Towards Design of Shear Walls Retrofitted with Shape Memory Alloys

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SUMMARY:
A retrofitting procedure using shape memory alloys (SMA) is presented for squat reinforced concrete shear walls designed and constructed without seismic detailing according to pre-1970s standards. The SMA retrofit consists of external rigid steel elements that are coupled with SMA rods whose length is optimized to ensure maximum re-centering. A prototype structure is assessed with the 2010 National Building Code of Canada, while the corresponding retrofitted structure is assessed with the capacity spectrum method as described in the ATC-40 and FEMA-440 documents. The SMA retrofit is simulated with the finite element program VecTor2 and compared with a traditional retrofit that utilizes externally bonded steel plates. Results from the seismic assessment indicate that retrofitting techniques with SMAs have the potential to improve the flexural response and energy dissipation of shear walls, while reducing damage to the concrete. The wall retrofitted with SMA sustained less concrete damage and less permanent deformation than the companion wall retrofitted with steel plates.

Keywords: Seismic Retrofit, Shape Memory Alloys, Reinforced Concrete, Squat Shear Walls

1. INTRODUCTION

Shape memory alloys (SMAs) have attracted the attention of the scientific community owing to the ability of the material to recover its initial shape. Specifically for retrofitting reinforced concrete (RC) shear walls, external SMAs possess characteristics that can potentially address deficiencies of traditional retrofitting strategies, such as external steel plates. Traditional strategies can improve strength and ductility capacities; however, the shear associated with the additional strength can trigger brittle failure of the shear wall before reaching its ultimate flexural capacity. Retrofitting strategies with SMAs, on the other hand, can improve re-centering capabilities (minimize residual deformations) and control shear deformations, while improving strength, ductility and energy dissipation. This results in a reduction of damage sustained by the concrete and a control of brittle shear failure.

Retrofitting techniques in the form of external steel plates and bonding of fibre reinforced polymers (FRP) have previously been investigated to improve the seismic response of squat RC shear walls. Tagdhi et al. (2000) investigated a retrofitting scheme with bi-diagonal and vertical steel plates bolted to squat shear walls with aspect ratios of 1.0. The cyclic response of the retrofitted walls, compared to a companion control wall, demonstrated improvement in strength, ductility and energy dissipation capacities. This flexural improvement was accompanied by extensive damage throughout the wall, as well as permanent deformation resulting from the yielding of the internal reinforcing steel and external retrofitting steel. Seismic retrofit of squat shear walls with FRP has been investigated by Lombard et al. (2000) and Hsiao et al. (2008) among others. Lombard et al. conducted reverse cyclic tests on RC shear walls retrofitted with vertically oriented carbon FRP sheets bonded on both sides of the wall to improve flexural response. Hsiao et al. investigated the use of externally bonded diagonal FRP strips to improve the cyclic response of squat shear walls with barbell cross section and aspect ratios of 0.57 and 0.80. In general, the retrofitted walls responded with marginal enhancement in strength, ductility and energy dissipation capacities. Although dissimilar in shape and properties, the shear walls retrofitted with FRP by Lombard et al. and Hsiao et al. were highly damaged from the base to the top of the wall and responded with significant residual deformation.

The only attempt to date, known by the authors, to retrofit squat shear walls with SMA has been conducted by Liao et al. (2006). The effectiveness of SMA was assessed by connecting two diagonal SMA bars from the base to the top of the barbell-shaped walls. The SMA bars were inclined at an angle of 27 degrees, and the walls had an aspect ratio of 0.5. The retrofitting scheme was successful in increasing strength; however, energy dissipation and re-centering capabilities of SMA were not completely utilized mostly due to the length of the bars. Significant damage and shear degradation observed in the post-peak response of the retrofitted walls was similar to that of the non-retrofitted walls indicating that the SMA retrofit scheme fell short in improving the cyclic response and energy dissipation of the shear walls. These shortcomings can be addressed by optimizing the length of the SMA in the diagonal bracing to obtain a hybrid system that promotes flag-shaped cyclic response with substantial energy dissipation and marginal residual deformation (Figure 1-1c)).

