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SITE EFFECTS AND SOIL-STRUCTURE INTERACTION ON AN ISOLATED HIGHWAY OVERCROSSING ON PILE FOUNDATIONS

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ABSTRACT

In this paper the effects of soil-structure interaction on the dynamic response of isolated overcrossings are investigated with reference to a case study, constituted by a real highway bridge. An accurate geotechnical characterization of the site is available from which the dynamic and mechanical properties of the soil are obtained. The seismic input is defined at the outcropping bedrock by means of real accelerograms and local response analyses are performed to compute the free-field input motions accounting for the non linear soil behaviour. The soil-structure interaction effects are evaluated on isolated and non-isolated bridges by comparing the seismic response of the structure with that obtained from a conventional fixed base model. Results demonstrate that the non-isolated bridge is more sensitive to soil-structure interaction effects, particularly in the transverse direction, where the pier stiffness is very high. Nevertheless, on the isolated bridge, soil-structure interaction produces an increase of the total shear strain in the isolators that should be taken in account in the verifications.

Keywords: Base isolation, bridges, pile foundations, site effects, soil-structure interaction

INTRODUCTION

The usual seismic design of structures is based on the ductility concept, namely on the structural capability of undertaking plastic deformations, without significant loss of resistance, and dissipating energy by hysteresis through structural damages. Even if the structure failure is prevented, generally significant repairs are necessary after strong earthquakes. In recent years the base isolation seismic design has becoming a popular strategy to reduce seismic structural damages. By uncoupling the structure and the ground, making use of isolators, suitably placed, it is possible to avoid the structure damage during strong earthquakes. This approach is generally adopted for strategic structures, such as bridges, for which the functionality after earthquakes is essential and repair costs may become important. In bridges, isolators are generally located at the top of piers and reduce the superstructure response by lengthening the fundamental periods and enhancing the energy dissipation. The performance of the devices depends on the shear strain that is a function of the relative displacements and rotations between the top and the bottom plates of the isolators, as well as of the axial force. Many works are available in the literature dealing with the seismic performance of isolated bridges (Ghobarah and Ali, 1988; Turkington et al., 1988; Saiidi et al., 1999; Tongaonkar and Jangid, 2000 among others) but only few of them account for the foundation flexibility (Thakkar and Maheshwari, 1995; Chaudhary et al., 2001; Tongaonkar and Jangid, 2003). Soil-Structure Interaction and site effects may be significant in the design of isolated

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bridges since the actual flexibility of the soil-foundation system, combined with site amplification effects, may modify the whole dynamic response and increase the isolators shear strains. In particular, depending on the site configuration, the actual amplification effects may lead to an increase of the spectral acceleration in correspondence of the fundamental period of the isolated structure while the soil-foundation flexibility may determine a significant rocking component of the seismic action that may increase the relative rotation of the isolator plates.

In this work the Soil-Structure Interaction (SSI) effects on the seismic response of isolated bridges are evaluated with reference to a case study constituted by a real highway overcrossing. Dynamic and mechanical properties of the soil are derived from an accurate geotechnical characterization of the site. According to a substructure approach, a numerical method for the SSI analysis of bridges is firstly presented. The kinematic interaction analysis of the soil-foundation system is performed in the frequency domain obtaining the foundation dynamic impedances and the Foundation Input Motions (FIMs). The seismic input is defined at the outcropping bedrock by means of real accelerograms and local response analyses are performed to compute the free-field input motions. The inertial interaction analysis of the superstructure is performed in the time domain by adopting suitable Lumped Parameter Models (LPMs) to account for the frequency dependent soil-foundation impedances. The procedure is adopted to investigate the effects of SSI on the seismic response of the case study, comparing the results with those derived from a conventional fixed base model. Furthermore, in order to highlight differences with non-isolated bridges, fixed base and SSI analyses are performed on the same bridge redesigned according to a conventional approach.

