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Effects of ground motion frequency content on performance of isolated bridges with SSI

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Abstract. The present study considers a multi-span continuous bridge, isolated by lead rubber bearing (LRB). Dynamic soil-structure interaction (SSI) is modelled with the help of a simplified, sway-rocking model for different types of soil. It is well understood from the literature that SSI influences the structural responses and the isolator performance. However, the above-mentioned effect of SSI also depends on the earthquake ground motion properties. It is very important to understand how the interaction between soil and structure varies with the earthquake ground motion characteristics but, as far as the knowledge of the authors go, no study has been carried out to investigate this effect. Therefore, the objectives of the present study are to investigate the influence of earthquake ground motion characteristics on: (a) the responses of a multi span bridge (isolated and non-isolated), (b) the performance of the isolator and, most importantly, (c) the soil-structure interaction.

Statistical analyses are conducted by considering 14 earthquakes which are selected in such a way that they can be categorized into three frequency content groups according to their peak ground acceleration to peak ground velocity (PGA/PGV) ratio. Lumped mass model of the bridge is developed and time history analyses are carried out by solving the governing equations of motion in the state space form. The performance of the isolator is studied by comparing the responses of the bridge with those of the corresponding uncontrolled bridge (i.e., non-isolated bridge). On studying the effect of earthquake motions, it is observed that the earthquake ground motion characteristics affect the interaction between soil and structure in such a way that the responses decrease with increase in frequency content of the earthquake for all the types of soil considered. The reverse phenomenon is observed in case of the isolator performance where the control efficiencies increase with frequency content of earthquake.

Keywords: bridge; elastomeric bearings; dynamic soil structure interaction; earthquake; frequency content

1. Introduction

Bridges are generally recognized as important components of the transportation network and they form a vital link in the lifeline of a nation in managing emergencies caused by natural or manmade disasters. Historically, catastrophic bridge failure examples are found all over the world during moderate to strong earthquakes. Although considerable progress has been achieved in the design and construction of earthquake resistant bridges, there are many gap areas which still remain unexplored in understanding the seismic behavior of bridges.

The conventional methods of design, in which the bridges are designed to resist a severe earthquake elastically, are recognized to be uneconomical. The effort towards protection of bridges against earthquakes should therefore be focused on minimizing the forces, in particular, the shear forces, to be carried by the piers.

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A typical earthquake acceleration record has dominant periods of about 0.1 to 1 second, with maximum severity often in the range 0.2 to 0.6 second. Considering this aspect, bridges are more vulnerable to the seismic forces due to resonance, as majority of bridges have their fundamental natural periods in the range of 0.2 to 1.2 s. (Naeim and Kelly 1999).

The seismic forces on bridges can be reduced if the fundamental period of a bridge is lengthened or its energy dissipating capability is increased. Seismic isolation is an effective and promising alternative in this regard as it reduces the forces on bridges by lengthening the fundamental period of the bridge, and/ or by increasing the energy dissipating capacity. Numerous studies had been done in this area and seismic isolation is found to be one of the most successful techniques to mitigate the risk to life and property during strong earthquakes (Skinner *et al.* 1993). Isolation systems have found extensive application in both buildings and bridges (Kunde and Jangid 2003, Patil and Reddy 2012, Vasiliadis 2016). There have been several studies investigating the effectiveness of isolation devices for seismic design of bridges. Ghobarah and Ali (1988), and Turkington *et al.* (1989) have shown that the lead-rubber bearings are quite effective in reducing the seismic response of bridges by shifting the natural period of the structure. Jangid (2007) and Matsagar and Jangid (2004) studied the effect of isolator characteristics on the response of the

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building structure. Parametric studies to investigate the effect of lead rubber bearing (LRB) isolator and ground motion characteristics on the response of seismic isolated bridges had also been conducted (Park *et al.* 2002, Hameed *et al.* 2008, Neethu and Das 2015). All these studies show that LRB can effectively reduce the seismic responses of bridge structures. It is to be noted that in all these studies, the seismic analyses of bridge structures are performed by assuming that the soil around the base is rigid and the ground motion is directly transferred to the structure.

However, soil around the foundation interacts with each other and as a result, the fundamental period and additional damping of the superstructure are increased, which is called the soil-structure interaction (SSI) effect. Hence, the consideration of SSI effect becomes inevitable in case of structures whose seismic responses are strongly influenced by the time period variation. Several studies (Krishnamoorthy 2013, Raheem and Hayashikawa 2013, Wang *et al.* 2014, Luco 2014) show that SSI greatly influences the structural responses.

The seismic response characteristics of isolated bridges are strongly governed by the fundamental structural period. Only in few of the literature on seismic isolation of bridges, the effect of SSI is considered due to the complications involved in modelling. Dicleli *et al.* (2005) studied the effect of SSI for friction pendulum bearings and the study revealed that the SSI effects need to be considered for bridges with light superstructures and heavy substructures, regardless of the stiffness of the foundation soil. Papathanasiou and Tsopelas (2008), considering two bridge systems, showed that SSI causes higher isolation system drifts as well as, in many cases, higher pier shears when compared to the fixed-pier bridges (no SSI). Tongaonkar and Jangid (2003) had studied the effect of SSI on the peak responses of a bridge, emphasizing on the significance of the parameters affecting the response of the system.

Hence, it is well understood from the above-mentioned studies that SSI influences the structural responses and the isolator performance. However, the above-mentioned effect of SSI also depends on the earthquake ground motion properties. It is very important to understand how the interaction between soil and structure varies with the earthquake ground motion characteristics but, as far as the knowledge of the authors go, no study has been carried out to investigate this effect.

As far as the earthquake ground motion characteristics are concerned, the ratio of peak ground acceleration(PGA) to peak ground velocity(PGV) is an informative measure that can account for the frequency content of input motions (Jafarian *et al.* 2010). The PGA/PGV ratio obtains practical information to characterize the damage potential of ground motions and can be considered as a measure of destructiveness. In fact, ground motions having lower PGA/PGV values result in larger damage potential (Zhu *et al.* 1988). Gradation of soils may also influence whether low or high frequency waves are more damaging to the structure (Garg *et al.* 2016).

Therefore, the objective of the present study is to investigate the influence of earthquake ground motion characteristics (in terms of PGA/ PGV ratio) on the soil and structure interaction. This investigation is carried out by studying the variation of (i) responses of a multi-span bridge (isolated and non-isolated) and (ii) performance of the isolator with the PGA/PGV ratio.

