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Effects of ground motion spatial variation and SSI on the response of multiple-frame bridges with unseating restrainers

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ABSTRACT: Bridge restrainers are the most commonly used retrofit device to limit large relative displacement that could lead to unseating failure. The present methods of designing restrainers calculate the relative displacement caused by the out-of-phase vibrations of adjacent bridge components with different fundamental frequencies, but neglect the relative displacements that are caused by differential ground motion input and Soil Structure Interaction (SSI) along the length of bridge. Previous investigations have revealed that neglecting spatial variability of ground motion and SSI could significantly underestimate relative displacements and thus lead to the inadequate design of the restrainers. This study evaluates the adequacy of the present restrainers design method as well as the overall bridge response subjected to spatially varying ground motion and SSI. The study particularly focuses on the influences of ground motion spatial variation and SSI on balanced frames that are emphasized by the modern codes. The effects of spatially varying ground motions and SSI on the response of the multiple-frame bridges with unseating restrainers are discussed through inelastic bridge response analysis.

1 INTRODUCTION

During an earthquake, bridge segments can vibrate out-of-phase owing to their dynamic characteristics and variation of ground motions at multiple bridge supports. This could result in pounding of adjacent bridge segments due to closing relative displacement exceeding the provided gap and more catastrophic unseating failure of the spans due to opening relative displacement exceeding the available seat width. Seismic restrainers are used as a retrofit measure in bridges with narrow supports to prevent excessive relative movement. Additionally, it is also used in new bridges with wide supports as a backup system. During the earthquakes such as the 1989 Loma Prieta, 1994 Northridge, 1995 Kobe etc. restrainers were found to be effective in protecting the bridges against unseating failure. However, a few bridges retrofitted with cable restrainers still collapsed due to unseating at joints (Moehle 1995), indicating the need for understanding the performance of restrainers during strong shaking and improvement in design methods. Many research works (Saiidi et al. 1996, Trochalakis et al. 1997, DesRoches and Fenves 2000, DesRoches and Muthukumar 2002) have been carried out since to understand the influencing factors on the behaviour of restrainers and its effect on performance of bridge structures. It should be noted that all these studies neglected the inevitable spatial variation of ground motions, which as will be demonstrated later is a principle factor that generates relative displacement between the bridge segments. Recent studies have highlighted that the relative displacement is significantly affected by spatial variations of ground motions and the characteristics of soil-structure system (Chouw and Hao 2005, 2008, Bi et al. 2011, Shrestha et al. 2015).The prevalent design method of restrainers (DesRoches and Fenves 2000) takes into account only the relative displacement caused by variation of fundamental frequencies of adjacent bridge segments. There are very limited studies on the response of bridge structures with cable restrainers to spatially varying ground motion including the SSI effects. Shrestha et al. (2014) presented the comparison of the response of different restrainers to Spatially Varying Ground Motions (SVGM); however, Soil Structure Interaction (SSI) was neglected.

This paper presents numerical investigations on the effects of SVGM and SSI on the response of bridge structures with cable restrainers. The analyses and discussions mainly are focused on the bridges with adjacent frames having close fundamental periods as required by the present codes. To realistically represent the structural response nonlinear Finite Element Model (FEM) with fibre section is adopted. The model includes energy dissipation due to pounding, foundation flexibility and damping, friction at isolator, non-uniform input motion and superstructure-abutment soil interaction. The present paper is an extension of the previous paper from the authors (Shrestha et al. 2015), where only longitudinal spatially varying ground motion was considered. This paper presents the response of the bridge structures to longitudinal and vertical ground motions applied simultaneously hence provide more realistic results.

2 FEM

2.1 Bridge Model

Four five-span bridge models are used for the analysis representing two bridge geometries. The decks of the bridge are composed of the concrete box-type girder with pre-stressing tendons. The bridge models are adopted from the studies of Feng et al. (2000) and Kim et al. (2000) and are representative of typical Californian bridges. 2-D nonlinear FEM are used for the analyses. The bridge models are shown in Figure 1. Following list the bridge models:

- Bridge 1: a five-span bridge with single intermediate joint, pier height of 19.83 m and diameter of 2.44 m. The adjacent segments of the bridge have fixed base period ratio of 0.91.
- Bridge 2: a five-span bridge with two intermediate joint, pier height of 19.83 and diameter of 2.44 m. The adjacent segments of the bridge have fixed base period ratio of 0.72.