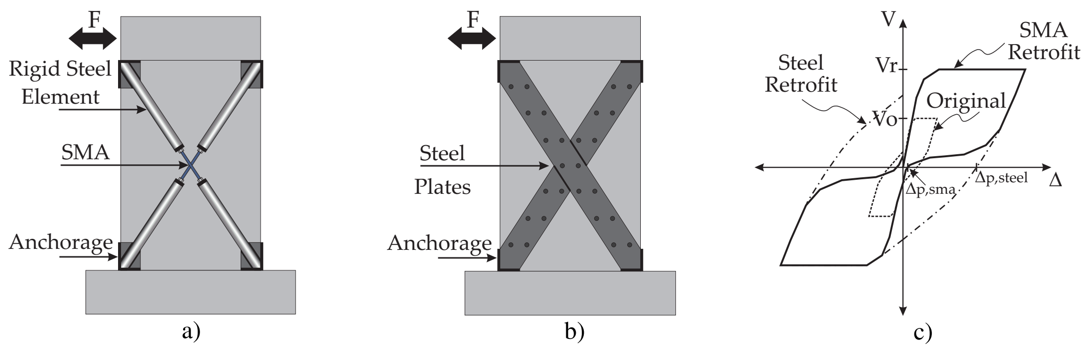

Figure 1-1. External retrofitting systems for shear walls: a) SMA diagonal bracing; b) Steel plate; c) Ideal response of retrofitting systems.

A proposed retrofitting system with optimized SMA, as illustrated in Figure 1-1a), is assessed in this study to improve the seismic response of RC squat shear walls that were designed and constructed prior to the 1970s. The assessed SMA retrofitting system (Figure 1-1a)) is compared with a traditional retrofit system with steel plates (Figure 1-1b)). The retrofitting procedure follows a performance-based design philosophy where capacity responses of the retrofitted walls determined from pushover curves are compared with corresponding reduced elastic demand, which are developed through the provisions of ATC-40 (ATC, 1996) and FEMA 440 (ATC, 2005). This approach was selected over other procedures, such as the coefficient method adopted by ASCE-41 (ASCE, 2007). The graphical representation of the capacity spectrum method in ATC-40 and FEMA-440 permits a more comprehensive estimate of the energy dissipation capacity and, thus, a more rational assessment of the seismic performance of structures with atypical hysteretic characteristics. Seismic requirements for the original (non-retrofitted) and retrofitted walls were evaluated with the 1965 and 2010 versions of the National Building Code of Canada (NBCC) (NRC, 1965; NRC, 2010), respectively. The retrofitting systems were designed in accordance with the required strength and ductility to match the calculated reduced elastic demand diagram. The pushover curves for both SMA and steel plate retrofitting systems were established from the backbone of the calculated cyclic responses, and the reduced elastic demand diagrams were calculated in accordance with the ATC-40 and FEMA-440 provisions using simple earthquake engineering concepts, such as equivalent energy dissipation. Both SMA and steel plate retrofit techniques were numerically simulated with the non-linear, two-dimensional finite element program VecTor2, which is capable of predicting behavioural features such as strength, ductility, residual deformation, energy dissipation, and damage.

2. SEISMIC REQUIREMENTS

2.1 Design of the original wall

An assumed two-storey prototype office building located in Vancouver, British Columbia, was selected to investigate SMA seismic retrofit of shear walls subjected to significant seismic demand. The analysis and design of the building followed the 1965 version of the NBCC (NRC, 1965) in which no seismic detailing was prescribed for shear walls. The lateral force resisting system consisted of 300 mm thick reinforced concrete shear walls in both orthogonal directions, while the vertical force resisting system consisted of 400 mm x 400 mm reinforced concrete columns. The floor system consisted of 200 mm thick flat plates. Dimensions of the structural elements are illustrated in Figure 2-1. The prototype building conforms to FEMA Type C2 concrete shear wall building, and is very similar to the Type C2 FEMA model building seismically assessed in FEMA 440.