2. Theoretical development

This study investigates the performance of a passive control scheme of typical multi-span continuous bridges considering the effect of soil structure interaction. The theoretical development of the isolated-structure involves mathematical modelling of the bridge, isolator and also the soil-structure interaction. The numerical simulations of the dynamic responses of the bridge, subjected to various near field and far field earthquakes, are carried out using a full-state feedback control loop. MATLAB, Simulink, and a complementary set of toolboxes are used to conduct the numerical simulation and computations in the work that follows. The related formulations are presented in this section.

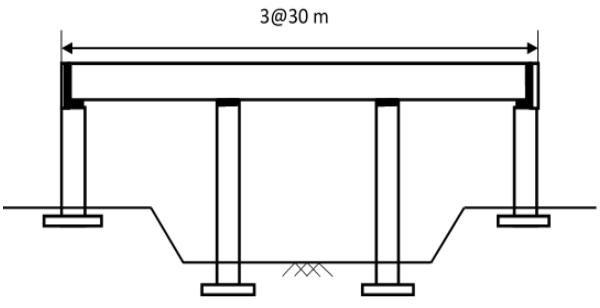
2.1 Modeling of the bridge

This study investigates the seismic response and performance of a three-span continuous bridge. The three-span bridge deck, isolated by laminated rubber bearing placed at the top of pier is shown in Fig. 1(a). A simplified model of the deck-pier system, attached with the LRB, is shown in Fig. 1(b). The equivalent mechanical model for the same is depicted in Fig. 1(c). The pier is discretized into a number of nodes with lateral degrees of freedom (as in shear building) and the bridge deck is treated as rigid mass, and the piers and the deck are assumed to remain elastic. The model considered in the present study has been used by researchers previously (Ghobarah and Ali 1988, Kunde and Jangid 2006, Jangid 2008 and Wang *et al.* 1998). It is to be noted that Kunde and Jangid (2006) had investigated the accuracy and computational efficiency of three different mathematical models of the isolated bridge "for the analytical seismic response by considering and ignoring the flexibility of the deck and piers". The model used in the present study is one of those three models. Kunde and Jangid (2006) observed that "the seismic responses of the bridge obtained from different equivalent mathematical models are comparable" and concluded that the earthquake response of the bridge can be effectively obtained by the simplified model used in the present study. Thus, it may be stated that even though the mathematical model considered in the present study is a simplification of the actual bridge model, the primary objective of the present study, which is to investigate the influence of earthquake ground motion frequency content as mentioned above, is fulfilled.

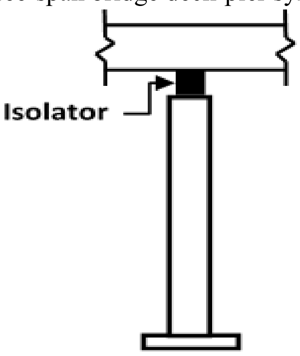
The governing equation of motion for the bridge (multi-degree of freedom system without SSI) subjected to ground motion is expressed in matrix form as

$$[M]\{\ddot{X}\}+[C]\{\dot{X}\}+[K]\{X\}+[D]\{F\}=-[M][r]\{\ddot{X}_g\} \quad (1)$$

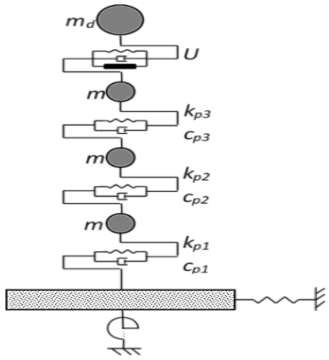
where, [M], [C] and [K] are mass, damping and stiffness matrices, respectively, of the bridge structure of the order [4 × 4]; $\{X\} = \{X_1, X_2 \ldots X_n, X_d\}^T$, $\{\dot{X}\}$ and $\{\ddot{X}\}$ are structural



(a) Three-span bridge deck-pier system



(b) Simple structural model for deck-pier system attached with LRB



(c) Equivalent mechanical model

Fig. 1 Schematic of the bridge model

displacement, structural velocity and structural acceleration vectors, respectively; X_d is the displacement of the bridge deck relative to ground; [D] is the location matrix for the restoring forces; {F} is vector containing the restoring forces; $\{r\} = \{1,1,.........,1\}^T$ is influence coefficient vector and $\{\ddot{X}_g\}$ is earthquake ground acceleration.

2.1.1 Modelling of soil structure interaction

The soil structure interaction is modeled using the sway-rocking model available in literature (Mikami and Sawada 2004) (Fig. 2). For modelling the SSI in this study, the effect of kinematic interaction is ignored, considering the foundation geometry such that its depth is small enough compared to its width (Mikami and Sawada 2004). It is to be noted that, the same bridge model has been used in a previous study (Tongaonkar and Jangid 2003) in which the SSI for the soil supporting the pier foundation has been modelled as spring and damper acting in the horizontal and rotational directions. The foundation was represented for all motions using a spring-dashpot-mass model with

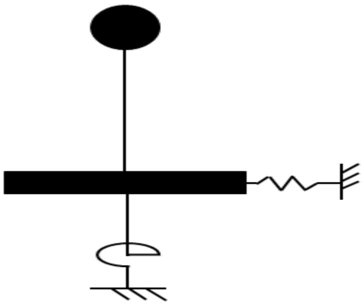


Fig. 2 Sway-rocking model of soil-structure interaction

frequency-independent coefficients. Referring to available literature (Spyrakos 1990), Tongaonkar and Jangid (2003) have mentioned in their study that, “a sufficiently accurate consideration of soil behaviour can be obtained if the soil stiffness and damping coefficients of a circular mass less foundation on soil strata are evaluated by the frequency independent expressions”. Thus, the same type of frequency-independent SSI model for mass less foundation is considered in the present study.

The present study is to have a global assessment of the modification of structural response and isolator effectiveness due to the change in SSI effect brought about by the earthquake ground motion frequency content. The reason behind this is that the performance of isolators are influenced by the flexibility of the soil surrounding the foundation because, structural system with SSI becomes more flexible than the rigidly founded isolated structure and also because of the energy dissipation that takes place in the soil medium. Thus, to be more precise, objective of the present study is to investigate how this SSI effect, in turn, is dependent on the earthquake ground motion frequency content and how, as a result, the responses of the structure and performance of the isolator are influenced by the ground motion. Moreover, in order to study the effect of ground motion frequency content in a more comprehensive way, fourteen real earthquakes have been taken. If any frequency-dependent model is considered, the number of earthquake records that may be taken will be much less due to the complexity involved in the procedure and this will lead to loss of generality of the conclusions, without much gain in terms of accuracy of the results.

