SEISMIC RETROFIT OF CONCRETE SHEAR WALLS WITH TENSION-ONLY SMA BRACES

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Abstract

A retrofitting technique, using tension-only superelastic Shape Memory Alloy (SMA) braces, was developed to enhance the performance of seismic deficient squat concrete shear walls. The SMA braces were designed to improve re-centering (minimize residual deformations) and to control shear deformations, while improving strength, ductility and energy dissipation capacities. Two squat reinforced concrete walls: an original and a companion retrofitted wall were tested to investigate the benefits of the tension-only SMA braces on the seismic response. Retrofitting of the concrete wall was based on a bi-diagonal layout where two crossing tension-only SMA braces were connected to the base and top of the wall. The walls were tested to failure with incremental cyclic loading. The experimental results demonstrated that the tension-only SMA braces are capable of improving the seismic response of squat reinforced concrete walls. The retrofitted wall experienced higher strength, greater energy dissipation and reduced permanent deformation. Furthermore, the re-centering properties of the SMA contributed to the reduction of pinching of the hysteretic response due mainly to the clamping action of the SMA bracing while recovering its original length.

Keywords: Seismic Retrofit; Shape Memory Alloy; Tension-Only Braces; Reinforced Concrete; Squat Shear Walls

1. Introduction

Superelastic SMAs have the ability to recover deformations and return to its original shape after removal of stress. This unique property makes superelastic SMAs an attractive material for improving the seismic response of concrete structures. Specifically, superelastic Nickel-Titanium (NiTi) SMAs display stable hysteretic cycles that have the capacity to recover up to 98% of the deformations experienced by the material when subjected to strains up to 6% and provide equivalent viscous damping of up to 7% [1]. Superelastic SMAs can potentially address deficiencies of traditional retrofitting techniques such as external steel plates and external FRP sheets. Traditional strategies can improve strength and ductility capacities; however, the shear associated with the additional strength can trigger brittle failure of concrete before the structure reaches its ultimate flexural capacity. Retrofitting strategies with SMAs, on the other hand, can improve re-centering (minimize residual deformations) and control shear deformations, while improving strength, ductility and energy dissipation capacities. This results in reduction of damage sustained by the concrete and mitigation of shear-related failure.

Superelastic NiTi SMA rods and composite SMA bars have been experimentally investigated as an alternative seismic reinforcement for concrete columns [2], concrete beams [5], [6], concrete beam-column joints [7][8], and concrete shear walls [9]. The use of SMA contributed to reductions of permanent deformations and damage of concrete relative to companion concrete specimens reinforced with traditional reinforcing steel. These benefits have been further investigated through numerical analyses of concrete structural components that are either fully or partially reinforced with SMA: columns [10], [11], beam-column joints [12], and full-scale frames [13], [14], [15]. The numerical studies suggest that residual deformation, inter-storey drift, and damage of concrete are reduced by the presence of SMAs. The application of diagonal SMA bars as external reinforcement has improved the strength capacity of squat concrete shear walls [16]. However, the small deformations, typical of squat walls, and buckling of long diagonal bars in compression limited the benefit of the energy dissipation and re-centering of the SMA.

This study presents the development of a tension-only retrofitting bracing system incorporating superelastic NiTi Shape Memory Alloys (SMAs) to improve the behaviour of seismically deficient squat concrete walls. The SMA braces recover post-elastic deformations (up to 6% strain) and return to approximately its original length after the removal of imposed axial stresses. The SMA braces address shortcomings of external long SMA bars by optimizing the length of the SMA material and by devising a mechanism to permit tension-only forces, resulting in a hybrid structural retrofitting system with substantial energy dissipation and reduced residual deformations. The SMA tension-only braces were installed on a reinforced concrete squat shear wall that was internally reinforced with the minimum reinforcement required by the 1965 edition of the National Building Code of Canada [17]. The objective was to experimentally assess the performance of a smart SMA retrofitting technique to mitigate seismic damage on existing concrete shear walls that were designed and constructed before the 1970s.

