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THE ROLE OF FOUNDATION FLEXIBILITY ON THE SEISMIC DESIGN OF BRIDGE PIERS

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ABSTRACT

The main objective of the present work is to explicitly describe and to further investigate the role of foundation flexibility effects on the overall seismic performance of bridge piers. In a performance-based design framework, the advantages of these phenomena are examined thoroughly through implementation of detailed analytical and computational models. A motivating premise is that when properly designed for, compliance owing to foundation-soil interaction may alleviate significantly the introduced deformation demands in the piers, at the expense of some permanent residual deformation at the foundation. Using this concept a very attractive alternative bridge design strategy is assembled, based on the controlled rotation of the foundation. A counterargument is that excessive soil deformations could have a detrimental impact on the structural integrity and the stability of the entire bridge system. Therefore, it would be of great importance not only to further investigate this soil-structure interaction type, but also to prescribe the limit states in which the foundation flexibility effects can be exploited in order to mitigate the pier seismic demands. Herein, two alternative foundation configurations are investigated in order to assess the seismic performance of the entire bridge system. In this context, the seismic performance of typical enlarged stiff pile-caps is thoroughly computed and compared with the anticipated performance of flexible discrete pile-caps that support each bridge column separately. Detailed nonlinear response history (NRHA) and pushover analyses are performed for the foundation-pier assembly using sophisticated constitutive models to represent structural and soil material behaviour. Recognizing that the pier-foundation system is critical for the stability of the entire bridge, the contribution of soil-structure interaction effects is examined and evaluated with reference to the soil properties and the structural type of the bridge system. Based on the above, design strategies are proposed for the dimensioning of the pier-foundation system of a typical highway overpass in order to properly mitigate nonlinear demands in the pier, by accommodating a very limited amount of rotation in the foundation system.

INTRODUCTION

It has been shown through experiment and analytical reasoning that Soil-Structure Interaction (SSI) effects may modify considerably the dynamic response of RC Bridges (Kotsoglou and Pantazopoulou, 2013; Antonellis and Panagiotou, 2013]. Many different interaction phenomena may be identified in the literature with a significant impact on the overall performance of the system. Soil-pile (Mylonakis et al., 1997), pier-foundation (Mylonakis and Gazetas, 2000) and bridge-embankment (Kotsoglou and

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Pantazopoulou, 2007; Goel, 1997; Sextos et al. 2003) interaction effects are considered to be of critical importance during design and assessment. Depending on the bridge system configuration and the foundation design, implications of soil-bridge interaction may range from favourable to detrimental, as a means that modifies the demands that occur in the bridge piers. In the present work, pier-foundation interaction effects are thoroughly investigated in an attempt to mitigate the seismic demands in pier columns through some marginal engagement of the flexible foundation, to be achieved by accommodating a degree of controlled deformation in the foundation soil, particularly in the ultimate limit state.

The established bridge design tradition in Europe has always opted for stiff pile-foundations of significant depth, in order to achieve perfect fixity conditions. The intention behind this design strategy in the Eurocodes and other EU, standards is to secure that no damage would take place in the less accessible for repair, foundation soil. On the other hand, a considerable number of highway overcrossings may be found in the US, with monolithic pier-superstructure connection and a relatively flexible foundation configuration. From recorded responses of heavily instrumented overpasses located in California, it has become apparent that in many cases the field performance of bridges designed according with the latter approach, where the soil foundation has some degree of participation in the overall response scheme, was significantly improved (Kotsoglou and Pantazopoulou, 2009; Antonellis and Panagiotou, 2013; Inel and Aschheim 2004; Saiidi 2011). Therefore, the idea that SSI effects under controlled conditions may have a positive impact on the seismic performance of the entire system, has recently been acknowledged openly throughout the scientific community. Its basic premise is that the more parts of the structure participate in dissipating the kinetic energy imparted by the earthquake through inelastic deformation, the less permanent damage will occur to any of its individual components; and this is achieved by rendering the flexibilities of the individual parts compatible through proper design (Kotsoglou and Pantazopoulou, 2013).

In defending this emerging design concept, a significant number of important issues are still to be addressed, such as for example: (a) whether SSI can be engineered so that it is mobilized only in controlled conditions (b) how much permanent damage to the foundation is acceptable, at what levels of seismic intensity (i.e., defining acceptable limit states for the foundation soil), (c) how can the design of the bridge substructure better engage the foundation while at the same time limiting the extent of damage to both pier and foundation, (d) engaging the foundation appears to be a mutually conflicting option with the use of support hardware at the pier top (e.g. bearings); design guidelines are needed in integrating these alternative technologies in different bridge types or at different levels of performance. The above mentioned issues are at the core of the present research study, with emphasis into alternative foundation configuration design strategies of two typical, instrumented, US highway overcrossings. Stiff enlarged pile-caps integrating the foundation of the entire pier frame are compared with more flexible, separate column pile-caps which may mobilize appreciably the foundation soil. In this context, detailed nonlinear response history analyses (NRHA) are performed for the foundation-pier assembly, using sophisticated constitutive models to represent structural and soil material behaviour.

