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PERFORMANCE OF REPAIRED SMA SHEAR WALL

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Abstract

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

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

Keywords: Repair; Shape Memory Alloys; Reinforced Concrete; Self-Consolidating Concrete; Shear Walls

1. Introduction

A significant amount of resources is required to repair structures after a major seismic event; the cost to repair may not be economical and reconstructing may prove to be more viable. Thus, in an attempt to reduce the cost of repairs, research is evolving in the area of novel materials and techniques that could minimize structural damage during seismic events. According to modern building codes, structures subjected to a major earthquake are expected to undergo large displacements to dissipate the energy of the earthquake, such that the structural materials yield and respond in the inelastic range. This requires large ductility capacity that could result in major structural and non-structural damage.

Shear walls are structural elements commonly used to resist the lateral forces imposed on a structure due to seismic activity and/or wind. During severe lateral loading, the principal reinforcement at the extremities of the wall or in the boundary elements yield to dissipate energy, which results in large permanent deformations of the overall structure. For structures exhibiting this type of performance, repairing is not, typically, economically and, therefore, demolition becomes the only alternative. To address this problem, research was recently conducted on hybrid-reinforced shear wall by Abdulridha [1], with shape memory alloy rods were used as the principal reinforcement in the boundary regions and traditional deformed reinforcement were used in the web section of the wall. This study demonstrated that SMA reinforced shear walls have the capacity to minimize permanent deformations, while provide similar lateral strength and drift capacities to traditional reinforced shear walls.

Shape Memory Alloys (SMAs) are a new group of alloy. Current applications of SMAs include medical and orthodontic equipment, fighter jets, shoes, and clothing. Shape memory alloys are smart materials that exhibit the unique property of recovering deformations after being subjected to extreme straining. This phenomenon results in self-centering, making SMAs attractive for seismic applications. In theory, a structure that self-centers may be serviceable after experiencing a major earthquake. In addition, the self-centering would reduce the extent of repairing.

The objective of this research is to study the performance of repaired hybrid SMA-deformed steel reinforced shear wall subjected to reverse cyclic loading. Furthermore, the focus is to assess whether a hybrid SMA-steel reinforced shear wall, which was previously damaged, can be restored without compromising on seismic performance.

2. Experimental Program

A slender, hybrid SMA-deformed steel reinforced shear wall was constructed and tested under reverse cyclic loading by Abdulridha [1]. The wall was built on a stiff foundation (1700 mm in length, 500 mm in height, and 1400 mm in width) to provide full fixity to the laboratory strong floor during testing. The wall also had a stiff top loading beam (1500 mm in length, 400 mm in height, and 400 mm in width) to distribute the lateral loading. The wall had height of 2200 mm, length of 1000 mm, and thickness of 150 mm. The original wall was named W2-NR, where W denotes wall, N denotes Nitinol (type of SMA rod used in the boundary regions) and R denotes reverse cyclic loading. For the current research, the wall was renamed RW2-NR, where the initial R denotes “repaired”

Wall W2-NR was reinforced with two layers of orthogonal steel reinforcement in the web region, spaced at 150 mm in both directions. The vertical reinforcement consisted of three pairs of 10M (11.3 mm-diameter, and 100 mm^2 of area) deformed bars and the horizontal reinforcement consisted of 15 pairs of 10M deformed bars. The resulting reinforcement ratios were 0.88% in both directions. In the plastic hinge region and within the boundary regions, 12.5 mm-diameter superelastic SMA bars were used as the principal reinforcement. The SMA bars were connected to deformed bars in the foundation and above the plastic region into the wall using mechanical couplers. The total length of the SMA bars was 1200 mm from center to center of the couplers. To prevent buckling of the vertical reinforcement in the boundary zones, 10M closed ties were spaced at 70 mm in the plastic hinge region and 100 mm above the plastic hinge region. To increase the resistance to shear sliding at the interface between the wall and the foundation, four pairs of 10M starter bars were also provided. Fig. 1 provides a layout of the reinforced in Wall W2-NR.