Eq. (2) gives the governing equation of superstructure-foundation-soil system if the superstructure is modeled as a single-degree-of-freedom system.

$$\begin{bmatrix} m_s & m_s & m_sH \\ m_s & m_s+m_f & m_sH \\ m_sH & m_sH & m_sH^2+J_f \end{bmatrix}\begin{pmatrix} \ddot{u} \\ \ddot{u}_f \\ \ddot{\theta}_f \end{pmatrix}+\begin{bmatrix} c_s & 0 & 0 \\ 0 & c_{hh} & 0 \\ 0 & 0 & c_{rr} \end{bmatrix}\begin{pmatrix} \dot{u} \\ \dot{u}_f \\ \dot{\theta}_f \end{pmatrix}+\begin{bmatrix} k_s & 0 & 0 \\ 0 & k_{hh} & 0 \\ 0 & 0 & k_{rr} \end{bmatrix}\begin{pmatrix} u \\ u_f \\ \theta_f \end{pmatrix}=-\begin{pmatrix} m_s \\ m_s+m_f \\ m_sH \end{pmatrix}\ddot{u}_g \quad (2)$$

where u = horizontal displacement of the structure relative to the foundation, u_f = displacement of the foundation relative to free-field, u_g = earthquake ground acceleration, θ_f = rocking angle of the foundation, m_s = mass of the structure, H = height of the structure, m_f = mass of the foundation, θ_k = base rocking of massless foundation slab due to kinematic interaction, c_{hh}, c_{rr} = damping coefficients for foundation, k_{hh} ,k_{rr} = stiffness for foundation.The values of c_{hh}, c_{rr}, k_{hh} and k_{rr} can be calculated from the relations given by Eqs. (3)-(6) (Gazetas 2006).

$$k_{hh} = 8Gr / (2 - \mathrm{u}) \tag{3}$$

$$k_{rr} = 8Gr^3 / 3(1 - \mathrm{u}) \tag{4}$$

$$c_{hh} = 4.6\rho v_s r^2 / (2 - u) \tag{5}$$

$$c_{rr} = 0.4\rho v_s r^4 / (1 - u) \tag{6}$$

where, ρ is the unit weight of soil, u is the Poisson's ratio, v_s is the shear wave velocity, r is average dimension of circular plate and G is the shear modulus of soil

Based on the equation for soil structure interaction for a single degree of freedom (Eq. (2)) the equation for bridge soil structure interaction is developed. The bridge structure is discretized along the length of the pier with masses lumped as shown in Fig. 1(c). Considering the equation of motion of the bridge (Eq. (1)), Eq. (2) is modified and is given as follows

$$\left[M_{sf}\right]\{\ddot{X}\}+\left[C_{sf}\right]\{\dot{X}\}+\left[K_{sf}\right]\{X\}+[D]\{F_b\}=-\left[M_{sf}\right][r]\{\ddot{X}_g\} \tag{7}$$

where, $[M_{sf}]$, $[C_{sf}]$ and $[K_{sf}]$ are mass, damping and stiffness matrices considering the effect of SSI respectively, of the bridge structure of the order [6×6]; $\{X\}=\{X_1, X_2 \ldots X_n, X_d, X_f, \theta_f\}^T$, $\{\dot{X}\}$ and $\{\ddot{X}\}$ are structural displacement, structural velocity and structural acceleration vectors, respectively; X_d is the displacement of the bridge deck relative to ground; X_f is the displacement of the foundation relative to free-field, θ_f is the rocking angle of foundation, $[D]=\{0,0,0,1,0,0\}^T$ is the location matrix for the restoring forces of damper; $\{F_b\}$ is a vector containing the restoring forces of isolator; $\{r\}=\{0,0,0,0,0,1,0\}^T$ is the influence coefficient vector; $\{\ddot{X}_g\}$ is earthquake ground acceleration.

State Space Formulation

The solution of the equations of motion [Eqs. (1) and (7)] are obtained by writing them in state-space form

$$\dot{z}(t) = Az(t) + Bu(t) + Hf(t) \tag{8}$$

where $z(t) = \begin{bmatrix} x(t) \\ \dot{x}(t) \end{bmatrix}$ is the 2n-dimensional state vector, where n=6

$A = \begin{bmatrix} 0 & I \\ -M^{-1}K & -M^{-1}C \end{bmatrix}$ is the [2n ×2n] system matrix, and $B = \begin{bmatrix} 0 \\ M^{-1}D \end{bmatrix}$ and $H = \begin{bmatrix} I \\ M^{-1}E \end{bmatrix}$ are [2n × r] and [2n × 1] location matrices specifying respectively, the locations of controllers and external excitations in the state-space. 0 and I denote respectively, the null matrix and the identity matrix of appropriate dimensions.

2.2 Modeling of laminated rubber bearing (LRB)

The LRB isolator is a multi-layered laminated rubber bearing along with a central lead-core to add damping to the isolation system. The LRB isolator provides the combined features of vertical load support, horizontal flexibility, restoring force and damping in a single unit. The schematic diagram of the combined mechanism is shown in Fig. 3. The ideal force-deformation behaviour of the LRB system is generally represented by non-linear characteristics following a hysteretic nature as shown in Fig. 3.

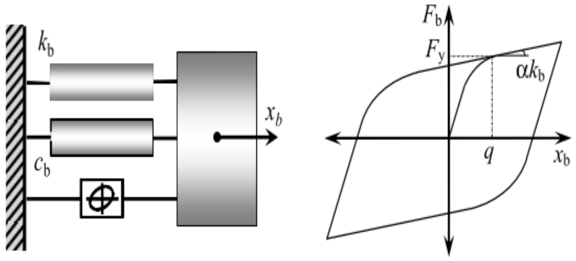


Fig. 3 Schematic and force-deformation behavior of Lead-rubber bearing (LRB)

One of the most important parameter of LRB isolator is F_o. The yield strength of the bearing is normalized with respect to the total weight of the isolated building and expressed by the parameter F_o defined as

$$F_o = \frac{Q_d}{W} \tag{9}$$

where W = Mg is the total weight of the isolated building; and g is the acceleration due to gravity. This parameter is largely related to the responses of the structure under earthquake.