ADOPTED ANALYSIS METHODOLOGY

The SSI problem for a generic bridge founded on pile groups (Figure 1) is conveniently studied in the frequency domain, under the assumption that both soil and structure behave linearly. By adopting a finite element approach and according to the substructure method, the following partitioned system of complex linear equations governs the dynamics of superstructure (Figure 1a):

$$\left[\begin{array}{c:c} \boldsymbol{Z}_{SS} & \boldsymbol{Z}_{SF} \\ \hdashline \boldsymbol{Z}_{FS} & \boldsymbol{Z}_{FF}+\Im \end{array}\right]\left[\begin{array}{c} \boldsymbol{d}_S \\ \hdashline \boldsymbol{d}_F \end{array}\right]=\left[\begin{array}{c} \boldsymbol{0} \\ \hdashline \Im \boldsymbol{d}_{FIM} \end{array}\right] \tag{1}$$

in which $\boldsymbol{d}_S$ and $\boldsymbol{d}_F$ are the sub-vectors containing the nodal displacements of the superstructure and the foundations, while $\boldsymbol{d}_{FIM}$ is the vector containing the FIM, namely the displacements of the pile caps, evaluated from kinematic interaction analyses of the soil-foundation systems. The dynamic stiffness matrix of the system is partitioned consistently with the displacement vector and subscripts S and F are used to define sub-matrices relevant to the superstructure and the foundation. $\Im$ is the frequency dependent impedance matrix accounting for the foundation compliance. According to the substructure method, soil-foundation impedances and FIM are evaluated by considering a separate model for the soil and the foundations; in this paper the numerical method proposed by Dezi at al. (2009) is adopted (Figure 1b).

To take advantage of common computer codes, which generally allows only for time domain analyses, the SSI problem has been reformulated by introducing frequency independent quantities. For the superstructure the classical stiffness, damping and mass matrices are introduced while suitable LPMs are considered for the soil-foundation systems (Wolf, 1994). LPMs are characterized by frequency independent springs, dashpots and masses, conveniently assembled and calibrated so as to reproduce the frequency dependent impedances of the soil-foundation system.

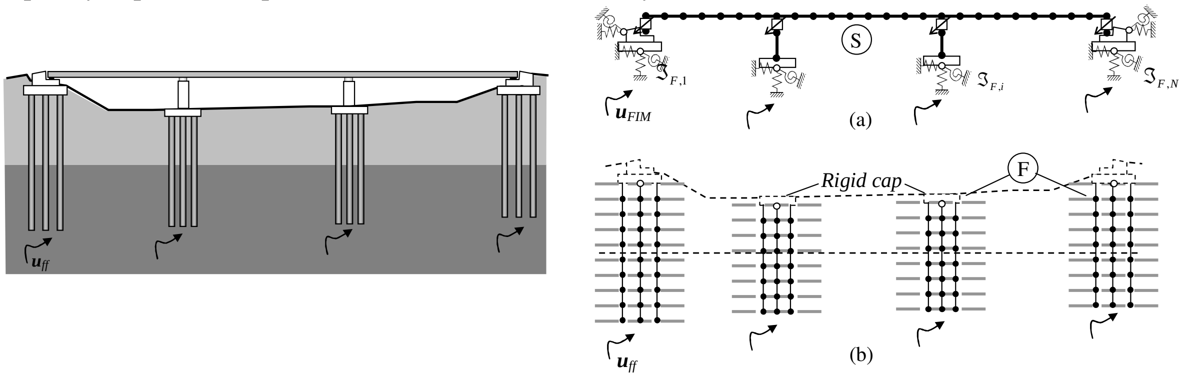


Figure 1. Substructure approach: (a) bridge on compliant base and (b) soil-foundation system

