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RESIDUAL DRIFT BASED SEISMIC DESIGN OF SHAPE MEMORY ALLOY REINFORCED CONCRETE BRIDGE PIER

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Abstract: Experience from past earthquakes and laboratory experiments suggest that structures designed according to current seismic standards are susceptible to residual deformation even during a design level earthquake. With the advancement in performance based seismic design, residual deformation has received very little attention in design codes and guidelines. Under seismic excitation, residual deformation is a direct outcome of nonlinear structural response. Researchers have developed several recentering structural systems to reduce the residual deformation during a seismic event. This paper proposes a residual drift-based design methodology for Shape Memory Alloy reinforced concrete (SMA-RC) bridge piers, which consists of defining the performance objectives, developing performance based damage states and formulating a performance based design guideline considering residual drift. The procedure anticipates the allowable residual drift based on target performance level, calculates the maximum allowable drift, and ensures that those deformation demands remain below the allowable residual and maximum drift.

1 INTRODUCTION

Over the last decade, there is growing consensus among researchers to consider residual drift as a design parameter for seismic design of structures. Observation from recent earthquakes and research results have evidenced that residual drift sustained by a structure after an earthquake plays a significant role in defining the seismic performance of a structure and need to be considered in the seismic design (Christopoulos et al. 2008, Erochko et al. 2011). Specifically for highway bridges, significant residual deformations can result in the total loss of a bridge if static incipient collapse is reached. Furthermore, they can also result in restriction of traffic loads and speed if not collapsed. These aspects are not reflected in current seismic design procedures. Researchers have developed different recentering structural systems for example, unbonded post-tensioned RC bridge columns (Kwan and Billington 2003), shape memory alloy (SMA) reinforced concrete structures (Billah and Alam 2015), and bracing systems with SMA (Haque and Alam 2017) to control or completely eliminate residual deformations. The objective of this study is to develop a performance-based seismic design guideline for shape memory alloy reinforced concrete (SMA-RC) bridge piers considering residual drift as the key performance indicator.

2 RESIDUAL DISPLACEMENT-BASED DAMAGE STATES

Performance-based seismic design largely relies on the identification and selection of proper limit/damage states. Often damage states are defined in terms of drift or displacement. Damages are usually defined as discrete observable damage states (e.g., rebar yielding, concrete spalling, longitudinal bar buckling, bar fracture) (Marsh and Stringer 2013). Although residual drift dictates the post-earthquake functionality of highway bridges, no other design guidelines except the Japanese code for highway bridge design (JRA 2006) provide any residual drift limit of bridge piers.

Five different SMAs are used in this study to develop the residual drift-based damage states for SMA-RC bridge piers. The bridge pier is assumed to be located in Vancouver, BC and was seismically designed following Canadian Highway Bridge Design Code (CHBDC 2014). Figure 1 shows the cross section and elevation of the bridge pier. The diameter of all the columns was fixed to be 1.83 m; the columns were reinforced with 48 longitudinal reinforcement of different diameter bars for different SMAs and 16 mm-diameter steel spirals at 76 mm pitch. The height of the pier is 9.14m with an aspect ratio of 5 which ensured the flexure dominated behavior.