PIER-FOUNDATION INTERACTION PRINCIPLES

In general, pier-foundation interaction effects under horizontal ground excitation, comprise two additive components: (a) rotations from flexural pier deformation and (b) rigid body pier rotation owing to deformations in the soil, caused primarily by the large overturning moments that are transferred from the fixed end support of the pier. Herein, a rather straightforward method for the evaluation of the dynamic response of the bridge superstructure-foundation-soil system was implemented in the time domain, based on nonlinear springs of Bouc-Wen type to model the soil impedances. The above mentioned springs were calibrated to match the anticipated response which should be compatible with the imposed soil shear strain, in terms of deformation and damping forces. In this context, computational estimations of the dynamic response of the pier-foundation system could be carried out on a more convenient basis rather than modelling the soil half-space.

In the above mentioned framework, based on a purely static approach, the effective, or work equivalent (W.E.) lateral stiffness contribution of the overall bent substructure, is calculated through the consideration of the structure in lateral displacement, when it attains a unit displacement at the reference degree of freedom (the so-called control node) using virtual work principles. Therefore, the pattern of lateral displacements of the pier-foundation system is evaluated based on Fig.1, where Φ is the part of the shape of the fundamental mode of vibration that concerns the pier-foundation assembly, normalized with respect to the displacement of the point of reference considered as "control node – CN" (Φ_{CN}=1). Subscripts on Φ refer to the coordinates in the normalized shape that concern subcomponents or specific points of the pier-foundation assembly. For example Φ_C, Φ_B, Φ_A, are the normalized transverse displacements at the connection of the pier with the deck, at the face of the base-support and at the tip of the piles, respectively (see Fig. 1).

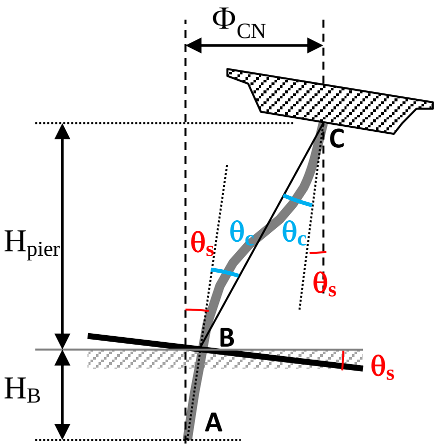


Figure 1. Pier-Foundation Substructure. Contributions to work-equivalent stiffness for cases with compliant foundation

In this context, the total relative displacement from the tip of the piles to the deck is $\Delta\Phi_{pier\text{-}foundation} = \Phi_C - \Phi_A$, whereas the relative displacement owing to deformation of the pier is:

$$\Delta\Phi_{pier} = \Phi_C - \Phi_B - \left[H_{pier} \cdot \left.\frac{\partial\Phi}{\partial z}\right|_{z=H_B} \right] \tag{1}$$

where, H_B is the height of the foundation system with reference to the pile tip (Fig 1). Thus, the equivalent stiffness of the entire pier-foundation system, is:

$$K^{eq}_{pier-foundation} = K_{pier} \cdot \Delta\Phi^2_{pier} + K^{transl.}_{found} \cdot \Delta\Phi^2_{found} + K^{rotat}_{found} \cdot \left(\left.\frac{\partial\Phi}{\partial z}\right|_{z=H_B} \right)^2 \tag{2}$$

where:

$K_{pier} \cdot \Delta\Phi^2_{pier}$: work done by pier deformation, where, $\Delta\Phi_{pier} = \theta_c H_{pier}$ (Fig. 1)

$K^{transl.}_{found} \cdot \Delta\Phi^2_{found}$: work done by lateral deformation of the foundation-soil system, where, $\Delta\Phi_{found} = \theta_s H_B$

$K^{rotat}_{found} \cdot \left(\left.\frac{\partial\Phi}{\partial z}\right|_{z=H_B} \right)^2$: work done by rocking of the pile-cap and flexural deformation of the piles

In Eq. (2) K_{pier} is the translational stiffness of the pier considered fixed at both ends if monolithically connected to the deck (e.g. $K_{pier} = 12EI / H^3_{pier}$); $K^{transl.}_{found}$ is the translational stiffness of the foundation system below grade (against lateral translation of point B relative to A), and $K^{rotat.}_{found}$ is the rotational stiffness of the pile cap and the system of piles against rotation at point B.

Based on Eq. (2), useful observations may illustrate the implications of a non-compliant foundation: A very stiff foundation where $K_{found}^{transl.} \rightarrow \infty$ and $K_{found}^{rotat.} \rightarrow \infty$, will cause the contribution of the last two terms of Eq. (2) to diminish (because the values of $\Delta\Phi_{found}$ and $\partial\Phi/\partial z\,|_{z=H_B}$ will tend to zero), thereby causing $\Delta\Phi_{pier} = \Phi_C$, i.e. the maximum possible value of pier relative end displacement owing purely to pier deformation alone– thereby causing maximum damage to the pier (this is owing to the general principle that the stiffer elements participate significantly less in the work equivalent stiffness, as the work performed is marginal in the absence of noticeable deformation).