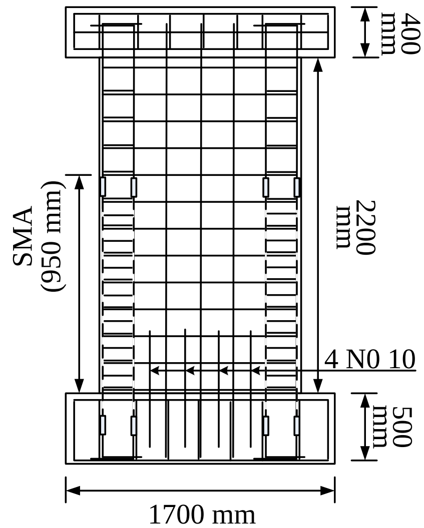
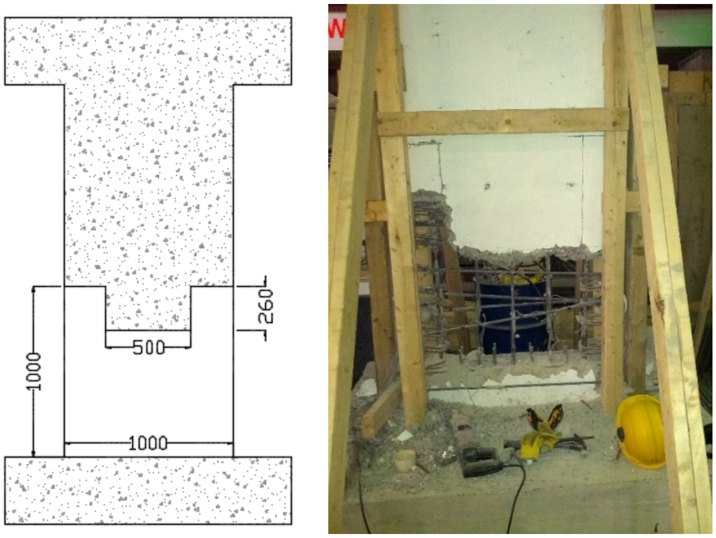


Fig. 1 – Reinforcement Layout of Wall W2-NR (modified form Abdulridha [1])

During the original testing, Wall W2-NR suffered extensive damage in the plastic hinge region. Few, but larger cracks surfaced on the wall. This was attributed to the smooth surface of the SMA bars. In addition, the wall experienced a major crack just above the termination of the starter bars, where most of the damage was experienced. This crack location controlled the behavior of the wall to failure. Spalling of the concrete in the boundary regions of the wall was also evident. Fig. 2 illustrates the state of Wall W2-NR prior to failure.

Fig. 2 – Condition of Wall W2-NR prior to failure (Abdulridha [1])

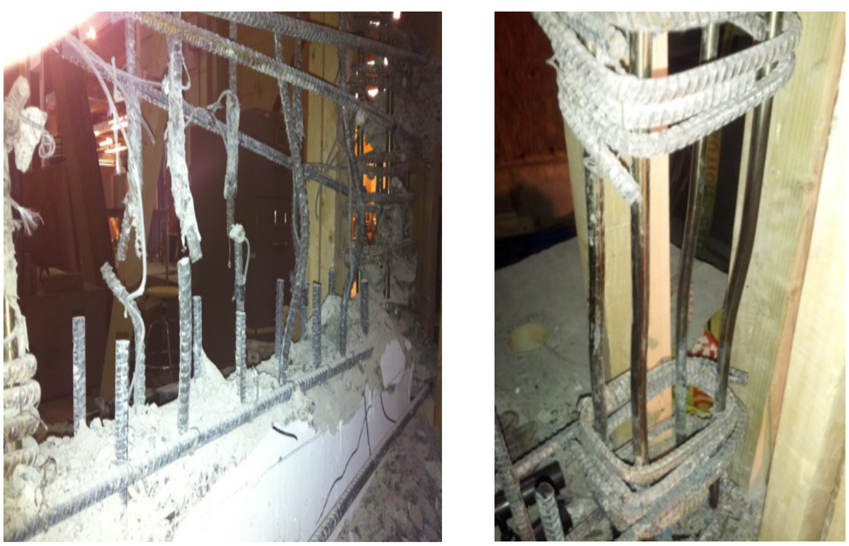
The first step in the repair strategy was to remove damaged concrete in the plastic hinge region. The concrete in the web was removed up to approximately 740 mm above the foundation level; while in the boundary region concrete was removed up to 1000 mm above the foundation, corresponding to the end of the couplers that were used to connect the SMA bars with the conventional steel. The latter was necessary to examine the condition of the SMA bars adjacent to the mechanical couplers. The extent of the concrete removed in Wall W2-NR is illustrated in Fig. 3.