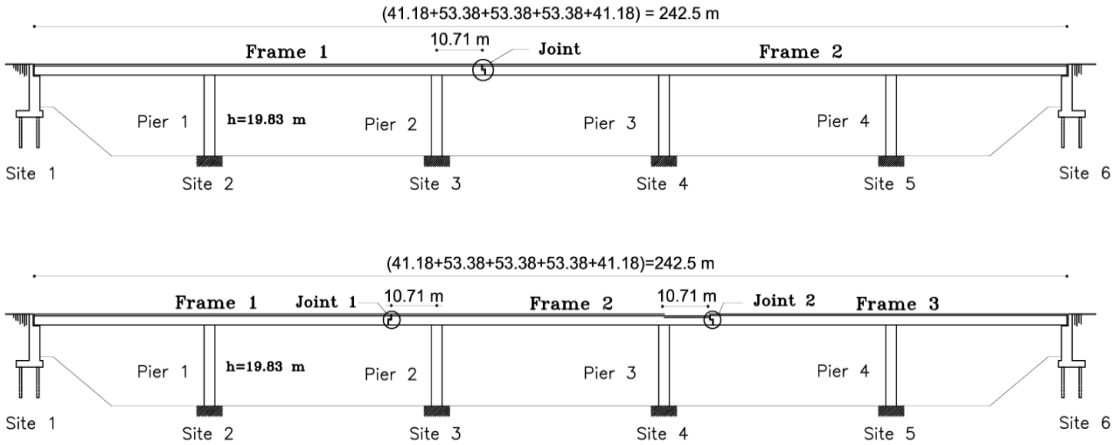

Figure 1. Bridge model 1 (top) with a joint and model 2(bottom) with two joints.

The bridge structures are modelled using the fibre element using force based inelastic reinforced concrete beam-column elements. The compressive strength of the concrete in taken as 27 *MPa* and yield strength of rebar is 415 *MPa*. The superstructure of the bridge is modelled using elastic beam column elements. Each deck element is discretized into six elements. Elastomeric bearing at the bridge abutments and joints are modelled using the elastic perfectly plastic link elements representing the friction provided by the bearings. The bridges consist of 6 elastomeric bearing of 0.4 m x 0.3 m. The initial stiffness of each bearing is 4.41 kN/mm and the coefficient of friction, μ at the interface of concrete and bearing is taken as 0.20.

2.2 Soil Structure Interaction

SSI in the model is incorporated using frequency-independent lumped spring-dashpot system. The size of the spread footing is 7 m x 7 m. Dynamic soil stiffness (spring and dashpot) of the foundation are calculated based on the study of Mylonakis et al. (2006) for surface foundation on homogeneous half space. Two linear translational and one rotation springs and dashpot are used to represent the stiffness and damping. Coefficients of springs and dashpots for sway, vertical and rocking degrees of freedom are summarized as follows:

$$K_z = k_z \left(\frac{4.54GB}{1-\nu}\right), \qquad C_z = (\rho V_{La} A_b)\hat{c}_z \tag{1}$$

$$K_x = k_x \left(\frac{9GB}{2-\nu}\right) \quad , \quad C_x = (\rho V_s A_b)\hat{c}_x \tag{2}$$

$$K_{ry} = k_{ry} \left(\frac{0.45GB^3}{1-\nu}\right), \quad C_{ry} = (\rho V_{La} I_{by})\hat{c}_{ry} \tag{3}$$

where K_z, C_z, K_x, C_x, K_{ry}, C_{ry} are the vertical stiffness, vertical viscous damping, sway stiffness, sway viscous damping, rocking stiffness and rocking viscous damping, respectively. k_z, k_x, k_{ry} are vertical, sway and rocking dynamic stiffness coefficients. $\hat{c}_z$, $\hat{c}_x$, $\hat{c}_{ry}$ are vertical, sway and rocking dynamic dashpot coefficients. B, A_b, I_{by} are the length, area and moment of inertia about the transverse axis of the square foundation. G, ρ, v, V_s and V_{La} are the shear modulus, soil density, Poisson's ratio, shear wave velocity and Lysmer's analogue wave velocity, respectively. In this study without losing generality only the local soil site classes presented in Table 1 are considered based on Caltrans (2013). Soil with shear wave velocity 400 m/s, Poisson's ratio 0.4 and density 19 kN/m^3, respectively is adopted for soil site class C. For soil site class D, the shear wave velocity of 220m/s, Poisson's ratio 0.4 and soil density 1.8 kN/m^3, respectively is adopted.