In the other end of the range of possible deformation patterns, such as in the case of a rocking foundation, the deformation and thus damage, occurring in the pier is substantially reduced, as stated by Fig. 1, with a consequent attenuation of the pier stiffness to the work equivalent stiffness of the overall system and a commensurate increase of the participation of the compliant foundation stiffness. Although the above mentioned scenario sounds ideal for the implementation of novel design strategies, significant implications may be identified under strong intensity ground excitations as a result of disproportionally increased soil deformation levels. For example, in near-field events an increase of acceleration is observed in the range of low frequencies due to soft soil amplification (e.g. Kobe earthquake). Also additional rotational inertial forces may increase the seismic loading of the structure due to soil flexibility at the base (Mylonakis et al., 1997) with increased P-Δ effects. In the case of diaphragmatic superstructures, considerable fraction of the deck displacement demand may be owing to excessive foundation rotation at the base.

From the above, it is evident that foundation flexibility effects may contribute under certain circumstances, to mitigate damage in the pier at the expense of some damage to the foundation soil. A prerequisite for the implementation of the above mentioned strategy is to control soil deformation and to secure that it will be within the prescribed limits. For this scenario, Kotsoglou and Pantazopoulou, (2013), proposed a set of limit states for all the sub-systems that compose the pier system, as illustrated in Table 1, reflecting an increasing tolerance for foundation damage with increasing earthquake intensity.

It should be also mentioned that although this type of performance could be considered unacceptable for the usual "serviceability" earthquake, at higher intensity events a marginal rotation at the support alleviates excessive damage to the pier, the functionality of which is essential for the collapse prevention performance objective.

According to the Table.1, three reference performance levels are specified in terms of the acceptable damage to the individual components (i.e., A=serviceability associated with frequent earthquakes, B=repairable damage or even partial replacement for rare events, C=collapse prevention for very rare events). To secure minimal soil foundation contribution and damage in frequent seismic events, whereas allowing for some acceptable damage at higher intensity rare events, the proposed values of peak soil shear strain are selected so that (a) the limit corresponds to an elastic apparent soil behaviour in the former case, and (b) damage to the foundation in the latter case corresponds to a comparable ductility as that used conventionally for design of the piers (for DCM, i.e. q≈3.5).

Table 1: Proposed Deformation Limits at performance stages for a well detailed superstructure

Performance Limit:	A	B	C
Soil Strain limit, γ	10^{-5}	10^{-4}	10^{-3}
Pier Column Relative Drift Ductility Ratio, μ_Θ	1	2	3
System displacement ductility, μ_Δ.	1.5	3	5

DETAILED COMPUTATIONAL SIMULATION OF SSI EFFECTS IN TWO INSTRUMENTED MONOLITHIC OVERPASSES

In the absence of a monolithic connection at the pier superstructure connection, foundation flexibility may result in unseating of the deck and catastrophic collapse– this is why these two alternatives for moderating distress to the piers may be considered incompatible and mutually exclusive. However, there is also extensive evidence available of the favourable implications of soil compliance on the seismic response of well designed, redundant bridge systems. A characteristic example of the favourable contribution of SSI is a typical highway overpass with monolithic deck-pier connections, known in the literature as Painter Street Overcrossing (PSO) which is an instrumented bridge at Rio Dell in California. It was observed that mobilization of the embankments in transverse displacement during earthquakes caused an effective increase in ductility demand of the central pier columns of this bridge; yet no damage was observed, with the columns delivered from damage despite their brittle design. The observed improved response of the bridge was attributed by many, to partial rotation at the pier foundations, owing to shear deformation of the soil, leading to a commensurate reduction in the relative drift ratios demanded of the pier columns (Kotsoglou and Pantazopoulou, 2013).

A primary objective of the present study is to quantify and better understand the effectiveness of SSI in reducing the seismic demands of critical bridge subsystems such as the bent columns. Because the SSI contribution is more prevalent in the case of bridge systems supported on shallow or flexible pile-foundation, the study is restricted to typical overpasses that belong to this class, such as the PSO; another overpass bridge example considered here is the so called Meloland Road Overcrossing –MRO, both bridges being instrumented samples of typical 1970's US design practice (in terms of bridge morphology and choice of foundation type), resting on flexible pile-foundation. Furthermore, the behavior of variants of the basic bridge prototypes is studied, in order to examine alternative design scenarios regarding the engineered exploitation of the SSI effects in short monolithic bridges with compliant abutments. The main objective of the implemented analytical studies is to investigate the contribution of the foundation configuration to the seismic performance of the bridge, with emphasis on SSI effects. In this framework, flexible discrete pile-caps are compared with stiff, enlarged, heavy assemblies, in order to assess the contribution of SSI effects to the entire performance of the pier-foundation system.

