Optimal design of superelastic-friction base isolators for seismic protection of highway bridges against near-field earthquakes

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SUMMARY

The seismic response of a multi-span continuous bridge isolated with novel superelastic-friction base isolator (S-FBI) is investigated under near-field earthquakes. The isolation system consists of a flat steel-Teflon sliding bearing and a superelastic NiTi shape memory alloy (SMA) device. The key design parameters of an S-FBI system are the natural period of the isolated bridge, the yielding displacement of the SMA device, and the friction coefficient of the sliding bearings. The goal of this study is to obtain optimal values for each design parameter by performing sensitivity analysis of a bridge isolated by an S-FBI system. First, a three-span continuous bridge is modeled as two-degrees-of-freedom with the S-FBI system. A neuro-fuzzy model is used to capture rate- and temperature-dependent nonlinear behavior of the SMA device. Then, a set of nonlinear time history analyses of the isolated bridge is performed. The variation of the peak response quantities of interest is shown as a function of design parameters of the S-FBI system and the optimal values for each parameter are evaluated. Next, in order to assess the effectiveness of the S-FBI system, the response of the bridge isolated by the S-FBI system is compared with the response of the non-isolated bridge and the same bridge isolated by pure-friction (P-F) sliding isolation system. Finally, the influence of temperature variations on the performance of the S-FBI system is evaluated. The results show that the optimum design of a bridge with the S-FBI system can be achieved by a judicious specification of design parameters.

KEY WORDS: superelasticity; structural control; damper; optimization; shape memory alloy; earthquake

1. INTRODUCTION

In past decades, various control strategies have been proposed in order to prevent structural and nonstructural failures in civil engineering structures such as buildings, bridges, nuclear reactors and liquid storage tanks during earthquakes. Among these strategies, base isolation has become most widely used technique and proven to be an effective means for reducing seismic response of structures. In general, a seismic isolation system should have (i) high lateral flexibility in order to lengthen the period of the structure to reduce lateral earthquake forces, (ii) adequate energy dissipation capacity and a good re-centering mechanism to avoid excessive bearing deformations and instability, (iii) a means of providing rigidity under service load levels and (iv) high vertical load carrying capacity. Also, high durability against cyclic loads and ability to recover residual deformations on the bearing after an excitation are among the favorable properties of an isolation device. A number of isolation systems have

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been suggested and developed for various types of structures [1]. Sliding isolation systems are among the most popular isolation schemes that have been proposed. These types of isolation systems include but are not limited to pure-friction (P-F) base isolation system, friction pendulum system (FPS), and resilient-friction base isolation system (R-FBI). The simplest sliding isolator is a P-F isolation system which consists of a stainless steel plate bearing against a low friction interface material such as polytetrafluoroethylene (PTFE). In a P-F system, the shear force transmitted to the superstructure is limited to the maximum friction force developed across the sliding interface of the bearing. The energy is consistently dissipated through Coulomb damping. Also, since the center of mass of the bearings automatically coincides with the center of mass of the supported structure, torsional effects are minimized. Yet, P-F systems lack re-centering ability. The R-FBI system overcomes this disadvantage by introducing a rubber central core which provides elastic restoring force and the FPS isolators combine the concept of sliding systems with the action of the pendulum to provide restoring force.

Although base isolation system has been proven to be an effective method of reducing seismic response of structures, the performance of base-isolated structures against near-field earthquakes has been questioned in recent years [2]. Near-field earthquakes are characterized with long period and large velocity pulses in the velocity time history. Since the period of these pulses usually coincides with the period of isolated structures, ground motions with near-field characteristics amplify seismic response of the isolation system. Another characteristic of the near-field motions that adversely influences base isolation systems is that the ground motion normal to the fault trace is richer in long-period spectral components than that parallel to the fault. Isolation bearings experience large deformations due to this normal component of the near-field motions [3]. To accommodate large isolator displacements, the size of the isolation device and the required seismic gap significantly increases. Besides these requirements, the need for flexible utility connections adds extra cost [4]. Furthermore, if an adequate seismic gap is not provided, undesirable pounding effects may occur.

