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SEISMIC SOIL-STRUCTURE INTERACTION IN BRIDGES: DOES THE ANSWER LIE IN SOIL OR STRUCTURE?

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Abstract

Effect of soil-structure interaction (SSI) and pier-column inelasticity on the seismic response of a 4-span bridge typically used for urban elevated highways was investigated using FEM analytical models and sub-structuring scheme. The bridge system was designed in a moderate seismic zone for five soil conditions with deep foundations and five rock profiles with shallow foundations. Impedances of both types of foundation systems were computed by methods available in the literature. Non-linear behavior of the reinforced concrete pier column was modeled for material and geometric non-linearities and incorporated in the analysis scheme by equivalent linear model. Foundation impedance was modeled as Winkler springs in six directions. FEM model of the bridges was subjected to fifteen actual ground motions varying in PGA from 0.01g to 0.64g. Results of more than 300 FEM analysis cases were evaluated to delineate the relative contribution of the elastic part, SSI and pier column non-linearity to bridge displacement, column shear force and modal parameters. Contribution to all three parameters was the largest for the elastic part (60% to 95%) followed by pier column inelasticity (4% to 25%) and SSI (1% to 15%) for various combinations of investigated parameters. Contribution of SSI to bridge displacement and column shear force was found to be significant only in bridges founded on weaker rock (Rock Class IV and V) and weaker soil (Site Class III). Contribution of pier column inelasticity was significant for more than 86% of the bridge analysis cases. It was thus concluded that pier column inelasticity contributed more significantly to the bridge design parameters than SSI in the class of bridges examined in the study.

Keywords: soil-structure interaction; bridge pier; pile foundation; seismic effect; reinforced concrete;

1. Introduction

Soil Structure Interaction (SSI) is interdisciplinary (geotechnical and structures) and tends to be misunderstood by both sides when it comes to assessing the overall behavior of a bridge system. The geotechnical side had focused mostly on elaborate linear/nonlinear/equivalent linear modeling of the soil and foundation system with a critical assumption of elastic structural components (concrete piers, abutments and super-structure elements). Whereas the reverse is true for the structural studies that included non-linearity in the structural elements while ignored the SSI [1]. Few examples [2 – 4] incorporated non-linearity in bridge sub-structure components while conducting SSI studies and concluded that incorporation of SSI in certain classes of bridges can have adverse impact on seismic demand for inelastic sub-structure components. Some other studies [5, 6] included a wide range of properties for the soil/rock-foundation system and the pier columns in order to 'generalize' the effect of SSI on such structures. This quest for generalization mostly resulted in wide deviations from realistic bridge designs and therefore arrived at some results which were theoretically plausible but of little practical significance.

Proper analysis of the field-recorded data may be the best way of understanding the SSI effects in bridges. However, in light of the very limited investigations that have focused on identifying SSI in bridges from field investigations or from analysis of recorded seismic motions [7 – 9], the usefulness of analytical studies cannot be under-estimated as these provide relatively low-cost solutions and cover a wide range of parameters that cannot be practically covered in field investigations. Such studies also provide valuable clues for augmenting and improving the field observations and instrumentation as well. Therefore, an effort was made in this study to investigate the seismic performance of a class of bridge which was designed based on the current AASHTO [10] design practices for foundation as well as bridge sub-structure components.

The purpose of the current study was to investigate the relative importance of SSI and pier column non-linearity on the seismic response of simple, straight and non-skew multi span bridges of medium span length using equivalent linear properties of concrete columns and soil-foundation system. Use of equivalent linear model was to reduce the computational effort while not significantly sacrificing accuracy as demonstrated for analytical studies [11] and back-calculated system identified parameters [8]. Finite Element Method (FEM) analytical studies were undertaken with varying rock/soil-foundation system properties and including non-linear behavior of pier columns to answer the question whether SSI in the bridge class studied herein is more influenced by the contribution of soil or by the structure?

2. Bridge description

This study focused on ordinary and standard bridges which are defined as "*those using normal weight concrete, with span lengths less than 90 m, and located in areas with no liquefiable soil*" [12]. A multi-span continuous bridge with medium span length (30 m), which is extensively used for long elevated urban viaducts as shown in Fig. 1, was investigated in this study. An interior part of the bridge was selected such that the influence of abutments could be safely ignored.

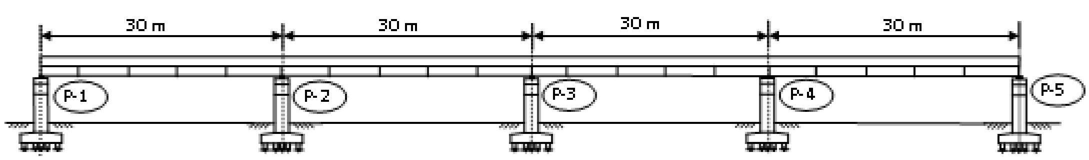


Fig. 1 - Longitudinal elevation of the bridge

The bridge comprised of AASHTO Type V prestressed concrete girder superstructure and 11 m tall two-column reinforced concrete bent as sub-structure. Foundations consisted of spread footings for AASHTO site classes A to C while pile foundations were used in AASHTO site classes C and D. Site classes A and B in the AASHTO code [10] are rock profiles, while class C is a 'soil rock' and class D represents the normal soil profiles. Rock site classes in the AASHTO code were further divided into five rock classes in this study based on the CSIR classification [13]. Similarly, the soil sites represented by classes C and D were further divided into five soil profiles to fill in the gaps in the wide range of V_s in these profiles. The rock and soil profiles used in the study are summarized in Table 1 and Table 2 respectively along with their salient mechanical properties.

