Seismic response of isolated bridges with soil–structure interaction

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Abstract

The modern transportation facilities demand that the bridges are to be constructed across the gorges that are located in seismically active areas and at the same time the site conditions compel the engineers to rest the pier foundation on soil. The purpose of this study is to assess the effects of soil–structure interaction (SSI) on the peak responses of three-span continuous deck bridge seismically isolated by the elastomeric bearings. The emphasis has been placed on gauging the significance of physical parameters that affect the response of the system and identify the circumstances under which it is necessary to include the SSI effects in the design of seismically isolated bridges. The soil surrounding the foundation of pier is modelled by frequency independent coefficients and the complete dynamic analysis is carried out in time domain using complex modal analysis method. In order to quantify the effects of SSI, the peak responses of isolated and non-isolated bridge (i.e. bridge without isolation device) are compared with the corresponding bridge ignoring these effects. A parametric study is also conducted to investigate the effects of soil flexibility and bearing parameters (such as stiffness and damping) on the response of isolated bridge system. It is observed that the soil surrounding the pier has significant effects on the response of the isolated bridges and under certain circumstances the bearing displacements at abutment locations may be underestimated if the SSI effects are not considered in the response analysis of the system.

Keywords: Bridge isolation; Elastomeric bearings; Soil–structure interaction; Effectiveness; Earthquake; System parameters

1. Introduction

Spectacular failure of bridges due to complete collapse of piers has been observed in every major seismic event. The Hyogo-ken Nanbu (Kobe) earthquake on 17 January 1995 and Gujarat earthquake of 26 January 2001 have demonstrated that the strength alone would not be sufficient for the safety of bridges during the earthquake. For past several years, the research is focused on finding out more rational and substantiated solutions for protection of bridges from severe earthquake attack. Seismic isolation is a strategy that attempts to reduce the seismic forces to or near the elastic capacity of the member, thereby eliminating or reducing inelastic deformations. The main concept in isolation is to reduce the fundamental frequency of structural vibration to a value lower than the predominant energy-containing frequencies of earthquake. The other purpose of an isolation system is to provide means of energy

dissipation, which dissipates the seismic energy transmitted to the system. The isolation device, which replaces conventional bridge bearings, decouples the bridge deck (which alone is responsible for majority of the pier base shear) from bridge substructure during earthquakes thereby significantly reducing the deck acceleration and consequently the forces transmitted to piers.

There had been several studies in the past for investigating the effectiveness of isolation devices for a seismic design of bridges. Constantinou et al. [1], Kartoum et al. [2] and Tsopelas et al. [3] conducted experimental and analytical studies on seismic response of bridges using sliding isolation system and found that such devices are quite effective for seismic isolation of bridges. Ghobarah and Ali [4] and Turkington et al. [5] showed that lead-rubber bearings are quite effective in reducing the seismic response of bridges. Saiidi et al. [6] studied the effectiveness of seismic isolators in reducing the force and displacement of the superstructure of a six-span bridge. It was shown that the use of isolators does not necessarily increase the displacement of the superstructure. Tongaonkar and Jangid [7] evaluated the performance of three span continuous deck

bridges using linear and non-linear model of elastomeric bearing to show that these bearings are quite effective in reducing the seismic response of the bridges. Kim et al. [8] studied the response of highway bridges with energy dissipating restrainers. Hwang and Sheng [9] evaluated specified effective stiffness and equivalent damping ratio for an equivalent elastic system of isolated bridges and proposed empirical formulae for calculations of the equivalent period shift and equivalent damping ratio of equivalent system.

It is to be noted that in all these studies the foundation of the bridge piers is assumed as rigid (embedded in solid rock) and there has not been any attempt to investigate the effects of SSI on the response of isolated bridges. Since the seismic isolation is based on the principle that it is possible to uncouple a structure from the damaging components of earthquake motion by a mechanism that provides flexibility and energy absorption capacity. Both these properties are modified by the flexibility of surrounding soil first, because soil-base-isolated-structure system is more flexible than the rigidly founded isolated structure; and secondly there is energy dissipation in the soil by radiation of waves and by hysteretic damping. Recent experimental and analytical studies conducted by Crouse et al. [10], Spyrakos [12,13] and Somani [11] have identified the significant role of soil-structure interaction (SSI) during seismic excitation of non-isolated bridges. It is shown that the SSI effects decrease the seismic response of bridge leading to reduction

in design cost. This is expected since the flexibility of the surrounding soil will tend to reduce the overall stiffness of the system and thus reduce its frequencies and modify the response. In all the three methods of analysis (viz. the elastic seismic response coefficient, the single mode spectral analysis, and the multi-mode spectral analysis) recommended by the AASHTO [14] specific consideration is given to soil site effects, thus recognising the importance of the local soil conditions.