Bridges play an important role in the transportation network on which goods and people are transported, and failure of them not only will result in an interruption of this basic need but also impede the relief and rescue efforts. In recent years, the damaging effects of near-field motions on highway bridges have revealed the lack of conventional design methods and emphasized the need of innovative design strategies. Numerous bridges have been damaged or collapsed during the 1994 Northridge, 1995 Kobe, 1999 Duzce and 1999 Chi Chi earthquakes [5-9]. In the most recent 2008 Winchuan earthquake, many highway bridges are either severely damaged or completely collapsed in China, leading to not only significant economic losses but also large loss of lives due to the transportation supply disruption and the lack of access to medical care [9].

In order to reduce large displacement response of isolated bridge structures during near-field earthquakes, several researchers have proposed the use of supplemental dampers. Some studies are focused on the use of passive devices for additional energy dissipation [10-12]. A considerable number of studies have explored the effectiveness of semi-active devices for mitigating the response of isolated bridges [13-15]. Symans and Kelly [16] investigated the performance of the variable fluid dampers operated by fuzzy logic controllers for seismic protection of isolated bridges. Sahasrabudhe and Nagarajaiah [17] experimentally and analytically studied the performance of a base-isolated bridge model that employs sliding bearings and MR dampers. Madhekar and Jangid [18] evaluated the response of an isolated
highway bridge with variable dampers developed from MR dampers by conducting extensive numerical simulations.

Over the past decade, shape memory alloys (SMAs) have received considerable attention as a smart material that can be employed in vibration control of civil structures [19]. SMAs possess two unique properties: shape memory effect and superelastic effect. Both of these peculiar capabilities of SMAs depend largely on solid-to-solid phase transformations from a crystallographically more ordered phase known as austenite to a crystallographically less ordered phase known as martensite [20]. These transformations can be induced by temperature (shape memory effect) or stress (superelastic effect) variations. Although a few researchers have studied to exploit shape memory effect of SMAs as actuators in active vibration control [21-22], most of the studies have focused on implementing superelastic SMAs for passive structural control.

Superelastic SMAs can fully regain their original shape upon being deformed. This shape recovery is also accompanied by a considerable hysteresis loop which signifies energy dissipation during loading-unloading cycle. Besides its unique re-centering ability and considerable energy dissipating capacity, superelastic SMAs have favorable mechanical behaviors such as the ability to undergo large deformations, excellent fatigue resistance and high corrosion resistance [23]. Due its attractive properties, many researchers investigated the use of superelastic SMAs in a wide range of seismic applications [24-32].

Among various applications of superelastic SMAs, the development of new base isolation systems based on superelastic behavior of SMAs has recently attracted interest of several researchers. Some researchers have proposed isolation systems that combine a rubber bearing with an SMA device [33-34] while some other researchers have studied sliding isolation systems which are coupled with SMA elements. As an example of SMA-based sliding isolators, Casciati et al. [35] designed an innovative sliding isolation device that employs CuAlBe shape memory alloys to limit the displacement of the isolator, dissipate energy and provide re-centering force. Cardone et al. [36] evaluated the effectiveness of an SMA-based isolation system that consists of a flat sliding bearing and SMA re-centering device by conducting full-scale experimental tests on a three-story reinforced concrete structure. Attanasi et al. [37] performed a parametric study to investigate the feasibility of integrating superelastic SMAs to seismic isolation devices.

In a recent study, Ozbulut and Hurlebaus [38] explored the effect of temperature changes on the performance of sliding-type isolators with SMA device used for the seismic protection of highway bridges. They used a genetic algorithm for selection of design parameters of SMA device. By conducting extensive time-history analyses of a five-span continuous bridge equipped with sliding bearings and SMA devices, it was shown that the temperature has a modest effect on the performance of isolated bridge. In particular, it was found that the behavior of sliding bearings and SMA device counterbalance each other in the case of a change in ambient temperature and SMA-based sliding isolators can be a viable system for seismic protection of bridges.

