# On the effectiveness of rotational friction hinge damper to control responses of multi-span simply supported bridge to non-uniform ground motions

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**Abstract:** Base isolation techniques have been extensively used to improve the seismic performance of bridge structures. The decoupling of the bridge decks from piers and abutments using rubber isolator could result in significant reduction of seismic forces transmitted to bridge substructures. However, the isolation devices could also increase the deck displacement thus enhance the possibility of pounding and unseating damage of bridge decks. Moreover, previous investigations have shown pounding and unseating damages on isolated bridges exacerbate due to the spatial variation of earthquake ground motions. Recent earthquakes revealed that isolation bearing could also be damaged due to the excessive movements of decks during large earthquake events. This study proposes the use of Rotational Friction Hinge Dampers (RFHD) to mitigate the damages that could be induced by large displacement of bridge decks, particularly focusing upon pounding and unseating damages and bearing damages. The device is capable of providing large hysteretic damping and the cost of installing the devices is relatively economical. This paper presents numerical investigations on the effectiveness of these devices on a typical Nepalese simply supported bridge subjected to spatially varying ground motions. The results indicate that RFHDs are very effective in mitigating relative displacement and pounding force, as well as controlling the bearing deformation and pier drift. It is also revealed that the effectiveness of the device is not significantly affected by small changes in the slip forces, thus small variations of the optimum slip forces during the lifetime of the bridge do not warrant any adjustment or replacement of the device.

**Key words:** rotational friction hinge device, pounding, unseating, spatially varying ground motions, simply supported bridges.

(Accepted in the journal **Advances in Structural Engineering**)

# 1. INTRODUCTION

Highway bridges are one of the key components of a transportation network and they carry significant importance in providing emergency services after an earthquake. Past and recent earthquakes, such as 1971 San Fernando earthquake, 1994 Northridge earthquake, 1995 great Hanshin earthquake, 2010 Chile earthquake and many more revealed that bridges are vulnerable to large ground shakings. In order to improve the seismic performance of both new and existing bridges, seismic isolation devices have been widely used since last few decades. Seismic isolation is an innovative seismic resistant design approach that decouples the bridge superstructure from the substructures, reducing the transmitted forces to the piers and abutments. The incorporation of the seismic isolator introduces flexibility at isolation level. As a result, the displacements of bridge decks increase. Since adjacent bridge decks/abutments might have different vibration frequencies, and ground motion input at different bridge supports are not exactly the same owing to seismic wave propagation, the adjacent bridge decks and abutments usually do not vibrate exactly in phase. This out-of-phase vibration results in relative displacement responses between bridge decks and between a deck and an abutment that leads to two main problems. Firstly, poundings between adjacent decks or between a deck and an abutment occur if the closing relative displacement exceeds the provided gap size at bridge expansion joints. Pounding of adjacent bridge structures could cause damages at expansion joints and could damage adjoining bearings and piers. It can also amplify relative displacements and contribute towards unseating of bridge spans (Otsuka et al. 1996). The bridge design codes, such as the Japanese Road Association (2004) specify that the gap size between bridge segments should be large enough to avoid poundings. However, the sizes of the expansion joints have to be limited to allow the traffic to flow smoothly. Therefore, it is often impossible to avoid pounding between adjacent bridge components with conventional expansion joints during large earthquake ground excitations. On the other hand, unseating of the bridge spans occur if the opening relative displacement is larger than the provided seat length. Unseating of a bridge span can lead to complete closure of the bridge. As bridges are key components in transportation networks and essential for providing emergency rescue and relief operation after a major earthquake, it is desirable for bridges to not only avoid collapse but also remain functional immediately after an earthquake. Therefore it is necessary to mitigate unseating and pounding damages of bridges induced by large relative displacement between adjacent bridge components.

In addition to pounding and unseating damages large relative displacement during an earthquake could also damage isolation bearings. For example, during the Tohoku earthquake, Japan in 2011, bearing rupture was observed in multiple bridges, such as the Tobu viaduct where the rupture was caused possibly due to the interaction of adjacent bridge components (Takahashi 2011). The failure of the bearing could result in large residual vertical gaps between the girders (Zhu et al. 2004). According to the design specification of highway bridges in Japan (JRA 2004), the shear strain in the isolation bearing shall be within 250%. Though several researches have been done in the past on preventing unseating damages in bridges, only limited researches have focused on the damages to the bearing during seismic events. Zhu et al. (2004) using a 3D model evaluated the serviceability of highway bridge with pounding countermeasures. The authors concluded that bearings to be the weakest link in the bridge and are likely to fail during strong earthquake resulting in permanent vertical gaps that could impede the traffic flow. Bi and Hao (2013) used a detailed 3D model of an isolated bridge and reported that bridge girder could dislocate from the bearing and the dislocated girder could pound against rubber bearing leading to further damages. Only limited studies have focused on the bearing protection devices (Ghosh et al. 2011; Wilde et al. 2000; Choi et al. 2005; Ozubulut and Hurlebaus 2011). It should be noted that these studies did not consider pounding between adjacent bridge components that could amplify/reduce the bridge displacement. The performance of bearings during earthquake excitations, especially when pounding between adjacent bridge decks occurs, is not well studied yet.

In order to mitigate the adverse effect of relative displacement in bridges different devices have been used. Among them, cable restrainers are the most widely used retrofitting method. However, cable restrainers are only effective to mitigate unseating damages caused by opening relative displacement but could not directly mitigate pounding impacts caused by closing relative displacement. Moreover, the commonly used cable restrainers relies primarily upon their stiffness to limit the opening relative displacement, which can induce a large tensile force which could result in either failure of restrainers or connecting element. The large tensile forces transferred to adjoining frame/deck/abutment may also alter the seismic responses of the bridge. To overcome the limitation of cable restrainers Feng et al. (2000) and Kim et al. (2000) investigated the use of energy dissipating restrainers to mitigate the damages at expansion joint. These studies reported that energy dissipating devices could be a practical solution to the seismic problem arising on bridges with expansion joints. Additionally, it was found that the supplemental damping could be significantly more effective than the stiffness on reducing the relative displacement at bridge expansion joints. Other researcher such as (Rungrassamee and Kawashima 2003; Guo et al. 2009) investigated the active and semi-active devices such as Magneto-Rheological (MR) dampers to improve the seismic responses of bridges.

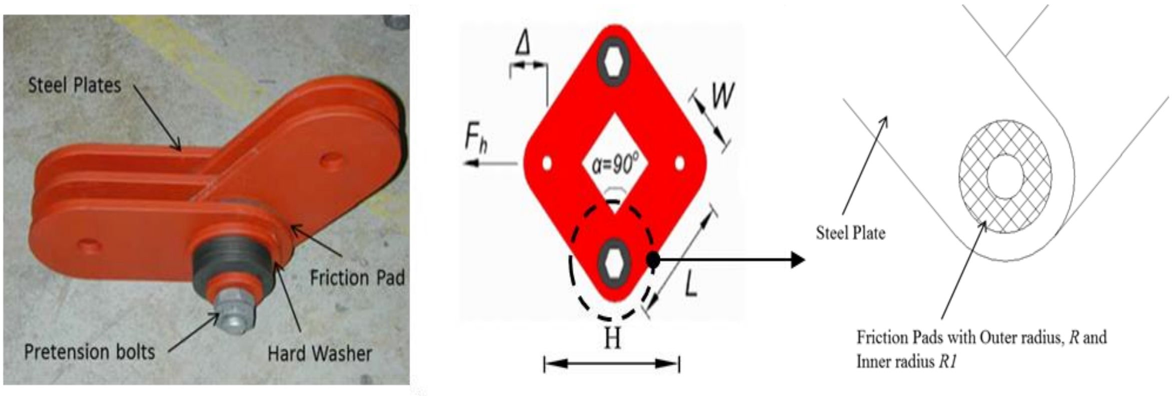
One of the important factors affecting the relative displacement responses between adjacent bridge components that have been commonly neglected is the spatial variation of seismic ground motions. Spatial variation of the ground motions along the length of an extended bridge is inevitable due to the different arrival times of seismic waves at different locations of a bridge and loss of coherency due to scattering of seismic waves and different soil conditions. Some previous studies, e.g., (Bi and Hao 2013; Zanardo et al. 2002; Chouw and Hao 2008; and Li et al. 2012) have demonstrated that structural response of bridges subjected to spatially varying ground motions can be drastically different from that under the usually adopted uniform ground motions. Despite the presented facts, most of the previous studies have either neglected the spatial variability of ground motions by assuming uniform ground motion or only partially considered it by including the wave passage effects (Jankowski et al. 2000) when studying the effectiveness of retrofit devices to mitigate relative displacement induced damages. To the best knowledge of authors, none of the previous studies, apart from the study of Shrestha et al. (2014, 2015) have modelled the ground motions spatial variability in detail in evaluating the effectiveness of pounding and unseating mitigation devices. Since ground motion spatial variation is inevitable, and it causes significantly different responses between adjacent bridge components, the study on the retrofit devices to mitigate the relative displacement induced damages without considering the spatial ground motion variations along the length of bridge may provide unrealistic results.