In this paper, the earthquake response of three-span continuous deck bridges seismically isolated by the elastomeric bearings is investigated considering the SSI effects. The specific objectives of the study are: (i) to present a method in time domain for finding out the seismic response of isolated bridges by duly considering the SSI effects, (ii) to study the effects of flexibility of the founding strata on the responses of seismically isolated bridges, (iii) to study the effects of SSI under parametric variation; the important parameters considered are pier flexibility, period and damping of the isolation system, and (iv) to distinguish the effects of SSI of between seismically isolated and non-isolated bridges.

2. Modelling of bridges and soil medium

Consider a typical three-span continuous deck highway bridge such as shown schematically in Fig. 1.

Fig. 1. (a) General elevation of seismically isolated bridge; (b) mathematical model of seismically isolated three-spans continuous bridge with SSI.

The substructure of bridge consists of rigid abutments and reinforced concrete piers. The structure is assumed to consist of a series of line column-beam elements. For the present study, the elastomeric bearings consisting of alternate layers of rubber and steel plates are considered. Due to the presence of alternate steel plates, these bearings are very stiff in vertical direction but quite flexible in horizontal direction. The horizontal flexibility and damping characteristics of the bearing provides desired isolation effects in the system. Horizontal flexibility transmits a limited earthquake force from ground to the superstructure. On the other hand, the damping of the bearing dissipates the seismic energy thereby reducing the design displacement of the bridge. The elastomeric bearings are placed over the piers as well as the abutments at cap level and will be rigidly connected to it. The bridge deck is simply supported over the row of such bearings. The following assumptions are made for the earthquake analysis of isolated bridge under consideration:

1. Bridge superstructure and piers are assumed to remain in the elastic state during the earthquake excitation. This is a reasonable assumption as the base isolation attempts to reduce the earthquake response in such a way that the structure remains within the elastic range.
2. The deck of the bridge is straight and is supported at discrete locations along its longitudinal axis by cross diaphragms. The abutments of the bridge are assumed as rigid and the angle of skew is 0° .
3. Both superstructure and substructure are modelled as lumped mass systems divided into number of small

Table 1
Properties of various bridges

Properties	Bridge		
	Bridge-1	Bridge-2	Bridge-3
Span length (m)	3@30	3@30	3@30
Pier height (m)	10	10	10
Pier shape
Sectional area (m^2)	1.767	1.767	1.767
Moment of inertia of pier in transverse direction (m^4)	0.902	0.113	0.063
Moment of inertia of pier in longitudinal direction (m^4)	0.902	0.451	0.253
Young's modulus of elasticity (GPa)	36	36	36
Mass density (kg/m^3)	2.4×10^3	2.4×10^3	2.4×10^3
Fundamental time period of non-isolated bridge in longitudinal direction (s)	0.53	1.5	2
Fundamental time period of non-isolated bridge in transverse direction (s)	0.53	0.75	1

Table 2
Dynamic properties of different types of soil

Properties of soil	Type of soil		
	Hard	Medium	Soft
Bulk modulus, G (MPa)	3.57	0.357	0.179
Shear wave velocity, C_s (m/s)	394	134	99.6
Horizontal stiffness of soil medium, K_h (10^8 N/m)	4.29	0.429	0.214
Rocking stiffness of soil medium, K_r (10^9 N/m)	1.80	0.18	0.0899
Horizontal damping coefficient, C_h (10^7 N s/m)	1.04	0.307	0.206
Rocking damping coefficient, C_r (10^7 N s/m)	0.967	0.285	0.191

discrete segments. Each adjacent segment is connected by a node and at each node two degrees of freedom are considered (Fig. 1(b)). The mass of each segment is assumed to be distributed between the two adjacent nodes in the form of point masses.

4. Stiffness contribution of non-structural elements such as kerbs, parapet walls and wearing coat is neglected. However, their mass producing the inertial forces is considered.
5. The force-deformation behaviour of the bearing is considered to be linear.
6. The bearings provided at the piers and abutments have the same dynamic characteristics.
7. The soil supporting the pier foundation is modelled as spring and damper acting in the horizontal and rotational directions. Viscous damping is used to simulate the radiation damping in the soil, which is developed through the loss of energy emanating from the foundation in the semi-infinite soil medium.
8. The foundation is represented for all motions using a spring-dashpot-mass model with frequency-independent coefficients. The modelling of the foundation on deformable soil is performed in the same way as that of the structure and is coupled to perform a dynamic SSI analysis [15].