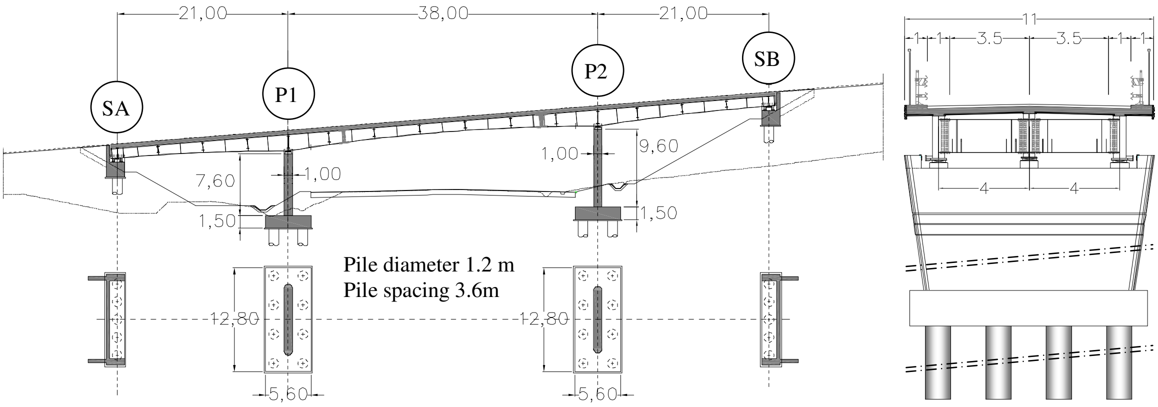


Figure 2. Lateral view and sections of the bridge

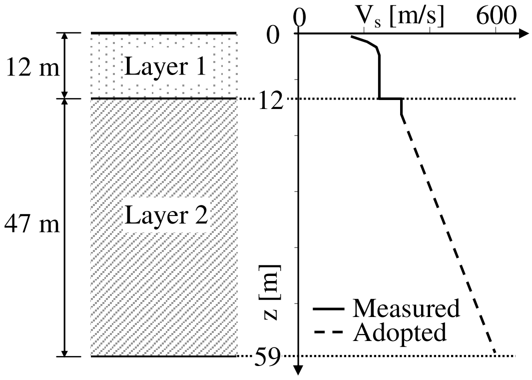
SSI EFFECTS ON AN ISOLATED HIGHWAY OVERCROSSING

The procedure is applied for the evaluation of the SSI effects on the seismic response of an isolated highway overcrossing. The bridge constitutes one of the new structures designed in the framework of the A14 Italian Highway third lane construction and is located in the Marche region (central Italy). The bridge static scheme is shown by Figure 2: the structure has a total length of 80 m and is divided into three spans of 21, 38 and 21 m. The steel-concrete composite deck is 11 m wide and is characterized by a 0.24 m thick slab and three steel I-shaped girders with variable high. Both piers are equipped with 3 Lead Rubber Bearings (LRBs) while multidirectional sliding bearings are placed at the abutments. The design structural period is about 1.5 s in order to avoid excessive displacements at the abutments and limit the cost of expansion joints. LRBs are characterized by an effective horizontal stiffness k_{ef} = 5500 N/mm and an equivalent damping factor ξ = 25%. The substructures are founded on 1.2 m diameter bored concrete piles as shown in the scheme of Figure 2.

The present-day geological configuration of the region can be represented by two main formations: a Plio-pleistocene marine deposite prevalently composed of marly clay with a thickness of hundreds of meters underlying a Quaternary (Holocene-Pleistocene) continental covering soil of variable thickness (generally not greater than 40 m) which mainly consists of alluvial, eluvial-colluvium and coastal deposits. In particularly, the area under study, where the bridge is located, completely lies on the marine Plio-

Pleistocene marly clay that crops out in this part of the coast and presents a weathered layer in its upper part.

(a)



	Layer 1	Layer 2
γ_t [Mg/m^3]	19÷20	20
φ' [°] / φ'_r [°]	26÷28 / 20÷22	
c' [kPa]	5	5÷10
OCR	6	>6
C_u [kPa]	75÷120	120+10·z

(b)

Figure 3. (a) Soil and V_s profiles and (b) soil properties estimated from available data

A large number of laboratory testes (such as oedometer, triaxial and simple shear test) and in situ testes (including dilatometer Marchetti, cone penetrometer, piezocone, standard penetrometer, multichannel analysis of surface waves and cross hole) have been carried out to characterize the soil. The soil profile deduced from the borehole investigations consists of two layers: a first layer of weathered marly clay with thickness of approximately 12 m and shear wave velocities V_s increasing for the first meters (3 m) up to a constant mean value of about 246 m/s; and a second layer of intact marly clay where the shear wave velocity is assumed to vary linearly with a gradient of 6.67 m/s/m. The seismic bedrock is fixed at a depth of 59 m from the ground surface, where the shear wave velocity reaches 600 m/s. The adopted V_s profile is shown by Figure 3a, while soil mechanical properties are reported by table in Figure 3b.

