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SEISMIC FRAGILITY ASSESSMENT OF SUPERELASTIC SHAPE MEMORY ALLOY REINFORCED CONCRETE SHEAR WALLS

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ABSTRACT

Mitigation of seismic damage can be achieved through self-centering techniques. One of the potential techniques involves the use of Superelastic Shape Memory Alloy (SE-SMA) bars in Reinforced Concrete (RC) structures. This study explores the use of such bars in the plastic-hinge regions of RC walls. The seismic performance and vulnerability of SE-SMA RC walls of ten- and twenty-story buildings are analytically assessed using fragility curves. The maximum inter-story drift, residual drift, and fragility are evaluated using multi strip analysis. The results clearly demonstrate the superior seismic performance of SE-SMA RC walls as compared to steel RC walls.

KEYWORDS: Shear wall; shape memory alloy; multi-strip analysis; fragility analysis; inter-story drift ratio; residual displacement

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1. INTRODUCTION

The main function of reinforced concrete (RC) structural walls is to resist lateral loads. Extensive studies have been conducted to explore their behaviour under various load conditions [1, 2, 3]. The seismic design philosophy, which aims at preserving life, leaves RC walls vulnerable to damage during strong seismic excitations. This damage was observed following many earthquakes including the 1985 Mexico earthquake [4], the 1999 Chi-Chi earthquake [5], the 2010 Maule earthquake [6], and the 2011 Christchurch earthquake [7].

Residual drift is one of the measures to evaluate the seismic performance of a structure. FEMA P-58-1 [8] introduced four damage states related to residual drift ratios. The limit for repairable structural elements was set at 1% residual inter-story drift [8]. McCormick et al. [9] concluded that the economical limit is 0.5%. To mitigate the residual displacements of RC walls, self-centering methods that rely on unbounded post-tensioned tendons and supplementary energy dissipation devices were proposed [10, 11, 12]. Although these methods have resulted in improved seismic performance, researchers are still exploring new techniques.

Superelastic shape memory alloy (SE-SMA) can recover its inelastic deformations upon the removal of the applied load. This unique property has been utilized by many researchers [13, 14, 15, 16, 17, 18]. The flag-shaped hysteresis of SE-SMA can eliminate the seismic residual drifts on the cost of lower energy dissipation as compared to steel reinforcement. Also, the lower modulus of elasticity of SE-SMA bars leads to higher seismic deformations. Researchers have addressed these disadvantages by minimizing the amount of SE-SMA material [17, 18]. The potential use of SE-SMA bars was extended to RC walls by a number of researchers. Effendy et al. [19] used external diagonal SE-SMA

bars to upgrade the seismic performance of existing squat walls. Test results showed a significant reduction in the residual displacements combined with a 16% to 26% increase in the peak shear strength. Abdulridha [20] experimentally studied the cyclic behaviour of a concrete wall. The boundary elements were reinforced with longitudinal SE-SMA bars at the plastic hinge region [20]. The SE-SMA bars increased the wall ductility and significantly reduced the residual displacements. Abraik and Youssef [21] conducted an analytical study to identify the performance of SE-SMA RC squat and intermediate walls considering different SE-SMA bar locations. The results highlighted that location of the SE-SMA bars have a significant effect on the wall seismic performance.

Research addressing the seismic vulnerability of tall concrete walls reinforced with SE-SMA bars is missing in the literature. The paper addresses this topic. It starts by identifying the plastic hinges for 10 and 20-story steel RC walls that are designed and detailed per CSA A23.3 [22] and NBCC [23]. The influence of using SE-SMA bars is then evaluated. Fragility curves are presented considering various damage states.