The second parameter is the post-yield stiffness, k_b of the LRB bearing is designed in such a way that it provides the specific value of the isolation period T_b expressed as

$$T_b = 2\pi \sqrt{\frac{M}{k_b}} \tag{10}$$

Another important parameter is the viscous damping c_b in the bearing due to rubber, is evaluated by the damping ratio ξ_b expressed as

$$\xi_b = \frac{c_b}{2M\omega_b} \tag{11}$$

where $\omega_b = 2\pi/T_b$ is the base isolation frequency. The mathematical modelling is done with the help of a non-linear Wen's (Wen 1976) model to characterize the hysteretic behaviour of the LRB systems. The restoring force developed in the isolation bearing is given by,

$$F_b = c_b\ddot{x}_b + \alpha k_b x_b + (1-\alpha)F_y Z \tag{12}$$

where, F_y is the yield strength of the bearing; α is an index which represent the ratio of post to pre-yielding stiffness; k_b is the initial stiffness of the bearing; c_b is the viscous damping of the bearing; and Z is the non-dimensional hysteretic displacement component satisfying the following non-linear first order differential equation expressed as,

Table 1 Earthquake ground motion characteristics

Earthquake	M_w	Station	Dist (km)	PGA (g)	PGV (cm/s)
Imperial Valley	6.5	Coachella Canal #4	50.1	0.115	12.47
Coalinga	6.4	Parkfield- Cholame	55.7	0.039	4.22
San Fernando	6.6	2516 Via Tejon PV	55.2	0.025	3.82
Victoria Mexico	6.3	SAHOP Casa Flores	39.3	0.101	7.77
Chalfant Valley	6.2	Convict Creek	31.1	0.071	3.84
Whittier Narrows	6	Canyon Country	48.1	0.109	7.32
Landers	7.3	Baker Fire Station	87.9	0.107	9.32
Kobe	6.9	KJMA	18.2	0.831	95.75
Chichi	7.62	TCU065	26.6	0.831	129.5
Northridge	6.69	LA DAM	11.7	0.576	77.09
Tabas	7.35	Tabas	1.2	0.851	121.22
Morgan hill	6.19	Anderson dam	16.6	0.449	29.01
Loma prieta	6.9	LGPC	3.5	0.57	102.0
Mendocino	7.1	Petrolia	8.5	0.59	100.0

Table 2 Classification of earthquake ground motions based on PGA/PGV ratio

Range	Earthquake	PGA/PGV (g/m/s)	2πPGV/PGA (s)
PGA/PGV<0.8 (Low Frequency)	Loma Prieta	0.559	11.24
	Mendocino	0.590	10.65
	Chichi	0.641	9.79
	San Fernando	0.654	9.60
	Tabas	0.702	8.95
	Northridge	0.747	8.40
0.8<PGA/PGV<1.2 (Intermediate Frequency)	Kobe	0.868	7.24
	Imperial Valley	0.922	6.81
	Coalinga	0.924	6.80
	Landers	1.148	5.47
PGA/PGV>1.2 (High Frequency)	Victoria Mexico	1.300	4.83
	Whittier Narrows	1.489	4.22
	Morgan Hill	1.548	4.06
	Chalfant Valley	1.849	3.40

$$q\dot{Z} = A\dot{x}_b - \beta\,|\,\dot{x}_b\,|\,z\,|\,z\,|^{n-1} - \tau x_b\,|\,z\,|^{n} \tag{13}$$

where, q is the yield displacement; dimensionless parameters β, τ, A and n are selected such that predicted response from the model closely matches with the experimental results (Constantinou and Tadjbakhsh 1985). The parameter n is an integer constant, which controls smoothness of transition from elastic to plastic response.

3. Numerical study

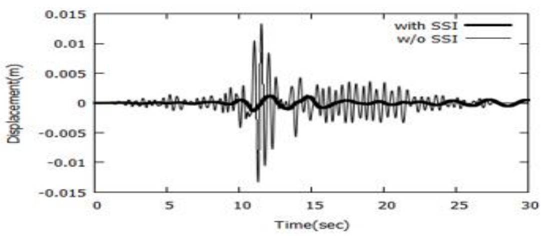
The present study investigates the seismic response of a three-span continuous bridge, the properties of which correspond to the bridge studied by Wang *et al.* (1998). The properties are: deck mass = 771.12×10^3 kg; mass of each pier = 39.26 × 10^3 kg; moment of inertia of piers = 0.64 m^4;

Table 3 Properties of different types of soil considered in the study

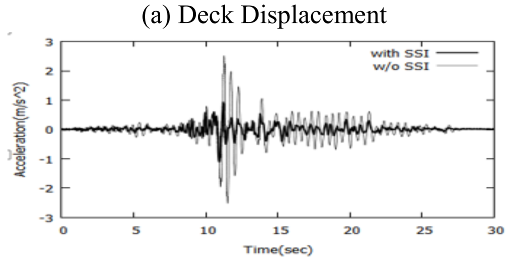
Soil Properties	Hard (Clayey Soil)	Medium (Sandy Soil)	Soft (Silty Soil)
Unit Weight (ρ) (kN/m^3)	20	19	18
Poisson Ratio (μ)	0.3	0.35	0.4
Damping of Soil	0.02	0.04	0.06
Shear wave velocity (v_s)(m/s)	1050	309	83
G (kN/m^2)	269×10^4	192310	12500
E (kN/m^2)	700×10^4	500 ×10^3	35 ×10^3

Table 4 Variation of time period and frequency of bridge with SSI effect

Soil Conditions	Time Period (s)	Frequency (Hz)
Fixed Base(no SSI)	0.45	2.22
Hard Soil	0.9293	1.076
Medium Soil	2.6041	0.384
Soft Soil	3.657	0.2734



(a) Deck Displacement



(b) Deck Acceleration

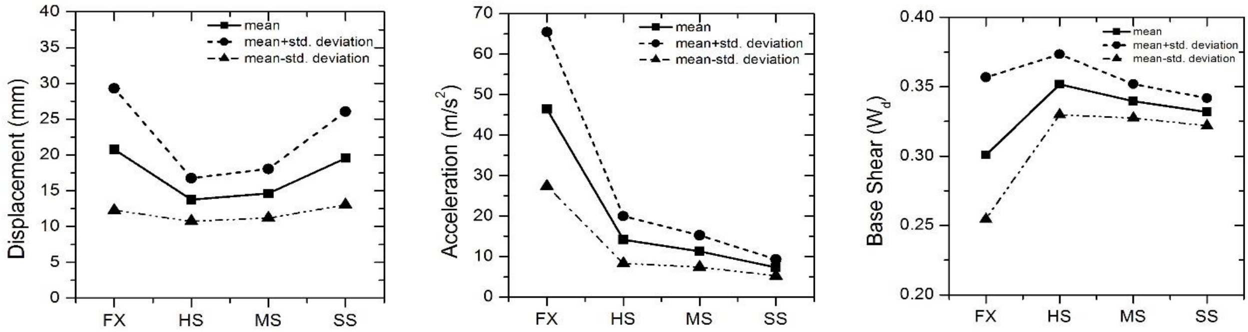
Fig. 4 Time history of responses of the bridge under Imperial Valley earthquake

Young's modules of elasticity = 20.67 × 10^9 N/m^2; pier height = 8 m; and total length of bridge = 90 m. The fundamental time period of the bridge is 0.45 s.

