Multi-Objective Optimization of Three Different SMA-LRBs for Seismic Protection of a Benchmark Highway Bridge against Real and Synthetic Ground Motions

Reyhaneh Hosseini 1,*, Maria Rashidi 1,*, Borko D. Bulajić 2 and Kamyar Karbasi Arani 1

1 Centre for Infrastructure Engineering, Western Sydney University, Penrith, NSW 2751, Australia; k.karbasi@westernsydney.edu.au
2 Department of Civil Engineering, University of Novi Sad, 21000 Novi Sad, Serbia; borkobulajic@uns.ac.rs

* Correspondence: r.hosseini@westernsydney.edu.au (R.H.); m.rashidi@westernsydney.edu.au (M.R.)

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Abstract: Many researchers have taken advantage of adding shape memory alloy (SMA) wires to base isolators to control displacements and residual deformations. In the literature, different arrangements of SMA wires wrapped around the rubber bearings can be found, as examples, straight, cross and double-cross arrangements. SMA wires with various configurations and radii lead to the different characteristics of the isolator system and thus various shear hysteresis. Therefore, the aim of this study is to evaluate the performance of these three SMA wire’s configurations in the seismic retrofitting of a benchmark highway bridge by implementing them in the bridge’s existing lead rubber bearings (LRB). This system is referred to as SMA-LRB isolator. Firstly, because of the crucial influence of the wire’s radius, this parameter is determined using a multi-objective optimization algorithm (non-dominated sorting genetic algorithm (NSGA)-II). This algorithm simultaneously minimizes the deck acceleration and mid-span displacement. Secondly, the optimized SMA-LRBs are implemented in the highway bridge and nonlinear dynamic analysis is conducted. For the nonlinear response history analysis, two strong ground motion records are selected from the PEER database, by studying the site’s conditions. In addition, ten synthetic ground acceleration time histories are generated. The result illustrates that the double-cross SMA-LRB reduces the maximum and residual displacements more than two other devices; however, it causes the largest base shear force and deck acceleration. Besides, the cross-configuration results in the least displacement reduction and has the least shear force and acceleration. To find SMA-LRB with the best overall performance, a multi-objective decision-making method is utilized and the straight SMA-LRB is recognized as the most effective isolator.

Keywords: passive control; base isolation; multi-objective optimization; NSGA-II; synthetic ground motions; shape memory alloy; iron-based shape memory alloy; benchmark highway bridge

1. Introduction

Bridges play a critical role in transportation networks, and any disruption in their function causes huge direct and indirect economic and life losses. Thus, it is of paramount importance to protect bridges against earthquakes or other severe loadings. In recent major earthquakes such as Northridge 1994 [1], Kobe 1995 [2], Chi-Chi 1999 [3], Wenchuan 2008 [4], Chile 2010 [5], and Great East Japan 2011 [6] a variety of damage types have occurred in highway bridges and proved the limitation of the conventional design methods. The conventional seismic design approaches are based on the dissipation of the input earthquake energy through partial inelastic deformations in the designed areas of the structural elements, and at the same time, maintaining the overall stability and integrity of
the structure [7]. In the current design practice, the bridge superstructures are mainly designed to stay in their elastic region during earthquakes, and foundation failure is hard to assess. Therefore, only the bridge substructure is allowed to dissipate input energy through inelastic deformations. However, unlike buildings, bridges lack redundancy as a result of their simple structural configurations. Due to the lack of redundancy, bridges cannot provide adequate ductility demand enforced by large earthquakes [8]. To tackle this problem, over the past decades, researchers have focused on employing vibration control strategies to either reduce the earthquake force acting on a structure or to absorb the seismic energy [9].

Amongst various control strategies, base isolation is a prevalent technique for mitigating seismic response and is applied in bridges worldwide. Seismic isolation aims to lengthen the natural period of a structure in order to reduce the applied seismic loads and provide additional damping capacity to the structure. Different types of isolation systems have been suggested and developed for various structures. Commonly adopted systems are rubber bearing (RB), lead rubber bearing (LRB), high damping rubber bearing (HDRB), friction pendulum (FP), and resilient friction bearing, amongst which LRB is the most broadly used device [10]. However, base isolations have several downsides. The major one is the occurrence of the large displacements under strong ground excitations, which may result in the pounding of adjacent parts or even unseating, instability due to large deformation, unrecovered residual deformation, and an urgent need for the replacement of bearings after strong earthquakes [11]. A novel way to minimize excessive displacements is to employ intelligent materials, such as shape memory alloys (SMAs), to produce additional damping and re-centering ability.

SMAs are intelligent materials with extraordinary properties. The eminent properties of these materials are super-elasticity and shape memory effect [12]. In the super-elastic behavior, the strain due to loads can be recovered entirely after unloading. However, in the shape memory effect, the initial shape can be returned by heating [13]. Since Bohler first found Nitinol in 1963, various types of SMAs have been presented [14]. The most common ones are nitinol-based, copper-based [15], and iron-based [16] SMAs. Recently, an iron-based alloy (i.e., FeNiCoAlTaB) was found which is able to hold 13.5% super-elastic strain [16].

In the last two decades, many researchers have used SMAs to improve the seismic performance of base isolators due to their high damping capacity, erosion resistance, fatigue strength, and high level of super-elastic strain. Wilde et al. [17] took advantage of two different systems, including a rubber bearing with an SMA bar damper and a lead core rubber bearing (LRB) with lateral bracing named NZ device. By comparing these systems, it was observed that the first system has more damping capacity, and because of using SMA, this system was able to recover its deformation. However, the force applied to the base isolator with SMA was three times more than the other one. To control the instability and residual deformation in a three-span continuous highway bridge, Choi et al. [18] took advantage of SMA wires, which were wrapped around an elastomeric bearing. Findings proved that the rubber bearing with Nitinol shape memory alloy was able to decrease the deck displacement, and no residual displacement was observed. Nevertheless, in extensive shear deformations (200 percent), the Nitinol wire experienced plastic strains, and the device was not able to work successfully anymore. Andrawes and DesRoches [19] also introduced and compared the performance of an SMA retrofit device with three other devices in limiting the displacements at the bridge’s intermediate joints. They concluded that the super-elastic behavior of the SMA is more effective in minimizing the joint opening. Hedayati Dezfuli and Alam [16] numerically investigated two different configurations of SMA wires in conjunction with elastomeric bearings. They studied the effects of various parameters such as shear strain amplitude, SMA type, dimension ratio of the base isolation, and the level of pre-strain of the SMA wires on the seismic behavior of base isolators. According to their results, the wire diameters should be selected based on the lateral stiffness and equivalent viscous damping to achieve the high performances. Zhu and Qiu [20] proposed a novel SMA self-centering isolator to protect highway bridges against earthquakes. The results illustrated that the SMA isolators are able to protect the superstructure of the bridge and limit the residual deformation. Hedayati Dezfuli and
Alam [21] assessed the seismic vulnerability of bridges isolated by natural rubber bearings with SMA wires and concluded that SMA wires can increase the reliability of the bearings and the bridge. Xiang and Alam [22] investigated four different retrofitting devices to improve the seismic performance of an LRB-isolated bridge, namely yielding steel cables (YSCs), viscous dampers (VDs), friction dampers (FDs), and super-elastic shape memory alloy cables (SMAs). The results indicated that the SMA device was better than the other measures at self-centering capability and led to the least residual superstructure displacement. Hosseini et al. [23] optimized the radius of the SMA wire wrapped around the lead rubber bearings of a benchmark bridge using a multi-objective optimization algorithm. According to their study, optimized SMA isolators can noticeably reduce the maximum displacement and residual deformation of the bridge and keep the base shear and deck acceleration less than those of the non-isolated benchmark bridge.

In the literature, the effectiveness of adding SMA wires to the LRBs of the bridges in controlling displacement and residual deformation is well studied. However, the comparison of different wire’s arrangements with optimum radii was not investigated. Researchers have wrapped SMA wires around the LRBs in different configurations, namely, double-cross [24,25], straight [18], and cross [16,26], each of them leads to specific characteristics of the isolator system. Besides, studies show that as a side effect of employing SMA wires into the LRBs, the base shear and deck acceleration increase. The more the radius of the wire, the more increase occurs in the acceleration and shear force, and the more decrease occurs in the displacements. To alleviate this problem, in a previous study by Hosseini et al. [23], a multi-objective non-dominated sorting genetic algorithm (NSGA-II) was utilized to find the optimum wire radius by simultaneously minimizing the base shear and the mid-span displacement. However, only the double-cross arrangement was considered. Thus, here, as a supplemental work of the previous study, the goal is to evaluate various optimized configurations of SMA wire to find the best arrangement of wires for retrofitting of the benchmark highway bridge. Thus, firstly, the NSGA-II algorithm is utilized to simultaneously minimize the deck acceleration and the mid-span displacement and find the optimum wire radius for each configuration. Then, the optimized SMA-LRBs are implemented in the benchmark highway bridge to assess and compare their seismic performance. A further contribution of this research is that instead of selecting ground motions from places other than the bridge’s site and modifying them considerably, strong motion records that match the site conditions are carefully picked out and ten synthetic ground acceleration time histories are generated to be used in the nonlinear response history analysis.

2. Benchmark Highway Bridge

The highway bridge considered in this study is a benchmark bridge. For many years, researchers have suggested control systems and implemented them in different bridges; therefore, it was hard to compare their results. For facilitating comparison among various control devices, the ASCE Committee on structural control in 2003 decided to develop a benchmark structural control model for highway bridges. This model is based on an actual bridge in Southern California.