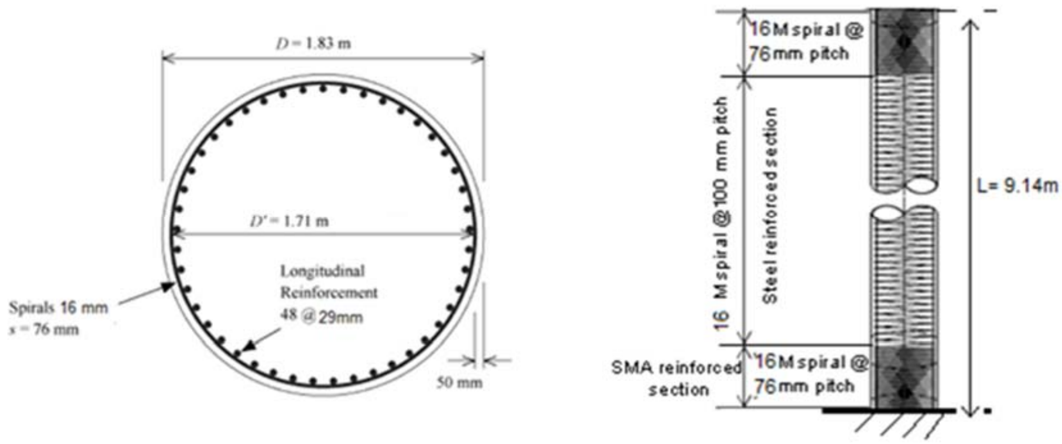

Figure 1: Cross section and elevation of SMA reinforced concrete bridge pier

In order to develop the residual drift based damage states (DS) for SMA-RC bridge pier a probabilistic approach has been adopted in this study. Based on the existing literature (O'Brien et al. 2007, Billah and Alam 2015), four different damage states have been identified and a range of limiting residual drifts were considered. It was assumed that a residual drift below 0.25% would meet the serviceability requirement (DS-1) while a residual drift larger than 1% would be characterized as a collapse damage state (DS-4). The intermediate damage states DS-2 and DS-3 are assumed to take place at a residual drift larger than 0.5% and 0.75%, respectively.

Incremental dynamic analysis (IDA) (Vamvatsikos and Cornell, 2002) was employed to determine the performance limit states of different bridge piers using an ensemble of ten selected ground motions. The incremental dynamic analyses were carried out using the 10 selected ground motions obtained from the PEER NGA ground motion database (2011).

Once the damage sates have been identified, fragility curves for residual drifts were developed using the IDA results for three different seismic hazard levels. In this study, fragility functions were developed using equation 3 which take the form of lognormal cumulative distribution functions having a median value of *θ* and logarithmic standard deviation or dispersion of *β*.

[1] $$F(RD) = \phi\left(\frac{\ln(RD/\theta}{\beta}\right)$$

where, *F(RD)* represents the conditional probability that the bridge pier will be damaged to a given DS as a function of the residual drift (*RD*); *F* denotes the standard normal cumulative distribution function; and *θ* and *β* are the median value of the probability distribution and the logarithmic standard deviation corresponding to the DS, respectively.

Figure 2 shows the fragility curves for SMA-RC bridge piers for different damage states at three different hazard levels. Here, the fragility curves are plotted irrespective of the SMA types to generalize the associated damage states. Using these fragility curves, the residual drift based damage states for SMA-RC bridge pier have been developed. From the fragility curves corresponding to each damage state, the RD value with a 50% probability of occurrence indicates the limiting value for the corresponding damage state. The limiting RD values with a 50% probability of occurrence at different damage states and hazard levels are developed as outlined in Table 1. From Table 1 it can be observed that as the ground motion return period decreases (probability of occurrence increases) the limiting residual drift corresponding to different DS decreases. Observation from Table 6 indicates that, as the damage level increases (DS-1 to DS-4) the difference in limiting RD values at different hazard levels decreases. For instance, at DS-2, the limiting RD value corresponding to 2475 years return period is 11% and 22.5% higher than that of 975 and 475 years return period, respectively. However, this difference goes down to 6.5% and 13.1% for DS-4.

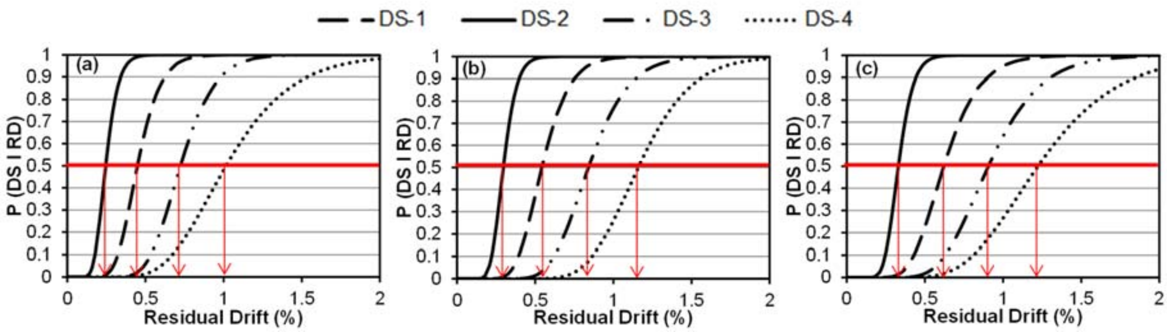

Figure 2: Fragility curves in terms of residual drift at (a) 10% in 50 years (b) 5% in 50 years and (c) 2% in 50 years probability of exceedance