GEOMETRIC CONFIGURATION AND CONSTITUTIVE PROPERTIES OF THE BRIDGE SYSTEMS UNDER CONSIDERATION

The bridges considered in the study rest on flexible foundations with small diameter piles designed primarily for gravity loads without significant lateral resistance. It should be mentioned that such a design scheme is at odds with modern European design approaches that require heavy foundation with deep caisson piles of a relatively large diameter. Both bridges have monolithic superstructure-column connections, and end abutments supported on earth embankments, encouraging the occurrence of significant SSI effects between the embankment and the bridge. The bridge geometries have been described before in detail (Kotsoglou and Pantazopoulou, 2010)- the MRO overpass is a regular two-span structure with length 63.4m (two equal spans of 31.7m). The central bent has a single column of circular cross-section 1.52m in diameter, which is supported on a pile group of twenty five piles of 0.43m diameter (arranged in a 5x5 rectangular pattern, Fig. 2a). The abutment foundations are supported on a series of seven piles arranged along a line and embedded in the embankment soil.

Similarly, the PSO is also a reinforced concrete bridge with multi-cell box type cross section. The bridge is skewed at an angle of 39°, and is monolithically connected with the supports at the abutments and central bent. It comprises two unequal spans of 44.5m and 36.3m. Again the abutments are supported on the end embankments, with the east side resting on fourteen and the west side on sixteen piles arranged in a line series, having a circular dimension of 0.36m each. The central bent is a two column frame, of 1.56m diameter and a free length of 9.6m, each of them supported on twenty piles arranged in a 4x5 rectangular pattern having a 0.36 m diameter each (Fig. 2b).

To examine the basic thesis of the paper, namely that damage to the piers may be mitigated by design, by using foundations that become increasingly compliant at higher limit states, variants to the basic bridge prototypes were developed as model case studies. The variants were designed with the intention to increase the ductility capacity of the bridge system without affecting the load bearing capacity of the bridge; for this reason the number of columns in the bents was increased while their diameter was reduced (i.e. the gravity axial load ratio is a criterion in that direction). This effectively produced individual bridge piers having a reduced stiffness to lateral translation. Another objective of the study was to compare different pile-cap configurations as a means to control soil damage at the higher limit states where SSI is to be encouraged. Thus, enlarged stiff pile-caps are compared with discrete flexible pile-caps supporting each individual pier, by comparison of the amounts of permanent deformation occurring throughout the bridge pier-foundation system.

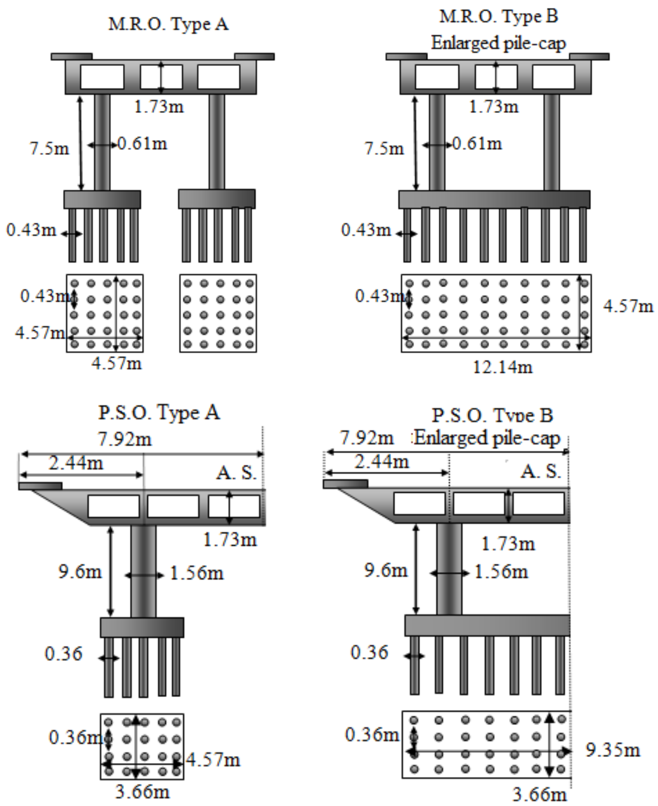


Figure 2. Alternative pilecap configurations under examination (MRO and PSO)

In this framework, based on the existing MRO bridge, the response of an identical bridge variant (MRO Type A in Fig. 2) having a frame bent of two 0.61m diameter columns was studied. Two alternative case studies were investigated: (a) circular cross section columns, founded on a separate pile group each, with the piles having identical geometry as those of the MRO and (b) circular cross section columns, founded on one stiff enlarged pile-group. Regarding the PSO bridge, a simple variant, denoted in Fig. 2 as PSO Type B had the original two-column frame in the central bent (PSO Type A) with a modified, enlarged pile-cap configuration.

SIMULATION AND ANALYSIS OF THE BRIDGES

In order to investigate the contribution of SSI effects to the overall response of the system in the transverse direction, detailed three-dimensional frame models were implemented, in conjunction with nonlinear time history analyses (Fig. 3). Those parts of the structure that are expected to remain elastic during the analysis (the prestressed deck, end abutments, cap beam and pile caps) were modelled using linear elastic constitutive laws, whereas the critical subsystems such as the bent columns and piles were modelled using plastic hinges in all possible locations with elasto-plastic skeleton curves based on calculated moment-curvature envelopes.