2. NUMERICAL MODEL

A multi-story RC wall is shown in Figure 1a. The walls are modeled using the Shear-Flexural Interaction Multi-Vertical Line Element Model (SFI-MVLEM), Figure 1b. This model was implemented in the Open System for Earthquake Engineering Simulation software (OpenSees) [24] by Kolozvari [25]. It allows simulating the seismic response of RC walls using two-dimensional membrane panels. The edge and interior panels represent the boundary elements and the wall web, respectively. The panels are modeled using a fixed angle crack approach. The rigid beams at the top and bottom enforces a plane section assumption. The flexural response of a wall is captured through

the axial deformations of the RC panels in the vertical direction. The average normal vertical strain is determined by dividing the average vertical deformation by the element height h . The relative rotation between the top and bottom of the wall element is assumed to happen at a height ch , measured from the bottom face. The value of the height coefficient c was recommended to be taken as 0.4 by Orakcal and Wallace [26]. The shear deformation of the SFI-MVLEM element is determined by transferring the external deformation components to a point at a height ch . The shear strain for each panel is calculated by dividing the shear deformation by the element height h . The normal strain in the horizontal direction is obtained by dividing the horizontal deformation at the internal degrees of freedom by the panel width. The effect of increasing or decreasing the number of RC panels or the number of SFI-MVLEM on the total displacement was found to be insignificant [25].

The built-in nonlinear material constitutive relationship, proposed by Menegotto and Pinto [27], was used to model the steel reinforcing bars (Figure 2a). The concrete was modeled using the constitutive relationships proposed by Chang and Mander [28], shown in Figure 2b. It should be noted that the origin of the tension envelope curve depends on the unloading strain [25]. Figure 2c illustrates the symmetric SE-SMA self-centering model. The SE-SMA model parameters were evaluated experimentally by Tazarv and Saiidi [14] and Varela and Saiidi [29]. The modulus of elasticity of the SE-SMA bars (E_{SMA}), the stress at which inelastic deformations initiate (f_{SMA}), and the post-yield strength (K_2) are assumed to be 38,000 MPa, 380 MPa, and 1725 MPa, respectively. The ultimate strain for SE-SMA is assumed to correspond to the point at which it loses the ability to recover its original shape (strain of 7%).

Local failure is defined when the strain in the longitudinal steel reinforcement reaches the yield strain ε_y and the concrete compressive strain reaches 0.2% [32, 33]. The structural and non-structural elements are expected to have sustained significant damage at this stage [32].

An experimental shake table test of a slender eight-story concrete wall, tested by Ghorbanirenani et al. [30], was selected to further validate the SFI-MVLEM model. The wall, shown in Figure 3a, was designed per NBCC 2005 [31] with a force reduction factor of 2.8.

Figure 1. MVLE model (a) RC wall; (b) One-story model

Figure 2. Materials model (a) steel bars; (b) concrete; (c) SE-SMA

A simulated time history ground motion developed for eastern North America was used to experimentally test the wall. Inelastic flexural response was developed at the wall base and at the sixth story. Flexural-shear cracks at the wall base and flexural cracks at the sixth level were also observed. The wall was modeled using eight SFI-MVLEM elements. The predicted results matched closely the experimental ones, as shown in Figure 4a. Slight discrepancies between the numerical predictions and experimental data at 7 seconds can be observed.

Abdulridha [20] performed a large-scale test on an SE-SMA RC wall to evaluate its performance when subjected to lateral loading as shown in Figure 3b. The experimental load-displacement response, shown in Figure 4b, is compared with predictions of Finite Element Analysis (FEA) by Abdulridha [20] and SFI-MVLEM model. The initial stiffness predicted by the FEA and Opensees are similar and higher than the experimental results by about 12%. This discrepancy is related to the bond slip effect, which is not modeled. The yield displacement from both models is 8% higher than the experimental value. The SFI-MVEL model predicted accurately the ultimate strength at a

displacement of 73 mm, whereas the ultimate strength predicted by the FEA model was higher by about 3%.

(a)

(b)

Figure 3. Tested walls: (a) Steel RC Wall [30]; (b) SE-SMA RC wall [20] (dimensions are in mm)

Figure 4. Numerical model versus the experimental data: (a) Steel RC wall tested by Ghorbanirenani [30]; (b) SE-SMA RC wall tested by Abdulridha [20]

3. STEEL RC WALLS

The structural plan of the considered RC building is shown in Figure 5. The building is assumed to be in southern British Columbia. The concrete shear walls are designed and detailed using CSA A23.3 [22] and NBCC [23]. The overstrength (R_0) and ductility (R_d) factors are 1.6 and 3.5, respectively. The concrete compressive strength and the yield strength of the steel rebars are assumed to be 30 MPa and 450 MPa, respectively. The structural lumped mass includes the self-weight and 25% of the floor live load, assumed 2.8 kN/m². Characteristics of the considered walls, shown in Figure 5c, are summarized in Table 1.