The selected bridge is 91/5 freeway overcrossing located in Orange County of Southern California, shown in Figure 1. This is a continuous two-span, cast-in-place prestressed concrete box-girder bridge supported by an outrigger bent at the center. The Whittier–Ellsinore fault is 11.6 km (7.2 miles) to the northeast, and the Newport–Inglewood fault zone is 20 km (12.5 miles) to the southwest of the bridge. The bridge has two spans of 58.5 m (192 ft) long and has two abutments skewed at 33°. The width of the deck along the east span is about 12.95 m (42.5 ft), while along the west span is about 15 m (49.2 ft). The cross-section of the deck consists of three cells. The deck is supported by a 31.4 m (103 ft) long pre-stressed outrigger, which rests on two pile groups, each consisting of 49 driven concrete friction piles. The columns are approximately 6.9 m (22.5 ft) high. Readers are referred to the definition paper for more information about the benchmark bridge [27].
A three-dimensional finite element model of this bridge was developed using ABAQUS software, in two phases. In phase I, the semi-isolated model, the deck of the bridge is fixed to the center outrigger, and eight seismic isolators are applied to separate the bridge from the end abutments [27]. In phase II, the completely isolated model, two lead rubber bearings are installed between the deck and the center outrigger in addition to the eight aforesaid isolators [28]. Many researchers have implemented their control systems in phases I and II of the benchmark bridge. For example, Tan and Agrawal [29] presented sample designs for three control systems and applied them in phase I. Madhekar et al. [30–32] investigated the performance of different passive and semi-active devices on the phase I bridge. Saha et al. [33–35] also applied passive control systems on this bridge. Regarding sample controllers in phase II, Zhang et al. [36] proposed a damper with SMA for controlling the displacements and studied the impact of temperature changes on the SMA behavior. Casciati et al. [37] suggested a new SMA device as a passive damper to reduce the maximum displacement of the base isolators. Li et al. [38] investigated the performance of a novel negative stiffness device (NSD) and a damper system in seismic control of the benchmark bridge. In this research, the completely isolated benchmark bridge in phase II is applied, which is modelled by Nagarajaiah et al. [28]. A brief review of the bridge modelling is provided in the following section.

2.1. Structural Model

To model the completely isolated highway bridge, Nagarajaiah et al. [28] took advantage of ABAQUS finite element software. The model has a total of 108 nodes, 4 rigid links, 70 beam elements, 24 springs, 27 dashpots, and 10 user-defined bearing elements. The superstructure, including the deck and the beam, is assumed to be elastic; also, the abutments and deck-ends are assumed to be rigid and have a skew angle of 33°. Nonlinearities are included in members to capture inelastic moment-curvature behavior of columns and the shear-displacement relationship of bearings. The bearings are idealized using the bi-directional bilinear plasticity model, as described by Makris and Zhang [39], and defined as a subroutine in ABAQUS. The bearings are modelled as a shear element, which means they express infinite vertical stiffness and zero torsional rigidity and bending stiffness. In addition, the mass and damping of these bearings are eliminated. The soil–structure interaction is considered at the end of abutments and embankments, and the pile foundations at both abutments are assumed to behave like an equivalent linear visco-elastic element. More details on this model, in conjunction with their dynamic properties, are described in the definition paper [27].

After developing the three-dimensional model of the bridge with 430 degrees of freedom, all element mass matrices and initial elastic element stiffness matrices obtained are summed at nodal masses to assemble global stiffness and mass matrices within MATLAB environment. The global damping matrix C is a combination of the distributed 5% inherent Rayleigh damping in the first two
modes and soil radiation damping. The fundamental period of this phase II benchmark isolated bridge is 5.1 s. The dynamic equation of this model is expressed in the form of Equation (1).

\[
M \ddot{U}(t) + C \dot{U}(t) + K(U(t)) = -M \ddot{y}(g) + bF(t)
\]  

In this equation, the incremental displacement, ground acceleration, the incremental controlling force, mass, and stiffness matrices are denoted by \(U\), \(\dot{U}\), \(F\), \(M\), and \(K\), respectively. Note that the stiffness matrix of the structure includes linear and nonlinear sub-matrices. \(\eta\) and \(b\) are the loading vectors for the ground acceleration and control forces, respectively. Equation (1) can be solved with the help of the Newmark integration method. MATLAB software has various tools for solving ordinary differential equations; however, they are not able to find the nonlinear structural responses. Thus, Nagarajaiah et al. [28] developed an S-Function program in the Simulink for the bridge model. Figure 2 shows the Simulink of the passive control device. Researchers can modify the contents of the ‘control device’ to accommodate their control designs.

![Phase II Benchmark Control Problem for Seismically Excited Highway Overcrossing](image)

**Figure 2.** Passive Simulink model of the benchmark bridge.

### 2.2. Earthquake Excitations

In this study, for the nonlinear dynamic analysis of the bridge under earthquakes, the nonlinear response history analysis is carried out by selecting a set of ground motion time histories. For this purpose, researchers often select ground motion time histories from a database of past recorded earthquakes. However, recorded motions are not always available for all types of earthquakes and in all regions. Therefore, researchers often have to choose records from locations other than the project site and modify these records by scaling or spectrum matching techniques to fit their desired conditions. Sometimes, they are forced to scale recorded motions by factors as large as 10 or 20 or modify the frequency contents to achieve desired intensity or frequency characteristics [40]. Such modified motions may not accurately characterize real earthquake ground motions. To tackle this deficiency, researchers have developed techniques for the generation of synthetic ground motions that simulate realistic ground motions for specific design scenarios [40,41].

In the benchmark bridge package, six ground motions from past earthquakes were selected without scaling or spectrum matching [27]. These earthquakes were North Palm Springs (1986), TCU084 component of Chi-Chi earthquake Taiwan (1999), El Centro component of Imperial Valley earthquake (1940), Rinaldi component of Northridge earthquake (1994), Bolu component of Duzce Turkey earthquake (1999), and Nishi-Akashi component of Kobe earthquake (1995). Because of the weaknesses of selecting ground motions from inappropriate locations with unreasonable scaling or spectrum matching, in this study, strong motion records are carefully selected to match site conditions.
Besides, 10 synthetic motions are generated to be used along with the real records in the nonlinear response history analysis. The approach for selecting real records of strong ground motion and generating synthetic records are presented in the next section.

3. Ground Motion Time Histories

For nonlinear response history analyses and the optimization algorithm in this study, instead of selecting input ground motion time histories from past earthquakes in areas other than the bridge site and modifying them significantly to match the current conditions, real records of strong motions are utilized that resemble the following conditions:

1. Distance to the causative fault approximately the same as the distance of the bridge to the closest active seismic fault.
2. Size of the earthquake equal to the probable maximum magnitude at the closest fault.
3. Soil conditions and deeper geological surroundings of the recording station similar to those beneath the bridge.

The bridge is located in Orange County of Southern California, and the two closest fault zones are the Whittier–Ellsinore, which is 11.6 km to the northeast, and the Newport–Inglewood, which is 20 km to the southwest of the bridge. Both faults are right-lateral strike-slip faults [42,43]. The Whittier–Ellsinore fault has a slip rate of 2.5 to 3.0 mm/year and is estimated to be capable of a 6.0–7.2 magnitude ($M_w$) earthquake, while the Newport–Inglewood fault has a slip rate of approximately 0.6 mm/year and could generate a 6.0–7.4 magnitude ($M_w$) earthquake [44]. Therefore, in this study, it is decided to analyze the benchmark bridge when exposed to an $M_w = 7.2$ earthquake with the causative strike-slip fault at around 11 km from the site.

According to Makris and Zhang [45], a geotechnical exploration at the location of the piers and near the end abutments was conducted before construction, down to a depth of about 35 m. The moderately stiff soil was identified, and the shear wave velocity of 200 m/s was estimated [45]. Hence, real strong motion records at the stations with similar local soil conditions were looked for. A further requirement is for the deeper geological conditions of the recording sites to resemble the geological surroundings of the benchmark bridge. The reason is that the geological characteristics up to depths of hundreds of meters or even a few kilometers may strongly affect the severity of surface seismic waves with both the shorter and longer oscillation periods [46]. Several recent seismic microzonation and strong motion studies have shown that this influence is also regionally dependent [47,48]. Therefore, to properly capture deeper geological conditions, it is decided to use the strong motion time histories recorded in Southern California as close to the benchmark bridge as possible.

Usually, it is very difficult to find strong motion records that will match all defined criteria above, but in this study, fortunately, such records are found. Table 1 illustrates the two records (each with three orthogonal components, i.e., two horizontal and one vertical) available at PEER Ground Motion Database [49], which closely match the defined earthquake and site conditions. Table 1 includes PEER Record Sequence Numbers (RSN), station name, pulse period, significant duration, Peak Ground Acceleration (PGA), earthquake name, magnitude, fault mechanism, distance to the rupture, and shear wave velocity for the first 30 m of the stratigraphic profile. For both records, the causative earthquake was the 2010 “El Mayor-Cucapah Mexico” earthquake with $M_w = 7.2$ and strike-slip fault mechanism. Moreover, both were recorded around 10 km from the fault, in Southern California, with the shear wave velocity parameter, Vs30, relatively close to 200 m/s. Another substantial point is that both records represent near-source ground motions that exhibit distinguishable strong acceleration and velocity pulses. In addition to the two real records, 10 synthetic accelerograms are generated to account for stochastic features of strong ground motion. Figure 3a,b show the recorded acceleration time histories described in Table 1, and the corresponding synthesized accelerograms, for both horizontal ground motion components.
Table 1. Basic features of the two selected records from the PEER Ground Motion Database [49].

| RSN | Station                       | Period (s) | 5–95% Duration (s) | PGA (g) | Earthquake Year and Name          | Mw | Mechanism    | Rrup (km) | Vs30 (m/s) |
|-----|-------------------------------|------------|--------------------|---------|-----------------------------------|----|--------------|-----------|------------|
| 8161| “El Centro Array #12”         | 8.722      | 32.9               | 0.31    | 2010, “El Mayor-Cucapah Mexico”   | 7.2| strike slip | 11.26     | 196.88     |
| 8606| “Westside Elementary School”  | 7.084      | 25.3               | 0.28    | 2010, “El Mayor-Cucapah Mexico”   | 7.2| strike slip | 11.44     | 242        |

Figure 3. (a). Acceleration time histories of the two horizontal components of the real record from Table 1 for the station “El Centro Array #12” and five corresponding spectrum-compatible and energy-compatible synthetic acceleration time histories; (b) Acceleration time histories of the two horizontal components of the real record from Table 1 for the station “Westside Elementary School” and five corresponding spectrum-compatible and energy-compatible synthetic acceleration time histories.