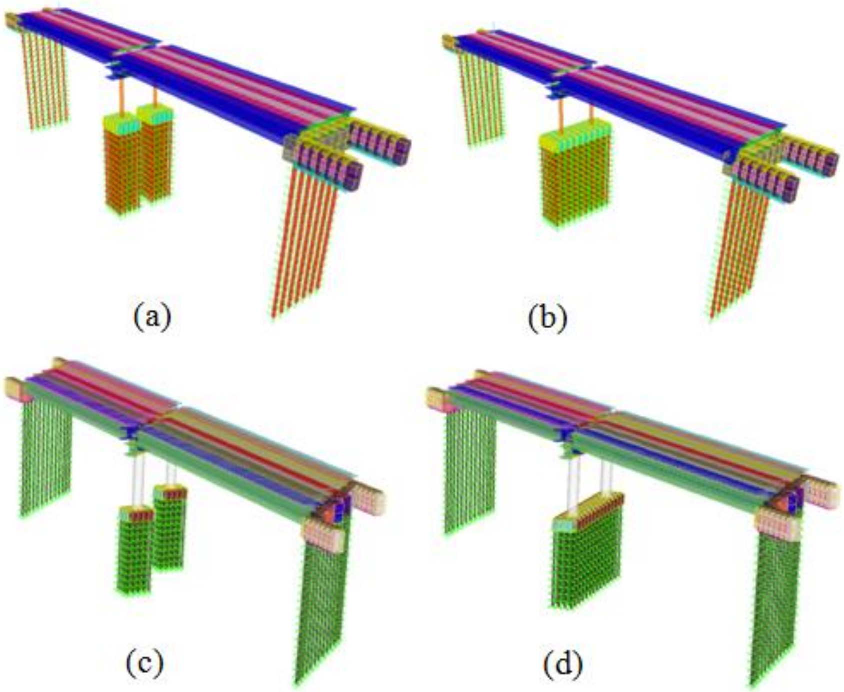


Figure 3. Implemented F.E. models (a,b) MRO and (c,d) PSO

Nonlinear response was simulated based on the implementation of lumped plasticity models. In this context, the plastic hinge length, ℓ_p, was taken according to, as:

$$\ell_p = 0.08 \cdot L + 0.022 \cdot D_b \cdot f_y \tag{3}$$

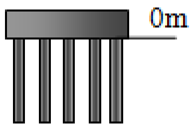
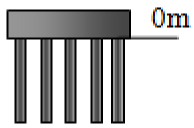
where L is the span of the member, D_b is the typical longitudinal bar diameter of the critical cross section and f_y the yield stress of the reinforcement (Priestley et al., 1996).

As it has been already mentioned, apart from the plastic hinge regions in the bridge structure, additional source of considerable nonlinearity in the response is the foundation soil. It is generally known that for cohesive soils increasing shear deformation leads to a commensurate reduction in the effective soil shear constant, G, resulting in significant nonlinearities in the calculated response. It is therefore essential that this effect be explicitly accounted for in the model, with consideration of any accompanying dynamic effects, such as inertial and kinematic soil-structure interactions.

In the model, inertial interaction effects were considered through the use of pertinent nonlinear springs of the Bouc-Wen type; constitutive laws were obtained from the G-γ and the equivalent viscous damping (G-ξ) relationships (Wen, 1976; Drosos et al., 2012). Foundation soil was modelled using non-linear springs spaced at a distance of 1m along the pile lengths, according with soil profile results obtained on site through investigative drilling (Table 2). Force-displacement (P-y) curves were developed for the springs from the (K-y) soil envelopes, based on relevant literature (Comodromos and Pitilakis, 2005; Brown et al. 1988). At this stage the pile interaction effects (within the pile group) were also considered.

To properly represent the mechanics of the bridge-system, it is critical to account for the kinematic and inertial interactions that occur between abutment and the supporting embankment soil. In this framework, embankment mobilization was accounted for in all bridge models according with Kotsoglou and Pantazopoulou, (2007). Using pertinent lumped masses and nonlinear springs in the F.E. model of the bridge, it was possible to account for the mass participation, additional stiffness and damping that are introduced into the system through these interactions. For a complete investigation of the problem each separate bridge type that was investigated in the study, was modelled considering all possible alternative support conditions in the central bent foundation, in order to illustrate the implications of foundation compliance: this includes the variants of (a) separate pile-caps supporting each pier column, (b) enlarged stiff pile-cap supporting the pier frame.

Table 2. Soil profiles for the MRO and the PSO case studies below grade (level 0 corresponds to the underside of the pile cap).

