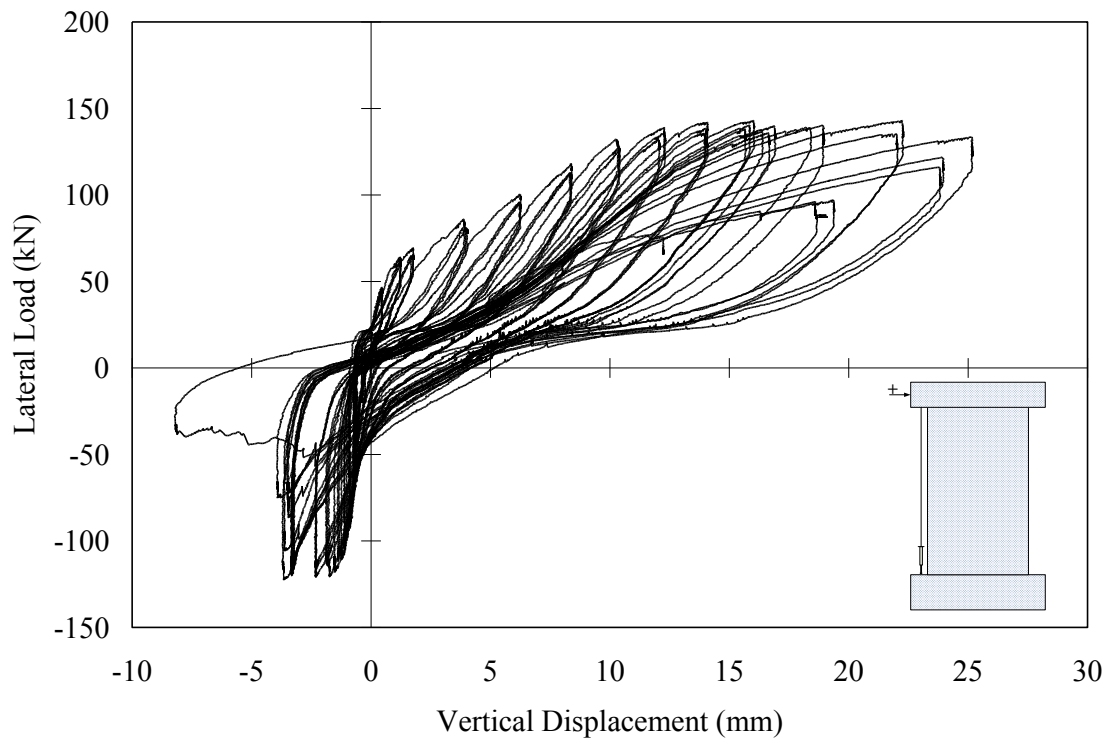


Figure D.20 DCT6 Response in W2-NR

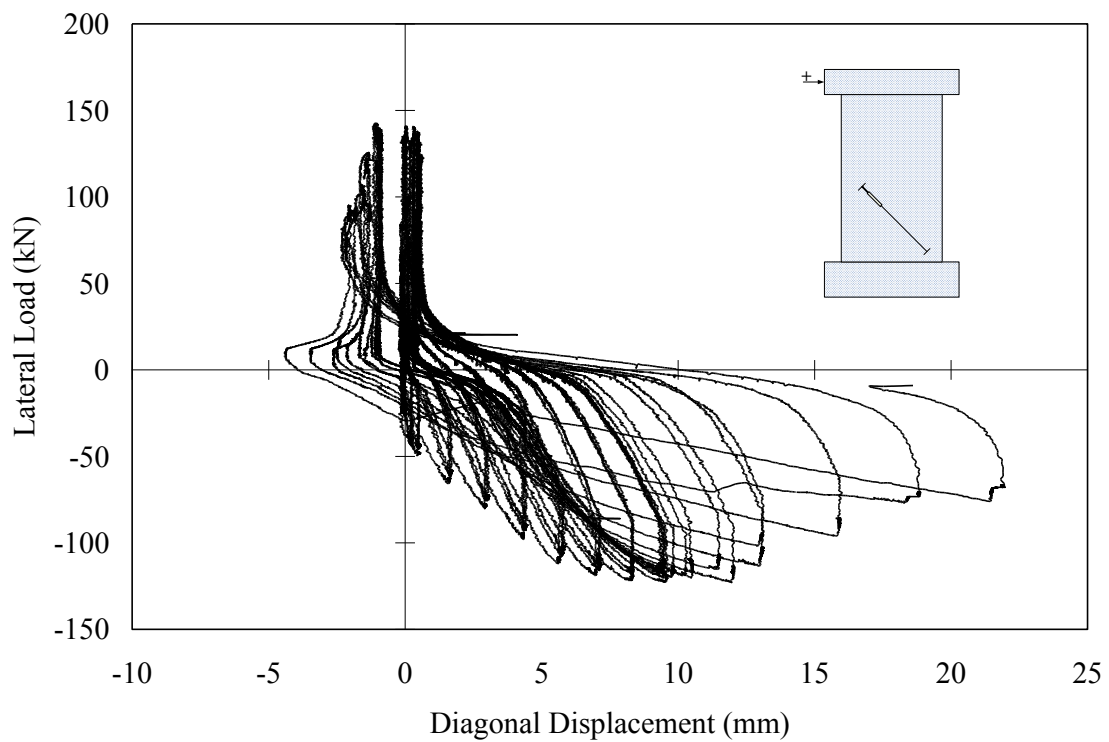


Figure D.21 DCT7 Response in W2-NR

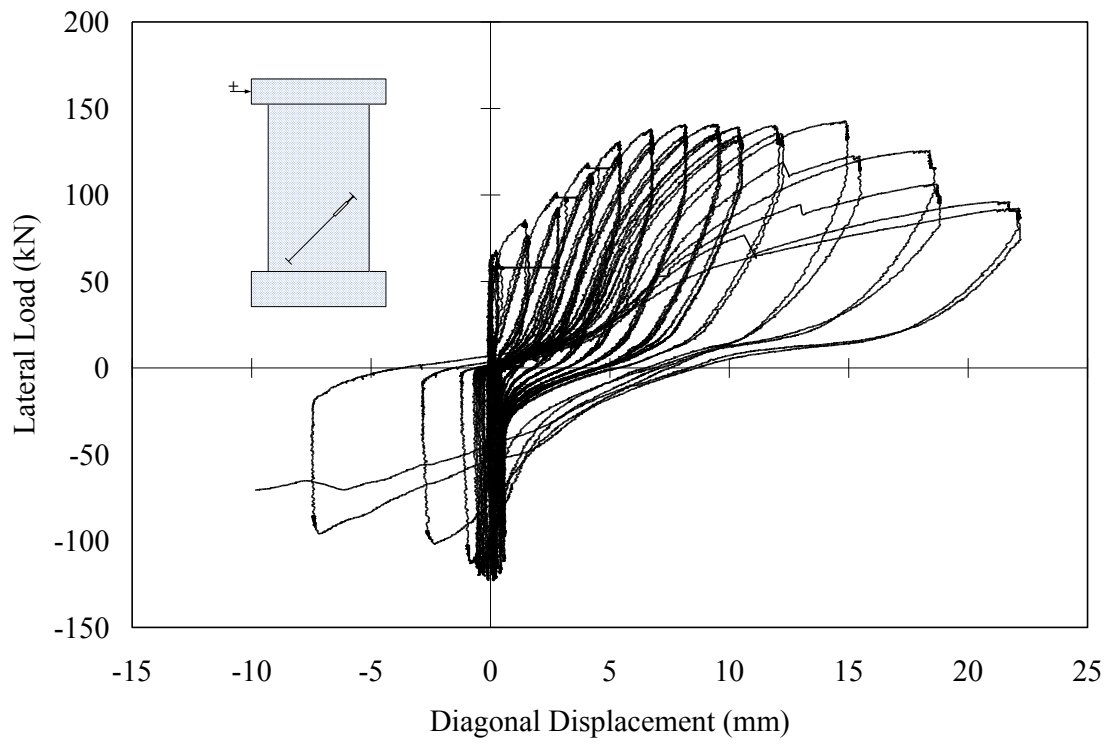


Figure D.22 DCT8 Response in W2-NR

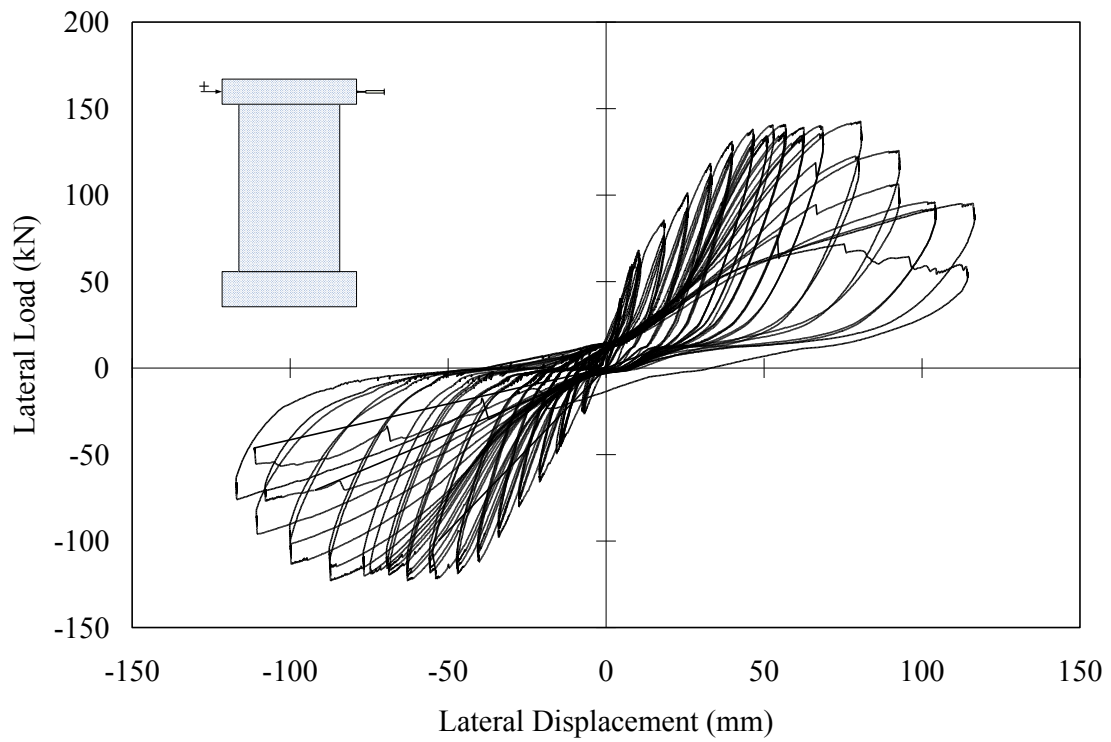


Figure D.23 DCT9 Response in W2-NR

#### D.4 Linear Variable Differential Transducer (LVDTs) Responses in Wall W2-NR

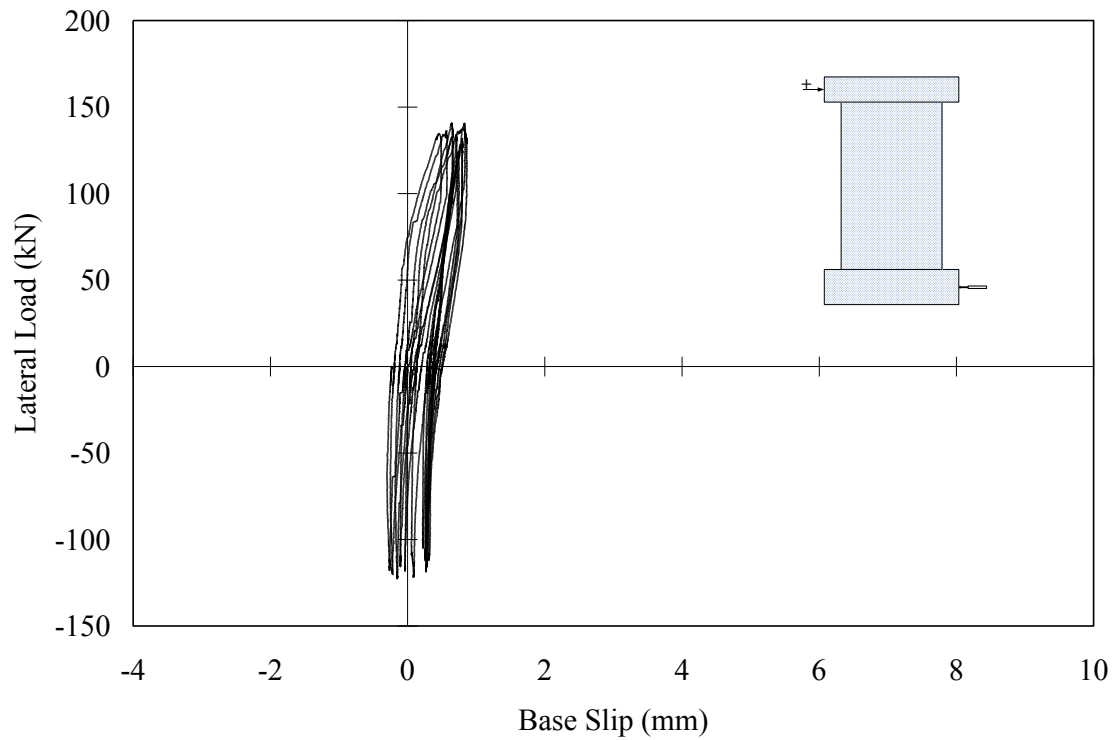


Figure D.24 LVDT1 Response in W2-NR

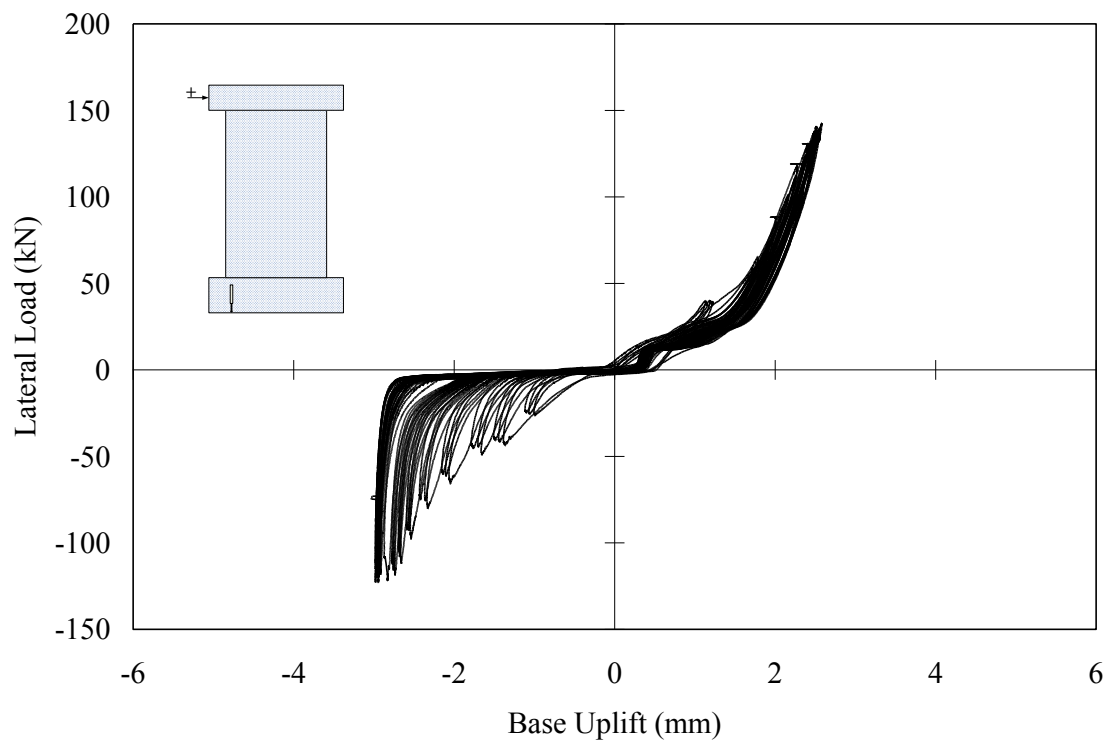


Figure D.25 LVDT2 Response in W2-NR

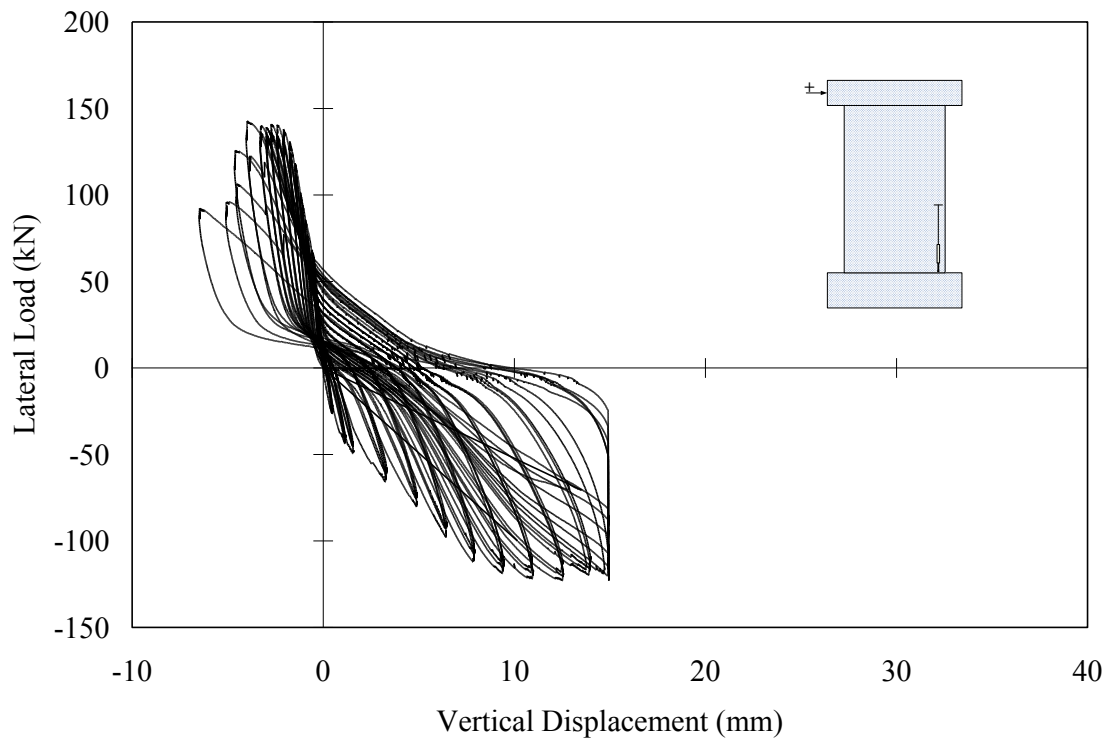


Figure D.26 LVDT3 Response in W2-NR

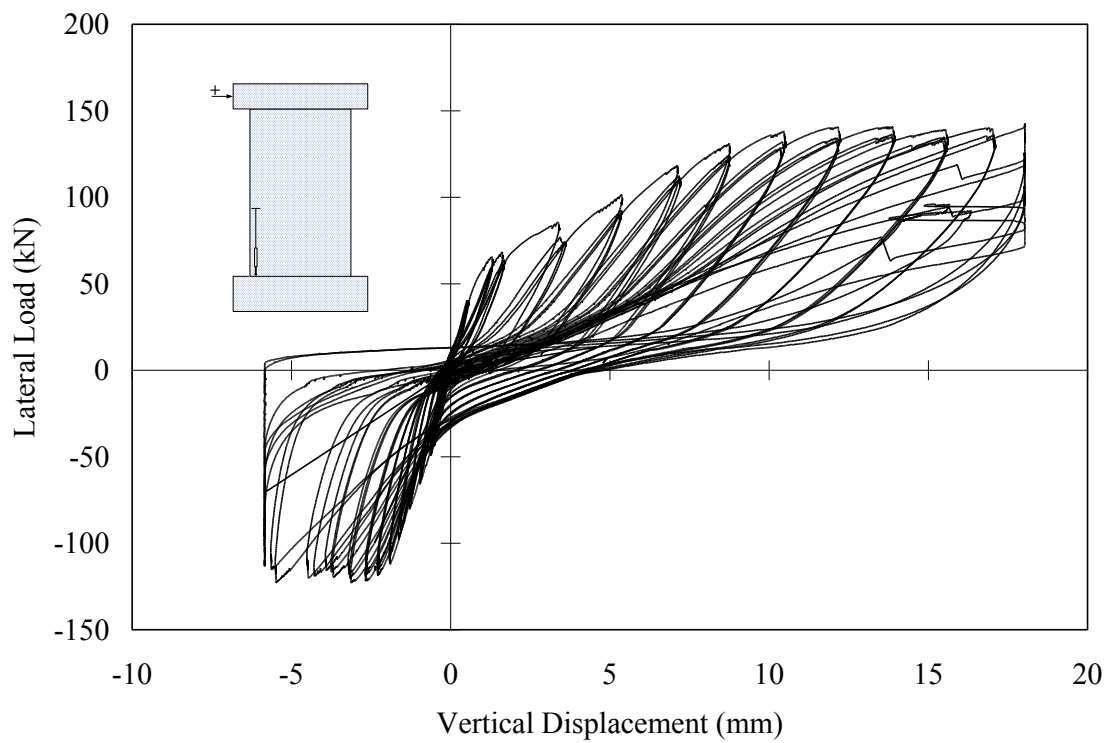


Figure D.27 LVDT4 Response in W2-NR

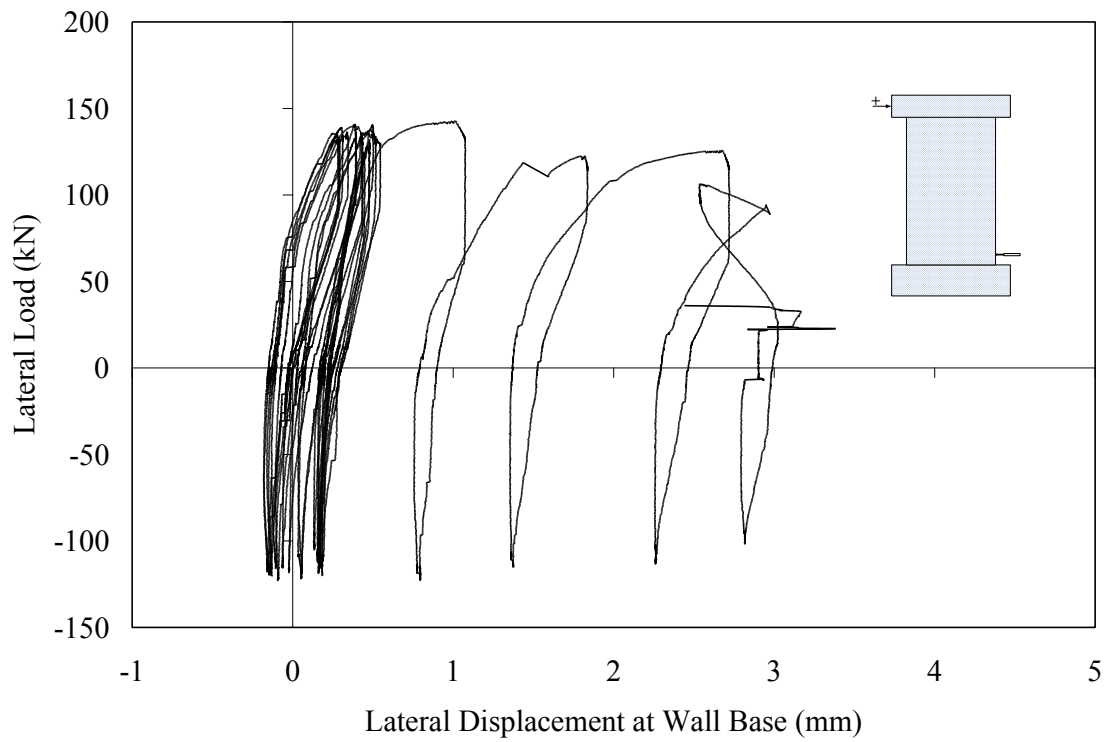


Figure D.28 LVDT5 Response in W2-NR

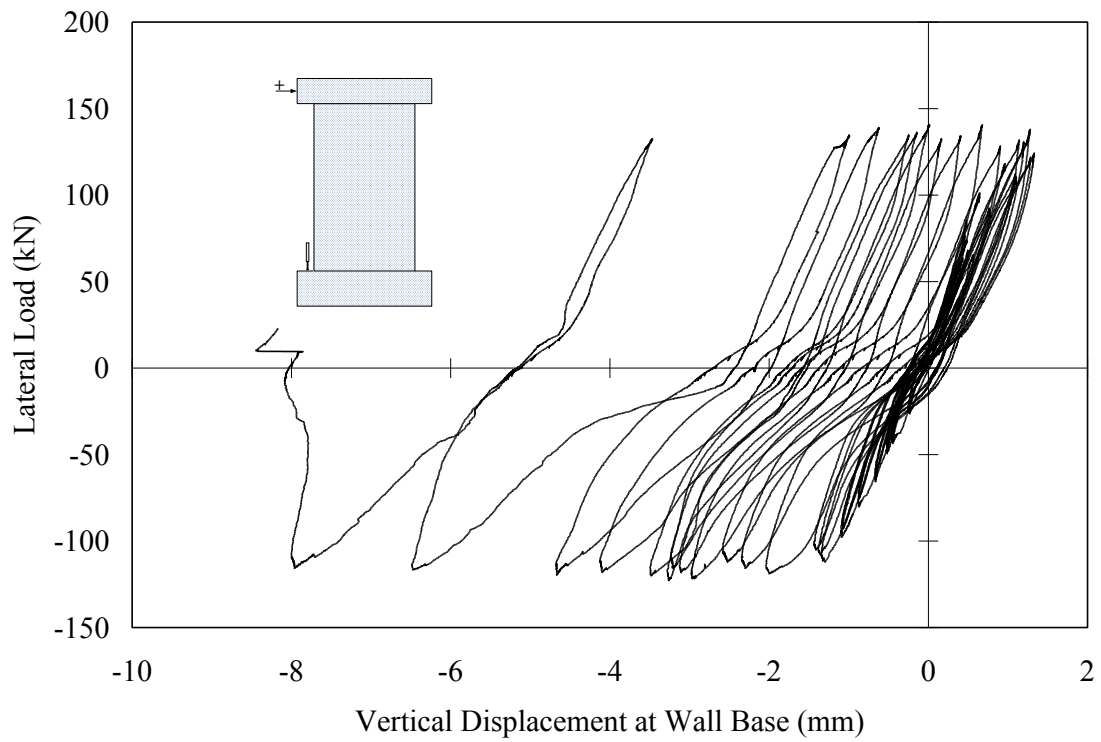


Figure D.29 LVDT8 Response in W2-NR

## APPENDIX E

### Strain Gauge Responses in Beams and Shear Walls

#### E.1 Strain Gauge Responses in Beams

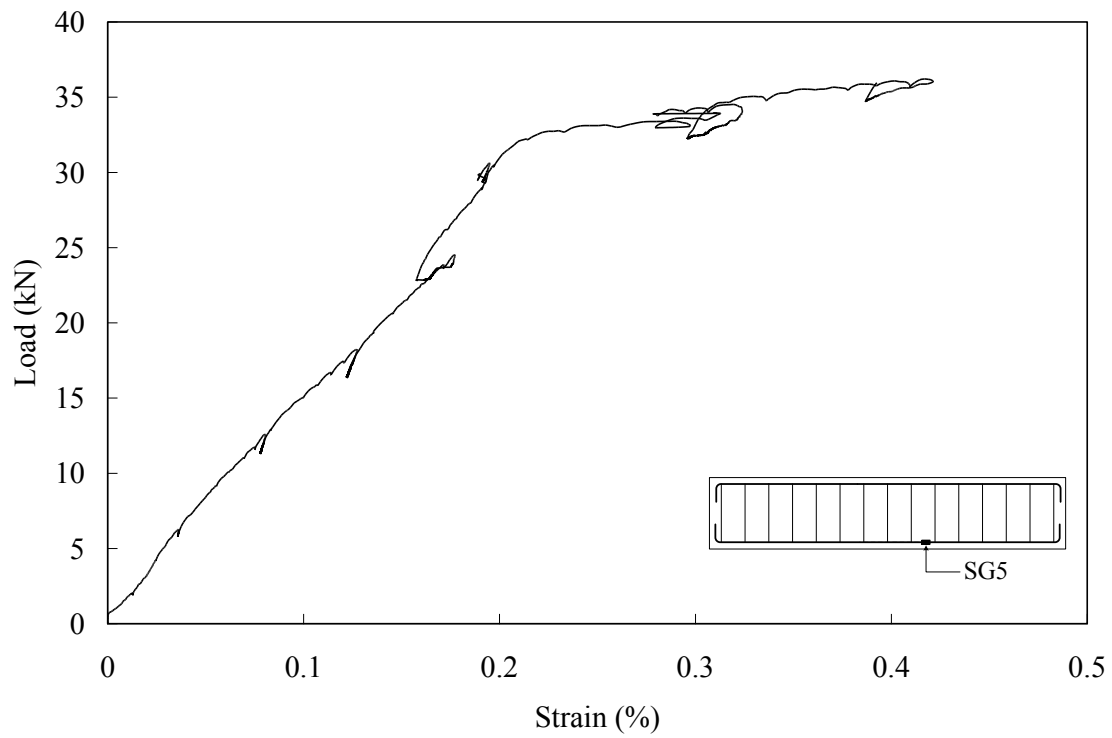


Figure E.1 SG5 Response in B1-SR



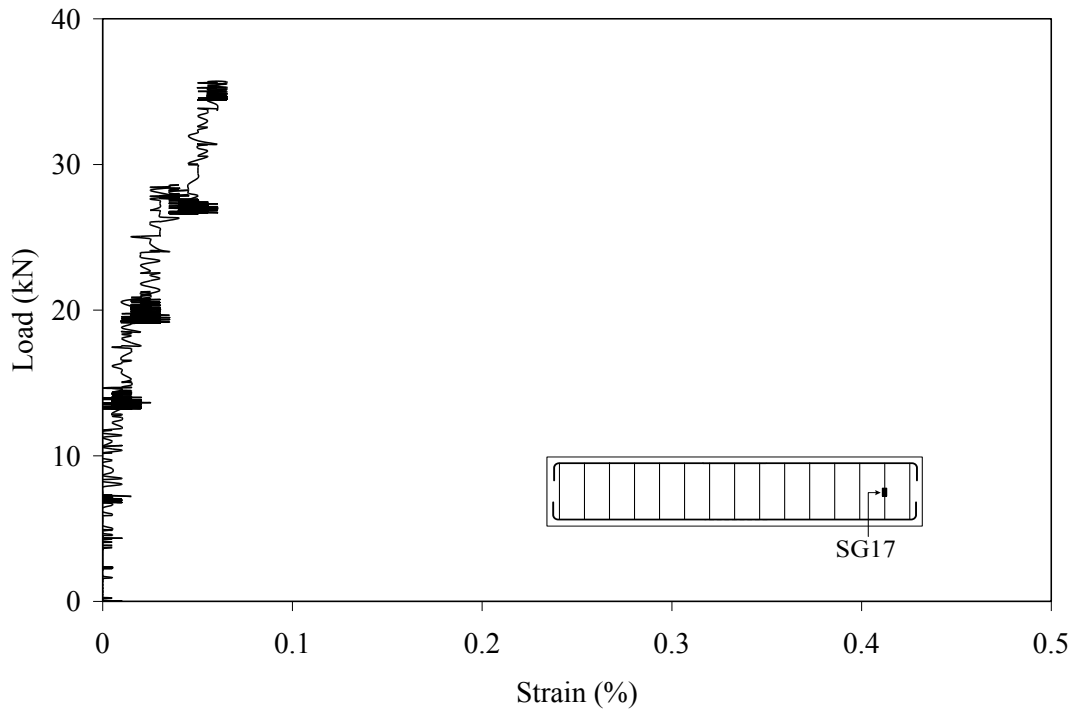