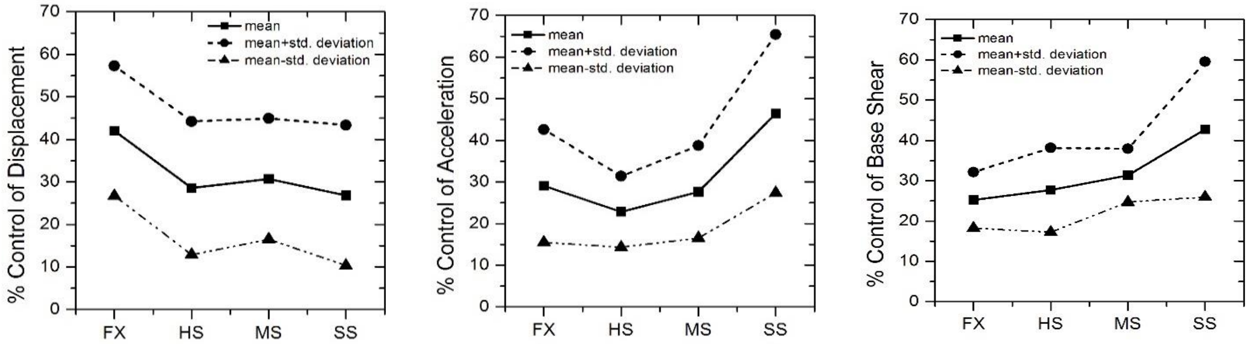
The seismic response of the bridge and the effectiveness of the passive control system (LRB isolator) are investigated for seven far-field and seven near-field earthquake ground motions. The near-field (NF) earthquake ground motions considered are: Kobe, Northridge, Chichi, Tabas, Morgan Hill, Loma Prieta and Mendocino. The far-field (FF) earthquake ground motions considered are Imperial Valley, Coalinga, San Fernando, Victoria Mexico, Whitter Narrows, Chaflant, Landers.

Table 1 shows the details of the ground motion characteristics and the earthquake data taken from the PEER ground motion database. For the seismic response analysis, all the ground motions are scaled to 0.3 g.

The seven far-field ground motion records considered in the study have been identified from the list provided in the



(a) Maximum absolute responses



(b) Percentage control

Fig. 5 Variation of performance of LRB in terms of mean and mean ± standard deviation

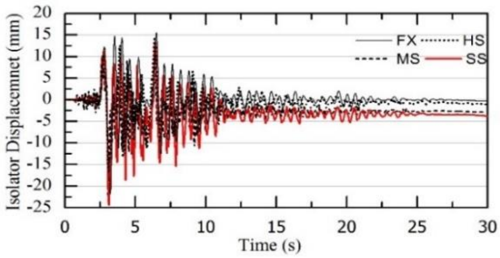


Fig. 6 Time history of isolator displacement under Northridge earthquake

literature (Davoodi *et al.* 2012). Similarly, out of the seven near-field ground motion records considered in the present study, four (Kobe, Northridge, Chichi and Morgan Hill earthquakes) have been identified from the first set of near-field earthquakes presented by Davoodi *et al.* (2012). These four near-field earthquakes, listed in the said literature, are characterized with the forward-directivity effects and no information regarding the distance is available in the said literature. The remaining three near-field records (Tabas, Loma Prieta and Mendocino) have been identified from list of near-field earthquakes provided in Zade.M. and Najafi.L. (2008). For these three earthquakes, the fault distances are less than 10 km.

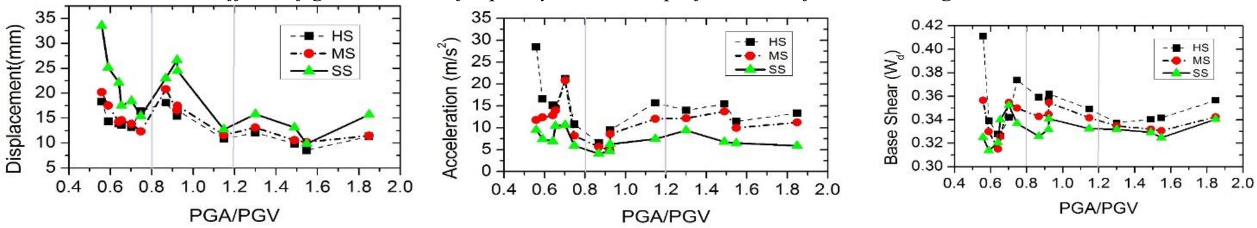
The earthquake ground motions are selected in such a way that they can be categorized into three groups according to their PGA/PGV ratio. These three PGA/PGV ranges correspond to the three categories: low (PGA/PGV < 0.8g/m/s), intermediate (0.8 g/m/s < PGA/PGV < 1.2 g/m/s) and high (1.2 g/m/s < PGA/PGV) (Zhu *et al.* 1988). This classification of low, intermediate and high PGA/PGV values ground motions subjectively corresponds to earthquake ground motions having low, moderate and high frequency contents, respectively. For earthquake ground motions that include several frequencies, the parameter 2πPGV/PGA can be interpreted as the period of vibration of an equivalent harmonic wave, thereby providing an indication of the predominant period of ground motion (Agarwal and Shrikhande 2006). Table 2 provides the classification of three categories of earthquakes and the corresponding PGA/PGV and 2πPGV/PGA values for each earthquake.

In order to choose the optimum values of the parameters of the isolation system, proper understanding of the impact of these parameters on the seismic behavior of the isolated bridge is required. In a preliminary study (Neethu and Das 2015), a rigorous parametric study was conducted to find out the parameters for getting maximum response control. The parameters selected for the present study are: (a) Isolation Time Period, T_b (2.5s), (b) Normalized Yield Strength Ratio, $F_o=Q_d/W$ (0.3), (c) Damping Ratio, ξ_b (0.10) and (d) Elastic Stiffness to Post Yield Stiffness Ratio, $\alpha=K_u/K_d$ (0.1).

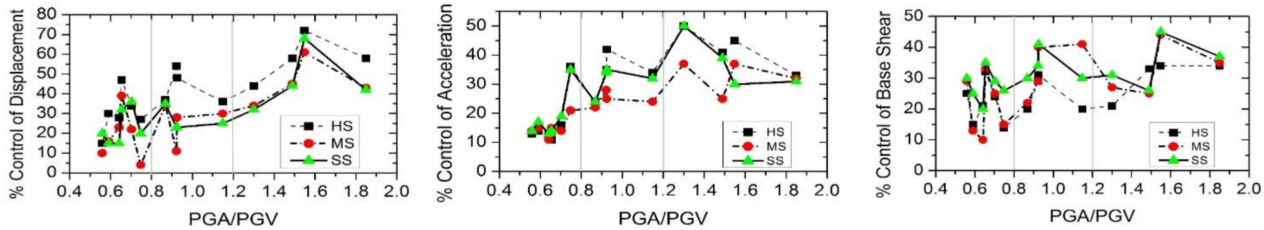
The effect of SSI is studied for the three soil conditions (viz. hard soil, medium soil and soft soil) which are detailed in Table 3.