Meloland Street Overcrossing MRO		**Painter Street Overcrossing PSO**	
Depth in m 	**MRO Soil Profile [15]**	Depth in m 	**PSO Soil Profile [16]**
from 0-0.3m	Soft Sandy Clay	From 0-1.0m	Soft Brown clayey silt with sand
from 0.3-1.5m	Compact Fine Silty Sand Interbedded with stiff Silty Clay	From 1.0-3.1m	Slightly compact brown silty medium to fine sand with clay binder
from 1.5-3.6m	Slightly Compact Fine Silty Sand Interbedded with Stiff Silty Clay (Sand 50% Clay 50%)	From 3.1-4.7m	Slightly compact olive-brown silty fine sand with some clay binder
from 3.6-6.1m	Stiff Clay	From 4.7-5.8m	Compact olive sandy silt with clay binder
from 6.1-8.5m	Slightly Compact Silt to Fine Silty Sand Interbedded with Thin Stiff Clay Seams	From 5.8-7.2m	Very stiff brown clayey fine sand
from 8.5-10.7	Compact Fine Silty Sand	From 7.2-8.9m	Compact light brown silt
from 10.7-13.7	Stiff Clay Interbedded with Slightly Compact Fine Sand	From 8.9-11.0m	Compact to slightly compact light brown silty fine sand

ANALYSIS RESULTS

Detailed nonlinear time history analyses for the bridge models examined were conducted using ground acceleration records from two significant earthquake events that characterize the seismicity of the greater region of the bridge locations, namely: (a) the Petrolia, Rio Dell (1992) earthquake, with an epicenter which was located 29km away from the instrumented PSO overpass in the Cape Mendocino area, California, and (b) the Imperial Valley (1979) earthquake with an epicentre at just 0.5km away from the instrumented MRO bridge (Fig. 4a and Fig. 4b respectively).

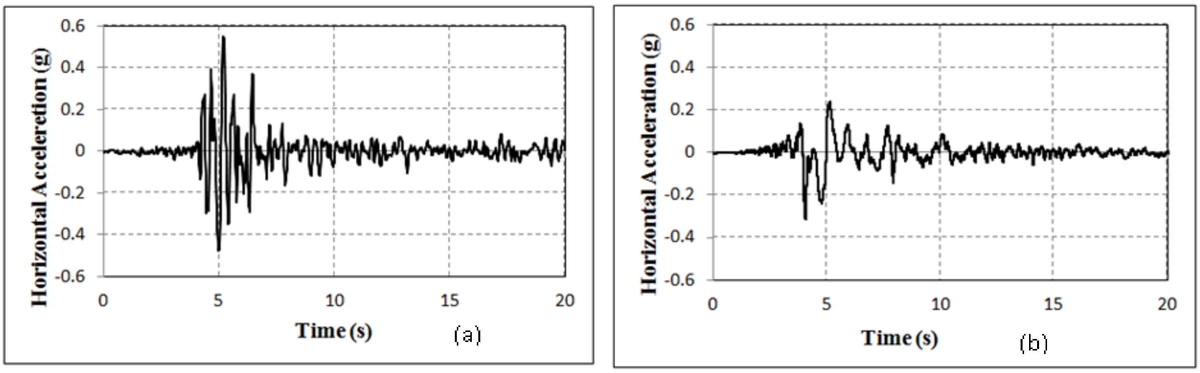
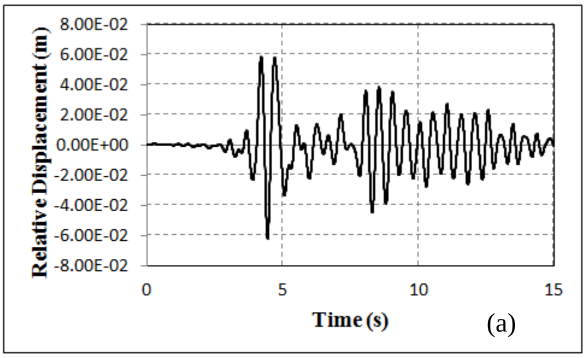


Figure 4. Ground acceleration records: (a) 1992 Petrolia earthquake (b) 1979 Imperial Valley

Figures 5 and 6 plot time-history results for the seismic response and plastic rotation demands of the central bent columns; the plots refer to the above mentioned Types of Bridges, while the contribution of the embankment flexibility is considered in Fig. 5. It is observed that embankment flexibility is overpowering all other effects, to the extent that the imposed displacement history of the pier is prescribed by the deck. The seismic performance of the MRO-Type A pier was significantly improved due to foundation rotation effects. As depicted in Fig. 7, plastic rotation demands in the pier were considerably decreased in the case of flexible foundations, at the expense of controlled foundation rotation. From the significant difference in the magnitude of plastic rotations occurring in the piers when comparing cases with and without embankment flexibility the "driving" role of the embankments is highlighted. For the same embankment condition, Fig. 7 indicates that in the case of stiff foundation systems, the displacement ductility demands are increased significantly.