Table-1: Residual drift damage states of SMA-RC bridge pier

Damage State	Functional Level	Description	Residual Drift, R_Δ (%)		
			Probability of Exceedance		
			10% in 50	5% in 50	2 % in 50
Slight (DS=1)	Fully Operational	No structural realignment is necessary	0.24	0.28	0.33
Moderate (DS=2)	Operational	Minor structural repairing is necessary	0.48	0.55	0.62
Extensive (DS=3)	Life safety	Major structural realignment is required to restore safety margin for lateral stability	0.73	0.82	0.87
Collapse (DS=4)	Collapse	Residual drift is sufficiently large that the structure is in danger of collapse from earthquake aftershocks	1.04	1.16	1.22

3 RESIDUAL DISPLACEMENT-BASED DESIGN OF SMA REINFORCED BRIDGE PIERS

The proposed design method for SMA-RC bridge piers is developed following a displacement-based approach. Unlike other displacement-based approach, the required design base shear is calculated corresponding to a target residual drift and target performance level corresponding to a selected seismic hazard. The procedure adopted in this study follows the procedure developed by Kowalsky et al. (1995) and Priestley et al. (2007), but is specifically tailored to SMA-RC bridge piers using the damping-ductility relationship developed in this study. The design steps adopted in this study are outlined in a simple flowchart in Figure 3.

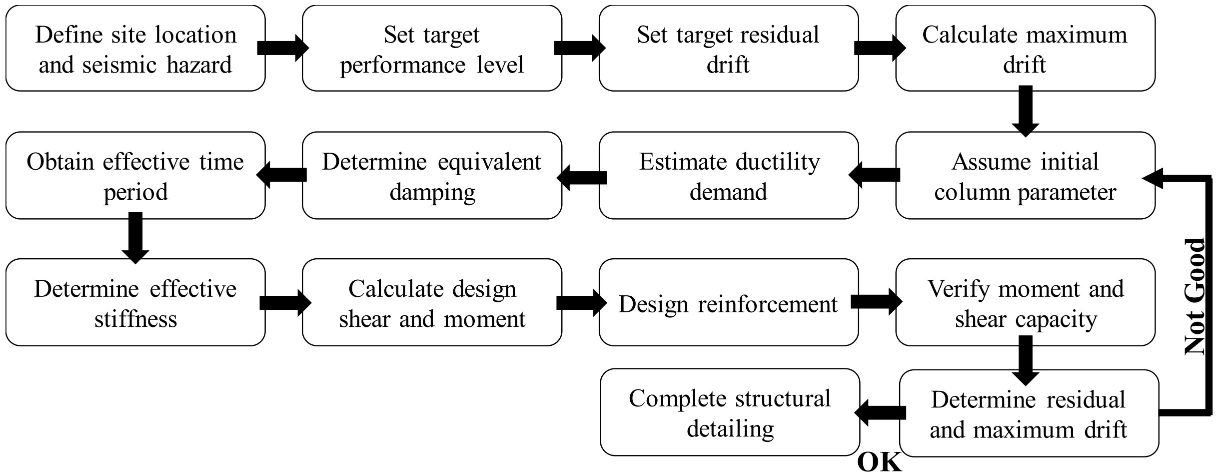

Figure 3: Flow diagram for residual drift based design of SMA-RC bridge pier

In this study, a damping-ductility relationship for SMA-RC bridge pier has been developed using a total of 100 ATC55/FEMA440 ground motions (Miranda and Garcia 2003) and following the method described by Dwairi et al. (2007). A set of new damping-ductility equations, in accordance with the previous expressions proposed by other researchers were developed in order to best approximate the damping-ductility relationship. Equation 2 represents the general form of the proposed equivalent viscous damping equation based on ductility for the SMA-RC bridge pier:

[2] $$\xi_{eq} = \xi_0 + \frac{a}{\pi}\left(1 - \frac{1}{\mu^b}\right)$$

In this equation, a and b are the two regression coefficients, and μ is the ductility demand. In order to obtain a generic damping-ductility relationship for SMA-RC bridge piers, all the examined bridge piers were considered together and the following expression was developed for the SMA-RC bridge pier:

[3] $$\xi_{eq} = 5 + \frac{32}{\pi}\left(1 - \frac{1}{\mu^{0.56}}\right)$$

The coefficient of determination or R^2 value obtained from this expression was higher than 85%.

3.1 Illustrative example

The following example is presented to demonstrate the performance-based design procedure for SMA-RC bridge piers.

The bridge pier is assumed to be located at Vancouver, BC in site soil class-C (stiff soil). The corresponding design spectrum is selected according to CHBDC-2014 which corresponds to 2% probability of exceedance in 50 years with a return period of 2475 years (Figure 4).