Table 1. Local soil site classes

Site Class	Shear wave velocity, Vs (m/s)	Selected value of Vs (m/s)
C. Very dense soil	360-760	400
D. Stiff soil	180-360	220

2.3 Impact element

Simplified bilinear impact model (Muthukumar 2003) is used to capture the effects of pounding including the energy dissipation during the impact. The impact model is an approximate representation of the Hertz damp model (Muthukumar and DesRoches 2006). The coefficient of restitution, *e* is assumed to be 0.8. The initial and the post-yielding stiffness of the impact element are taken as 10.68 GN/m and 3.68 kN/m, respectively, determined based on the work of Muthukumar (2003). Without losing the generality the gap width of intermediate joints are assumed as 25 mm and 50 mm at abutments.

3 GROUND MOTION MODELLING

Method proposed by Bi and Hao (2012) is used to simulate the spatially varying ground motion time histories that are compatible with the design spectra specified in the Japanese Highway Code (2004). The acceleration response spectrum for Type II ground motions, representing ground motions generated by an inland earthquake at short distance, at site class 2 (medium soil site) are used to generate the artificial ground motions. Theoretical coherency loss function (Sobczky 1991) is used to model spatial variation between two location j and k on ground surface.

$$\gamma_{jk}(i\omega) = \left|\gamma_{jk}(i\omega)\right| \exp(-i\omega d_{jk} \cos\alpha / v_{app}) = \exp(-\beta\omega d_{jk}^2 / v_{app}) \times \exp(-i\omega d_{jk} \cos\alpha / v_{app}) \tag{4}$$

In order to gain wider perspective on the response variations due to the non-uniform ground motions, simulation is carried out for three levels of coherency losses, i.e. β = 0.0005, 0.0010 and 0.0015 representing highly, intermediately and weakly correlated ground motions, respectively. To obtain relatively unbiased structural responses, three sets of ground motion time histories are simulated for each coherency loss levels. Apparent wave velocity, v_{app} is taken as 400 m/s, which is compatible with the measured shear wave velocities of typical highway bridge sites in California (Zhang and Markis 2000). Vertical ground motion level is considered to be 2/3 of the horizontal ground motion. Figure 2 shows spectral acceleration of horizontal and vertical ground motions for a set of spatially varying motions at six sites of the bridge shown in Figure 1.

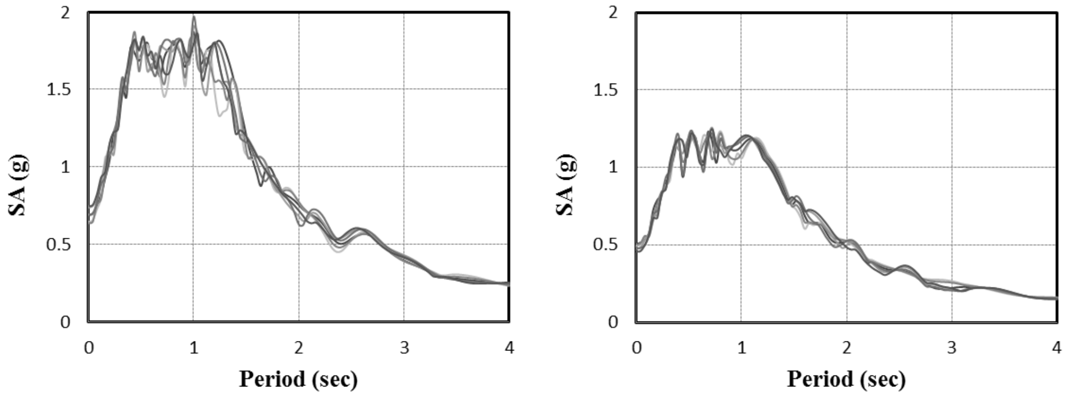

Figure 2. Response spectra of horizontal (left) and vertical ground motions (right)

