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# RESPONSE OF SEISMIC ISOLATED BRIDGES INCLUDING SOIL STRUCTURE INTERACTION EFFECTS

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## ABSTRACT

With the wide spread of the seismic isolation in the last decade all over the world, there is a pressing need to understand the contribution of soil structure interaction (SSI) in the response of seismically isolated structures and especially bridge structures, which are essentially determined structures with low ability to redistribute loads in cases of component failures. The effects of soil structure interaction (SSI) on seismically isolated bridge structures are investigated through a parametric study. The pier is assumed to have a constant elastic stiffness where two soil shear wave velocity levels were considered. A suite of 20 seismic motions was used to provide the excitations in this study. The isolated bridge system was modeled around a typically build highway over-pass. The analyses results are compared with the results of the isolated bridge without soil structure interaction effects (fixed pier). These comparisons indicate that the SSI on seismic isolated bridges could increase both the isolation drift and the pier shear force of the bridge system. Variation of the responses for the system with SSI as compared to the systems without SSI reach +25% which is rather high level and could, if not accounted for in the design process, lead to failures of the bridge structure.

Keywords: Bridge, Seismic Isolation, Soil Structure Interaction

## INTRODUCTION

Elevated highways and bridges are very important elements of the infrastructure in modern societies. Due to their importance, loss of functionality after a seismic event is not an acceptable performance criterion for the vast majority of those structures. However, the performance of the bridges during the recent Northridge (1994) and Kobe (1995) earthquakes alarmed the bridge engineering community, which started looking towards alternative design methods to minimize the seismic hazard. One such design method, which has shown significant potential of improving the seismic performance over the conventional design philosophy, is seismic isolation (e.g. Tsopelas and Constantinou, 1997). On the other hand if seismically isolated bridges are not designed properly, could suffer severe damage (e.g. Bolu Viaduct, in Duzce Earthquake 1998) if not total collapse under seismic events (Roussis et al. 2003).

Extensive research work has been conducted in the past thirty years regarding the effects of soil-structure-interaction (SSI) on the seismic response of civil engineering structures. Until recently, it was thought that SSI effects are beneficial to the response of civil engineering structures, since similar to seismic isolation systems, SSI provides additional flexibility to the structure or naturally isolates the structure from the shaking ground. For example in the NEHRP-97 specifications it was stated that the response of a structure under earthquake loading could be conservatively evaluated without taking the

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SSI effects into consideration. Even though the above statement may be true for a class of structures (for example, structures with large redundancy) there are case histories that the above-mentioned assumption may lead to an unsafe design.

A number of studies have investigated the influence of SSI on the earthquake response of conventionally designed bridges in recent years. Ciampoli and Pinto (1995) conducted a large parametric study on conventionally designed bridges founded on shallow foundations and concluded that SSI effects consistently decreased the ductility demands of the piers. On the contrary, Mylonakis and Gazetas (2000) using a simplified model for the bridge and the foundation have showed that if the SSI effects are not accounted correctly in the analysis, especially for single degree of freedom like structures, could lead to failures. Jeremic et al. (2003) conducted a detailed finite element study on the seismic response of the I-880 viaduct in Oakland and concluded that SSI can have both beneficial and detrimental effects on the response of the structure depending on the characteristics of the ground motion.

The effects of SSI on seismically isolated bridge structures have not yet been addressed in the literature. This might be because; on a first sight it is not unreasonable to argue that SSI effects would have a beneficial effect in response of seismic isolated bridges, given that SSI induces additional flexibility into the bridge-foundation-soil system thus reinforces the "isolation" effect by further lengthening the periods of the structural system. Investigating the validity of this premise is of great importance in earthquake engineering, especially now that seismic isolation becomes more and more popular among the practicing engineering community around the world.

The present parametric study investigates the effects of SSI on the seismic response of a typical seismically isolated freeway over-crossing, which is founded on soft, non-liquefiable soil through a pile group foundation. Using a nonlinear hysteretic model, which could account for the non-linear hysteretic behavior of the isolation system and possibly the pier, the inertial interaction between the foundation-soil system and the superstructure is studied. The results are compared with those obtained from fixed-pier analyses (no SSI effects). Since the structural configuration investigated herein is a seismic isolated highway over crossing which is typically used all over the word, the results presented in this study are expected to provide important insight on the SSI effects to practicing engineers throughout the world.