In this study, Rotational Friction Hinge Dampers (RFHD) devices are proposed to mitigate damages in bridge structures subjected to spatially varying ground motions. These devices have large hysteretic energy dissipation capability at a reasonable cost and are easy to install and maintain. The behavior of the device are nearly unaffected by amplitude, frequency or the number of the applied loading cycles (Mualla and Belev 2002). Recently, several friction devices have been tested experimentally and some of these have been implemented in buildings around the world (Mualla and Belev 2002; Nielsen et al. 2004). However, its efficacy in mitigating the relative displacement induced damages in bridge structures has not been explored yet. This study focuses on evaluating the effectiveness of the RFHD on mitigating relative displacement induced damages in simply supported bridges caused by spatially varying earthquake ground motions. This study does not focus on comparison of responses of isolated simply supported bridges to spatially varying ground motions with those subjected to uniform ground motion which could be found elsewhere (Zanardo et al. 2002). It focuses on mitigating the adverse responses of isolated multi-span simply supported bridges subjected to spatially varying ground motions, particularly pounding and unseating damages, as highlighted by (Zanardo et al. 2002). The analysis is conducted on a typical Nepalese simply supported bridge with four spans of 25 meters each. Extensive numerical analysis is conducted to identify the effectiveness of RFHD on mitigating the damages in bridge structures. Parametric analyses have been conducted to ascertain the optimum slip force of the RFHD. The investigation also compares the bridge structural responses with two configurations of RFHD.

## 2. ROTATIONAL FRICTION HINGE DAMPER

In recent years, friction dampers have found several applications in both steel and concrete buildings for seismic rehabilitations and up-gradation of the existing structures as well as applications in newly constructed structures (Mualla and Belev 2002). A key point in the use of the friction dampers in seismic protection of structures is that their response is not affected by frequency and duration of ground motions. However, their mechanical behavior is likely to induce residual displacement that may require some recovering operations after the earthquake event.

In this study, a type of friction damper, Rotational Friction Hinge Damper (RHFD), is used to mitigate the damages arising in bridge structures due to relative displacements of adjacent bridge components. RFHD consists of rigid steel plates connected in rotational hinge, and the plates are separated by several shims of friction pads as seen in Figure 1(a). The moment-rotation behavior in the hinge is elastic-frictional. The hinge connection is meant to increase the amount of relative rotation between the rigid plates, which in turn enhances the energy dissipation in the system. During the seismic events the distance between connection points and the angle between the damper plates in the hinge changes due to the induced seismic motion. Upon reaching the frictional resistance of the device in torsion, slip and relative rotation between the damper plates take place, thus dissipating a portion of the kinetic energy of the structure. The sticking and sliding modes of the RFHDs succeed each other until the end of motion (Nielsen et al. 2004).




(a) (b) (c)

**Figure 1.** (a) Damper details, (b) details of Rhombus shape damper with double hinges and (c) sectional detail of hinge including friction pads.

In order to investigate the effects of different configurations of dampers on the bridge responses two damper configurations as shown in Figure 1(a) and 1(b) are studied. The damper configurations are referred as V-type and R-type, respectively. The geometrical features of the friction dampers are provided in Figure 1 (b). Figure 1 (c) shows details of friction pads at rotational hinge. The friction plate has length L, width W and thickness t. The angle between the two adjacent plates is α. The slip force of the friction damper is calculated using the relations given by Chen and Hao (2013).

$$F_h = \frac{nM}{\sqrt{L^2 - (H/2)^2}} = \frac{nM}{L\cos(\alpha/2)} \tag{1}$$

where M is the rotational friction resistant moment at each hinge, n is the number of hinges in a device, L is the effective length of plate. The included angle is given by $\alpha$, and the height of the device is H.

The value of rotational friction resistant moment, *M*, depends on the friction coefficient, the preload and the frictional area. The friction force is given by

$$dF = \mu dN = \mu pds = \mu p \times 2\pi r dr \tag{2}$$

The resistant moment is given by

$$dM = rdF = 2\pi r^2 \mu p dr \tag{3}$$

$$M = \int_{R_1}^{R} dM = \int_{R_1}^{R} 2\pi r^2 \mu p dr = \frac{2}{3}\pi\mu p(R^3 - R_1{}^3) \tag{4}$$

where F is the rotational friction force at each joint; $\mu$ is the friction coefficient; p is the preloading provided by the bolts; $R_1$ is the inner radius of friction pad ; R is the outer radius of friction pad as shown in Figure 1(c).

The two ends of a V-type damper can be connected to pier and deck of the bridge, respectively, as shown in Figure 2 to mitigate relative displacement response. The connection detail of the R-type damper is presented in Figure 3. As shown, for a R-type damper the connection is between deck to deck at intermediate joints. In the figures only the connection scheme to control the longitudinal bridge motion are presented as this study considers only the longitudinal bridge responses that are responsible for pounding and unseating. The connection scheme could be easily extended to control both the longitudinal and the transverse bridge responses.

As presented, the damper has a very simple mechanism that makes it easy to be assembled and installed. The simplicity allows for installing devices with multiple units in order to meet the required frictional resistance. While applying, the dampers should be placed parallel to the longitudinal axis of the bridge to mitigate relative displacement responses in the longitudinal direction. In addition, a hydraulic lock-up device that allows slow movements such as thermal expansion but transmit the shocks from high frequency movement such as earthquake could be placed along with the device.

Previous investigation had showed that the behavior of friction damper is essentially bilinear (Mualla and Belev 2002; Mualla 2000). Due to this behavior it is quite common to represent a friction damper using rigid plastic link (Mualla and Belev 2002; Vafai et al. 2001) or elastic perfectly-plastic link in numerical modelling. Bhaskararao and Jangid (2006) studied the response of MDOF structures connected using friction dampers modelled using fictitious springs. The fictitious spring was assumed to having large stiffness during the non-slip mode and zero stiffness during the slip mode. The same concept is utilized here to model the RFHD with a high initial stiffness ($k_d$) during non-slip mode as shown in Figure 4. The slip takes place whenever the force in the dampers exceeds the slip force $F_h$, which is the limiting force in that friction damper.