Table 1: Rock profiles and their mechanical properties

AASHTO Site Class	Rock Class	Rock Description	V_s	ρ	ν	G	q_a
			m/s	kg/m³		GPa	kN/m²
A	I	Very good	3350	2920	0.15	32.60	3816
	II	Good	2250	2610	0.20	13.18	2051
B	III	Fair	1300	2320	0.25	4.00	839
	IV	Poor	780	2090	0.30	1.22	385
C	V	Very Poor	600	2060	0.35	0.74	215

Table 2: Soil profiles and their mechanical properties

AASHTO Site Class	Soil Profile	V_s (m/s)	ρ (kg/m3)	ν	G (MPa)	β (%)
C	II_{upper}	600	2060	0.35	741	3
	II_{avg}	475	2020	0.35	456	4
D	III_{upper}	350	1980	0.40	243	5
	III_{avg}	275	1900	0.40	144	7
	III_{low}	175	1850	0.42	57	8

The 2 m square reinforced concrete pier columns, group of eleven 1 m diameter pile foundations (for soil profiles), spread footings of various plan dimensions (for rock profiles) and superstructure components were designed according to the AASHTO code [10] stipulated load combinations of dead, live and seismic loads. The bridge was designed for HL-93 live load and was located in a moderate seismic zone with PGA of 0.2g.

3. Methodology

This study utilized a sub-structuring method in which the SSI problem was split into two parts. First, frequency-independent dynamic impedance of shallow footings in rock (Table 3) and pile foundations in the soil profiles (Table 4) was computed in the lateral, vertical and rocking modes by the procedures available in the literature [14, 15]. These impedances were incorporated in the 3-D FEM analytical model of the bridge as Winkler springs at the foundation level. Second, the superstructure was modeled with beam and plate finite elements and seismic ground motions were applied as acceleration time-history at the foundation nodes. Use of frequency independent foundation impedance for seismic design of bridges is a well established and accepted procedure [16, 17].

The bridge system was modelled in a commercially available FEM package utilizing equivalent linear properties of non-linear soil-foundation system and sub-structure components. The FEM bridge model was subjected to a suite of fifteen actual ground motions with PGA varying between 0.036g and 0.64g [18]. Acceleration spectra of these ground motions are presented in Fig. 2.

Member forces and displacements in various components of the bridge system were computed along with the modal properties of the bridge for each seismic ground motion simulation. Stiffness of the pier columns was adjusted for the computed displacement values in the next iteration to account for inelasticity in the pier column as determined from the load-deflection curve shown in Fig. 3.

Table 3: Shallow foundation stiffness in various modes for the selected rock profiles

Rock Description	Foundation size (L x B x D) (m x m x m)	K_v x10^7 (kN/m)	K_{Hx} x10^7 (kN/m)	K_{Hz} x10^7 (kN/m)	K_{Rx} x10^8 (kN-m/rad)	K_{Rz} x10^9 (kN-m/rad)
Very good	12.6 x 3.2 x 1.75	61.9	61.7	56.6	19.4	15.7
Good	12.6 x 3.4 x 1.75	27.1	26.0	23.8	9.43	6.99
Fair	12.8 x 4.0 x 2.00	9.29	8.49	7.79	4.28	2.56
Poor	13.2 x 5.0 x 2.25	3.25	2.82	2.62	2.25	0.99
Very Poor	14.8 x 6.0 x 2.50	2.47	1.97	1.83	2.33	0.95

Table 4: Pile group stiffness in various modes for the selected soil profiles

Soil Profile	Pile length (m)	K_v x10^6 (kN/m)	K_{Hx} x10^6 (kN/m)	K_{Hz} x10^6 (kN/m)	K_{Rx} x10^8 (kN-m/rad)	K_{Rz} x10^8 (kN-m/rad)
II_{upper}	22	29.15	11.23	10.97	13.28	8.22
II_{avg}	25	19.54	7.64	7.40	8.94	5.55
III_{upper}	30	11.78	4.78	4.54	5.43	3.39
III_{avg}	32	7.31	3.16	3.01	3.93	2.14
III_{low}	35	3.07	1.53	1.47	1.47	0.95

4. Finite Element Method (FEM) model

The analytical model of the bridge was made in FEM package STAAD [19] and is depicted in Fig. 4. The bridge super-structure was modeled by two different finite elements. Beam elements with six degrees of freedom were used for modeling the girders and diaphragms, while four node plate elements were employed for the bridge deck. Full composite action was assumed between the girders and the deck slab in the FEM model. Sub-structure consisted of pier columns, pier cap and pile cap; all of which were modeled by beam elements. Foundation-soil system was modeled by Winkler springs. The pile cap which connects the two columns was modeled as a rigid beam to accurately estimate the effect of seismic forces transferred to piles and soil.