Figure E.2 SG17 Response in B1-SR

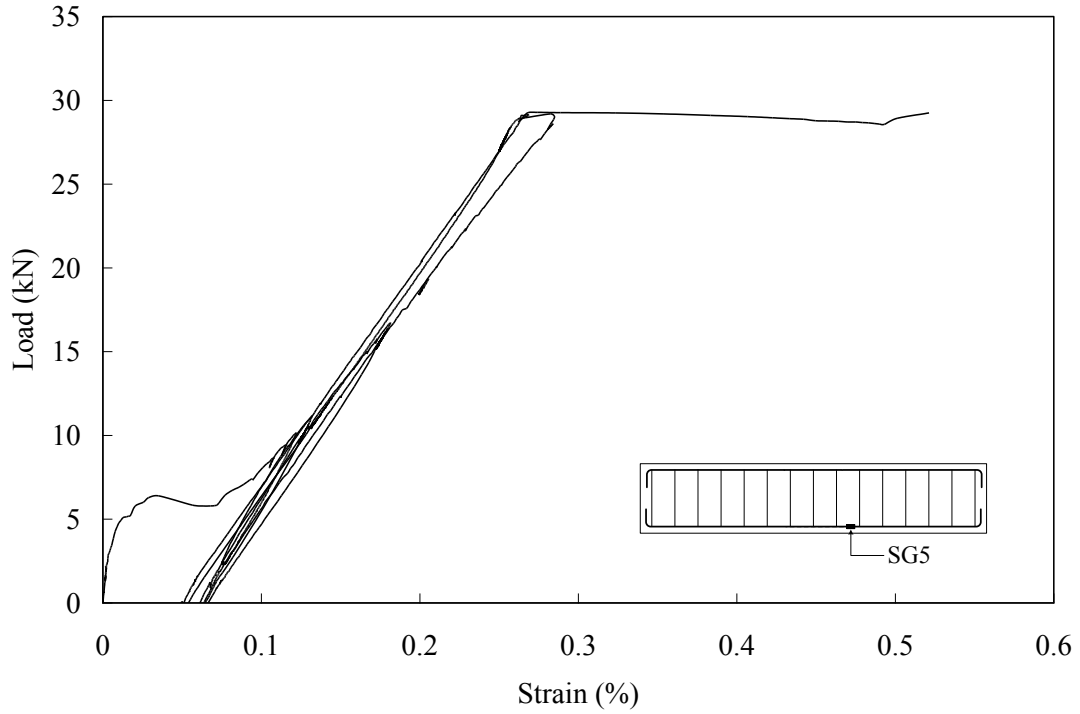


Figure E.3 SG5 Response in B2-SC

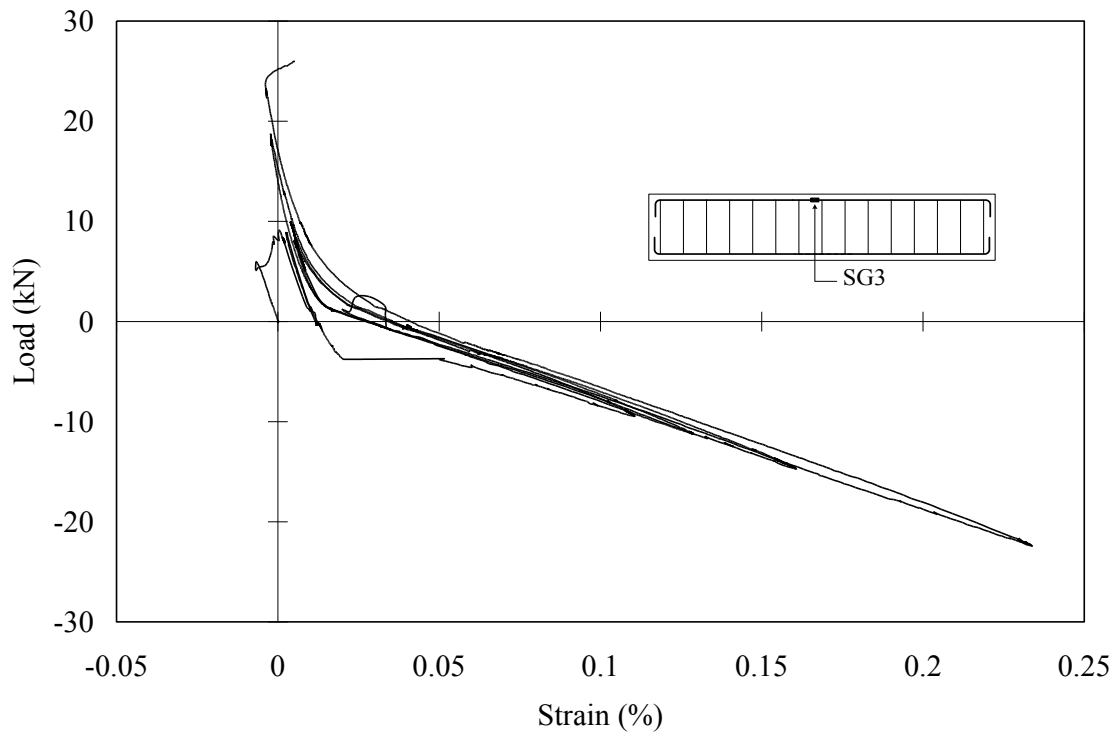


Figure E.4 SG3 Response in B3-SR

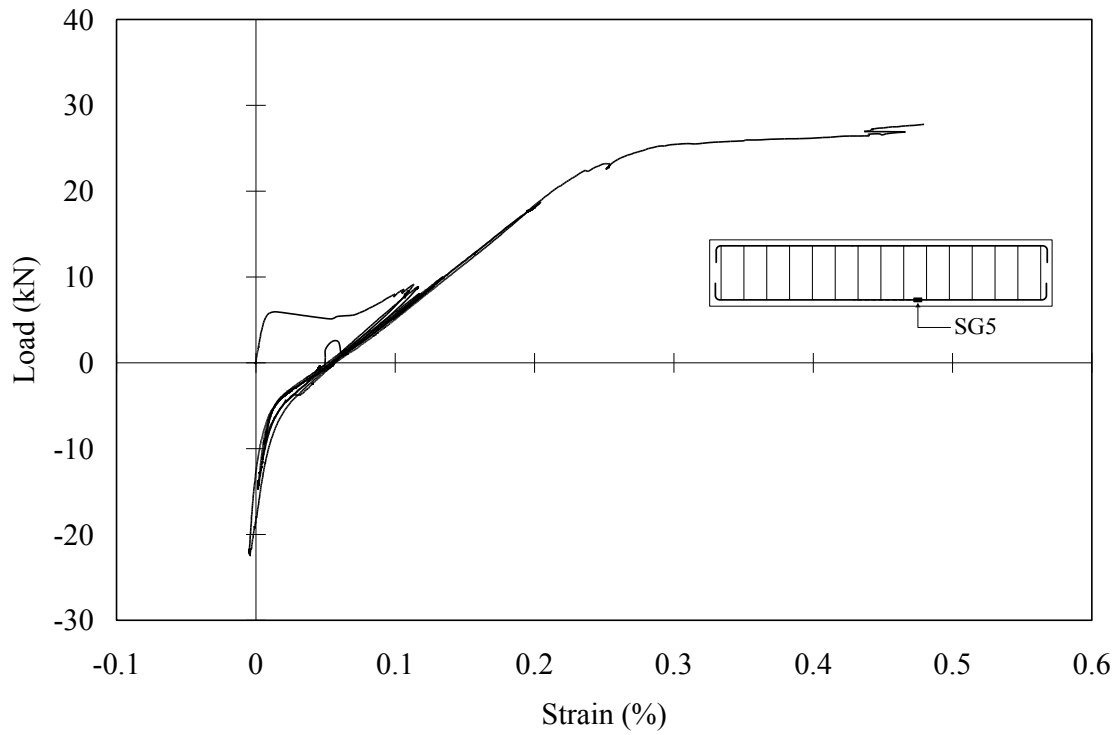


Figure E.5 SG5 Response in B3-SR

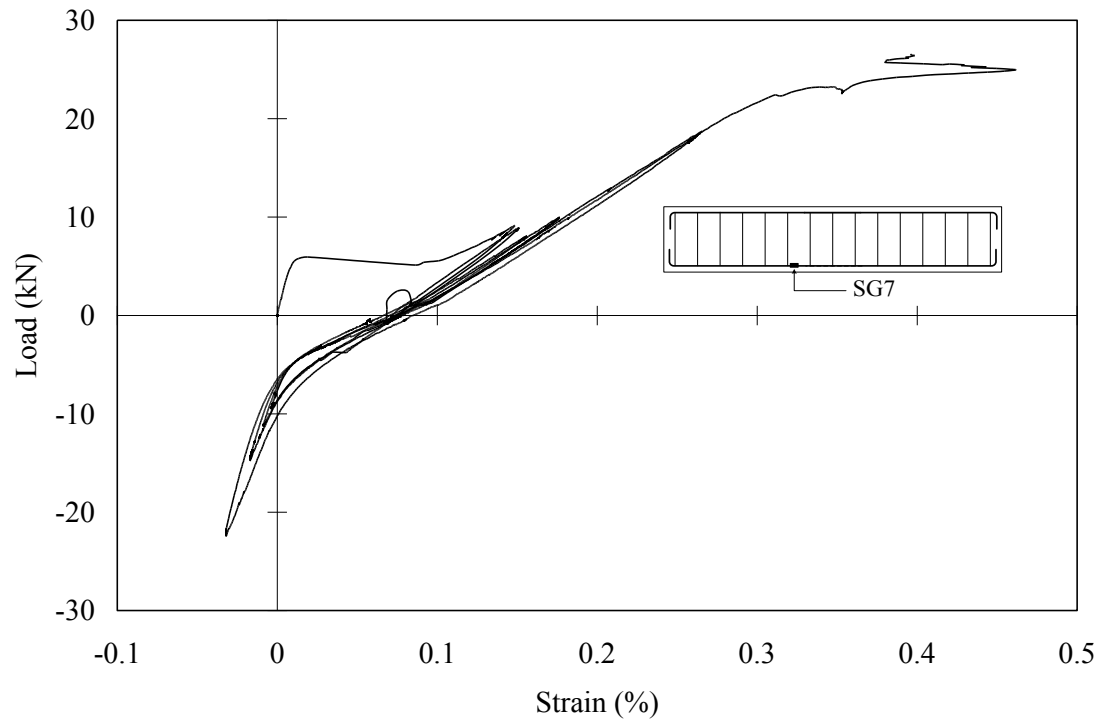


Figure E.6 SG7 Response in B3-SR

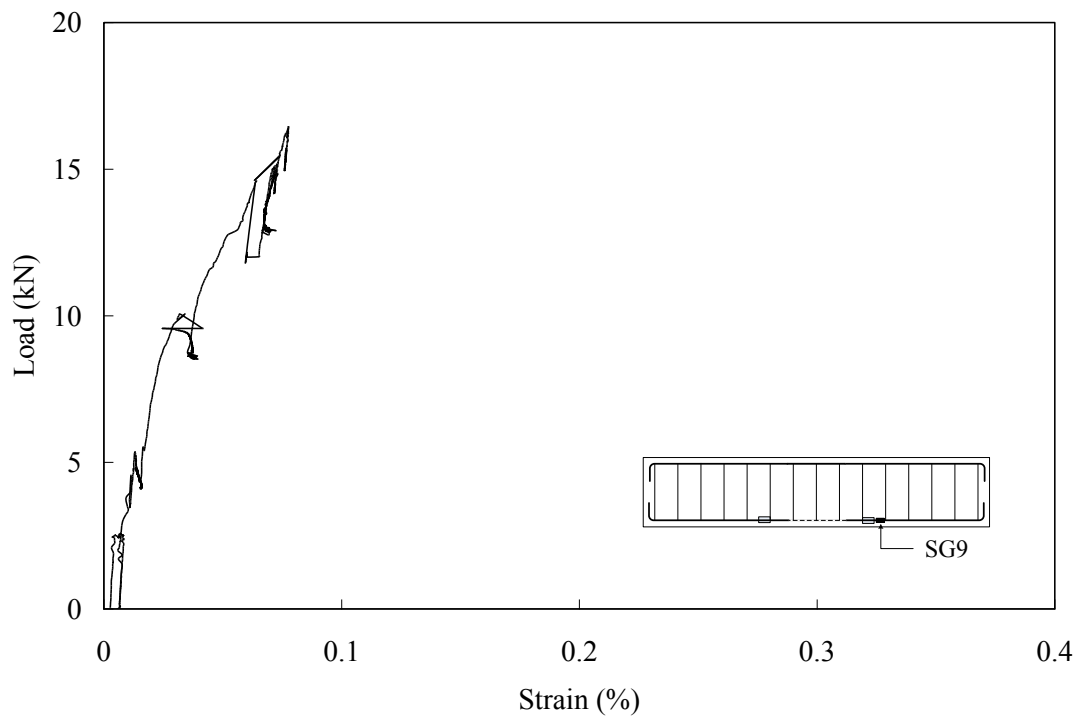


Figure E.7 SG9 Response in B4-NR

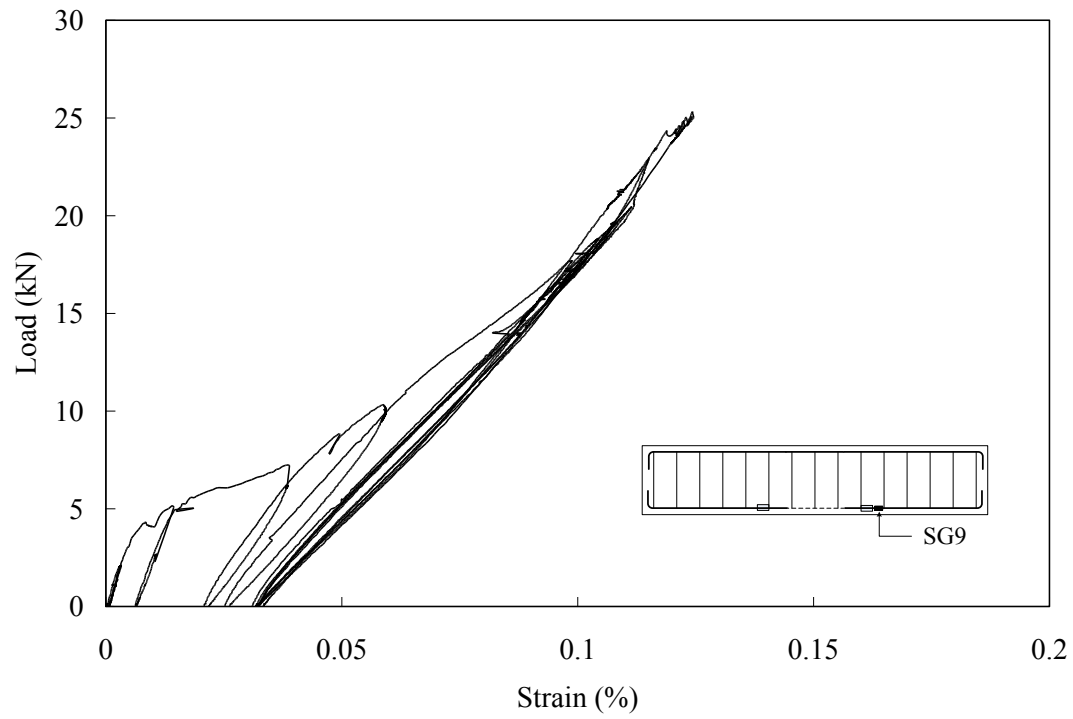


Figure E.8 SG9 Response in B5-NC

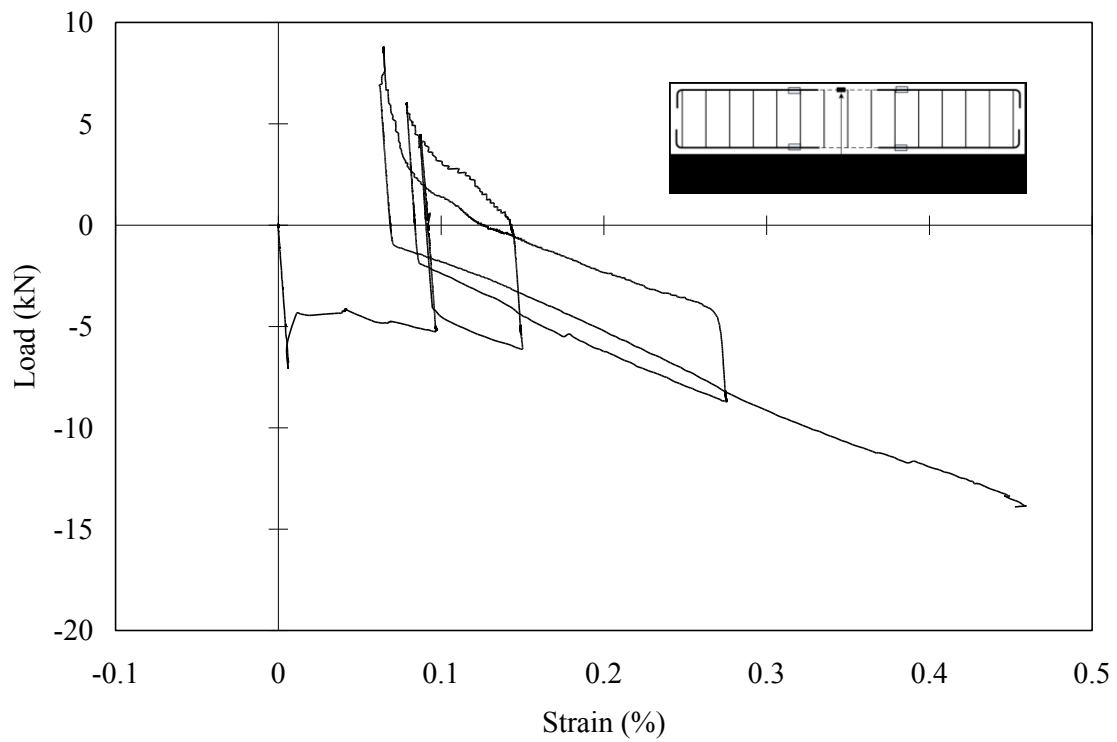


Figure E.9 SG3 Response in B6-NR

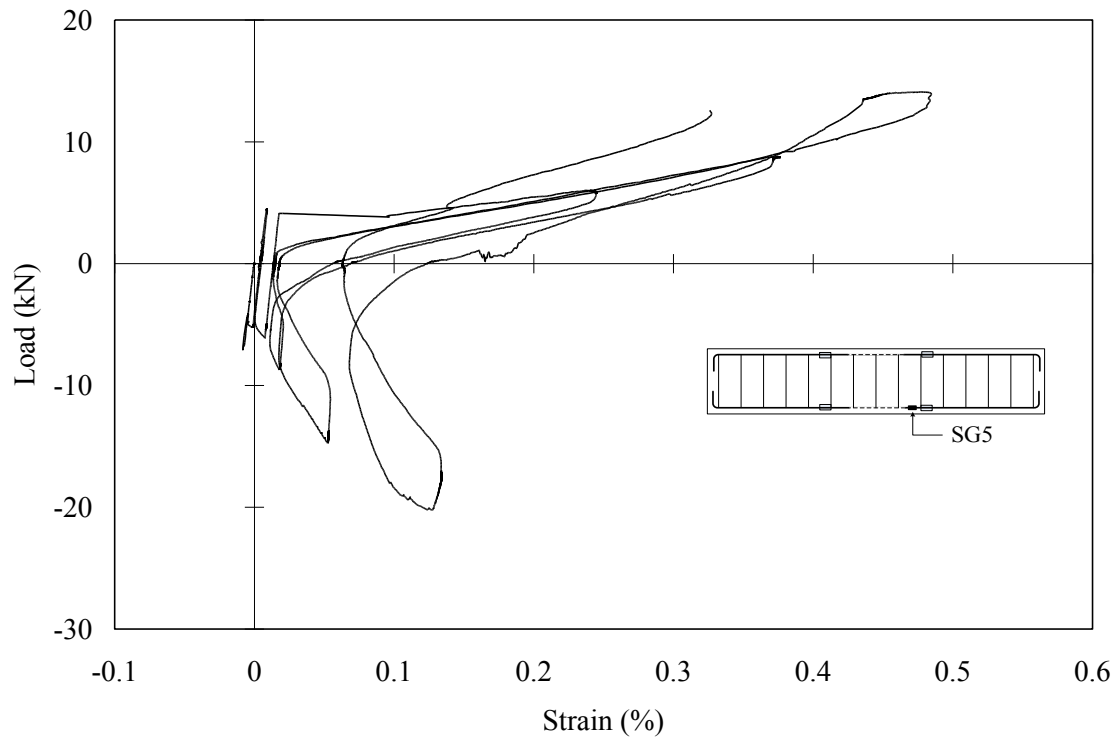


Figure E.10 SG5 Response in B6-NR

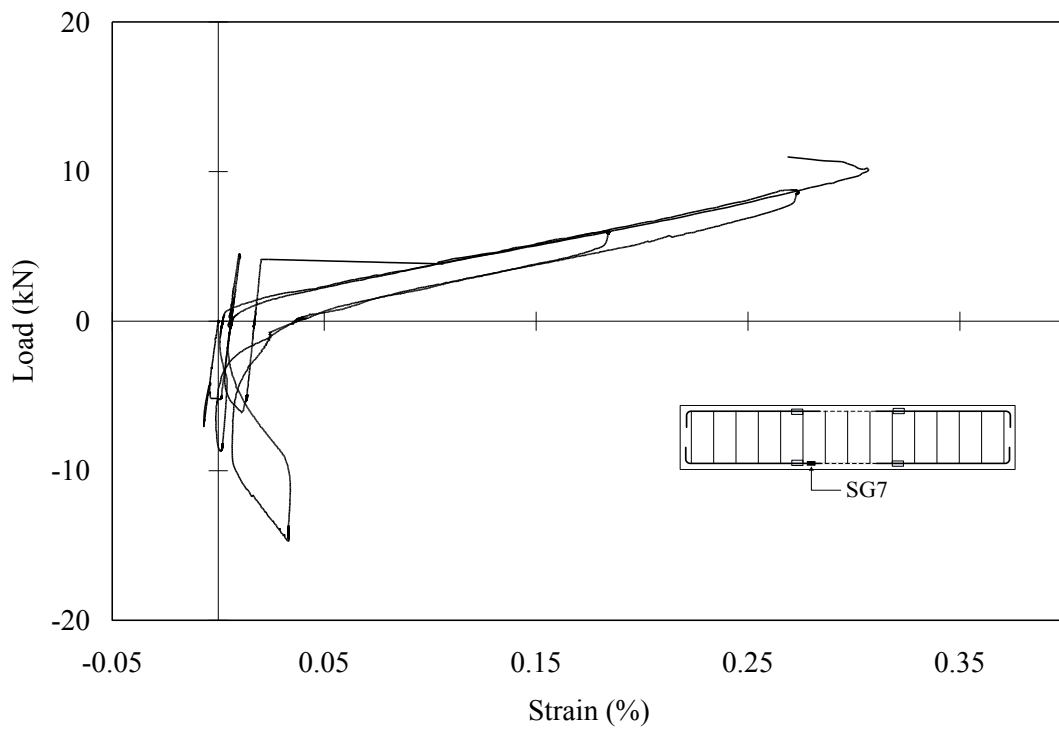


Figure E.11 SG7 Response in B6-NR

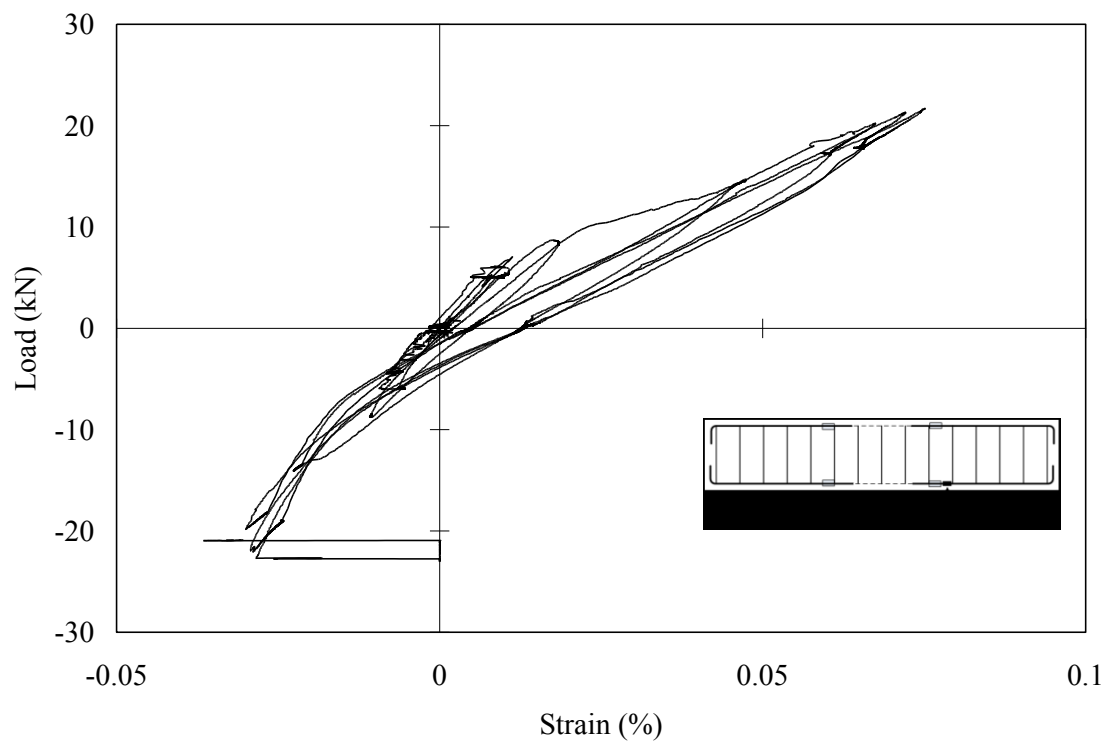


Figure E.12 SG9 Response in B6-NR

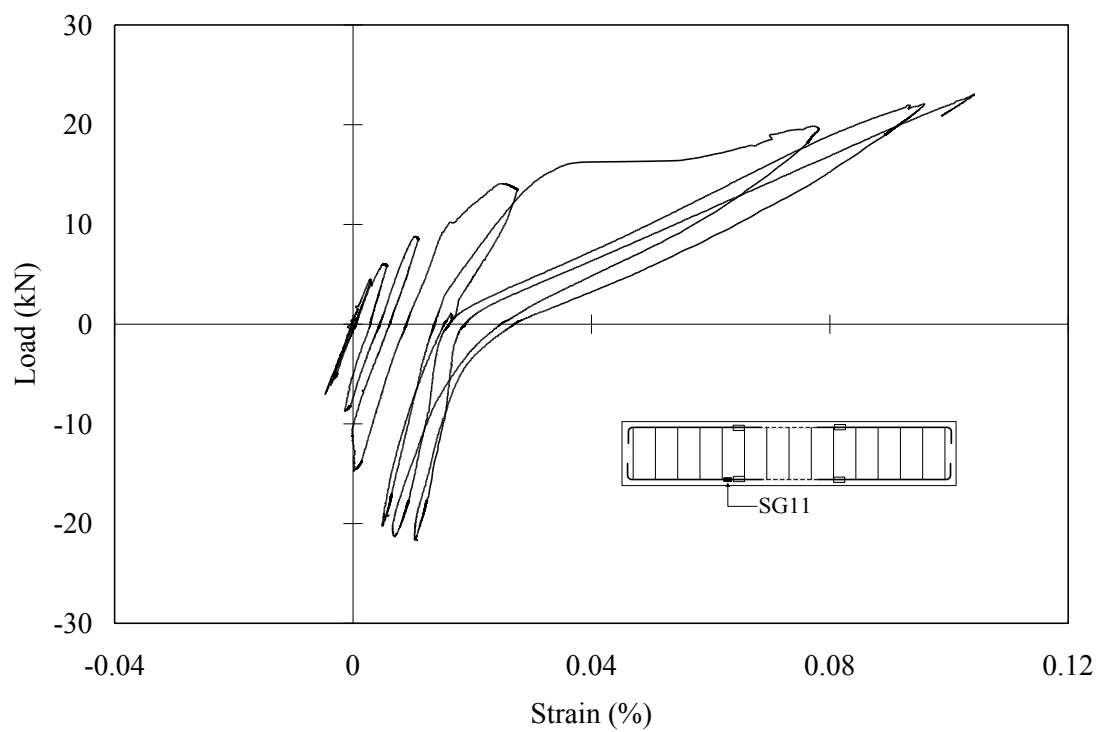


Figure E.13 SG11 Response in B6-NR

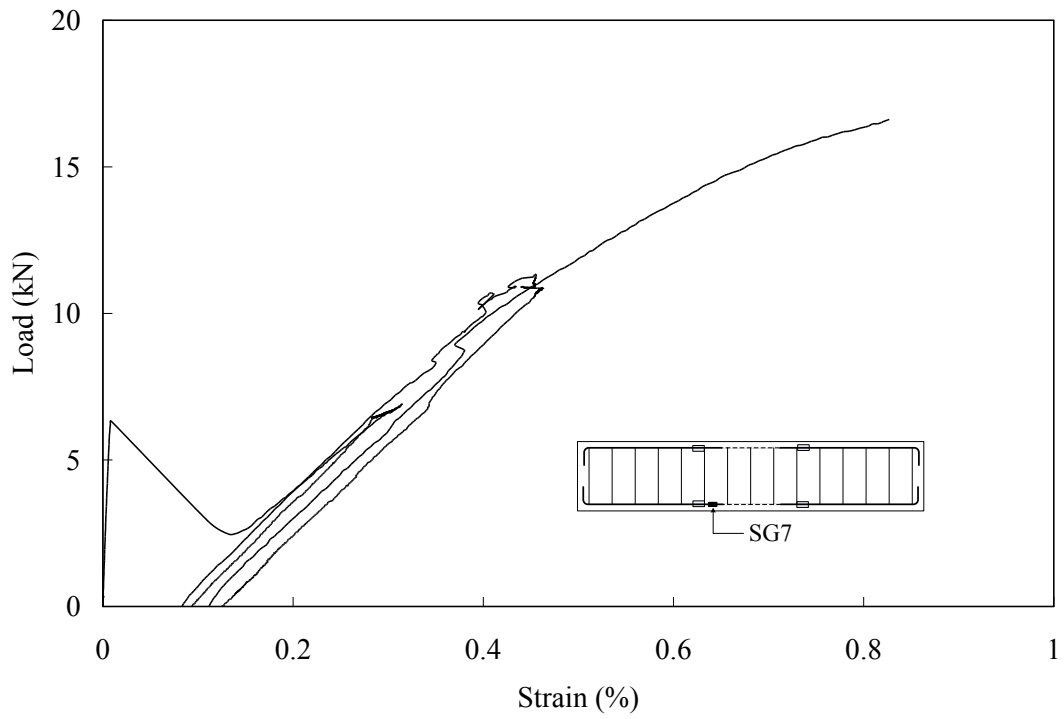


Figure E.14 SG7 Response in B7-NCM

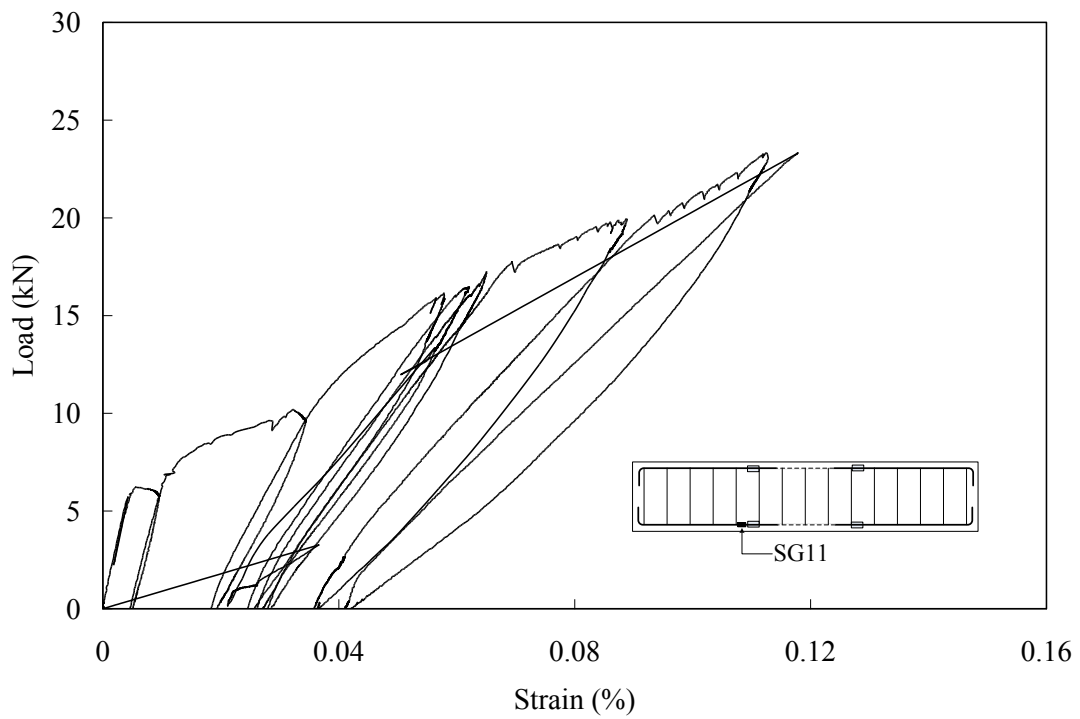


Figure E.15 SG11 Response in B7-NCM