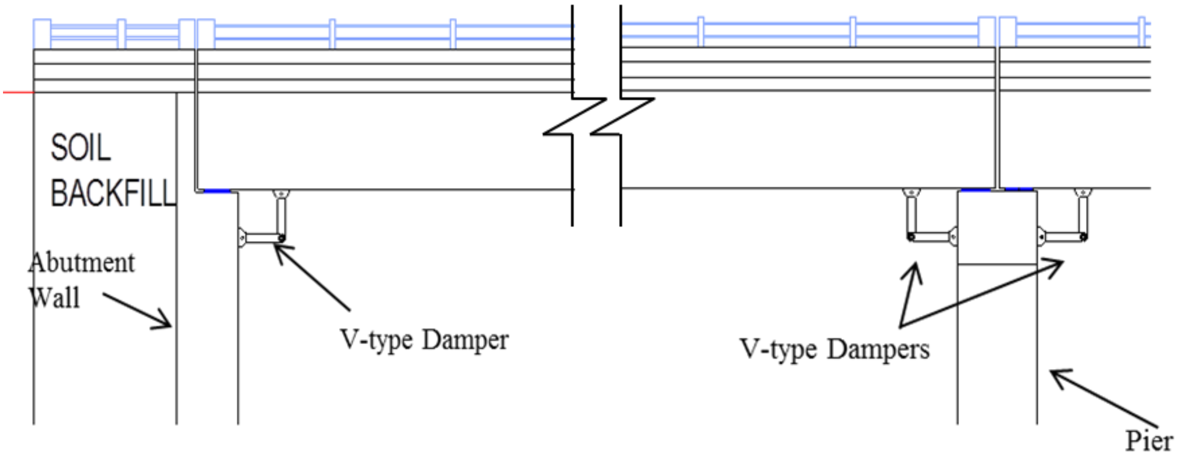



**Figure 2.** Connection scheme for V-type dampers

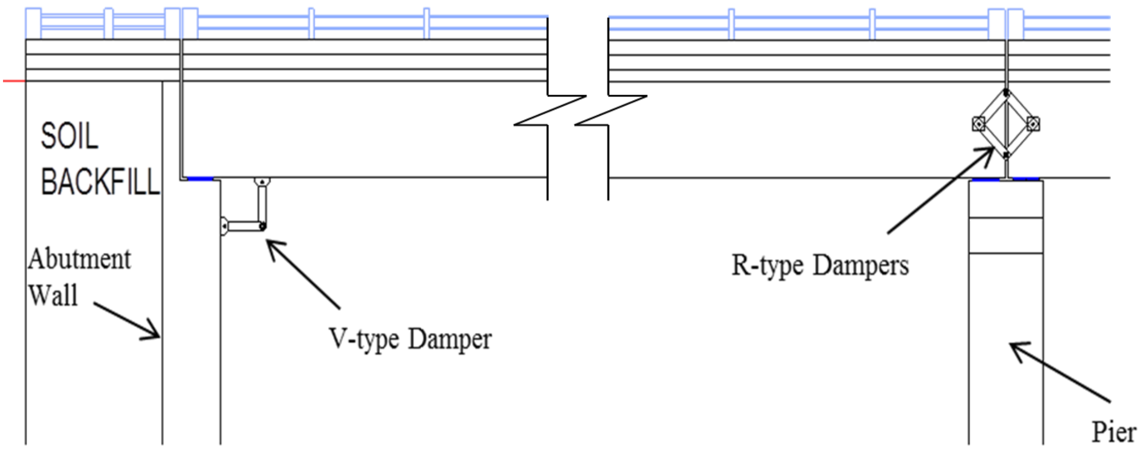



**Figure 3.** Connection scheme for R-type dampers

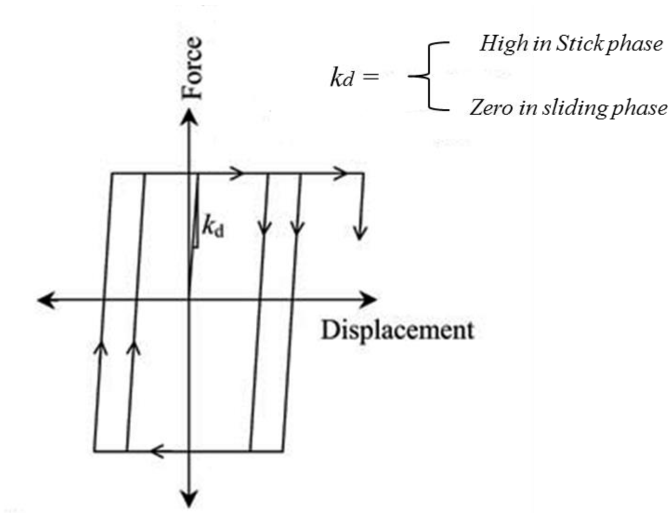



**Figure 4.** Force-displacement relationship for RFHD

## 3. BRIDGE MODEL

### 3.1 Bridge Description

Figure 5 shows the details of the simply supported bridge considered in this study. The bridge has 4 spans of 25 m each and the total length is 100 m. These are typical simply supported bridges commonly found in Nepal. The bridge is supported on 3 piers and 2 abutments. The piers are of circular geometry with 1.6 m diameter. The total height of the bridge piers from the top of foundation is 6 m. The bridge deck is slab on girder type construction with 3 girders of 2 m depth. The total weight of each 25 m deck is 2.13 MN. The details of deck and pier to deck connections are presented in Figure 6. The deck is supported on elastomeric bearings of area 0.4 m by 0.3 m and thickness of 0.05 m. The piers and abutments are provided with shear keys that inhibit the lateral movement of bridge decks. The abutment is a seating type with back wall of 2 m in height and 7.2 m in width. The length of the seat at the abutment is 0.94 m. All the bridge piers and both the abutment rest on a well foundation of diameter 6 m and depth 13 m.

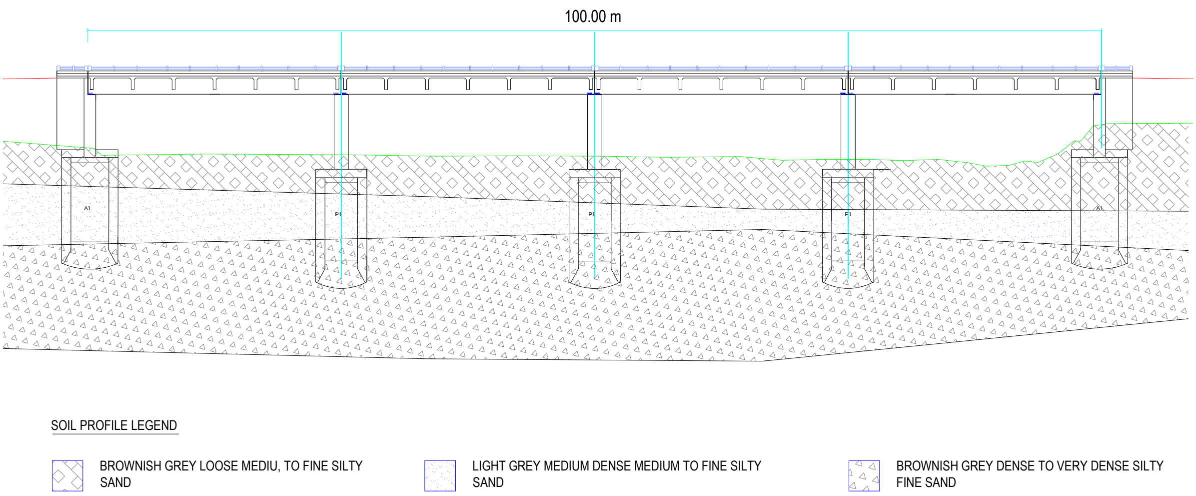



**Figure 5.** Sectional view of the bridge

### 3.2 Numerical modelling

In this study, 2-D finite element models of the bridge is developed. The geometrical property of the bridge is calculated based on the details of the bridge designs illustrated in Figure 6. The superstructures of an isolated bridge are usually designed to remain elastic under seismic events. Therefore an elastic beam-column element with the calculated properties is used to model the bridge deck. The piers are modelled using nonlinear beam-column element. Fiber element modelling, also known as discretized-section model for non-linear analyses, is used in this study to represent non-linear behavior of the reinforced concrete bridge piers. Reinforced concrete sections are constructed from three materials, namely unconfined concrete, confined concrete and reinforcing steel. The unconfined and confined concrete behavior is modelled using the nonlinear concrete model that follows the constitutive relationship proposed by Mander et al. (1988) and the cyclic rules proposed by Martinez-Rueda and Elnashai (1997). The confinement effects provided by the lateral transverse reinforcement are incorporated through the rules proposed in (Mander et al. 1988), whereby constant confining pressure is assumed throughout the entire stress-strain range. To represent the behavior of the steel re-bars, Menegotto-Pinto steel model (Menegotto and Pinto 1973) is used. The yield strength of the rebar is 500 MPa, and the elastic modulus, $E_s$ is 200GPa. Reinforcement details of the piers are shown in Figure 6 (b).

The foundation of the bridge is assumed to be fixed at the top of well foundation. To simplify the problems, interaction between soil and the foundation of bridge structure is neglected in the present study. It is common in engineering practice to use a simplified bilinear model with kinematic hardening rules, as shown in Figure 7(b), to represent the behavior of elastomeric bearings (Naeim and Kelly 1999). The bilinear model can be completely described by the elastic stiffness, $K_1$, characteristic strength $Q$ and post-yielding stiffness $K_2$. The characteristics strength $Q$ of bearing is taken as 10% of the weight carried by bearings. This value has been widely accepted among the bearing designers (Ali and Adbel Ghaffer 1995; Adbel Raheem 2009). The elastic stiffness to post-yielding stiffness ratio, $K_2/K_1$ is taken as 0.10. The elastic stiffness of elastomeric bearings is taken as 13.25 MN/m, the post-yielding stiffness is 1.32 MN/m and the characteristic strength is 98.60 KN.

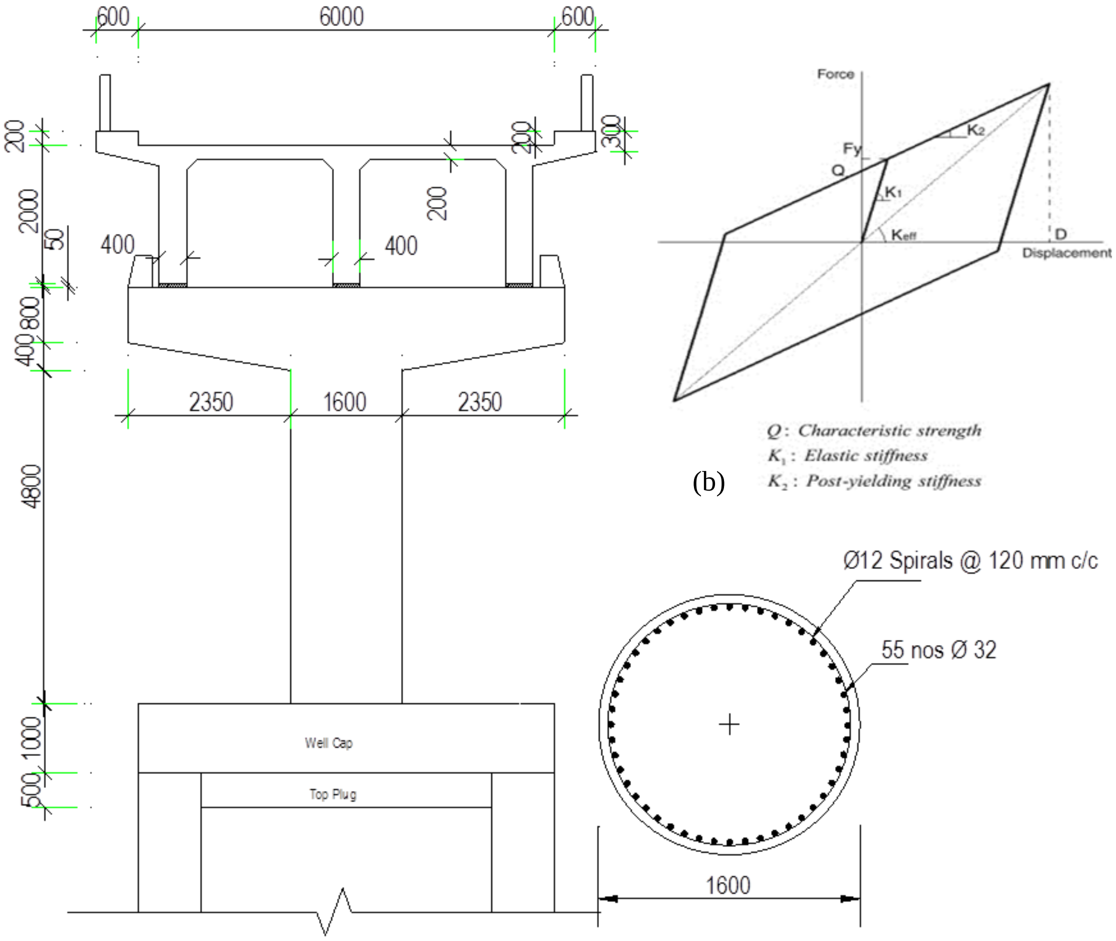
Pounding between two decks or deck and abutment is modelled using a linear impact spring element with gap of 25mm. The stiffness of impact element $K_i$, and the impact force $F_i$ at the impact spring element are expressed as

$$K_i = \gamma \frac{AE}{L} \tag{5}$$

$$F_i = K_i \cdot \Delta u \tag{6}$$

where $\Delta u$ is the relative closing displacement between the adjoining bridge superstructures beyond the provided gap width. In Equation (5), $A$ is the sectional area of the deck, $\gamma$ is the ratio of impact spring stiffness to the stiffness of the superstructure and E is the modulus of elasticity of the deck material. In this study $\gamma$ is taken as 2 based on the previous studies on similar bridges (Adbel Raheem 2009; Ruangrassamee and Kawashima 2003). The stiffness of the impact spring is calculated to be 7884 MN/m. Abutment of the bridge is modelled using linear spring. The stiffness of abutment spring, $K_{abut}$ used in the analysis is 174 MN/m. The abutment springs get activated only in passive direction of the abutment.