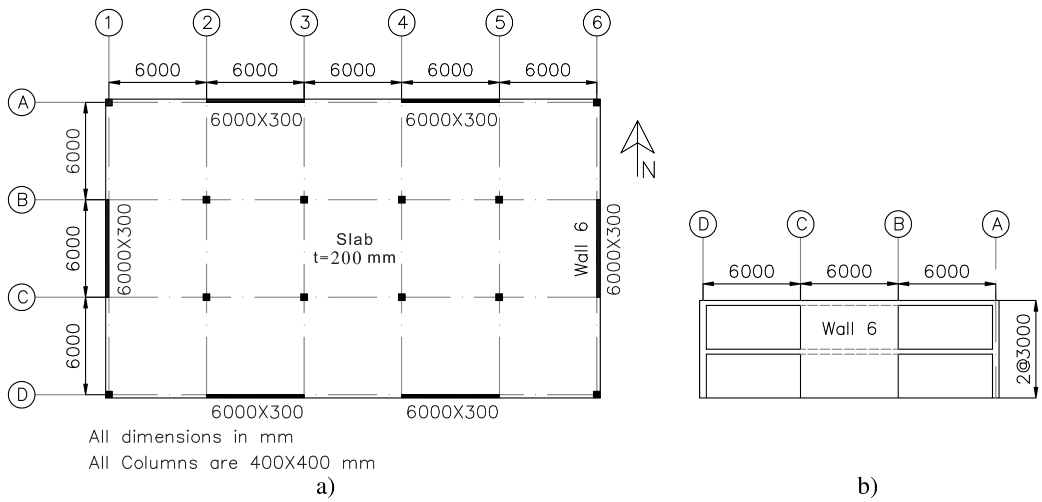

Figure 2-1. Prototype building: a) Plan view; b) Transverse elevation view

Seismic analysis of the building was performed using the equivalent static force procedure with the assumption of rigid diaphragms without bending deformations to be consistent with the methods of analysis commonly used in the 1960s. Wall 6 (6000 mm x 6000 mm x 300 mm) was selected for the numerical assessment of the retrofitting scheme with SMA. Design of Wall 6 for seismic actions in the North-South direction followed the ultimate limit state method as specified by the NBCC-1965, which was an alternative to the commonly used working stress method. In general, the use of the ultimate limit state method in the 1960s yielded less flexural reinforcement that resulted in lightly reinforced concrete walls with significant seismic deficiencies. Seismic loads, according to the NBCC-1965, were based on the minimum shear force at the base of the building, *V* (see Equation 2-1). The base shear force was a function of the total weight of the structural and non-structural elements, *W;* the hazard factor, *R*; the type of construction, *C*; the importance factor, *I*; the foundation soil factor, *F*, and the height of the structure, *S*. The factor *S* is an inverse function of the number of storeys, *N*, of the building above ground.

$$V = RCIFSW; \qquad S = 0.25/(9+N) \tag{2-1}$$

For the analysis of the prototype building, factor *R* was taken as 4 for the city of Vancouver, factor *C* was taken as 1.25 for building types other than portal frames or ductile shear walls, factor *I* was taken as 1 for buildings with normal importance, and factor *F* was taken as 1.0 for an assumed rock sub-soil condition. The factor *S* was calculated as 2.27 for the two-storey prototype building. A seismic force of 556 kN, including 5% accidental torsion, was calculated for Wall 6 and linearly distributed to the floor levels. This distribution resulted in a shear span of 0.81. Material properties were assumed to reflect construction practices of the 1960s: cylinder compression strength of concrete, f'_c, was taken as 21 MPa (3000 psi), and yield stress of reinforcing steel, f_y, was 280 MPa (40ksi). The design resulted in a horizontal reinforcement ratio, ρ_h, of 0.21% and a vertical reinforcement ratio, ρ_v, of 0.18%. The wall was not designed with any special detailing for ductility and anti-buckling since this was not required by the NBCC-1965.

2.2 Seismic assessment of the original wall

The seismic response of the original, non-retrofitted, wall was initially assessed with the NBCC-2010 to satisfy the increase in base shear requirements relative to the NBCC-1965, specifically in terms of seismic force and ductility. The ductility capacity of the retrofitted shear wall is predetermined as nominal (minimum ductility capacity) with seismic reduction factor R_d of 1.5. No over strength reduction factor, R_o, was considered in the calculation of the required strength since expected material values of f'_c = 25 MPa ($1.20f'_c$) and f_y = 350 MPa ($1.25f_y$) were used in the non-linear analysis of the non-retrofitted and retrofitted walls as recommended by the ATC-40 and FEMA-440. The design seismic force (design base shear V) was calculated as follows:

$$V = \frac{S(T_a)M_V I_E W}{R_d} \leq \frac{(2/3)S(0.2)I_E W}{R_d} \qquad (2\text{-}2)$$

where $S(T_a)$ is the 5% damped spectral response acceleration of the structure at the fundamental period, T_a; M_v is the factor to account for higher modes effect (M_v=1.0 for structures with short fundamental periods); I_E is the importance factor; $S(0.2)$ is the 5% damped spectral response acceleration at T_a=0.2 seconds; and W is the seismic weight of the structure. The fundamental period, T_a, is a function of the height of the building, h_n, and is calculated as follows for shear wall structures:

$$T_a = 0.05 h_n^{3/4} \qquad (2\text{-}3)$$

Using Equations 2-2 and 2-3, NBCC-2010 yields a design base shear of 1972 kN for Wall 6, including accidental torsion. The resulting design base shear is 3.55 times larger than that calculated with the NBCC-1965. This suggests that Wall 6 is in need of retrofit for enhancing strength to meet the NBCC-2010 required levels. A comprehensive assessment of the seismic response of Wall 6 is performed using the capacity spectrum method as well as the nonlinear finite element program VecTor2. Based on this assessment, the proposed SMA retrofit to improve seismic response is designed.

3. DESIGN OF SMA RETROFIT

3.1 Seismic assessment of the retrofitted wall

The performance of the retrofitted wall was assessed following the capacity spectrum method (ATC 40 and FEMA 440), which has been widely used for other retrofitting strategies. The approach requires a capacity diagram (lateral load vs. spectral displacement), which is evaluated against a reduced elastic (equivalent inelastic) demand diagram (base shear vs. spectral displacement). To develop the capacity diagram, a pushover response (lateral load vs. displacement) is determined from a trace of the predicted hysteretic behaviour. Program VecTor 2 is used to predict hysteretic responses from which the pushover curves are developed for the SMA retrofitted wall. The pushover response is converted to a capacity diagram by relating the roof displacements, Δ_{roof}, to spectral displacements, S_d. The roof displacements are divided by the product of the first modal participation factor, PF_1, and the first modal roof displacement, $\phi_{1,roof}$ as follows:

$$S_d = \Delta_{roof} / PF_1\, \phi_{1,roof} \qquad (3\text{-}1)$$

Based on a fundamental triangular modal distribution, which is typical of buildings with short fundamental periods, and the mass distribution of the prototype building, a value of 1.23 was calculated for $PF_1\phi_{1,roof}$ for Wall 6.

The development of the demand diagram requires the Uniform Hazard Spectrum (UHS) for the location of the structure. For the assessment of the retrofitted prototype building, the UHS for the City of Vancouver was developed based on the NBCC-2010. The spectral acceleration ordinates, S_a, are converted to base shear, V, and the structural period ordinates are converted to spectral displacements, S_d, by using the following equations:

$$V = S_a \alpha_1 W \; ; \qquad S_d = \left(T^2/4\pi^2\right)S_a g \tag{3-2}$$

where α_1 is the first modal mass coefficient, W is the weight of the structure, T is the fundamental period of the structure, and g is the gravitational acceleration.

The elastic demand diagram is then reduced to account for energy dissipation of the retrofitted shear wall by applying reduction factors SR_A and SR_V that are calculated based on the effective damping, β_{eff}, of the structure, following the formulations below:

$$SR_A = \left[3.21 - 0.68\ln\left(100\beta_{eff}\right)\right]/2.12 \quad ; \qquad SR_V = \left[2.31 - 0.41\ln\left(100\beta_{eff}\right)\right]/1.65 \tag{3-3}$$

SR_A is applied to the constant acceleration portion of the linear elastic design spectrum (plateau), while SR_V is applied to the constant velocity portion of the linear elastic design spectrum (descending branch). The effective damping, which is a combination of the inherent elastic viscous damping of the structure, β_o, and the equivalent viscous damping of the hysteretic loops, β_{eq}, is calculated as follows:

$$\beta_{eff} = \beta_o + k\beta_{eq} \quad ; \qquad \beta_{eq} = \left(1/4\pi\right)\left(E_D/E_{So}\right) \tag{3-4}$$

where E_D corresponds to the energy dissipated by hysteretic damping (area under the hysteretic loop at the target displacement) and ES_o corresponds to the maximum strain energy (area of the triangle defined by the target spectral displacement and corresponding strength) as shown in Figure 3-1. In the formulation presented in ATC-40 and FEMA 440, a reduction factor k is included to account for the difference between the actual hysteretic loop and the ideal elasto-plastic loop. The k factor was not used in the assessment of the retrofitted walls since the equivalent damping values were established from the predicted numerical hysteretic loop at the selected target displacement. Elastic viscous damping, β_o, was calculated using a model proposed for Farrar and Baker (1995) for squat shear walls. For the required strength of 1972 kN and corresponding shear stress of 1.10 MPa (calculated with the NBCC-2010), Farrar and Baker's formulation yielded a value of 1.38% for β_o.