The synthetic ground acceleration time histories have been generated as spectrum-compatible and energy-compatible with the two selected real records, based on the method of Li et al. [50], see Equations (2)–(6), and by using the corresponding Matlab subroutines. The method of Li et al. [50] is
partially based on the same procedure used by Gasparini and Vanmarcke [51], i.e., on the fact that any periodic function can be expanded into a series of sinusoidal waves:

$$x(t) = \sum_{i=1}^{n} A_i \sin(\omega_i t + \phi_i)$$  (2)

where $A_i$ is the amplitude and $\phi_i$ is the phase angle of the $i$-th contributing sinusoid. By fixing an array of amplitudes and generating different arrays of phase angles, one obtains different motions with the same general appearance but different details. The computer uses a random number generator to produce strings of phase angles with the uniform likelihood in the range between 0 and $2\pi$. In order to match the target spectrum, the amplitudes $A_i$ have to be iteratively adjusted:

$$A_{i,j+1} = A_{i,j} \frac{PSAt}{PSA_{computed}}$$  (3)

where $j$ is the number of the current iteration and $PSAt$ is the target pseudo-acceleration spectrum. To simulate the transient character of real earthquakes, the steady-state motions are multiplied by an envelope function $I(t)$. The artificial motion $z(t)$ then becomes:

$$z(t) = I(t) \sum_{i=1}^{n} A_i \sin(\omega_i t + \phi_i)$$  (4)

Usually, the envelope function is a deterministic function that does not change during the iteration process. In their method, Li et al. [50] adjust $I(t)$ according to the differences between the target and simulated energy distributions. For this purpose, they proposed iterative adjustment of $I(t)$ based on the values of Arias Intensity (AI). The Arias Intensity is the measure of the energy contained in the accelerogram and can be defined as:

$$AI = \frac{\pi}{2 g} \int_{t=0}^{t_d} a_s^2(t) dt$$  (5)

where $t_d$ is the total duration of strong ground motion and $a_s$ is the acceleration time history. In order to match the target $AI$, the envelope $I(t)$ has to be iteratively adjusted:

$$I(t)_{j+1} = I(t)_j \left( \frac{a_s^2(t)_j}{a_s^2(t)_{computed}} \right)^p$$  (6)

where $j$ is the number of the current iteration, $a_s$ is the target pseudo-acceleration spectrum, and $p$ is a factor controlling the convergence speed. Table 2 demonstrates the PGA and AI values of the synthetic accelerograms compatible with the real records for the stations “El Centro Array #12” and “Westside Elementary School”. Furthermore, Figures 4 and 5, respectively, show pseudo-acceleration spectra and Arias intensities of the real and synthesized accelerograms. For more details on the whole procedure, the readers are referred to [50].
Table 2. Peak ground acceleration (PGA) and Arias Intensity (AI) values for the five synthetic accelerograms that are spectrum-compatible and energy-compatible with the real records from Table 1 for the stations “El Centro Array #12” and “Westside Elementary School”.

| Synthetic Accelerogram # for the station “El Centro Array #12” | 1st Horizontal Component | 2nd Horizontal Component |
|---------------------------------------------------------------|--------------------------|--------------------------|
|                                                                | PGA (g) | Target PGA (g) | AI (m/s) | Target AI (m/s) | PGA (g) | Target PGA (g) | AI (m/s) | Target AI (m/s) |
| 1                                                                 | 0.33    | 2.72           | 0.30     | 3.21            | 0.34    | 2.75           | 0.36     | 3.42            |
| 2                                                                 | 0.34    | 2.75           | 0.36     | 3.42            | 0.30    | 2.93           | 0.31     | 3.21            |
| 3                                                                 | 0.30    | 2.75           | 0.31     | 3.21            | 0.31    | 2.86           | 0.35     | 3.20            |
| 4                                                                 | 0.32    | 2.86           | 0.34     | 3.27            | 0.31    | 2.87           | 0.34     | 3.27            |
| 5                                                                 | 0.31    | 2.87           | 0.34     | 3.27            | 0.31    | 2.87           | 0.34     | 3.27            |

| Synthetic Accelerogram # for the station “Westside Elementary School” | 1st Horizontal Component | 2nd Horizontal Component |
|------------------------------------------------------------------------|--------------------------|--------------------------|
|                                                                        | PGA (g) | Target PGA (g) | AI (m/s) | Target AI (m/s) | PGA (g) | Target PGA (g) | AI (m/s) | Target AI (m/s) |
| 1                                                                      | 0.27    | 1.23           | 0.36     | 1.87            | 0.29    | 1.27           | 0.31     | 1.99            |
| 2                                                                      | 0.29    | 1.27           | 0.31     | 1.99            | 0.28    | 1.27           | 0.35     | 2.07            |
| 3                                                                      | 0.28    | 1.27           | 0.35     | 2.07            | 0.22    | 1.19           | 0.31     | 1.97            |
| 4                                                                      | 0.22    | 1.19           | 0.39     | 2.09            | 0.25    | 1.29           | 0.39     | 2.09            |

Figure 4. (a). Pseudo-acceleration spectrum of the real record from Table 1 for the station “El Centro Array #12” and the spectra of five corresponding synthetic acceleration time histories shown in Figure 1a, for both horizontal ground motion components. (b) Pseudo-acceleration spectrum of the real record from Table 1 for the station “Westside Elementary School” and the spectra of five corresponding synthetic acceleration time histories shown in Figure 1b, for both horizontal ground motion components.
4. The SMA-LRBs

Adding the SMA wires to base isolators as a supplementary element can mitigate the seismic response of the structures in terms of displacement and internal forces, and offers some benefits such as re-centering capability, stability, high fatigue resistance, high energy dissipation capacity, etc. For these reasons, many researchers have used SMA wires in conjunction with LRBs, which some of them are mentioned in the introduction.

In this study, the newly discovered iron-based SMA alloy, i.e., FeNiCoAlTaB with 13.5% super-elastic strain is used, and the finite element approach is applied to investigate the effects of different wire configurations on the seismic performance of a benchmark bridge. To do so, some milestones should be reached, including the development of an appropriate constitutive model for the SMA-LRBs. Due to the complexity of the shear behavior of the SMA-LRBs, Hedayati Dezfuli and Alam [24] suggested using the idea of superposition to divide the smart isolation into two separate systems, namely, SMA wire and LRB to simplify the model. Then, to obtain the shear hysteresis model for the SMA-LRB, first, the model is found for each device; second, the models are added to each other to form the hysteresis behavior of the isolation system. It is worth mentioning that Hedayati Dezfuli and Alam verified the superposition method through the finite element model validated by experimental results [24].
4.1. The Hysteresis Model of the LRBs

In order to simplify the finite element analysis procedure, the hysteresis response of the LRBs can be modelled using a bilinear model [24,37,52,53]. Here, it is assumed that the behavior of LRB can be simulated by the bilinear kinematic hardening (BKH) model. Three parameters exist in this model, including yield force $F_y$, initial stiffness (i.e., stiffness in the elastic region) $K_0$ and post-yield hardening ratio $r$ (i.e., the ratio of the stiffness in the plastic region to the initial stiffness). These parameters are required for finding the shear force–displacement curve. The BKH model is available in some software. In this case, the available code in the benchmark bridge package is applied. Characteristics of benchmark bridge LRBs are shown in Table 3.

| Isolator Type       | Yield Displacement (m) | Post-Yield Stiffness (kN/mm) | Initial Stiffness (kN/mm) |
|---------------------|------------------------|-----------------------------|---------------------------|
| Abutment Isolators  | 0.015                  | 0.6                         | 4.8                       |
| Mid-pier Isolators  | 0.015                  | 1.2                         | 9.6                       |

4.2. The Hysteresis Model of SMA Wires

Researchers have wrapped SMA wires around the base isolators in three different arrangements, namely, straight [18], cross [16,26], and double-cross [24,25]. These isolators are schematically shown in Figure 6. It should be noted that, in these isolators, wires pass through steel hooks, which are mounted on the top and the bottom supporting plates. It is assumed that there is a smooth contact between the steel hook and the SMA wire. In order to reduce the complexity of the finite element simulation, Hedayati Dezfuli and Alam [16] proposed to consider the applied forces on the isolator due to the SMA wires, instead of modelling the steel hooks and their contact with the wire. They calculated the force generated in SMA wires as a function of time, and then applied the effect of SMA wire on the isolator. For calculating this axial force of the wire, first, by specifying the geometrical properties of the isolator and configuration of wire, the strain in the wire at each time step can be found. Second, with the help of the stress–strain relationship of SMA, the axial stress in the wire is determined. Finally, using the amount of the axial stress and the direction of wire at each time step, the force vectors exerted from the SMA wire to the LRB isolator as an external force are computed.

![Figure 6. (a) Straight, (b) cross and (c) double-cross configuration of shape memory alloy (SMA)-LRB.](image_url)

As mentioned above, the strain in SMA wire at each time step depends on the geometry of the isolator and wires arrangement. In this study, the SMA wires are wrapped around the LRBs of the benchmark bridge, so the geometry of the LRB is constant. Therefore, calculations must be performed for the three different configurations of wires separately.
4.2.1. SMA-LRB with Double-Cross Wires

In double-cross arrangement, each of the two continuous SMA wires is passed through eight steel hooks welded to the top and bottom supporting plates of the LRB, as can be seen in Figure 6. Hedayati Dezfuli and Alam proposed and utilized this configuration of wires to increase the length of the SMA wire with the purpose of reducing the wire’s strain. For an LRB with specific dimensions, the required SMA wire to wrap around the isolator is the longest for the double-cross arrangement. As a result, the effective strain in the SMA wires generated due to the shear deformation in the LRB decreases because wires have a longer initial length.