Table 3
Peak acceleration of the earthquake ground motions

Earthquake	Component	Bridge direction	Peak acceleration, g
El-Centro, 1940	N90E	Longitudinal	0.21
	N00E	Transverse	0.34
Northridge, 1994	Channel 1: 90°	Longitudinal	0.58
	Channel 2: 90°	Transverse	0.55
Kobe, 1995	N00E	Longitudinal	1.28
	E90W	Transverse	0.92

The system has additional degrees-of-freedom at the base of pier due to flexibility of foundation or SSI effects (refer degrees-of-freedom x_n and y_n in Fig. 1(b)). The above assumptions will lead to the mathematical model of the isolated bridge system as shown in Fig. 1(b).

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 [12,16]. The stiffness and damping coefficients of soil medium are expressed by

$$K_h = \frac{8Ga}{2-\gamma} \left(1 + \frac{1}{2\bar{H}} \right) \quad \text{for } \bar{H} > 1 \quad (1)$$

$$K_r = \frac{8Ga^3}{3(2-\gamma)} \left(1 + \frac{1}{6\bar{H}} \right) \quad \text{for } 1 < \bar{H} \leq 4 \quad (2)$$

Fig. 2. Time variation of isolator displacements, deck acceleration and pier base shear for Bridge-1 subjected to El-Centro, 1940 earthquake excitation in longitudinal direction ($T_b = 2$ s, $\xi_b = 0.125$).

$$C_h = \frac{4.6Ga^2}{(2 - \gamma)C_s} \quad (3)$$

$$C_r = \frac{0.4Ga^4}{(1 - \gamma)C_s} \quad (4)$$

where K_h and K_r represent the horizontal and rocking stiffness of soil medium, respectively; C_h and C_r are

the horizontal and rocking viscous damping coefficients for radiation soil damping, respectively; G is the soil shear modulus; C_s is the shear wave velocity for soil; a is the radius of circular footing, γ is Poisson's ratio for the soil and H is the depth of the soil stratum overlying a rigid bedrock, and $\bar{H} = H/a$. The above expressions are also valid for the limiting case of a large soil stratum, in which the term $\bar{H}$ diminishes.

Fig. 3. Time variation of isolator displacements, deck acceleration and pier base shear for Bridge-1 subjected to El-Centro, 1940 earthquake excitation in transverse direction ($T_b = 2$ s, $\xi_b = 0.125$).

3. Governing equations of motion

The equations of motion of the isolated bridge model with SSI effects (Fig. 1(b)) under two horizontal components of earthquake ground motion is expressed in the following matrix form

$$[M]\{\ddot{z}\} + [C]\{\dot{z}\} + [K]\{z\} = -[M][r]\{\ddot{z}_g\} \quad (5)$$

$$\{z\} = \{x_1, x_2, x_3, \dots, x_n, y_1, y_2, y_3, \dots, y_n\}^T \quad (6)$$

$$\{\ddot{z}_g\} = \begin{Bmatrix} \ddot{x}_g \\ \ddot{y}_g \end{Bmatrix} \quad (7)$$

where $[M]$, $[K]$ and $[C]$ represents the mass, stiffness and damping matrix, respectively, of the foundation–bridge structure system; $\{\ddot{z}\}$, $\{\dot{z}\}$ and $\{z\}$ represent structural acceleration, structural velocity and structural displacement vectors; $[r]$ is the influence coefficient matrix; $\{\ddot{z}_g\}$ is the earthquake ground acceleration vector; $\ddot{x}_g$ and $\ddot{y}_g$ represents the earthquake ground acceleration in longitudinal and transverse directions, respectively; and x_i and y_i are the displacements of the i th node of the bridge in longitudinal and transverse directions, respectively as shown in Fig. 1(b).

4. Incremental solution procedure

The four components of isolated bridge system with SSI effects (i.e. superstructure, piers, bearing and soil) have the linear force–deformation behaviour but the classical modal superposition method still cannot be used due to different damping characteristics of these components. Hence, for the present study, the response is obtained using the complex modal analysis method. In complex modal analysis the second order differential equations of the system are converted into first order differential equations. These equations are decoupled through complex eigen values and eigen vectors of the associated matrices. In order to convert the N (i.e. $N = 2n$) second order differential equations given by Eq. (5) define a vector $\{Y\}$ of the size $(2N \times 1)$ as

$$\{Y\} = \begin{Bmatrix} \{\dot{z}\} \\ \{z\} \end{Bmatrix} \quad (8)$$

The Eq. (5) can be expressed as $2N$ first order equations as

$$\{\dot{Y}\} + [A]\{Y\} = \{P\} \quad (9)$$

Table 4

Peak response quantities for Bridge-1 for isolated and non-isolated condition with SSI and for rigid foundation condition ($T_b = 2$ s and $\xi_b = 0.125$)