4 RESTRAINER DESIGN

In this study, the restrainers are designed based on the procedure proposed by DesRoches and Fenves (2000). In the procedure, the relative hinge displacements are calculated using two-degree-of-freedom modal analysis. The relative hinge displacement, D_{eq} is estimated by combining the modal response using the complete quadratic combination rule,

$$D_{eq} = \sqrt{D_1^2 + D_2^2 - 2\rho_{12} D_1 D_2} \tag{5}$$

where D_1 and D_2 are the modal displacements and ρ_{12} is the correlation coefficient between the response of the two modes defined as:

$$\rho_{12} = \frac{8\sqrt{\xi_1\xi_2}\left(\xi_1 + \eta\xi_2\right)\eta^{3/2}}{\left(1-\eta^2\right)^2 + 4\xi_1\xi_2\eta\left(1+\eta^2\right) + 4\left(\xi_1^2 + \xi_2^2\right)\eta^2} \tag{6}$$

where η is the ratio of the frame fundamental period and ξ is the corresponding modal damping ratio. After calculating D_{eq}, the restrainers' stiffness, K_r, required to limit hinge displacement is determined from the sensitivity of the hinge displacement to restrainers' stiffness given by

$$\frac{\delta Deq}{\delta Kr} = -\frac{1}{Km + Kr} Deq \tag{7}$$

where $1/Km=1/K_1+1/K_2$ is the sum of the flexibilities of the two frames. Performing a Taylor series expansion about the current estimate of the hinge displacement, Deq_j, and solving for an improved estimate of the restrainer stiffness at the next step, Kr_{j+1}, gives

$$Kr_{j+1} = Kr_j + (Km+Kr_j)\frac{(Deq_j - Dr)}{Deq_j} \tag{8}$$

In this study available hinge seat width, D_{hinge} is taken as 200 mm, which represents narrow seated bridges before San Fernando earthquake era. The restrainers slack is assumed as 25 mm. Minimum required bearing length is assumed as 87 mm. The target yield displacement of the restrainers is then calculated to be 88 mm. Assume the yielding strain of high strength cable restrainers at 1.75%, the restrainers length is calculated to be 5.04 m. Consequently the stiffness of the restrainers required to restrict the hinge movement to the prescribed value, 113 mm (88+25), is determined using the above-mentioned modal analysis with multiple trials for each bridge models defined previously.

Typical 19 mm diameter high-strength cables used in Caltrans Bridge with cross sectional area of 143 mm^2 and yield strength of 1210 *MPa* are used for restrainers. The numbers of restrainer required are calculated as two and eight for bridge model 1(a) and 2(a). In this study without losing generality ten restrainer are provided at each joint for both bridges implying a slight over design. The restrainers are modelled as two node elasto-plastic cable element with tension only behavior. Slack of 25 mm is provided to accommodate the thermal movement of the bridge. To take an account of failure of restrainers under the extreme relative displacement ultimate strain of 4.5% is assigned to the element.

5 RESULTS AND DISCUSSION

5.1 Effect of SVGM

Effects of non-uniform ground motions on the response of bridge are evaluated for three sets of ground motions for five cases, i.e. uniform motion, spatially varying motion considering wave passage effect only and wave passage effect together with the three levels of coherency losses discussed previously. Without losing the generality, only the responses of the bridge on soil site C are presented here. Figure 3 shows the comparison of the Maximum Relative Hinge Displacement (MRHD) and Restrainers Deformation (RD) for the ground motions for bridge 1. The results are normalized by the averaged response under uniform ground motions. In the figure Uniform represents the case of uniform ground motion, WP (400) represents the motion considering wave passage effects only and CH (H), CH (I) and CH (W) represents the cases with highly, intermediately and weakly coherent motions, respectively. As shown the normalised MRHD for all the cases of spatially varying ground motions are significantly higher than those under uniform ground motions. The largest relative displacement at intermediate hinge corresponds to the ground motions with wave passage effects only. The dotted red line in the figure indicates available hinge seat width, D_{hinge} which is exceeded in a few cases with spatially varying ground motions, indicating unseating of the deck are likely. However, this result is used only for indicative purpose and the unseating of bridge spans is not explicitly modelled. The result presented also suggests that restrainers are deformed beyond its elastic limit for all cases of spatially varying ground motions and fracture (strain at 4.5%) is narrowly escaped in a case of ground motion with wave passage effects. Figures 4 presents the normalised MRHD and normalised RD for joint 1 of bridge model 2. In this case, the spatially varying ground motions considering coherency losses resulted in higher relative displacements. These results indicate that the effects of the coherency losses on the response of the bridge structures are not only dependent upon the ground motions, but also on the bridge geometry. Considering only the wave passage effects of the ground motion spatial variations may not be sufficient for accurate predictions of bridge structural responses. Moreover, the results again confirm that uniform excitation assumption significantly underestimates the MRHD and RD. The restrainers designed for the uniform ground motion may lose its functionality due to yielding or may even get fractured due to the large relative displacement caused by ground motion spatial variations.