The mechanical model of as-built bridge and bridge installed with V-type and R-type RFHD are illustrated in Figure 7(a), 7(b) and 7(c), respectively. In the figure, RLink*i*L and RLink*i*R refer to the rigid link connecting the $i_{th}$ pier with the deck on the left and right side of the $i_{th}$ pier, respectively; AbutSpr1 is the abutment spring at abutment 1, Br2L refers to the left bearing at Pier 2, Vtype2L refers to V-type RFHD at left side of Pier 2. Rtype 2 indicates R-type RFHD placed above pier 2.




(a) (c)

**Figure 6.** (a) Bridge geometrical details, (b) hysteretic behavior of elastomeric bearing and (c) pier reinforcement details

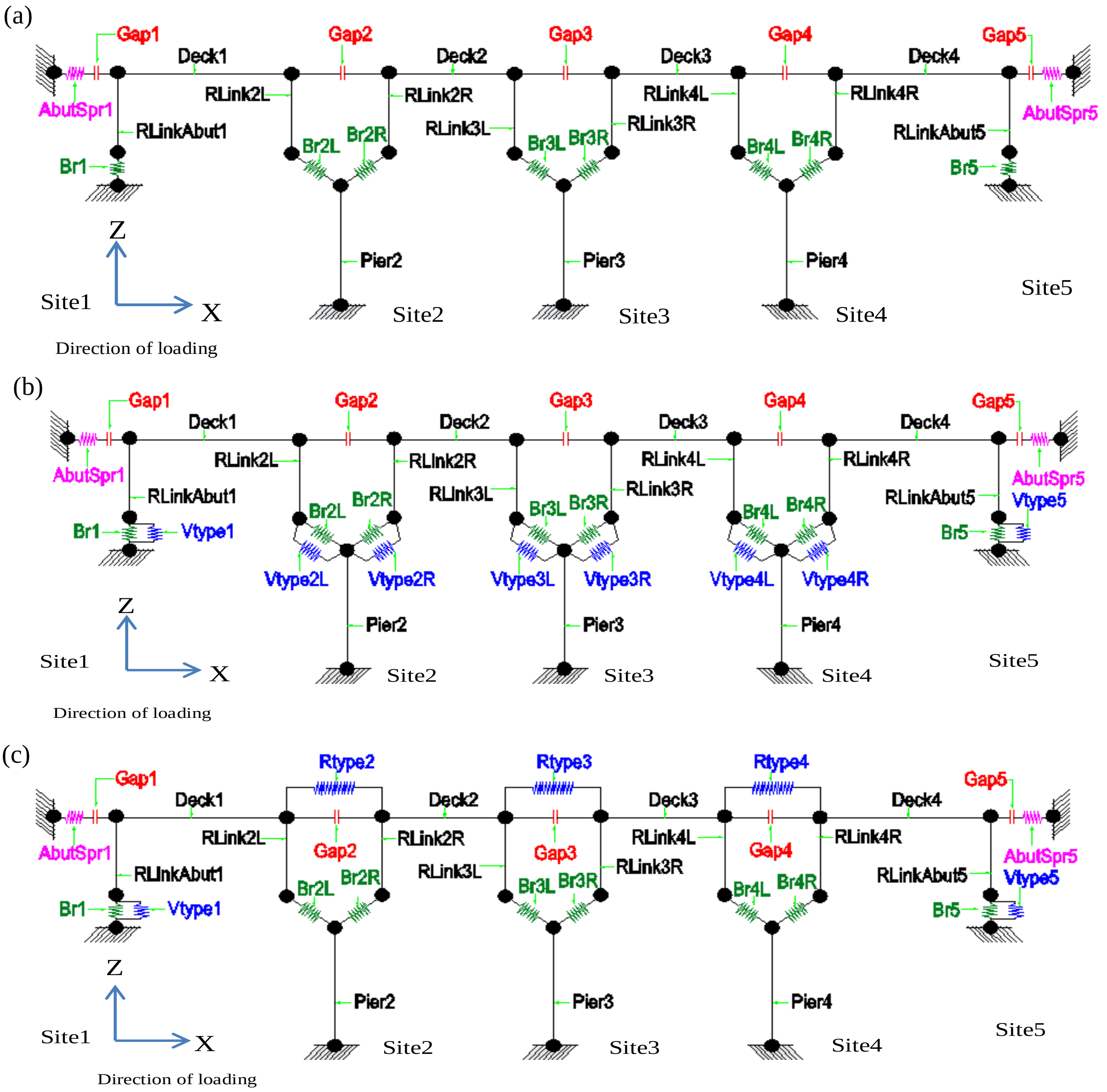



**Figure7.** Mechanical model of (a) as-built bridge, (b) bridge with V-type dampers and (c) bridge with R-type dampers at intermediate joints

## 4. GROUND MOTIONS

The method proposed by Bi and Hao (2012) is used to simulate spatially varying ground motion time histories. The ground motions are simulated to be compatible with the design response spectrum defined in Indian code IS1893 (2002) for Type III (soft soil) condition normalized to PGA 0.65g. The PGA value adopted in this study was determined in recent Probabilistic Seismic Hazards Analyses (PSHA) (Parajuli 2009; Ram and Wang 2013; Mahajan et al. 2010) for regions in Nepal and adjoining areas for rare earthquake events that should be used for designing lifeline bridge structures.

The spatial variation between ground motions recorded at two locations j and k on ground surface is modelled by a theoretical coherency loss function (Sobczky 1991)

$$\gamma_{jk}(i\omega) = |\gamma_{jk}(i\omega)| \exp(-i\omega d_{jk} \cos\alpha / v_{app}) = \exp(-\beta\omega d_{jk}^2 / v_{app}) \cdot \exp(-i\omega d_{jk} \cos\alpha / v_{app}) \tag{7}$$

where $\beta$ is a constant reflecting the level of coherency loss, $d_{jk}$ is the distance between the two locations j and k in the wave propagation direction, f is the frequency in Hz, $v_{app}$ is the apparent wave velocity, and $\alpha$ is the seismic wave incident angle. In this study, $\beta$ = 0.001, $v_{app}$ and $\alpha$ are assumed to be 500 m/s and 45°, respectively. The adopted value of $\beta$ represents intermediate coherency losses of the spatial ground motions at supports of the bridge. To obtain a relatively unbiased response accounting for the random phase angles of ground motions, 5 sets of spatial ground motion time histories (referred as GM1 to GM5) are simulated independently. Sampling frequency is set to 100 Hz, and duration of the ground motion is 20.47 seconds in simulation. Figure 8 compares the response spectra of the simulated spatial ground motions with the target response spectrum. As shown, the response spectra of the simulated spatial ground motions at the five sites are all compatible with the design response spectrum.

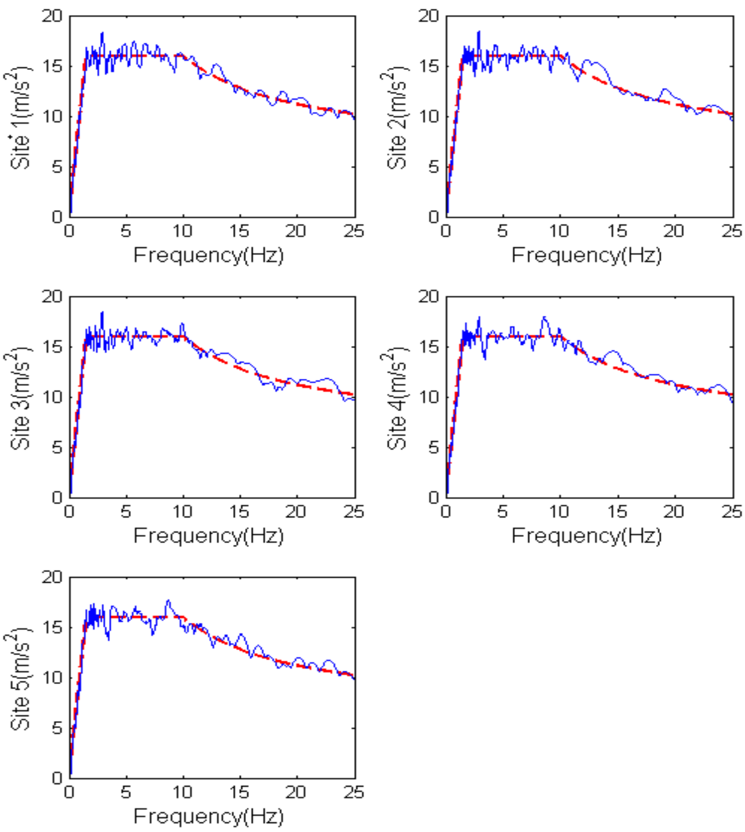



**Figure 8.** Comparison of response spectra of simulated ground motions with the design response spectra

The comparison of the empirical coherency loss function defined by Eq. (7) between Site 1 and the other sites is presented in Figure 9. A good match can be observed except for $|\gamma_{15}|$ in the higher frequency range. This is expected since Site 5 is the furthest from site 1 and the spatial ground motions at these two sites are least correlated. The cross correlation between spatial motions or the coherency loss decreases with frequency, but the numerically calculated coherency loss between any two spatial ground motion time histories is more than 0.3. This is because the numerically calculated coherency loss has a threshold value of about 0.3-0.4, the value corresponding to the numerically calculated coherency loss between two white noise series as revealed in previous studies, e.g. Hao et al. (1989).