## SOIL STRUCTURE INTERACTION PROBLEM

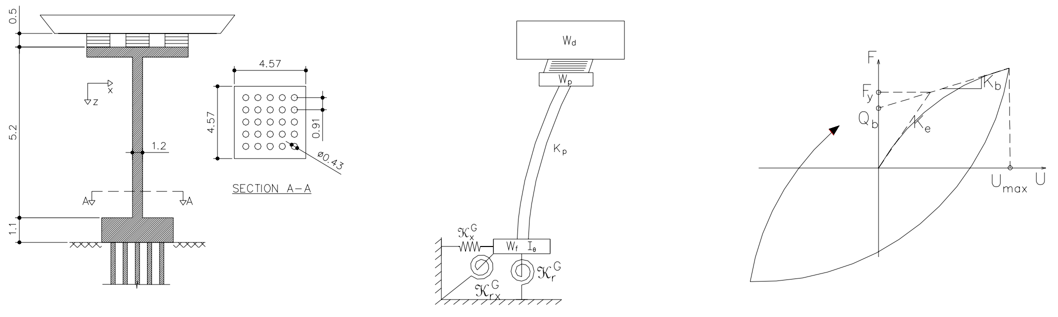
The seismic SSI problem involves two major components. The first is the response of the soil itself as seismic waves travel through the soil deposit. The second is the coupled foundation-superstructure response, which is usually assumed to be a superposition of the response of the foundation itself to the excitation in the absence of the superstructure (kinematic response), and the effect of the additional flexibility caused by the foundation to the response of the superstructure (inertial response).

The soil response analyses is one of the most important aspects of earthquake engineering field, since it will determine the ground motion that will be experienced at the top of soil without the presence of a structure or the so called free filed response. The analysis involves estimation of the seismologic characteristics of the region, determination and modeling of the soil profile and its dynamic characteristics. Furthermore it accounts for the multiple reflections and refractions that will occur at the soil layer interfaces as the seismic waves propagate though the soil deposits. Although special purpose computer programs exist for this purpose, the validity of the results depend greatly on how accurate dynamic soil properties are estimated, which in spite the improvements in the in-situ testing, is still a challenging task. In the present study no soil amplification analysis was performed, rather the considered acceleration time histories, which were recorded in a variety of soil conditions, were used directly to excite the structure and the springs modeling the soil.

Extensive research has been conducted on the kinematic response of piled foundations. The results presented by Gazetas et al. (1992), and Gazetas and Mylonakis (1998) indicate that, as a result of the kinematic response, a scattered wave field might be created which could cause differences in the free-field response and the motion that the structure will experience at the pile cap. This difference from the free-field response or the so-called kinematic interaction is a function of a) the relative stiffness of the pile with respect to the stiffness of the soil (manifested by the ratio of their respective moduli, $E_p/E_s$), b) soil profile, c) pile boundary conditions, and d) the excitation frequency. Nevertheless, the same studies show that for excitations with relatively low frequency content or for cases where the dimensionless wave parameter $a_o$ ($a_o = \omega d/V_s$) is lower than 0.25 the effect of the kinematic response on the free field response could be ignored and the free-field motion could be used as the input motion. Since the isolated bridge is expected to be excited by relatively low frequencies, which in turn will result in low dimensionless wave parameters, a homogeneous soil profile assumption is adapted and the kinematic soil-pile interaction is ignored in the current study. The inertial interaction on the other hand is accounted for, by replacing the pile foundation with springs, and dashpots as explained in detail in a later section.

## DESCRIPTION OF THE BRIDGE MODEL

The bridge under consideration and the model utilized in this study are shown in Figures 1(a) and (b) respectively. The model consists of a cantilever pier with it's mass concentrated on its top, the non-linear hysteretic isolation system, and the tributary deck weight on top of the isolation system. The properties of the structure are selected such that it represents a typical highway crossing used all over the world and are listed in Table 1. The foundation system consists of a 5x5 vertical floationg pile group, which is founded on a homogenous medium. The effect of foundation flexibility is incoorparated into the mechanical model via horizontal, rocking and cross-rocking springs. Only the lateral response of the system (x direction ) is considered.