2. Tension-Only Braces

SMA tension-only braces were designed and fabricated to retrofit 2000 mm x 2000 mm squat concrete shear walls. The SMA braces were 2540 mm-long and consisted of 635 mm-long superelastic NiTi SMA links that were coupled to 953 mm-long rigid steel elements (Fig. 1). The SMA links are resettable fuses that provide re-centering flag-shaped hysteretic cycles up to 6% strain and sustain all the damage of the braces, while the rigid steel elements sustain negligible deformations and remain in the elastic range. After being subject to seismic loading, the SMA link can return to approximately its original length and be reused for re-retrofitting. The dissipative characteristics of the SMA brace benefit the flexural response of reinforced concrete walls, while mitigating damage.

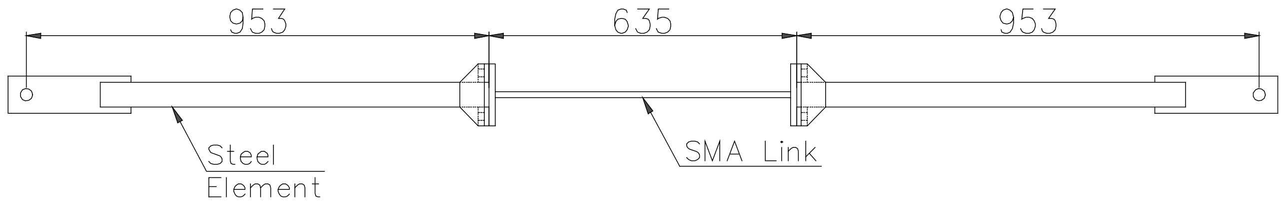


Fig. 1 – Tension-Only SMA brace

2.1 Design

The braces were designed to concentrate all damage (recoverable damage) in the SMA links. The length of the SMA links was selected to ensure forward transformation response of the SMA and such that the SMA could sustain the calculated maximum brace deformation of 35 mm based on the predicted ultimate capacity of the original wall. The brace elongation of 35 mm results in 5.5% strain within the SMA link. Achieving forward transformation permits the deformation recovery capacity of the brace, while not exceeding the superelastic strain limit of 6% permits full reusability of the brace. The rigid steel elements were designed to sustain a maximum working stresses of 25% of the yield stress (approximately 80 MPa), thus they behave within the elastic range and experience negligible deformations.

2.2 Fabrication

The SMA links were fabricated with 12.7 mm-diameter NiTi rods with cross-sectional area of 126 mm^2, 25 mm-thick connecting end plates, and reinforcing bar couplers (Fig. 2(a)). The steel rigid elements were fabricated with 50 mm x 50 mm x 4 mm hollow structural steel (HSS) section and 76 mm x 25 mm steel plates (Fig. 2(b)). To ensure tension-only action, one of the couplers of the link was welded to a connecting end plate (Fig. 2(c)), while the other was free to slide into the hollow steel section. Eighteen shear-off bolts, torqued to 50 ft-lb (68 N-m), ensured adequate grip of the couplers, preventing slip of the SMA rods.