Earthquake motion	Direction	Soil type	Base shear, W_d		Deck acceleration, g		Bearing displacement (mm)	
			Non-isolated	Isolated	Non-isolated	Isolated	Abutment	Pier
El Centro, 1940	Longitudinal	Rigid	0.705	0.117	0.671	0.118	85.02	84.09
		Hard	0.619	0.111	0.669	0.119	93.92	78.61
		Medium	0.576	0.065	0.663	0.122	132.16	45.41
		Soft	0.478	0.049	0.366	0.130	155.82	26.17
	Transverse	Rigid	0.956	0.126	0.903	0.169	105.9	100.29
		Hard	0.772	0.126	0.836	0.160	107.62	98.42
		Medium	0.655	0.088	0.802	0.184	165.35	64.49
		Soft	0.465	0.064	0.638	0.195	181.98	39.73
Northridge, 1994	Longitudinal	Rigid	1.439	0.210	1.396	0.238	171.75	170.22
		Hard	1.264	0.207	0.919	0.229	178.10	151.10
		Medium	1.089	0.109	0.356	0.189	202.35	69.07
		Soft	0.639	0.072	0.336	0.174	213.39	39.11
	Transverse	Rigid	0.824	0.093	0.822	0.141	45.52	50.78
		Hard	0.473	0.090	0.518	0.141	46.08	47.17
		Medium	0.441	0.036	0.477	0.109	48.21	21.25
		Soft	0.33	0.027	0.347	0.154	50.83	12.96
Kobe, 1995	Longitudinal	Rigid	3.034	0.620	2.903	0.810	584.97	578.46
		Hard	1.347	0.573	1.268	0.769	592.11	509.17
		Medium	1.196	0.272	1.129	0.541	571.15	212.44
		Soft	0.652	0.188	1.214	0.473	578.91	116.10
	Transverse	Rigid	1.833	0.303	1.719	0.427	279.19	296.11
		Hard	1.629	0.301	1.491	0.419	280.20	260.36
		Medium	1.459	0.191	1.298	0.410	352.30	146.38
		Soft	0.614	0.133	0.836	0.383	375.94	84.90

where

$$[A] = \begin{bmatrix} [M]^{-1}[C] & [M]^{-1}[K] \\ -[I] & 0 \end{bmatrix} \quad (10)$$

and

$$P(t) = \begin{bmatrix} -[r]\{\ddot{z}_g\} \\ 0 \end{bmatrix} \quad (11)$$

Let matrix $[A]$ have a complete set of eigen values $a_1, a_2, a_3, \dots, a_j, \dots, a_{2N}$ and let $u_1, u_2, u_3, \dots, u_j, \dots, u_{2N}$ be the corresponding eigen vectors. Define matrix $[U]$ formed by the eigen vectors of matrix $[A]$ as

$$[U] = [u_1, u_2, \dots, u_{2N}] \quad (12)$$

leading to

$$[U]^{-1}[A][U] = \text{diag}[a_1, a_2, a_3, \dots, a_j, \dots, a_{2N}] \quad (13)$$

Defining the transformation

$$\{Y\} = [U]\{X\} \quad (14)$$

Substituting for $\{Y\}$ in Eq. (9) and multiplying by $[U]^{-1}$

$$\{\dot{X}\} + [D]\{X\} = \{Q\} \quad (15)$$

where $\{Q\} = [U]^{-1}\{P\}$ and $[D] = [U]^{-1}[A][U]$

Eq. (15) represents a system of $2N$ -uncoupled equations of the form

$$\frac{dX_j}{dt} + a_j X_j = Q_j(t), \quad j = 1, 2, \dots, 2N \quad (16)$$

The general solution of Eq. (15) is given by

$$X_j(t) = \exp[-a_j(t - t_0)]X_j(t_0) + \int_{t_0}^t \exp[-a_j(t - \tau)]Q_j(\tau)d\tau \quad (17)$$

where $X_j(t_0)$ is the value of X_j at time t_0 , which is known from the initial conditions of the system. Knowing the vector $\{X\}$, determine vector $\{Y\}$ from Eq. (14), which provides the displacement vector $\{z\}$ and velocity vector $\{\dot{z}\}$ of the bridge system (Eq. (8)).

5. System parameters

For the present study, three basic bridges referred as Bridge-1, -2 and -3 are considered. The superstructure of these bridges consists of three-span continuous pre-stressed concrete box girder and the substructure is in the form of reinforced concrete piers. The geometric and dynamic properties of the bridge deck and piers are given in Table 1.