(a) (b)

Fig. 3 – Extent of concrete removal: (a) Schematic drawing, and (b) Photo

After concrete was removed, the exposed reinforcing bars were examined. It became evident that three steel deformed vertical bars in the web section of the wall had fractured, while the others had experienced buckling. This damage was concentrated along the major crack adjacent to the termination of the starter bars. In addition, the SMA bars in the boundaries regions suffered moderate buckling along the same crack. No slip of the SMA bars was evident at the location of the couplers; however, one SMA bar had fractured adjacent to the coupler. The fracturing surfaced along the plane where the sharp-end screws of the coupling system pierced through the SMA bar. Fig. 4 illustrates the state of the reinforcement in Wall W2-NR after removal of the concrete.

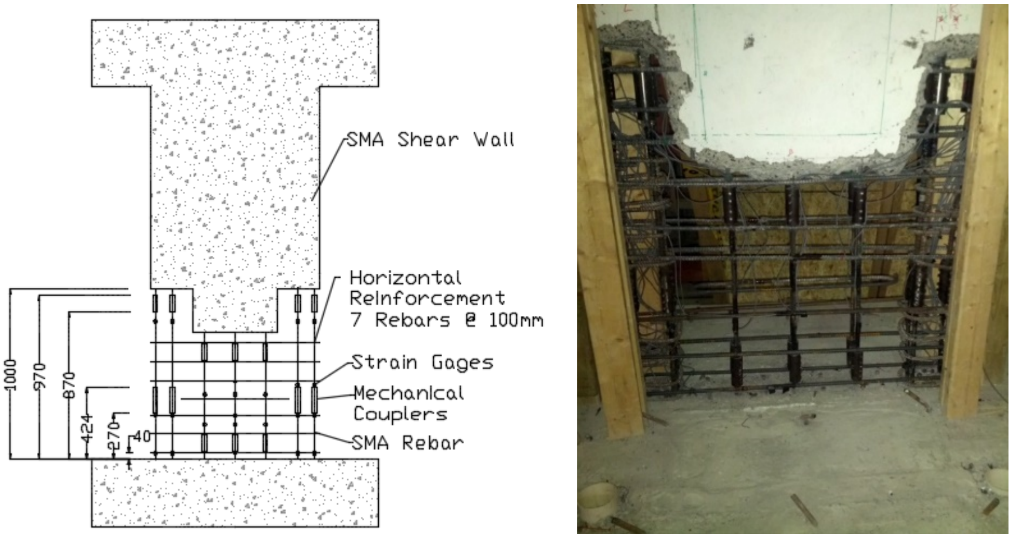


(a) (b)

Fig. 4 – Condition of reinforcement in W2-NR: (a) fractured and buckled web vertical bars, and (b) buckled SMA bars

The buckled and fractured reinforcing bars and starter bars in the web section of the wall were removed. The presence of the starter bars resulted in higher stiffness near the base of the wall. This led to the major crack that surfaced just above the starter bars where damage and failure of Wall W2-NR occurred.