The focus of this study is to investigate the optimum design parameters of a superelastic-friction base isolator (S-FBI) that is installed between piers and superstructure on a bridge to reduce structural responses during near-field earthquakes. The S-FBI system consists of a steel-Teflon sliding bearing that filters out the earthquake forces by providing frictional sliding interfaces and a superelastic SMA device that provides a re-centering mechanism and absorbs seismic energy through hysteresis of SMA elements. First, a neuro-fuzzy model of
superelastic SMAs that is capable of simulating the behavior of NiTi wires considering loading rate and temperature effects is introduced. Then, the model of a three-span continuous bridge with the S-FBI system is developed as a two-degrees-of-freedom system. In order to generate near-field earthquakes that are used as external excitations in the simulations, the time domain response spectral matching of six historical records is performed with the program RspMatch2005. Next, a sensitivity analysis is performed to evaluate the optimum values of design parameters of the S-FBI system for mitigating the response of the highway bridges against near-field earthquakes. In order to assess the benefits of the S-FBI system, the performance of a bridge isolated by an optimal S-FBI system is compared with the response of a comparable non-isolated bridge and same bridge isolated by the P-F isolation system. Finally, the effect of the outside temperature on the performance of the S-FBI system is investigated.

2. A NEURO-FUZZY MODEL FOR SHAPE MEMORY ALLOYS

A reliable model that describes highly complex behavior of superelastic SMAs is needed in order to explore potential application of SMAs as a seismic isolation system component. Earlier studies indicate that temperature and loading-rate have a significant influence on the superelasticity of SMAs [39-40]. In order to illustrate the temperature and loading-rate dependent behavior of SMAs, Figure 1 shows strain-stress curves of superelastic NiTi wires with a diameter of 1.5 mm at various conditions. The left subplot shows hysteresis loops at different temperatures for a loading frequency of 1 Hz, while the right subplot is given for various loading-rates at room temperature. It can be seen from the left subplot that an increase in the temperature shifts markedly hysteresis loops upward. A similar but less pronounced vertical shift is also notable when the loading frequency increases. Note that also the lower transformation plateau shifts more than the upper transformation plateau and thus the area of hysteresis loops gets smaller with the increasing loading-rate.

Since SMAs will be exposed to dynamic effects and outside temperature changes when they are implemented as a bridge isolation system component, it is important to consider the degree to which the mechanical response of SMAs is affected by variations of loading-rate and temperature. In this study, a neuro-fuzzy model described thoroughly in the work by Ozbulut and Hurlebaus [41] is employed to simulate the superelastic behavior of NiTi shape memory alloys. This model is capable of capturing rate- and temperature-dependent material response while it remains simple enough to carry out numerical simulations.
The first step to create a neuro-fuzzy model is to collect experimental data. The SMA wires considered here are obtained from SAES Smart Materials and have a chemical composition of 55.8% nickel by weight and the balance titanium. According to tests conducted by the wire manufacturer, the austenite start and finish temperatures are \( A_s = -10^\circ C \) and \( A_f = 5^\circ C \), respectively. The uniaxial tensile tests are conducted on SMA wires with a diameter of 1.5 mm at a frequency range 0.5-2 Hz and at a temperature range of 0-40ºC to obtain data. In order to stabilize hysteretic loops, a training test procedure that consists of 10 load cycles with strain amplitude of 6% at 0.04 Hz and at room temperature is applied to all samples [41].

Then, adaptive neuro-fuzzy inference system (ANFIS) which can create a model that approximate a real data set by mapping a set of inputs to a target output is employed to represent superelastic behavior of SMAs. ANFIS combines the abilities of fuzzy theory and neural networks. In particular, ANFIS employs neural network strategies to develop a fuzzy inference system (FIS) whose parameters (membership functions and rules) cannot be predetermined by user’s knowledge. ANFIS can learn information about the data set by employing a hybrid method which combines backpropagation algorithm and least square estimation. Then, it adjusts fuzzy system parameters based on input/output pairs of data.

Here, an initial FIS with three inputs (strain, strain rate and temperature) and a single output (stress on the SMA wire) is created. After several trials, three Gaussian membership functions are selected for strain, and two Gaussian membership functions are chosen for both strain rate and temperature to fuzzify each input. Also, twelve if-then rules are defined to map the input variables to single output. This initial FIS and its membership function and rules are randomly generated, and need to be tuned by ANFIS to predict the correct output for given inputs. Data collected from experimental tests is concatenated in order to set up training, checking and validation data sets for ANFIS simulations. ANFIS uses training data to learn about the characteristics of SMA material response while it employs checking data to avoid overfitting. Finally, the validation data which is a new set of data that has not been used during training or checking is used to validate the trained FIS. Figure 2 illustrates stress-strain relationship of superelastic SMAs for the experimental results and fuzzy model prediction. Each subplot of Figure 2 is given for a different maximum strain amplitude and loading frequency as well as different temperature. The strain-stress curves are plotted for 3
loading cycles in each subplot. It can be seen that the obtained hysteresis loops are stable and the developed model satisfactorily imitate the mechanical response of NiTi SMA wires at each condition.