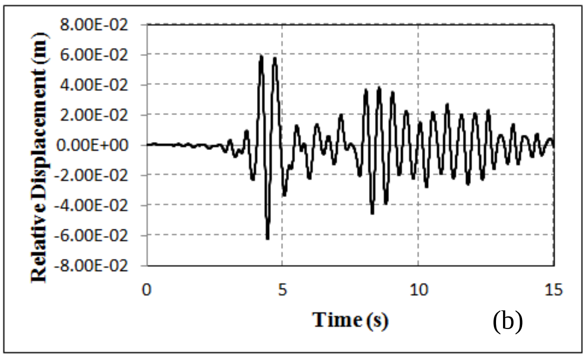
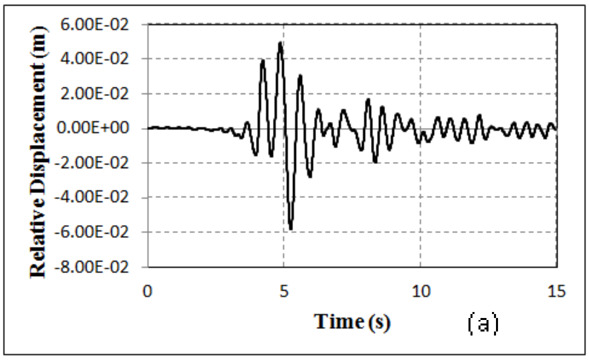


Figure 5. Pier relative displacement of the two MRO bridge variants, considering bridge-embankment interaction effects: (a) Type A- flexible foundation (b) Type B- Stiff foundation (enlarged pile-cap).



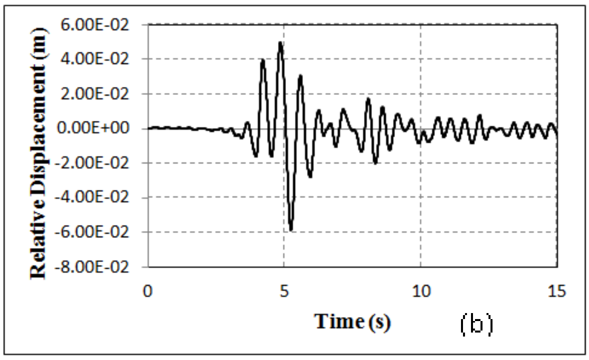


Figure 6. Pier relative displacement of the two MRO bridge variants, without bridge-embankment interaction effects: (a) Type A- flexible foundation (b) Type B- Stiff foundation (enlarged pile-cap).

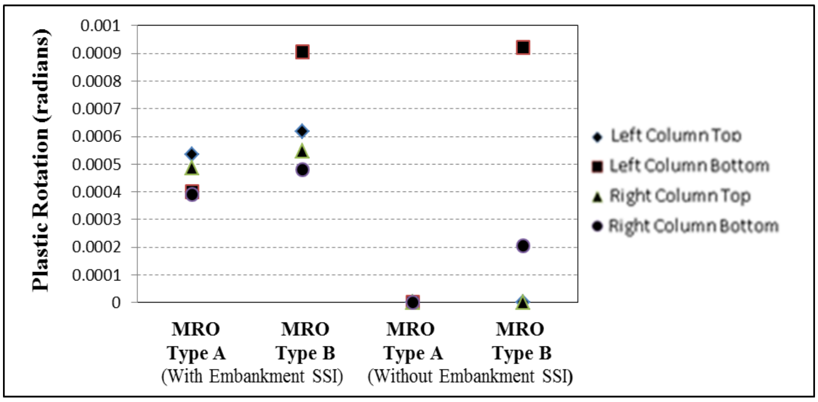


Figure 7. MRO (Types A and B): Plastic rotation for alternative bridge-foundation configurations

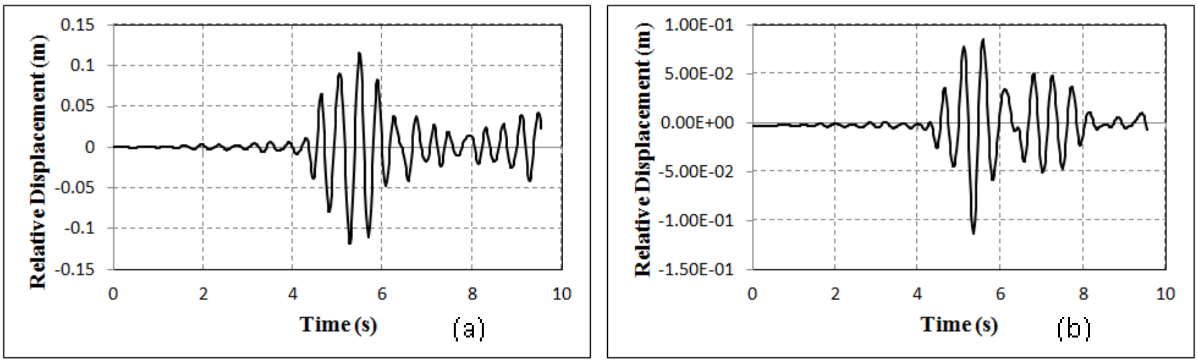


Figure 8 Pier relative displacement of the two PSO bridge variants, considering bridge-embankment interaction effects: (a) Type A- flexible foundation (b) Type B- Stiff foundation (enlarged pile-cap).