Table 1. Characteristics of considered walls

	10-story	20-story
Wall thickness (b_w)	250 mm	350 mm
Wall length (l_w)	4060 mm	6000 mm
Length of wall boundary element (l_{bl})	500 mm	600 mm
Floor height	2800 mm	
Axial load per story	233 kN	
Weight per story	1248 kN	
Vertical and horizontal steel ratio in the web (ρ_{vw}, ρ_{hw})	0.25%	
Horizontal steel ratio in the boundary elements (ρ_{hb})	0.67%	
Vertical and horizontal steel ratio in the web at plastic hinge (ρ_{vw}, ρ_{hw})	0.5%	
Vertical steel ratio in the boundary elements (ρ_{vb})	1.28%	1.90%
Maximum axial load ratio ($\frac{P}{A_g f_c}$)	0.1	0.12

Figure 5. Considered building (a) structural plan; (b) wall elevation; (c) typical wall section

Steel RC walls are designed based on A23.3 [22], which assumes that the plastic hinge develops at the wall base. Detailing requirements of A23.3 [22] ensure a certain level of ductility along the wall height by modifying the factored moment M_f , as shown in Figure 6. The static analysis procedure of A23.3 [22] assumes a linear variation of the factored moment starting at the top of the plastic hinge as shown in Figure 6. Although CSA ensures adequate level of ductility to mitigate yielding at any point outside the plastic hinge zone, there is a possibility for spread of plasticity along the wall height [30, 32, 33, 34]. The design shear forces found from the analysis are to be increased over the wall height by the ratio between the moment of resistance M_r and the factored moment M_f . The corresponding shear values must exceed the smaller of the shear corresponding to the probable moment capacity and the shear demand calculated assuming $R_d R_0$ equal to 1.3.

Figure 6. Capacity design moment envelopes for steel RC wall: (a) detailing requirements; (b) variation of moments along wall height

4. SEISMIC GROUND MOTIONS

The Conditional Mean Spectrum (CMS) proposed by Baker [35] was utilized to select the ground motions. The method is based on choosing the spectrum that has a target amplitude for a specific structure period. An eigenvalue analysis was used to determine the structure period for the steel RC walls. The resulting periods were 1.67 s and 3.06 s for the 10 and 20-story walls, respectively. Seven level of hazards with return periods ranging from 72 years to 2475 years were selected, as shown in Table 2. Soil class D with shear wave velocity ranging from 180 to 360 m/s was assumed. Each of the hazard levels is represented by 20 ground motions having the spectral accelerations shown in Table 2.

Table 2. Hazard levels considered for Southern British Columbia

Hazard Level	Return Period (years)	$S_{a10}(T_1 = 1.67)(g)$	$S_{a20}(T_1 = 3.06)(g)$
1	72 [50% in 50]	0.06	0.05
2	224 [20% in 50]	0.17	0.09
3	336 [20% in 75]	0.21	0.12
4	475 [10% in 50]	0.25	0.15
5	975 [5% in 50]	0.34	0.19
6	1462 [5% in 75]	0.40	0.26
7	2475 [2% in 50]	0.46	0.30

5. SE-SMA RC WALLS

Multi Strip Analysis (MSA) was used to evaluate the seismic response of the 10 and 20-story steel RC walls. Strain profiles along the wall heights were used to determine the length and locations of the SE-SMA bars. Figure 7b shows the mean strains in the longitudinal bars of the 10 and 20 story steel RC walls, when subjected to 20 earthquakes with 2475 years return period. The main plastic hinge is formed at the base. However, an additional plastic hinge is formed at mid-height of the wall. The steel strain at the 5th and 6th stories of the 10-story steel RC wall exceeded the yield strain. The

same trend is observed for the 20-story wall, where the strain in the rebars at 10th, 11th, 12th, and 13th stories exceeded the yield strain. In both buildings, plastic hinge length is about 20% of the total wall height.