## E.2 Strain Gauge Responses in Shear Walls

### E.2.1 Strain Gauge Responses in Wall W1-SR

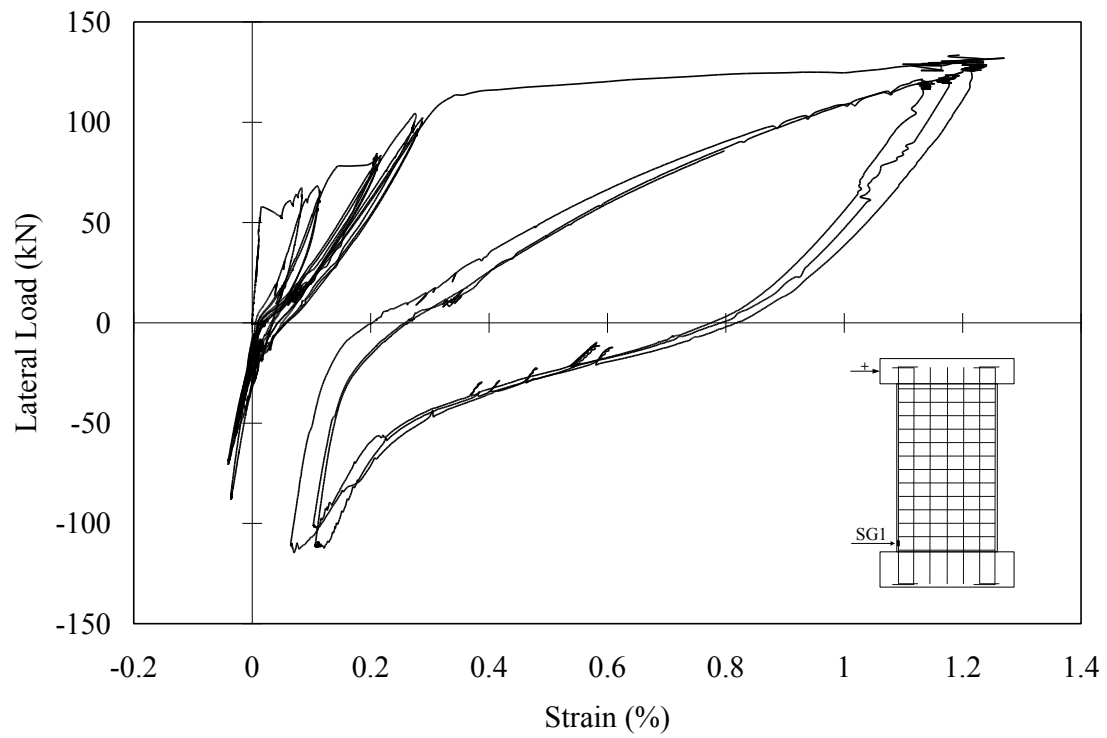


Figure E.16 SG1 Response in W1-SR



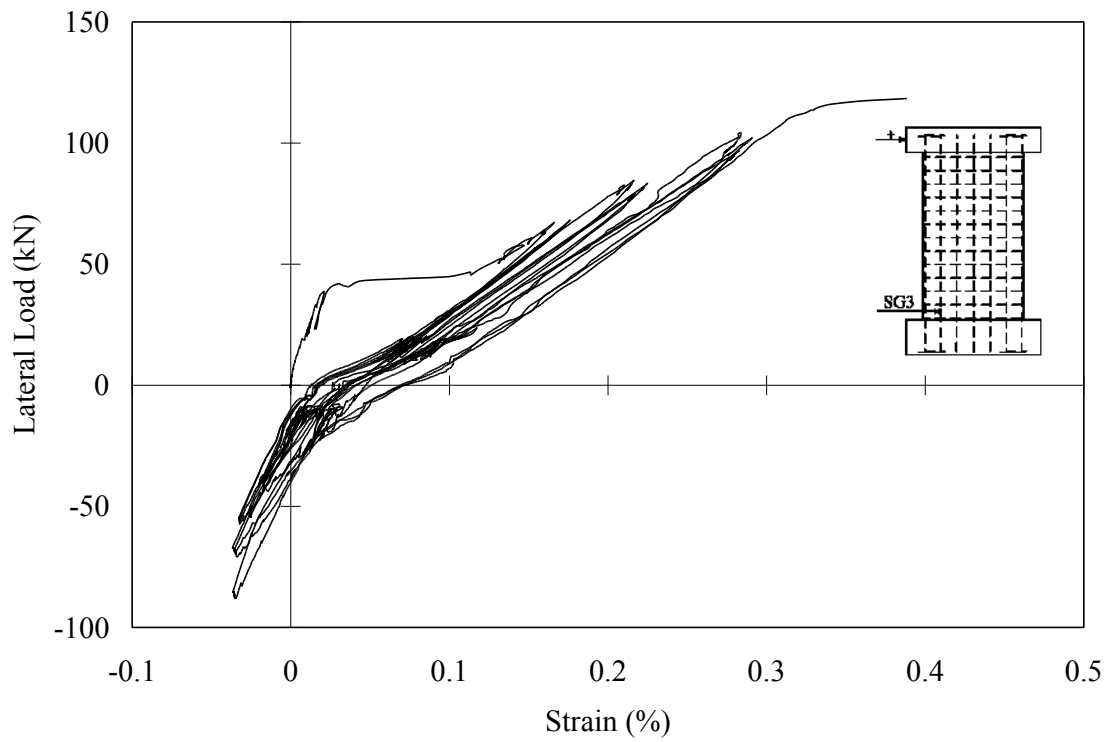


Figure E.17 SG3 Response in W1-SR

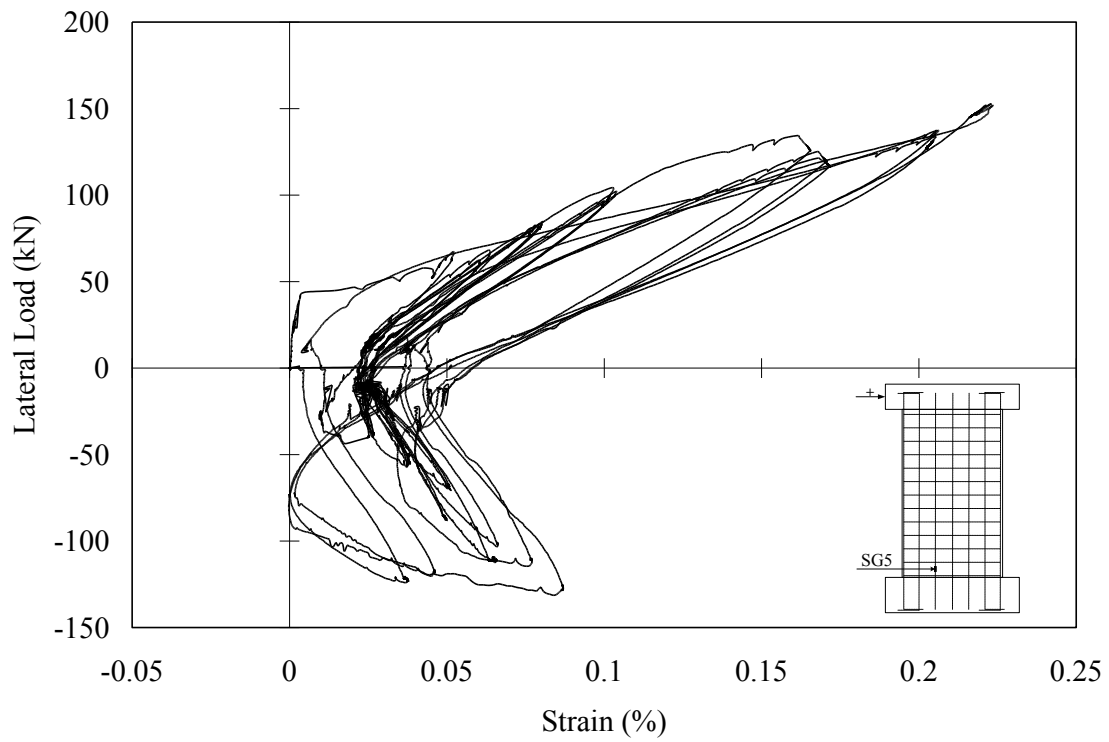


Figure E.18 SG5 Response in W1-SR

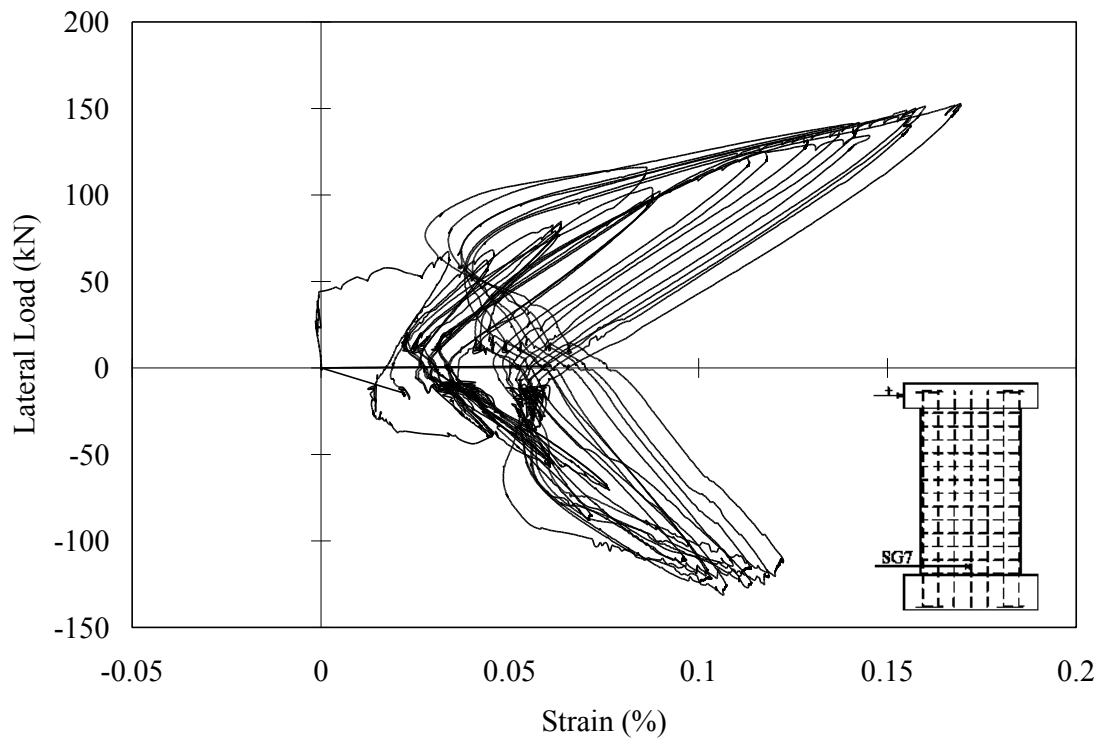


Figure E.19 SG7 Response in W1-SR

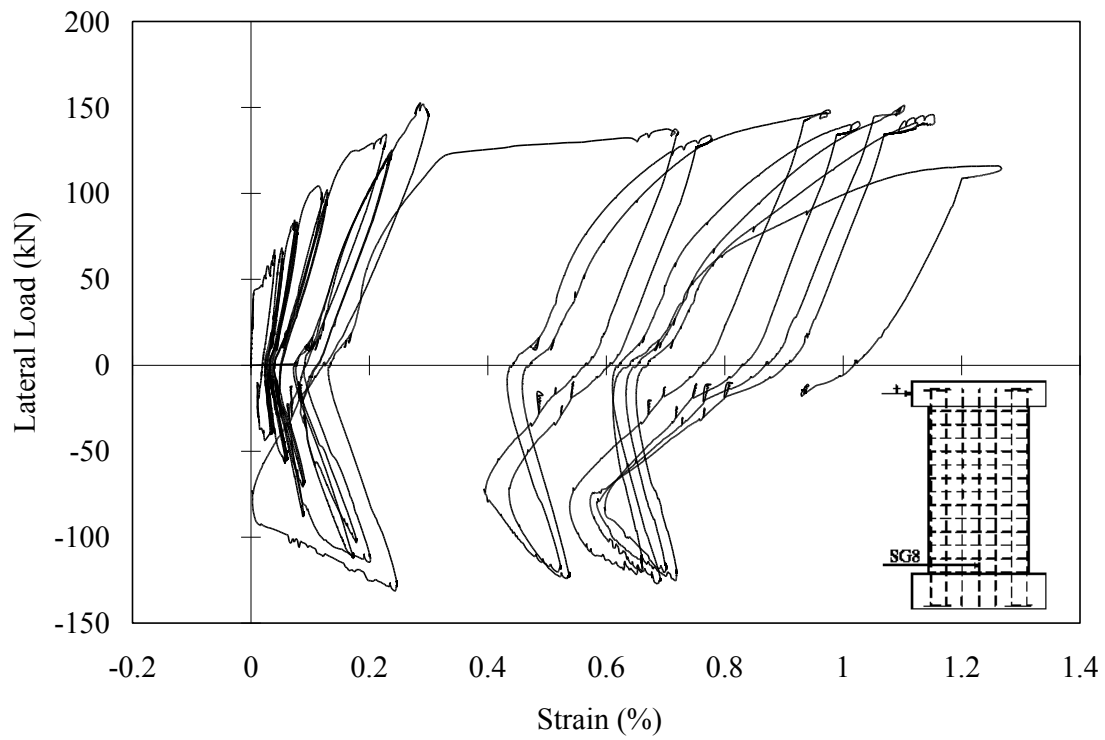


Figure E.20 SG8 Response in W1-SR

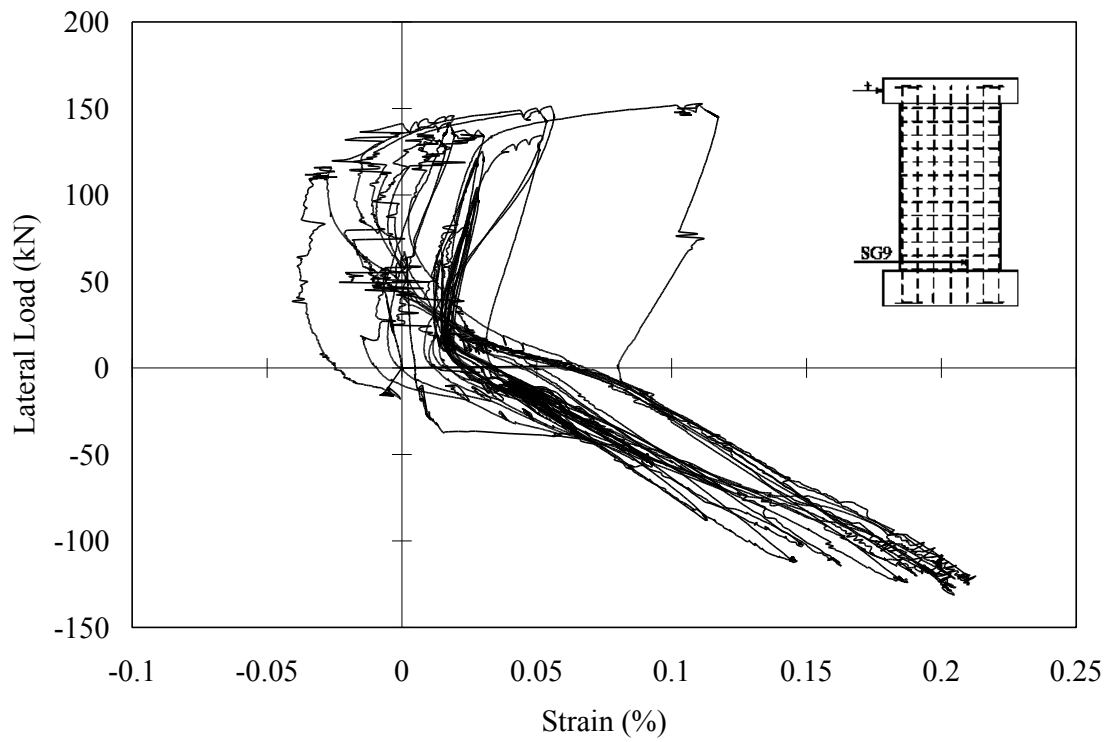


Figure E.21 SG9 Response in W1-SR

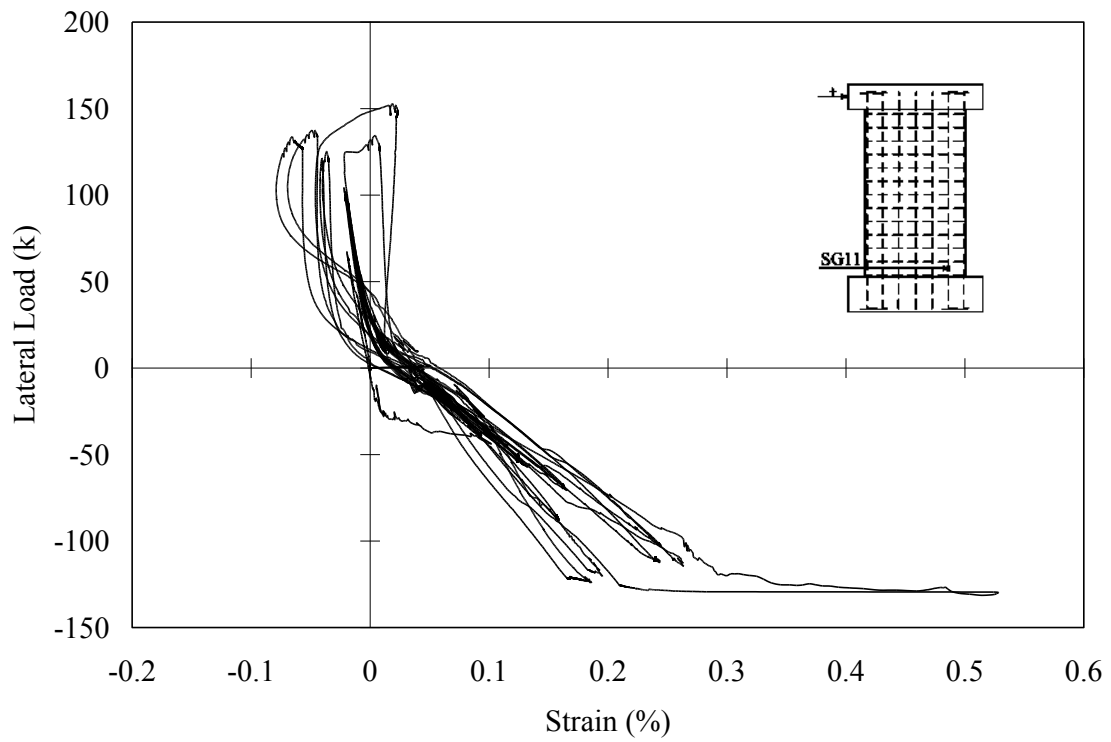


Figure E.22 SG11 Response in W1-SR

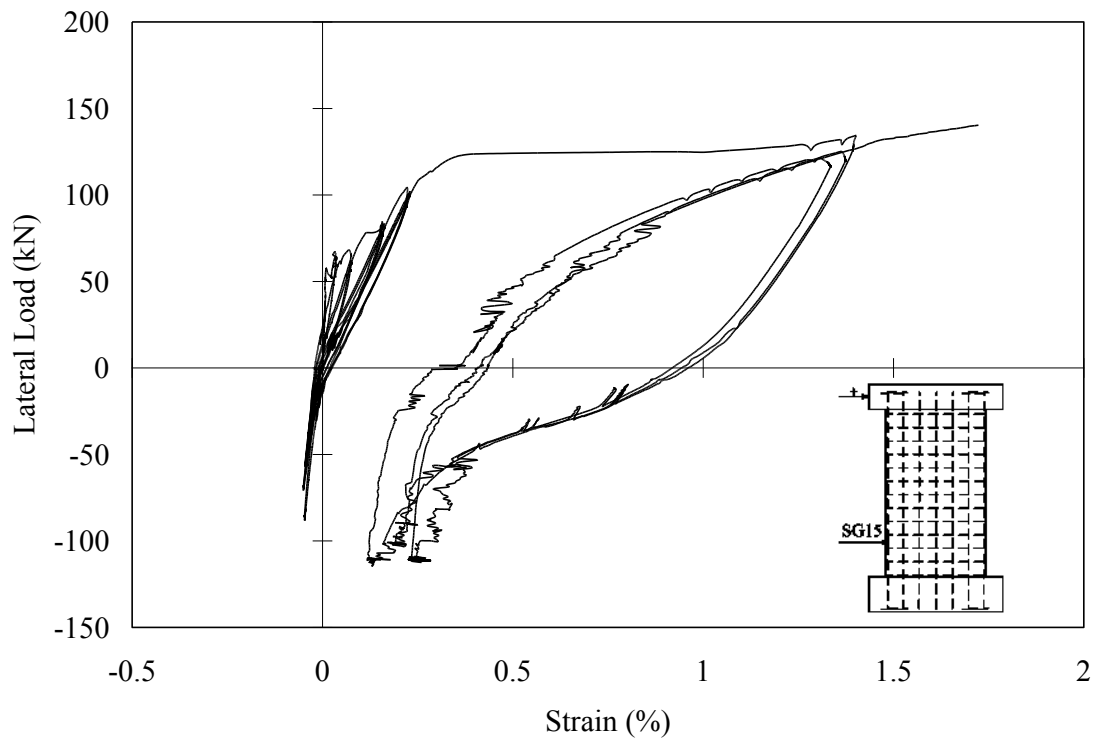


Figure E.23 SG15 Response in W1-SR

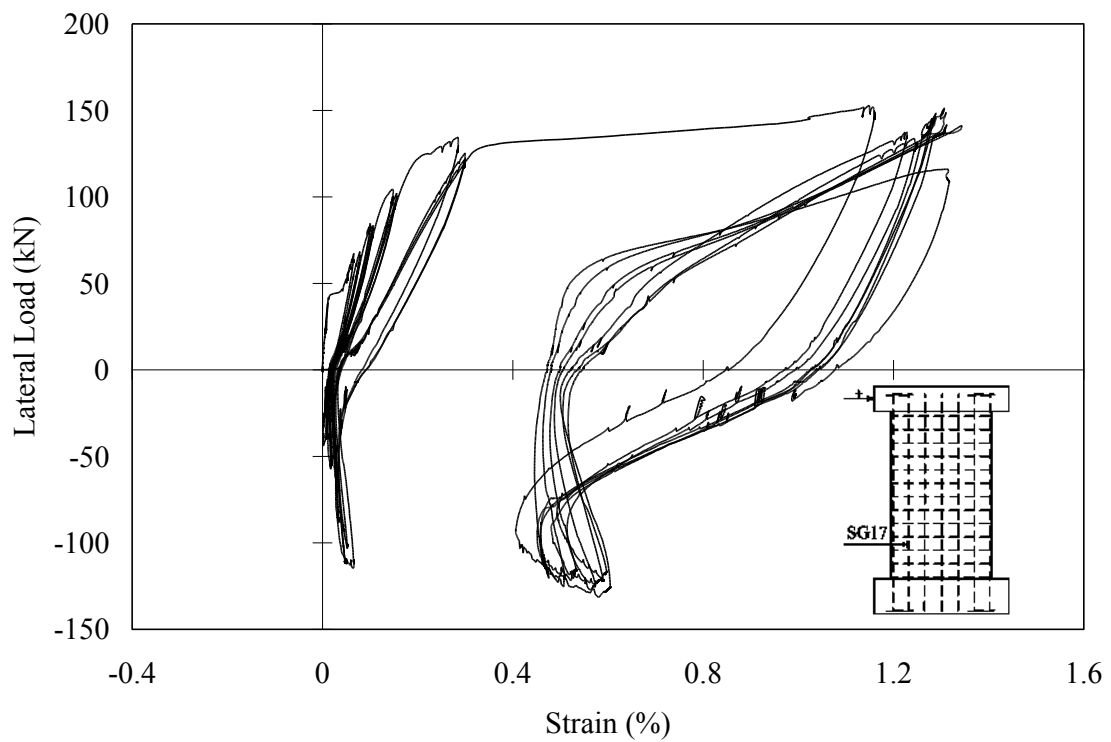


Figure E.24 SG17 Response in W1-SR

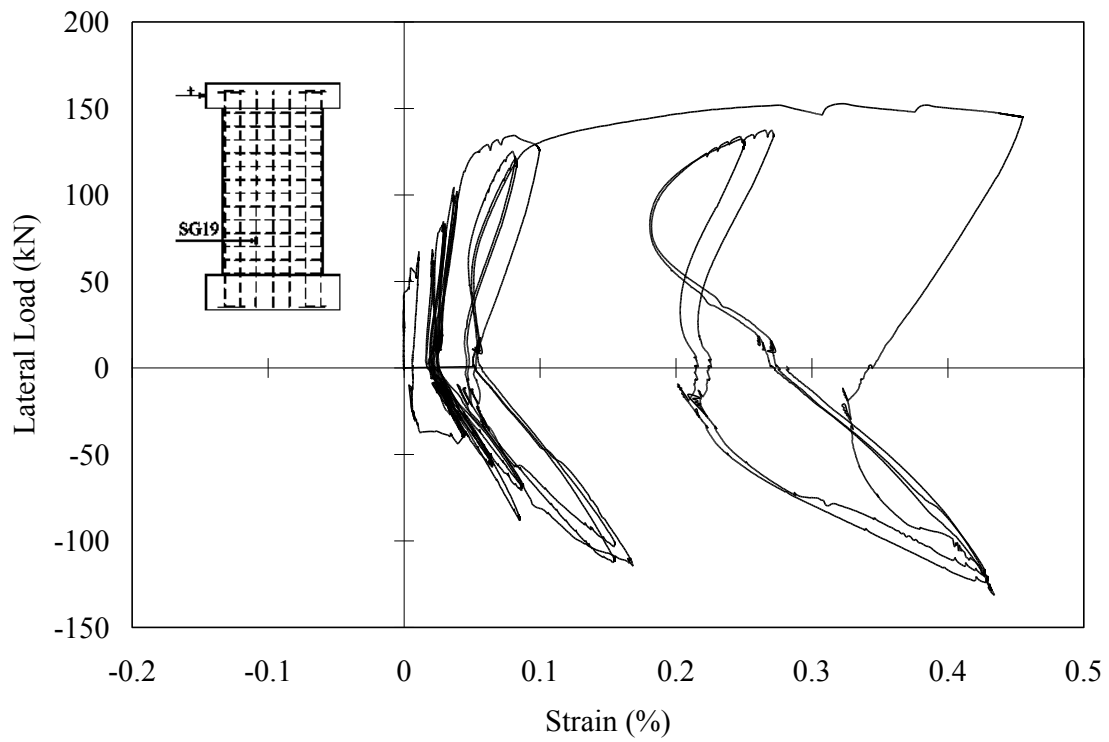


Figure E.25 SG19 Response in W1-SR

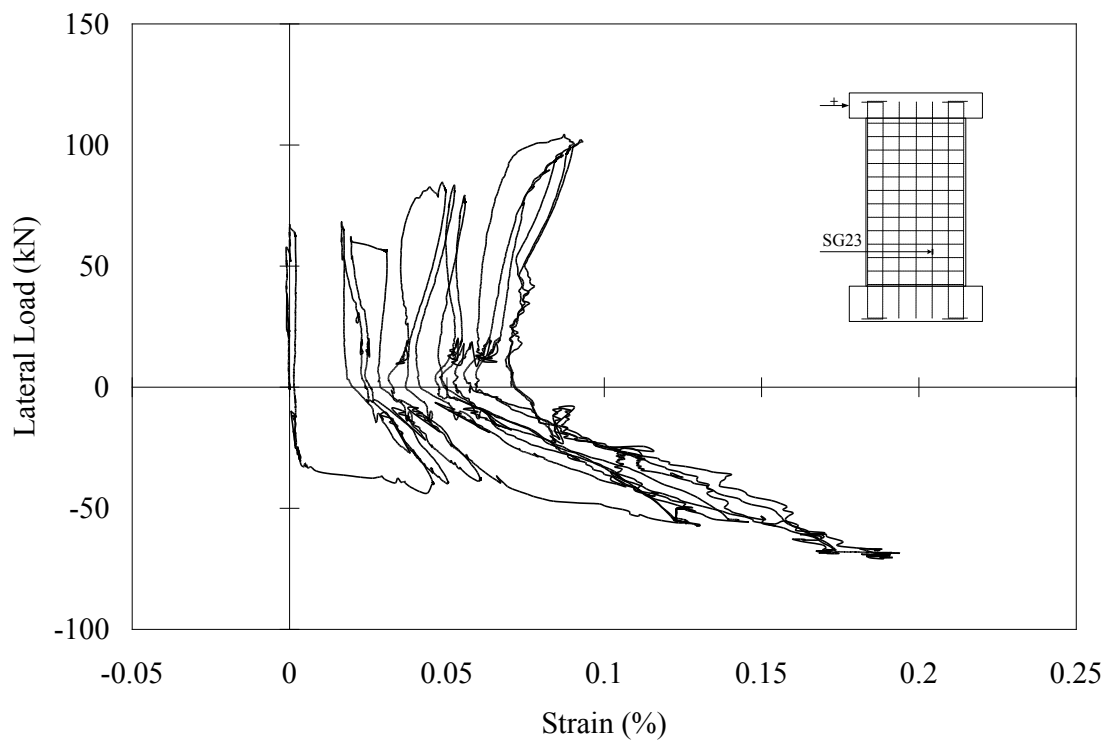


Figure E.26 SG23 Response in W1-SR

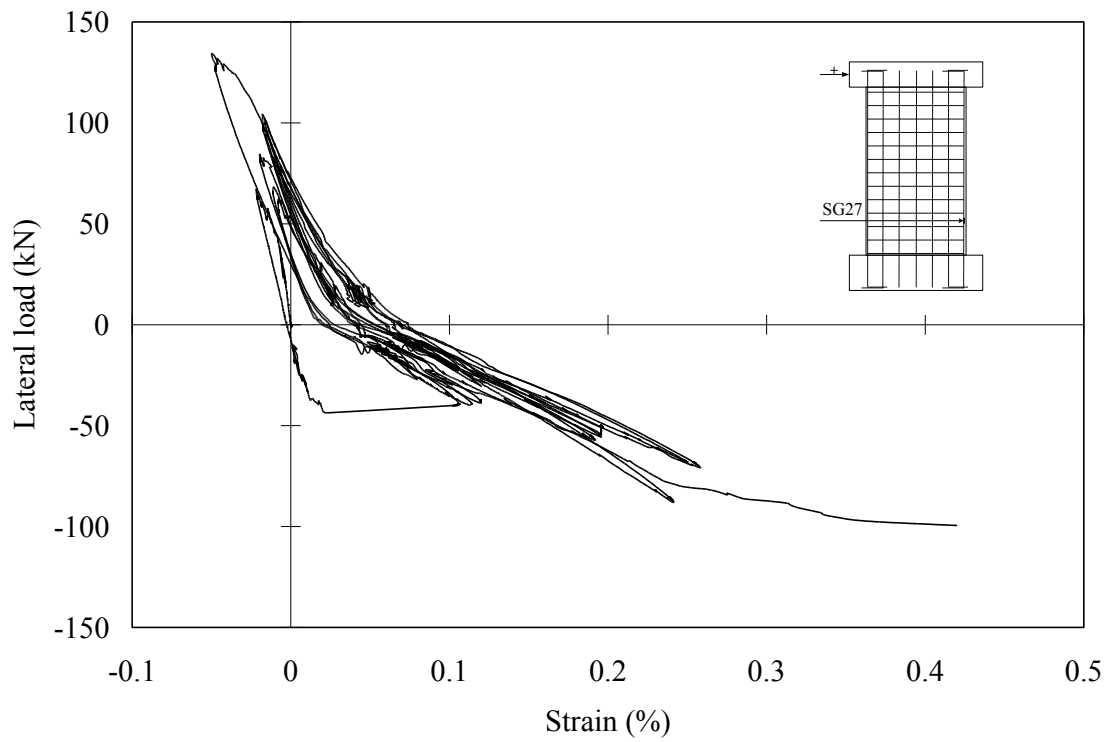


Figure E.27 SG27 Response in W1-SR

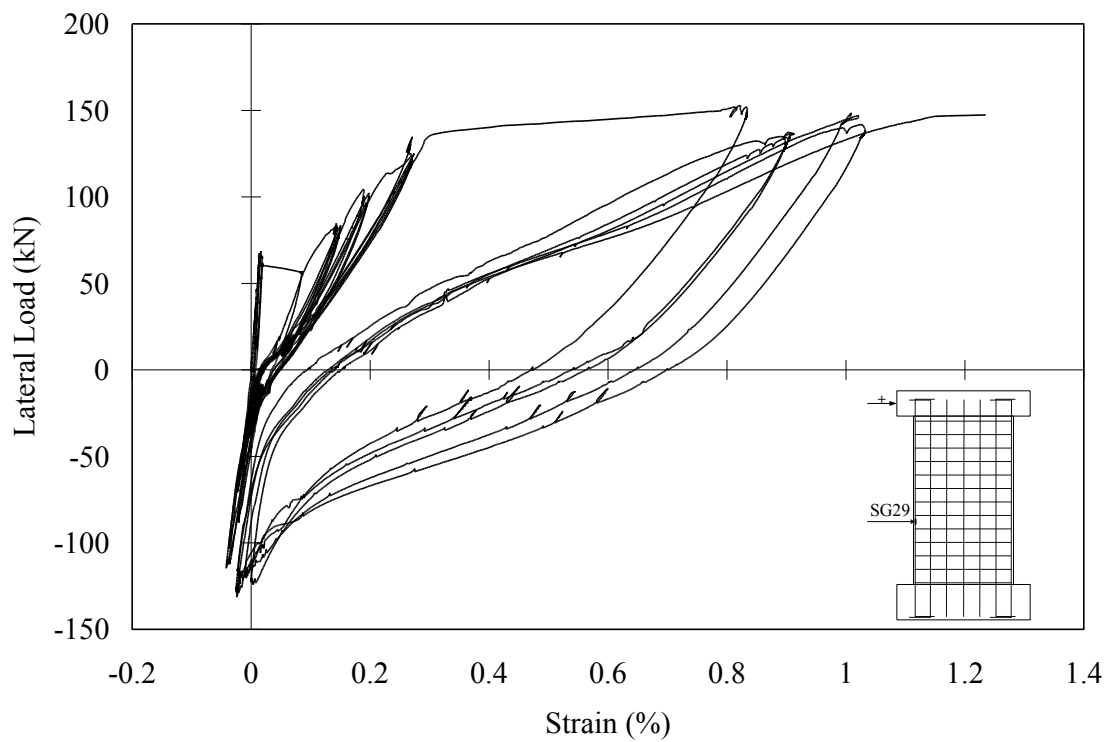


Figure E.28 SG29 Response in W1-SR

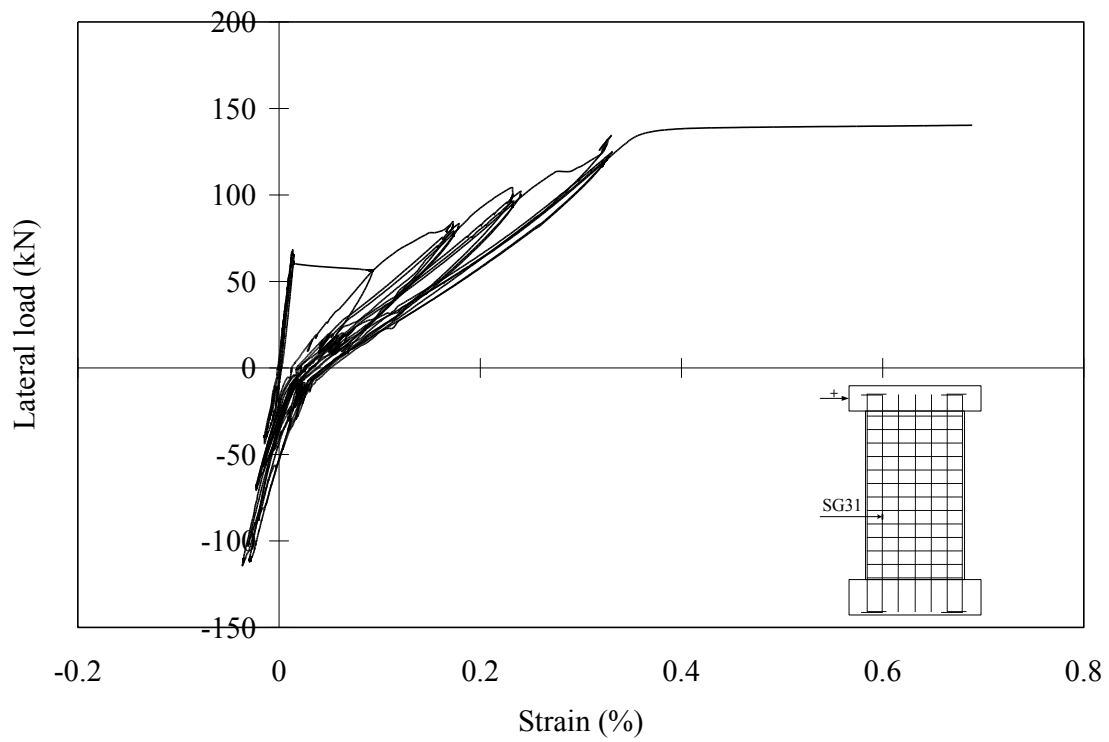