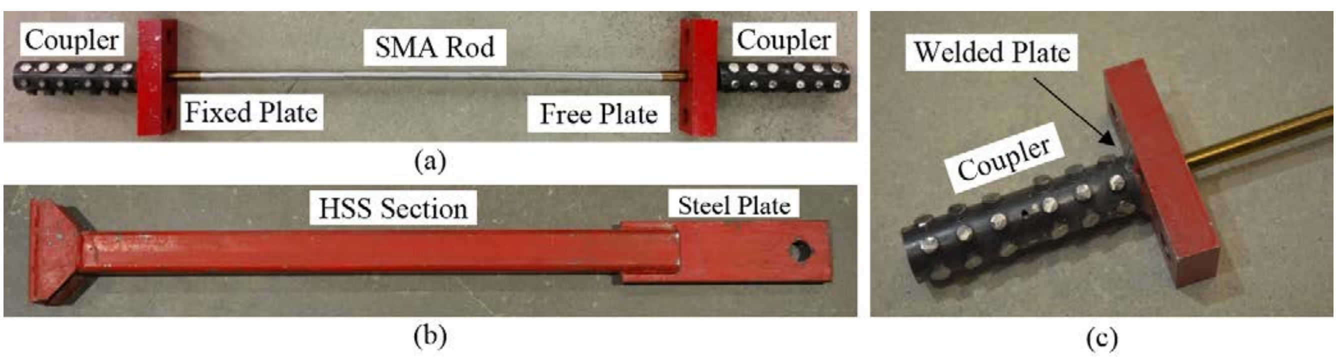


Fig. 2 – Retrofitting SMA brace: (a) SMA link, (b) rigid steel element, and (c) detail of fixed end of SMA link

Mechanical properties of the SMA rods were obtained from the stress-strain response of coupons as specified by the ASTM F2516-07 standard [18] (Fig. 3): upper plateau stress of 407 MPa at 3% strain; lower plateau stress of 186 MPa at 2.5% strain, ultimate stress (rupture) of 1054 MPa at 19.8% strain; and residual strain of 0.6% for a cycle to 6% strain.

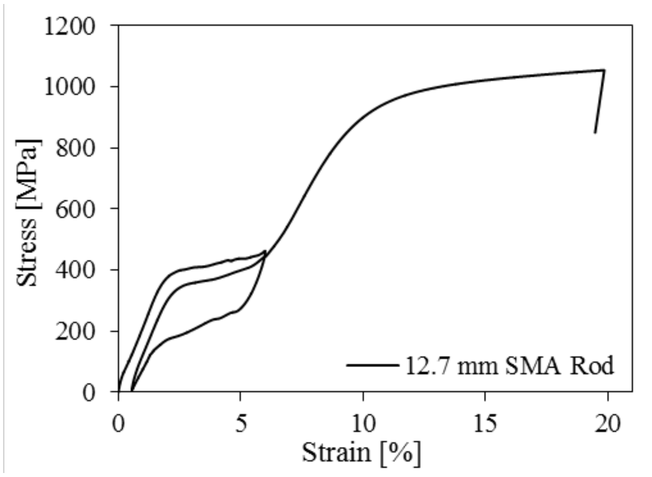


Fig. 3 – Stress-Strain response of SMA rods

2.3 Testing

Three superelastic SMA tension braces were tested under cyclic loading to validate the design, verify the performance of the brace components, and establish the actual cyclic response of the braces. Braces SMA1 and SMA2 were constructed with a single superelastic, 12.7 mm-diameter, SMA link with cross sectional area of 126 mm^2, while the Brace SMA3 used a half-length (317.5 mm) shape memory SMA link with reduced cross-sectional area of 110 mm^2. Brace SMA1 was tested to the target elongation of 35 mm (5.5% strain of the SMA link), based on the quasi-static cyclic testing protocol specified by FEMA 461 [19]. After testing Brace SMA1, the SMA link was removed and reused for testing Brace SMA2 to failure. The objective of retesting the SMA brace was to evaluate the reusability of the retrofitting system. Brace SMA3 was tested to failure to assess the effect of the reduced cross-sectional area on improving the deformation capabilities of the SMA links.

Testing of the braces was conducted with an axial loading system, where the braces were set up in the horizontal position and connected to a hydraulic actuator at one end and a concrete block at the other end (Fig. 4). Displacement transducers measured the elongations of the SMA braces and SMA links, and strain gauges measured localized straining of the SMA rods at mid-length of the SMA links.