The design of the SE-SMA RC is taken similar to the steel RC wall. SE-SMA bars are assumed to replace the vertical steel bars at the boundary elements for the plastic hinge length. The SE-SMA bars are connected to steel bars using mechanical couplers as proposed by Youssef et al. [15]. Resulting periods for SE-SMA RC walls were 1.71 s and 3.1 s for the 10 and 20-story buildings, respectively.

Figure 7. Plastic hinges in the considered walls: (a) location of plastic hinges; (b) strain profile along the wall height

6. SEISMIC PERFORMANCE OF STEEL AND SE-SMA RC WALLS

Figures 8 and 9 compare the shear forces and bending moments for the steel and SE-SMA RC walls at the first and second periods. The shear forces for the steel RC walls were higher than the SE-SMA walls by about 5% to 10%. Similar trend is noticed for the bending moments, which were higher by about 8% to 12% at the wall base and 3% to 15% at mid-height. The flexibility of the SE-SMA rebar slightly lengthened the wall natural period, which resulted in a decrease in the bending moments and shear forces.

(a)

(b)

(c)

(d)

Figure 8. Shear force along wall height at: (a) 10-story first period; (b) 20-story first period; (c) 10-story second period; (d) 20-story second period

(a)

(b)

(c)

(d)

Figure 9. Bending moment along wall height at: (a) 10-story first period; (b) 20-story first period; (c) 10-story second period; (d) 20-story second period

The lateral displacement envelopes are plotted in Fig. 10 for 2475-, 475-, and 72-year events. The displacements of the considered walls follow approximately a linear trend. The lateral displacement of the SE-SMA RC walls are higher than the steel RC walls by 6 to 16%. This increase is due to the lower stiffness of the SE-SMA bars.

The residual displacement envelopes are plotted in Figure 11. For low intensity seismic ground motions, the residual displacements of the SE-SMA RC walls are not significantly different from the steel RC walls as the behaviour was in the elastic range. The use of SE-SMA bars reduces the residual displacements by 19% to 50 % for moderate and high level seismic ground intensities. The reduction was more pronounced for the 10-story wall, which is less flexible as compared to the 20-story wall. Lateral displacements and residual displacements envelopes for different ground motion intensity levels are summarized in Table 3.

Figure 10. Lateral displacement envelopes for: (a) Return period 2475 yrs/ 10-story wall; (b) Return period 475 yrs/ 10-story wall; (c) Return period 72 yrs/ 10-story wall; (d) Return period 2475 yrs/ 20-story wall; (e) Return period 475 yrs/ 20-story wall; (f) Return period 72 yrs/ 20-story wall

Figure 11. Residual displacement envelopes for: (a) Return period 2475 yrs/ 10-story wall; (b) Return period 475 yrs/ 10-story wall; (c) Return period 72 yrs/ 10-story wall; (d) Return period 2475 yrs/ 20-story wall; (e) Return period 475 yrs/ 20-story wall; (f) Return period 72 yrs/ 20-story wall

Table 3. Peak lateral displacements and residual displacements

Events	10-story wall				20-story wall			
	Steel	SE-SMA	Steel	SE-SMA	Steel	SE-SMA	Steel	SE-SMA
	Lateral Displacement (m)		Residual Displacement (m)		Lateral Displacement (m)		Residual Displacement (m)	
2475	0.86	1.00	0.042	0.020	1.30	1.54	0.06	0.040
475	0.36	0.40	0.016	0.005	0.73	0.77	0.05	0.03
72	0.11	0.12	0.005	0.003	0.21	0.21	0.02	0.02

7. FRAGILITY FUNCTION

Seismic damage can be assessed using story drift ratio [36] or inter-story drifts [37]. The damage level for steel RC walls and SE-SMA RC walls can be judged as similar based on the story drifts (Figure 10) or significantly different based on the residual drifts (Figure 11). In this section, fragility curves are presented for both inter-story and residual drifts.