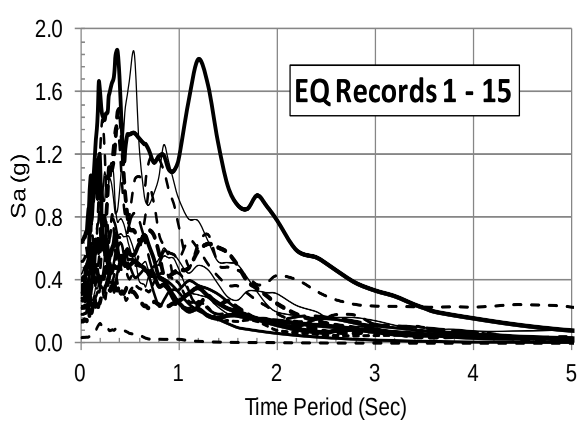


Fig. 2 - Spectra of used seismic motions

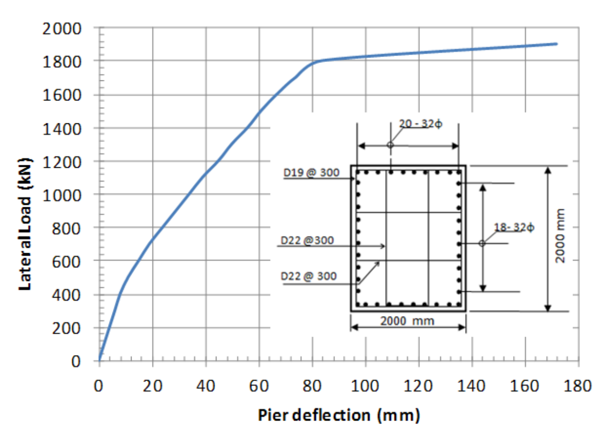


Fig. 3 - Load deflection curve for pier

Non-linear behavior of the pier columns was captured by employing equivalent linear stiffness of the pier columns computed from the load-displacement (P-Δ) curve of the columns as shown in Fig. 3. P-Δ curve of the column was computed by integration of the moment-curvature (M-φ) relationship of the column section that was found by conducting a layer-by-layer strain compatibility analysis of the cross section using non-linear stress-strain model for 27.6 MPa concrete [20] and standard model for 420 MPa yield strength steel. Reduction in pier column stiffness was computed for the maximum elastic displacement determined from the case of linear elastic pier columns.

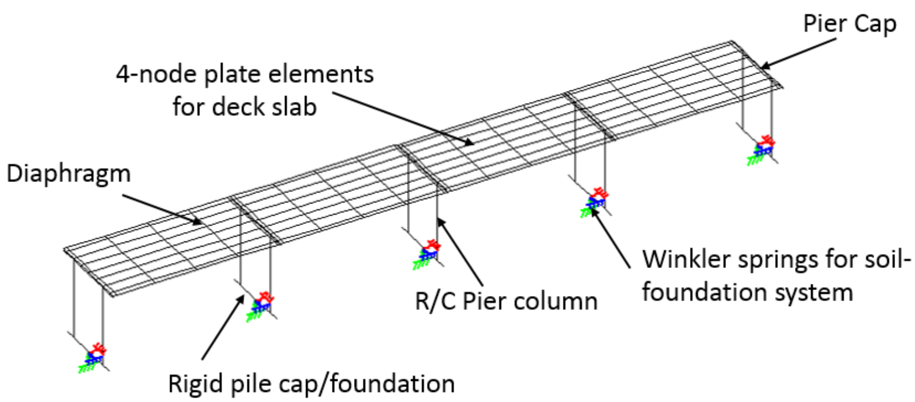


Fig. 4. 3D FEM model of the bridge

FEM analysis of the bridge model was conducted for the suite of fifteen selected ground motions for two basic conditions, i.e. linear elastic pier columns and non-linear pier columns. For each of the two pier column conditions, eleven boundary conditions were investigated. One boundary condition was with a fixed foundation, i.e. no SSI and the remaining ten represented boundary conditions with SSI with varying values of the rock/soil-foundation springs corresponding to the five rock and five soil profiles as outlined in Sections 2 and 3. Altogether 330 FEM analyses were conducted for all cases of pier elasticity, rock/soil-foundation systems and ground motion records. Results of these analyses related to the design parameters of the bridge are presented in the next section.

5. Various components of bridge deck displacement and column shear

5.1 Methodology of computations

The total bridge deck displacement and column shear consisted of three components: (i) elastic part, (ii) contribution of SSI and (iii) effect of pier inelasticity. These three components were inferred from the results of the FEM analysis as follows:

Taking example of total bridge deck displacement, the three components were computed as:

(i) Elastic deformation:
Elastic deformation was taken as the deformation found from the fixed based model with elastic pier.