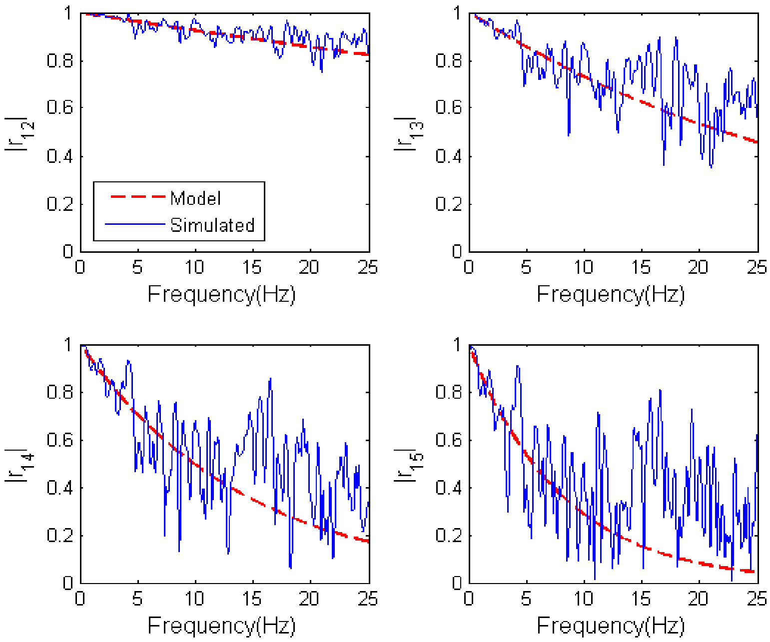



**Figure 9.** Ideal and simulated coherency losses

## 5. RESULTS AND DISCUSSIONS

### 5.1 Effects of Pounding

Prior to assessing the effectiveness of the friction hinge dampers, the effects of seismic pounding on the response of the structure must be understood. It is well-known that seismic pounding results in damaging impact between the adjacent bridge components, however, its effect on the relative opening displacements at joints of simply supported bridges subjected to non-uniform ground motions has not been well documented. As relative opening displacement may result in unseating damage, it is important to understand the influence of pounding on the relative opening displacement response. To study this, the as-built bridge model with expansion gap of 25 mm and assumed gap large enough to avoid contact between the adjacent bridge components are analyzed. As shown in Figure 10, without pounding the deck could move beyond the gap size (negative or closing relative displacement more than 25 mm) and the response is more stable for the duration of the earthquake. When pounding occurs the closing relative movement is limited approximately to 25mm at each joint as the gap closes. As shown in the figure, peak joint opening displacements at Joint 1 and Joint 2 due to pounding experience an increase of 33% and 250%, respectively. This indicates that the relative joint separations could be amplified by the pounding of adjacent segments. As a consequence, the unseating displacements (i.e. opening relative displacement between the bridge deck and supporting pier) of the bridge deck, as presented in Figure 11, may increase, which may lead to unseating failure of the bridge deck if the provided seat width is smaller than the unseating displacement.

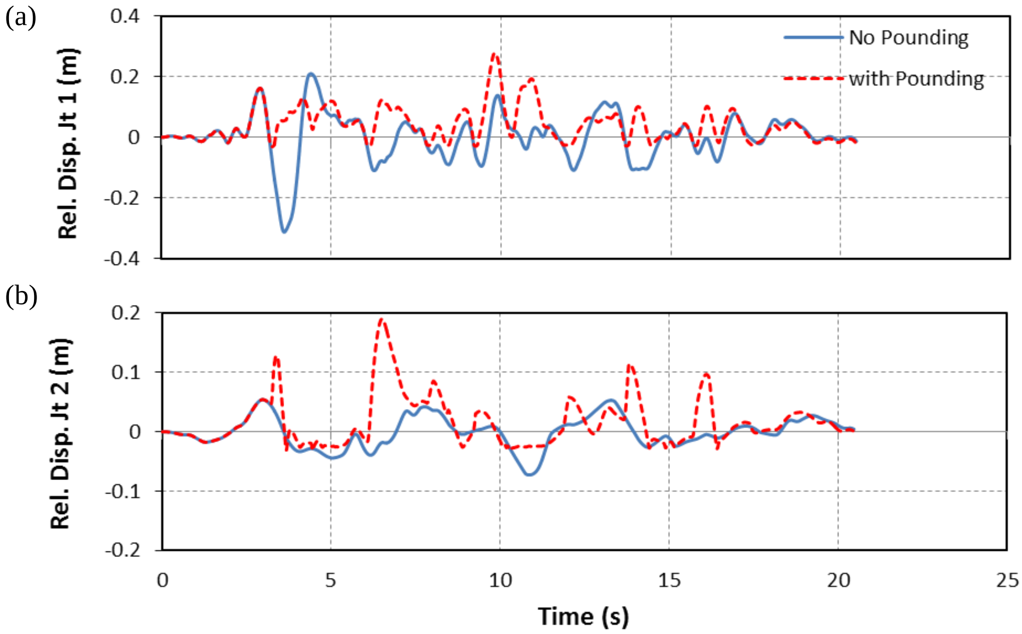



**Figure 10.** Relative displacements between adjacent decks at (a) joint 1 and (b) joint 2 with and without pounding

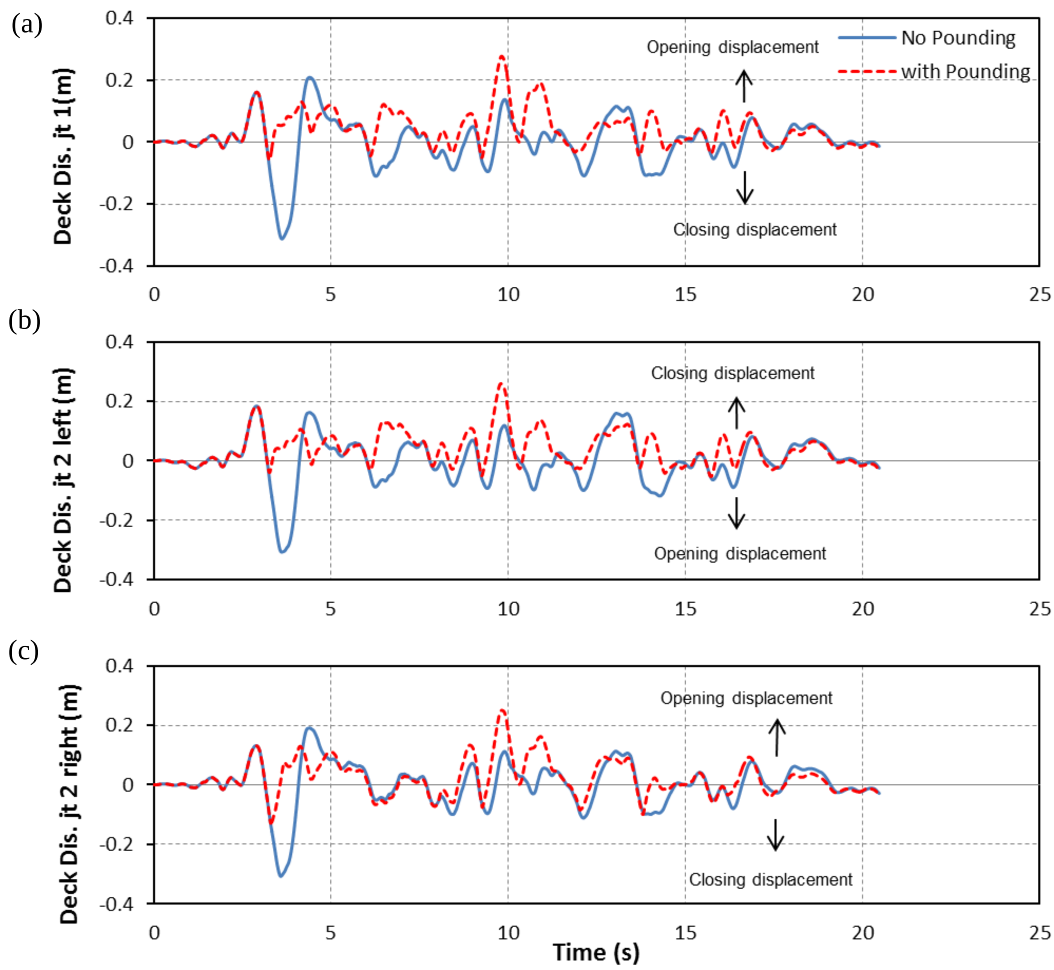



**Figure 11.** Relative displacements between bridge deck and supporting pier at (a) joint 1 , (b) left side of joint 2 and (c) right side of joint 2 with and without pounding

### 5.2 Effectiveness of RFHD

To evaluate the effectiveness of the RFHD in bridge structures subjected to spatially varying ground motions, responses of the as-built bridge model and the bridge model with V-type damper are analyzed and compared. In this section, without losing generality only the case with the total damper slip force of 186kN, i.e. with two dampers with slip force 93 KN each placed at two outer girders of the deck as shown in Figure 12, is presented. The performance of the bridges is compared in terms of the peak and standard deviation of pounding forces, peak and standard deviation of relative displacement, residual displacement, bearing deformation and pier drift.