Figure E.29 SG31 Response in W1-SR

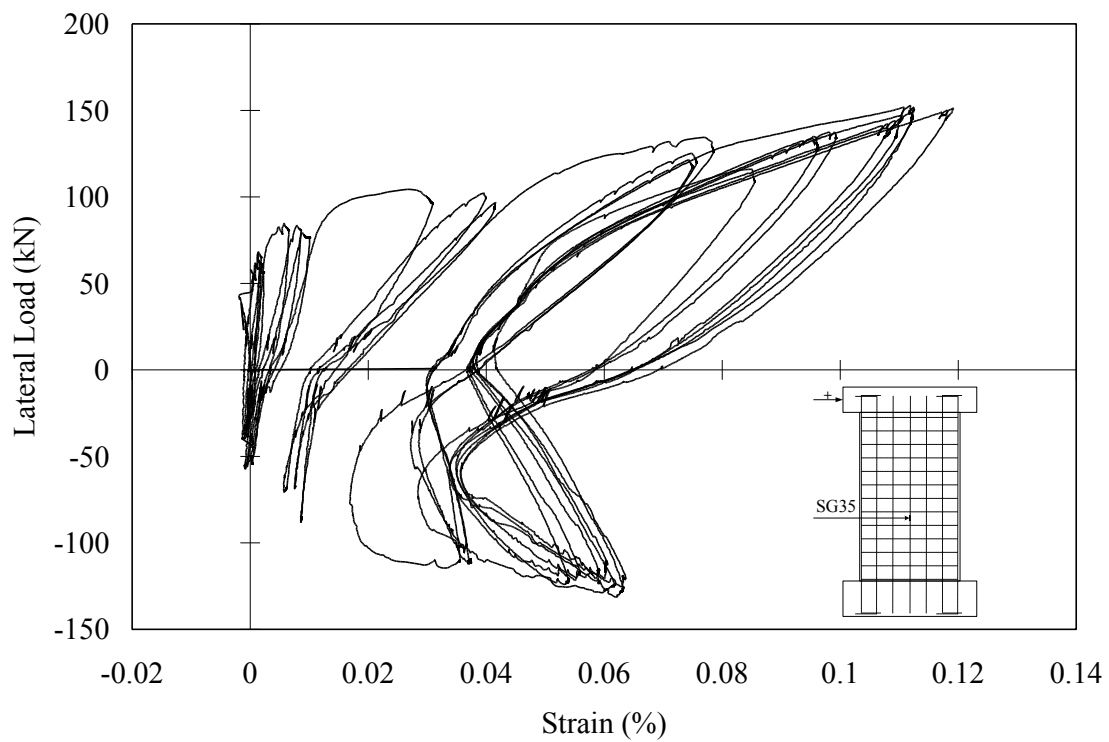


Figure E.30 SG35 Response in W1-SR

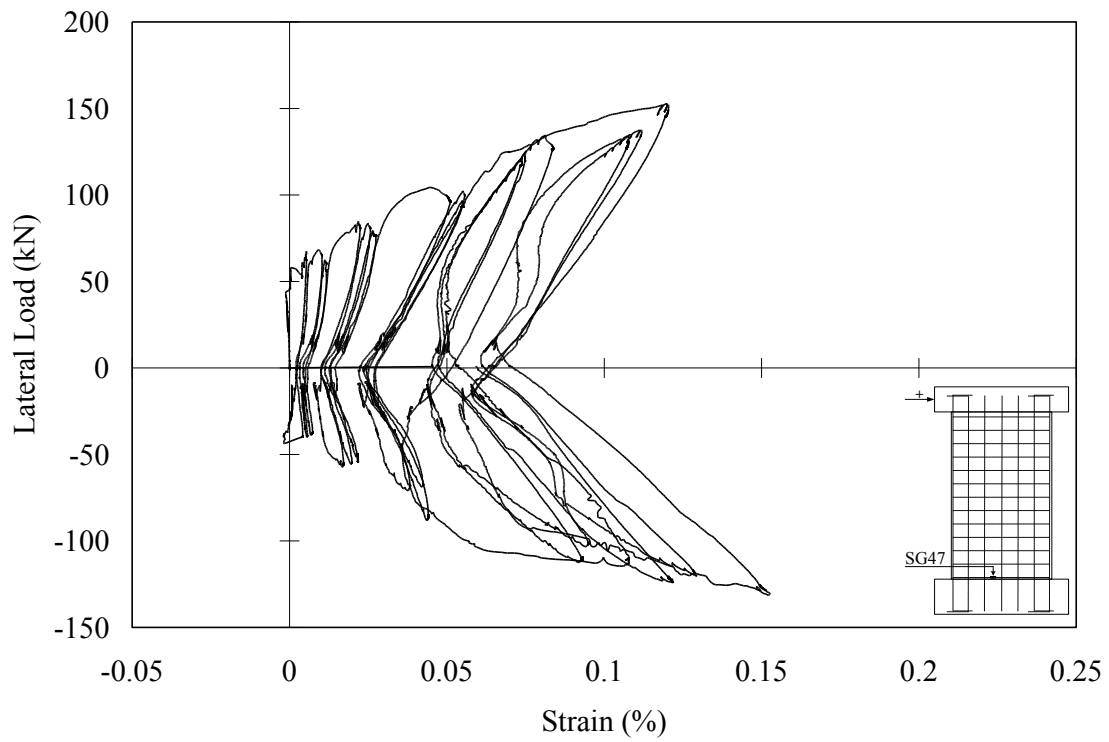


Figure E.31 SG47 Response in W1-SR

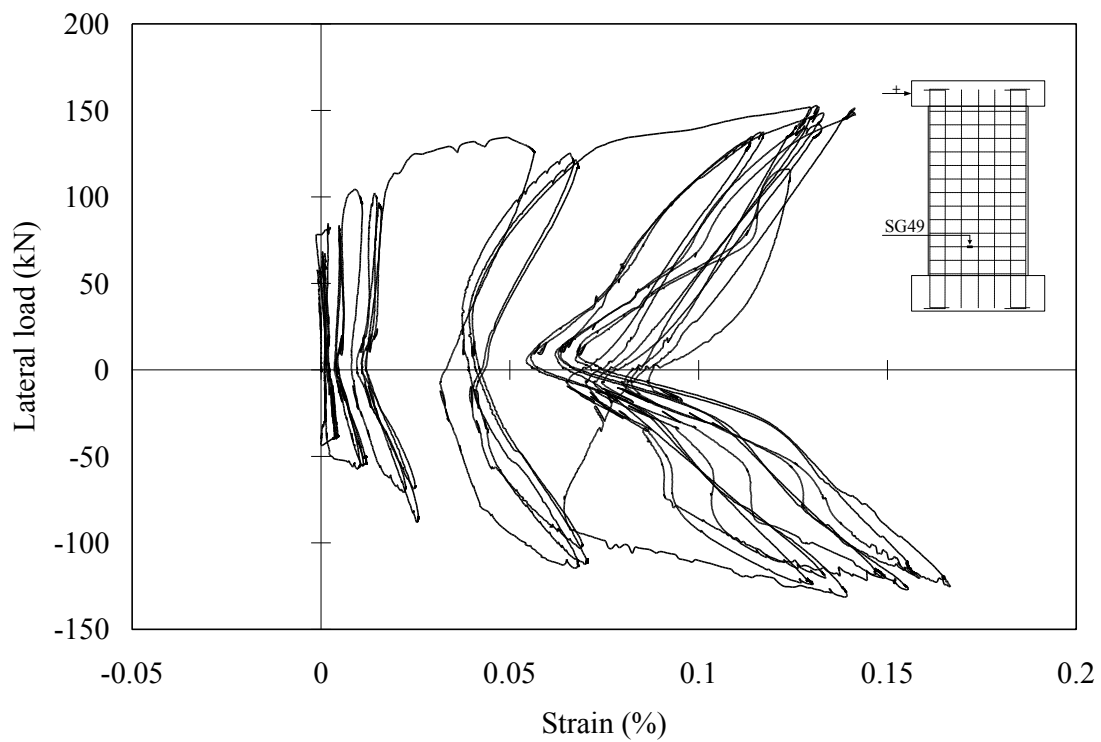


Figure E.32 SG49 Response in W1-SR



### E.2.2 Strain Gauge Responses in Wall W2-NR

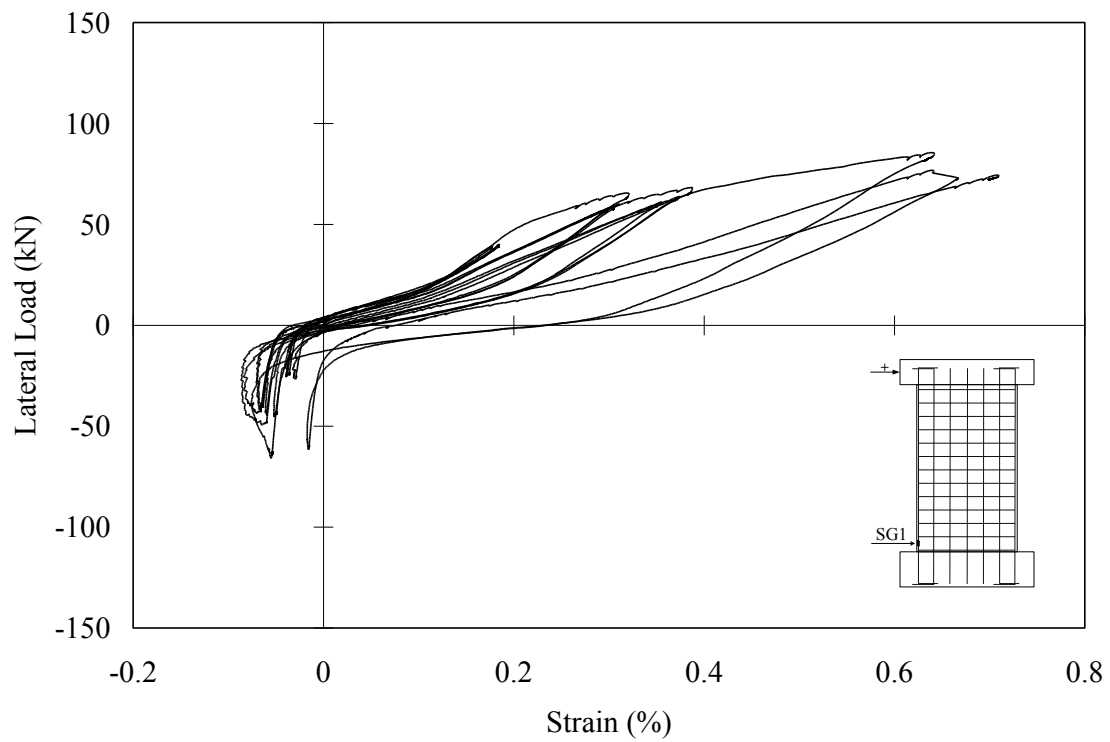


Figure E.33 SG1 Response in W2-NR

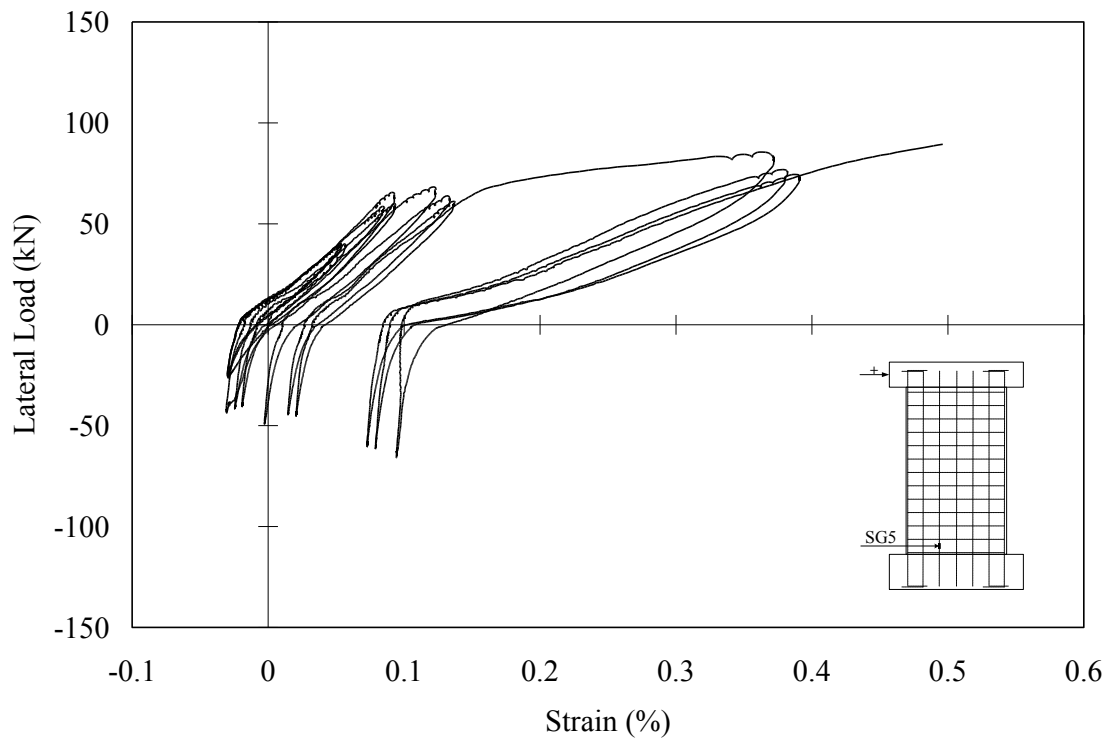


Figure E.34 SG5 Response in W2-NR

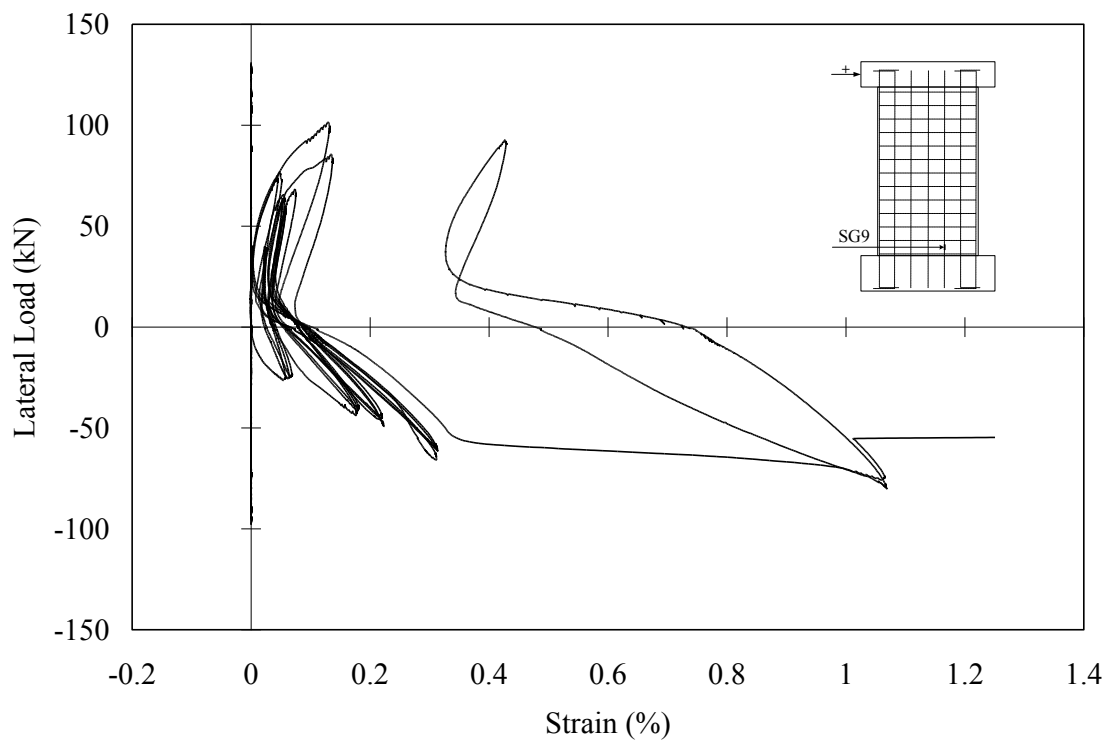


Figure E.35 SG9 Response in W2-NR

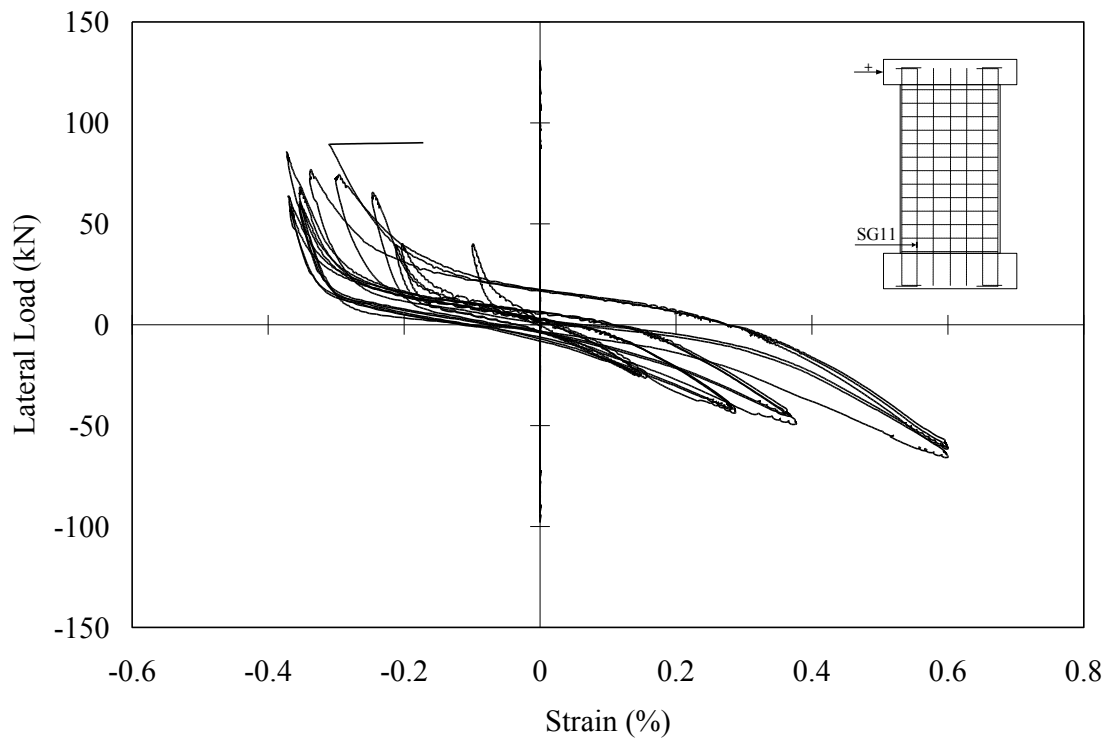


Figure E.36 SG11 Response in W2-NR

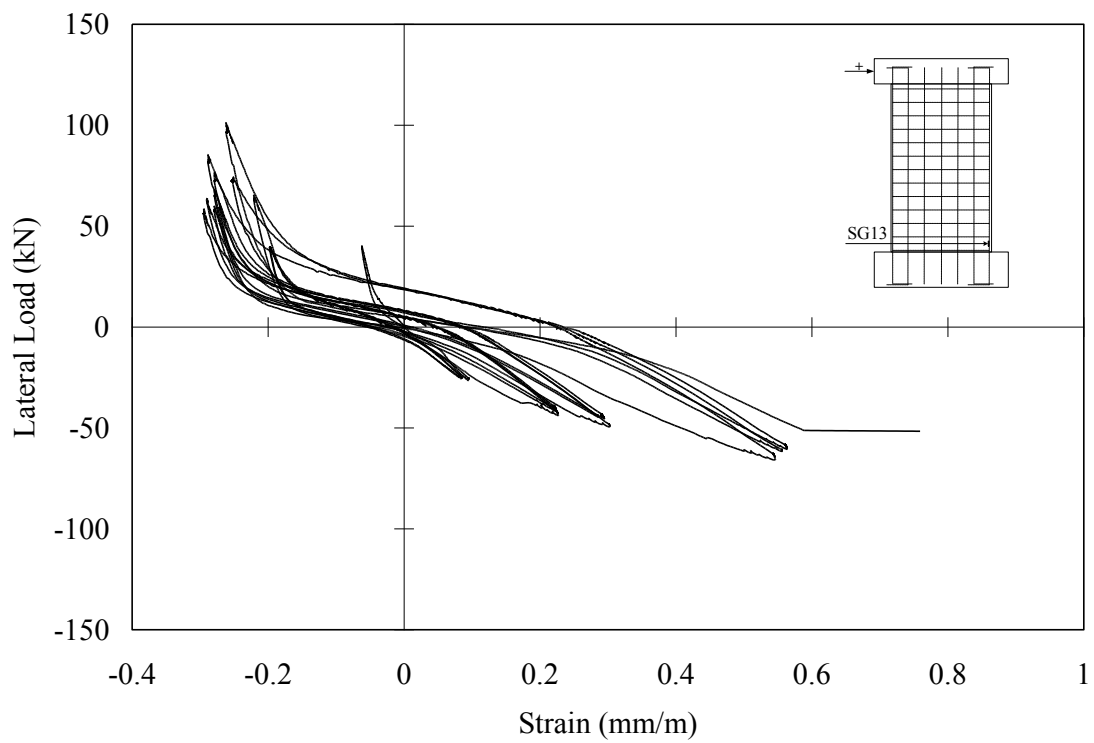


Figure E.37 SG13 Response in W2-NR

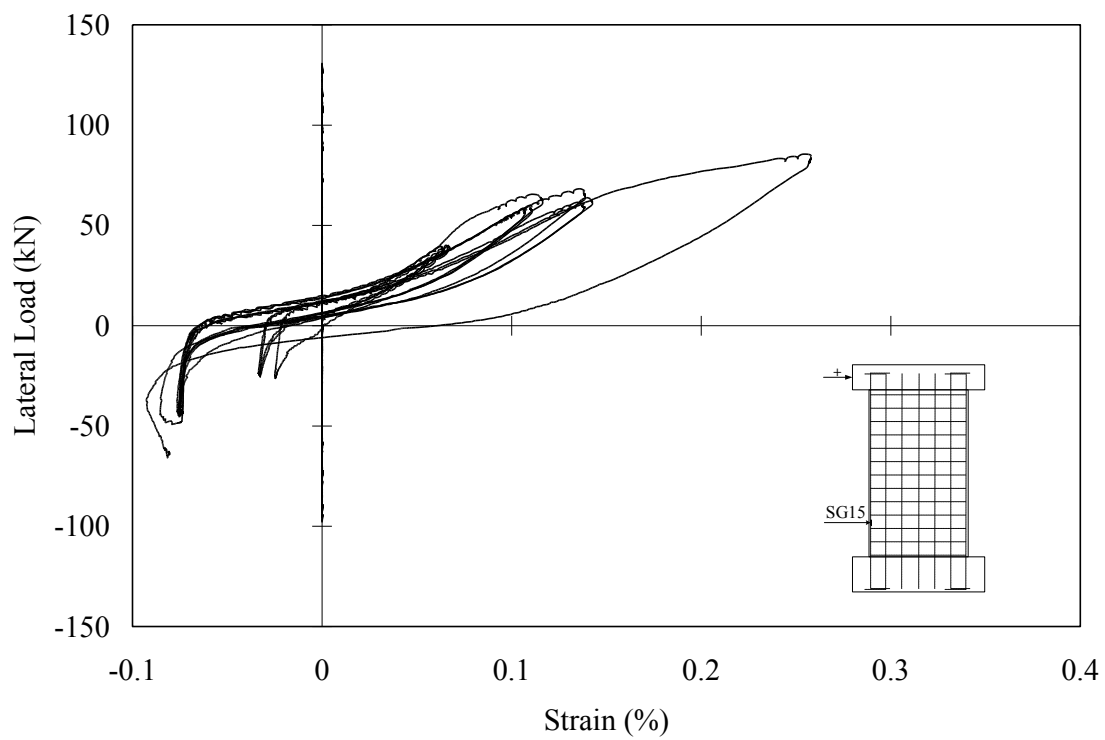


Figure E.38 SG15 Response in W2-NR

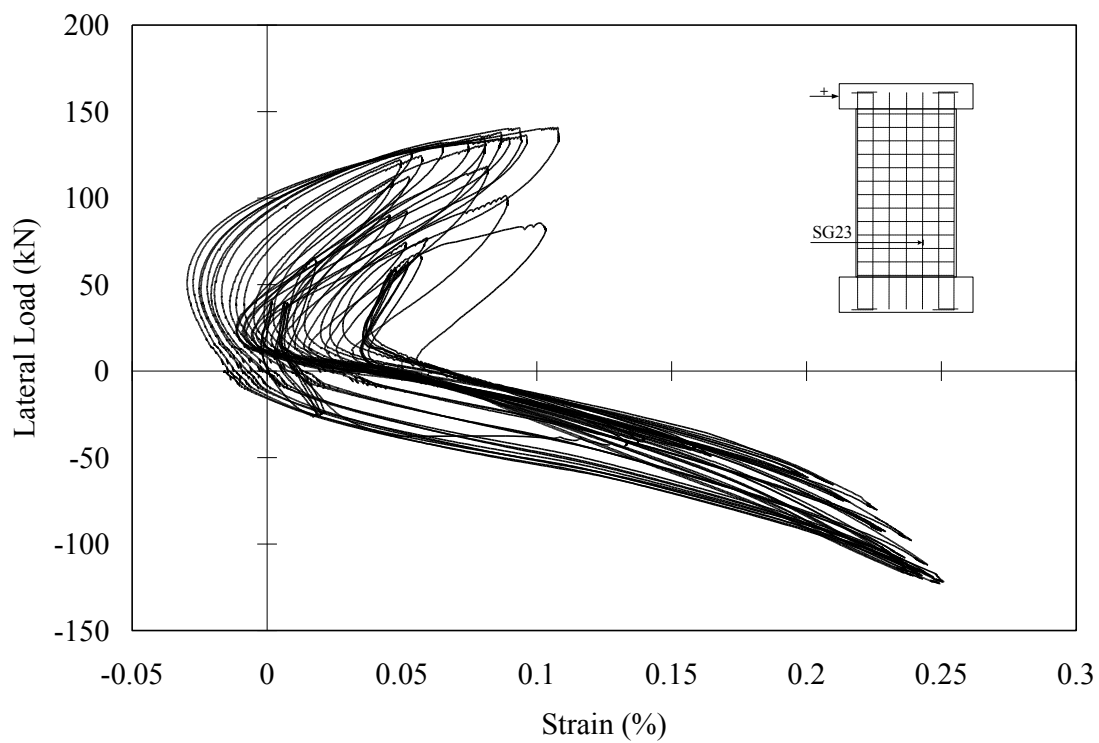


Figure E.39 SG23 Response in W2-NR

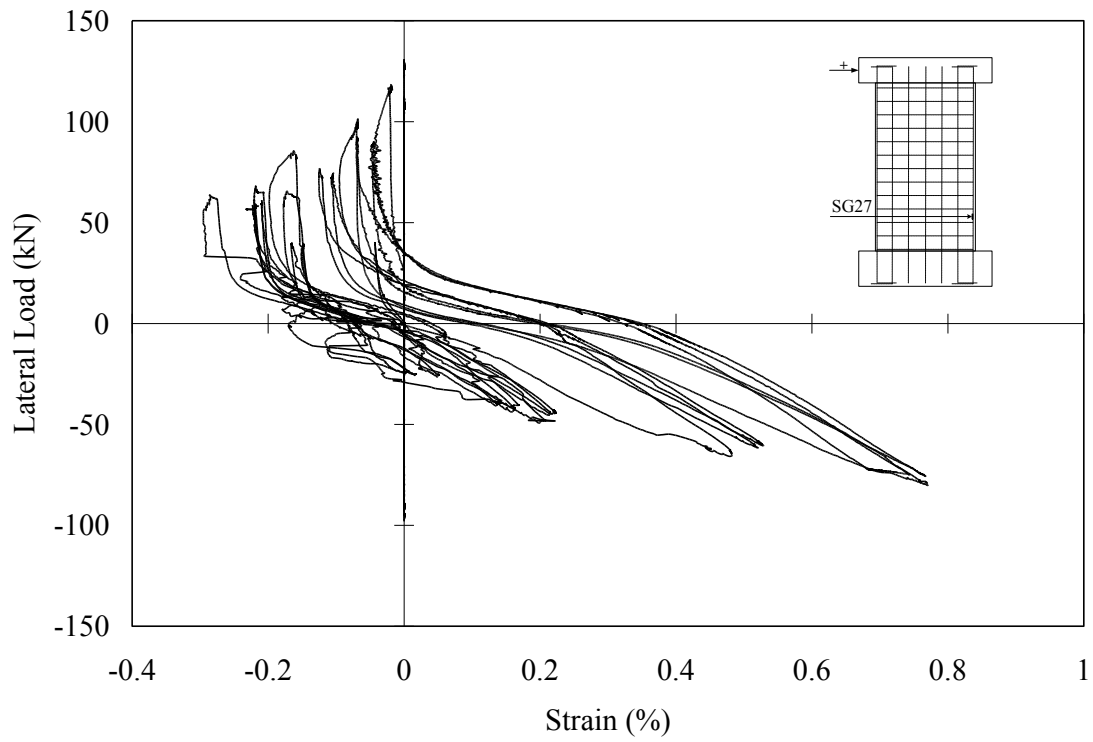


Figure E.40 SG27 Response in W2-NR

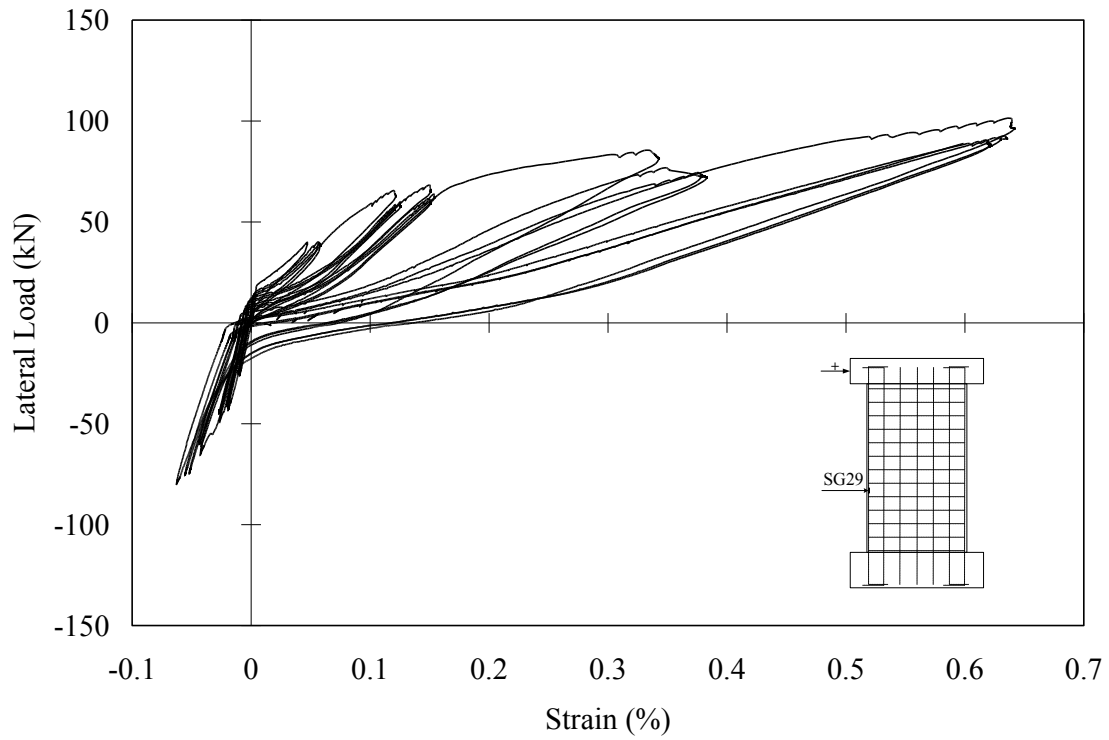


Figure E.41 SG29 Response in W2-NR

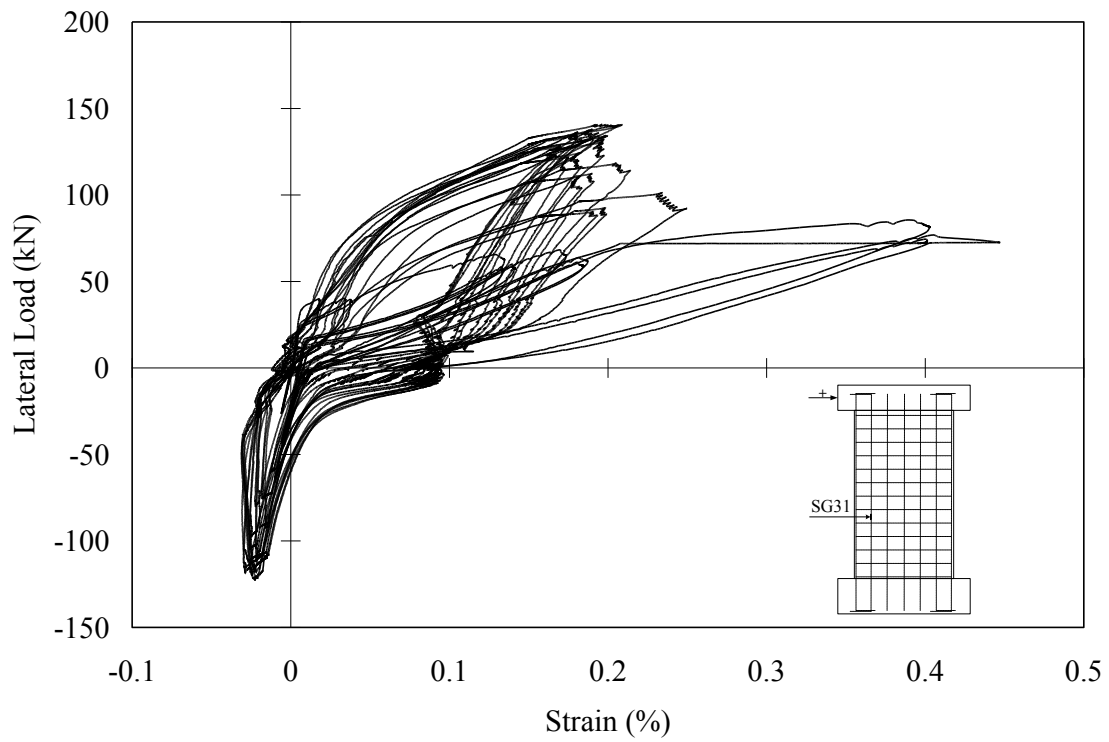


Figure E.42 SG31 Response in W2-NR