4. Results and discussions

In the present study a three-span continuous bridge is selected from the literature (Wang *et al.* 1998) and a lumped mass model of the bridge is numerically developed using Matlab. In order to evaluate the performance of the isolator,



(a) Absolute Maximum Values of responses



(b) Percentage control of responses

Fig. 7 Variation in seismic responses of the isolated bridge with PGA/PGV ratio

the variation of the seismic responses of the bridge, namely, maximum deck displacement, maximum base shear and maximum deck acceleration, are considered.

4.1 Effect of dynamic soil structure interaction

Table 4 shows the change in the time period and frequency of the bridge on considering the effect of SSI for different soil conditions. On considering the effect of SSI, the time period of the structure increases and the values of the time history responses are found to be more for softer soils. The reason for this is that the consideration of SSI makes the structure more flexible.

Fig. 4 shows the time histories of acceleration and base shear of the isolated bridge without and with SSI for soft soil. The figure shows that there is a reduction in the time history responses of the bridge on inclusion of SSI.

The percentage controls of the responses of the isolated bridge under different earthquakes are studied for the different soil conditions. The term percentage control indicates the percentage reduction of response quantities (displacement, acceleration and base shear) of the isolated structure with respect to those of the non-isolated structure. It is calculated using the following expression

$$\frac{(Uncontrolled\ Response - Controlled\ response)}{Controlled\ Response} \times 100$$

The mean of the percentage control of responses is found to be the best representative value for the set of 14 earthquakes. Figs. 5(a) and 5(b) compare the mean, mean (+) standard deviation and mean (-) standard deviation values of the absolute maximum values and the percentage control of the responses, respectively, for fixed base (FX), hard soil (HS), medium soil (MS) and soft soil (SS) conditions. It is evident from the figures that the performance of the controller varies from one soil to the other.

The results of Fig. 5(a) show that the absolute maximum values of the displacement responses increase while the acceleration and base shear reduce when SSI is considered. The opposite phenomenon occurs for the percentage control (Fig. 5(b)), i.e., since the absolute maximum values of displacement responses increases on consideration of SSI, the percentage control or reduction of displacement is less. On the other hand, the absolute maximum values of the deck acceleration and base shear responses reduce and hence the percentage control of the responses increases.

The results explain that the percentage control of base shear and acceleration improves as the soil becomes softer; however, the control reduces in case of displacement responses. This may be attributed to the fact that as the soil becomes more and more soft, the structure-soil system becomes more and more flexible. The isolator is observed to be efficient in controlling all the response of the isolated bridge.

Fig. 6 shows the time histories of isolator displacement of the bridge (with and without SSI) subjected to Northridge earthquake. The figure shows that when SSI is considered, the bridge has a residual displacement of around 1- 4.9 mm when subjected to the earthquake. For the fixed base condition no residual displacement of the isolator is observed. However, on considering SSI, the residual displacement is found to increase on moving from hard soil to soft soil. The soft soil condition causes maximum isolator displacement (25 mm), as well as the maximum residual displacement (4.9 mm). This is due to the deformation of the underlying soil, which increases as the soil becomes softer. It is important to note that this type of residual displacement may cause a permanent increase in the size of the thermal expansion gap and may render the bridge unusable.

4.2 Effect of ground motion characteristics

In this section the effect of ground motion characteristics on the performance of the isolated bridge with different soil conditions is studied. The variation of the seismic responses of the isolated bridge with increase in PGA/PGV ratio is studied.

Fig. 7(a) shows the variation of absolute maximum values of responses of the isolated bridge with PGA/PGV ratio. The first plot in Fig. 7(a) shows the variation of the deck displacement with the PGA/PGV ratio, which is an estimate of the ground motion time period. In the figure,