The strain in the SMA wire is calculated by Equation (7). Where \( l_{0,SMA} \) is the initial length of the SMA wire and \( l_{SMA} \) is the length of the SMA wire after the isolator undergoes a deflection of \( \Delta x \) in the X-direction. For the double-cross arrangement of wires, these parameters are determined from Equations (8) and (9). In which \( l, w, \) and \( h \) are the distances between hooks in X, Y, and Z directions, respectively. Having calculated the strain, the axial stress in SMA wires can be determined from the idealized stress–strain relationship of shape memory alloy based on Auricchio’s super-elasticity model [54]. The idealized super-elastic behavior of SMA is shown in Figure 7. After that, the axial force in the wire, \( F \), can be found from Equation (10). The resultant force exerted to the LRB in the X-direction, and the consequent hysteresis of the SMA wires is comprehensively explained in [24] and is not repeated here. In order to achieve the hysteresis curves of the SMA wire, a MATLAB function is developed by the authors based on the presented algorithm by Hedayati Dezfuli and Alam [24] and was verified in a previous study [23]. This algorithm calculates the force applied by the SMA wire to the LRB at each time step for a specific shear displacement.

\[
\varepsilon_{SMA} = \frac{l_{SMA} - l_{0,SMA}}{l_{0,SMA}} 
\]

(7)

\[
l_{0,SMA} = 4\sqrt{\left(\frac{l}{2}\right)^2 + h^2 + \sqrt{\left(\frac{w}{2}\right)^2 + h^2}} 
\]

(8)

\[
l_{SMA} = 2\sqrt{\left(\frac{l}{2} + \Delta x\right)^2 + h^2 + 2\left(\frac{l}{2} - \Delta x\right)^2 + h^2 + 4\Delta x^2 + \left(\frac{w}{2}\right)^2 + h^2} 
\]

(9)

\[
F = \sigma_{SMA} \pi r_{SMA}^2 
\]

(10)

\( \sigma^M_{s} = \) Martensite start stress  
\( \sigma^F_{s} = \) Martensite finish stress  
\( \sigma^M_{f} = \) Austenite start stress  
\( \sigma^F_{f} = \) Austenite finish stress  
\( \varepsilon^M_{s} = \) Martensite start strain  
\( \varepsilon^F_{s} = \) Martensite finish strain  
\( \varepsilon^M_{f} = \) Austenite start strain  
\( \varepsilon^F_{f} = \) Austenite finish strain

Figure 7. Idealized stress-strain diagram of SMA [54].

4.2.2. SMA-LRB with Straight Wires

This arrangement of wires was proposed by Choi et al. [18]. As can be seen in Figure 6, in straight configuration, two continuous SMA wires are wound in two opposite sides of the isolator. Wires are passed through steel hooks with smooth contact. Since the wire is continuous and is not fixed at the corners of the isolator, the initial length of the SMA wire is longer, and its effective strain is lower.
In this arrangement the strain in SMA wire is also determined by Equation (7); however, the parameters for straight wires are obtained based on its configuration as in Equations (11) and (12). After that, the strain and axial force in the SMA wire is determined like in the double-cross arrangement. The proposed algorithm by Hedayati Dezfuli and Alam [24] can be modified slightly to account for the straight configuration. This code is written by the authors as a function in MATLAB to input $\Delta X$ at each time step and calculate the resultant force that SMA wire exerts to the LRB.

$$l_{0,SMA} = 2\left(\sqrt{l^2 + h^2} + \sqrt{w^2 + h^2}\right)$$  \hspace{1cm} (11)

$$l_{SMA} = \sqrt{(l + \Delta x)^2 + h^2 + \sqrt{(l - \Delta x)^2 + h^2 + 2\sqrt{\Delta x^2 + w^2 + h^2}}}$$  \hspace{1cm} (12)

4.2.3. SMA-LRB with Cross Wires

In the cross configuration, two continuous SMA wires are wound around the LRB diagonally, as can be seen in Figure 6. Hedayati Dezfuli and Alam firstly suggested and used this configuration of wires [16,30]. The reason was that the wire in this figure is longer compared with the straight arrangement, and thus, the induced strain in the wire is lower. In this case, again, the wire’s strain is calculated by Equation (2), but the parameters of this equation are determined by Equations (13) and (14). Similar to the previous two configurations, by modifying the Hedayati Dezfuli and Alam [24] algorithm, a new MATLAB function is obtained that can calculate the hysteresis shear response of SMA wires with cross configuration.

$$l_{0,SMA} = 2(l + h)$$  \hspace{1cm} (13)

$$l_{SMA} = 2(l + \sqrt{\Delta x^2 + h^2})$$  \hspace{1cm} (14)

4.2.4. Efficiency of Different Wire Configurations

An essential prerequisite for the efficient performance of SMA-LRB is for the SMA wires to stay in their super-elastic range under lateral displacements. The SMA alloy that is used in this study is FeNiCoAlTaB, with 13.5% maximum super-elastic strain. As explained above, the strain in the wire is a function of wires’ configuration and LRB’s dimension and is independent of the wire’s radius. Here, LRBs geometry is constant; therefore, the strain is only dependent on the wires’ arrangement.

In order to find the range of shear strain that each configuration can remain applicable, the induced strain in wires due to a continuous range of shear strain (i.e., lateral displacement amplitude divided by total rubber height), $\gamma$, ranging between 0% and 250% calculated and plotted for all arrangements of wires in Figure 8. The wire strain is calculated using Equations (7) to (14). As can be observed, shear strain in the cross configuration is lower than in the others, but the performance of double-cross and straight wires are almost the same. At the ultimate shear strain, the induced strain in straight, cross and double-cross wire is 10.44%, 3.23%, and 11.39%, respectively, which is below the super-elastic strain limit of the iron-based SMA alloy. Thus, with this LRB dimension and this SMA alloy, wires in all three configurations can perform effectively.

It is worth mentioning that, for shear strain levels between 50% and 150%, the strain in SMA wire with cross arrangement varies between 0.13% and 1.18%, which is not high enough to create the flag-shaped hysteresis of SMA. Therefore, in lower shear strains, the cross configuration is not as effective as the other arrangements.

4.2.5. Force–Displacement Curves

In the next section, the radius of the SMA wire is found for each configuration through an optimization algorithm. However, the optimized radius is different for each arrangement. Thus, to observe and compare the sole effect of various arrangements on the hysteresis diagram of the SMA wires, the force–displacement curves for the three wire’s configurations with a fixed wire radius of 2.5 mm, under three different lateral displacement scenarios, are plotted in Figure 9. As illustrated,
for specified excitation and wire radius, the stiffness of the double-cross configuration is more than the straight one and more than the cross one. By increasing the radius of the wire in each arrangement, the stiffness could be increased. Thus, it is expected that SMA-LRBs with cross arrangement need thicker wires, and with double-cross arrangement, require the thinnest wire. Additionally, the cross wires create less flag-shaped hysteresis because of their lower levels of strain in the SMA wires (see Figure 8). The input displacements are extracted from Hedayati Dezfuli and Alam [24] and are named in the same way as E5, E6, and E7. They selected these displacement scenarios in a way to follow different functions like ramp, step, sinusoidal, and a combination of them with various peak amplitudes to assess the model in any condition. As mentioned before, the written code in this study is based on the algorithm developed by Hedayati Dezfuli and Alam [24] and was verified in a previous paper by Hosseini et al. [23].

![Figure 8. Strain in SMA wires due to shear strain of SMA-LRB for the three different wires arrangement.](image)

**Figure 8.** Strain in SMA wires due to shear strain of SMA-LRB for the three different wires arrangement.

![Figure 9. Excitations (E5, E6 and E7) in terms of shear strain versus time, and corresponding shear hysteresis responses of the three SMA wire configurations in terms of force versus displacement.](image)

**Figure 9.** Excitations (E5, E6 and E7) in terms of shear strain versus time, and corresponding shear hysteresis responses of the three SMA wire configurations in terms of force versus displacement.
5. Optimizing the SMA-LRBs

In the previous section, the approach for obtaining the hysteresis models for SMA-LRBs with three different configurations of wires was explained. As discussed, the influential parameters on the hysteresis behavior of the SMA-LRBs include the geometry of the isolator, wires arrangement, and the radius of the SMA wire (see Equation (10)). This research uses the SMA wire in three arrangements, as a supplementary element to the existed LRBs of the benchmark bridge, to enhance its seismic performance without changing its isolators. Therefore, the only unknown yet significant parameter is the radius of the wire. It should be determined carefully since it has a substantial effect on the force applied to the LRB isolator. By increasing the radius of the wire, the re-centering force generated in the SMA wire and applied to the LRB increases, and, as a result, the horizontal stiffness of the isolator increases. Consequently, the residual deformation declines, but the base shear force and the deck acceleration of the bridge rise. The main goal of this research is to minimize the residual displacements of the bridge after strong earthquakes; however, a detrimental increase in the base shear and acceleration is not acceptable. Therefore, it is decided to take advantage of a multi-objective optimization algorithm to simultaneously minimize displacement and acceleration and find the optimum radius of the wire.

An evolutionary optimization algorithm named NSGA-II (non-dominated sorting genetic algorithm) is used to find the best radius of the SMA wire for each arrangement under the most severe earthquake. The NSGA-II is a multi-objective genetic algorithm that was proposed by Deb et al. [55]. It is an extension and development of NSGA, which was proposed previously by Srinivas and Deb [56]. In the structure of the NSGA-II, in addition to genetic operators (i.e., crossover and mutation), two specified operators are defined and employed. The first one is non-dominated sorting, which sorts and partitions the population into fronts (F1, F2, etc.), where F1 indicates the approximated Pareto front. The second one is crowding distance, which specifies a mechanism of ranking amongst members of a front, which are either dominated by or are dominating each other. These ranking mechanisms are utilized with genetic selection operators to generate the population of the next generation. The flowchart of NSGA-II is demonstrated in Figure 10 [57].