![Figure 2](attachment:image.png)

Figure 2. Strain-stress curves at various conditions for experimental results and ANFIS prediction.

3. MODEL OF ISOLATED BRIDGE STRUCTURE

A three-span continuous bridge shown in Figure 3 is selected for the sensitivity analysis [42]. The deck of the bridge has a mass of $771.12 \times 10^3$ kg, and the mass of each pier is $39.26 \times 10^3$ kg. The bridge has a total length of 90 m, and each pier is 8 m tall. The moment of inertia of piers and Young’s modulus of elasticity are given as $0.64$ m$^4$ and $20.67 \times 10^9$ N/m$^2$, respectively. The fundamental period of the non-isolated bridge is 0.45 s. The isolated bridge is modeled as a two-degree-of-freedom system with the S-FBI system. It is assumed that required separation distance between bridge deck and abutment is provided, i.e., pounding effects are avoided. Since the isolation systems installed at the abutment and pier have similar characteristics and therefore, the seismic response of the bridge at the abutment and pier have the same trend, only an internal span is modeled. The equations of motion are given as

$$m_1\ddot{u}_1(t) + c_1\dot{u}_1(t) + k_1u_1(t) - F_{IS}(u_1, \dot{u}_1, u_2, \dot{u}_2, t) = -m_1\ddot{u}_g (t)$$

$$m_2\ddot{u}_2(t) + F_{IS}(u_1, \dot{u}_1, u_2, \dot{u}_2, t) = -m_2\ddot{u}_g (t)$$

where $m_1$, $m_2$ and $u_1$, $u_2$ are the masses and displacements of the pier and deck, respectively, $c_1$ and $k_1$ represent the coefficient of damping and stiffness of piers, and $\ddot{u}_g$ is the ground acceleration. $F_{IS}$ denotes the restoring force of the S-FBI system. Hence, $F_{IS}$ is the sum of the nonlinear force of the SMA device and frictional resistance force of steel-Teflon sliding bearings. A hysteretic model is used to simulate the force of the sliding bearings [43]. The frictional force at a sliding interface is given by

$$F_f = \mu WZ$$

where $\mu$ represents the coefficient of friction, $W$ is the normal load carried by the bearing interface, and $Z$ is a hysteretic dimensionless quantity computed from following equation

$$Y\dot{Z} + \gamma|\dot{u}_b|Z|Z|^{n-1} + \beta\dot{u}_b|Z|^{n} - \dot{u}_b = 0,$$
where $Y$ is the yield displacement of the sliding bearing chosen as 0.0005 m and, $\gamma$, $\beta$, and $n$ are dimensionless parameters that control the shape of the hysteretic curve and have the values of 0.9, 0.1 and 1, respectively. Also, $u_b = u_2 - u_1$ is the deformation of the sliding bearings. Fuzzy model described above is used to compute the instantaneous force from the SMA elements.

Figure 3. Model of a three-span isolated bridge

4. GROUND MOTIONS USED FOR ANALYSES

Selection and modification of ground motion records that are used in dynamic time history analyses have a significant influence on the results of the analyses. The ground motion records mainly fall into three categories: (i) artificial accelerograms, (ii) synthetic accelerograms generated from seismological source models and (iii) real accelerograms recorded during earthquakes [44]. Several programs are developed to generate seismic records belong to the first two categories mentioned above and they are freely available for engineers. Neverthless, it has been revealed that the strong-motion accelerograms generated by using these methods have several drawbacks such as having unreasonably high energy content or requiring definition of many parameters to characterize earthquake source [44]. Since real accelerograms do not possess these shortcomings by definition and are increasingly available via digital seismic networks, they are becoming the most attractive option for dynamic time-history analyses. However, selection and scaling of real records for dynamic analysis of structures requires special attention. A review on different strategies used for selection of real ground motion records can be found in [45]. An evaluation of the earlier
works related to record selection points out that the spectral shape is the most important ground motion characteristic in modification of real records [46].