Figure 13 shows the peak pounding forces at five joints of the bridge for the two bridge models, i.e. as-built and with V-type dampers as shown in Figure 2. The middle point represents the mean peak pounding forces while vertical line represents the mean plus or minus one standard deviation of the peak pounding force at the bridge joints obtained with the 5 sets of independently simulated spatially varying ground motions. Thus the tip of the line represents the 84$^{th}$ percentile value of peak pounding force while bottom end of the line represents the 16$^{th}$ percentile value of the peak pounding force. As shown, the V-type dampers are effective in mitigating peak pounding forces at all joints of the bridge. Figure 14 compares the peak relative displacement at five joints of the bridges. The V-type dampers are also effective in reducing the peak relative opening of the joints. As shown the dampers significantly reduce the relative displacement at all joints except at joint 4, which has the least relative displacement without the dampers. This is because the dampers are effective only when the relative displacement is relatively large as damping capacity depends upon the opening of the joints and have only limited effect if the relative displacement is small as in the case for joint 4.

A factor that could limit the application of friction damper is its mechanical behavior which is likely to induce residual displacement in the structure that could limit the serviceability of the bridge after an earthquake. In order to evaluate the residual deformation that dampers can induce at the bridge joints, residual deformations at all joints are measured and compared with the corresponding residual deformations of the as-built bridge model. As shown in Figure 15, residual deformations at joints are not significantly altered by use of the friction dampers. The residual deformation could widen the gap or completely close the gap, however, the calculated residual deformations are within a limited range (less than 3cm) for the considered ground motions, thus would not impede the traffic flow.

Damper constraints the movement of bridge deck and this limits the deformation on the bearings. Bearing deformations without dampers could be large and could result in the failure of the bearings, potentially generating vertical gaps between the two adjacent decks or deck and approach slab. This study verifies the failure of rubber bearing by observing its peak deformation. Though the bridge codes (Japan Road Association 2004) suggest 250% shear strain as the ultimate shear strain limits, the modern isolation bearings can sustain shear strain up to 400% before failure. In this study without losing generality, a failure criterion of shear strain 300% for rubber bearing is adopted as in a previous study (Zhu et al. 2004). Figure 16 shows the peak deformation of the bearing to five sets of simulated ground motions for two bridge models. As presented, the bearing deformations in as-built bridge model are large and most of the bridge bearing will be damaged due to the earthquake ground motions. Installing the V-type dampers significantly reduces the deformation demand of the bearings and limits the bearing deformations within the permissible limit.

Figure 17 compares the peak drift of the three piers of the two bridge models. As shown, applying the V-type dampers results in an increase in the drift of the bridge piers due to the transfer of forces from superstructure to the pier. However, this does not significantly affect the bridge pier responses as indicated by only the slight increase in the bridge drift because the slip force of the damper is relatively lower and damper dissipates some of the kinetic energy. Application of V-type dampers leads to slightly higher forces on the bridge piers, however, this would not significantly reduce the effectiveness of bridge isolation and only slight increase in peak displacement demand would be expected to bridge piers. Despite this undesirable influence on pier responses, the advantages of using friction dampers to mitigate relative displacement responses of bridge superstructures are obvious.

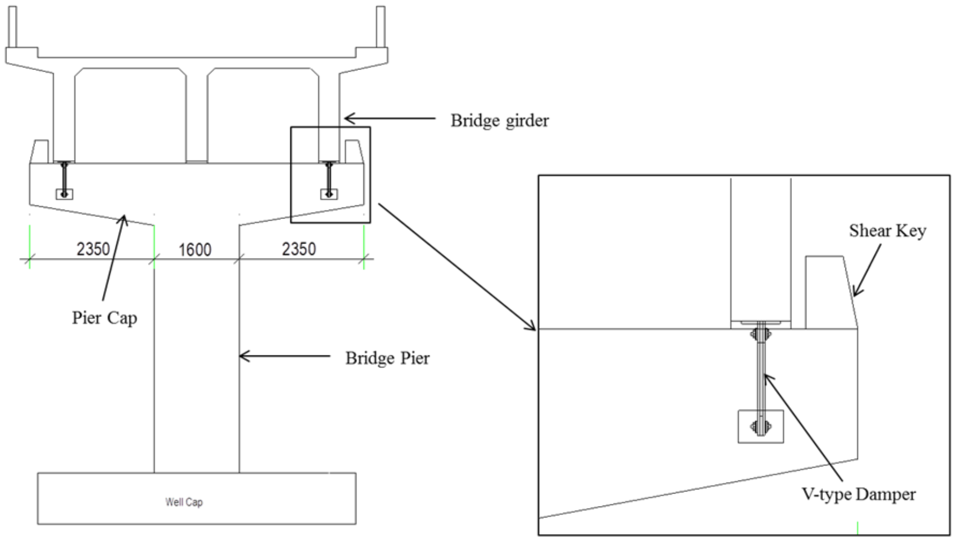
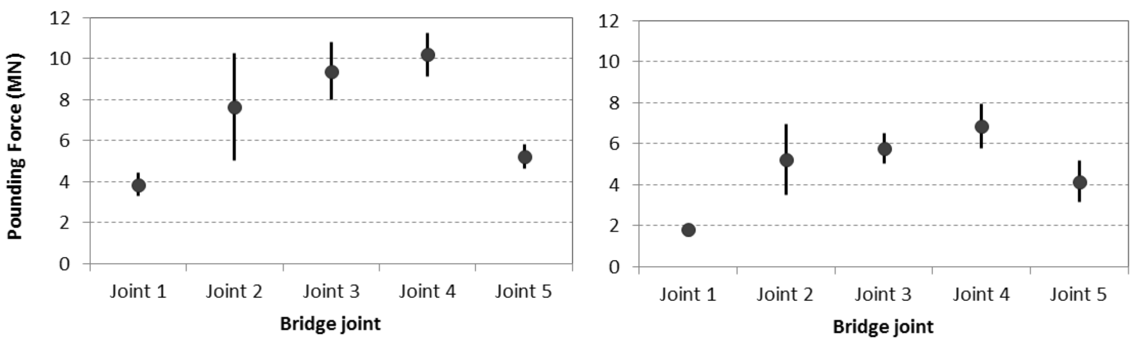
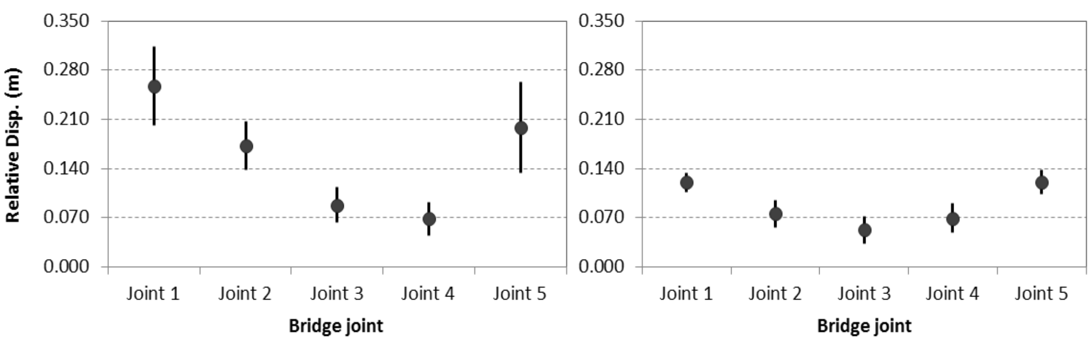



**Figure 12.** Symmetrical placement of dampers at outer bridge girders




(a) (b)

**Figure 13.** Pounding forces at five joints (a) as-built bridge; (b) bridge with V-type RFHD




(a) (b)