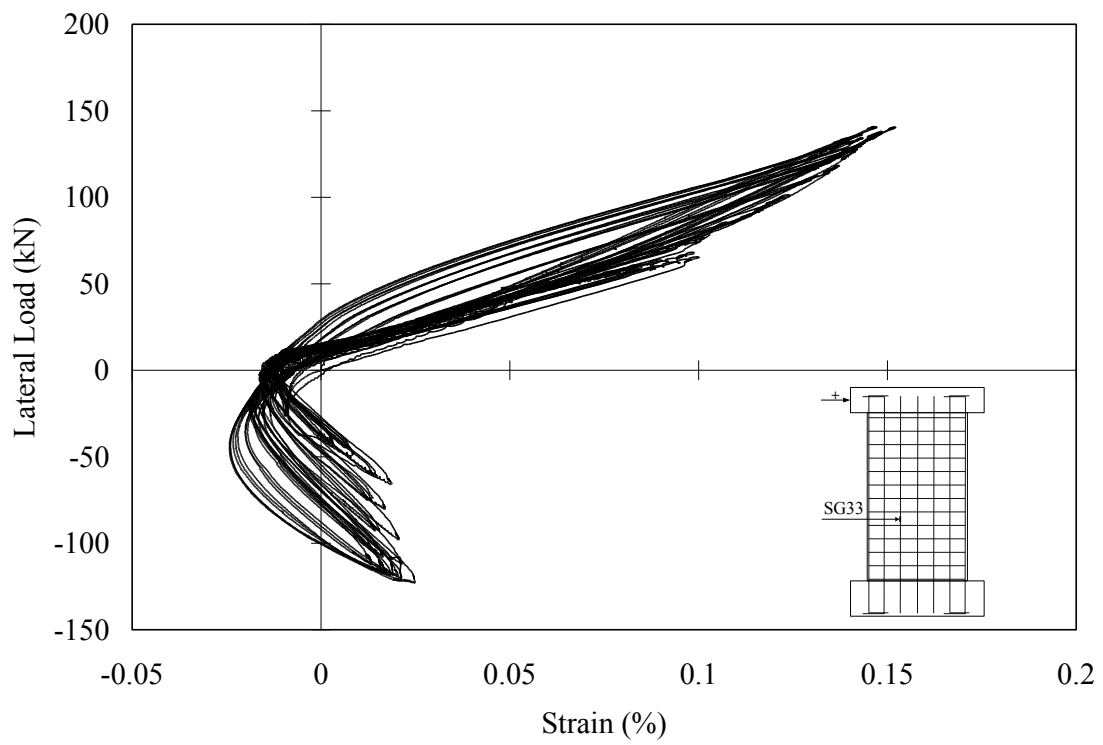


Figure E.43 SG33 Response in W2-NR

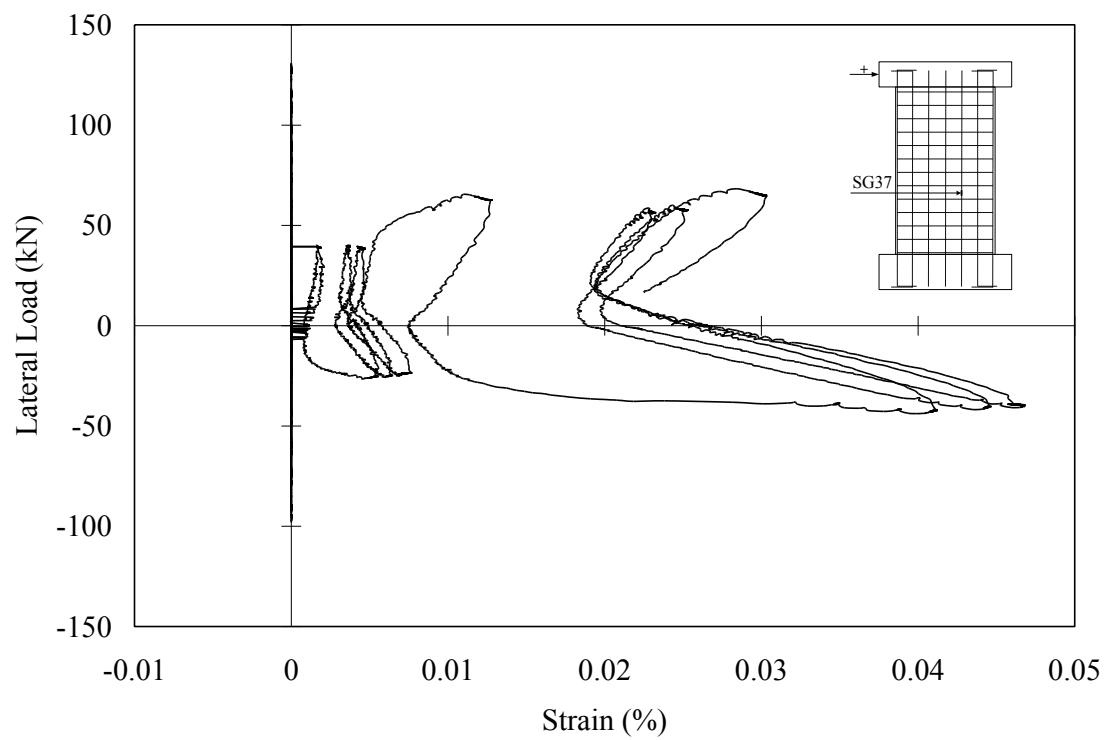


Figure E.44 SG37 Response in W2-NR

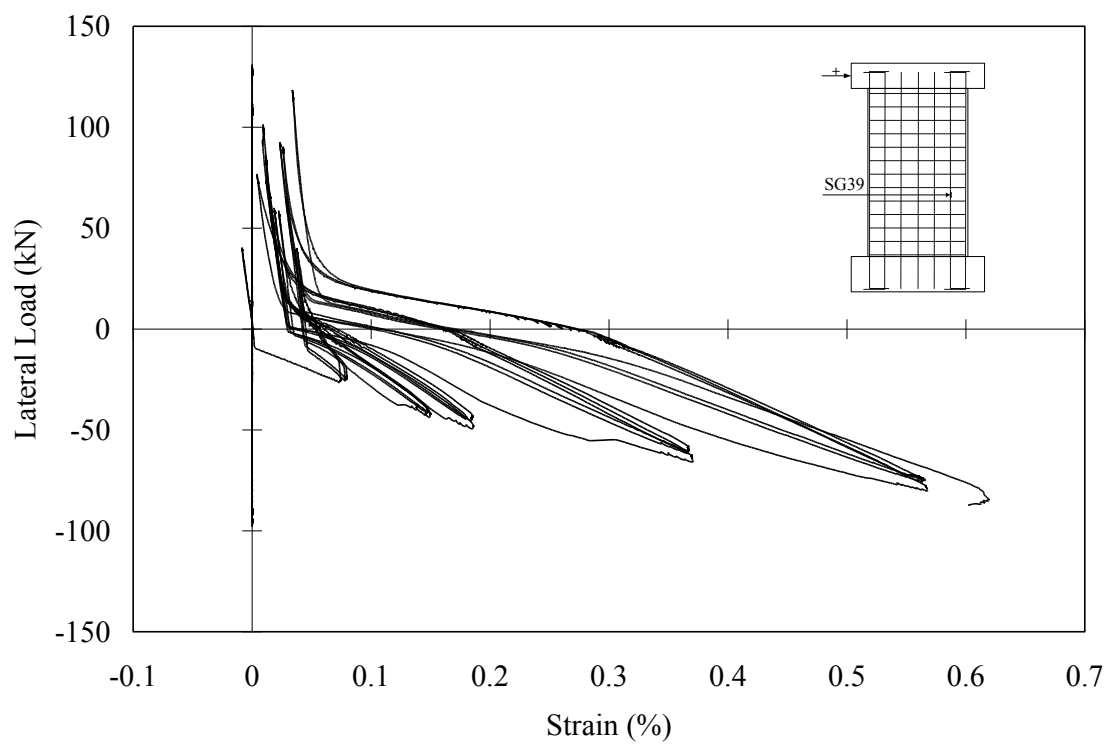


Figure E.45 SG39 Response in W2-NR

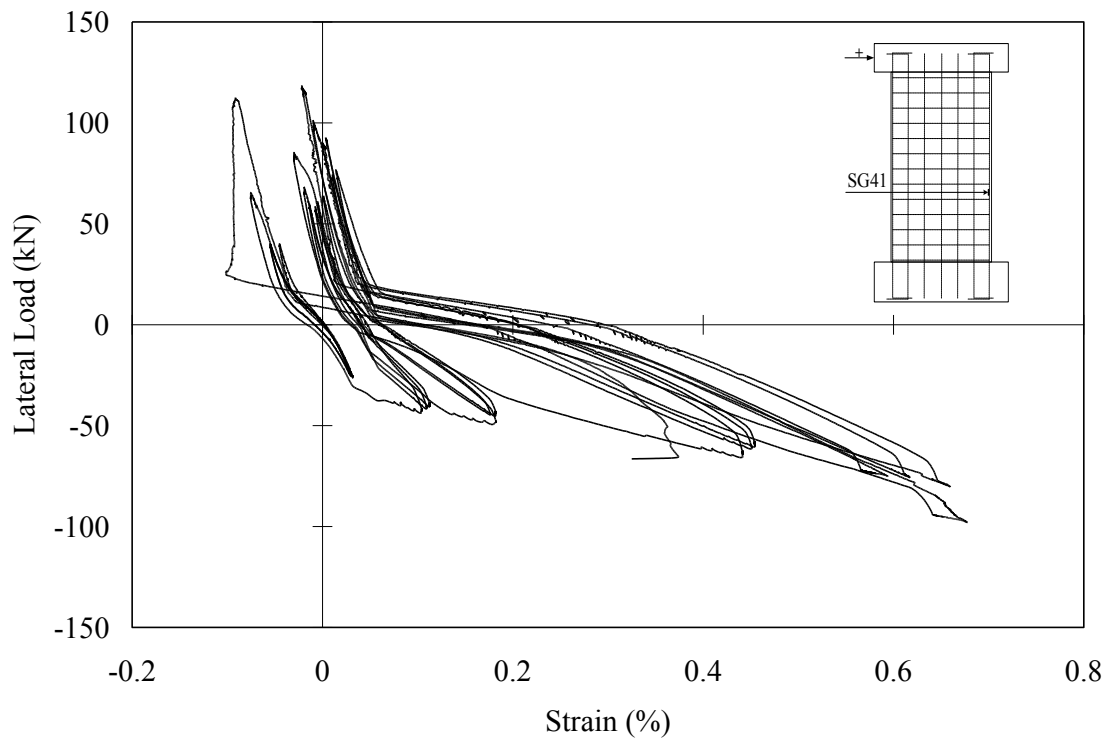


Figure E.46 SG41 Response in W2-NR

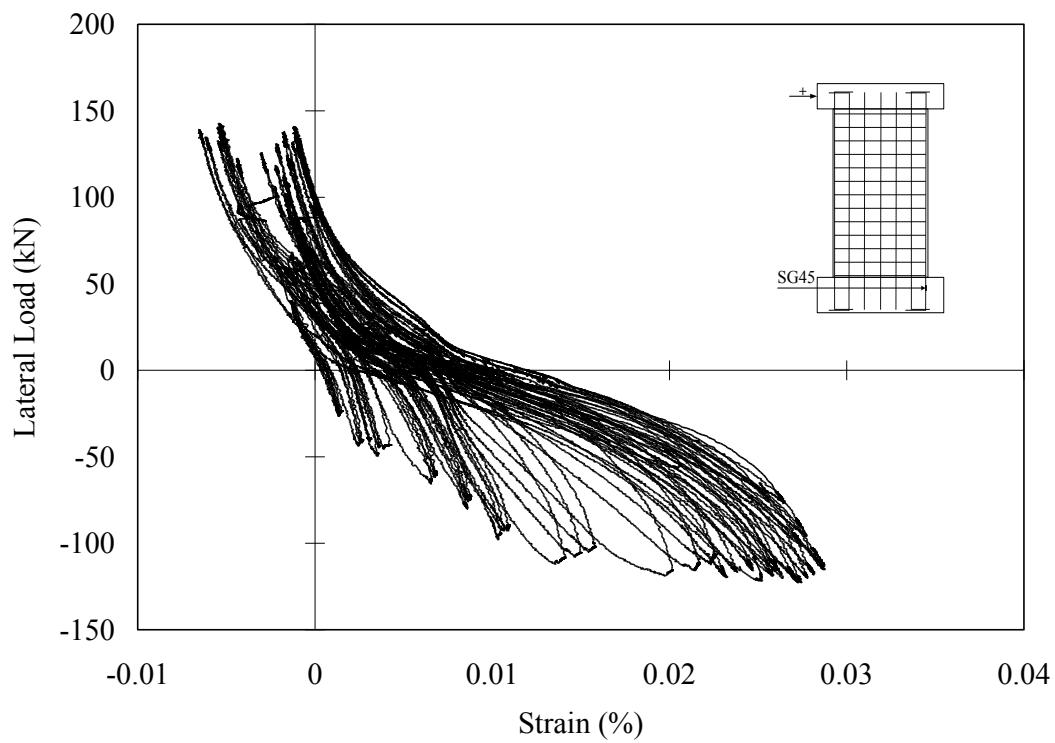


Figure E.47 SG45 Response in W2-NR



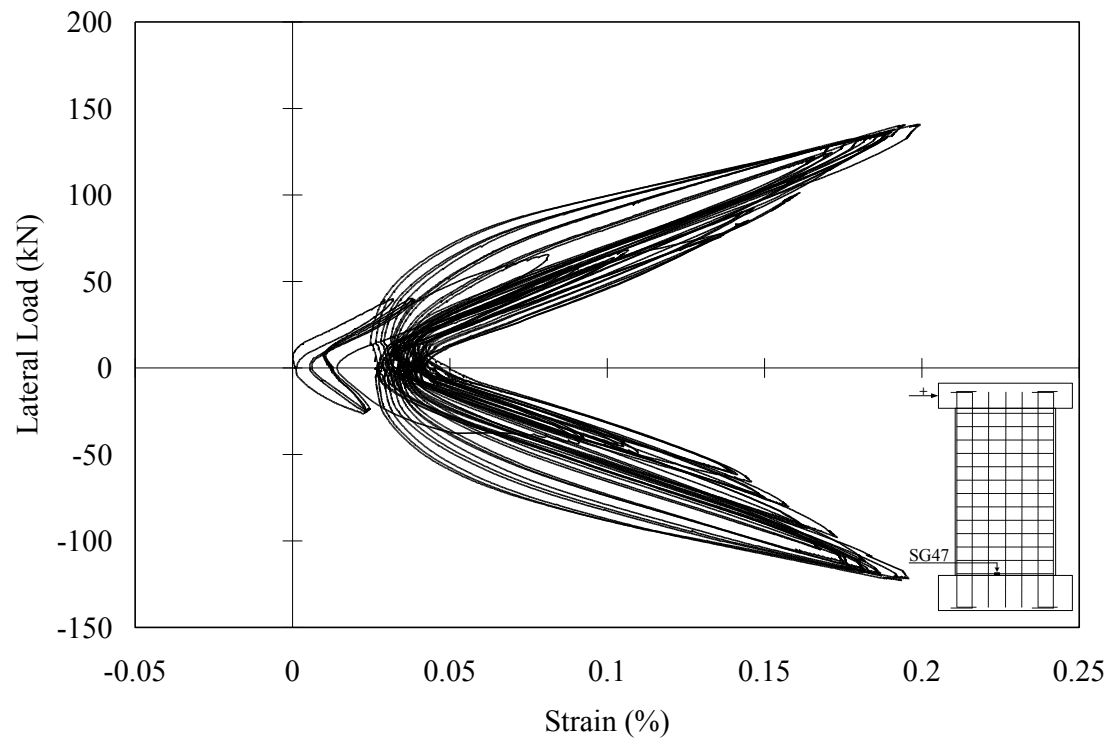


Figure E.48 SG47 Response in W2-NR

## APPENDIX F

### Cracking Characteristics in Beams and Shear Walls

#### F.1 Cracking Characteristics in Beams



Figure F.1 B2-SC Beam After ( $\Delta y/4$ ) Displacement



Figure F.2 B2-SC Beam After ( $\Delta y/2$ ) Displacement

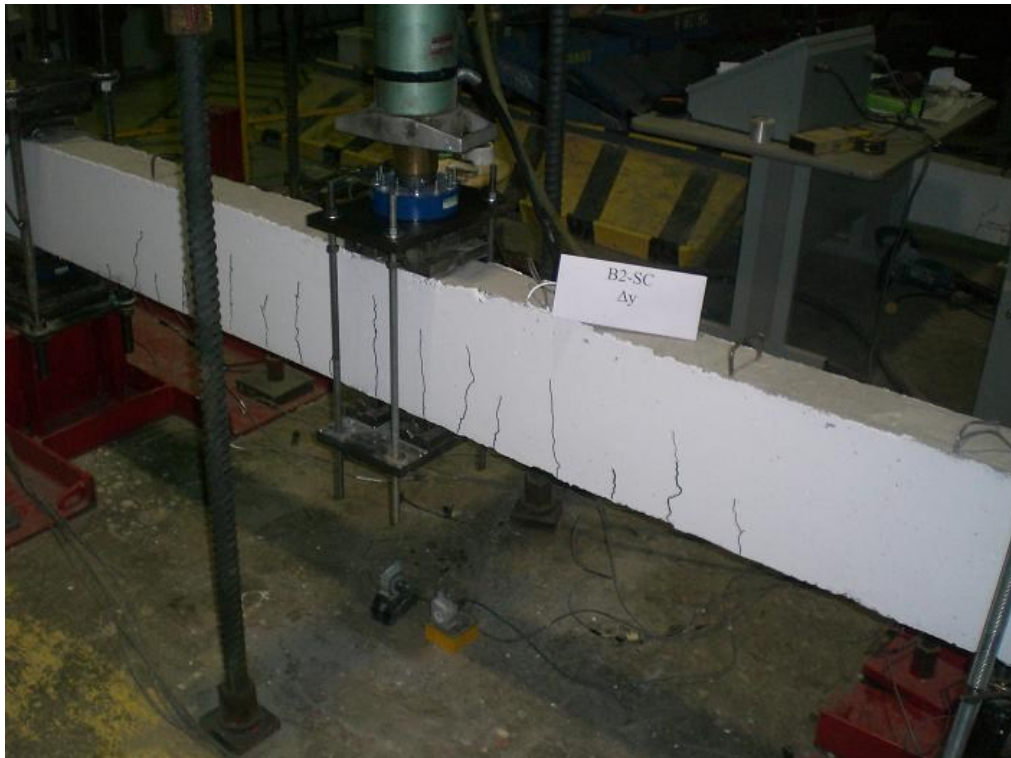


Figure F.3 B2-SC Beam After ( $\Delta y$ ) Displacement



Figure F.4 B2-SC Beam After ( $2\Delta y$ ) Displacement



Figure F.5 B2-SC Beam After ( $3\Delta y$ ) Displacement



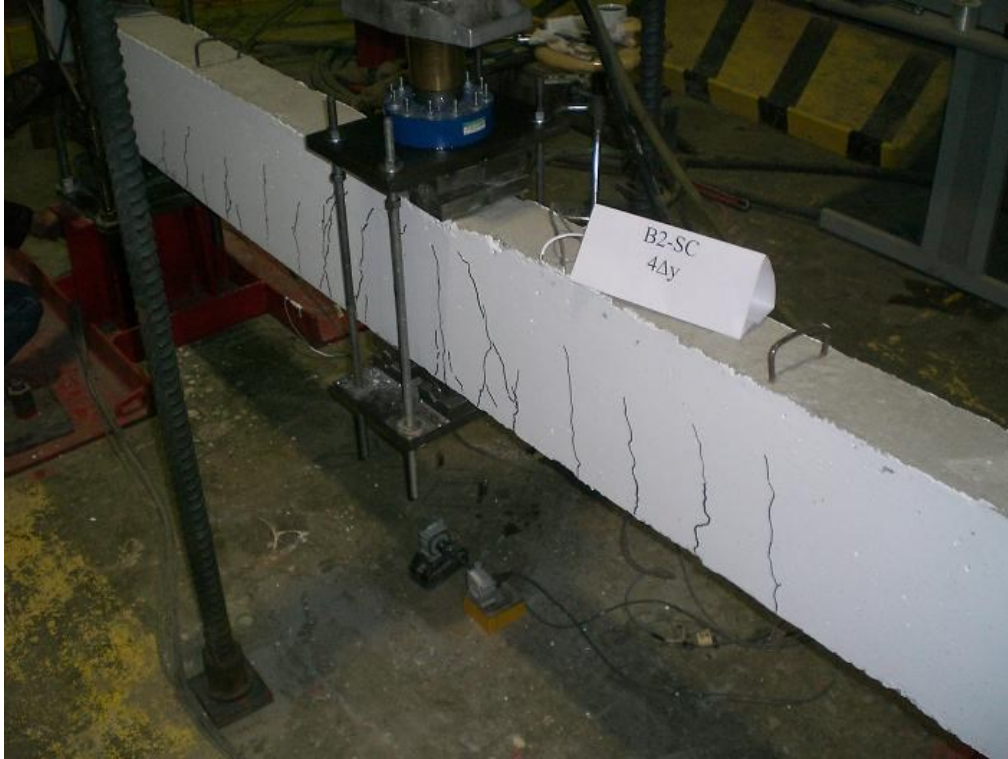


Figure F.6 B2-SC Beam After ( $4\Delta y$ ) Displacement



Figure F.7 B2-SC Beam After ( $5\Delta y$ ) Displacement

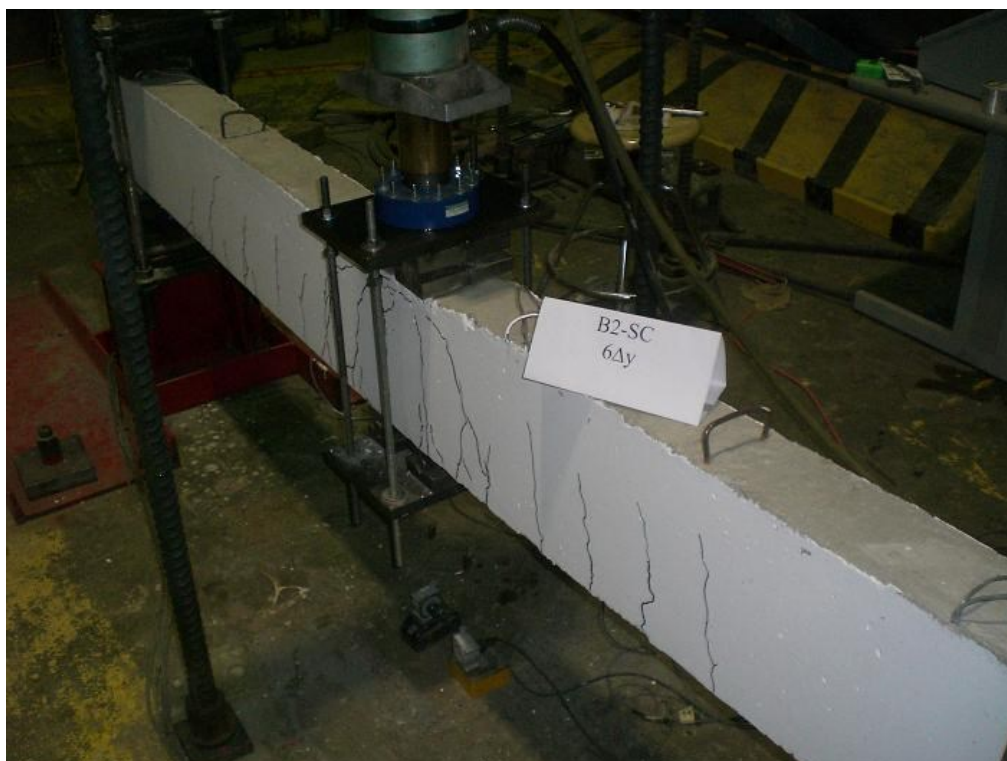


