Seismic performance of EBFs equipped with an innovative shape memory alloy damper

Nadia M. Mirzai and Reza Attarnejad *

School of Civil Engineering, College of Engineering, University of Tehran, P.O. Box 4563-11155, Tehran, Iran
* Corresponding author. Tel.: +98 9121439530, E-mail: attarnjd@ut.ac.ir

Abstract

Given their unique characteristics, Shape Memory Alloys (SMAs) have significant potential for use in different areas of engineering. The phase shift characteristics of these alloys allow them to memorize a certain shape, and if deformed, revert back to that shape through a thermal process. Given the vast potentials of SMAs, they can be utilized to address the limitation of conventional eccentrically braced frames (EBFs) with vertical links in order to achieve better residual and maximum interstory drifts. This paper presents a vibration control system equipped with SMAs to achieve improved operational domain. The Compared to conventional EBFs, the proposed system named recentering damping device (RDD) is easy to fabricate and implement and allows for the redesign of fuse members. A numerical analysis is performed for a 9-story steel frame building using nonlinear analysis program OpenSees to evaluate the system performance. Results of time history analysis demonstrate better self-centering behavior and lower residual interstory drifts of the proposed system as compared to EBF.

Keywords: Shape memory alloys; damper; residual drift; eccentrically braced frames.

1. Introduction
Eccentrically braced frames (EBFs) are lateral force resisting systems that provide not only desirable elastic stiffness but also manageable inelastic behavior realized through controlled deformation of the link member under heavy loads such as earthquake motion. The history of research on this controlled deformation under seismic loads can be traced back to the works published about three decades ago by [1-3].

The most important disadvantage of horizontal EBFs is the difficulty of repairing the deformed link member. To resolve this issue, researchers such as Aristizabal-Ochoa [4], Ghobarah and Abou Elfath [5] and Vetr et al. [6] proposed the use of vertical EBFs, in which a shear panel serves as a vertical link between the story beam and the \( \Lambda \)-shaped braces. This approach not only facilitates the repairing of damaged links but also easier to repair in their horizontal counterpart. This approach also provides a convenient solution for the cases where the presence of extensive gravity loads necessitates ensuring that floor beams remain in elastic region. The disadvantage of this approach is that it may be sometimes difficult to provide lateral bracing for vertical links [7].

Shape memory alloy (SMA) is an alloy that can memorize a certain shape, and if deformed, can recover the memorized shape when heated. SMAs have extensive potential for use in several industries, and particularly in structures, automobiles, and aerospace vehicles [8]. Given the superelastic behavior, recentering ability, and damping capacity of SMAs, their potential use in the vibration control of structures has been the subject of many studies [9-26]. A comprehensive review of civil engineering applications of SMAs can be found in [27, 28]. Regardless of their potential, using these alloys in real structures needs a perfect understanding of their mechanical behavior in different scenarios, which still requires further research on the subject [29]. This is also true in the case of EBFs, as the possible behaviors of SMAs in these frames are still under investigation [23].
In passive vibration control schemes, energy dissipation mechanisms are expected to reduce the demand on the primary structural members as well as any consequent plastic deformation. Meanwhile, the recentering mechanism attempts to return the structures to its original geometry, thereby preventing the aggregation of inelastic deformations [30]. The problem in using SMAs in EBFs is the low energy dissipation capacity of recentering mechanisms, especially under high rate loadings, which is hard to reconcile with almost completely linear elastic behavior of SMAs. One approach to resolving this issue is the use of Nitinol-based devices proposed by Dolce et al. [31, 32], which provides a good combination of recentering capability and energy dissipation. Some other researchers demonstrated that the combination of base isolation and SMA leads to more structural integrity and concluded that re-centering LRB isolators causes an improvement of seismic performance in terms of energy dissipation and re-centering influence (e.g. [33]).

Haque and Alam [34] presented a piston based bracing which is able to recentering after large deformation. A study by DesRoches et al. has investigated the use of SMA bars in the beam to column connections of steel moment resisting frames and the seismic performance of the resulting system [35]. Ellingwood et al. have also studied the seismic demand of steel moment resisting frames with SMA connections, but have used a probabilistic seismic demand assessment approach for this purpose [36]. The applications of SMAs in the vibration control systems typical to other domains of civil engineering have also been researched [23, 37-41]. Moreover, several researchers employed the special properties of SMAs to improve the response of structures. For example in the references [16, 23, 41, 42] it can be seen that using SMAs could significantly reduce the response of structures in terms of interstory drift ratios, residual drifts and absolute acceleration of floors.
This paper proposes a recentering damper with enhanced self-centering and energy dissipation capabilities that is simple to setup and can be utilized for seismic design of new structures as well as retrofitting of existing structures. The paper also provide the results of tests conducted to determine the impact of displacement amplitude and frequency on the mechanical behavior of the device and the shaking table tests performed to evaluate its ability to control the seismic response.