$$\delta_{elastic} = \delta_{fixed\ base}^{elastic\ pier} \tag{1}$$

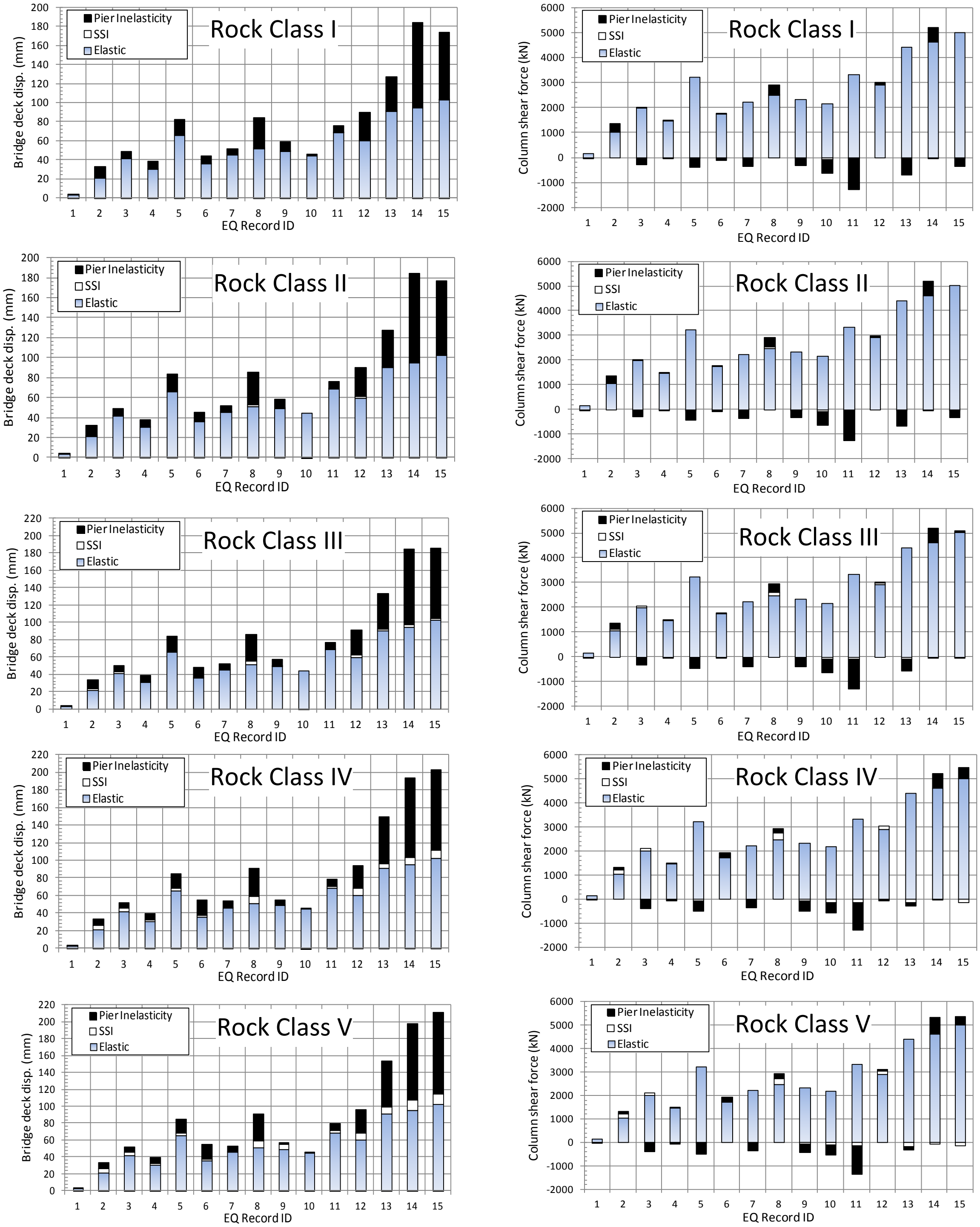
(ii) Contribution of SSI:
Contribution of SSI to bridge displacement was computed from the difference between displacements found from the SSI case with elastic pier and the fixed base model with elastic pier.

$$\delta_{SSI} = \delta_{SSI}^{elastic\ pier} - \delta_{fixed\ base}^{elastic\ pier} \tag{2}$$

(iii) Effect of pier inelasticity:
Effect of pier inelasticity was inferred from the results of SSI case with inelastic pier and SSI case with elastic pier.