Fig. 4 – Testing of SMA braces

2.4 Force-Elongation response

The responses of the SMA braces were governed by the flag-shaped behaviour of the SMA link as demonstrated by the force-elongation response (Fig. 5). Damage of the braces was concentrated in the links, while the steel components sustained marginal elastic deformations. Upper plateau strength, at 3% elongation of the SMA links, was observed at 45.5 kN and 47.2 kN (361 MPa and 375 MPa) for Braces SMA1 and SMA2, respectively. Braces SMA1 and SMA2 experienced lower plateau strength of 19.6 kN and 22.8 kN (156 MPa and 181 MPa), respectively, at 2.5% strain of the SMA links. The SMA braces exhibited excellent elongation recovery. The maximum residual elongation experienced by the braces was less than 2.5 mm (0.1% of the brace length). The measured strength of the braces at the cycle to 35 mm (1.4% strain in the brace and 5.5% strain in the link) was 52.1 kN and 54.3 kN (413 MPa and 431 MPa) for Braces SMA1 and SMA2, respectively. The SMA link of Brace SMA2 ruptured near one of the end couplers at a strength of 66.1 kN (525 MPa) and corresponding elongation of 41 mm.

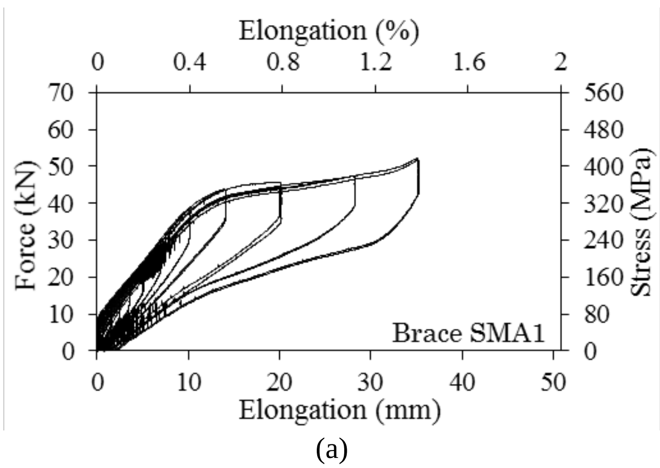


Fig. 5 – Response of SMA braces: (a) Brace SMA1, and (b) Brace SMA2

Testing of Brace SMA3 demonstrated that reshaping of the SMA rods delays the fracture of the SMA link. Rupture of the SMA link occurred at the transition of the cross-sectional area at 8.8% strain and corresponding strength of 75 kN (682 MPa). The improved elongation capacity provides a safety margin to avoid undesirable premature rupture of the SMA retrofitting bracing system.

3. Testing of Shear Walls

The experimental program was conducted on squat reinforced concrete shear wall specimens with aspect ratio of 1.0 and shear span of 0.83. Two sets of walls were designed: lightly reinforced shear walls (Walls SQ1) and shear walls with boundary columns (Walls SQ2). Results from testing the second set of walls, consisting of an original Wall SQ2 and a retrofitted Wall SQ2S, will be presented. Wall SQ2 represents a squat shear wall that is prone to combined diagonal tension and sliding failure.

3.1 Specimens

Wall SQ2 was designed with vertical reinforcement concentrated at the ends and to investigate normal concrete strength walls governed by diagonal tension failure. Wall SQ2 was 2000 mm long, 2000 mm high, and 125 mm thick. The reinforcement layout complied with the minimum requirements specified in the 1965 edition of the NBCC for shear walls (Fig. 6(a)). The web region of Wall SQ2 contained 5-ϕ9.5 mm vertical reinforcing bars (reinforcement ratio of 0.18%) and 7-ϕ9.5 mm horizontal reinforcing bars (reinforcement ratio of 0.20%).

Boundary columns (225 mm x 125 mm) were reinforced with 4-ϕ9.5 mm longitudinal (vertical) reinforcing bars (reinforcement ratio of 1.0%) and ϕ6.35-mm transverse (horizontal) ties spaced at 100 mm (reinforcement ratio of 0.51%). The concrete of Wall SQ2 had a compressive strength of 20.3 MPa on the day of testing.

The walls were constructed with heavily reinforced 500 mm-high foundation blocks, and 125 mm x 160 mm embedded loading beams, reinforced with 4-10M longitudinal steel bars and 6.35 mm-diameter transverse stirrups. The foundation blocks were designed to experience minimum cracking and remain elastic during the tests. The embedded beams, which coincided with the location of the loading actuator at 1660 mm from the base of the wall (shear span of 0.83), provided additional reinforcement along the wall to transfer the lateral forces imposed by the actuator without damaging the concrete. Vertical reinforcement of the walls was anchored with epoxy within 250 mm of existing foundation blocks.