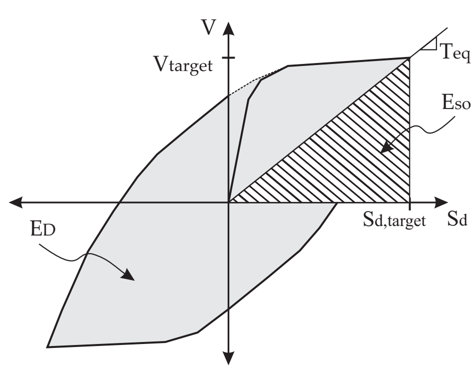

Figure 3-1. Graphical representation of hysteretic damping

Following the capacity spectrum method, the retrofitting reinforcement for the prototype wall was continually redesigned until the capacity diagram crossed the reduced elastic demand diagram at the target displacement as shown in Figure 3-2. The retrofitted system is thus designed to satisfy the strength and ductility requirements of the reduced elastic demand diagram based on the properties of the SMA retrofit. The resulting SMA retrofit for Wall 6 included external rigid steel elements coupled with two-30 mm diameter nickel-titanium SMA rods (1400 mm^2 cross sectional area) on each face of the wall (Figure 1-1a)). The length of the SMA rods was optimized to sustain maximum deformation recovery. The retrofitted wall with SMA is compared with a companion wall retrofitted with bi-diagonal steel plates with sectional area of 1200 mm^2 attached to each face of the wall in a cross pattern (Figure 1-1b)). Yield stress of the retrofitting SMA and steel was taken as 420 MPa. Both the SMA and steel retrofitting strategies were aimed to improve strength, ductility and energy dissipation capacities. In addition, the SMA retrofitting strategy was focused to reduce permanent residual deformation of the shear wall.

The reduced elastic demand diagram for the SMA bracing and steel plate retrofit systems for the City of Vancouver is illustrated in Figure 3-2. Calculated reduction factors and effective damping values for the retrofitted walls are also provided.

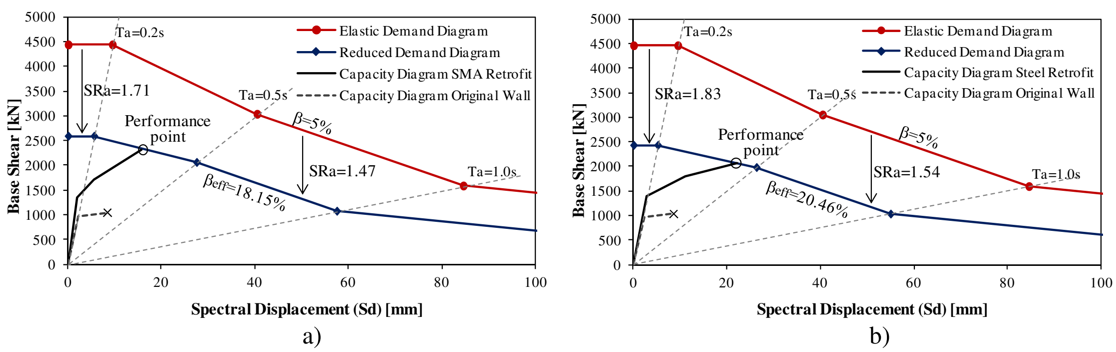

Figure 3-2. Seismic assessment of retrofitted walls: a) SMA retrofit; b) Steel plate retrofit

Figure 3-2 shows similar seismic performance for both the SMA and the steel retrofitting systems. The calculated effective damping values and reduction factors indicate similar energy dissipation for both retrofitting systems. The designed SMA retrofit, however, resulted in slightly more strength and less ductility. A more detailed discussion of the response for the retrofitted walls is presented with the results of the numerical simulations.

3.2 Numerical simulation

Numerical simulations of the retrofitting strategies were performed with the nonlinear finite element program VecTor2. The finite element approach provided a detailed assessment, including: failure load, failure mode, displacements, ductility, energy dissipation, and residual deformations of the original and retrofitted walls. VecTor2 is based on the Modified Compression Field Theory (MCFT) and the Disturbed Stress Field Model (DSFM) and employs a smeared rotating crack approach to model cracked concrete. The program includes comprehensive constitutive models for concrete and reinforcing materials, such as steel, FRP, and SMA. Details of these models can be found elsewhere (Wong and Vecchio, 2002). Specifically for SMA, Abdulridha, Palermo, and Foo (2010) used a simple model for SMA reinforcement with full plastic deformation recovery (Figure 3-3). This model was satisfactorily validated through the analysis of SMA reinforced concrete beams and shear walls.