Table 5

Peak response quantities for Bridge-2 for isolated and non-isolated condition with SSI and for rigid foundation condition ($T_b = 2$ s and $\xi_b = 0.125$)

Earthquake motion	Direction	Soil type	Base shear, W_d		Deck acceleration, g		Bearing displacement (mm)	
			Non-isolated	Isolated	Non-isolated	Isolated	Abutment	Pier
El Centro, 1940	Longitudinal	Rigid	0.207	0.106	0.196	0.121	89.73	82.60
		Hard	0.178	0.089	0.191	0.144	128.31	78.91
		Medium	0.093	0.059	0.12	0.1228	140.32	40.74
		Soft	0.076	0.047	0.109	0.1243	159.90	24.95
	Transverse	Rigid	0.656	0.126	0.623	0.202	105.64	99.14
		Hard	0.381	0.119	0.318	0.196	114.55	100.17
		Medium	0.121	0.086	0.31	0.185	167.42	63.90
		Soft	0.083	0.064	0.308	0.159	183.02	39.49
Northridge, 1994	Longitudinal	Rigid	0.609	0.193	0.585	0.239	180.16	165.41
		Hard	0.281	0.164	0.459	0.234	205.62	128.97
		Medium	0.191	0.093	0.363	0.190	209.92	62.67
		Soft	0.152	0.068	0.326	0.175	219.76	35.61
	Transverse	Rigid	0.305	0.095	0.295	0.180	45.63	50.69
		Hard	0.205	0.106	0.241	0.161	49.93	47.93
		Medium	0.078	0.036	0.238	0.155	48.38	21.15
		Soft	0.056	0.026	0.212	0.123	51.12	12.97
Kobe, 1995	Longitudinal	Rigid	1.778	0.599	1.717	0.812	606.16	556.56
		Hard	0.654	0.461	1.523	0.744	657.10	410.84
		Medium	0.415	0.236	1.367	0.534	589.49	189.03
		Soft	0.336	0.173	1.218	0.478	591.47	111.19
	Transverse	Rigid	3.585	0.302	3.412	0.488	295.32	281.16
		Hard	2.233	0.305	3.159	0.433	307.59	262.58
		Medium	0.569	0.188	2.277	0.426	379.23	187.87
		Soft	0.353	0.132	2.135	0.382	432.66	86.10

Table 6

Peak response quantities for Bridge-3 for isolated and non-isolated condition with SSI and for rigid foundation condition ($T_b = 2$ s and $\xi_b = 0.125$)

Earthquake motion	Direction	Soil type	Base shear, W_d		Deck acceleration, g		Bearing displacement (mm)	
			Non-isolated	Isolated	Non-isolated	Isolated	Abutment	Pier
El Centro, 1940	Longitudinal	Rigid	0.212	0.101	0.201	0.155	93.55	81.07
		Hard	0.138	0.085	0.185	0.126	144.48	74.94
		Medium	0.084	0.055	0.14	0.124	146.20	37.59
		Soft	0.069	0.045	0.134	0.123	163.07	23.73
	Transverse	Rigid	0.576	0.126	0.55	0.205	105.21	100.26
		Hard	0.372	0.114	0.402	0.197	119.49	98.07
		Medium	0.125	0.084	0.299	0.191	170.38	62.87
		Soft	0.080	0.063	0.282	0.160	184.53	39.02
Northridge, 1994	Longitudinal	Rigid	0.277	0.185	0.271	0.240	185.46	159.95
		Hard	0.251	0.142	0.206	0.228	213.25	113.17
		Medium	0.169	0.084	0.187	0.189	214.92	57.32
		Soft	0.131	0.065	0.154	0.176	222.86	33.75
	Transverse	Rigid	0.264	0.094	0.277	0.189	50.59	45.79
		Hard	0.162	0.094	0.249	0.163	48.66	46.11
		Medium	0.070	0.035	0.301	0.157	51.16	20.83
		Soft	0.052	0.026	0.296	0.122	51.99	12.86
Kobe, 1995	Longitudinal	Rigid	0.901	0.809	0.896	0.700	620.86	535.91
		Hard	0.503	0.377	0.627	0.578	624.02	348.58
		Medium	0.353	0.210	0.573	0.529	638.49	170.33
		Soft	0.329	0.161	0.461	0.479	655.86	106.02
	Transverse	Rigid	2.720	0.302	2.606	0.497	293.88	284.02
		Hard	2.119	0.298	2.016	0.437	320.11	256.32
		Medium	0.571	0.183	1.46	0.426	360.65	142.55
		Soft	0.359	0.131	1.263	0.385	382.20	85.63

Fig. 4. Spectra showing variation of response of seismically isolated Bridge-1 with bearing period under El-Centro, 1940 earthquake excitation (soil type: medium and $\xi_b = 0.125$).

Fig. 5. Spectra showing variation of response of seismically isolated Bridge-1 with bearing period to Northridge, 1994 earthquake excitation (soil type: medium and $\xi_b = 0.125$).

Fig. 6. Spectra showing variation of response of seismically isolated Bridge-1 with bearing period to Kobe, 1995 earthquake excitation (soil type: medium and $\xi_b = 0.125$).