A variety of methods have been proposed to modify a historical time history so that its response spectrum is compatible with a given target spectrum [47]. One approach used commonly for generating response spectrum compatible ground motions is to adjust Fourier amplitude spectra in the frequency domain. Although it is a straightforward method and provides a close match to the target spectrum, it also has significant potential problems such as distorting the energy characteristics of accelerograms and producing very unrealistic seismic demands [48]. Hancock et al. [49] have proposed an alternative approach that performs spectral matching in time domain using wavelets. The method, known as RspMatch2005, can simultaneously match spectra at multiple damping values while preserving the non-stationary character of the reference time history. Unlike the spectral matching in the frequency domain, RspMatch2005 does not corrupt the velocity and displacement time histories and avoids creating ground motions with unrealistic energy content. In this study, the program RspMatch2005 is used to generate spectrum compatible real ground motion records for dynamic time history analyses of the isolated bridge.

A response spectrum constructed as per the International Building Code [50] for a site in southern California, assuming firm rock conditions is selected as the target spectrum [51]. A total of six historical California earthquakes which presents near-field characteristics are selected as seed accelerograms. The characteristics of the ground motions such as magnitude, the closest distance to the fault plane, peak ground acceleration and velocity, and significant duration are given in Table I. Figure 4 shows the target response spectrum used in the analysis and response spectra of selected ground motions for 5% damping level.

<table>
<thead>
<tr>
<th>Earthquake</th>
<th>Magnitude ($M_w$)</th>
<th>Distance (km)</th>
<th>PGA (g)</th>
<th>PGV (cm/s)</th>
<th>Duration (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1979 Imperial Valley</td>
<td>6.5</td>
<td>1.0</td>
<td>0.44</td>
<td>109.8</td>
<td>8.5</td>
</tr>
<tr>
<td>1986 N. Palm Springs</td>
<td>6.0</td>
<td>8.2</td>
<td>0.59</td>
<td>73.3</td>
<td>4.5</td>
</tr>
<tr>
<td>1994 Sylmar</td>
<td>6.7</td>
<td>6.2</td>
<td>0.90</td>
<td>102.8</td>
<td>9.0</td>
</tr>
<tr>
<td>1971 San Fernando</td>
<td>6.6</td>
<td>2.8</td>
<td>1.22</td>
<td>112.5</td>
<td>3.8</td>
</tr>
<tr>
<td>1992 Landers</td>
<td>7.3</td>
<td>1.1</td>
<td>0.72</td>
<td>97.6</td>
<td>13.1</td>
</tr>
<tr>
<td>1989 Loma Prieta</td>
<td>6.9</td>
<td>6.1</td>
<td>0.56</td>
<td>94.8</td>
<td>10.2</td>
</tr>
</tbody>
</table>

The selected seed accelerograms are adjusted using RspMatch2005 in order to simultaneously match 5%, 10% and 25%-damped response spectra. Figure 5 shows the spectrally matched response spectrum of Landers and Loma Prieta earthquakes for different damping levels. Note that spectral misfit is reduced significantly for all damping levels.
Figure 4. The target response spectrum compared to response spectra of the selected ground motions

Figure 5. The spectrally matched response spectra of Landers and Loma Prieta earthquakes for different damping levels

5. NUMERICAL STUDIES

In this section, the optimum design parameters of the S-FBI system for seismic protection of bridges against near-field earthquakes are investigated. Figure 6 shows the typical force-deformation curves of the sub-components of the S-FBI system, i.e. the SMA device and steel-Teflon sliding bearing and the combined hysteresis. Here, the SMA device has a simple design, which avoids extra fabrication costs. It simply consists of multiple loops of superelastic NiTi wires wrapped around two wheels. The S-FBI system is characterized by the natural period of the isolated bridge $T_b$, the yield displacement of the SMA device $u_y$, and the friction coefficient of sliding bearings $\mu$. The natural period of the isolated bridge can be computed as
where $m_d$ is the mass of the bridge deck and $\alpha k_{SMA}$ denotes post-yield stiffness of the SMA device. Here, $\alpha$ which represents the ratio of post-yield stiffness and initial stiffness of the SMA device is taken as 0.1 and the yield strain of SMA wires is chosen to be 1%, which are typical values for NiTi shape memory alloy wires. Hence, the design of the S-FBI system requires the specification of three parameters: $T_b$ and $u_y$ (to determine the area and the length of SMA wires) and $\mu$.