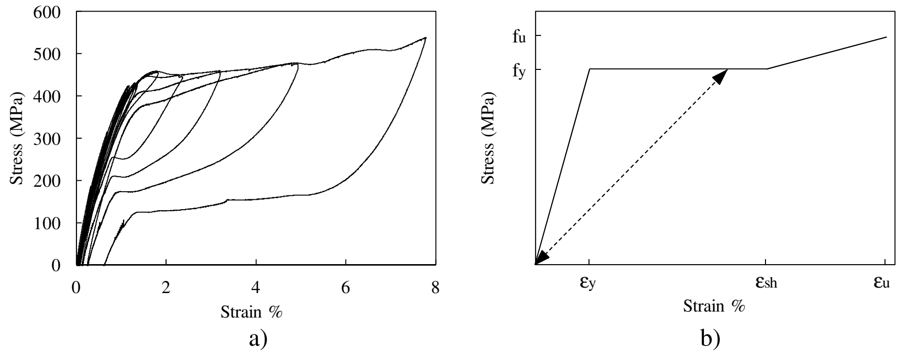

Figure 3-3. Cyclic response of SMA rods (Abdulridha, Palermo, and Foo, 2010): a) Coupon test; b) Model

The original wall was modelled with two RC zones, one corresponding to the web, and the other to the slabs (Figure 3-4). The web consisted of forty rectangular RC elements in the horizontal direction and thirty-eight rectangular RC elements in the vertical direction. The horizontal and vertical reinforcement was smeared in the reinforced concrete elements.

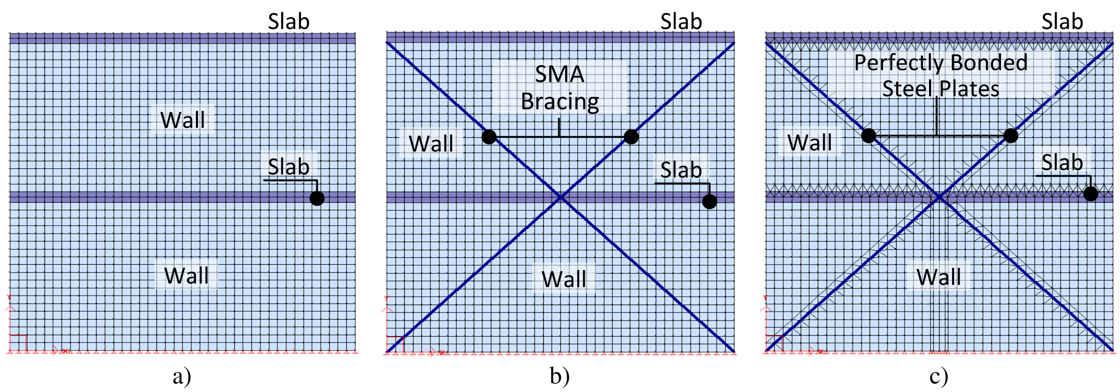

Figure 3-4. Finite element models for shear walls: a) Original; b) SMA retrofitted; c) Steel plate retrofitted

The wall retrofitted with SMA bracing was modelled with the same number of RC elements as the original wall. The SMA links in the cross bracing were designed with a length of 350 mm, but were simulated by single truss elements with length of 8500 mm to account for the length of the adjoining rigid steel elements (Figure 1-1a)). The truss elements were connected to opposite corners at the base and top of the wall. The single truss elements were assigned an equivalent stress-strain relationship assuming that all deformation would be experienced by the SMA links. Thus, given that the SMA links were assigned a length 25 times larger than their design length, the stress and strain of the material model assigned to the truss element were reduced 25 times, while the area of the SMA links was increased 25 times. This model ensures that the stiffness of the SMA links is maintained and also makes certain that the axial force and strain demands imposed on the SMA are equivalent for the model and actual structure. The wall retrofitted with steel plates was modelled with ductile steel truss elements perfectly bonded to rectangular and triangular RC elements, following recommendations of Cortés-Puentes and Palermo (2012). Triangular RC elements were limited to regions with geometric constraints.