Figure F.8 B2-SC Beam After ( $6\Delta y$ ) Displacement



Figure F.9 B3-SR Beam After ( $\Delta y/4$ ) Displacement





Figure F.10 B3-SR Beam After ( $\Delta y/2$ ) Displacement



Figure F.11 B3-SR Beam After ( $\Delta y$ ) Displacement

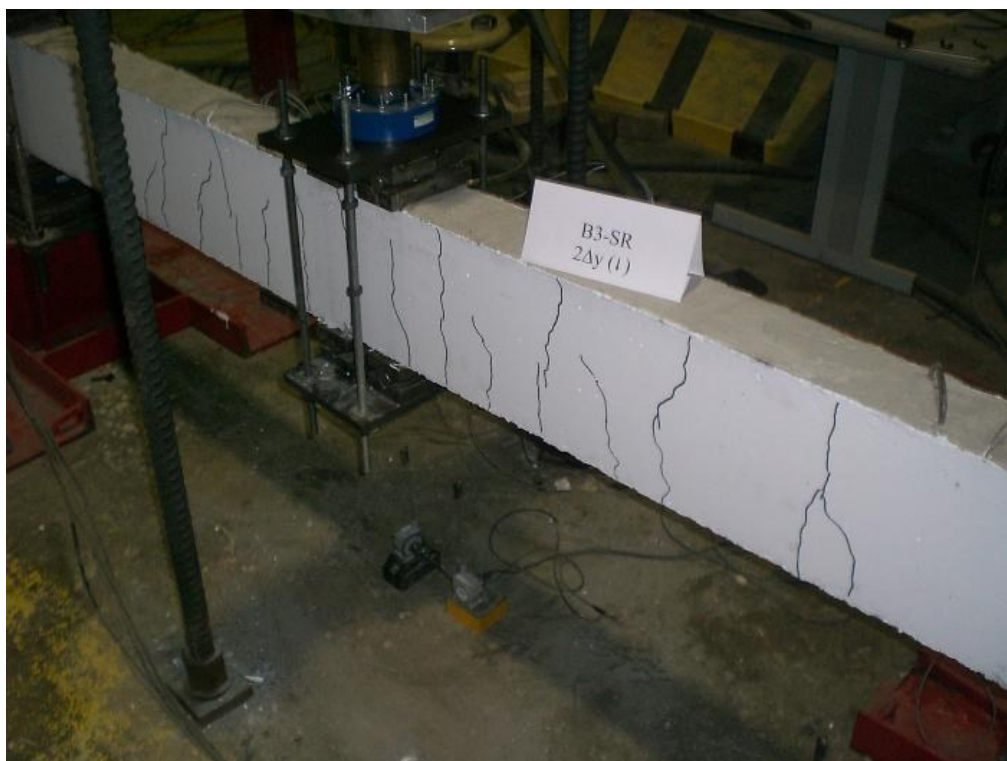


Figure F.12 B3-SR Beam After ( $2\Delta y$ ) Displacement

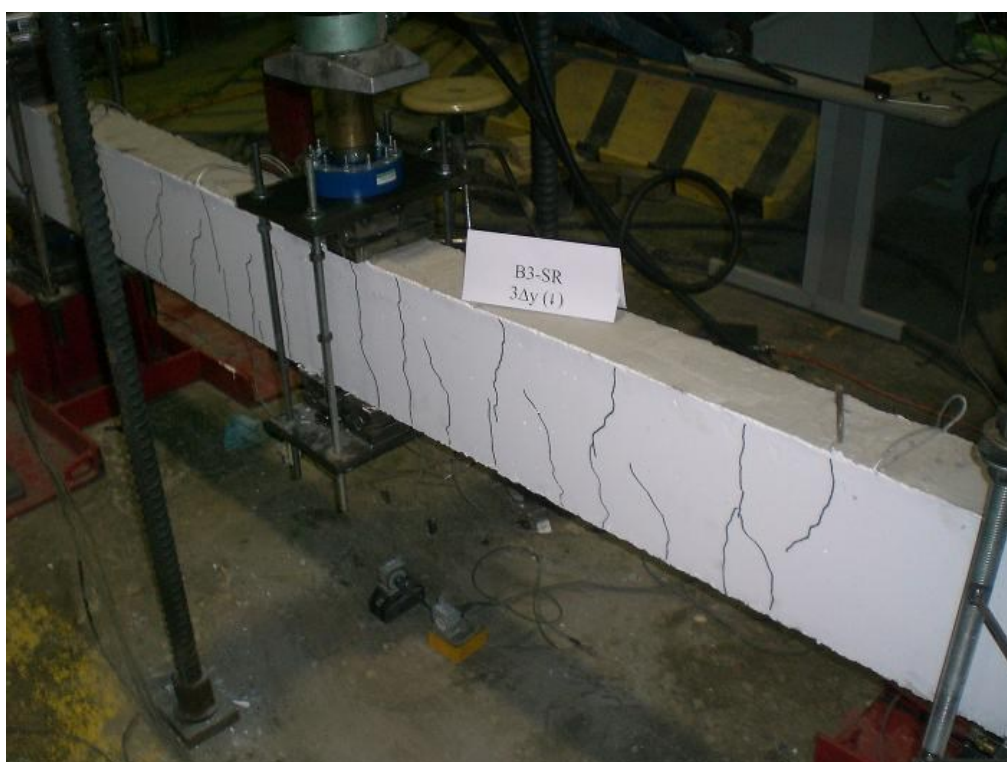


Figure F.13 B3-SR Beam After ( $3\Delta y$ ) Displacement





Figure F.14 B3-SR Beam After ( $4\Delta y$ ) Displacement

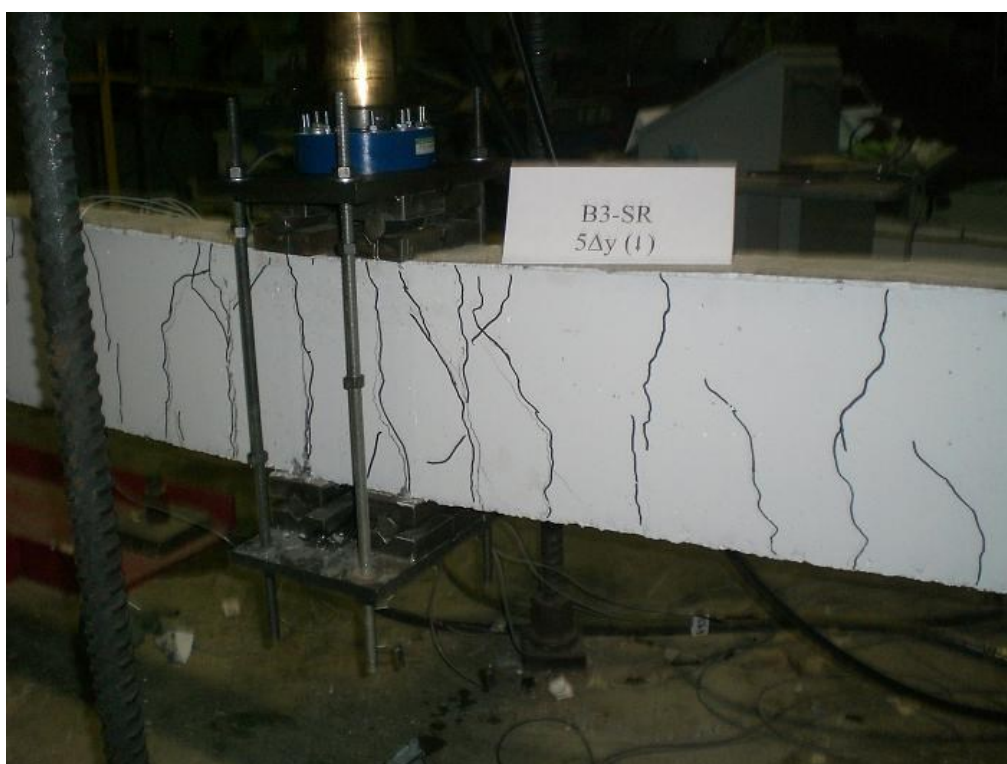


Figure F.15 B3-SR Beam After ( $5\Delta y$ ) Displacement



Figure F.16 B3-SR Beam After ( $7\Delta y$ ) Displacement



Figure F.17 B3-SR Beam After ( $8\Delta y$ ) Displacement



Figure F.18 B5-NC Beam After ( $\Delta y/4$ ) Displacement



Figure F.19 B5-NC Beam After ( $\Delta y/2$ ) Displacement





Figure F.20 B5-NC Beam After ( $\Delta y$ ) Displacement



Figure F.21 B5-NC Beam After ( $2\Delta y$ ) Displacement



Figure F.22 B5-NC Beam After ( $3\Delta y$ ) Displacement



Figure F.23 B5-NC Beam After ( $4\Delta y$ ) Displacement