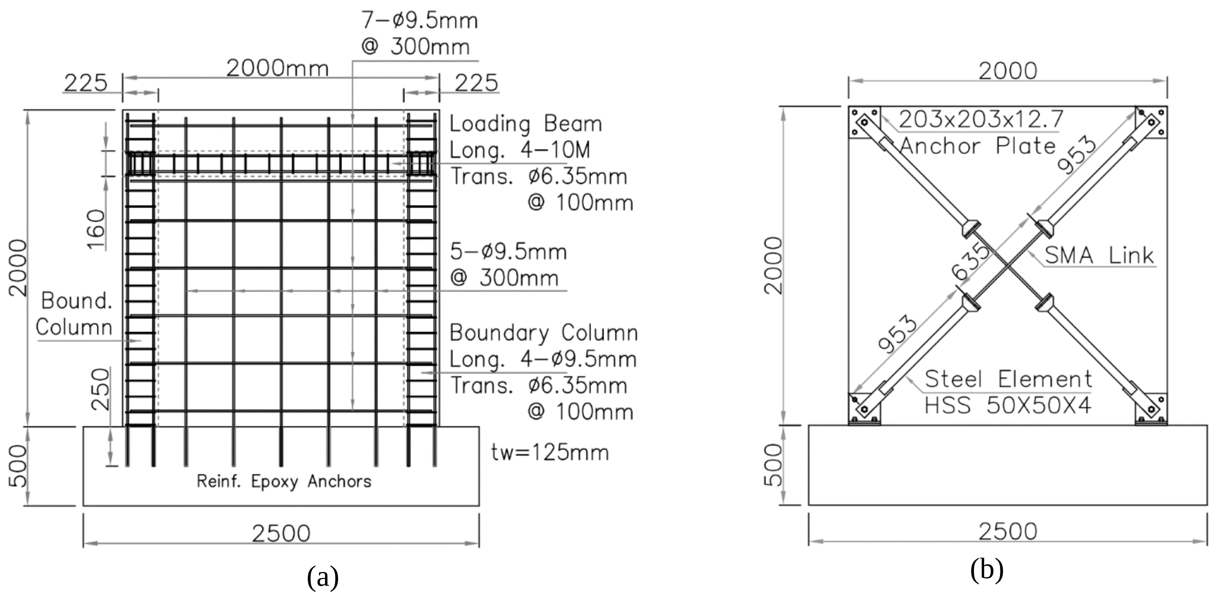


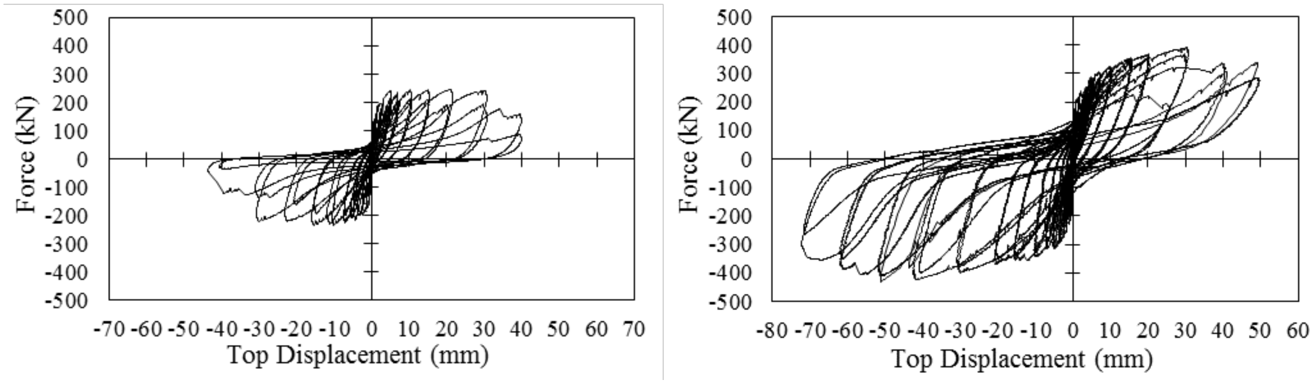
Fig. 6 – Shear wall specimens: (a) Wall SQ2, and (b) Wall SQ2S