![Figure 10. Flowchart of non-dominated sorting genetic algorithm (NSGA-II).](image-url)
In the NSGA-II algorithm of this project, there are two objective functions, namely, maximum mid-span displacement and maximum deck acceleration. For simplicity, these parameters are normalized with respect to the quantities of the uncontrolled bridge. The optimization should be carried out in the worst scenario of the earthquake, so the real record from the station El Centro Array #12 (RSN 8161) is used in the algorithm, as it has larger Arias Intensity and larger PGA value (Tables 1 and 2). The population includes 30 members, and the maximum number of iterations is equal to 30. The crossover and mutation rates are 0.7 and 0.4, in turn. Under these defined circumstances, the algorithm is run three times for the three various configurations of wires, and the Pareto front is achieved for each case. The Pareto front indicates the points that simultaneously and independently minimize the normalized maximum mid-span displacement and the normalized maximum deck acceleration. Figure 11 depicts the optimized point for the three wire configurations. Selecting the desired radius amongst the optimized points of a Pareto front depends on the designer’s perspective. In this research, the main goal is to decrease displacements; hence, the final radius is determined based on this idea. The final optimized and rounded radii for the double-cross, straight, and cross arrangements are 5 mm, 6 mm and 7 mm, respectively. It should be mentioned that, by increasing the population size, this algorithm can find more counts of optimum points at the expense of increasing the runtime.

![Pareto Front of NSGA-II for the three wire configurations.](image)

### 6. Numerical Analysis

In order to assess the performance of the three optimized SMA-LRBs, the seismic responses of the highway bridge equipped with the SMA-based isolators are calculated and compared with those of the LRB isolated and the non-isolated bridges. These responses include the base shear at the piers, the deck acceleration, relative displacement at the mid-span of the bridge, and the mid-span residual deformation. For the purpose of the comparison, five distinctive sets of dynamic time history analyses must be conducted on the (1) non-isolated bridge, (2) LRB isolated bridge, (3) SMA-LRB isolated with double-cross wires, (4) SMA-LRB isolated with straight wires and finally (5) SMA-LRB isolated with cross wires. All these models are excited under two real and 10 synthetic ground motion records. The detailed characteristics of these records are mentioned in Section 3. These records are exerted on the bridge only in the longitudinal direction, and between the two horizontal components for each record, the one that leads to more critical responses is chosen. It is worth mentioning that the responses of the non-isolated bridge are calculated by analyzing the non-isolated model (phase I) of the benchmark bridge, which was developed and released by Agrawal et al. [27].

#### 6.1. Base Shear

The peak base shear responses of the benchmark bridge in five different conditions, i.e., non-isolated bridge, LRB isolated bridge, and SMA-LRB isolated bridge with double-cross (denoted as DC-SMA-LRB), straight (denoted as S-SMA-LRB) and cross (denoted as C-SMA-LRB) wire arrangements resulted from the nonlinear dynamic time history analysis are tabulated in Table 4. As can be seen in this Table, base shear for the non-isolated benchmark bridge is by far the largest under all ground motion records. After implementing LRB isolators, the base shear decreases significantly due to the
reduction in the stiffness and transmitted load from the base to the superstructure. The largest drop in the base shear after isolating by the LRBs occurs under the real record of RSN 8606 earthquake, with 91%. However, by isolating the superstructure, its displacements increase. That is why SMA wires are employed in this study, to control the excessive displacements of the bridge. Adding SMA wires to the LRBs leads to a marginal rise in the horizontal stiffness and, as a consequence, in the base shear. According to Figure 12, it is true that SMA-LRBs increase the base shear of the benchmark bridge more than the LRBs, but the figures are still far less than those of the non-isolated bridge. Consequently, both the LRBs and SMA-LRBs are effective measures for reducing the base shear of the bridge.

Table 4. Maximum base shear for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

| Earthquake | Non-Isolated | LRB Isolated | DC-SMA-LRB Isolated | S-SMA-LRB Isolated | C-SMA-LRB Isolated |
|------------|--------------|--------------|---------------------|--------------------|--------------------|
| RSN 8161   | 7.09 × 10^6  | 1.58 × 10^6  | 1.96 × 10^6         | 1.83 × 10^6        | 1.71 × 10^6        |
| Synthetic #1 | 7.07 × 10^6  | 1.30 × 10^6  | 1.70 × 10^6         | 1.59 × 10^6        | 1.46 × 10^6        |
| Synthetic #2 | 6.82 × 10^6  | 0.91 × 10^6  | 1.06 × 10^6         | 1.04 × 10^6        | 1.02 × 10^6        |
| Synthetic #3 | 6.80 × 10^6  | 1.22 × 10^6  | 2.02 × 10^6         | 1.87 × 10^6        | 1.35 × 10^6        |
| Synthetic #4 | 6.61 × 10^6  | 0.88 × 10^6  | 1.69 × 10^6         | 1.57 × 10^6        | 1.49 × 10^6        |
| Synthetic #5 | 8.25 × 10^6  | 1.18 × 10^6  | 1.75 × 10^6         | 1.64 × 10^6        | 1.25 × 10^6        |
| RSN 8606   | 7.17 × 10^6  | 0.64 × 10^6  | 0.94 × 10^6         | 0.84 × 10^6        | 0.79 × 10^6        |
| Synthetic #1 | 6.29 × 10^6  | 0.97 × 10^6  | 1.07 × 10^6         | 1.06 × 10^6        | 1.01 × 10^6        |
| Synthetic #2 | 6.39 × 10^6  | 0.84 × 10^6  | 1.18 × 10^6         | 1.13 × 10^6        | 1.10 × 10^6        |
| Synthetic #3 | 7.90 × 10^6  | 0.72 × 10^6  | 0.96 × 10^6         | 0.94 × 10^6        | 0.90 × 10^6        |
| Synthetic #4 | 6.71 × 10^6  | 0.66 × 10^6  | 0.91 × 10^6         | 0.84 × 10^6        | 0.71 × 10^6        |
| Synthetic #5 | 6.29 × 10^6  | 0.60 × 10^6  | 1.04 × 10^6         | 0.96 × 10^6        | 0.83 × 10^6        |

Figure 12. Graphical demonstration of the base shear for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

6.2. Mid-Span Acceleration

Table 5 and Figure 13 show and compare the maximum mid-span acceleration in the non-isolated, LRB isolated, and three SMA-LRB isolated benchmark bridges. As reflected, both isolation systems, i.e., LRB isolators and SMA-LRBs substantially decrease the peak acceleration under all ground motion records, compared to the non-isolated state. By introducing isolators in the central pier of the benchmark bridge, its stiffness and load transferred from the base to the superstructure reduce, so the acceleration reduces too. After wrapping SMA wires around the LRB isolators, however, the stiffness and the acceleration increase slightly. This increase is the least for SMA-LRBs with cross wires and the largest for the double-cross arrangement of the wires, and still far less than the figures for the non-isolated
bridge. Furthermore, as an example, the time history curves of the mid-span acceleration for the five different benchmark bridges under the synthetic record number 3 of the RSN 8161 earthquake are plotted in Figure 14.

Table 5. Maximum mid-span acceleration for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

| Earthquake      | Non-Isolated | LRB Isolated | DC-SMA-LRB Isolated | S-SMA-LRB Isolated | C-SMA-LRB Isolated |
|-----------------|--------------|--------------|---------------------|--------------------|-------------------|
| RSN 8161        | 4.45         | 1.92         | 3.33                | 3.05               | 2.97              |
| Synthetic #1    | 4.27         | 1.29         | 2.71                | 2.45               | 2.07              |
| Synthetic #2    | 4.34         | 1.45         | 2.45                | 2.37               | 1.99              |
| Synthetic #3    | 4.26         | 1.76         | 3.37                | 3.12               | 2.07              |
| Synthetic #4    | 4.30         | 1.22         | 3.06                | 2.89               | 2.70              |
| Synthetic #5    | 5.07         | 1.40         | 2.74                | 2.44               | 1.98              |
| RSN 8606        | 4.50         | 1.01         | 2.00                | 1.88               | 1.39              |
| Synthetic #1    | 5.71         | 0.99         | 2.14                | 1.89               | 1.47              |
| Synthetic #2    | 4.02         | 1.45         | 2.24                | 2.10               | 1.87              |
| Synthetic #3    | 4.90         | 1.79         | 2.20                | 2.02               | 2.01              |
| Synthetic #4    | 4.23         | 1.28         | 1.90                | 1.74               | 1.47              |
| Synthetic #5    | 4.04         | 1.38         | 1.98                | 1.76               | 1.75              |

Figure 13. Graphical demonstration of the mid-span acceleration for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

Figure 14. Time history curves of the mid-span acceleration under synthetic#3 of the RSN8161 record.
6.3. Mid-Span Displacement

The benchmark highway bridge in the non-isolated phase, in which the deck is connected to the central pier, experiences the smallest superstructure displacements, and the highest base shear and deck acceleration (see Sections 6.1 and 6.2). After isolating the deck from the central piers via LRBs, the acceleration, and base shear drop, but mid-span displacement rises dramatically. This rise is evident in Figure 15 and Table 6. For example, under synthetic record number 3 of the RSN 8606 record, the maximum mid-span displacement jumps from 81 to 650 mm after isolation. The extreme deck displacement resulting from the isolating process can have detrimental effects, such as the collision of the adjacent parts of the superstructure, deck unseating, or plastic deformation of the bearings. To tackle this issue, SMA wires are added to the LRBs to control displacements.

![Graphical demonstration of the maximum mid-span displacement for the non-isolated, LRB-isolated and the three SMA-LRB isolated bridges.](image)

**Figure 15.** Graphical demonstration of the maximum mid-span displacement for the non-isolated, LRB-isolated and the three SMA-LRB isolated bridges.