**Figure 14.** Relative displacement at five joints (a) as-built bridge; (b) bridge with V-type RFHD

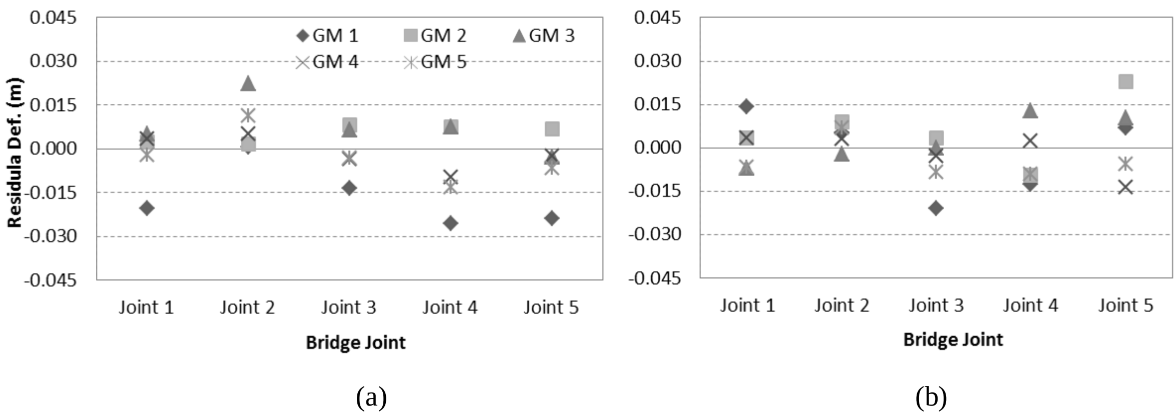



**Figure 15.** Residual displacement at five joints (a) as-built bridge; (b) bridge with V-type RFHD

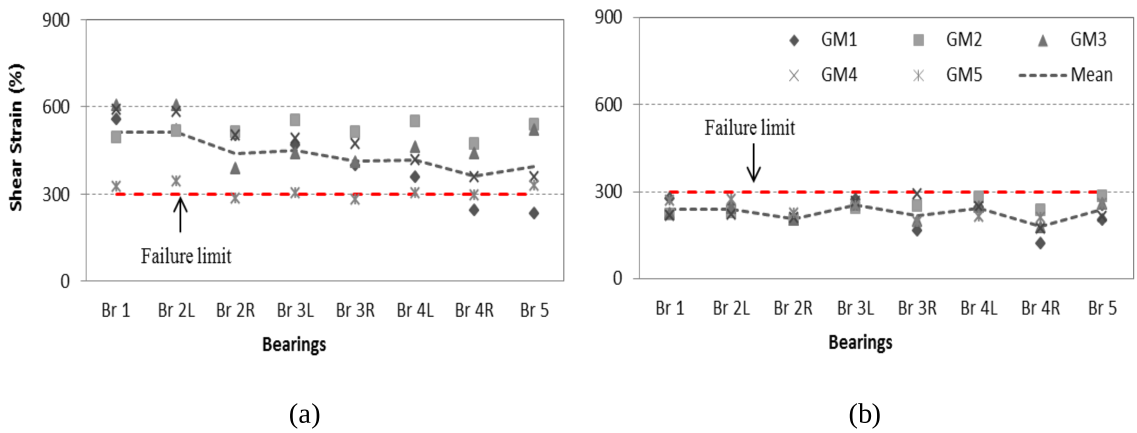



**Figure 16.** Peak deformation of bearings (a) as-built bridge; (b) bridge with V-type RFHD

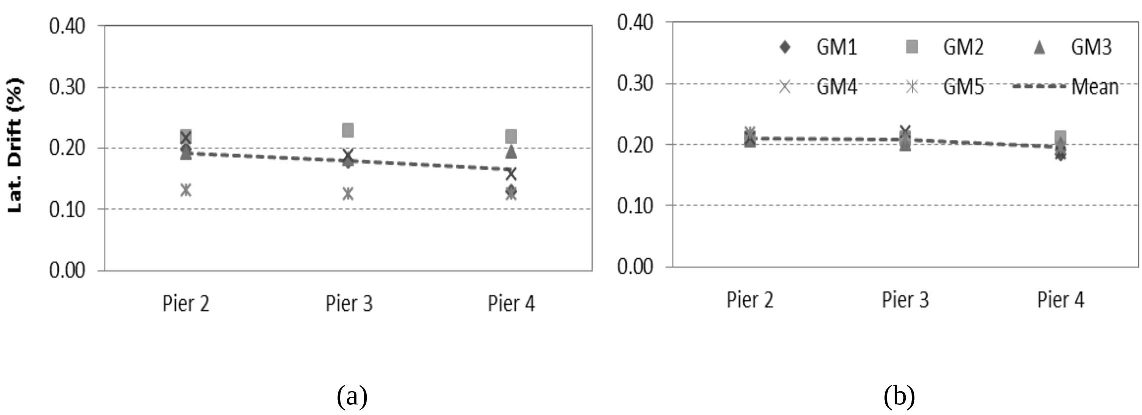



**Figure 17.** Comparisons of peak pier drifts (a) as-built bridge; (b) bridge with V-type RFHD

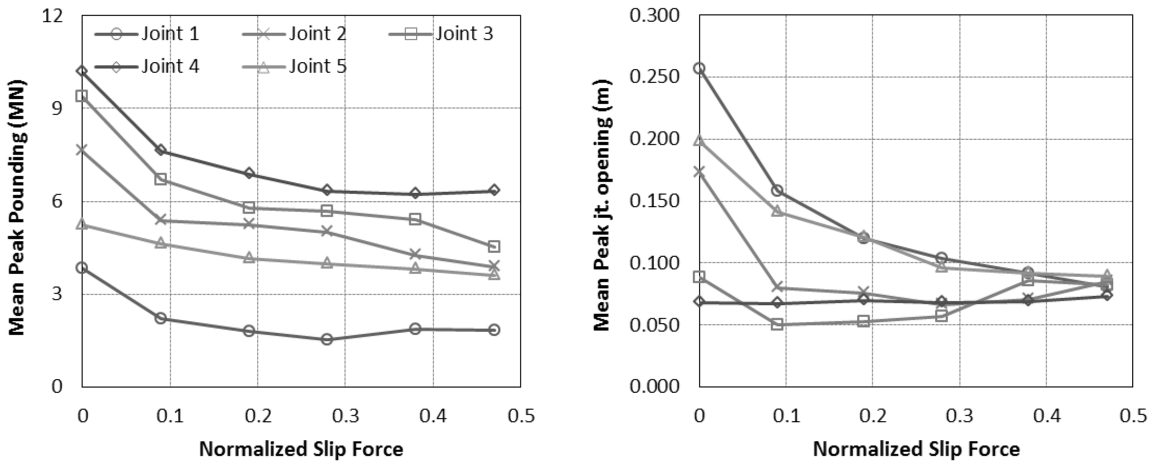
### 5.3 Optimum damper slip force

In order to find out the optimum slip force of the dampers to mitigate relative displacement responses without significantly increasing the pier responses, analyses are carried out with varying slip forces of the dampers. The slip forces of the dampers can be practically controlled by increasing or decreasing bolt pretension and/or by increasing or decreasing the number of friction plates. In this study, five damper slip forces, i.e. 93, 186, 280, 373 and 466 kN are considered to identify the effects of the damper slip force on the bridge response. This represents the normalized damper slip forces, defined as the ratio of slip force over weight of the bridge deck on bearing supports, of 0.09, 0.19, 0.28, 0.38 and 0.47, respectively. In order to investigate the optimum slip force of the dampers normalized damper slip forces are used to compare the bridge responses.

Figure 18 compares the mean peak pounding forces and mean peak joint opening at five joints of the bridge for 5 sets of ground motions. As shown, the pounding forces and relative joint opening are significantly reduced due to the application of RFHD. In general, increasing the RFHD slip forces result in reductions of peak pounding forces and joint opening. However, the rate of pounding force and joint opening reduction decreases with the higher slip force. When the normalized slip force is larger than 0.28, further increasing the slip force has insignificant effect on reduction of pounding force and joint opening displacement. This is because, as will be discussed subsequently, the energy dissipated by the dampers reduces with the higher slip forces. The reduction in the energy dissipation reduces the effectiveness of dampers to mitigate pounding forces between adjacent bridge components.

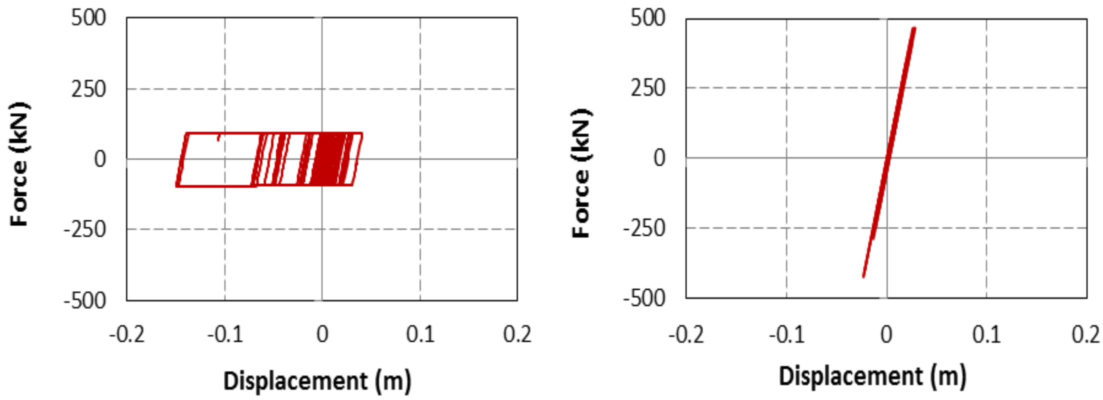
The energy dissipated by the dampers is affected by the damper slip force. Figure 19 presents comparison of hysteretic responses of a damper, Vtype4R with normalized damper slip force of 0.09 and 0.47 subjected to GM2. It is observed that the increase in the slip forces could result in a reduction of damper deformation and in some cases the device may form an incomplete hysteretic loop, suggesting a reduction in energy dissipation as well as presence of some residual displacements. Comparison of bearing shear deformation and pier drift demand subjected to GM2 is presented in Figure 20 (a). As shown, the bearing deformations of the bridge model without dampers are large and exceed the ultimate limit state. Placing the dampers with the normalized damper slip forces of 0.09 reduces the bearing deformations; however the deformations are still large enough to result in bearing failure. Installing the dampers with the normalized damper slip forces of 0.19 or above reduces the shear deformations below the ultimate strain limit of 300%. The higher is the normalized slip forces of the dampers, the more is the reduction of bearing shear strain. However, when the normalized damper slip force is larger than 0.38, further increase in slip force has insignificant effect on bearing deformation. Figure 20 (b) shows that as the normalized slip force is larger than 0.19, further increase in the slip force results in an increase in the peak drift of the bridge piers. This is because large damper slip force reduces the effectiveness of bearing isolation of the bridge deck, therefore results in more seismic forces being transferred from bridge decks to the piers.

The above results indicate that increasing the damper slip force is generally beneficial to mitigating relative displacement responses, however, would also result in reduction of energy dissipation and larger pier responses. Therefore a balance needs be found for a practical application of dampers for better protection of not only the bridge super structures (decks) and connection members (bearings), but also the bridge piers. The results presented also suggest that damper effectiveness is not significantly affected by slight variations in the slip force of dampers. Hence, small variations in optimum slip force over the life of the bridge do not warrant any adjustment or replacement of friction dampers.