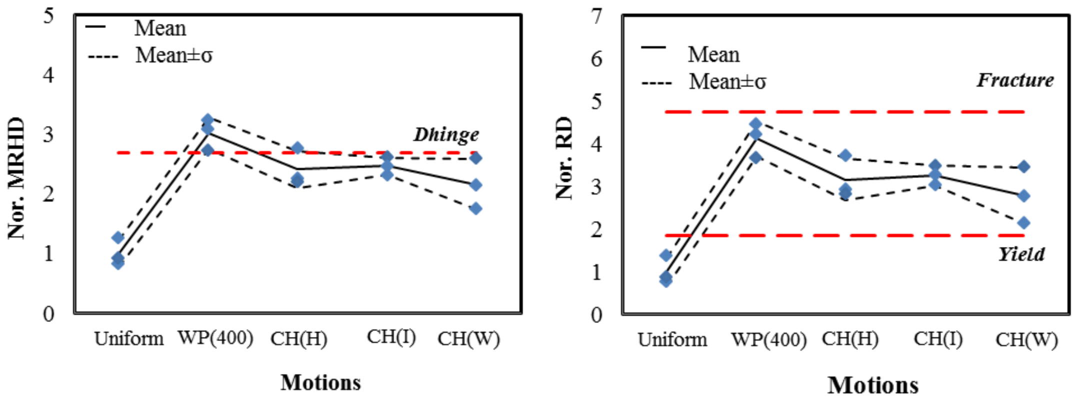

Figure 3. Normalised MRHD (left) and Normalised RD (right) at the joint of bridge 1

Figure 5 shows response of Pier 2 and Pier 3 of the bridge model 1 located at the stiffer frame and the flexible frame, respectively. The results indicate that pier drift are reduced due to spatially varying ground motion compared to uniform ground motions. Figure 6 presents the normalised peak pounding forces at the joint of bridge 1 and joint 1 of bridge 2. As shown the spatial variation of ground motions increases out-of-phase motion between adjacent bridge components which lead to severe pounding between adjacent bridge segments. Moreover, the frequencies of pounding between the bridge segments also increase. This impedes displacement of bridge frame and usually reduces the response of frame of both the stiff and flexible frames particularly when the fundamental period ratios of the frames are close. Though significant localised damages are expected at impacting locations due the pounding, response of the piers is reduced due to the spatial variation of ground motions.

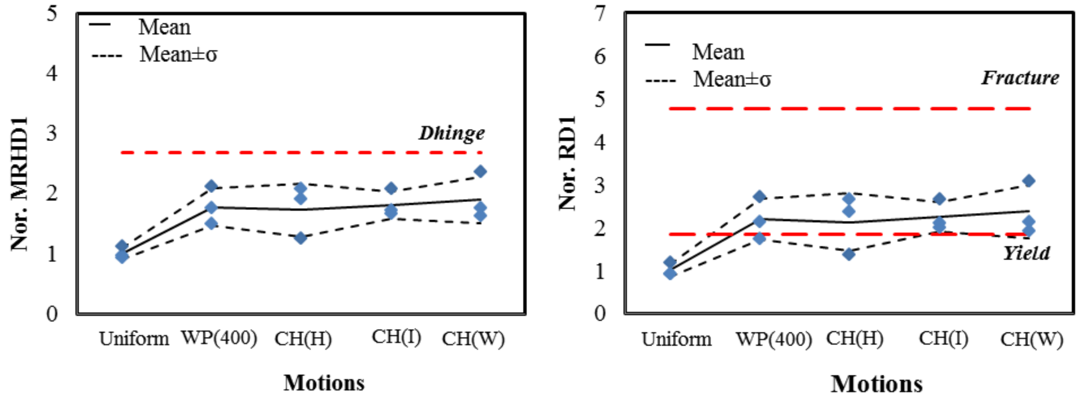

Figure 4. Normalised MRHD (left) and Normalised RD (right) at joint 2 of bridge 2(a)