(a) (b) (c)

**Figure 1 (a) Bridge and pile foundation system; (b) Mechanical model for the bridge and foundation system utilized in this study; (c) Bilinear force displacement relation of the seismic isolation system.**

The isolation system is simulated as bilinear spring utilizing smooth hysteretic behavior. The model is based on Bouc-Wen model as utilized by Tsopelas and Constantinou (1997). Although this behavior is typical to lead rubber bearings (LRB) or an isolation system consisting of lubricated sliding bearings and metallic yielding devices, the results presented in this study could also be applicable for isolation systems consisting of sliding bearings with restoring force capabilities such as the friction pendulum system (FPS) isolators. As depicted in Figure 1(c) the variables controlling the behavior of the seismic isolation system are the characteristic strength ($Q_b$) or the yield strength ($F_y$), the elastic stiffness ($K_e$) and the post yielding stiffness ($K_b$). The choice of Kb and $F_y$ will determine the level of the force transmitted into the pier, and the peak displacement that the isolation system will experience. In the United States design codes, a strong restoring force approach is favored, where $K_b$ is sufficient enough to reduce the uncertainty in the peak seismic displacement and minimize any potential permanent displacement, where in other design philosophies (e.g. Italian codes) a low $K_b$ value might be favored in order to reduce the uncertainties of the peak force (isolation force) transmitted to the pier.

The seismic isolation period of the bridge in the present study is defined based on the post yielding stiffness of the isolation system from $Tb=2\pi(W_d / g / K_b)1/2$. The values characterizing the isolation system considered in this study are shown in Table 1. In addition the elastic stiffness of the isolation system was assumed to be 10 times the post yielding stiffness ($K_e=10K_b$) or the post to pre-yielding stiffness ratio was assumed constant equal to 0.1.

**Table 1 Properties of Isolation Systems Considered**

| System Properties | Values |
|---|---|
| Deck Weight (kN) | 2,600 |
| Pier weight (kN) | 110 |
| Foundation Weight (kN) | 650 |
| Pier Elastic Stiffness (kN/m) | 1.24 E5 |
| Isolation period, $T_b$ (sec) | 2.0 |
| Isolation Strength Ratio ($F_y/W_d$) | 0.12 |

The design practice of the conventionally build bridges allows inelastic action in the pier through the ductility and/or strength reduction factors concepts. In isolated bridges, however, allowing excessive inelastic action to occur in the pier, in addition to the risk that might be carrying (bridges are essentially determined structures), affects the performance of the isolation system by making it ineffective (large displacements occur in the pier and not in the isolation system). This is the main reason behind the latest design codes (AASHTO 2002) for isolated bridges where the piers are designed essentially elastic or allowing some limited inelastic action (ductility less than 1.5 or 2). Complying with this requirement, in the present study the bridge pier is assumed to behave in an elastic manner, unless it is noted otherwise.

## DYNAMIC STIFFNESS OF PILE FOUNDATIONS

The dynamic impedance of a single pile can be expressed as:

$$\mathcal{K}_j^S = K_j^S + i\omega C_j^S \tag{1}$$

where, the subscript j denotes the direction of the frequency dependent spring, i.e. x for horizontal, z for axial, r for rocking, and x-r for cross term horizontal-rocking; $K_j^S$ is the real part of the dynamic stiffness, $C_j^S$ is the dashpot coefficient, and $\omega$ is the vibration frequency. The real part of the above given equation can be re-written as:

$$K_j^S = K_j k_j \tag{2}$$

where, $K_j$ is the static stiffness of the pile, $k_j$ is the dynamic stiffness coefficient. The imaginary part of equation 1 on the other hand, can be evaluated using closed formed solutions available in the literature or using the integral expressing given in Gazetas and Dobry (1984)

$$C_j^S = \int_0^L c_j(\omega) Y_{sj}^2(z)\,dz \tag{3}$$

where, $c_j(\omega)$ is the distributed damping coefficient that accounts for both radiation and soil inherent damping (its expression can also be found in Gazetas and Dobry, 1984), $Y_{sj}(z)$ is the static deflection profile of the pile normalized to its maximum amplitude on the top. In this study the static deflection profiles are determined using closed formed solutions of a beam-on-Winkler foundation model with constant sub-grade reaction along its depth, which are available in the literature. The interested reader is referred to Poulos and Davis (1980) for more information. The static cross-rocking stiffness and the dynamic stiffness coefficients in all directions under consideration are evaluated using closed form equations given in Gazetas et al. (1992). Once the impedance of a single pile is calculated the pile group impedance (of the 5x5 pile group) can be evaluated by applying the simplified super position method proposed by Dobry and Gazetas (1988).