The region is a medium seismicity area, namely characterised by frequent events of medium local macroseismic intensity (6.5 – 7.5 MCS), due to seismic activity originating from the central part of the Appennino mountains where several seismogenetical sources are present especially close to the Adriatic coast (Valensise and Pantosti, 2001). By considering a return period of 475 years, the PGA-rock of 0.18 g and the relative response spectrum are deduced from the Italian seismic hazard map (INGV, 2004). Seven triplets (horizontal and vertical directions) of real accelerograms are selected from the European Strong Motion Database and used as input motion for the seismic site response analyses.

Due to the prevalently flat geomorphological characteristics of the site, and to the low-to-medium strain levels expected for medium intensity shakings, a 1D linear equivalent method has been adopted to investigate the seismic local response. The shear modulus degradation and damping ratio evolution curves (Figure 4d), used to account for the non-linear soil behaviour in the seismic local response analyses; have been assumed according to resonant column test results (Crespellani and Simoni, 2007).

Site response analysis and input motions

The free field motion within the deposit is obtained by means of a site response analysis. In particular, 1D wave propagation analyses are performed from the seismic bedrock to the ground surface to capture amplifications of the seismic signal. Figure 4a shows the response spectra of the selected accelerograms for the seismic input in the longitudinal and transverse directions and Figure 4b shows the relative response spectra obtained at ground surface from the site analyses. Finally, Figure 4c shows the mean spectra at the outcropping bedrock and at the ground surface for the horizontal directions compared with the relevant code reference spectra. Site amplification effects are evident in the frequency range 0.8÷5 Hz (period 0.2÷1.25 s) with maximum value attained at about 3.6 Hz (period 0.28 s). The time histories obtained from the site analyses are used directly in the kinematic interaction analyses to represent the

Free-Field Motion at the ground surface and at different depths within the deposit.

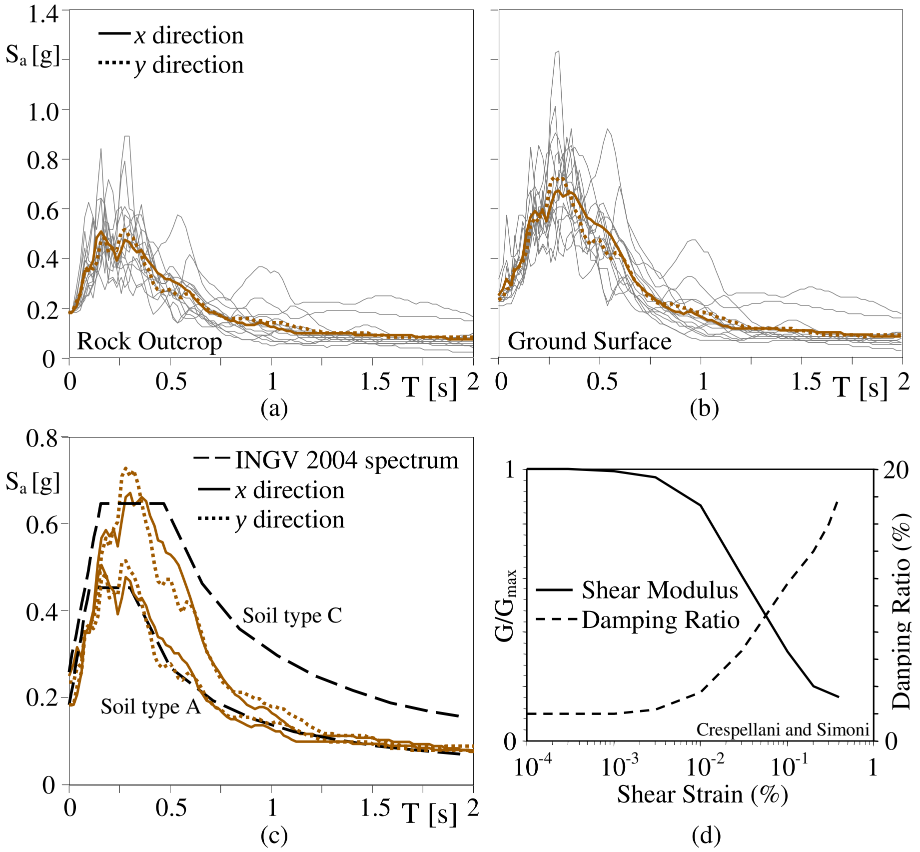


Figure 4. Response spectra (a) at outcropping bedrock; (b) at ground surface; (c) mean spectra compared with reference spectra and (d) normalized shear modulus and damping ratio curves