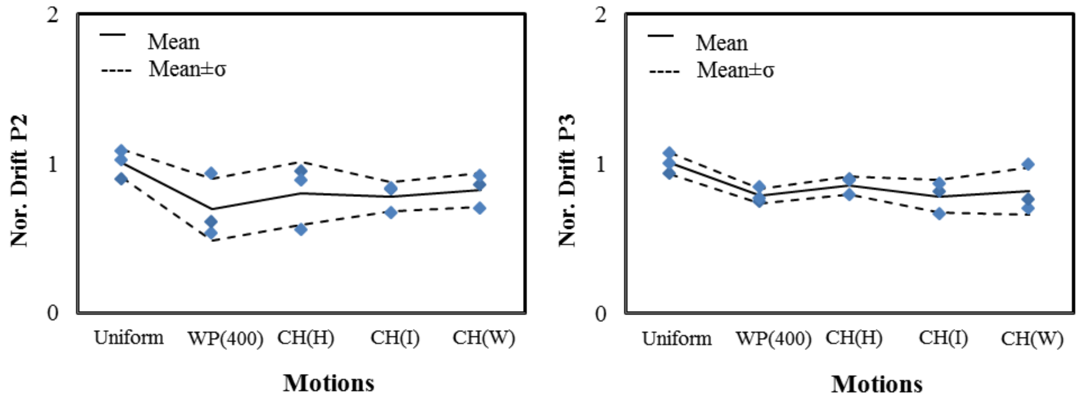

Figure 5. Normalised peak drift of Pier 2 (left) and Pier 3 (right) of bridge 1(a)

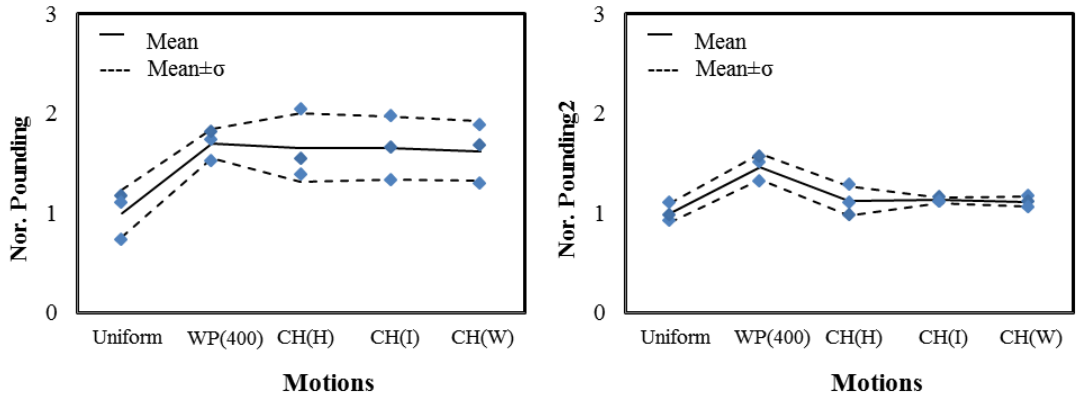

Figure 6. Normalised Pounding at the joint of Bridge 1 (left) Joint2 of Bridge 2 (right)

5.2 Effect of SSI

To identify the effects of SSI on the seismic response of the bridge to uniform and spatially varying ground motions comparative study of bridges with fixed base and with SSI (for two soil site conditions presented in Table 1) are conducted. Figure 7 presents MRHD and RD at the joint of bridge 1(a). SSI reduces the MRHD and the RD of the adjacent bridge due to the increase in vibration period of adjacent frames. Due to the shift in the vibration period, the ratio of the periods of the adjacent structures shifts closer to unity thus resulting in the reduction of out-of-phase vibration of the adjacent bridge frames. Figure 8 presents the drift of Pier 2 (stiffer frame) and pier 3 of bridge 1(a). The result presented suggests the absolute displacement of the both frames increases with soil flexibility. It is interesting to notice that SSI reduces the relative displacement but increases the overall drift of both the frames. The effects of SSI on section curvature demand at the base of the bridge piers are plotted in Figure 9. Though the drift demand of the bridge piers is increased, the curvature demand is apparently reduced. This result indicates that the higher drift demand is absorbed by the foundation due to its flexibility and damping and reduces the curvature demand at the base of bridge piers.