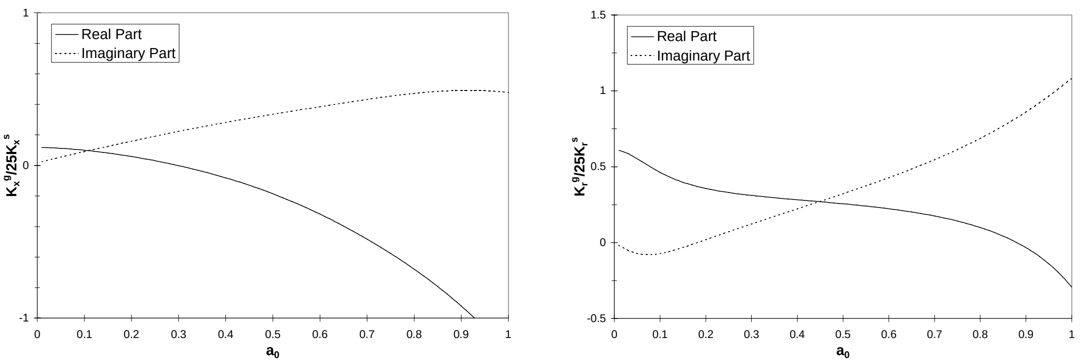



**Figure 2 Lateral and rocking impedances of the 5x5 pile group for Vs=110 m/s.**

Figure 2 presents the group stiffness and dashpot values for lateral and rocking motions, normalized with the sum of the static stiffness of the individual piles as a function of dimensionless wave parameter $a_o$ (=ωd/Vs), with d being the pile diameter and Vs the shear wave velocity of the soil. The group impedance functions presented in the above figure are evaluated assuming linear-elastic soil and pile behavior and assuming that no slippage or discontinuity occurs between the pile and the surrounding soil. Herein the adapted values for soil Poison's ratio, mass density and inherent damping are 0.4, 1.8 t/m3 and 0.05 respectively. Two values for the shear wave velocity are used in this study, Vs=110 m/sec and Vs=80 m/sec. In Figure 2 the stiffness at zero frequency ($a_o$=0) corresponds to the well-known static group efficiently factor, which could alternatively be determined using 'p-y' curves. The significance of pile-to-pile interaction is apparent from Figure 2, since the static group stiffness values are well below unity. Furthermore the same figure shows the frequency dependency of the group dynamic stiffness values except the lateral dashpot coefficient that is constant over a wide range of frequencies.

While in a frequency domain analyses one could directly account for the frequency dependency of foundation springs and dashpots, the non-linear behavior of the isolation system requires a time domain solution that in turn makes it impossible to account for the frequency dependence foundation impedance functions. Moreover, with the isolation period being fairly large it is expected that the isolated bridge will be excited by low frequencies that correspond to dimensionless wave parameter less than 0.05. For this reason the spring constants for the foundation system are assumed to be frequency independent and are evaluated at $a_o$~0. To check the effect of foundation rocking damping on the response of the structure for each case two time history analyses are carried out, namely 'CASE A' where the rocking dashpot coefficient equals to zero and 'CASE B' for which the rocking dashpot coefficient is evaluated at $a_o$=1. The final values for the foundation springs are given in Table 2.

**Table 2 Values for the foundation spring utilized in the present study**

| | Vs=110 m/sec | | Vs=80 m/sec | |
|---|---|---|---|---|
| | Case A | Case B | Case A | Case B |
| $K_x$ (MN/m) | 265 | | 160 | |
| $K_r$ (MN·m/rad) | 7373 | | 5105 | |
| $K_{xr}$ (MN/rad) | -330 | | -240 | |
| $C_x$ (MN·s/m) | 7 | | 5 | |
| $C_r$ (MN·m·s/rad) | 0 | 54 | 0 | 34 |

## SEISMIC EXCITATIONS

A total of 20 acceleration time histories from 6 actual earthquakes are used in the present study. The average of the spectral accelerations are shown in Figure 3, while their detailed information is shown in Table 3.