Kinematic interaction analysis

The kinematic interaction analysis is performed in the frequency domain by means of the numerical procedure proposed by Dezi et al. (2009). 1 m long finite elements are used to discretize each pile in order to guarantee a good level of accuracy for the results in terms of soil-foundation impedances and FIMs. The pile density is 2.5 Mg/m^3 and the modulus of elasticity is $2.4 \cdot 10^4$ N/mm^2 to take into account concrete cracking. Master nodes are introduced at the centroid of the rigid pile caps, at the level of pile heads. The transient seismic free field motion at each pile node is determined by means of 1D site analyses. Results of the kinematic interaction analysis consist of the frequency-dependent impedances for each foundation and the relevant FIMs. Considering the pile layout symmetry and collecting the master node displacements in the vector $[u_x, u_y, u_z, r_x, r_y, r_z]^T$, the impedance matrix of the generic pile group assumes the form

$$\Im = \begin{bmatrix} \Im_x & 0 & 0 & 0 & \Im_{x-ry} & 0 \\ & \Im_y & 0 & \Im_{y-rx} & 0 & 0 \\ & & \Im_z & 0 & 0 & 0 \\ & & & \Im_{rx} & 0 & 0 \\ & sym & & & \Im_{ry} & 0 \\ & & & & & \Im_{rz} \end{bmatrix}_i \tag{2}$$

where the only non-null coupling terms are the roto-translational components. Notice that as a consequence of the roto-translational coupling, also the FIM is characterized by translational and rotational components. Figure 5 shows, with continuous lines, the non-null components of the impedance matrix of the pier foundation. Furthermore, in Figure 6 the dimensionless displacement response factors

$$I_{u,x} = \frac{d_{FIM,x}}{d_{ff,x}} \qquad I_{u,y} = \frac{d_{FIM,y}}{d_{ff,y}} \tag{3a, b}$$

$$I_{\varphi,rx} = \frac{d_{FIM,rx} D}{d_{ff,y}} \qquad I_{\varphi,ry} = \frac{d_{FIM,ry} D}{d_{ff,x}} \tag{4a, b}$$

relevant to the pier foundation are shown. In equations (3) and (4), D is the pile diameter and subscripts x, y, rx and ry refer to the horizontal and rocking components of vectors $\boldsymbol{d}_{ff}$ and $\boldsymbol{d}_{FIM}$ which contain the free-field displacements at ground surface and the rigid cap displacements, respectively. It is worth noticing that, since the soil mechanical properties depend on the shear strain amplitudes, different response factors are associated to each group of accelerograms.