**Table 6.** Maximum mid-span displacement for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

| Earthquake | Non-ISOLATED Bridge | LRB Isolated | DC-SMA-LRB Isolated | S-SMA-LRB Isolated | C-SMA-LRB Isolated |
|------------|---------------------|-------------|---------------------|-------------------|-------------------|
| RSN 8161   | 0.73 x 10^-1       | 2.98 x 10^-1 | 1.53 x 10^-1       | 48.66             | 1.62 x 10^-1     | 45.64             | 2.20 x 10^-1     | 26.17             |
| Synthetic #1 | 0.71 x 10^-1       | 4.60 x 10^-1 | 1.54 x 10^-1       | 66.52             | 1.59 x 10^-1     | 65.43             | 1.64 x 10^-1     | 64.35             |
| Synthetic #2 | 0.69 x 10^-1       | 3.12 x 10^-1 | 1.54 x 10^-1       | 50.64             | 1.70 x 10^-1     | 45.51             | 2.00 x 10^-1     | 35.90             |
| Synthetic #3 | 0.73 x 10^-1       | 3.81 x 10^-1 | 1.63 x 10^-1       | 57.22             | 1.69 x 10^-1     | 55.64             | 2.44 x 10^-1     | 35.96             |
| Synthetic #4 | 0.68 x 10^-1       | 3.87 x 10^-1 | 1.77 x 10^-1       | 54.26             | 1.78 x 10^-1     | 54.01             | 2.43 x 10^-1     | 37.21             |
| Synthetic #5 | 0.86 x 10^-1       | 2.86 x 10^-1 | 1.73 x 10^-1       | 39.51             | 1.80 x 10^-1     | 37.06             | 1.84 x 10^-1     | 35.66             |
| RSN 8606   | 0.75 x 10^-1       | 3.92 x 10^-1 | 1.15 x 10^-1       | 70.66             | 1.22 x 10^-1     | 68.88             | 1.25 x 10^-1     | 68.11             |
| Synthetic #1 | 0.42 x 10^-1       | 3.45 x 10^-1 | 1.31 x 10^-1       | 62.03             | 1.38 x 10^-1     | 60.00             | 1.48 x 10^-1     | 57.10             |
| Synthetic #2 | 0.69 x 10^-1       | 4.81 x 10^-1 | 1.19 x 10^-1       | 75.26             | 1.34 x 10^-1     | 72.14             | 1.84 x 10^-1     | 61.75             |
| Synthetic #3 | 0.81 x 10^-1       | 6.90 x 10^-1 | 1.22 x 10^-1       | 81.23             | 1.30 x 10^-1     | 80.00             | 2.17 x 10^-1     | 66.62             |
| Synthetic #4 | 0.70 x 10^-1       | 4.06 x 10^-1 | 1.21 x 10^-1       | 70.20             | 1.30 x 10^-1     | 67.98             | 1.80 x 10^-1     | 55.67             |
| Synthetic #5 | 0.68 x 10^-1       | 4.79 x 10^-1 | 1.00 x 10^-1       | 79.12             | 1.13 x 10^-1     | 76.41             | 1.97 x 10^-1     | 58.87             |

As illustrated in Table 6 and Figure 15, SMA wires are quite beneficial in minimizing the mid-span displacements. For instance, the maximum mid-span displacement under synthetic record number 3 of the RSN 8606 decreases from 650 to 122 mm using double-cross SMA-LRBs. According to Table 6, SMA-LRBs with various wire configurations reduce the maximum mid-span displacements in a range from 26% to 81%. Another interesting point is that SMA-LRBs with
double-cross wires reduce displacements more than the straight wires, and the cross configuration has the least displacement reduction.

6.4. Residual Mid-Span Displacement

One of the main goals of implementing SMA wires in LRBs is to reduce the residual displacement of the deck after strong earthquakes and decrease the repairing cost. Residual displacement is the lateral displacement that remains in the deck at the end of the earthquake. This value can be extracted from the displacement history curve of each point of the bridge’s deck. Here, the mid-span residual displacement is calculated, which also represents the residual displacement at the place of the mid-pier isolators. The mid-span residual displacement under all ground motion records, for the LRB isolated and the three SMA-LRB isolated bridges, is calculated and illustrated in Table 7. Besides, the reduction ratios of the residual displacement of the SMA-LRBs with respect to the LRBs are shown. The most significant reductions are under synthetic records number 3 of the RSN 8606 and RSN 816 records, for which the displacement history curves are demonstrated in Figures 16 and 17. As indicated, the mid-span residual displacement in the LRB isolated benchmark bridge is 13.19 cm under synthetic record number 3 RSN 8606, and by using double-cross, straight, and cross SMA wires, it drops to 0.18 cm, 0.89 cm, and 2.76 cm, respectively.

Table 7. Maximum residual displacement for the non-isolated, LRB-isolated and three SMA-LRB isolated bridges.

| Earthquake     | LRB Isolated | DC-SMA-LRB Isolated | S-SMA-LRB Isolated | C-SMA-LRB Isolated |
|----------------|--------------|---------------------|---------------------|--------------------|
| RSN 8161       | 0.1182       | 0.0300              | 74.62               | 0.0349             | 70.47              | 0.0538 | 54.48 |
| Synthetic #1   | 0.0657       | 0.0033              | 94.98               | 0.0035             | 94.67              | 0.0216 | 67.12 |
| Synthetic #2   | 0.0741       | 0.0349              | 52.90               | 0.0359             | 51.55              | 0.0426 | 42.51 |
| Synthetic #3   | 0.1617       | 0.0150              | 90.72               | 0.0185             | 88.56              | 0.0372 | 76.99 |
| Synthetic #4   | 0.0995       | 0.0476              | 52.16               | 0.0481             | 51.66              | 0.0553 | 44.42 |
| Synthetic #5   | 0.1248       | 0.0348              | 72.12               | 0.0413             | 66.91              | 0.0653 | 47.68 |
| RSN 8606       | 0.1882       | 0.0408              | 78.32               | 0.0414             | 78.00              | 0.0553 | 71.57 |
| Synthetic #1   | 0.1818       | 0.0326              | 82.07               | 0.0599             | 67.05              | 0.0663 | 63.53 |
| Synthetic #2   | 0.1374       | 0.0157              | 88.57               | 0.0226             | 83.55              | 0.0245 | 82.17 |
| Synthetic #3   | 0.1319       | 0.0018              | 98.67               | 0.0089             | 93.25              | 0.0276 | 79.08 |
| Synthetic #4   | 0.0834       | 0.0155              | 81.41               | 0.0227             | 72.78              | 0.0314 | 62.35 |
| Synthetic #5   | 0.1481       | 0.0153              | 89.67               | 0.0451             | 69.55              | 0.0721 | 51.32 |

Figure 16. Time history curves of the mid-span displacement under synthetic#3 of the RSN8161 record.
Figure 16. Time history curves of the mid-span displacement under synthetic#3 of the RSN8161 record.

Figure 17. Time history curves of the mid-span displacement under synthetic#3 of the RSN8060 record.

Due to the self-centering capability of the SMA wires, the residual displacement of the benchmark bridge equipped with SMA-LRBs is much smaller than that of the bridge with LRBs. Furthermore, amongst the three wires’ arrangements, the optimized double-cross is the most successful device in reducing the residual displacement, from 52.16% to 98.67% under various records. This range for the straight arrangement is between 51.55% and 93.25%, and for the cross wires is from 42.51% to 82.17%. Figure 18 graphically compares the mid-span residual displacement for different isolators.

6.5. Comparing the Three SMA-LRBs

SMA wires can be wrapped around the LRB isolators in double-cross, straight, and cross arrangements to improve the seismic performance of the isolated structure. On the one hand, it is proven that adding SMA wires to the LRBs, results in a marginal rise in the base shear and deck acceleration. As evidenced in Tables 4 and 5, this rise is the least for the bridge with cross SMA-LRBs and is the highest for the bridge with the double-cross SMA-LRBs. On the other hand, adding SMA wires reduces the maximum and residual displacements because of their re-centering and super-elasticity capabilities. According to Tables 6 and 7, amongst the three configurations, cross wires have the least reduction ratio in the maximum and residual displacements, while double-cross wires have the highest ratio. Another difference between these three isolators is the length of the used SMA wire, which can affect the cost of the project. According to Equations (8), (11), and (13), for SMA-LRBs with double-cross, straight and cross configurations, 242.71 cm, 133.12 cm, and 205.85 cm SMA wire is required to wrap around each of the LRBs, respectively.
For a specified design perspective, in order to choose the best configuration of SMA wires to add to the LRBs, the weighted sum method (WSM) can be utilized. The WSM is the best known and commonly used multi-criteria decision-making method for evaluating a number of alternatives regarding a number of decision criteria. According to the WSM, in a decision problem with M alternatives and N criteria, a performance value can be calculated for each alternative by applying Equation (15). In which, \( A_{i}^{WSM} \) is the WSM score of the \( i^{th} \) alternative, \( w_j \) is the weight of importance of \( j^{th} \) criteria, and \( a_{ij} \) is the actual value of the \( i^{th} \) alternative regarding the \( j^{th} \) criterion.

\[
A_{i}^{WSM} = \sum_{j=1}^{N} w_j \times a_{ij} , \ i = 1, 2, 3, \ldots, M. \tag{15}
\]

\[
a_{ij}^{Normalized} = \frac{\text{Min} a_{ij}}{a_{ij}} \tag{16}
\]

In this multi-criteria decision-making problem, there are three alternatives, including SMA-LRB with double-cross, straight and cross wires, and five decision criteria are set as the maximum base shear, maximum deck acceleration, maximum and residual mid-span displacement and the length of the SMA wire. Moreover, the responses under the real record RSN 8161, as illustrated in Table 8, are considered here, since this record has larger PGA values. To calculate the score of each alternative, first of all, the values must be normalized. Since for all the five criteria, the lower value is desired, then normalization must be performed using Equation (16). The normalized decision matrix is shown in Table 8.