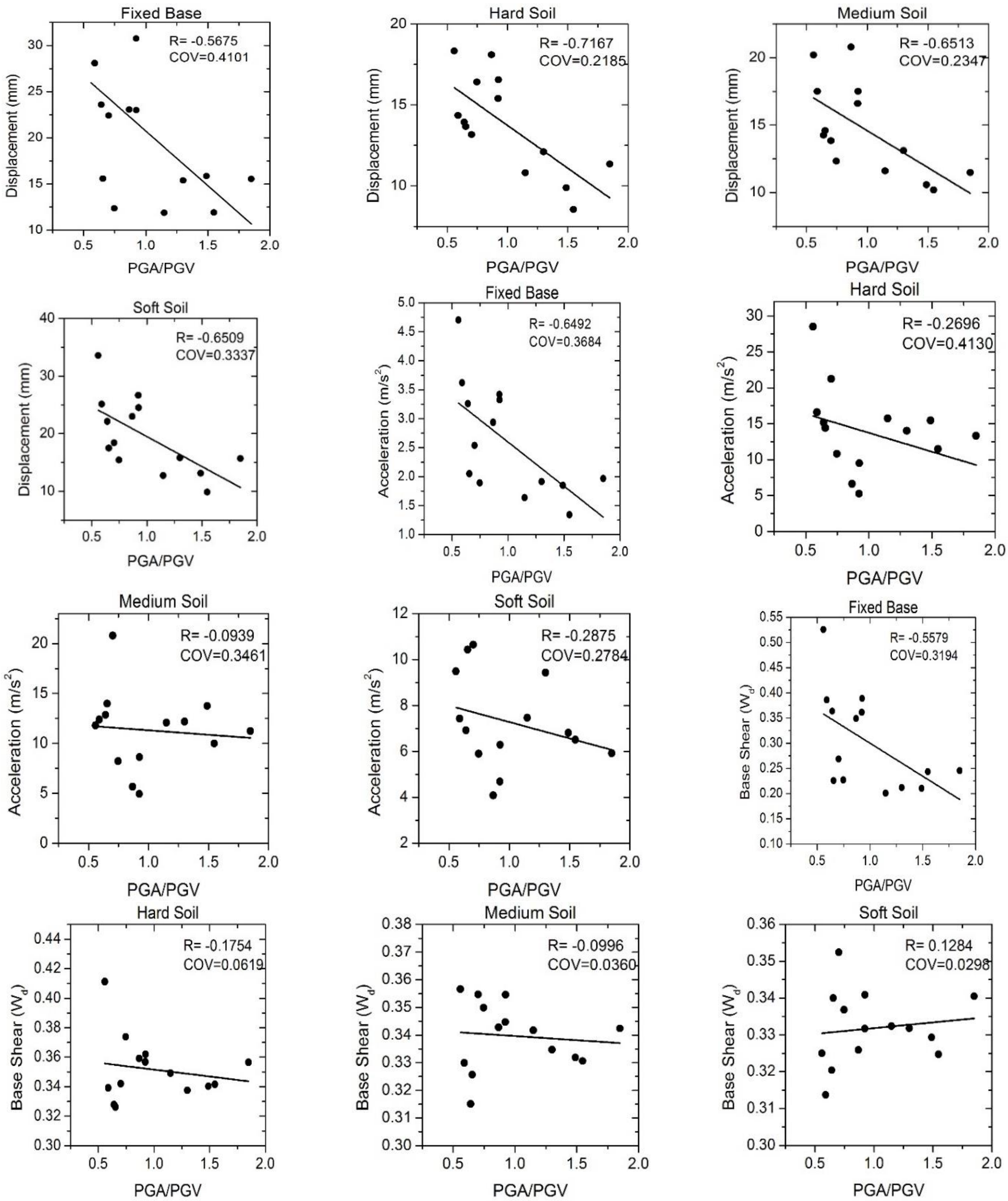


Fig. 8 Absolute maximum responses for different PGA/PGV ratios (R= correlation coefficient; COV = coefficient of variation; the straight shown is trend line)

these variations are shown separately for the different types of soil. It is observed that, for all the earthquakes, the displacement values are the highest in case of the soft soil condition as the soft soil makes the structure most flexible. Again, if the curve for the soft soil is observed, the trend is that as the PGA/PGV ratio decreases, the displacement increases. As the lower values of PGA/PGV ratio refer to lower frequency or longer period ground motions, it may be qualitatively stated the displacements increase with increase in time period of the ground motions. Therefore, the combination of isolation, soft soil and long period ground motions is very much deleterious for the structure because both the isolation and soft soil make the structure very much flexible and there is resonance effect when it is subjected to long period ground motions.

It may be noted that the frequencies of the isolated bridge falls somewhere in zones I and II (in the figure the left portion is I, the middle portion is II and right protion is III). For the bridge under consideration, all the responses are found to be sensitive to the PGA/PGV ratio up to a

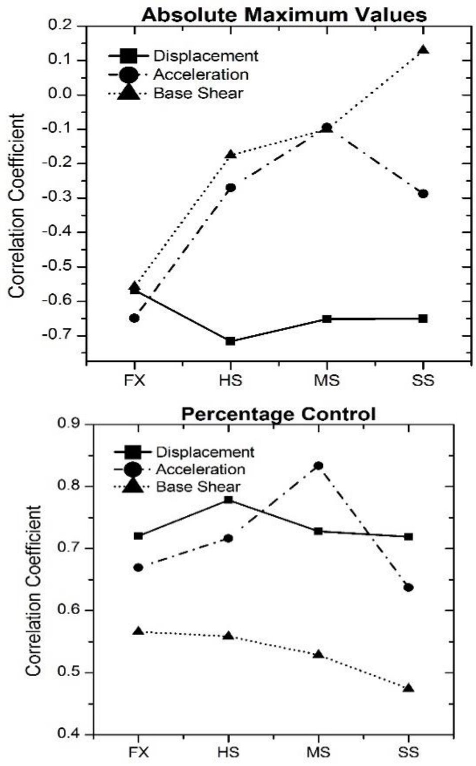
Table 5 Statistical results of correlation coefficient for different response quantities and different soil conditions

Responses	Soil Condition	R	Type of Correlation	Remarks
Displacement	FX	-0.5675	HN	High negative correlation for all soil conditions
	HS	-0.7167	HN	
	MS	-0.6513	HN	
	SS	-0.6509	HN	
Acceleration	FX	-0.6492	HN	Correlation reduces and then again increases with increase in soil stiffness. Lowest correlation for hard and medium soil.
	HS	-0.2696	MN	
	MS	-0.0939	LN	
	SS	-0.2875	MN	
Base Shear	FX	-0.5579	HN	Reversal of correlation with increase in soil softness. Negative correlation for harder soil. Positive correlation for softer soil. Lowest correlation for medium soil.
	HS	-0.1754	LN	
	MS	-0.0996	LN	
	SS	0.1284	MP	
Percentage Control of Displacement	FX	0.7204	HP	High positive correlation for all soil condition
	HS	0.7782	HP	
	MS	0.7281	HP	
	SS	0.7193	HP	
Percentage Control of Acceleration	FX	0.6695	HP	High positive correlation for all soil condition
	HS	0.7166	HP	
	MS	0.8338	HP	
	SS	0.6376	HP	
Percentage Control of Base Shear	FX	0.5662	MP	Positive Correlation for all soil conditions. Correlation increases with soil softness.
	HS	0.5589	MP	
	MS	0.7204	HP	
	SS	0.7782	HP	

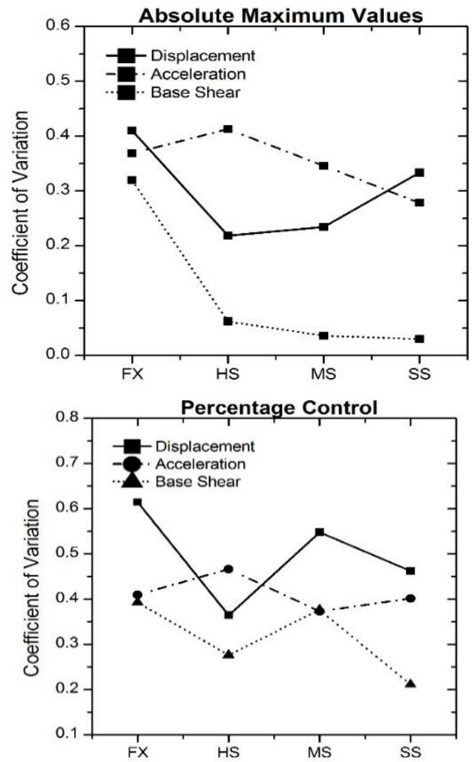
H=High; M=Medium; L=Low; N=Negative; P=Positive

value of 0.9 for all types of soil. Thereafter, the fluctuations reduce and the responses are steadier. Thus, the figures are indicative of some kind of a critical PGA/PGV value and consequently a critical value of, frequency content, which plays an important role as far as sensitivity is concerned. In the present study, no earthquake time history, with PGA/PGV equal to 1.0 was available. If available, this "critical value" could have been PGA/PGV=1.0, i.e., where PGA is numerically equal to PGV. The responses are also observed to be higher when PGA/PGV ratio (or in other words, frequency content) is less than the 'critical value'. The figure shows that with increase in softness of the soil, displacement increases but acceleration and base shear reduce.