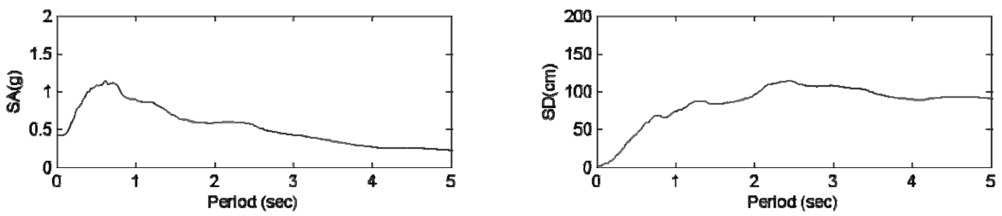



**Figure 3 Average response spectra of the 20 considered acceleration time histories.**

**Table 3 List of the considered Seismic Excitations and their corresponding scale factors**

| Record ID | Seismic Event | Station | Component | Scale Factor |
|---|---|---|---|---|
| 1, 2 | 1992 Landers | Joshua | 90, 0 | 1.48 |
| 3,4 | 1992 Landers | Yermo | 270, 360 | 1.28 |
| 5,6 | 1989 Loma Prieta | Gilroy 2 | 0, 90 | 1.07 |
| 7,8 | 1989 Loma Prieta | Hollister | 0, 90 | 1.46 |
| 9,10 | 1994 Northridge | Century | 90, 360 | 2.27 |
| 11,12 | 1994 Northridge | Moorpark | 180, 90 | 2.61 |
| 13,14 | 1949 W. Washington | 325 | N86E, N04W | 2.74 |
| 15,16 | 1954 Eureka | 022 | N79E, N11W | 1.74 |
| 17,18 | 1971 San Fernando | 241 | N00W, S90W | 1.96 |
| 19,20 | 1971 San Fernando | 458 | S00W, S90W | 2.22 |

## ANALYSES RESULTS AND DISCUSSION

The isolated bridge system is analyzed using the above given 20 ground motion time histories. The system responses considered are the displacement of the isolation system (isolation drift) and the shear force of the pier. The peak values were normalized by their corresponding values obtained from the cases without the SSI effects (fixed pier cases). The results therefore are presented in terms of pier shear ratio (PSR) and isolation drift ratio (IDR) which are defined as

$$Pier\,Shear\,Ratio = \frac{Pier\,Shear\,(V_s)}{Pier\,Shear\,(V_s = \infty)} \quad (4)$$

$$Isolation\ Drift\ Ratio = \frac{Isolation\ Drift\,(V_s)}{Isolation\ Drift\,(V_s = \infty)} \tag{5}$$