Figure F.24 B5-NC Beam After ( $5\Delta y$ ) Displacement



Figure F.25 B5-NC Beam After ( $6\Delta y$ ) Displacement





Figure F.26 B6-NR Beam After ( $\Delta y/4$ ) Displacement



Figure F.27 B6-NR Beam After ( $\Delta y/2$ ) Displacement



Figure F.28 B6-NR Beam After ( $\Delta y$ ) Displacement



Figure F.29 B6-NR Beam After ( $2\Delta y$ ) Displacement



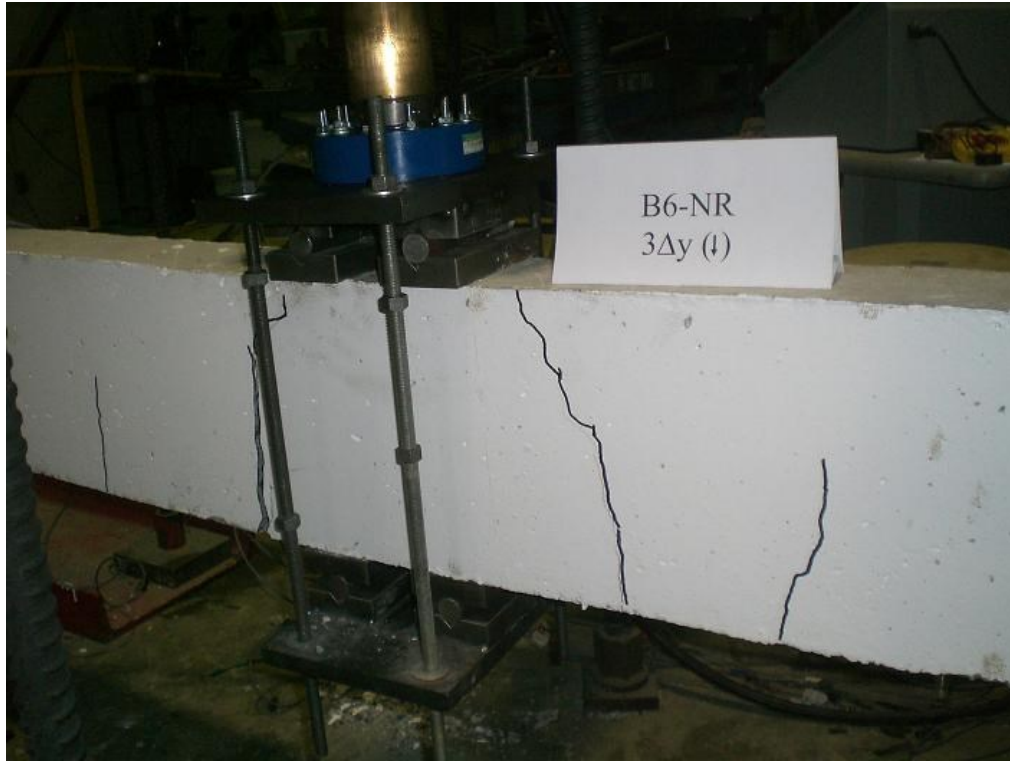


Figure F.30 B6-NR Beam After ( $3\Delta y$ ) Displacement



Figure F.31 B6-NR Beam After ( $4\Delta y$ ) Displacement



Figure F.32 B6-NR Beam After ( $5\Delta y$ ) Displacement

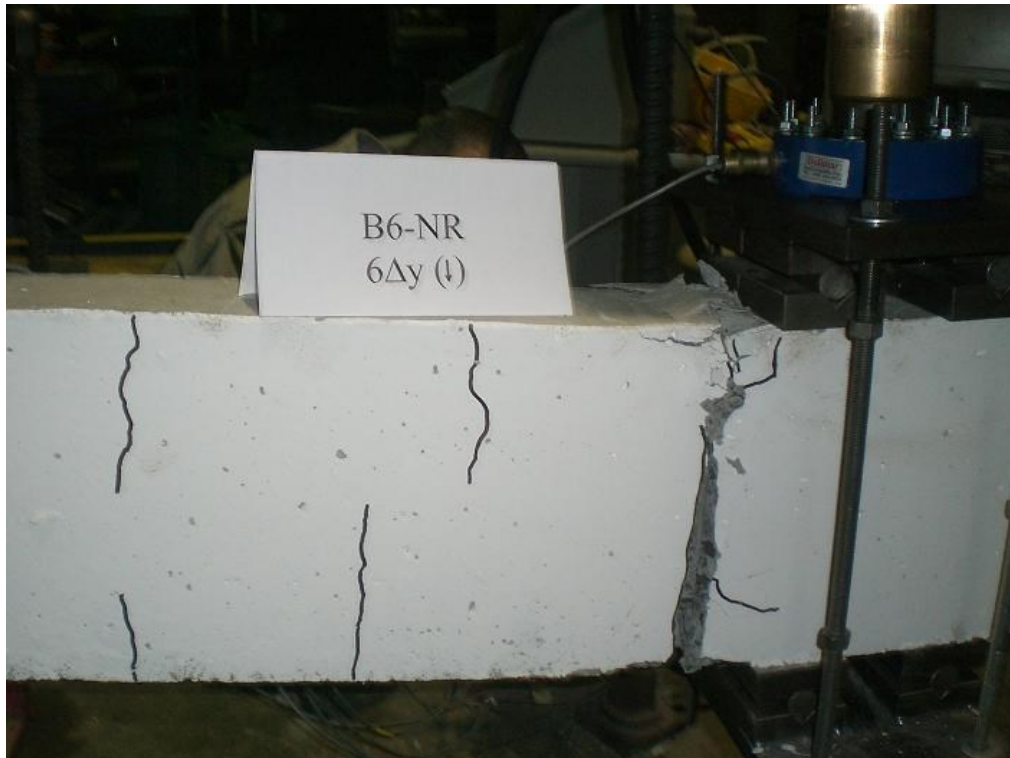


Figure F.33 B6-NR Beam After ( $6\Delta y$ ) Displacement

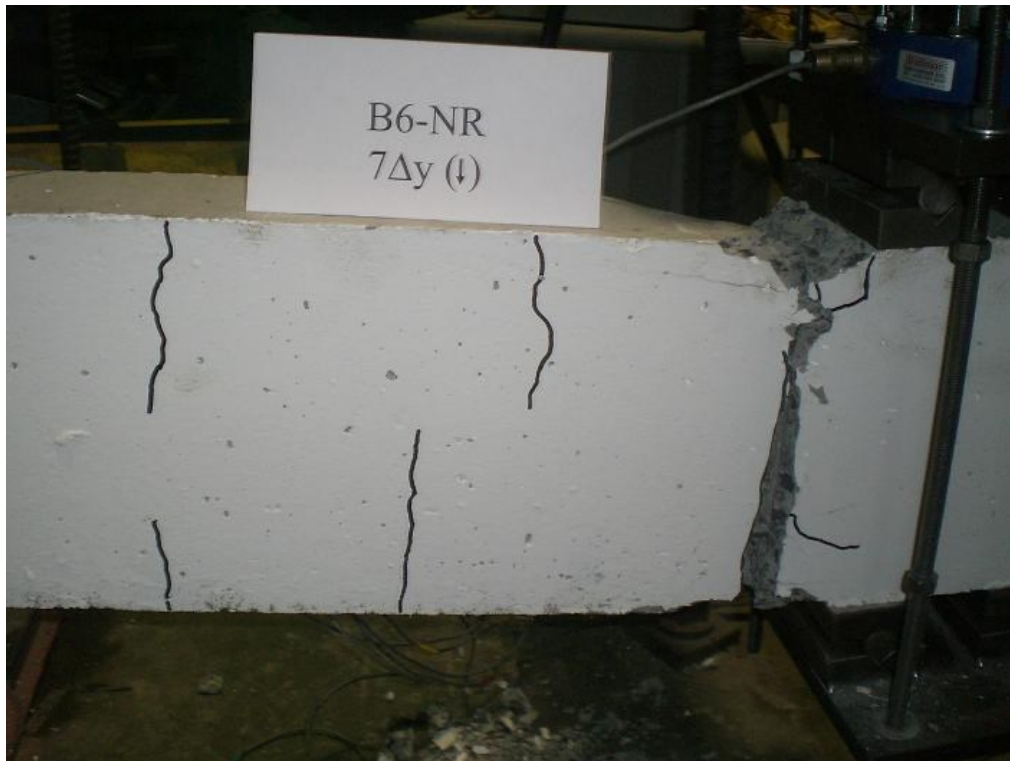


Figure F.34 B6-NR Beam After ( $7\Delta y$ ) Displacement

## F.2 Cracking Characteristics in Shear Walls



Figure F.35 W1-SR Wall After (0.1%) Drift





Figure F.36 W1-SR Wall After (0.2%) Drift



Figure F.37 W1-SR Wall After (0.25%) Drift



Figure F.38 W1-SR Wall After (0.5%) Drift



Figure F.39 W1-SR Wall After (0.75%) Drift





Figure F.40 W1-SR Wall After (1%) Drift