On studying the variation of percentage control of responses (Fig. 7(b)), it is observed that the isolator performs most effectively in zone III (PGA/PGV>1.2), which corresponds to earthquake ground excitation with high frequency content. As in the case of absolute maximum values of responses, the performance of the isolator in terms of percentage reduction of responses also is more sensitive up to the 'critical value' of PGA/PGV or frequency content. From the figures, though it becomes evident that SSI has great influence on the response



(a) correlation coefficient



(b) coefficient of variance

Fig. 9 Statistical measurements for different soil conditions

reduction by the isolator, no particular conclusion can be derived regarding the effect of the soil condition on the isolator performance. It is found to vary with the frequency content of the earthquake excitation. However, the effectiveness of LRB isolator seems to enhance with

Table 6 Statistical results of coefficient of variation for different response quantities and different soil conditions

Responses	Soil Condition	COV	Type of Correlation	Remarks
Displacement	FX	0.4101	HV	Greatest dispersion for infinitely hard (fixed base condition) soil and soft soil. Lower dispersion for hard and medium soil.
	HS	0.2185	MV	
	MS	0.2347	MV	
	SS	0.3337	HV	
Acceleration	FX	0.3684	HV	In an overall sense, dispersion decreases with increase in soil softness.
	HS	0.4130	HV	
	MS	0.3461	HV	
	SS	0.2784	MV	
Base Shear	FX	0.3194	HV	Greatest dispersion for infinitely hard soil. Very low dispersion for hard, medium and soft soil.
	HS	0.0619	LV	
	MS	0.0360	LV	
	SS	0.0298	LV	
Percentage Control of Displacement	FX	0.6147	HV	High dispersion for all soil condition
	HS	0.3641	HV	
	MS	0.5482	HV	
	SS	0.4621	HV	
Percentage Control of Acceleration	FX	0.4099	HV	High dispersion for all soil condition
	HS	0.4665	HV	
	MS	0.3732	HV	
	SS	0.4015	HV	
Percentage Control of Base Shear	FX	0.3927	HV	Fluctuations in nature of dispersion.
	HS	0.2762	MV	
	MS	0.3767	HV	
	SS	0.2110	MV	

H=High; M=Medium; L=Low; N=Negative; P=Positive

increase in earthquake frequency content.

The scatter plots in Fig. 8 show the dispersion characteristics of the peak response values with respect to PGA/PGV ratio. In order to study the effect of the PGA/PGV ratio on the responses of the bridge, statistical analyses are carried out and the corresponding correlation coefficients (R) and coefficients of variation (COV) are also shown in the figure. The higher the coefficient of variation, the greater the level of dispersion around the mean. The highest values of coefficient of variation is observed for the acceleration responses. The fact that the coefficient of variation for the acceleration are slightly larger than those for displacement and base shear may be ascribed to the larger sensitiveness of the acceleration responses to the variation in PGA/PGV ratio. The correlation coefficient R measures the strength and direction of the linear relationship between two variables on a scatter plot. It is clearly observed from the values of R that almost all the responses have a negative correlation with PGA/PGV ratio which shows that the responses decrease with increase in the ratio.

It is observed from Table 5 and Fig. 9(a) that the responses (absolute maximum values) have negative correlations with frequency content (PGA/PGV ratio), i.e., the responses decrease with increase in frequency content. The decrease rate is very high for displacement and moderate to low for acceleration and base shear. Acceleration values decrease with frequency content for hard soil, but as the soil becomes softer, the decrease rate reduces and ultimately for soft soil, acceleration increases with frequency content.

The performance of isolator (in terms of percentage control of response) has a positive correlation with frequency content, i.e., control efficiencies increase with frequency content for all soil types. The increase in isolator effectiveness with frequency content is more predominant for displacement and acceleration and less for base shear.

Table 6 and Fig. 9(b) show that greater scatter or dispersion about mean is observed for displacement and acceleration (absolute maximum values) than for base shear. Thus, displacement and acceleration are more sensitive to frequency content of earthquake ground motion (in terms of PGA/PGV ratio) than base shear. Control effectiveness of isolator (in terms of percentage control of responses) is very sensitive to ground motion frequency content, the sensitivity in case of base shear reduction being a little less than the other two response quantities.

5. Conclusions

The present study investigates the effect of SSI on the seismic responses of a multi-span isolated bridge. In order to study the effect of earthquake ground motion characteristics on the responses of the bridge as well as on the performance of the isolator, statistical analyses are conducted by considering 14 earthquakes. The earthquake ground motions are selected in such a way that they can be categorized into three frequency groups according to their PGA/PGV ratio. The performance of the isolated bridge is evaluated for hard, medium and soft soil condition. The isolator is effective in controlling all the responses of the bridge. However, the effectiveness of the isolator degenerates when SSI effect is included, which suggests that when the effect of SSI is considered, the efficiency of the isolator in terms of the percentage control or percentage reduction of the seismic responses (i.e., the reduction of responses of the isolated structure with respect to those of the non-isolated structure) decreases and the percentage control value becomes lesser than that obtained without considering the effect of SSI. This occurs because of the deleterious effect of the combination of isolation and SSI which makes the structure more flexible.

On considering SSI, both the isolator displacement and the residual isolator displacement are found to increase with increase in soil softness. It is important to note that this type of residual isolator displacement may cause a permanent increase in the size of the thermal expansion gap and may render the bridge unusable.

On investigating the influence of the earthquake ground motion charateristices, it is concluded that as far as the sensitivity of the responses to the PGA/PGV ratio (or in other words, frequency content) is concerned, the results indicate to a critical PGA/PGV value, at which PGA is numerically almost equal to PGV. The structural responses are observed to be much more sensitive to the PGA/PGV ratio up to the critical value; thereafter, the fluctuations reduce and sensitivity decreases. In addition to this, the responses are also observed to be higher when frequency content is less than the ‘critical value’.

The COV for acceleration responses are slightly larger than those for displacement and base shear. Therefore, among all the responses, acceleration is most sensitive to any variation in PGA/PGV ratio.

All the responses have a negative correlation with frequency content and the performance of isolator (in terms of percentage reduction of responses) has a positive correlation with frequency content of the ground motion. Thus, it is concluded that the ground motion characteristics affect the interaction between soil and structure in such a way that the responses decrease with increase in frequency content of the earthquake for all the types of soil considered; the reverse phenomenon is observed in case of the isolator performance where the control efficiencies increase with frequency content of earthquake. However, the increase in isolator effectiveness with frequency content is more predominant for displacement and acceleration and less predominant for base shear.

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