The SMA bars were shortened from the original length of 950 mm to approximately 425 mm from the base of the wall. The shortened length led to an investigation of the effect on deformation recovery in hybrid walls due to the length of the SMA bars. This was based on the assumption that the response would be controlled along a single horizontal crack, and therefore, a shorter SMA length would be sufficient. In the web, only 90 mm of the vertical deformed reinforcement from the base and 115 mm from the top of the web from where the concrete was removed remained intact such that mechanical couplers could be installed; the remainder was removed. The total length of each new segment of deformed reinforcing bar placed in the web and connected to the couplers was 540 mm. Note that the same size of reinforcing bar (10M) was reused in the web. In the boundary zones, new segments of conventional deformed reinforcing steel (15M, 16 mm-diameter, 200 mm^2 area) replaced the length of the SMA bar that was removed. Mechanical couplers were used to connect the remaining SMA bars to the new segments of reinforcing steel. Thereafter, transverse reinforcement running along the width of the wall and anti-buckling ties in the boundary zones were replaced to their original spacing. Fig. 5 illustrates the replacement of reinforcement for Wall RW2-NR.

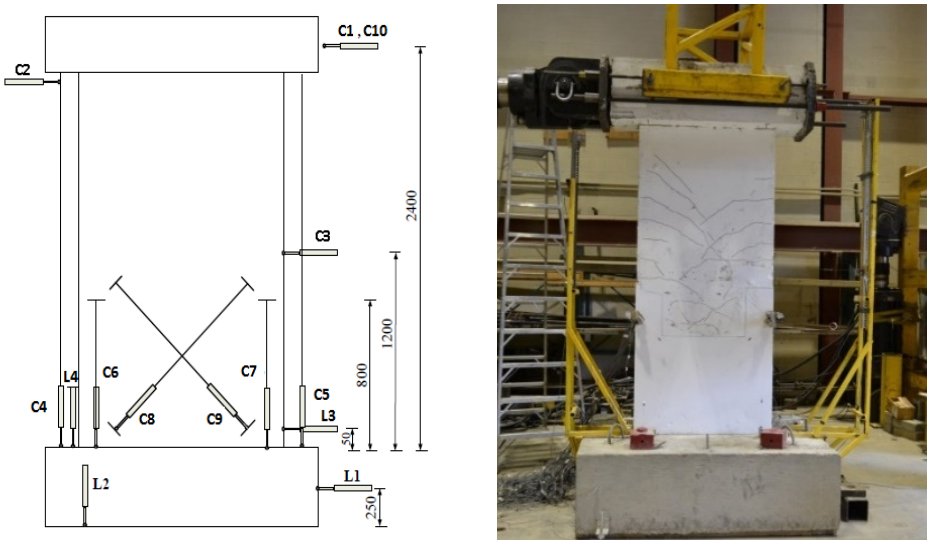


(a) (b)

Fig. 5 – Reinforcement layout in the repaired region of Wall RW2-NR: (a) Schematic drawing, and (b) Photo

The final step in the repair strategy involved casting high-strength Self-Consolidating Concrete (SCC) in the repair region. Self-consolidating concrete was selected given the tight formwork due to which the use of a vibrator to consolidate the concrete was not possible.

Linear Variable Differential Transducers (LVDTs) and Displacement Cable Transducers (DCTs) were placed at various locations on the wall to monitor displacements. The locations of the instrumentation and test setup are illustrated in Fig. 6. Reverse cycles of loading were applied to Wall RW2-NR at multiples of the yield displacement, following the same loading protocol used by Abdulridha [1] based on ATC- 24 [2].