$$T_b = 2\pi \sqrt{\frac{m_d}{\sum c k_{SMA}}}$$

In order to evaluate the effects of these design parameters of the S-FBI system on the seismic response of the bridge, nonlinear time-history analyses are performed by solving the governing equations of motion of the isolated bridge. As external excitation, the six near-field ground motion records described above are employed. The response quantities evaluated here are peak relative displacement of the deck, peak absolute acceleration of the deck and peak base shear normalized by the weight of the deck.

Figure 6. Force-deformation curves of the S-FBI system and its sub-components.

Figure 7 illustrates the variation of peak response quantities with the natural period of the isolated bridge for various earthquakes. The values of $\mu$ and $u_y$ are selected as 0.10 and 30 mm, respectively. It can be seen that the peak deck acceleration rapidly decreases when the isolation period increases. On the other hand, the peak deck drift starts to increase for some excitation cases with the increasing isolation period. Also, the peak normalized base shear decreases when isolation period increases from 2.0 s to about 3.5 s. However, for the higher values of isolation period, it stays nearly constant.

Figure 8 shows the effect of yield displacement of the SMA device on the peak response quantities for different near-field ground motions. The results are obtained for $T_b = 4.0$ s and $\mu = 0.10$. It is observed that there is not considerable change in the peak response of the isolated bridge for different values of $u_y$. Nevertheless, a moderate increase in peak deck acceleration and a modest decrease in peak deck drift for some excitation cases are present. Also, there is a slight increase in peak normalized base shear for some earthquakes for higher values of $u_y$. Since the larger values of $u_y$ imply longer SMA wire length, the value of $u_y$ can be kept small (between 20-30 mm) without any performance degradation in order to reduce the amount of SMA material needed. Note that further reducing the yield displacement ($u_y < 20$ mm) results in large strain values for SMA wire. Since the maximum superelastic strain is about 6% for NiTi wires and significant strain hardening occurs for the larger strain values, the seismic demand on piers may considerable increase for very small values of $u_y$. 

Figure 7. Variation of peak response quantities with the natural period of the isolated bridge.

Figure 8. Effect of yield displacement of the SMA device on the peak response quantities.
Figure 7. Variations of peak response quantities with the natural period of the isolated bridge.

Figure 8. Variations of peak response quantities with yield displacement of the SMA device.

The variation of peak response quantities for different values of friction coefficient of sliding bearings is shown in Figure 9 considering $T_b = 4.0$ s and $u_y = 30$ mm. It can be seen that increasing the friction coefficient regularly decreases the peak deck drift while increasing the peak deck acceleration. However, note that the increase in the peak deck acceleration occurs almost at a constant rate while the rate of the decrease in the peak deck drift reduces at larger values of $\mu$. Moreover, for most excitation cases, the peak normalized base shear initially decreases with an increase in $\mu$ and then it tends to increase for higher values of $\mu$. 
Figure 9. Variation of peak response quantities with friction coefficient of sliding bearings

The effect of interaction between the isolation period $T_b$ and friction coefficient of sliding bearings $\mu$ is examined through three-dimensional (3-D) plots shown in Figures 10-12. Each of these 3-D plots presents the variation of peak response quantities of the isolated bridge with respect to isolation period and friction coefficient simultaneously for different near-field earthquakes. As shown in Figure 10, the peak deck drift obtains large values for small values of the friction coefficient and it reduces with an increase in the friction coefficient almost for all isolation periods. However, the rate of this reduction rapidly decreases and the surface becomes substantially flat when the friction coefficient is over 0.15 for most of the excitations. Also, there exists an increase in the peak deck drift for the increasing values of isolation period, yet; the isolation period seems to have negligible effect when the friction coefficient is large. It can be seen from Figures 11 and 12 that the peak deck acceleration and normalized base shear attain high values for small values of isolation period for various friction coefficient values. The larger values of isolation period ameliorate the acceleration response of the deck and decrease the normalized base shear. Note that the highest values of peak deck acceleration and normalized base shear is observed for the smallest $T_b$ and largest $\mu$ while peak deck drift has its largest value for the largest $T_b$ and smallest $\mu$.