A fragility function describes the probability of damage for a given seismic intensity (IM). It can be expressed using Equation 1 [38]:

$$P(C \setminus IM = x) = \Phi \left[\frac{\ln(\frac{x}{\theta})}{\beta} \right] \quad (1)$$

Where, P is the probability of exceeding a specific damage level C , Φ is the standard normal cumulative distribution, θ is the median of the fragility function, and β is the standard deviation of $\ln(IM)$. The fragility curve can be obtained using incremental dynamic analysis [39]. In this

approach, the seismic intensity is incrementally increased until collapse. However, this method is time-consuming and raises the concern of whether scaling moderate intensity ground motions can represent high intensity ground motions [38, 40]. Multi-Strip Dynamic Analysis (MSA) is an efficient approach that addresses this concern [38]. In this approach, the structure is subjected to a number of ground motions representing each hazard level.

7.1 FRAGILITY CURVES

The mean roof inter-story drifts (ID) and the residual roof drifts (RD) for seven seismic hazard levels, representing return periods of 72 years to 2475 years, are summarized in Table 4. Figure 12 displays the MSA curves that depict the relationship of the mean roof inter-story drift against the seven hazard intensity levels. The mean roof inter-story drift for the 10-story SE-SMA wall is 19% higher than that of the steel RC wall. The 20-story walls have similar behaviour up to a hazard intensity of 0.2g. At higher intensity levels, the 10-story SE-SMA RC wall experiences slightly higher inter-story drifts compared to the 20-story SE-SMA RC wall. This apparent difference in the inter-story drift is due to differences in the wall heights and boundary element reinforcement ratios.

Figure 13 shows the relationship between the residual roof displacement and the ground motion intensity level. The residual displacements for the 10 and 20-story steel RC walls are higher than the corresponding SE-SMA RC walls.

(a)

(b)

(c)

(d)

Figure 12. Maximum inter-story drift ratios as function of ground motion intensity, for (a) 10-story steel RC wall; (b) 10-story SE-SMA RC wall; (c) 20-story steel RC wall; (d) 20-story SE-SMA RC wall

Figure 13. Maximum residual drift as function of ground motion intensity: (a) 10-story steel RC wall; (b) 10-story SE-SMA RC wall; (c) 20-story steel RC wall; (d) 20-story SE-SMA RC wall

Table 4. Mean inter-story drifts and residual drifts

Event (years)	10-story				20-story			
	Steel		SE-SMA		Steel		SE-SMA	
	ID (%)	RD (%)	ID (%)	RD (%)	ID (%)	RD (%)	ID (%)	RD (%)
2475	3.07	0.23	3.74	0.072	2.72	0.13	2.82	0.08
1462	2.69	0.18	3.20	0.071	2.77	0.11	2.90	0.07
975	2.28	0.10	2.84	0.054	1.93	0.095	1.95	0.053
475	1.63	0.034	1.43	0.025	1.52	0.060	1.71	0.049
336	1.52	0.050	1.60	0.030	1.26	0.063	1.36	0.060
224	1.36	0.035	1.49	0.025	1.00	0.069	1.12	0.054
72	0.40	0.004	0.40	0.003	0.48	0.039	0.46	0.04

The collapse fragility curve, which shows the collapse probability as a function of ground motion intensity (S_a), is provided in Figure 14a. The fragility function fitting method proposed by Baker [40] is used to generate the fragility curves. At low levels of seismic excitations, a significant reduction in wall fragility is observed for walls reinforced with SE-SMA bars. The collapse probabilities at S_a of 0.46g for the 10-story steel RC wall and 0.3g for the 20-story steel RC wall reach 80% and 73%, respectively. Utilizing SE-SMA bars at wall boundaries in the plastic hinge zones significantly diminishes the collapse probability of the 10 and 20-story walls by 66% and 50%, respectively. The effect of using SE-SMA bars is more pronounced for the 10-story wall. However, the considered walls reach the same probability of collapse at seismic intensity of about 0.65g, which is significantly higher than the spectral acceleration of the maximum considered earthquake.