2. SMA Characteristics

2.1 Phase Transformations
Shape memory alloys have two stable phases. The first phase, martensite, is the state where alloy has high stress and low temperature, and the second phase, austenite, is the state where alloy has low stress and high temperature. Martensite phase itself has two variants, twinned and detwinned. Therefore, a change in stress, temperature, or both can trigger a phase transformation in these alloys. The strain corresponding to the phase transformation is called transformation strain. In Figure 1, the transformation fronts are portrayed on the $T$-$\sigma$ plane. This phase transformation is the source of shape memory effect and pseudoelasticity that make SMAs a fascinating and potentially rewarding subject of research. Researchers such as reference [43] have identified more phases for SMAs to model other effects, but this is beyond the scope of the present paper.

2.2 Shape memory effect (SME)
Shape memory effect can be described as the process that SMA undergoes to memorize a shape and then recover that shape when heated. As shown in Figure 2, this effect is realized in the four
following stages:

**Stress application:** Subjecting the alloy to stress rearrange the twinned martensite into detwinned martensite, but leave a transformation strain.

**Stress removal:** Removing the stress eliminates the elastic strain but not the transformation strain, as the alloy remains in detwinned martensite phase.

**Heat application:** Once heated, the alloy undergoes a phase shift from martensite to austenite, which results in the recovery of transformation strain.

**Heat removal:** After removing the heat, the alloy undergoes another phase shift from austenite to twinned martensite.

At the end of fourth stage, the plastic strain will be removed and the alloy will recover its initial strain state.

3. **Device configuration and mechanism**

The mechanical layout of the SMA-reinforced lead rubber damper is displayed in Figure 3. As can be seen, this recentering damping device (RDD) is a lead rubber bearing equipped with SMA bars, top and bottom stiffeners that work in opposite directions, and steel plates that connect it at the top and bottom to the structure. Two yellow plates are in front of each other but one has been installed on the top plate and the other one on the bottom plate. It causes the yellow plates can get away and apply tensile force to the SMA bars. If the plates move away from each other, the device will create a tension force and a flag-shaped hysteresis, which, alongside the hysteresis of the lead core, will result in a large energy dissipation. The plates moving toward each other will create no compression force in the bars, as they can slip into the slots embedded for this purpose.

The SMA bars are connected to the steel palates using nuts. Some researchers conducted experimental tests equipped with the SMA bars in their model using nuts (e.g. [45, 46]).
4. Verification study
To ensure the accuracy of the modeling, all parts of the modeling have been verified by the experimental tests available in the literature. The verification study includes two different parts: 1) an SDOF vertical shear link studied by Bouwkamp et al. [47] and 2) modeling of the SMA material tested by Desroches et al. [48]. More details are addressed as follows.

4.1 Verification study of an SDOF vertical shear link
A one-story vertical shear link which is subjected to a cyclic loading in which two-cycle is applied at the displacement levels of $\pm 1.5 \text{ mm}$, $\pm 2.0$, $\pm 3.0$, $\pm 3.5$, $\pm 4.0$, $\pm 5.0$, $\pm 6.0$, ..., $\pm 10.0$, $\pm 12.0$, ..., $\pm 34.0$. The details of the SDOF frame is listed as the Table 1. Figure 4 illustrates the well matching between the results of the experimental test and OpenSees.

4.2 Verification study of SMA behavior
To evaluate the accuracy of the simulation of the SMA bar in OpenSees platform, the behavior of the material has been verified by the study of Desroches et al. [48]. Figure 5 shows the comparison between an SMA bar with 25.4 mm diameter and OpenSees which demonstrates the applied parameters in the numerical model are adopted, accurately.

5. Frame design and modeling
The performance of the system was evaluated using the models of 9-story steel buildings designed for a seismically active site (Tehran) composed of soil type III with $V_{30}=175$-375 m/s according to Iranian code of practice for seismic resistant design of buildings [49]. Every story
was assumed to have a height of 3.2 m. Buildings were designed using the gravity loads typically assumed for residential buildings in Iran. An image of the designed structures is displayed in Figure 6. The specifications of the buildings are provided in Table 2. To have a reasonable comparison, the natural period of both systems should be almost same. It has been tried the natural period of the RDD system to be close to the conventional EBF by controlling the amount of SMA in the structure. The natural period of the conventional structure and the RDD braced frame is 1.0 sec and 0.94 sec, respectively.