It can be concluded from the observations in Figures 7 to 12 that the optimal value of isolation period $T_b$ which effectively reduces the deck drift and simultaneously control superstructure acceleration is between 3.5s and 4s. Also, the optimum value of the friction coefficient $\mu$ of an S-FBI system used for seismic response control of bridges against near-field earthquakes is in the vicinity of 0.15. Increasing $\mu$ over 0.15 reduces deck displacement slightly more at an expense of considerable increase in peak acceleration response of the deck and peak normalized base shear.
Figure 10. Variations of peak deck drift with isolation period and friction coefficient.
Figure 11. Variations of peak deck acceleration with isolation period and friction coefficient.
Figure 12. Variations of peak normalized base shear with isolation period and friction coefficient.

The time histories of various response quantities of the bridge isolated by the S-FBI system are illustrated in Figures 13 and 14 for Imperial Valley earthquake and in Figures 16 and 17 for Loma Prieta earthquake. The peak values of each response quantity are also given in the figures. The S-FBI system parameters $T_b$, $u_y$, $\mu$ are specified as 4.0 s, 30 mm, and 0.10, respectively. In order to serve as a benchmark for evaluating effectiveness of the S-FBI
system, the responses of the bridge isolated by pure-friction (P-F) base isolation system with a friction coefficient of 0.10 are also demonstrated in the figures mentioned above. Moreover, the peak values of each response quantity of the non-isolated bridge are provided in Table II for the same comparison purposes. Also, the force-deformation curves of the SMA device and the steel-Teflon sliding bearings as well as the overall S-FBI isolation system are shown in Figures 15 and 18 for Imperial Valley and Loma Prieta earthquakes, respectively.

It can be seen that the isolation of bridge with the S-FBI system significantly decreases the peak deck acceleration when compared with the non-isolated bridge. Note that as compared to P-F system, the bridge isolated by the S-FBI system produces 63% and 66% more reduction in the peak deck acceleration for Imperial Valley and Loma Prieta earthquakes, respectively. Also, in comparison with the non-isolated bridge, peak normalized base shear experiences 69% and 73% decreases for Imperial Valley earthquake and 82% and 85% decreases for Loma Prieta earthquake when the bridge is isolated by the S-FBI system or the P-F system, respectively. Moreover, the S-FBI system successfully reduces the peak bearing deformations and recovers the deformations after the seismic event. On the other hand, the P-F system experience excessive deformations and has large residual deformations since it lacks re-storing force capability.

<table>
<thead>
<tr>
<th>Responses of non-isolated bridge</th>
<th>Imperial Valley</th>
<th>Loma Prieta</th>
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<tbody>
<tr>
<td>Peak deck drift (mm)</td>
<td>73</td>
<td>86</td>
</tr>
<tr>
<td>Peak deck acceleration (g)</td>
<td>1.92</td>
<td>2.27</td>
</tr>
<tr>
<td>Peak normalized base shear</td>
<td>1.62</td>
<td>1.99</td>
</tr>
</tbody>
</table>

Finally, since the mechanical properties of both SMA wires and steel-Teflon sliding bearings are considerably influenced by environmental temperature changes, the effect of outside temperature on the performance of the S-FBI system is investigated. In order to consider the temperature effect on the friction coefficient of steel-Teflon bearings, the friction coefficient is computed as

$$\mu = \mu_{\text{max}} - \Delta \mu \exp(-a|\mu|),$$

(5)

where $\mu_{\text{max}}$ is the coefficient of friction at very high velocities, $\Delta \mu$ is the difference between the coefficient of friction at very high and very low velocities, and $a$ is a constant. Dolce et al. [52] specified the parameters $\mu_{\text{max}}, \Delta \mu,$ and $a$ for different combination of bearing pressure, condition of interface and temperature. Here, the values of $\mu_{\text{max}}, \Delta \mu,$ and $a$ for three different temperatures and for a 28.1 MPa bearing pressure and non-lubricated bearing interface are approximated from the study of Dolce et al. [52] and given in Table III. The fuzzy model described earlier is used to predict the force of the SMA device at different temperatures. Also, the values of $T_b$ and $u_y$ are selected as 4.0 s and 30 mm, respectively.
Figure 13. Time histories of pier displacement and deck drift for a bridge isolated by the S-FBI system or the P-F system under Imperial Valley earthquake.