| Criteria                        | Alternatives | Normalized Alternatives | Weights | Weighted Sum Matrix |
|---------------------------------|--------------|-------------------------|---------|---------------------|
|                                 | DC-SMA-LRB   | S-SMA-LRB               | C-SMA-LRB | DC-SMA-LRB   | S-SMA-LRB | C-SMA-LRB | DC-SMA-LRB | S-SMA-LRB | C-SMA-LRB |
| Maximum Base Shear (N)          | 1,960,000    | 1,830,000               | 1,710,000 | 0.87          | 0.93      | 1.00      | 0.15      | 0.13      | 0.14      | 0.15      |
| Maximum mid-span Acceleration (ms\(^{-2}\)) | 3.33        | 3.05                    | 2.97     | 0.89          | 0.97      | 1.00      | 0.15      | 0.13      | 0.15      | 0.15      |
| Maximum mid-span Displacement (m) | 0.153       | 0.162                   | 0.22     | 1.00          | 0.94      | 0.70      | 0.3       | 0.30      | 0.28      | 0.21      |
| mid-span Residual Displacement (m) | 0.03        | 0.0349                  | 0.0538   | 1.00          | 0.86      | 0.56      | 0.3       | 0.30      | 0.26      | 0.17      |
| Length of the SMA wire (cm)     | 242.71       | 133.12                  | 205.85   | 0.55          | 1.00      | 0.65      | 0.10      | 0.05      | 0.10      | 0.06      |
| Performance Score               | 0.92         | 0.93                    | 0.74     |              |
| Rank                            | 2            | 1                       | 3        |              |

The next step is to assign weights to the criteria. These numbers are selected based on the design perspective. Here, for example, reducing displacements is more important than a minor increase in the shear force and acceleration, and the length of the wire is presumed as the least important criteria. As a result, weights are selected as 0.15, 0.15, 0.3, 0.3, and 0.1 for base shear, acceleration, mid-span displacement, residual displacement, and wire length, respectively. The final step is to substitute the normalized values in Equation (15) and calculate the performance score for each alternative. As illustrated in Table 8, the WSM score for the SMA-LRB with double-cross, straight, and cross wires is 0.92, 0.93, and 0.74, in turn. Consequently, the straight wire arrangement has the best overall...
performance (rank 1), just above the double-cross arrangement (rank 2), but the SMA-LRB with cross wires displays by far the weakest performance (rank 3).

7. Conclusions

For nearly two decades, researchers have been using SMA wires in conjunction with isolators of the bridges to control their displacements and residual deformations. Different configurations of the SMA wires around the LRB isolators have been reported, namely, double-cross, straight, and cross. In this paper, the performance of these three SMA-LRBs is compared by implementing them in the LRBs of a benchmark highway bridge. Each configuration leads to specific characteristics of the SMA-LRBs and thus specific shear hysteresis. In addition to the wire’s arrangements, LRB dimensions and the radius of the wire are also influential parameters in the SMA-LRB behavior. For a specified LRB dimension, by increasing the radius of the wire, the horizontal stiffness, and consequently, the base shear and deck acceleration of the bridge rise, but the maximum and residual displacement reduce.

To get the best performance out of SMA wires, a multi-objective non-dominated sorting genetic algorithm (NSGA-II) is applied to find the radius of the SMA wire for each configuration that simultaneously minimizes the deck acceleration and mid-span displacement. After optimizing the isolators, to evaluate and compare their performance, nonlinear dynamic analysis is conducted for the five distinctive states of the benchmark bridge, i.e., non-isolated, LRB isolated, and the three SMA-LRB isolated bridges. For the nonlinear response history analysis, researchers can select ground motions from past recorded earthquakes in places other than the bridge’s site and modify them to match desired conditions. However, because of the deficiencies of this practice, in this paper, strong ground motion records are selected in a way to fit the site conditions, and also ten synthetic spectrum-compatible and energy-compatible ground acceleration time histories are generated to be used in the analysis. The summarized results are as follows.

- As for the optimized double-cross SMA-LRB, this device decreases the maximum base shear and deck acceleration of the isolated bridge from 70.29% to 87.85% and from 20.89% to 62.57%, respectively, under various records compared with the non-isolated bridge. However, LRB still leads to the largest reduction in these figures, from 77.71% to 91.07%, and from 56.85% to 82.66% under various records. Moreover, SMA-LRBs with double-cross wires reduce the maximum mid-span and residual displacements in ranges between 39.51% and 81.23% and from 52.16% to 98.67%, in turn, under different ground motions compared with the LRB isolated bridge. In brief, amongst the three SMA-LRBs, it has the largest shear force and deck acceleration, and needs the longest SMA wire, but leads to the least maximum and residual displacements. Finally, the result of the WSM method indicates that DC-SMA-LRB is the second most successful device.

- Regarding the optimized straight SMA-LRB, it decreases the maximum base shear and deck acceleration of the isolated bridge slightly more than the double-cross SMA-LRBs, from 72.50% to 88.28% and from 26.76% to 66.90%, respectively, under different records, compared with the non-isolated benchmark bridge. Besides, SMA-LRBs with straight wires reduce the maximum and residual mid-span displacement slightly less than the DC-SMA-LRBs, from 37.06% to 80.00% and from 51.55% to 94.67%, in turn, under various excitations, compared with the LRBs. Moreover, the required length of the SMA wire is the least for this configuration. Lastly, the WSM method shows that S-SMA-LRB has the best overall performance.

- As regards the optimized cross SMA-LRB, it results in the reduction of the base shear and deck acceleration more than the other two SMA isolators, compared with the non-isolated bridge. However, it decreases the maximum and residual displacements less than the others and uses rather long SMA wires. Eventually, by conducting WSM, it is ranked as the least successful device.

In this research, the bridge model presented by Nagarajaiah et al. [28] is utilized in which the bearings are modelled as shear elements by infinite vertical stiffness and zero torsional rigidity, and the coupling between the horizontal and vertical responses is neglected. In future studies, more up-to-date
models for the bearings should be utilized [58]. Moreover, a sensitivity analysis of the optimization methodology is needed to be done. A further future research proposal is to carry out probabilistic seismic fragility analysis or risk-based assessment on the optimized SMA-LRB isolated bridge [59,60].

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**References**

1. Buckle, I.G. *The Northridge, California Earthquake on January 17, 1994: Performance of Highway Bridges*; Tech. Rep. NCEER-94-0068; National Center for Earthquake Engineering Research, State University of New York at Buffalo: Buffalo, NY, USA, 1994.

2. Kawashima, K.; Unjoh, S. The damage of highway bridges in the 1995 Hyogo-ken Nanbu earthquake and its impact on Japanese seismic design. *J. Earthq. Eng.* 1997, 1, 505–541. [CrossRef]

3. Hsu, Y.T.; Fu, C.C. Seismic effect on highway bridges in Chi Chi earthquake. *J. Perform. Constr. Facil.* 2003, 17, 47–53. [CrossRef]

4. Li, J.; Peng, T.; Xu, Y. Damage investigation of girder bridges under the Wenchuan earthquake and corresponding seismic design recommendations. *Earthq. Eng. Eng. Vib.* 2008, 7, 337–344. [CrossRef]

5. Kawashima, K.; Unjoh, S.; Hoshikuma, J.I.; Kosa, K. Damage of bridges due to the 2010 Maule, Chile, earthquake. *J. Earthq. Eng.* 2011, 15, 1036–1068. [CrossRef]

6. Buckle, I.; Yen, W.; Marsh, L.; Monzon, E. Implications of Bridge Performance during Great East Japan Earthquake for U.S. Seismic Design Practice. In *Proceedings of the International Symposium on Engineering Lessons Learned from the 2011 Great East Japan Earthquake*, Tokyo, Japan, 1–4 March 2012; pp. 1363–1374.

7. Ozbulut, O.; Hurlebaus, S.; Desroches, R. Seismic response control using shape memory alloys: A review. *J. Intell. Mater. Syst. Struct.* 2011, 22, 1531–1549. [CrossRef]

8. Roy, B.K.; Chakraborty, S.; Mishra, S.K. Seismic vibration control of bridges with excessive isolator displacement. *Earthq. Struct.* 2016, 10, 1451–1465. [CrossRef]

9. Rai, D.C. Future trends in earthquake-resistant design of structures. *Curr. Sci.* 2000, 79, 1291–1300.

10. Shinozuka, M.; Chaudhuri, S.R.; Mishra, S.K. Shape-Memory-Alloy supplemented Lead Rubber Bearing (SMA-LRB) for seismic isolation. *Probab. Eng. Mech.* 2015, 41, 34–45. [CrossRef]

11. Bhuiyan, A.R.; Alam, M.S. Seismic performance assessment of highway bridges equipped with superelastic shape memory alloy-based laminated rubber isolation bearing. *Eng. Struct.* 2013, 49, 396–407. [CrossRef]

12. Paiva, A.; Savi, M.A. An overview of constitutive models for shape memory alloys. *Math. Probl. Eng.* 2006, 2006, 1–30. [CrossRef]

13. Lagoudas, D.C. *Shape Memory Alloys Modeling and Engineering Applications*; Springer: Boston, MA, USA, 2008.

14. Arghavani, J. Thermo-mechanical Behavior of Shape Memory Alloys under Multiaxial Loading: Constitutive Modeling and Numerical Implementation at Small and Finite Strains. Ph.D. Dissertation, Sharif University of Technology, Tehran, Iran, 2010.

15. Ozbulut, O.E. Seismic Protection of Bridge Structures Using Shape Memory Alloy-based Isolation Systems against Near-field Earthquakes. Ph.D. Dissertation, Texas A&M University, College Station, TX, USA, 2010.