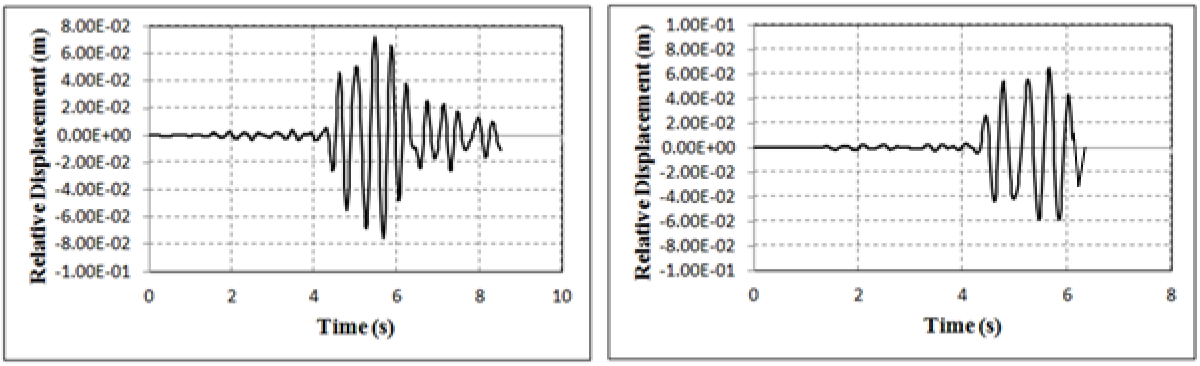


Figure 9 Pier relative displacement of the two PSO bridge variants, without bridge-embankment interaction effects: (a) Type A- flexible foundation (b) Type B- Stiff foundation (enlarged pile-cap).

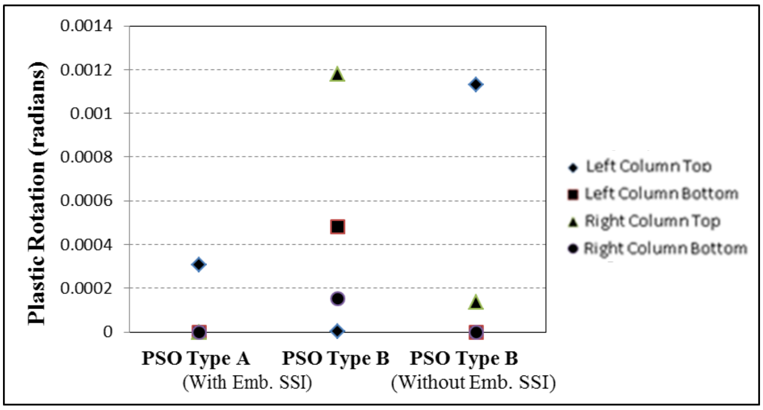


Figure 10. PSO Plastic rotation for alternative bridge-foundation configurations

Similarly the seismic response and plastic rotation of the central bent columns were estimated for the PSO Bridge (Fig. 8, 9). The seismic performance of the PSO pier was significantly improved due to foundation rotation effects. Again, plastic rotation demands were considerably decreased at the expense of a controlled amount of foundation rotation – this rotation lies within the performance limit C associated with the design earthquake in Table 1 for the soil and piers. With reference to Fig. 10, it may be easily concluded that in the case of stiff foundation systems, significantly increased displacement ductility demands would be anticipated in the piers.

The analysis results show conclusively that the intensity of displacement ductility demand is significantly decreased as a result of the implementation of flexible foundation strategies. On the other hand, when combining a stiff foundation, it served to contain all deformation demand in the column members thereby increasing the damage potential for the pier columns. It should be emphasized that in the case with flexible foundation, the computed rotation of the pile cap and soil-springs, remained below the prescribed soil deformation/pile settlement limits for the soil conditions under investigation.

CONCLUSIONS

The role of the foundation compliance in the seismic performance of the entire bridge system was investigated thoroughly in the present work. From response analyses, it became evident that alternative pile-cap configurations may lead to improved earthquake behaviour of the pier-foundation system; the concept exploits the SSI effects in more compliant foundations in order to reduce the deformation demands in the individual members of the superstructure by engaging more components of the bridge in inelastic activity. Flexible pile-caps that separately support each column, may have positive effects to the overall performance of the bridge, by mitigating the seismic demand to the soil, at the expense of some residual foundation soil deformation. Nevertheless, a wide range of critical indices and parameters should be considered in order to limit the anticipated nonlinearities within acceptable limits.

It was demonstrated through detailed time history analysis that stiff foundations introduce significant forces in the soil, causing larger rotations and damage (plastic hinges) in the upper end of the piles near the connection with the stiff pile caps, whereas flexible column designs may deliver the foundation from damage by undertaking larger share of the lateral drift demand without significant damage. Based on the preceding, separated pile-cap configurations under each bridge column performed better in both case studies considered. From the above it can be concluded that exploitation of the foundation flexibility is feasible through simple configuration strategies during initial design or even strengthening procedures, provided that rotation limits for the pile-groups at different levels of performance values lie within prescribed values. A note of caution is that superstructure seating hardware should be used in conjunction with pertinent stoppers to eliminate the risk of damage due to the large relative displacements at the more flexible pier-foundation substructures. Further investigation is warranted so as to further correlate the proposed soil deformation limit states with field results before this methodology be generally adopted in practical design.

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