Fig. 7. Spectra showing variation of response of seismically isolated Bridge-1 with damping ratio of bearing to El-Centro, 1940 earthquake excitation (soil type: medium and $T_b = 2$ s).

Fig. 8. Spectra showing variation of response of seismically isolated Bridge-1 with damping ratio of bearing to Northridge, 1994 earthquake excitation (soil type: medium and $T_b = 2$ s).

The piers of Bridge-1 are solid circular (same time period of pier in both the directions). However, to study the effect of flexibility of pier on the response, the piers for Bridge-2 and -3 are considered oblong (i.e. wall type piers). For the sake of simplicity, the section properties of the piers are kept the same throughout the pier height. Also, for all the bridges superstructure is kept identical. Thus, the mass ratio (i.e. ratio of mass of deck to mass of pier) remains same for all the bridges. The fundamental time period of Bridge-1 is 0.53 s in both directions. The fundamental time period of Bridge-2 and -3 in transverse direction is 0.75 and 1 s, respectively. The fundamental time period of the bridge-2 and -3 in longitudinal direction is twice of the corresponding time period of the bridge in the transverse direction. Since the Bridge-1 is stiffest and the Bridge-3 is most flexible, the comparison of the response of these bridges will be useful in studying the effects of bridge flexibility on the response of seismically isolated bridges. The damping in the deck and piers is taken as 5% of the critical in all modes of vibration. The number of elements considered in bridge deck and piers are 7 and 5, respectively.

The isolation system is characterized by the parameters namely: the lateral stiffness (k_b) and the damping constant (c_b) as shown in Fig. 1(b). The elastomeric bearings for the above bridges are designed to provide the specified

values two parameters namely the period of isolation, T_b and the viscous damping ratio, ξ_b expressed as

$$T_b = 2\pi\sqrt{\frac{m_d}{\sum k_b}} \quad (18)$$

$$\xi_b = \frac{\sum c_b}{2m_d\omega_b} \quad (19)$$

where m_d is the mass of the bridge deck; $\sum k_b$ is the total stiffness of the bearings; $\sum c_b$ is the total viscous damping of the bearings; T_b is the isolation time period of the bearings; ξ_b the damping ratio; and $\omega_b = 2\pi/T_b$ is the isolation frequency.

The soil surrounding the pier is considered as hard, medium and soft soil. The properties of these soils such as shear wave velocity, Young's modulus and Bulk modulus, etc., given in Table 2. For the results reported herein, the Poisson's ratio is taken as 0.4.

6. Numerical study

The seismic response of isolated bridge system is investigated for the three real earthquake excitations, namely the El-Centro (1940), Northridge (1994) (recorded

Fig. 9. Spectra showing variation of response of seismically isolated Bridge-1 with damping ratio of bearing to Kobe, 1995 earthquake excitation (soil type: medium and $T_b = 2$ s).

at La County fire station) and Kobe (1995). The components N90E, channel 1 and N00E of El-Centro, Northridge and Kobe earthquakes, respectively, are applied in the longitudinal direction of the bridge (the corresponding orthogonal component is applied in the transverse direction). The El-Centro (1940) excitation is extensively used in past by many researchers whereas during the Kobe (1995) earthquake many bridges have suffered extensive damages and even total collapse, the absolute acceleration is highest

for this earthquake. The various properties of these earthquake ground motions are shown in Table 3.

The response of three bridges (Table 1 for Bridge-1, -2 and -3) including SSI is investigated under three recorded earthquake ground motions discussed earlier. The effects of SSI are studied for both isolated and non-isolated conditions. The response quantities of interest are the deck acceleration, base shear in the piers and the relative displacement of the isolation devices at abutment and pier

Fig. 10. Spectra showing variation of response of seismically isolated Bridge-1 with soil stiffness to different earthquake excitations ($T_b = 2$ s and $\xi_b = 0.125$).

locations. The base shear in the pier indicates the forces exerted in the bridge due to earthquake ground motion. On the other hand, the relative displacement of the isolation system is crucial from the design point of view of the isolation system and expansion joints. In addition, the effects of SSI on parameters that are crucial for the design of isolation system are also studied. The isolation system requires the specifications of the parameters, namely, the period of isolation, T_b and damping ratio, ξ_b

(Eqs. (18) and (19)). For the purpose of present study the T_b is varied from 1.5 to 4 s. In addition, the effect of variation of damping in isolation devices on the seismic response of bridge is also studied. The damping in isolation devices is varied from 0.075 to 0.3, keeping T_b constant. The damping in the superstructure and substructure of the bridge is considered as 5% of the critical in all modes of the vibration.