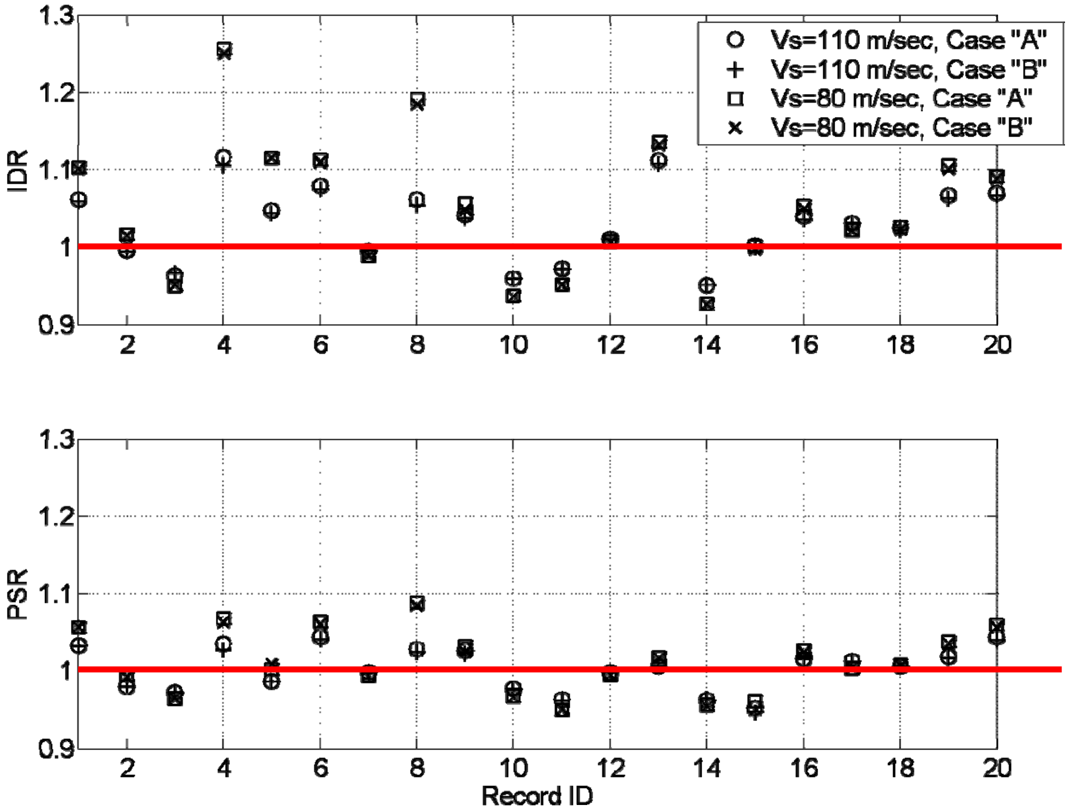



**Figure 4 PSR and IDR vs Seismic Motion Number**

Figure 4 summarizes the results obtained from non-linear time history analyses for the 20 seismic time histories considered. The Record ID in Figure 4 corresponds to the Record ID numbers presented in Table 3. As can be seen from Figure 4, SSI clearly affects the maximum responses of the isolated bridge-pier structure. The SSI effects seem to be either unfavorable or beneficial depending mainly on the details of the individual seismic records. Another important observation which can be made from Figure 4 is the effect of the foundation flexibility has on the response. As the foundation system becomes more flexible the effects of SSI seem to become more pronounced. Figure 4 show that when SSI effects are accounted for, the maximum isolation drift has a spread ranging between +25% and -%10 relative to the fixed pier solution. Similar observations can be made for the isolation drift ratio, however the spread relative to fixed pier solution is smaller ranging between $\pm 10\%$ for the softest soil condition.

The effect of including rockingdamping in the analysis is also studied. Results in Figure 4 for “CASE A”, which rocking damping is ignored are almost identical to “Case B’ results where rocking damping is included. This observation indicates that rocking damping has a negligible effect on the response of the isolated bridge structures.

Figure 5 presents the time histories of the isolation system drift and the pier drift under the seismic excitation record ID 6. The two graphs on the left depict the isolation system drift and pier drift for the time interval between 2 and 10 secs where the maximum responses occur. The difference in the response between the fixed and the flexible foundation are visible with the isolated structure experiencing higher response than the fixed structure. In this particular case the maximum response increase is of the order of 10% for the isolation drift and of the order of 5% for the pier drift. The graphs on the right depict the free vibration response after the end of the seismic input which occurs at approximately 39.5 seconds. The period shift of the fundamental period of the bridge system can be observed from these graphs. The difference in the vibration periods between the fixed bridge and the bridge with SSI amounts to approximately 15%, 0.7 secs for the fixed bridge and 0.8 secs for the bridge with SSI. This observed period shift seems to be large. However one has to remember that the bridge during the free vibration response beyond 40 seconds behaves as a linear 2DOF system with stiffness the elastic pier stiffness and the elastic isolation system stiffness (Ke). When the post yielding stiffness (Kb) of the isolation system is utilized (that happens when the isolation system displaces beyond the yield displacement) the period shift due to SSI is rather minimum amounting to less than 0.2 sec.