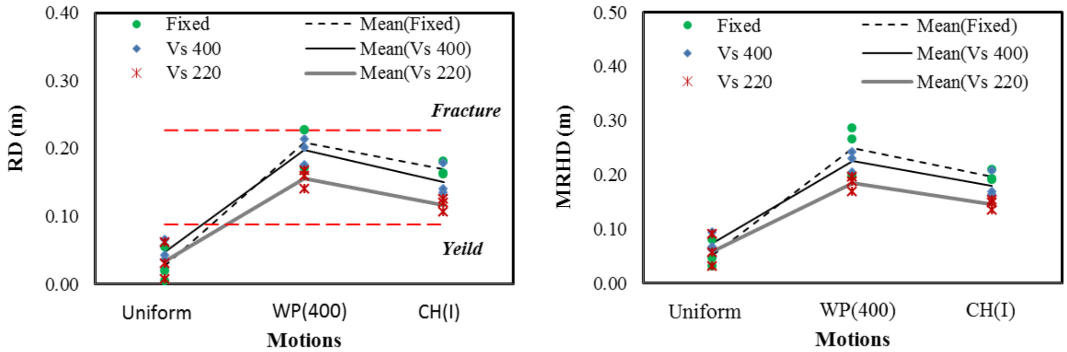

Figure 7. Calculated MRHD (left) and RD (right) at the joint of bridge 1

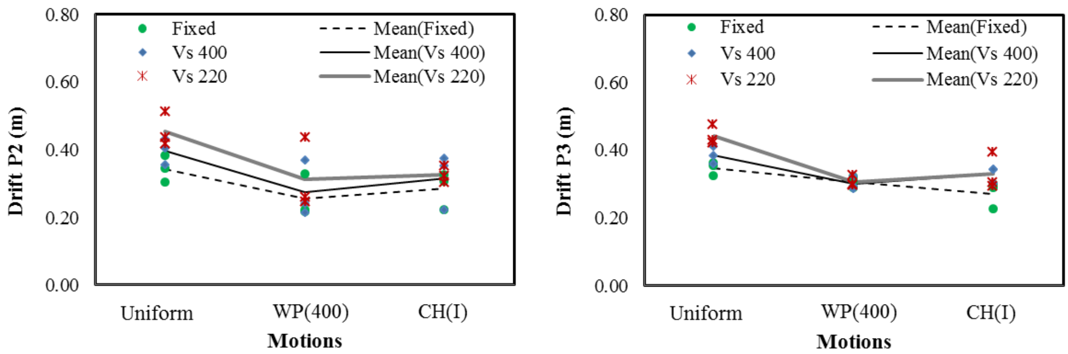

Figure 8. Calculated drift of Pier 2 (left) and Pier 3(right) of bridge 1

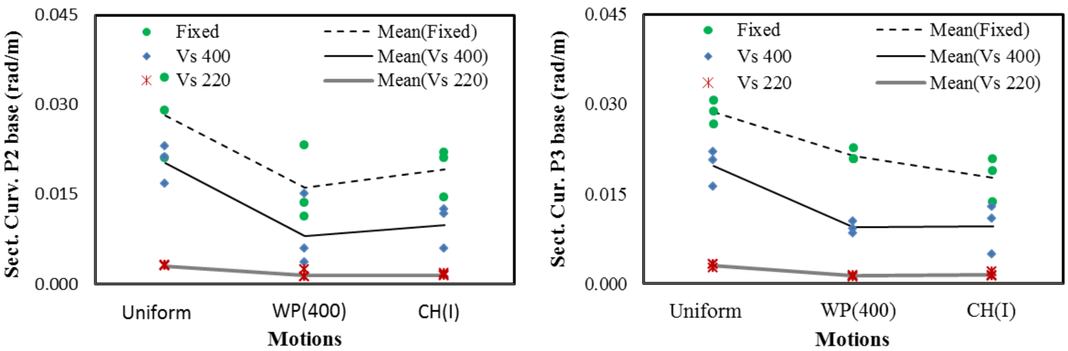

Figure 9. Peak curvature demand at the base of Pier 2 (left) and Pier 3 (right) of bridge 1

6 CONCLUSION

Numerical investigations conducted herein suggest that SVGM have a significant effect on the relative displacement response of the adjacent bridge segments. The existing design method that neglects the SVGM could result in the underestimation of required stiffness and strength of the restrainers to limit the joint opening movement. SVGM, however, have a positive effect on the response of bridge piers in the longitudinal direction.