(a) (b)

Fig. 6 – DCTs and LVDTs layout: (a) Schematic drawing (modified from Abdulridha [1]), and (b) Photo

3. Material Properties

The SMA bars that were removed from Wall W2-NR were tested under cyclic loading to measure the deformation recovery after the original test. The testing was performed using a 600 kN universal testing machine. A typical stress-strain response is provided in Fig. 7. The yield stress of the SMA bar was 305 MPa calculated using a 0.2% offset, and the modulus of elasticity was 27 GPa. At 7% strain, the stress in the SMA bar was 625 MPa. Beyond this strain level, the SMA bar was loaded to rupture. The ultimate (rupture) stress was 948 MPa at a strain of 16.4%. The stress-strain response demonstrated that the SMA bar retained the capacity to recovery large strains. Table 1 provides the properties of the SMA reinforcing bars tested as part of this research and the properties provided by Abdulridha [1] for Wall W2-NR.

Concrete cylinders for the repaired region were prepared on the day of casting and were subjected to the same curing conditions as Wall RW2-NR. The cylinders were tested on the day of the wall test and were subjected to a stress rate of 0.25 MPa/s in accordance with ASTM C39 [3]. The average concrete compressive strength of the repair concrete was 81 MPa.

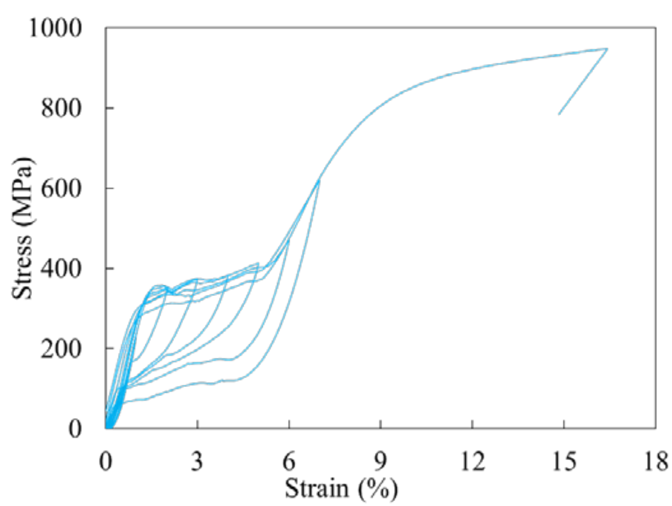


Fig. 7 – Stress-strain response of SMA

Table 1 – SMA Properties before and after the original test

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

4. Test Results

The lateral displacements recorded from DCT C1, located at the top of the wall, were used to generate the lateral load-lateral displacement hysteretic response for Wall RW2-NR, as illustrated in Fig. 8.

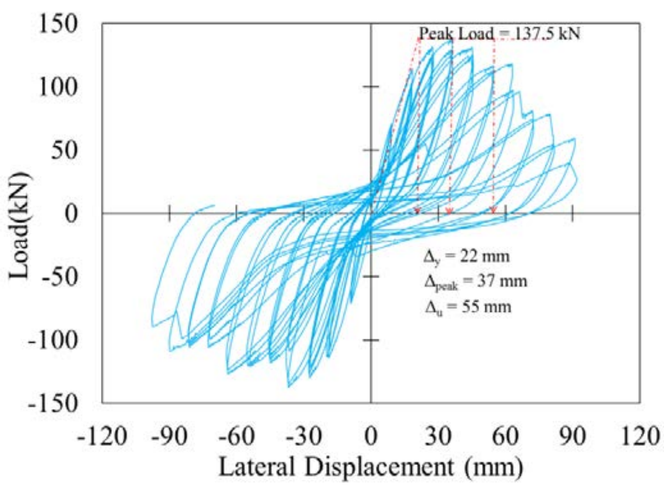


Fig. 8 – Lateral load-lateral displacement response of Wall RW2-NR

The response prior to yielding was linear, indicating elastic behaviour. After the first cycle, a reduction in strength was evident during the second and third repetitions of loading due to degradation in stiffness caused by reverse cyclic loading. Yielding of the wall 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 [4]). Yielding occurred at a displacement level of 22 mm, corresponding to a lateral load 119 kN. The peak lateral strength of 137 kN was recorded at a displacement of 37 mm. The ultimate displacement, corresponding to the displacement prior to the lateral load capacity of the wall dropping below 80% of the maximum load capacity (Park [4]) was 55 mm. The yield and ultimate laterals loads and displacements were based on the positive direction of loading.