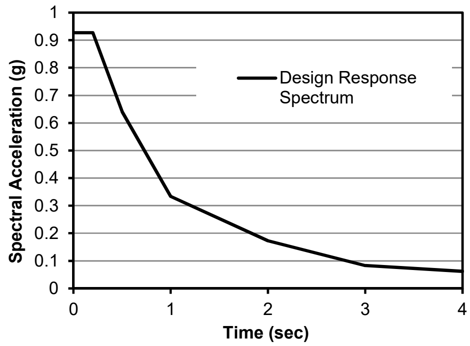

Figure 4: Design Acceleration Response Spectrum

The considered bridge is a lifeline bridge and according to CHBDC-2014 performance requirement, the bridge should be operational with limited service at the selected seismic hazard level. For the considered damage level, a target residual drift of 0.6% is selected to meet the performance objective. To restrict the residual drift within the target level, a Nitinol shape memory alloy with 6% superelastic strain (ε_s) is selected. In this design example, Nitinol SMA is considered since it is the most commonly used and commercially available SMA in the market. Moreover, a good number of research studies are also available on SMA-RC bridge piers using nitinol SMA.

Based on the target residual drift (RD) and superelastic strain (ε_s), the maximum drift (MD) is calculated using equation 3:

$$RD = 0.5 \times \left(\frac{\varepsilon_s}{100} \times MD^2 \right) - \left(\frac{\varepsilon_s}{100} \times MD \right) + \frac{1}{\varepsilon_s} \qquad [4]$$

or,

$$0.6 = 0.5 \times \left(\frac{6}{100} \times MD^2 \right) - \left(\frac{6}{100} \times MD \right) + \frac{1}{6}$$

Solving this quadratic equation we get, maximum drift, *MD* = 4.92%

Maximum displacement, Δ_m= 0.0492 × 5= 0.246 m

Initial column parameters:

Height of the pier = 5m

Lumped mass at the top of pier = 500,000 kg

Selected material properties of concrete, steel, and SMA are provided in Table-2.

Table-2: Material Properties

Material	Property	
Concrete	Compressive Strength (MPa)	42.4
	Elastic modulus (GPa)	23.1
Steel	Elastic modulus (GPa)	200
	Yield stress (MPa)	400
	Ultimate stress (MPa)	672
SMA	Modulus of Elasticity (GPa)	58.8
	Austenite-to-martensite starting stress (MPa)	401
	Austenite-to-martensite finishing stress (MPa)	510
	Martensite-to-austenite starting stress (MPa)	370
	Martensite-to-austenite finishing stress (MPa)	130

Superelastic strain (%)	6.0

For the considered hazard level, the yield drift is selected as 1.68% as developed by Billah and Alam (2016).

Yield displacement, Δy_T = 0.0168 × 5= 0.084 m

Equivalent viscous damping value corresponding to the design ductility is calculated using the following damping-ductility relationship developed in this study:

$$\xi_{eq} = 5 + \frac{32}{\pi}\left(1 - \frac{1}{\mu^{0.56}}\right) = 5 + \frac{32}{\pi}\left(1 - \frac{1}{2.93^{0.56}}\right) = 9.6\%$$

The spectral reduction factor (R_ξ) is calculated as:

$$R_\xi = \left(\frac{0.10}{0.05 + \xi}\right)^{0.5} = \left(\frac{0.10}{0.05 + 0.096}\right)^{0.5} = 0.83$$

Using the spectral reduction factor (R_ξ) of 0.83, the displacement spectrum corresponding to 9.6% damping is obtained. With this reduced displacement spectrum and the maximum displacement, Δ_m, the effective time period of the pier (T_{eff}) is calculated as 3.42 sec.