Figure 14. Time histories of deck acceleration and normalized base shear for a bridge isolated by the S-FBI system or the P-F system under Imperial Valley earthquake.

Figure 15. Force-deformation curves of the S-FBI system and its sub-components under Imperial Valley earthquake.
Figure 16. Time histories of pier displacement and deck drift for a bridge isolated by the S-FBI system or the P-F system under Loma Prieta earthquake.

Figure 17. Time histories of deck acceleration and normalized base shear for a bridge isolated by the S-FBI system or the P-F system under Loma Prieta earthquake.

Figure 18. Force-deformation curves of the S-FBI system and its sub-components under Loma Prieta earthquake.
Table III. Model parameters for different temperatures

<table>
<thead>
<tr>
<th>T (°C)</th>
<th>$\mu_{\text{max}}$ (%)</th>
<th>$\Delta \mu$ (%)</th>
<th>$a$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>11.07</td>
<td>6.78</td>
<td>23.3</td>
</tr>
<tr>
<td>20</td>
<td>10.26</td>
<td>7.13</td>
<td>22</td>
</tr>
<tr>
<td>40</td>
<td>9.86</td>
<td>7.11</td>
<td>18.7</td>
</tr>
</tbody>
</table>

The influence of temperature changes on peak response quantities of the isolated bridge is illustrated in Figure 19. It can be seen that the effectiveness of the S-FBI system in reducing peak deck drift mostly increases with an increase in temperature. As temperature reduces to 0°C compared to a reference temperature of 20°C, the peak deck drift experience a maximum of 31% increase for San Fernando earthquake; yet, the same increase is in the range of 2-10% for all other cases. On the other hand, there is a maximum of 19% reduction in peak deck drift when the outside temperature rises to 40°C from its reference value. It is also observed that peak deck acceleration attains larger values as temperature increases. In particular, it changes about ±10% when temperature differs ±20°C from the reference temperature of 20°C. Furthermore, seismic demand on piers experience an increase when temperature increase to 40°C compared to reference temperature, while it reduce as temperature drops to 0°C for all cases except San Fernando and Loma Prieta earthquakes. Note that the changes in the peak base shear are in the range of 2-19%. Overall, it can be concluded that a ±20°C variation in environmental temperature compared to the reference temperature of 20°C does not significantly affect the performance of the S-FBI system.

Figure 19. Variations of peak response quantities with environmental temperature.

6. CONCLUSION

This study explores the optimum design parameters of a superelastic-friction base isolation system for seismic protection of highway bridges subjected to near-field earthquakes. The S-FBI system consists of a steel-Teflon sliding bearing and an SMA device. While the sliding bearing decouples the superstructure of the bridge from its piers and dissipates energy through
friction, the SMA device provides restoring force and additional damping. The design parameters of the S-FBI system chosen for the investigation includes the natural period of the isolated bridge $T_h$, the yield displacement of the SMA device $u_y$, and the friction coefficient of sliding bearings $\mu$. In order to generate ground motions used in the simulations, a time domain method which employs wavelets to adjust real accelerograms to match a target response spectrum with minimum changes on the other characteristics of ground motions is used. Time-history analyses of the isolated bridge are performed to evaluate the variation of peak response quantities with the design parameters of the S-FBI system. It is found that the optimum value of $T_h$ based on restraining both displacement and acceleration response of the deck is in the range of 3.5 s - 4.0 s under near-field earthquakes. Also, it is observed that peak response quantities are not much influenced by the variation of the yield displacement of the SMA device. Therefore, smaller values are recommended for $u_y$ in order to reduce the length of the SMA elements used for the device. It is also observed that increasing the friction coefficient ameliorates peak displacement response of the deck while adversely affecting the peak acceleration response of the deck. It can be said that selecting a value between 0.10 and 0.15 for $\mu$ yields optimum results for isolated bridges subjected to near-field motions. Finally, it is shown that there is not a significant change in the response of the bridge isolated by the S-FBI system as temperature varies in the range of 0 - 40°C. The results indicate that the S-FBI system can effectively mitigate the response of highway bridges against near-field earthquakes when the design parameters of the S-FBI system are judiciously selected.

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