In the post-peak regime, Wall RW2-NR experienced a significant degradation of strength of approximately 15% due to rupture of one SMA bar in the boundary zone while loading to at a displacement level of 63 mm. Rupturing of the SMA bar affected the deformation recovery by 20% with respect to the first repetition of loading at a displacement level of 63 mm. During the subsequent loading stage to 72 mm of displacement in the positive direction, an additional SMA bar ruptured in the same boundary zone. This second fracture resulted in a significant reduction in the recovery capacity of the wall. Beyond this displacement level and up to failure of the wall, rupturing of the other SMA bars was evident. Rupturing of the SMA bars occurred along the plane where the sharp-end screws of the coupling system pierced through the SMA bars. Therefore, Wall RW2-NR did not realize its full lateral drift capacity. Future testing will investigate other coupling alternatives. Spalling of the concrete was first evident at a drift level of 1.5% (36 mm) in boundary zone at the location of coupler connecting the SMA bars to the new conventional reinforcing bar. Two probable reasons for spalling at a low drift level were: (1) reduced concrete cover caused by the presence of the mechanical couplers;

and (2) bursting stresses that developed in front of the mechanical coupler due to bearing against the surrounding concrete.

At the end of testing, damage was concentrated along a crack located at 260 mm from the base of the wall. No new cracks surfaced in the region above the plastic hinge zone that remained intact after the original test. However, existing cracks in this region re-opened during testing. Fig. 9 illustrates the damage state of the repaired wall after testing.

Fig. 9 – Damage state of Wall RW2-NR at the end of testing

5. Discussion of Results

The lateral load-lateral displacement hysteretic response of Wall RW2-NR, before and after repair, is illustrated in Fig. 10.

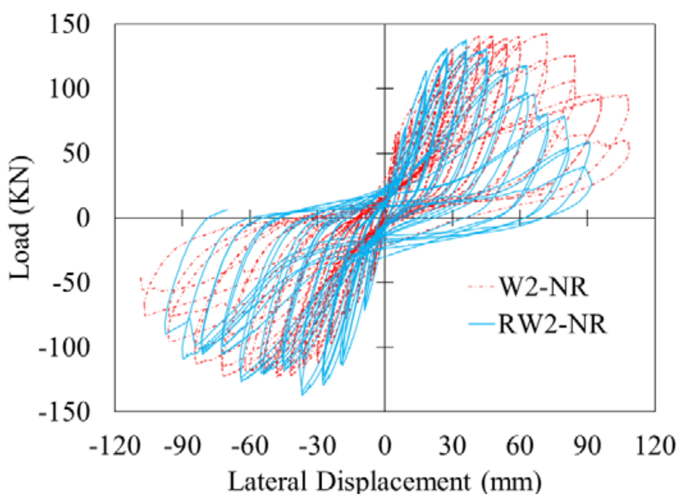


Fig. 10 – Lateral load-lateral displacement hysteretic response before and after repair

The pre-yielding behaviour of RW2-NR was very similar to W2-NR. After yielding, Wall W2-NR maintained a stable response up to the peak load at 3% drift (72 mm). However, RW2-NR rapidly reached its peak load after yielding at 1.5% drift (36 mm). The initial stiffness of Wall W2-NR was 4.24 kN/mm, while the repaired wall (RW2-NR) provided an initial stiffness of 7.75 kN/mm. The increased stiffness of RW2-NR was most likely due to the greater compressive strength of the repair concrete. The tests results, before and after repair, are summarized in Table 2.