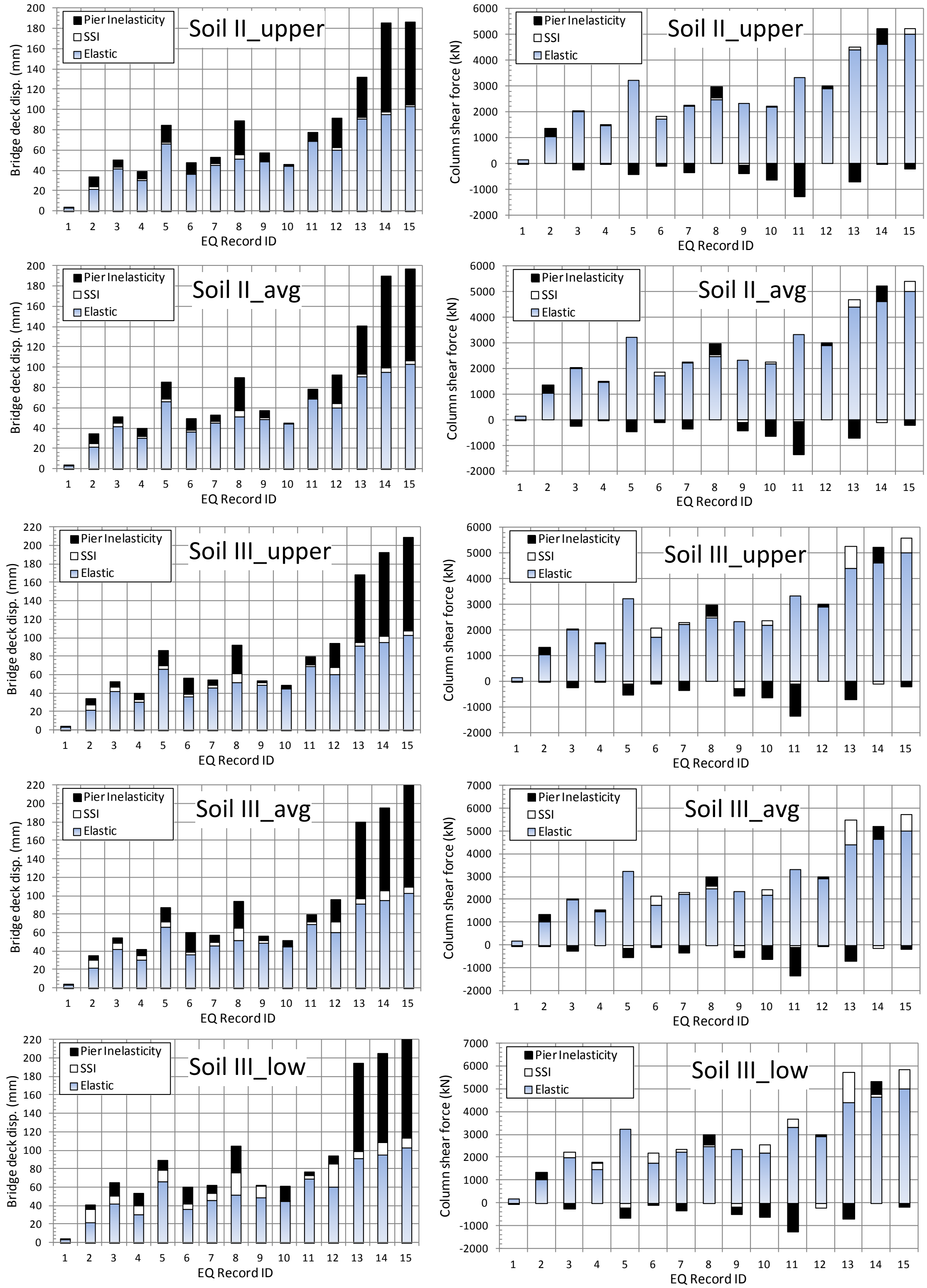
$$\delta_{pier\ inelasticity} = \delta_{SSI}^{inelastic\ pier} - \delta_{SSI}^{elastic\ pier} \tag{3}$$

Contribution of these components to the total displacement and total column shear force is summarized in Fig. 4 for the rock profile bridges and in Fig. 5 for the soil profile bridges.



(a) Bridge deck displacement (b) Column shear force

Fig. 4 - Contribution of elastic , SSI and pier inelasticity components to (a) total bridge displacement and (b) column shear force for bridge in various rock profiles



(a) Bridge deck displacement (b) Column shear force

Fig. 5 - Contribution of elastic, SSI and pier inelasticity components to (a) bridge deck displacement and (b) column shear force for bridge in various soil profiles

5.1.1. Observations and discussions

The following observations were made based on analysis of Figs. 4 and 5:

(a) Bridge displacement (Figs. 4a and 5a)

i. Contribution of the elastic component of deformation was the largest varying from 100% for small ground motions to 70- 80% for design ground motions and reducing to about 50% for extreme ground motions for both rock and soil profiles.

ii. The contribution of pier inelasticity component to overall displacement varied from 20% to 50% for most of the ground motions in both rock and soil profiles.

iii. SSI had a maximum contribution of 10% in a few of the ground motions for relatively weaker rock profiles. Its contribution was less than 5% for majority of the cases in rock profiles. Whereas, SSI had a maximum contribution of 30% in a few of the ground motions for relatively weaker soil cases (i.e. Type III).

iv. Effect of pier inelasticity was found to be more dominant than SSI to bridge displacement in both rock and soil profiles for most of the ground motions.

(b) Pier Column shear force (Figs. 4b & 5b)

i. Elastic component of column seismic shear force varied between 70% to 95% for various rock/soil profiles and ground motions, making it the largest component in total shear force.

ii. SSI and pier inelasticity decreased the elastic shear force for nearly 50% and 67% of the ground motions for bridges in rock and soil profiles respectively and increased it in rest of the cases. This observation was contrary to the bridge displacement which increased for almost all ground motions when SSI and pier inelasticity was included. This difference could be due the fact the first modal period of the bridge was within the dominant period range of earthquakes 1-6 and 13-15 for both elastic and inelastic bridge pier cases [21]. Whereas, the first modal period is outside the dominant period of earthquakes 7-12 for the inelastic pier cases which could be the possible reason for deamplification in seismic shear force for these ground motions.

iii. Contribution of SSI in increasing or decreasing the fixed base column shear force was observed to range from 3% to 20%.

iv. Negative contribution to shear force from SSI or pier inelasticity in Figs. 4b & 5b means a reduction in the magnitude of the shear force and hence it is a beneficial effect.

v. Pier inelasticity had a contribution in increasing or decreasing the fixed base column shear force ranging from 3% to 27%.

vi. Delineating the relative significance of SSI and pier column inelasticity on pier column shear force is not straightforward from Figs. 4b and 5b. Some cases showed SSI to be more significant than pier inelasticity and vice versa and in some cases both effects were equally present. Further examination of the results was conducted in Section 5.2 to get a better insight on relative significance of SSI and pier column inelasticity for shear force in pier columns.