The relative displacement responses of the bridges under this study are reduced to the inclusion of SSI. This phenomenon could be attributed to the reduction of relative out-of-phase motion caused by flexibility introduced by SSI. Though the overall drift increased, the curvature demands at the base of the piers are reduced.

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REFERENCES:

Bi K., Hao H., Chouw N. 2011. Influence of ground motion spatial variation, site condition and SSI on the required separation distances of bridge structures to avoid seismic pounding. *Earthquake Engineering & Structural Dynamics*. 30 (1). 141-153.

Bi K., Hao H. 2012. Modelling and simulation of spatially varying ground motions at sites with varying conditions. *Probabilistic Engineering Mechanics*. 29. 92-104.

Caltrans. 2013. Seismic Design Criteria: Design manual version 1.7. *California Dept. of Transportation,* California.

Chouw N., Hao H. 2005. Study of SSI and non-uniform ground motion effects on pounding between bridge girders. *Soil Dynamics and Earthquake Engineering*. 23 (7). 717-728.

Chouw N., Hao H. 2008. Significance of SSI and non-uniform near-fault ground motions in bridge response I: Effect on response with conventional expansion joint. *Engineering Structures*. 30 (1). 141-153.

DesRoches R., Fenves G.L. 2000. Design of Seismic Cable Hinge restrainers for bridges. *Journal of Structural Engineering*. 126 (4). 500-509.

DesRoches R., Muthukumar S. 2002. Effect of pounding and restrainers on seismic response of multiple-frame bridges. *Journal of Structural Engineering*. 128 (7). 860-869.

Feng M.Q., Kim J.M., Shinozuka M., Purusinghe R. 2000. Viscoelastic dampers at expansion joints for seismic protection of bridges. *Journal of Bridge Engineering*. 5 (1). 67-74.

Japan Road Association. 2004. Specification of highway bridges- Part V seismic design. 5th edition. (in Japanese).

Kim J.M., Feng M.Q., Shinozuka M. 2000. Energy dissipating restrainers for highway bridges. *Soil Dynamic and Earthquake Enginereing*. 19 (1). 65-69.

Moehle J.P. 1995. Northridge Earthquake of January 17, 1994: Reconnaissance Report, Volume1- Highway bridges and Traffic Management. *Earthquake Spectra*. 11 (4). 287-372.

Muthukumar S. 2003. A contact element approach with hysteresis damping of the analysis and design of pounding in bridges. *Phd thesis. Georgia institute of technology*. USA.

Mylonakis G., Sissy N., Gazetas G. 2006. Footing unders seismic loading: Analysis and design issues with emphasis on bridge foundation. *Soil Dynamic and Earthquake Enginereing*. 26 (9). 824-853.

Saiidi M., Maragakis E., Feng S. 1996. Parameters in bridge restrainer design for seismic retrofit. *Journal of Structural Engineering*. 122 (1). 61-68.

Shrestha B., Hao H., Bi K. 2015. Seismic response analysis of multiple-frame bridges with unseating restrainers considering ground motion spatial variation and SSI. *Advances in Structural Engineering*. 18 (6). 873-892.

Shrestha B., Hao H., Bi K. 2014. Effectiveness of using Rubber bumper and Restrainer on mitigating Pounding and Unseating damages of bridge structures subjected to spatially varying ground motions. *Engineering Structures*. 79. 195-210.

Sobczky K. 2011. Stochastic Wave Propagation. *Kluwer Academic Publisher*. Netherlands.

Zhang J., Markis N. 2002. Seismic response analysis of highway overcrossing including soil-structure interaction. *Earthquake Engineering & Structural Dynamics*. 31 (11). 1933-1966.