Table 2 – Test results before and after repair

Wall	**Yield**			**Peak**		
	Load (kN)	**Displacement (mm)**	**Drift (%)**	**Load (KN)**	**Displacement (mm)**	**Drift (%)**
RW2-NR	119	22	0.92	137	37	1.54
W2-NR	112	26.4	1.1	142	72	3

Fig. 11 provides the recovery capacities of Walls W2-NR and RW2-NR. The original wall (W2-NR) maintained its recovery capacity within 90% to 100% throughout testing. From such recovery of deformation, it was evident that SMA bars were capable of overcoming the resistance from the conventional deformed steel in web section of the wall. The repaired SMA wall (RW2-NR) with shortened SMA bars also maintained significant recovery capacity of more than 80% up to 2% drift; afterwards there was a constant decrease in the recovery capacity. This was a result of rupturing of SMA bars. From the results, it can be established that re-centering capacity was not significantly affected by shortening of SMA bars in the repaired wall, and deficiencies after 2% were attributed to the couplers imposing a weakness in the SMA bars. Note, however, that a majority of buildings are designed to respond within the 2%-2.5% drift range.

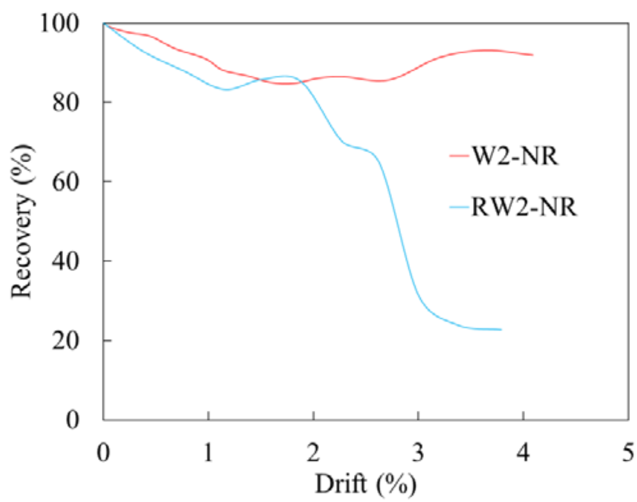
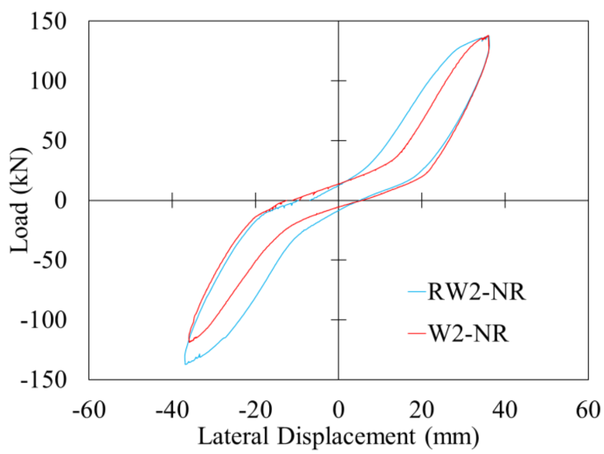
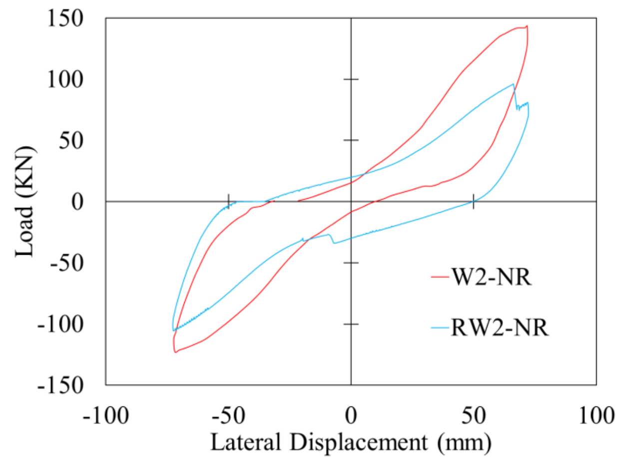