The buildings were modeled and analyzed in the OpenSees software. All models have been simulated based on a verified model in OpenSees [47, 50]. Internal frame of all structures was subjected to 2D nonlinear static and dynamic analyses. Steel behavior was modeled with the help of OpenSees material library by assigning the elements with desired bilinear kinematic stress–strain curve. To ensure that there would be no jump in local stiffness of elements or in the transition between elastic and plastic regions, a transition curve was defined for the tangent moduli (the intersection of the first and second tangent). Beam and column cross sections were modeled using a combination of displacement based beam-columns and fiber sections. According to [51, 52], force-based elements have an inherently lower stability than displacement-based elements. In the software, P-delta transformation of geometric stiffness matrix was used to make sure that P-delta effects are incorporated into the analysis.

The details of the lead rubber bearing which is used in the nine-story building are listed in Table 3.

In which $V$ is the vertical load, $K_e$ is effective horizontal stiffness, $x_e$ is equivalent viscous damping coefficient, $d_1, F_1, d_2, F_2$ define the bilinear curve, $D_g$ is external elastomer diameter, $t_e$ is total elastomer thickness, $h$ is height excluding outer steel plates, $H$ is total height
including outer steel plates, and \( Z \) is side length of outer steel plate. As can be seen in Table 3, the proposed hybrid device in this paper has a damping ratio of 29\% which leads to considerable reduction in the response of the structure.

Ricles and Popov [54] recommended that a form of nonproportional viscous damping should be employed to prevent unrealistic larger axial forces in braces and columns which are besides the shear links. For this aim, they recommended that Rayleigh damping is applied to all members except shear links. In this paper, this recommendation was employed such that the “Rayleigh Damping Command” in OpenSees was assign to all elements except the shear links using “Region Command”.

6. Performance assessment

The lateral load carrying capacity was evaluated by applying lateral loads both statically and dynamically on the models.

6.1 Pushover curve

The lateral strength and post-yield behavior was assessed through a static pushover analysis. Figure 7 presents the base shear vs. maximum roof displacement pushover diagram obtained for the 9-story model. To make sure that the inherent response of the building to lateral loading is accounted for, a load pattern based on the structure’s fundamental period was used to apply loading in a displacement-controlled state.

These results clearly demonstrate the proposed system has a better performance due to its higher capacity.

6.2 Nonlinear time history analysis

Beyond the nonlinear static analyses, nonlinear response history analyses were performed to further evaluate the inter-story drift responses of the system. This section discusses the
maximum inter-story drifts, the maximum residual inter-story drifts and residual displacement obtained from the nonlinear response history analyses of systems with and without proposed damper.

To employ the nonlinear time history analysis, ten far-field records are selected from FEMAP695 with the magnitude between 6.5 through 7.5. All the ground motions time histories have been scaled to 0.55g based on a method suggested by Iranian code of practice for seismic resistant design of buildings [49]. Figure 8 shows the scaling procedure according to Iranian seismic code. Also, the considered soil type for Iran is D (stiff soils) based on Geomatrix soil class in USGS (US Geological Survey). The details of the used ground motions are specified in Table 4.

6.2.1 Maximum interstory drift ratio (IDR)

Global and local interstory deformation are one of the causes of induced damage to the structure. Even elastic deformation of the structure may induce some damage to the nonstructural components. Therefore, a greater deformation will cause not only the nonstructural damage but also lead to the series structural damage. Error! Reference source not found.9 shows that the maximum interstory drift has been decreased up to 57%.

The response of the 9-story building subjected to one ground motion (record seq. No. 900 in Table 4) is shown in Error! Reference source not found.10 as an example.

6.2.2 Residual drift

The inelastic deformation caused under earthquake loads may lead to significant residual deformation which increases time and cost of the repair of the structure and, in some cases, leads to the structure to become unusable or to collapse, either partially or totally. As can be seen in Error! Reference source not found.11 and Error! Reference source not found.12, the RDD device can significantly decrease the residual drift in 9-story building. Error! Reference source
not found. 12 shows the residual drift of the two structures under Landers earthquake (see Table 4). The re-centering effect can be clearly seen in Error! Reference source not found.12. Based on this figure, 9-story conventional building is not usable after the earthquake and will probably collapse, partially or totally. However, in the RDD braced frame, the maximum residual drift is almost 0.2% in the 9-story building, therefore the rehabilitation is reasonable and affordable.