Figs. 2 and 3 show the time history response of pier base shear, deck acceleration and isolator displacements at

Fig. 11. Spectra showing variation of response of seismically isolated Bridge-2 with soil stiffness to different earthquake excitations ($T_b = 2$ s and $\xi_b = 0.125$).

abutment and pier locations for Bridge-1 under El-Centro, 1940 earthquake motion in longitudinal and transverse directions, respectively. These effects are studied for medium soil with the bearing parameters as, $T_b = 2$ s and $\xi_b = 0.125$. From these graphs it is observed that SSI influence the displacements of the bearings. In longitudinal direction, the peak bearing displacement at abutment location with and without SSI effects is found to be 84.98 and 132.16 mm, respectively, i.e. when the effects of SSI are

considered the bearing displacement increase by 55%. On the other hand, bearing displacements at pier location found to decrease from 84.09 to 45.43 mm i.e. when the effects of SSI are considered. Thus, the bearing displacement at pier location decreases whereas at abutment increases due to SSI effects. This implies that the design bearing displacements at abutment may be underestimated if the effects of SSI are ignored for the design of isolated bridge system, which can be crucial from the design point of

Fig. 12. Spectra showing variation of response of seismically isolated Bridge-3 with soil stiffness to different earthquake excitations ($T_b = 2$ s and $\xi_b = 0.125$).

view. It is also observed that there is a sizeable reduction in the pier base shear (about 45%) due to SSI effects. Thus, the flexibility of the surrounding soil tends to reduce the earthquake forces induced in the isolated bridges. Similar effects of SSI are observed in transverse direction.

The responses of the isolated and non-isolated bridges for different soil conditions and subjected to various earthquake motions are shown in Tables 4–6. The effects of SSI under Northridge, 1994 and Kobe, 1995 earthquake ground motions are similar to those observed for El-Centro, 1940 earthquake. As expected, the effects of SSI are significant for non-isolated bridges for hard soil condition. The reduction in pier base is found to be 32 and 52% in longitudinal and transverse direction; the corresponding reduction for isolated bridge is just 2.27 and 0.367%. However, it is seen that there is an increase of about 10% in abutment isolator displacements. Bearing displacement is a quantity of prime interest in the design of isolated bridges because and if it exceeds certain limits the bearing may fail resulting into the collapse of the bridge.

Figs. 4–6 shows the spectra of variation of bearing displacement, pier base shear and deck acceleration for Bridge-1 plotted against the bearing period for all the three earthquakes in both, longitudinal and transverse direction. To study the effects of flexibility of the supporting soil on the systems response these graphs are plotted for medium soil and for rigid foundation condition (i.e. ignoring SSI effects). Damping in the isolators (ξ_b) is held constant at 0.125. It is observed that, practically for all values of bearing period, the abutment bearing displacements are higher whereas pier bearing displacements are lower as compared to bridge on rigid foundation. It is also observed that for the bearing period higher than 2.5 s the spectra for both the cases shows a converging trend indicating that the effects of SSI diminishes with the increase in the flexibility of isolation bearings. Thus, the effects of SSI are more pronounced for stiff bearings and diminish with the increase in bearing period.

Effects of variation of bearing damping on the response of seismically isolated bridges with SSI are studied in Figs. 7–9 for all the three ground motions. The damping in bearing is varied from 0.075 to 0.3. As expected, the increase in damping has an overall effect of reducing all the response quantities. For Northridge and Kobe excitations, plotted ignoring the effects of SSI, a reverse trend in the variation of deck acceleration and pier base shear is observed (i.e. it first decreases attains minimum value and then increases with increase of bearing damping). This indicates that there exists an optimum value of bearing damping for which the earthquake forces in the bridge are minimum. This is similar to a well-known phenomenon that the higher damping in the isolation system can reduce displacements but it may transmit higher earthquake forces. It happens due to the fact that the higher bearing damping transmits more

acceleration into the system for the earthquake energy at higher frequencies. The effects of SSI are not much influenced by the bearing damping.

In order to study the effect of flexibility of the supporting soil on the response of the system, the behaviour of the isolated bridges is studied by varying the stiffness of soil from k_b to $20k_b$ (beyond this value of soil stiffness, bridge found to behave as if it is founded on rigid soil). The stiffness of the soil (k_s) is defined as

$$k_s = \alpha_s \sum k_b \quad (20)$$

where α_s is a stiffness factor and $\sum k_b$ is the bearing stiffness. The other parameters namely C_h , C_r , K_h and K_r are evaluated using Eqs. (1)–(4).