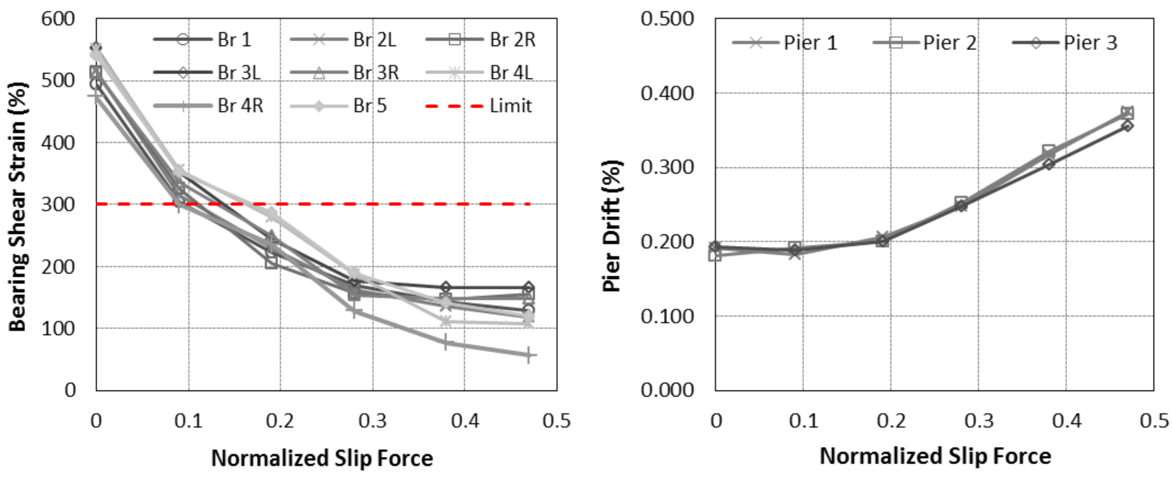

(a) (b)

**Figure 18.** Comparison of (a) mean peak pounding forces; (b) mean peak joint opening




(a) (b)

**Figure 19.** Comparisons of force-displacement curves of Vtype4R damper with normalized slip force (a) 0.09; (b) 0.47, subjected to GM2




(a) (b)

**Figure 20.** Comparison of (a) peak bearing deformations; (b) pier drift subjected to GM2

### 5.4 Effects of damper configuration

To investigate the effectiveness of damper types, responses of bridge model with applications of V-type and R-type dampers at the different joints are calculated and compared. The results corresponding to 5 normalized damper slip forces as described above subjected to the 5 sets of spatially varying ground motions are compared and discussed. Figure 21 (a) and (b) present the comparisons of mean peak pounding forces at joint 3 and mean peak joint opening at joint 1 and 3 for two damper types with varying normalized damper slip forces, respectively. As shown, V-type dampers are more effective in mitigating pounding impact and relative displacement at the joint as it is connected to the bridge piers. R-type dampers reduce peak opening joint displacement at joint 3 more effectively than V-type dampers. However, it should be noted that the peak joint opening at joint 3 is much smaller compared to that at joint 1. Figure 21 (c) and (d) present the comparison of shear strain in bearings and peak drift of three piers with two damper configurations to a set of spatially varying ground motion. V-type dampers are more effective than R-type dampers on mitigating bearing deformations; however, it also leads to transfer of large forces to bridge piers resulting in larger deformations. R-type dampers reduces the pier drift demand as connection is deck to deck and it dissipates some input energy by hysteretic response at superstructure of the bridge.

From the above results it can be concluded that, in general, V-type dampers are more effective in reducing pounding and joint opening at the bridge joints. It is to be noticed, however, that in the current numerical simulations, in the case of V-type damper two friction hinge devices are used at the both sides of each joint connecting the deck to the pier, in the case of R-type damper only one friction hinge device with equal slip force as a single V-type unit is used to connect the two adjacent decks. This assumption implies the force required to make V-type damper connected joint move is two times of that required to make R-type damper to move.

The appropriate damper configuration to control the bridge responses thus depends upon responses of the most vulnerable components of the bridge.  In the studied bridge the bearings were weaker components thus V-type dampers that connects the deck with the piers are the appropriate retrofit device as this will lead to reduction of displacement and shearing strains of the bearings. However, more forces are transmitted to the bridge substructures. In case where protection of bridge superstructures from pounding and unseating damages are desired without transfer of additional forces to bridge piers, R-type dampers are the appropriate selection.

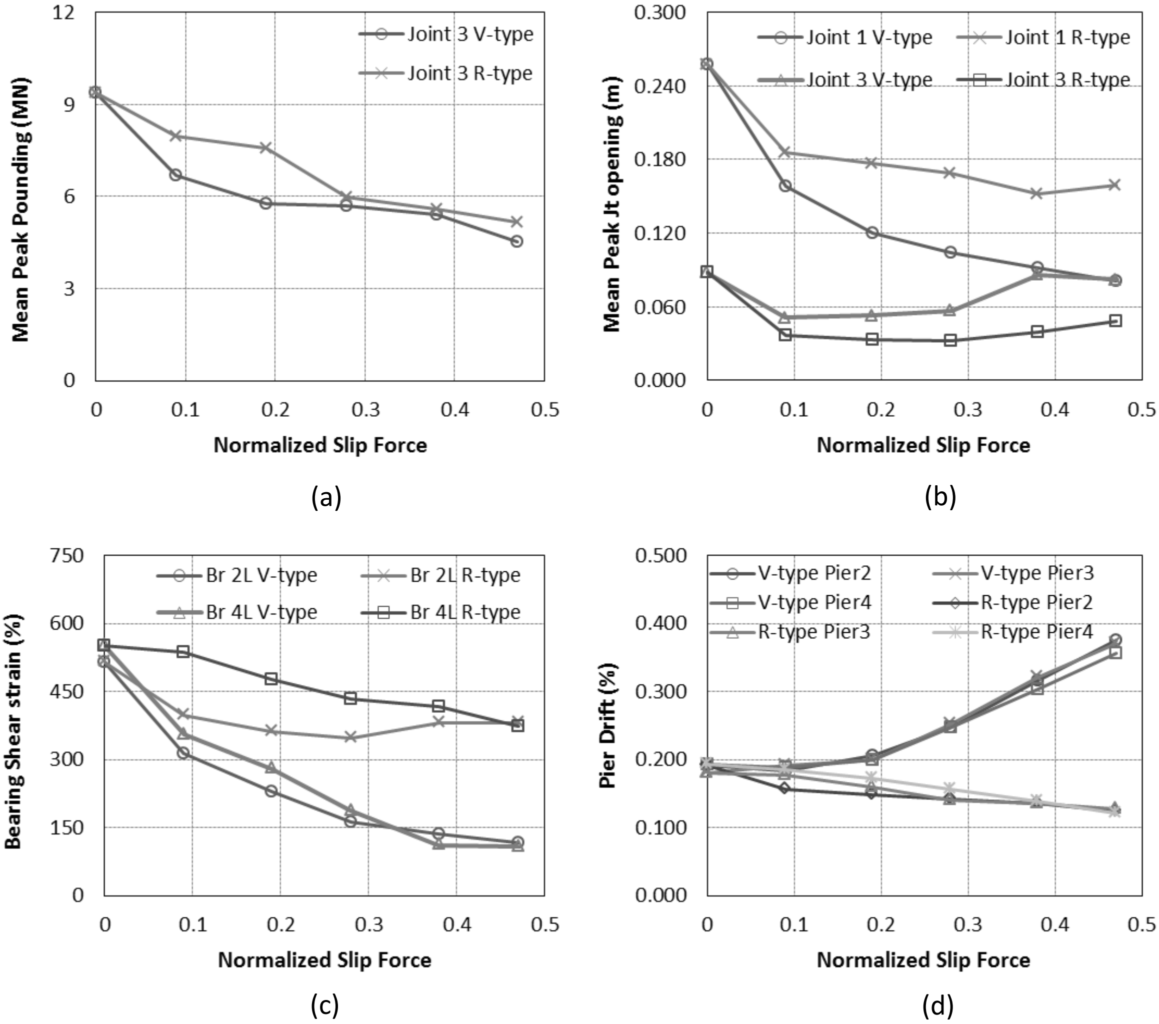



**Figure 21.** Comparison of (a) mean peak pounding force ; (b) mean peak joint opening; (c) bearing shear strain; (d) pier drift demand for two damper configurations

## 6. CONCLUSION

The paper presents investigations on the effectiveness of using RFHD to control responses of simply supported bridges subjected to non-uniform ground motions. Five sets of spatially varying ground motions compatible with the design spectrum and empirical coherency loss function along the supports of the bridge are used to simulate realistic relative displacement responses of the bridge. The bridge model is based on a typical Nepalese simply supported bridge. The study found that pounding between the adjacent bridge components could increase relative joint opening, thus enhancing the risk of unseating failures. The results presented in this paper suggest RFHD could be an ideal retrofit device to mitigate relative displacement induced damages, such as pounding and unseating damages, abutment back wall deformations and bearing failure. These devices are capable of reducing the response at bridge joints by dissipating some of the input energies.

For better mitigation of seismic responses of bridge, damper with optimum slip force should be provided. Increasing the slip force of the dampers beyond optimum slip force, in general, leads to slight reductions in bridge responses. However, it also increases the pier drift as more forces are transferred to the piers of the bridge. The result presented also shows the effectiveness of dampers to mitigate the relative displacement induced damages, such as pounding and unseating, are not significantly affected by small changes in optimum slip force of the dampers. Therefore, small variations on optimum slip forces of dampers during the life of the bridge do not warrant any adjustment or replacement of friction dampers.

V-type dampers are found to be more effective in mitigating pounding and relative opening displacement at bridge joints. The dampers are also significantly more effective in reducing the deformation demand of the bearings compared to the R-type dampers. However, V-type dampers could increase the drift demand of the piers because they transfer forces from the superstructure to the bridge piers. R-type dampers are relatively less effective on mitigating poundings, relative joint displacements and bearing deformations but their effectiveness on reducing the piers demands is superior compared to the V-type dampers.

## ACKNOWLEDGEMENT

The authors acknowledge the partial financial support from Australian Research Council (ARC) Linkage Project LP110200906 for carrying out this research.

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