Figure F.41 W1-SR Wall After (1.25%) Drift



Figure F.42 W1-SR Wall After (1.5%) Drift



Figure F.43 W2-NR Wall After (0.1%) Drift





Figure F.44 W2-NR Wall After (0.2%) Drift



Figure F.45 W2-NR Wall After (0.25%) Drift



Figure F.46 W2-NR Wall After (0.5%) Drift



Figure F.47 W2-NR Wall After (0.75%) Drift



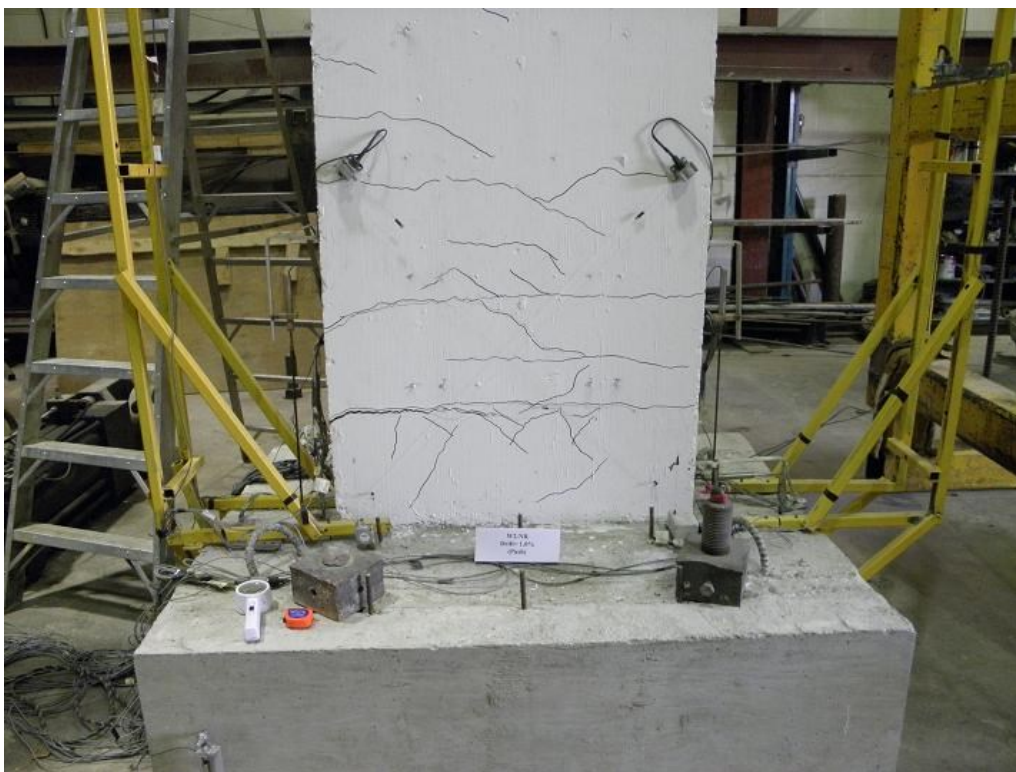


Figure F.48 W2-NR Wall After (1%) Drift

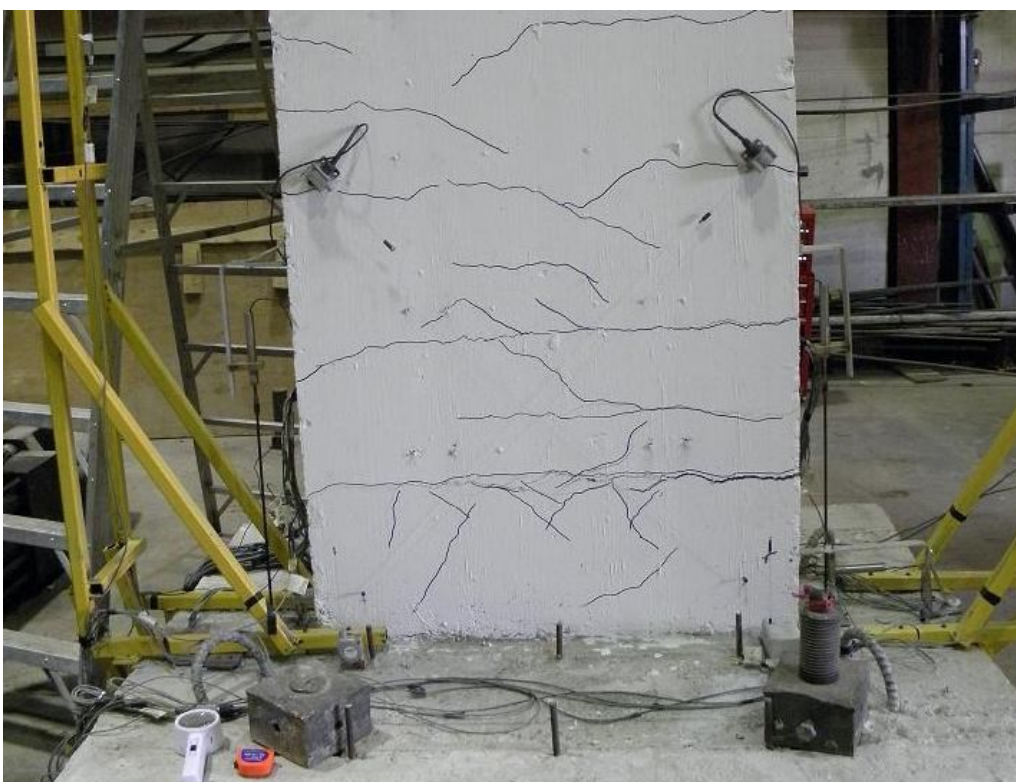


Figure F.49 W2-NR Wall After (1.25%) Drift



Figure F.50 W2-NR Wall After (1.5%) Drift

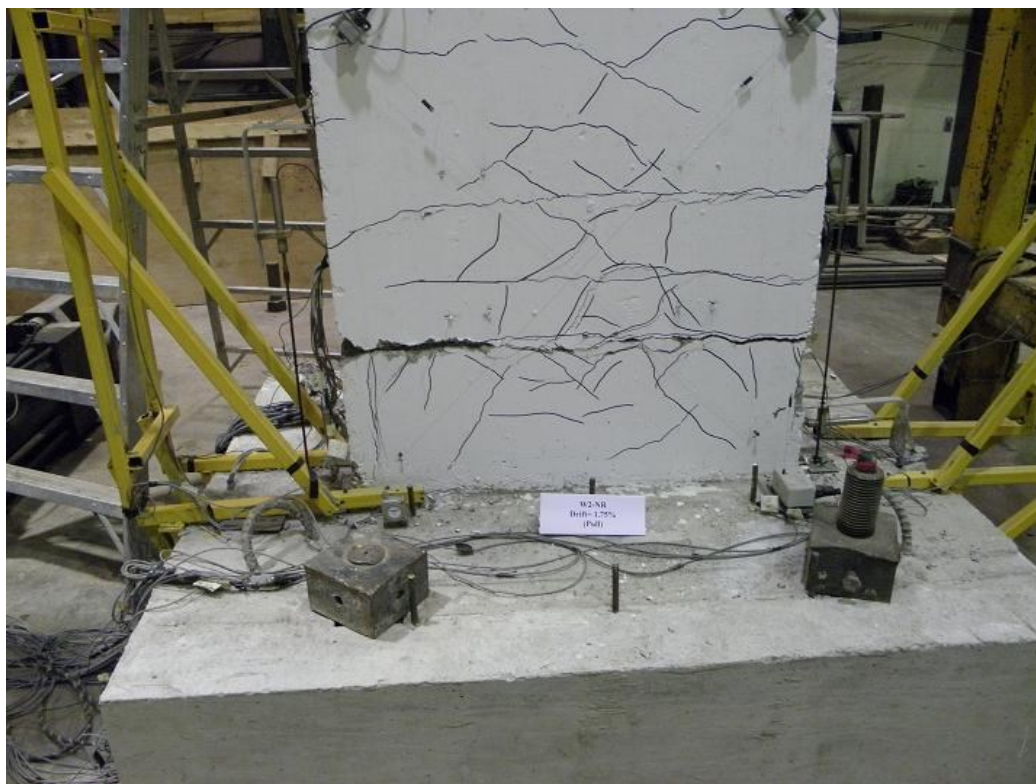


Figure F.51 W2-NR Wall After (1.75%) Drift





Figure F.52 W2-NR Wall After (2%) Drift



Figure F.53 W2-NR Wall After (2.25%) Drift

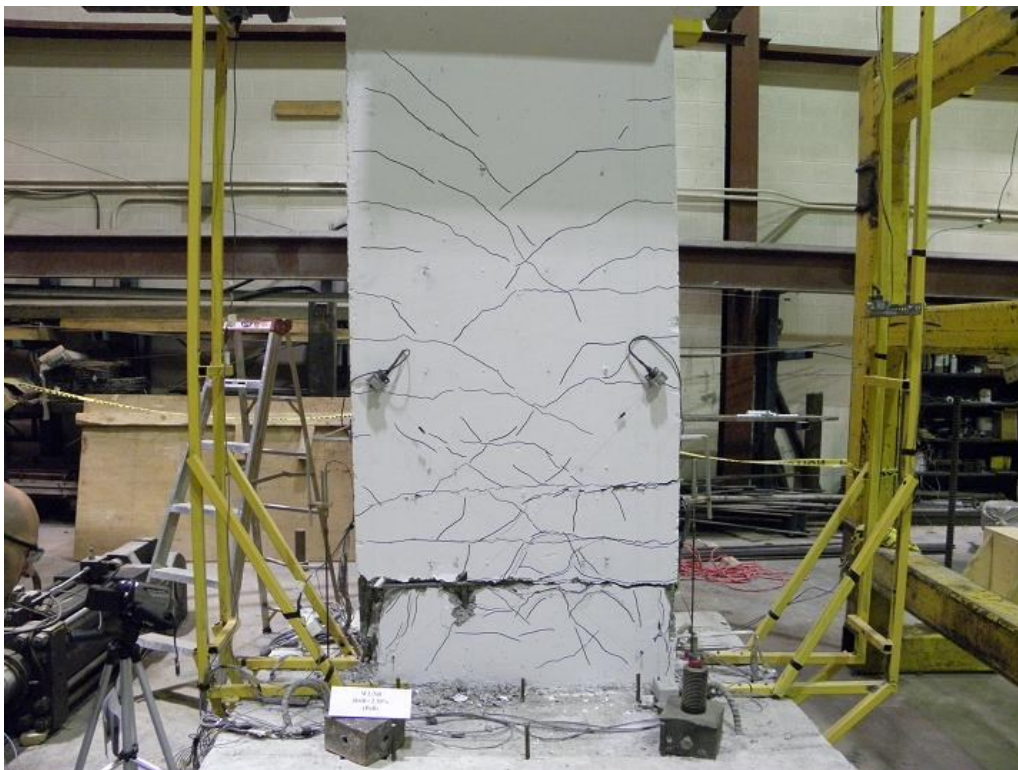


Figure F.54 W2-NR Wall After (2.5%) Drift

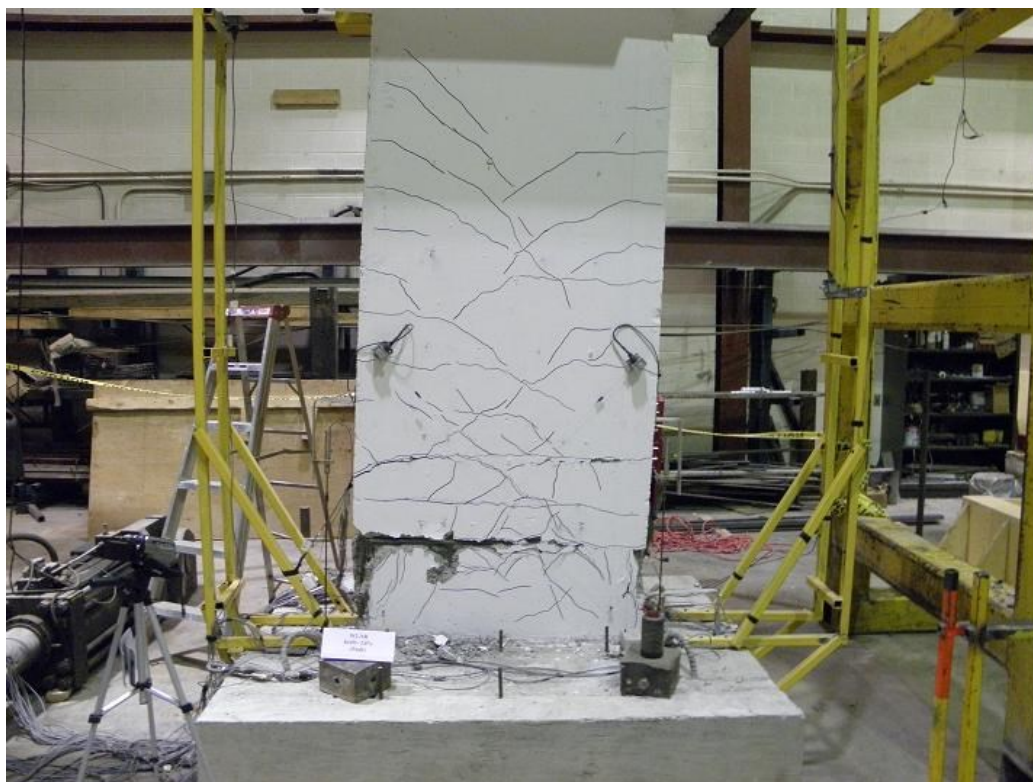


Figure F.55 W2-NR Wall After (3%) Drift





Figure F.56 W2-NR Wall After (3.5%) Drift

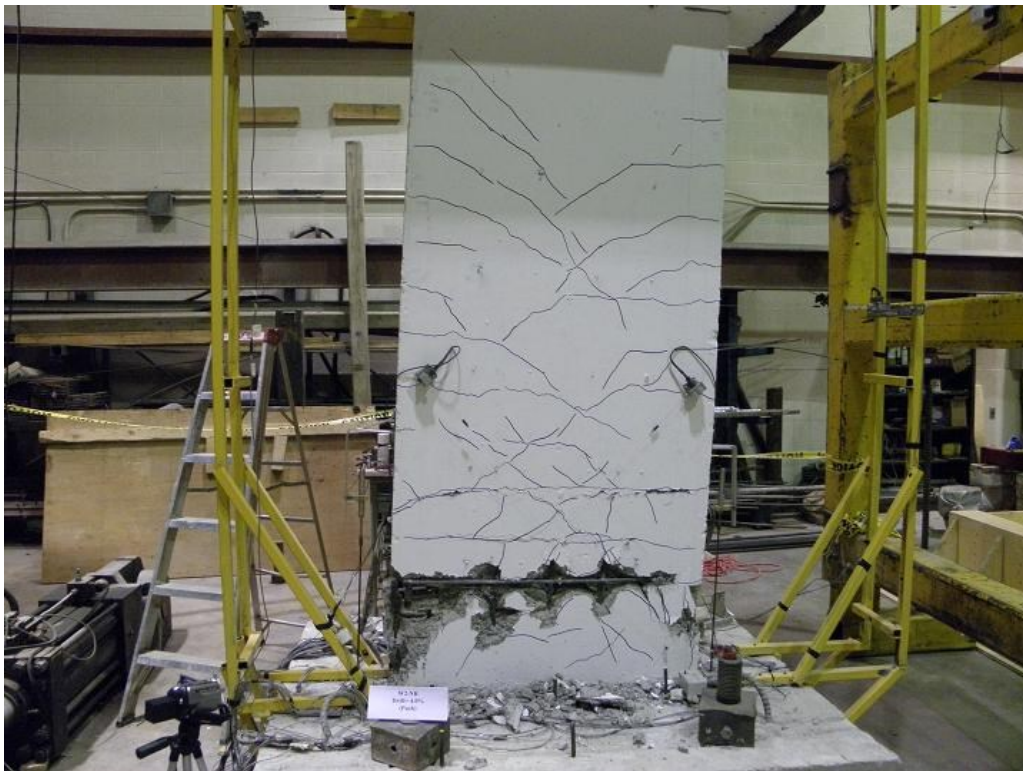


Figure F.57 W2-NR Wall After (4%) Drift

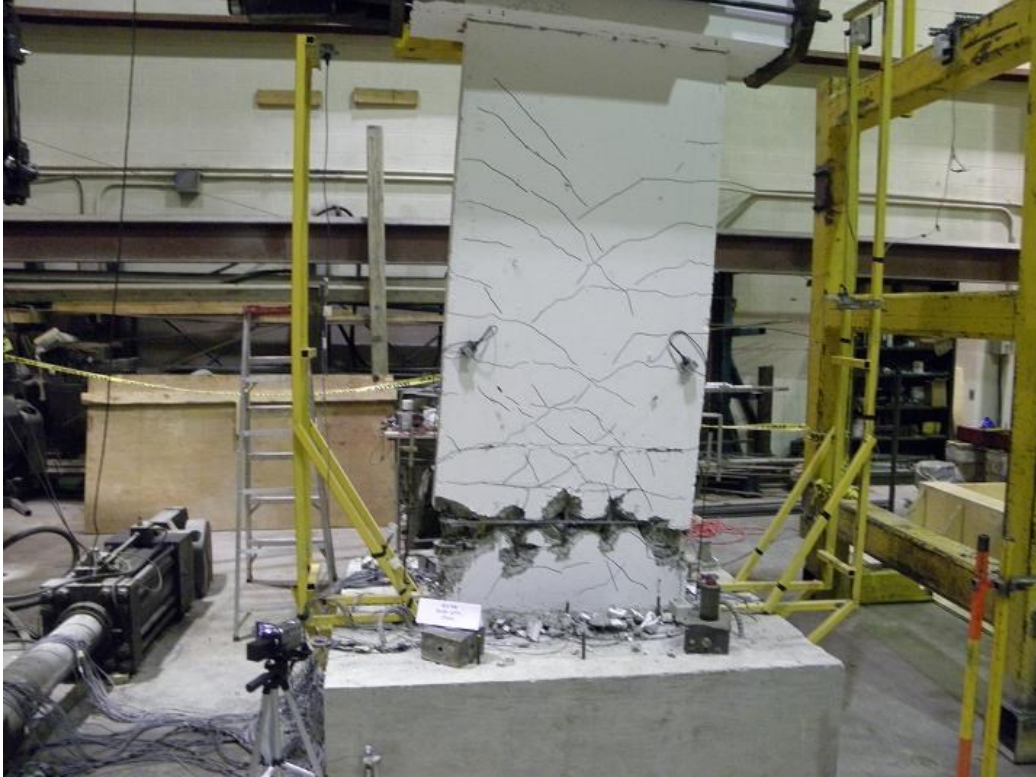


Figure F.58 W2-NR Wall After (4.5%) Drift