Figs. 10–12 show spectra of important response quantities plotted against k_s . The damping in the bearings (ξ_b) and time period of bearings (T_b) are held constant at 0.125 and 2 s, respectively. These graphs are plotted in longitudinal as well as in transverse direction for all the three ground motions. As expected the increase in soil stiffness (higher values of α_s) resulted in increase in pier base shear. The effects of variation of soil stiffness are more pronounced for Bridge-1 as compared to other two bridges, the increase in flexibility of bridge diminishes the effects of SSI. From these plots it is also observed that with the increase in the soil stiffness bearing displacements at pier location increase and those at abutment decrease. The displacement ratio (ratio of bearing displacements at pier and abutment) is lowest for Bridge-1 and highest for Bridge-3. The ratio goes on decreasing with the increase in soil stiffness and it is almost unity for rigid foundation condition. In longitudinal direction for value of $k_s = k_b$, the displacement ratio works out to 2.16, 2.4 and 2.61 for Bridge-1, -2 and -3, respectively, but it reduces to 1.24, 1.3, and 1.58 for $k_s = 10k_b$. It is observed that for values of $k_s > 10k_b$ (i.e. $\alpha_s > 10$) the displacement ratio is almost constant and the increase in pier base shear is also very insignificant. Thus, when the stiffness of soil is more than 10 times the isolator stiffness, the SSI effects may be ignored in design of the isolated bridges.

7. Conclusions

Effects of SSI on earthquake response of isolated and non-isolated bridges are investigated under deterministic ground motion. The soil medium is modelled by frequency independent coefficients of soil stiffness and damping. The circumstances under which SSI need to be included in the analysis of isolated bridges are identified and it is shown that consideration of SSI in the analysis will result in enhancement of safety and reduction in design costs. From trends of results of the present study following conclusions may be drawn

- (1) Considerations of SSI in the analysis of isolated bridges is warranted when the stiffness of the supporting soil medium is less than 10 times the stiffness of isolation bearings.
- (2) The SSI affects the bearing displacements at the abutment and ignoring these effects will underestimate the design displacement at abutment, which may be crucial from the design point of view.
- (3) The effects of SSI are found to be more pronounced for stiff bridges in comparison to the flexible bridges.
- (4) The effects of SSI are more pronounced for bridges with stiff isolation system in comparison to flexible system.
- (5) The variation in the damping in the isolation system does not have noticeable effects on the response of seismically isolated bridges with SSI effects.

References

- [1] Constantinou MC, Kartoum A, Reinhorn AM, Bradford P. Sliding isolation system for bridges: experimental study. *Earthquake Spectra* 1992;8:321–44.
- [2] Kartoum A, Constantinou MC, Reinhorn AM. Sliding isolation system for bridges: analytical study. *Earthquake Spectra* 1992;8: 345–72.
- [3] Tsopelas P, Constantinou MC, Kim YS, Okamoto S. Experimental study of FPS system in bridge seismic isolation. *Earthquake Engng Struct Dyn* 1996;25:65–78.
- [4] Ghobarah A, Ali HM. Seismic performance of highway bridges. *Engng Struct* 1988;10:157–66.
- [5] Turkington DH, Carr AJ, Cooke N, Moss PJ. Seismic design of bridges on lead-rubber bearings. *J Struct Engng, ASCE* 1988;115: 3000–16.
- [6] Saiidi M, Maragakis E, Griffin G. Effect of base isolation on seismic response of multi-column bridges. *Struct Engng Mech* 1999;8: 411–9.
- [7] Tongaonkar NP, Jangid RS. Earthquake response of seismically isolated bridges. *Eur Earthquake Engng* 2000;XIV:48–58.
- [8] Kim J-M, Feng MQ, Shinozuka M. Energy dissipating restrainers for highway bridges. *Soil Dyn Earthquake Engng* 2000;19:65–9.
- [9] Hwang JS, Sheng LH, Gates JH. Practical analysis of bridges on isolation bearings with bi-linear hysteresis characteristics. *Earthquake Spectra* 1994;10:706–27.
- [10] Crouse CB, Hushmand B, Martin GR. Dynamic soil structure interaction of single span bridge. *Earthquake Engng Struct Dyn* 1987;15:711–29.
- [11] Somani DR. Parametric study on soil–structure interaction of bridges with shallow foundation. *Proceedings of the Eighth World Conference on Earthquake Engineering* 1984;3:529–35.
- [12] Spyarakos CC. Assessment of SSI on the longitudinal seismic response of short span bridges. *Engng Struct* 1990;12:61–5.
- [13] Spyarakos CC. Seismic behaviour of bridge piers including soil structure interaction. *Comput Struct* 1992;43:373–84.
- [14] AASHTO, Standard specifications for highway bridges, 14th ed. Washington, DC: American Association of State Highway and Transport Officials; 1989.
- [15] Wolf JP. Spring–dashpot–mass models for foundation vibrations. *Earthquake Engng Struct Dyn* 1997;26:931–49.
- [16] Gazetas G. In: Fang H-Y, editor. *Foundation vibrations, foundation engineering handbook*, 2nd ed. ; 1991. p. 553–93.