Fig. 11 – Recovery capacity-drift behavior

The lateral load-displacement responses at the peak load for RW2-NR and W2-NR corresponding to 37 mm and 72 mm, respectively, are illustrated in Fig. 12. At lateral displacement of 37 mm, the repaired wall (RW2-NR) dissipated 29% more energy than original wall (W2-NR). This difference can be attributed to greater stiffness in the repaired wall leading to a wider hysteretic cycle. At lateral displacement of 72 mm, W2-NR dissipated 7750 Nm of energy, while RW2-NR dissipated 2.5% more energy. However, as illustrated in Fig. 12 (b), W2-NR had higher strength than RW2-NR. The SMA reinforcement had ruptured in RW2-NR affecting the strength capacity, reverse transformation stress, and the capacity of the wall to recover displacements, resulting in less punching and increased energy dissipation.



(a)



(b)

Fig. 12 – Hysteretic response at: (a) 37mm, and (b) 72mm displacement

Fig. 13 provides the energy dissipated by the walls during the first repetition of loading at each drift level. Overall, RW2-NR dissipated more energy than W2-NR due to a reduction of pinching. The largest difference in energy dissipation in the walls was at 2% drift where RW2-NR dissipated 27% more energy than W2-NR. At 3% drift, both walls dissipated similar energy (approximately 7700 Nm). Beyond 3% drift, W2-NR dissipated more energy than RW2-NR due to a loss in lateral load capacity in RW2-NR after a number of the SMA bars had fractured.

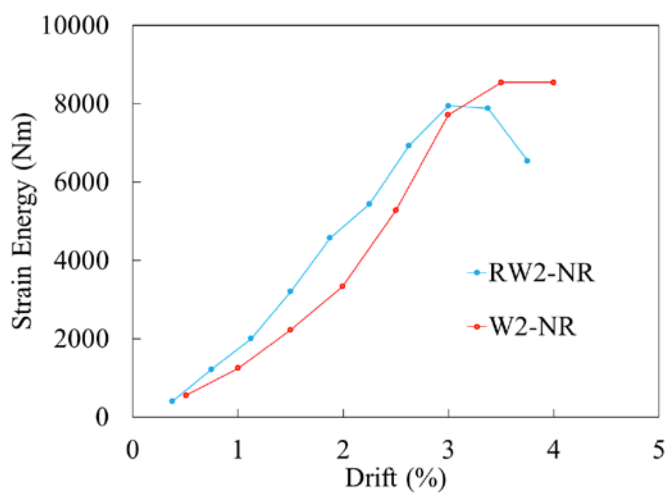


Fig. 13 – Energy dissipation-drift response

6. Conclusions

The performance of repaired slender reinforced concrete shear wall reinforced internally with superelastic SMAs in the boundary zones within the plastic hinge region was investigated and compared to the performance of the original wall, resulting in the following conclusions:

1. The SMA bars maintained sufficient strength and recovery capacities after testing the original wall.
2. The repaired SMA wall experienced higher stiffness at yield than the original wall. This increase was attributed to the significantly higher strength of the repair concrete used in the plastic hinge region.

3. The repaired SMA wall exhibited the ability to recover displacements up to the 2% drift level. Thereafter, the wall lost its recovery capacity due fracturing of the SMA bars.
4. The repaired wall dissipated more energy than the original wall due to a reduction in pinching.
5. Spalling of the concrete was evident in the repaired wall at a low drift level adjacent to the location of the mechanical couplers connecting the SMA bars to conventional deformed reinforcement. This was attributed to reduced concrete cover at the location of the couplers and bursting stresses that developed at the head of the coupler.
6. The repaired SMA wall failed due to rupturing of the SMA bars adjacent to the mechanical couplers; whereas, the original wall failed due to rupturing of the deformed vertical reinforcement in the web.

7. Acknowledgments

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8. References

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