5.2 Relative significance of pier inelasticity and SSI for bridge response parameters

In order to answer the question when SSI or pier inelasticity can be neglected for computation of bridge displacement and seismic shear force in the pier column, it was assumed that the analyses cases that included SSI and pier column inelasticity resulted in the ‘most accurate’ values of the bridge response parameters. Results of other three cases, viz. (a) fixed base with inelastic pier columns, (b) SSI with elastic pier columns and (c) fixed base with inelastic pier column were compared with the ‘most accurate’ case by plotting the following ratios for bridge displacement and pier column shear force:

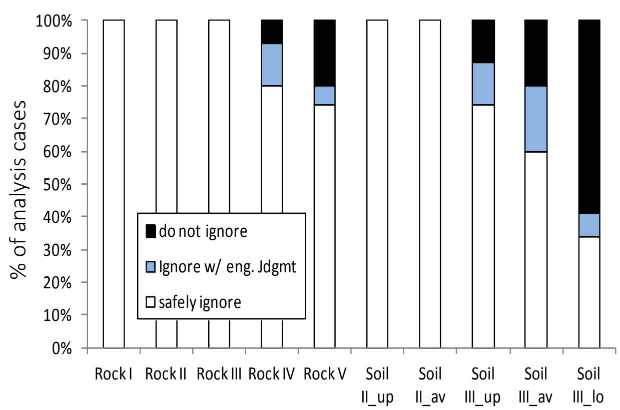
$$\frac{\delta_{SSI}^{inelastic\ pier}}{\delta_{fixed\ base}^{inelastic\ pier}} \ \&\ \frac{V_{SSI}^{inelastic\ pier}}{V_{fixed\ base}^{inelastic\ pier}} \qquad (4)$$

$$\frac{\delta_{SSI}^{inelastic\ pier}}{\delta_{SSI}^{elastic\ pier}} \ \&\ \frac{V_{SSI}^{inelastic\ pier}}{V_{SSI}^{elastic\ pier}} \qquad (5)$$

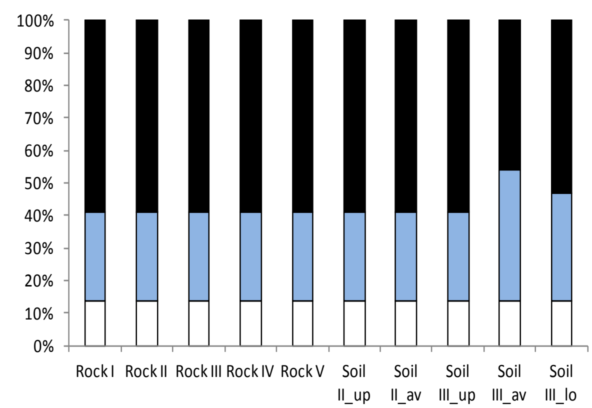
$$\frac{\delta_{SSI}^{inelastic\ pier}}{\delta_{fixed\ base}^{elastic\ pier}} \ \&\ \frac{V_{SSI}^{inelastic\ pier}}{V_{fixed\ base}^{elastic\ pier}} \qquad (6)$$

Ratios (4) and (5) represent the individual effects of SSI and pier column inelasticity respectively while ratio (6) represents the combined effect of SSI and pier inelasticity. Based on the common engineering practice, it was concluded that a particular effect could be safely neglected if the value of the pertinent ratio lied within ±10% of unity and it could be ignored based on engineering judgment if the ratio is between ±10% and ±20%. Whereas the effect should not be ignored if the ratio exceeded ±20% of unity.

The above observations are summarized in Figs. 6a and 6b for bridge displacement and column shear force respectively. It can be observed from Fig. 6a that the cases in which SSI can be safely neglected or ignored with engineering judgment for bridge displacement and column shear force are more than the pier inelasticity cases. This means that ignoring pier inelasticity in the computation of these parameters for such bridges is generally more detrimental than ignoring the SSI effect. Similarly, it can be observed from Fig. 6b that SSI effect is important and cannot be neglected in only selected soil/rock classes only. Whereas, pier inelasticity is important in all soil/rock classes.