For all the walls, the Smith-Young relationship was employed to model the pre-peak and post-peak response of concrete in compression. This model is generally better at capturing the gradual softening response that is exhibited in the post-peak range of lower concrete strengths. The other constitutive models that were employed to capture salient features of the response included: Vecchio 1992-A model for compression softening (Vecchio and Collins, 1993), modified Bentz model for tension stiffening (Vecchio, 2000), and Palermo and Vecchio 2002 model for hysteretic behavior (Palermo and Vecchio, 2003). Steel reinforcement was modelled with a tri-linear relationship with initial elastic response, yield plateau, and non-linear strain hardening. The hysteretic response of the steel was modelled with the Seckin model, which accounts for the Bauschinger effect. The concrete and reinforcing steel models were selected based on a parametric study conducted by Cortés-Puentes (2009). The SMA rods were modelled with a tri-linear relationship, similar to that used for ductile steel, with full strain recovery as depicted in Figure 3-3.

For each analysis, axial loads were distributed along the top row of finite elements corresponding to the top slab, while the lateral seismic load (lateral displacement) was assigned to the nodes located at 4900 mm in height with respect to the base of the wall. The location of the lateral loading corresponded to the shear span, which was calculated from the base overturning moment and base shear force from the equivalent static force procedure. The lateral loading followed a reverse cyclic protocol with two repetitions at each displacement level, which was incremented by a factor of 1.4. The simulation initiated with a cycle to 0.5 mm displacement.

3.3 Calculated response

The numerical analysis of the original wall predicted strength of 1039 kN at a corresponding lateral top displacement of 10 mm (Table 3-1 and Figure 3-5). Furthermore, the wall responded with significant pinching and residual plastic deformation of approximately 6 mm. Damage was concentrated along the base, which resulted in rocking behaviour of the wall (Figure 3-6) and failure in the form of rupture of vertical reinforcement near the edges of the walls at 14 mm of lateral displacement. This failure mechanism is consistent with observations during testing of lightly reinforced squat RC shear walls (Tagdhi et al., 2000).

Table 3-1. Summary of predicted response of shear walls

Wall	Strength (kN)	Ultimate Displacement (mm)	Residual Displacement (mm)	Max. Residual Crack Width (mm)	Failure Mode
Original	1039	10	6	33	Rocking behaviour due to major crack along the base followed by rupturing of steel
SMA Retrofit	2317	19	2	15	Rocking behaviour due to major crack along the base followed by rupturing of steel
Steel Plate Retrofit	2073	27	14	46	Extensive flexure and shear cracking followed by shear sliding at the quarter height of the wall

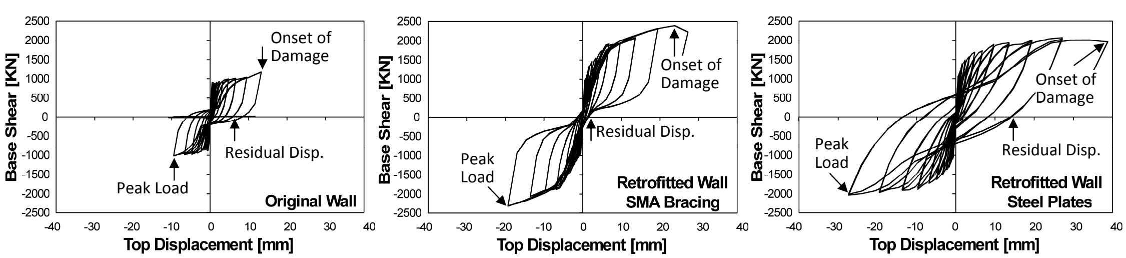

Figure 3-5. Predicted response of shear walls: a) Original; b) SMA retrofitted; c) Steel plate retrofitted