Wall SQ2S was designed and constructed with the same dimensions, and reinforcement layout. However, the compressive strength of the concrete of 27.4 MPa on the day of testing was slightly greater. Wall SQ2S was retrofitted with SMA tension bi-diagonal braces that were externally attached to the lower and upper corners on each side of the walls (Fig. 6(b)). A total of four tension-only SMA braces were installed on the retrofitted wall.

Testing of the walls followed an incremental reverse loading protocol, based on recommendations of FEMA 461 [19], where two repetitions of reverse cyclic lateral displacements were imposed on the loading beam. The protocol started with small top displacements of 0.3 mm and were gradually increased to reach the target displacement of 40 mm. Thereafter, increments of 10 mm were imposed until failure. To monitor the response of the shear walls, including the internal reinforcing steel and the retrofitting SMA braces, displacement transducers and strain gauges were placed at different locations. The results presented herein are based on the main displacements measured at the top of the walls and the elongations of the SMA braces.

3.2 Test Results

Fig. 7 illustrates the force-top displacement response of the original and retrofitted walls, and Fig. 8 shows the observed damage of the walls at ultimate, prior to rupture of the vertical reinforcing steel. The cyclic responses and the progression of damage of both the original and retrofitted walls will be described below.



(a) (b)

Fig. 7 – Cyclic response of walls: (a) original Wall SQ2, and (b) retrofitted Wall SQ2S

(a) (b)

Fig. 8 – Cracking of walls: (a) original Wall SQ2 (at 30 mm), and (b) retrofitted Wall SQ2S (at 40 mm)

3.2.1 Response of Original Wall SQ2

Wall SQ2 experienced satisfactory ductility as reflected by the force-displacement response (Fig. 7(a)), but shear related damage, in the form of diagonal cracking and sliding, promoted strength and stiffness degradation that resulted in a highly pinched response with limited energy dissipation capacity. First diagonal cracking at the lower corners of the wall was observed at +0.3 mm and corresponding strength of 115 kN. With increase in lateral displacement, diagonal cracking propagated toward the loading beam. The wall reached its peak strength at 5 mm displacement in the positive direction and at 7 mm displacement in the negative direction. The corresponding peak strengths were 243 kN and 225 kN, respectively. As indicated by the response, the vertical reinforcement in the boundary columns fully yielded at 7 mm displacement. After yielding, the majority of diagonal cracks formed and a major horizontal crack, where the yielding of the vertical reinforcement concentrated, surfaced at the base of the wall. Widespread cracking was followed by spalling, grinding, and signs of crushing at the wall toes, resulting in degradation of strength and stiffness, specifically during the second repetitions of loading. The yield strength was sustained to ultimate displacement of 30 mm. Residual displacement of Wall SQ2 at ultimate was 23 mm. The horizontal crack at the base had a maximum width of 20 mm, resulting in localized straining that ruptured six vertical bars (four in the boundary columns and two in the web). Rupture of the reinforcement initiated during the second repetition to 30 mm. Deterioration of the wall at the base occurred simultaneously with the appearance and propagation of new and existing diagonal cracks. The diagonal cracks, however, were controlled by the reinforcement and did not deteriorate to the same degree as the horizontal crack at the base. The largest diagonal crack initiated at the base of the wall near the left boundary column and propagated toward the half the height of the wall at the right end (Fig. 8(a)). This crack had a maximum width of 7 mm at ultimate. Other diagonal cracks were smaller than 2 mm. Due to the intensive damage of the concrete and rupture of the internal reinforcement, the strength of the wall in both directions decreased to an average of 53 kN at 40 mm.