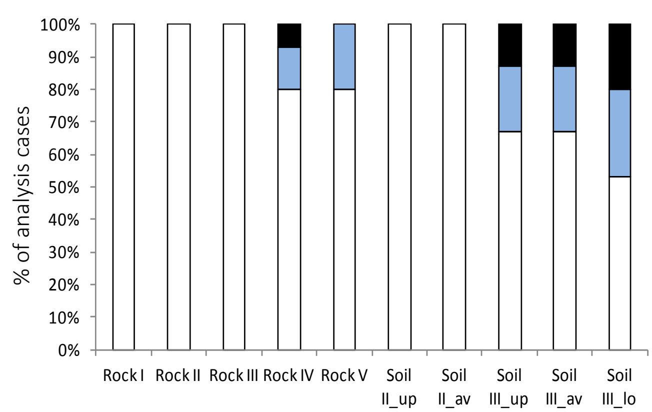


(i) SSI effect

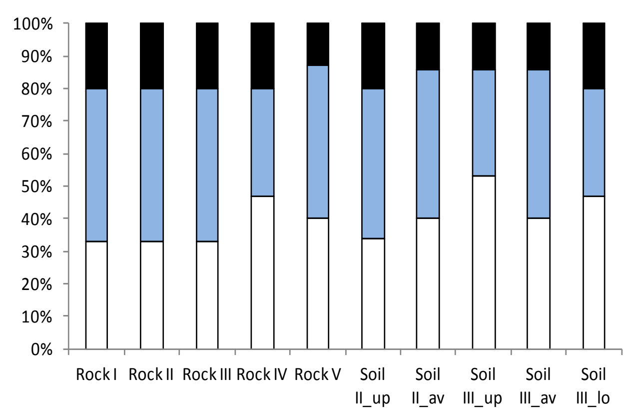


(ii) Pier inelasticity effect

(a) Bridge deck displacement



(i) SSI effect



(ii) Pier inelasticity effect

(b) Column shear force

Fig. 6 - Relative importance of SSI and Pier Inelasticity on (a) Bridge displacement and (b) Column shear force

5.3 Effect of SSI & pier inelasticity on modal frequencies

Modal parameters (frequencies, mode shape and damping ratio) are important structural parameters that can be determined from acceleration measurements at discrete locations in the bridge. Digital signal processing (DSP) and system identification (SI) techniques are employed to detect any anomaly in these parameters for used in structural health monitoring of bridges and other structures [22 – 24].

In the current study, modal frequencies remained constant for the elastic pier case for all levels of ground motions and showed variation only due to the change in the stiffness of the boundary support / foundation springs. However, for the case of inelastic pier, modal frequencies changed due to change in stiffness of the pier as well as foundation springs [21].

Figs. 7 and 8 present the breakdown of the components (i.e. SSI and pier column inelasticity) causing change in modal frequencies for soil profile and rock profile bridges respectively. The contribution of each component was computed by the following relationships:

$$Pier\ inelasticity\ component = (f_{fixed}^{elastic} - f_{fixed}^{inelastic})/f_{fixed}^{elastic} \tag{7}$$

$$SSI\ component = (f_{fixed}^{inelastic} - f_{SSI}^{inelastic})/f_{fixed}^{elastic} \tag{8}$$

Pier column inelasticity accounted for an average change of 66%, 49%, 40% and 84% in 1st to 4th modal frequencies respectively for the pile supported bridge in the weakest soil profile as shown in Fig. 7. Fig. 8 reveals that the share of pier column inelasticity to average change in 1st to 4th modal frequencies was 85%, 72%, 67% and 91% respectively for the weakest rock profile bridge. This indicates that most of the change that resulted in the lower modal frequencies was due to inelasticity in the pier columns. Contribution of pier inelasticity towards change in the 3rd modal frequency in both soil and rock profile bridges was relatively less. This was due to the fact that the 3rd mode was a torsional mode and torsional stiffness of the bridge sub-structure was substantially more than the torsional stiffness of the foundation which resulted in lesser contribution to change from structural components and more from the soil-foundation system.