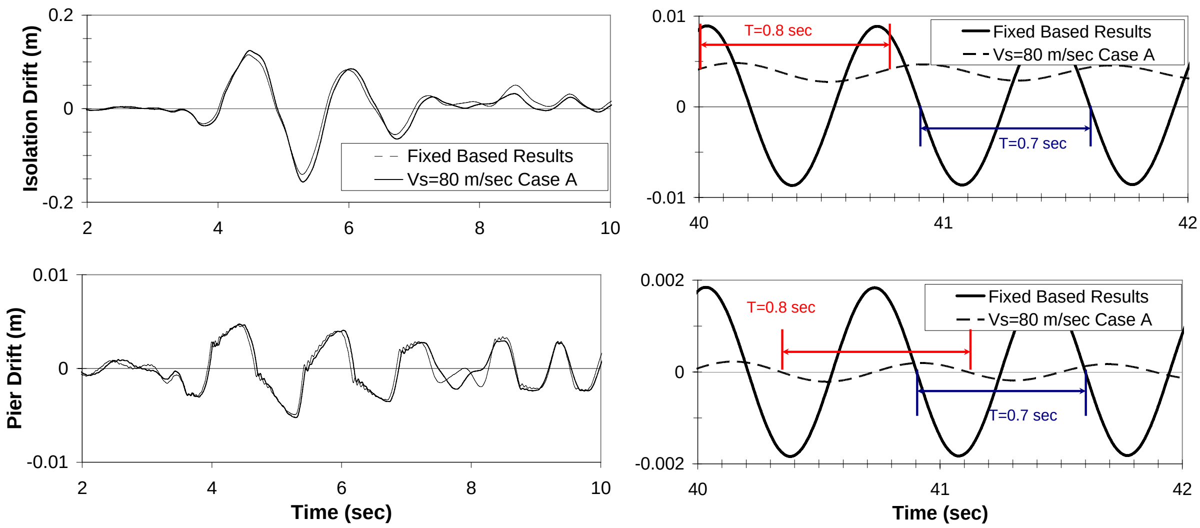



**Figure 5 Partial Response time histories of the system under Seismic Record ID 6**

All the aforementioned results are obtained for elastic pier behavior, which corresponds to a strength reduction factor well below 2 or the ductility based portion of the strength reduction factor below unity. To investigate the effect of pier inelastic action, additional time history analyses are carried out using record ID 6 and the results is presented in Figure 6. In the analyses it is assumed that the pier behaves as a bilinear spring with smooth hysteretic behavior and very small post yielding stiffness (post yielding stiffness to pre yielding stiffness ratio = 0.01).The yield force of the pier is set to the pier shear obtained from elastic-fixed base analyses, i.e. the ductility-based portion of the strength reduction factor is assumed to be unity.. As can be seen in Figure 6 the fixed base results are quite different from the results where soil structure interaction is considered. . While in the fixed base analyses the pier experiences ~5 mm residual displacement, in the case where soil structure interaction is considered the residual displacement in the pier doubles to ~10mm. Refereeing back to Figure 5 if the pier is linear elastic SSI effects seem to increase the pier shear ~%5. While one might think that this ~%5 increase is rather small for all practical purposes, if the pier has no capacity reserve this rather small increase can double the residual displacements that the system experiences.