6.2.3 Residual displacement

Residual displacement has an important effect in judging the post-earthquake safety of buildings, as well as in decision on the economic possibility of repair and reconstruction. Error! Reference source not found.13 illustrates the effect of the RDD device on mitigation of the residual displacement. As can be seen in Figure 13, the residual displacement has been decreased. Also, the residual displacement is more uniform in both buildings. Time history response of buildings is shown in Error! Reference source not found.14. It can be observed that the RDD device has a significant effect on the reduction of the residual displacement of the structures. Uniform drift can be related to uniform demand capacity stiffness ratios [55].

7. Conclusions

A recentering damping device (RDD), which includes an EBF equipped with SMA that acts as a rapid repair fuse, was proposed and numerically evaluated in this study. A nine-story steel frame was considered numerical models. A nonlinear time history analysis was conducted using 10 different ground motions simulated in the OpenSees platform. The main findings of the present study are summarized as follows.

1. The simulation results illustrate that RDD can mitigate maximum inter-story drift ratio
by 57% in the nine-story building.

2. Compared with conventional EBFs, the RDD system effectively reduces residual drift by up to 86.99% for the nine-story buildings at different ground motion records.

3. The RDD system also produces minimal residual displacement for all 10 ground motion records.

4. Considerably lower residual drifts that were observed in the proposed RRD system demonstrate that the repair costs of the steel buildings equipped with RRD are lower than those of the traditional steel frames after an earthquake.

5. The pushover curve shows that the RRD system has larger capacity in comparison with the conventional system.

6. However SMA material is expensive, their cost can, in fact, be kept quite modest by using reasonably priced materials.

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Figure captions

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Table 1. The details of the SDOF frame tested by Bouwkamp et al. [47]

<table>
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<tr>
<th>Specimen No.</th>
<th>Beam section</th>
<th>Vertical shear link section</th>
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<td>3</td>
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Table 2. The structural details of nine-story IY-Shaped brace frames

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<td>9-Story</td>
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Table 3. The details of LRB in nine-story building [53]

<table>
<thead>
<tr>
<th>LRB-S</th>
<th>$V$ (kN)</th>
<th>$K_v$ (kN/mm)</th>
<th>$\lambda_v$</th>
<th>$F_2$ (kN)</th>
<th>$F_1$ (kN)</th>
<th>$d_1$ (mm)</th>
<th>$d_2$ (mm)</th>
<th>$D_x$ (mm)</th>
<th>$t_e$ (mm)</th>
<th>$h$ (mm)</th>
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<td>3.2</td>
<td>29</td>
<td>267</td>
<td>155</td>
<td>10</td>
<td>83</td>
<td>800</td>
<td>128</td>
<td>223</td>
<td>283</td>
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Table 4. Far-field ground motions from FEMA P695

<table>
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<tr>
<th>Record Seq. No.</th>
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<th>NEHRP Class</th>
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<th>Name</th>
<th>Recording Station</th>
<th>Magnitude</th>
<th>PGA(g)</th>
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<td>D</td>
<td>1994</td>
<td>Northridge</td>
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<td>309</td>
<td>D</td>
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<td>Northridge</td>
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<td>6.7</td>
<td>0.48</td>
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<td>D</td>
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<td>Landers</td>
<td>Yermo Fire Station</td>
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<td>752</td>
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<td>1989</td>
<td>Loma Perieta</td>
<td>Capitola</td>
<td>6.9</td>
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</table>

Nadia M. Mirzai received her BSc degree in Civil Engineering from Shahid Bahonar University of Kerman in 2009, MSc degree in Structural Engineering from Shiraz University in 2011. She is currently a Ph.D. candidate of Structural Engineering in University of Tehran, working in fields such as applications of smart materials in earthquake engineering and passive control systems. Ten contributions have been published by her in international journals and conferences.

Reza Attarnejad received his Ph.D. degree in 1990 from Polytechnic University of Catalonia (UPC) in Spain focused on computational mechanics on continuous media. He is currently the (full) Professor of Structural Engineering Division of Civil Engineering Department at University of Tehran and also was the former chairman of the division. He has extensive publications on his specialties which are fluid-structure interaction, structural dynamics, FEM, applied mathematics, and semi-analytical methods. He has published near 70 papers in the ISI indexed international journals.