The effective stiffness (K_{eff}) based on the effective period (T_{eff}) is calculated as:

$$K_{eff} = \frac{4\pi^2 M}{T_{eff}^2} = \frac{4\pi^2 \times 500000}{3.42^2} = 1.68MN/m$$

The design base shear is calculated as: $V_{base} = K_{eff}\Delta_m = 1.68 \times 10^6 \times 0.246 = 413.3kN$

The design moment is calculated as: $M_d = V_{base} \times L = 413.3 \times 5 = 2066.5kN - m$

Finally, for the design moment of 2066.5 kN-m, the column section is designed according to CHBDC 2014 [19] considering a column diameter of 1 m. For this design moment, a longitudinal steel ratio of 1.73% is required which is provided using 28-25M SMA rebar (24.9 mm diameter) in the plastic hinge region and 28-25M steel (diameter 25.2 mm) rebar in the remaining portion. The shear reinforcement was designed following CHBDC 2014 seismic design requirements which yielded 15M spirals at 50mm pitch providing a spiral reinforcement ratio of 1.49%. The shear capacity of the column is checked using modified compression field theory (Vecchio and Collins, 1986) which predicts the experimentally determined shear failure within 1% error. The shear resistance of the pier is found to be 2264 kN which is much higher than the applied shear force.
Performance evaluation of SMA-RC bridge pier

In order to validate the proposed design approach, the performance of the designed bridge pier is evaluated using NLTHA with ten earthquake records. The bridge pier was modelled in Seismostruct, a fiber based finite element software. The bridge piers were modelled through a 3D inelastic beam–column element (force based element). The objective of this evaluation is to compare the performance objectives (residual drifts and maximum drifts) with the predicted performance under the ensemble of 10 selected ground motions.

The results of the analyses in terms of maximum and residual drifts are presented in Figures 5a and 5b, respectively. These figures show the maximum and residual drift response obtained from each nonlinear time history analysis along with the target maximum and residual drift (horizontal dotted line) used in the design.

From these figures it is evident that the bridge pier sustained maximum and residual drifts within 15% of the target maximum and residual drift. It was observed from the analysis that among ten earthquake records, two marginally exceeded the target residual drift of 0.6% and maximum drift of 4.92%. The remaining eight are below the design level residual drift and targeted maximum drift. These minor discrepancies can be attributed to the linearization of the displacement spectrum adopted during the design and scaling of ground motions. However, the average response in terms of both residual and maximum drifts was very close to the targeted drift levels. Priestley et al. (2007) suggested that the differences in the target drift and obtained drift from NLTHA is acceptable if the mean of the peak drifts remains close to the design drift.

4 SUMMARY AND CONCLUSIONS

This study presented a new residual drift based design method for shape memory alloy reinforced concrete bridge piers. The approach outlined in this paper is a comprehensive approach for performance-based design of SMA-RC bridge piers. This study developed necessary design equations and graphs for PBSD of SMA-RC bridge piers. The proposed method provides the owner to select expected performance of the bridge pier and allows the designer/engineer to select multiple hazard and performance expectation combinations. Following the DDBD guidelines of Priestley et al. (2007) the authors developed their own design method and damping-ductility relationship for SMA-RC bridge piers. In contrast to the conventional DDBD approach, the proposed procedure anticipates a target residual drift based on the expected performance during design earthquake, calculates the maximum drift demand, and ensures that those drift demands (maximum and residual) remain below acceptable limits for the design level earthquakes. The performance of the bridge pier was validated using NLTHA, and the maximum and residual drifts at the design level earthquakes were found to satisfy the performance expectations.

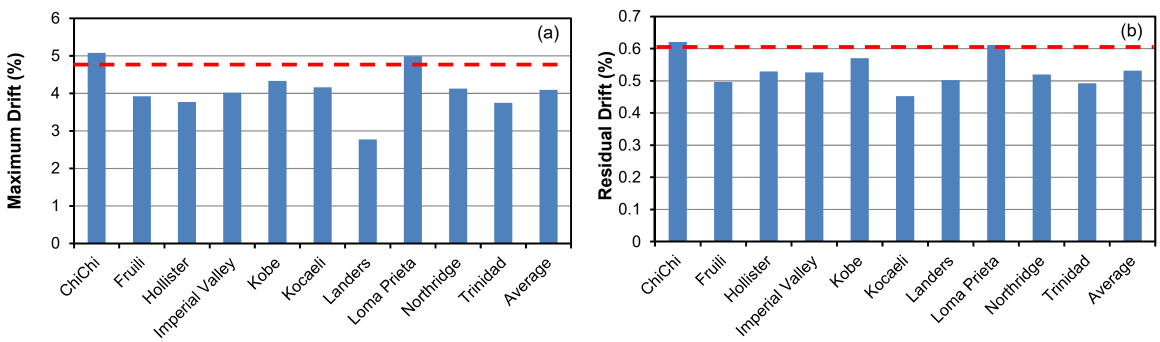

Figure 5: (a) Maximum and (b) residual drift value obtained from time history analysis of the designed pier (Red line showing the target maximum and residual drift)

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