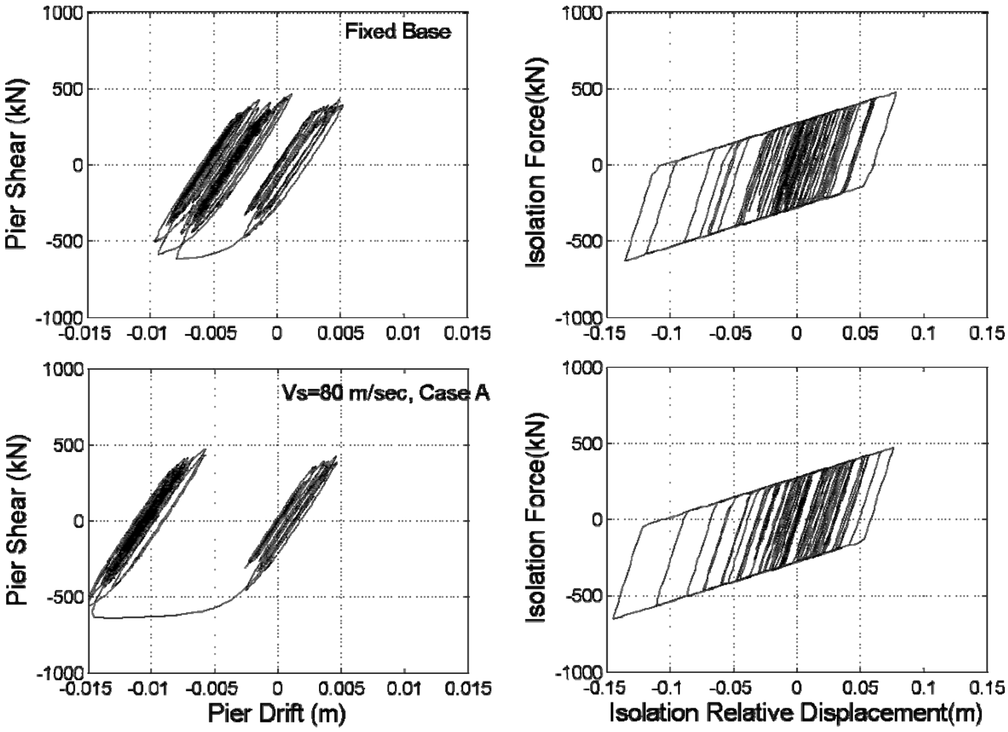



**Figure 6 Comparisosns of pier shear force vs pier drift and isolation system force vs isolation drift between the fixed bridge and the bridge with SSI under the excitation of record ID 6.**

## CONCLUSIONS

The effects of SSI on the response of seismic isolated bridges are investigated in this study. SSI is manifested through the flexibility of the foundation-soil system. The foundation considered consists of a 5x5 pile group in a homogeneous elastic half space. By varying the soil shear wave velocity between 80 and 110m/s the foundation-soil flexibility decreases and accordingly the SSI effects are dropping. The premise that the SSI effects, since they increase the flexibility of the system and thus the isolation effect, are benefiting the seismic isolated bridges does not seem to hold as it is shown in Figure 4. The range of variation of the isolation drift and the pier shear as compared to the same responses for the case of no SSI varies between +25% and -10%. This indicates that SSI effects could be either favorable or unfavorable on the response of the seismic isolated bridges. Even though such variations might not look spectacular, they are very large and could potentially lead in failures if the SSI effects are not accounted properly. The results were obtained using the suite of seismic motions, and thus the conclusions could be limited by this choice.

## REFERENCES

AASHTO (2002). “Guide Specifications for Seismic Isolation Design”, American Association of State Highway and Transportation Officials, Washington, DC.

Ciampoli, M., Pinto, P.E., (1995) “Effects of soil structure interaction on inelastic seismic response of bridge piers’ Journal of Structural Engineering, 121(5), 806-814

Dobry, R., Gazetas, G., (1988) “Simple method for dynamic stiffnes and damping of floating pile groups” Geotechnique, 38(4), 557-574

Jeremic, B., Kunnath, S., Xiong, F., (2003) “Influence of soil-foundation-structure interaction on seismic response of the I-880 viaduct”, Engineering Structures, 26, 391-402

Gazetas, G., and Dobry, R. (1984) “Horizontal response of piles in layered soil”, Journal of Geotechnical Engineering, 110(1), 20-40

Gazetas, G., Fan, L., Tazoh, T., Shimizu, K., Kavvadas M., Makris N., (1992) , "Siesmic pile-group-structure interaction", Piles Under Dynamic Loads, E., Prakash, .(ed.), ASCE, 56-93…

Gazetas, G. and Mylonakis, G. (1998). "Seismic Soil-Structure Interaction: New Evidence and Emerging Issues" Geotechnical Earthquake Engineering and Soil Dynamics III, ASCE, 1119-1174.

Mylonakis, G. and Gazetas, G. (2000). "Seismic Soil-Structure Interaction: Beneficial or Detrimental" Journal of Earthquake Engineering, 4(3), 277-301.

NEHRP (1997) "Recommended provisions for seismic regulations for new buildings and other structures", Parts 1 and 2, Building Seismic Safety Council, Washington DC

Poulos, H.G., Davis, E.H, (1980) "Pile foundation analysis and design", John Wiley and Sons, New York

Roussis, P.C., Constantinou, M.C., Erdik, M., Durukal, E., Dicleli M. (2003). "Assessment of Performance of Seismic Isolation System of Bolu Viaduct." Journal of Bridge Engineering, 8(4), pp.182-190

Tsopelas, P. and Constantinou, M.C. (1997). "Study of Elastoplastic Bridge Seismic Isolation System." Journal of Structural Engineering, 123(4), 489-498