The fragility curves for the considered walls as a function of the maximum inter-story drift ratio (ID) are shown in Figure 14b. The 10 and 20-story steel RC walls have similar probabilities of collapse. The 10 and 20-story SE-SMA RC walls exhibit lower probability of collapse compared to the steel RC walls.

The probability of collapse against the roof residual drift ratio (RRD), which is normalized by the maximum residual drift is presented in Figure 14c. The probability of collapse is negligible for RRD less than or equal 0.3. At RRD of 1.0, the 10-story and 20-story steel RC walls suffer major damage with a probability of collapse of about 80% and 73%, respectively. The SE-SMA walls have significantly lower probability of collapse.

Results confirm the significance of considering both ID and RID when evaluating the SE-SMA RC wall fragility. At inter-story drift ratio of 2.4%, both buildings exhibit the same fragility of collapse;

whereas, the dispersion of fragility results between the SE-SMA and steel building is large at different RID ratio levels.

Figure 14. Collapse fragility curves with respect, to (a) Spectra acceleration; (b) Maximum inter-story drift ratio; (c)

roof residual displacement ratio

8. CONCLUSIONS

This paper investigates the effect of utilizing SE-SMA bars in RC shear walls designed according to the current Canadian design standards. The strain profile along the height of 10 and 20-story steel RC walls revealed two plastic hinges at the base and at the mid-height sections. The length of each plastic hinge is about 20% of the wall's height. At these plastic hinge regions, steel bars within the boundary elements were replaced with SE-SMA bars. The seismic performance of the 10 and 20-story steel and SE-SMA RC walls is then compared using multi-hazard dynamic analysis. Conclusions of this paper are listed below.

1. The use of SE-SMA bars at the wall plastic hinges improved the seismic performance over steel RC walls. This improvement was characterized by lower shear forces and bending moments and reduced residual displacements (42% average reduction). This improvement needs to be confirmed for shear walls with other heights and for other seismic ground motions.
2. Although the steel RC walls perform well under low probability seismic events, SE-SMA RC walls have significantly reduced permanent lateral deformations.
3. The efficiency of SE-SMA RC walls to recover the inter-story drifts is reduced for low-intensity seismic events and higher walls.
4. The dispersion of fragility results associated with residual drifts is considerably larger than the dispersion of fragility results associated with inter-story drifts. Hence, fragility results relying on inter-story drifts cannot be used to assess damage state in steel versus SE-SMA RC walls.
5. Steel RC walls exhibit higher fragility than SE-SMA RC walls in terms of residual drifts. This renders SE-SMA RC walls as less vulnerable to seismic damage. However, a negligible difference exists between steel and SE-SMA walls in term of inter-story drifts.

6. The used analytical model accurately predicts the wall response for the steel RC wall and the SE-SMA RC wall. However, future studies to improve the accuracy of the analytical model are needed. This includes accounting for bond slip between SMA bars and mechanical couplers.
7. Additional experimental studies are needed to confirm results of this research. Also, future studies should account for the vertical ground excitation.

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NOMENCLATURE

CMS	Conditional mean spectra
A_g	Cross section area
f'_c	Concrete compressive strength
R_0	Ductility factor
M_f	Factored moment
M_r	Factored moment resistance
l_{bl}	Length of wall boundary element
θ	Median of the fragility function
IM	Ground motion intensity
ρ_{hw}	Horizontal steel ratio in the web
ρ_{hb}	Horizontal steel ratio in the boundaries
ID	Inter-story drift
MSA	Multi strip analysis
n	Number of ground motions considered
M	Number of intensity measure levels
P	Probability of exceeding a damage level
RDD	Residual displacement ratio
x	Realized condition of the ground motion intensity measure
RC	Reinforced concrete
SE-SMA	Superelastic shape memory alloy
ε_y	Steel yield strain

Φ	Standard normal cumulative distribution
C	Specific damage level
S_a	Spectra acceleration
β	Standard deviation
R_0	Overstrength factor
ρ_{vw}	Vertical steel ratio in the web
ρ_{vb}	Vertical steel ratio in the boundaries
b_w	Wall thickness
l_w	Wall length
H	Wall height