3.2.2 Response of Retrofitted Wall SQ2S

Wall SQ2S exhibited high ductility and significant energy dissipation, as indicated by the hysteretic response in (Fig. 7(b)). The wall experienced minimum strength, shear, and stiffness degradation that resulted in satisfactory flexural behaviour with moderate pinching, specifically during the first 30 mm of displacement. Pinching of the response was mostly due to the re-centering effect of the retrofitting SMA braces and not due to degradation of concrete, which was controlled by the retrofitting system. First cracking of the wall occurred during the cycle to 0.6 mm. A small horizontal crack at the base of the wall surfaced at approximately one-third of the length while imposing positive displacements and a diagonal crack developed from the quarter-height at the right end of the wall to mid-length at the base while imposing negative displacements. The corresponding strength at first cracking was 170 kN (positive direction). Diagonal cracks, along with horizontal flexural cracks in the boundary columns, propagated uniformly from the base of the wall up to the loading beam. A significant number of the cracks appeared during the first stage of loading until yielding. The force-displacement response indicates yielding of Wall SQ2S at 7 mm displacement. The corresponding strength at yielding was 298 kN in the positive direction and 302 kN in the negative direction. After yielding, the strength was gradually increased to the peak strength of 391 kN at +30 mm and 425 kN at -40 mm due to additional strength provided by the SMA braces. At +30 mm, the horizontal crack at the base opened to approximately 7 mm, while the majority of diagonal cracks had maximum widths between 2.5 mm and 6 mm. At this displacement level, the residual displacement was 13 mm. Damage of the right boundary column affected the post-peak strength and stiffness of the wall, specifically in the positive direction. Strength and stiffness degradation was observed while cycling the wall to 40 mm displacement. Furthermore, first rupture of the vertical reinforcement occurred during the second repetition to 40 mm. The strength decreased to 285 kN at +50 mm and to 265 kN at -70 mm (average of 275 kN). The corresponding residual displacements at +50 mm and -70 mm were 18 mm and 50 mm, respectively.

Damage of Wall SQ2S did not affect the performance of the retrofitting system as demonstrated by the force-elongation response of one of the SMA braces (Fig. 9). The SMA braces exerted tension-only forces that attempted to pull the wall back to its original position (zero lateral displacement). The forces developed in the braces, however, could not completely restore the wall due to yielding of the internal reinforcement as indicated by the residual elongations in Fig. 9. The SMA braces experienced negligible permanent damage. After removal from the wall, the two braces that were pulled to +50 mm wall displacement (response in Fig. 9) did not experience any permanent change in length. The other two braces, which were pulled to -70 mm wall displacement, experienced an increase in length of 2 mm. The maximum strain experienced by the SMA links at +50 mm was 5.5%, while the maximum strain experienced by the SMA links at -70 mm was 6.6%. Note that although the wall exceeded the full recovery capacity of the SMA braces at -70 mm, the permanent deformations were marginal, and the SMA braces and SMA links remained usable.

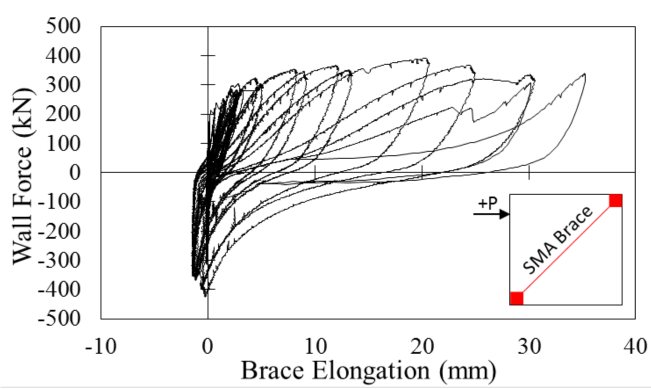


Fig. 9 – Wall force-elongation response of SMA brace pulled in the positive direction

3.3 Discussion

Comparison of the observed behaviour and the hysteretic response of Walls SQ2 and SQ2S indicates that retrofitting with SMA tension braces improved the strength, ductility, and residual displacement of the original Wall SQ2. As a result, the retrofitted wall provided hysteretic cycles with greater energy dissipation.