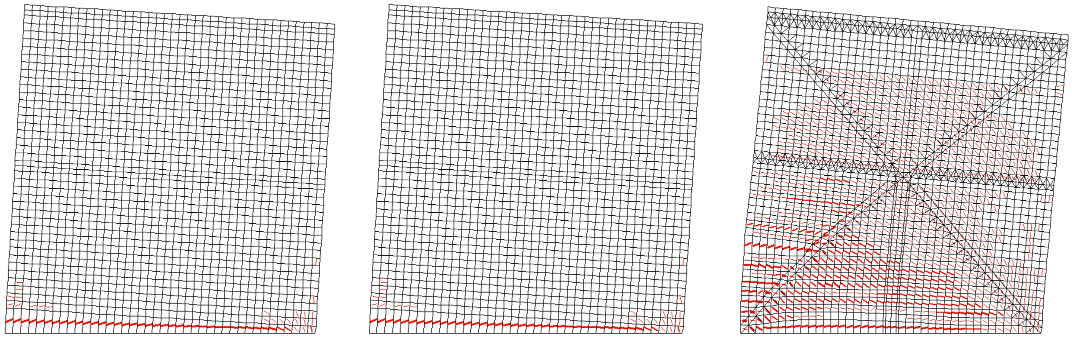

Figure 3-6. Predicted damage of shear walls: a) Original; b) SMA retrofitted; c) Steel plate retrofitted

The wall retrofitted with SMA cross bracing sustained an ultimate strength of 2317 kN at 19 mm (Table 3-1 and Figure 3-5), which represented an increase of strength of approximately 125%. The predicted failure mechanism involved rocking of the shear wall with subsequent rupture of the vertical reinforcement, similar to that calculated for the original wall. The SMA retrofit reduced the residual plastic deformation of the original wall by 66%. The numerical analysis of the wall retrofitted with steel plate predicted peak strength of 2073 kN (Table 3-1) and displayed initial stiffness similar to that of the wall retrofitted with SMA (Figure 3-5). The wall retrofitted with steel plates responded primarily in flexure with concrete damage throughout the wall (Figure 3-6), while the wall retrofitted with SMA was ultimately controlled by rocking with concrete damage localized at the base of the wall (Figure 3-6). The predicted cracking pattern of the wall retrofitted with steel plates indicates diagonal shear cracking in addition to flexural cracking, which was not predicted for the wall retrofitted with SMA. The predicted maximum residual crack width for the wall with SMA retrofit was approximately half of that predicted for the non-retrofitted wall and one-third of that predicted for the wall with steel plates (Table 3-1). The smaller residual crack widths for the wall retrofitted with SMA are a direct result of the re-centering properties of the SMA rods. At 19 mm top displacement, the cumulative energy dissipation of the wall retrofitted with SMA was approximately 30 % higher than that of the wall retrofitted with steel plates (Figure 3-7). At ultimate displacement, however, the total energy dissipated by the wall retrofitted with SMA was lower than the total energy dissipated by the wall retrofitted with steel plates (Figure 3-7).

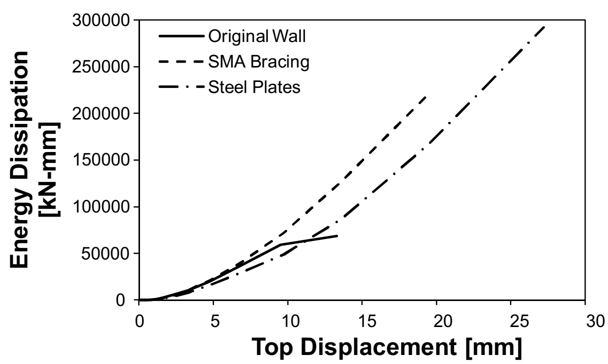

Figure 3-7. Cumulative energy dissipation

4. CONCLUSIONS

Emerging shape memory alloys possess re-centering capabilities and good energy dissipation capacity that can be utilized for improving seismic response of squat reinforced concrete shear walls. An SMA system was proposed to retrofit a seismically deficient squat reinforced concrete shear wall designed following pre-1970s standards. The SMA retrofitting system was assessed with the capacity spectrum method and numerically simulated with the finite element method. Furthermore, the seismic response of the SMA retrofitting system was compared with a traditional steel plate retrofitting system. Results from the seismic assessment indicate that retrofitting with SMA has the potential of improving strength and ductility, while reducing the permanent deformations and controlling cracking of concrete. The shear wall retrofitted with SMA displayed ductile behaviour with minimum residual deformation. Similar strength and ductility was predicted for both the wall retrofitted with SMA and the wall retrofitted with steel plates. Damage in the shear wall retrofitted with SMA was localized at the base of the wall, while damage in the wall retrofitted with steel plates was widespread throughout the wall. In general, the seismic assessment illustrated the capability of the proposed SMA retrofitting system to dissipate energy while reducing plastic deformations of squat reinforced concrete shear walls. Although the numerical assessment demonstrated promising results, further experimental research is needed to validate the use of SMAs for seismic retrofit of existing squat RC shear walls.

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