16. Hedayati Dezfuli, F.; Alam, M.S. Shape memory alloy wire-based smart natural rubber bearing. *Smart Mater. Struct.* 2013, 22, 45013–45030. [CrossRef]

17. Wilde, K.; Gardoni, P.; Fujino, Y. Base isolation system with shape memory alloy device for elevated highway bridges. *Eng. Struct.* 2000, 22, 222–229. [CrossRef]

18. Choi, E.; Nam, T.; Cho, B.S. A new concept of isolation bearings for highway steel bridges using shape memory alloys. *Can. J. Civ. Eng.* 2005, 32, 957–967. [CrossRef]
19. Andrawes, B.; DesRoches, R. Comparison between Shape Memory Alloy Seismic Restrainers and Other Bridge Retrofit Devices. *J. Bridg. Eng.* 2007, 12, 700–709. [CrossRef]
20. Zhu, S.Y.; Qiu, C.X. Incremental Dynamic Analysis of Highway Bridges with Novel Shape Memory Alloy Isolators. *Adv. Struct. Eng.* 2014, 17, 429–438. [CrossRef]
21. Hedayati Dezfuli, F.; Alam, M.S. Seismic vulnerability assessment of a steel-girder highway bridge equipped with different SMA wire-based smart elastomeric isolators. *Smart Mater. Struct.* 2016, 25, 75039–75055. [CrossRef]
22. Xiang, N.; Alam, M.S. Comparative Seismic Fragility Assessment of an Existing Isolated Continuous Bridge Retrofitted with Different Energy Dissipation Devices. *J. Bridg. Eng.* 2019, 24, 04019070–04019087. [CrossRef]
23. Hosseini, R.; Rashidi, M.; Hedayati Dezfuli, F.; Karbasi Arani, K.; Samali, B. Seismic Assessment of a Benchmark Highway Bridge Equipped with Optimized Shape Memory Alloy Wire-Based Isolators. *Appl. Sci.* 2019, 10, 141. [CrossRef]
24. Hedayati Dezfuli, F.; Alam, M.S. Hysteresis model of shape memory alloy wire-based laminated rubber bearing under compression and unidirectional shear loadings. *Smart Mater. Struct.* 2015, 24, 065022–065041. [CrossRef]
25. Hedayati Dezfuli, F.; Alam, M.S. Smart Lead Rubber Bearings Equipped with Ferrous Shape Memory Alloy Wires for Seismically Isolating Highway Bridges. *J. Earthq. Eng.* 2017, 22, 1042–1067. [CrossRef]
26. Hedayati Dezfuli, F.; Alam, M.S. Performance-based assessment and design of FRP-based high damping rubber bearing incorporated with shape memory alloy wires. *Eng. Struct.* 2014, 61, 166–183. [CrossRef]
27. Agrawal, A.; Tan, P.; Nagarajaiah, S.; Zhang, J. Benchmark structural control problem for a seismically excited highway bridge-Part I: Phase I Problem definition. *Struct. Control Health Monit.* 2009, 16, 509–529. [CrossRef]
28. Nagarajaiah, S.; Narasimhan, S.; Agrawal, A.; Tan, P. Benchmark structural control problem for a seismically excited highway bridge-Part III: Phase II Sample controller for the fully base-isolated case. *Struct. Control Health Monit.* 2009, 16, 549–563. [CrossRef]
29. Tan, P.; Agrawal, A.K. Benchmark structural control problem for a seismically excited highway bridge-part II: Phase I sample control designs. *Struct. Control Health Monit.* 2009, 16, 530–548. [CrossRef]
30. Madhekar, S.N.; Jangid, R.S. Variable dampers for earthquake protection of benchmark highway bridges. *Smart Mater. Struct.* 2009, 18, 115011–115029. [CrossRef]
31. Madhekar, S.N.; Jangid, R.S. Seismic performance of benchmark highway bridge installed with piezoelectric friction dampers. *IES J. Part A Civ. Struct. Eng.* 2011, 4, 191–212. [CrossRef]
32. Madhekar, S.N. Seismic Performance of Benchmark Highway Bridge Installed with Passive Control Devices. *Adv. Struct. Eng.* 2015, 2, 1377–1390.
33. Saha, A.; Saha, P.; Patro, S.K. Seismic response control of benchmark highway bridge using non-linear FV spring damper. *IES J. Part A Civ. Struct. Eng.* 2015, 8, 240–250. [CrossRef]
34. Saha, A.; Saha, P.; Patro, S.K. Polynomial friction pendulum isolators (PFPIs) for seismic performance control of benchmark highway bridge. *Earthq. Eng. Eng. Vib.* 2017, 16, 827–840. [CrossRef]
35. Saha, A.; Saha, P.; Patro, S.K. Seismic protection of the benchmark highway bridge with passive hybrid control system. *Earthq. Struct.* 2018, 15, 227–241.
36. Zhang, Y.; Hu, X.; Zhu, S. Seismic performance of benchmark base-isolated bridges with superelastic Cu-Al-Be restraining damping device. *Struct. Control Health Monit.* 2009, 16, 668–685. [CrossRef]
37. Casciati, F.; Faravelli, L.; Al Saleh, R. An SMA passive device proposed within the highway bridge benchmark. *Struct. Control Health Monit.* 2009, 16, 657–667. [CrossRef]
38. Li, H.N.; Sun, T.; Lai, Z.; Nagarajaiah, S. Effectiveness of Negative Stiffness System in the Benchmark Structural-Control Problem for Seismically Excited Highway Bridges. *J. Bridg. Eng.* 2018, 23, 04018001. [CrossRef]
39. Makris, N.; Zhang, J. Seismic Response Analysis of a Highway Overcrossing Equipped with Elastomeric Bearings and Fluid Dampers. *J. Struct. Eng.* 2004, 130, 830–845. [CrossRef]
40. Rezaeian, S. Stochastic Modeling and Simulation of Ground Motions for Performance-Based Earthquake Engineering. Ph.D. Dissertation, University of California Berkeley, Berkeley, CA, USA, 2010.
41. Lee, V.W. Empirical scaling of strong earthquake ground motion: Part III: Synthetic strong motion. *ISET J. Earthq. Technol.* 2002, 39, 273–310.
42. Hart, E.W.; Bryant, W.A.; Manson, M.W.; Kahle, J.E. Summary Report: Fault Evaluation Program 1984–1985, South Coast Ranges Region and Other Areas, Calif; Open-File Report 86-3; Depart of Conservation, Division of Mines and Geology: Sacramento, CA, USA, 1986.

43. Jennings, C.W.; Saucedo, G.J. Fault Activity Map of California and Adjacent Areas, with Locations and Ages of Recent Volcanic Eruptions; California Department of Conservation, Division of Mines and Geology: Sacramento, CA, USA, 1994.

44. Petersen, M.D.; Wesnousky, S.G. Fault slip rates and earthquake histories for active faults in southern California. Bull. Seismol. Soc. Am. 1994, 84, 1608–1649.

45. Makris, N.; Zhang, J. Structural Characterization and Seismic Response Analysis of a Highway Overcrossing Equipped with Elastomeric Bearings and Fluid Dampers: A Case Study; PEER Report 2002/17; Pacific Earthquake Engineering Research Center: Berkeley, CA, USA, 2002.

46. Trifunac, M.D.; Brady, A.G. On the correlation of seismic intensity scales with the peaks of recorded strong ground motion. Bull. Seismol. Soc. Am. 1975, 65, 139–162.

47. Bulajić, B.; Manić, M.; Ladinović, D. Effects of shallow and deep geology on seismic hazard estimates: A case study of pseudo-acceleration response spectra for the northwestern Balkans. Nat. Hazards. 2013, 69, 573–588. [CrossRef]

48. Bulajić, B.D.; Bajić, S.; Stojnić, N. The effects of geological surroundings on earthquake-induced snow avalanche prone areas in the Kopaonik region. Cold Reg. Sci. Technol. 2018, 149, 29–45. [CrossRef]

49. PEER Center. PEER Ground Motion Database; PEER NGA-West2 Database 2013/03; Pacific Earthquake Engineering Research Center Headquarters at the University of California: Berkeley, CA, USA, 2013.

50. Li, Z.; Kotronis, P.; Wu, H. Simplified approaches for Arias Intensity correction of synthetic accelerograms. Bull. Earthq. Eng. 2017, 15, 4067–4087. [CrossRef]

51. Gasparini, D.A.; Vanmarcke, E.H. Simulated Earthquake Motions Compatible with Prescribed Response Spectra; Department of Civil Engineering Research Report No. R76-4; Massachusetts Institute of Technology: Cambridge, MA, USA, 1976.

52. Attanasi, G.; Auricchio, F.; Fenves, G.L. Feasibility Assessment of an Innovative Isolation Bearing System with Shape Memory Alloys. J. Earthq. Eng. 2009, 13, 18–39. [CrossRef]

53. Mauro, D.; Donatello, C.; Roberto, M. Implementation and testing of passive control devices based on shape memory alloys. Earthq. Eng. Struct. Dyn. 2000, 29, 945–968.

54. Auricchio, F. A robust integration-algorithm for a finite-strain shape-memory-alloy superelastic model. Int. J. Plast. 2001, 17, 971–990. [CrossRef]

55. Deb, K.; Pratap, A.; Agarwal, S.; Meyarivan, T. A fast and elitist multiobjective genetic algorithm: NSGA-II. IEEE Trans. Evol. Comput. 2002, 6, 182–197. [CrossRef]

56. Srinivas, N.; Deb, K. Multiobjective Optimization Using Nondominated Sorting in Genetic Algorithms. Evol. Comput. 1994, 2, 221–248. [CrossRef]

57. Kim, H.; Chang, C.; Kang, J. Evaluation of microvibration control performance of a smart base isolation system. Int. J. Steel Struct. 2015, 15, 1011–1020. [CrossRef]

58. Tubaldi, E.; Mitoulis, S.A.; Ahmadi, H. Comparison of different models for high damping rubber bearings in seismically isolated bridges. Soil Dyn. Earthq. Eng. 2018, 104, 329–345. [CrossRef]

59. Rad, A.R.; Banazadeh, M. Probabilistic risk-based performance evaluation of seismically base-isolated steel structures subjected to far-field earthquakes. Buildings 2018, 8, 128.

60. Chimamphant, S.; Kasai, K. Comparative response and performance of base-isolated and fixed-base structures. Earthq. Eng. Struct. Dyn. 2016, 45, 5–27. [CrossRef]