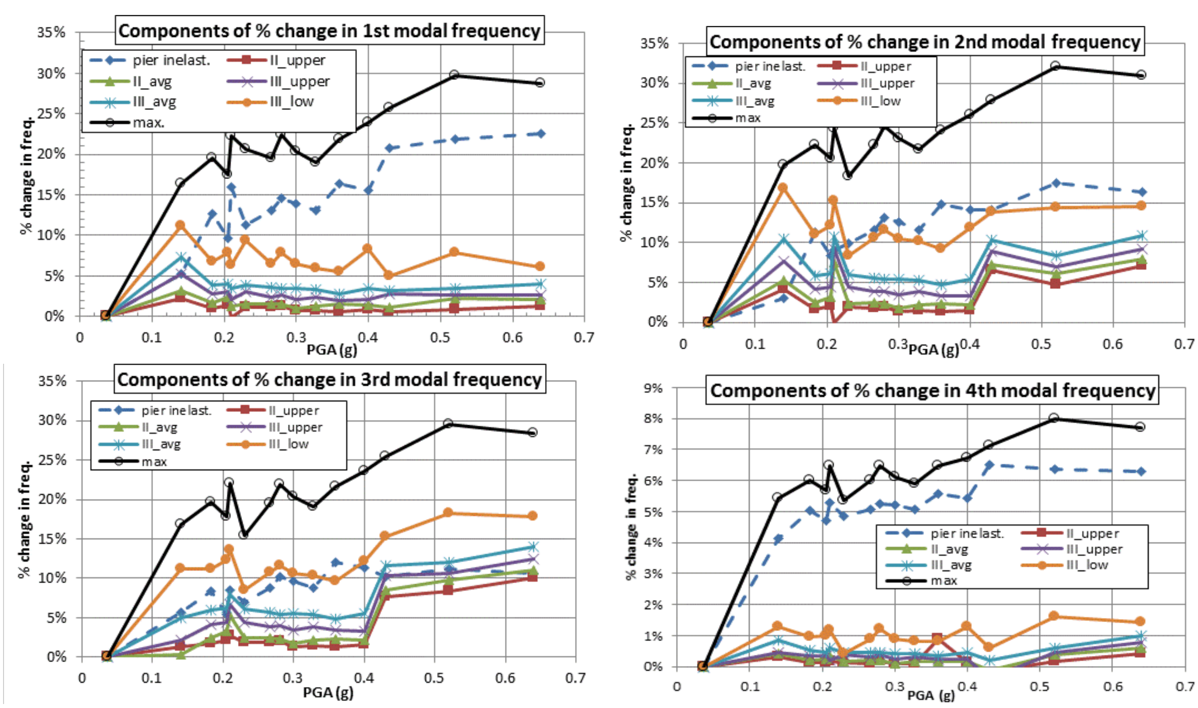


Fig. 7 - Components of change in modal frequencies – pile supported bridge (soil profiles)

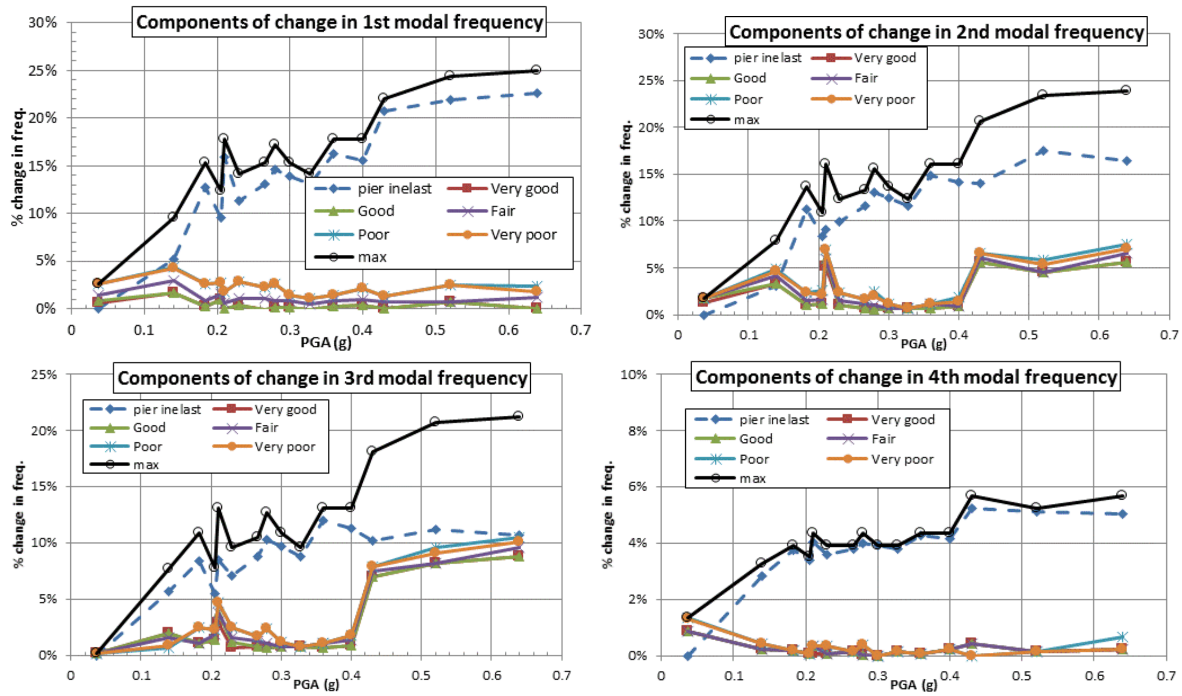


Fig. 8 - Components of change in modal frequencies – block foundation bridge (rock profiles)

6. Conclusions

The following conclusions are drawn from this study:

i- Pier column inelasticity contributed significantly to the bridge displacement and column shear force whether SSI was included or neglected. Therefore pier column inelasticity should not be neglected in the seismic design / evaluation of the class of bridge studied herein.

ii- SSI effect was moderately significant for displacement and shear force of bridge footings founded in class IV and V rock profiles. SSI should not be ignored in these rock profiles, contrary to the recommendations of AASHTO [10]. However, SSI can be neglected for foundations on class I to III rock profiles. SSI effect was significant in bridges founded on piles in soil profile III and should not be ignored.

iii- Bridge deck displacement and pier column shear force were affected more by pier column inelasticity than SSI. Therefore, more attention needs to be focused on including the effect of pier column inelasticity in design than SSI. This observation is in line with the conclusion of [4] that effect of SSI in bridges is more strongly influenced by the non-linear structural properties of bridge sub-structure components (piers and abutments) than by soil properties.

iv- Based on the results for rock class IV and V, it is anticipated that SSI effect will become pronounced for medium span bridges with shallow foundations supported by soil profiles (i.e. AASHTO site classes C and D). Therefore, use of shallow foundations for bridges in these soil profiles should be done with due considerations for SSI effect.

v- Majority of the lower modal frequencies were affected more by pier column inelasticity than by SSI in both rock profile and soil profile bridges.

vi- The study presented herein used equivalent linear models and needs to be extended for non-linear models to confirm the obtained results.

7. Acknowledgements

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