The cyclic response of Wall SQ2S was more stable than that of the original wall, specifically for the first 30 mm of displacement. The original wall lost substantial strength and stiffness during second repetitions after yielding of the vertical reinforcement. Conversely, the retrofitted wall maintained a great percentage of its yield strength and stiffness. Furthermore, the original wall sustained its yield strength to ultimate, while the retrofitted wall increased its strength from yielding to its peak capacity by approximately 30%. The retrofitted wall sustained its strength to 40 mm in the positive direction and 60 mm in the negative directions (2% and 3% drift, respectively), while the original wall sustained its strength to 30 mm (1.5% drift). This represents up to 100% increase of the displacement capacity. The average residual strength of 275 kN at failure (last imposed cycle) of the retrofitted wall was substantially greater than the average residual strength of 53 kN of the original wall.

The SMA braces reduced the residual displacement at zero load and the pinching effect resulting from strength and shear degradation of concrete due to cyclic loading. At 30 mm (ultimate displacement of Wall SQ2) the residual displacement of Wall SQ2 was 23 mm, while the residual displacement of the retrofitted Wall SQ2S was 13 mm (reduction of 43%). Damage of the retrofitted wall at 40 mm, mainly due to yielding and buckling of vertical reinforcement in the boundary columns, affected the stiffness and therefore the recovery capabilities of the retrofitted wall. Beyond 40 mm, Wall SQ2S experienced good recovery in the positive direction, but limited recovery in the negative direction. The integrity of the retrofitting braces was not compromised at large displacement demands, which strained the SMA beyond the design full recovery limit of 6%. The SMA links of two braces sustained up to 6.6% strain when the retrofitted wall was subjected to -70 mm. This resulted in a marginal permanent deformation of 2 mm, corresponding to an increase of 0.3% of the original length of the link.

4. Conclusions

A tension-only brace, consisting of a superelastic NiTi SMA link and rigid steel elements, was developed to improve the seismic response of squat reinforced concrete shear walls. The SMA link was designed to sustain the majority of the damage up to the full recovery limit of 6%, while the steel components were designed to sustain negligible elastic deformations. Three SMA braces were fabricated and tested under cyclic axial loading. The SMA braces resisted up to 75 kN before rupture and sustained straining of up to 8.8%, which exceeded the full recovery limit of the SMA. The SMA braces exhibited flag-shaped responses with permanent elongation of less than 2.5 mm (0.1% of the length of the brace) demonstrating the recovery capability. The small permanent elongations permitted reuse of one of the SMA links for retesting, which demonstrated the reusability of the SMA braces.

Testing of two squat shear walls with aspect ratio of 1.0 (original and retrofitted walls) demonstrated the application of the SMA brace for retrofitting. The SMA retrofitting system incorporated bi-diagonal braces that were connected to the lower and upper corners of the wall. The braces were intended to improve important features of seismic response, such as strength, ductility, displacement recovery, and energy dissipation. In general, the retrofitting system performed well and improved the hysteretic response of the shear wall. Retrofitting with the SMA braces increased the strength by 75% and the displacement capacity by up to 100%. Furthermore, the SMA braces reduced the residual displacement by 35%, specifically in the positive direction. Heavy damage of the boundary columns affected the stiffness and the response in the positive direction as well as the displacement recovery. Additional retrofit interventions aiming to improve the stability of the boundary columns may improve the response for large displacement demands. Although diagonal tension cracks developed in the retrofitted wall (similar to those observed in the original wall), the clamping effect of the retrofitting braces significantly reduced the post-peak strength and stiffness degradation observed in the original wall, specifically for the first 30 mm of displacement. As a consequence, the associated pinching of the response was reduced. Improvements of the response resulted in a retrofitted wall with improved recovery and energy dissipation. After testing the retrofitted wall, the SMA links and the SMA braces remained reusable for further retrofitting.

5. Acknowledgements

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