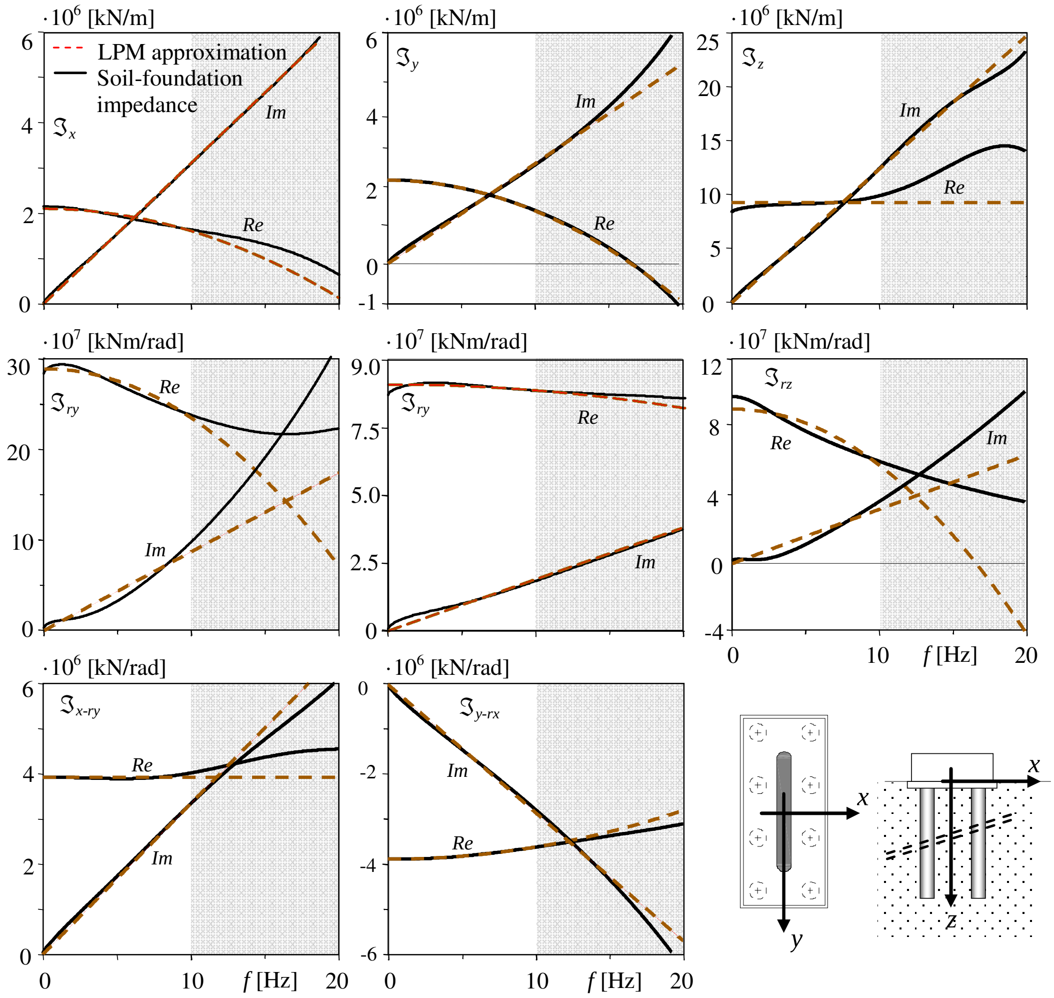


Figure 5. Non-null components of the impedance matrix of the pier soil-foundation system

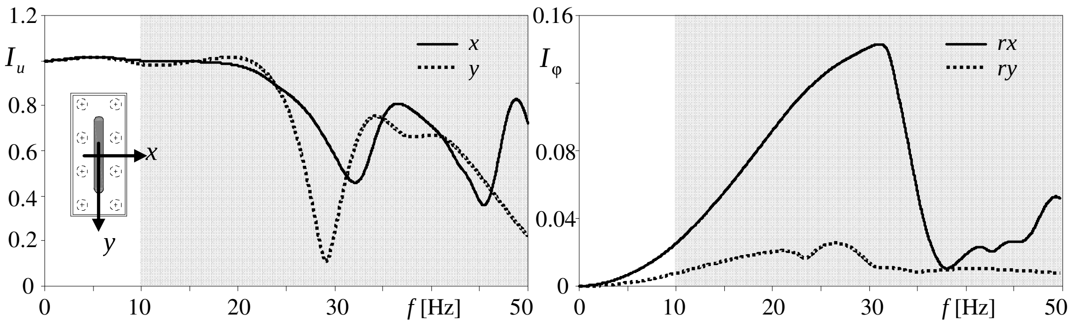
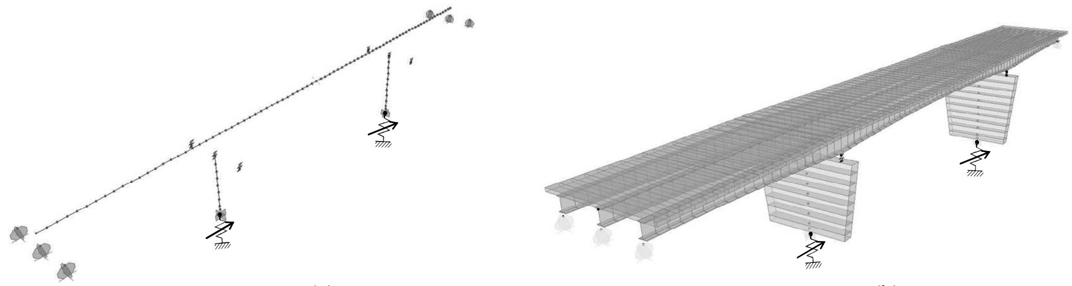


Figure 6. Dimensionless displacement response factors for one set of the selected accelerograms



(a) (b)

Figure 7. Finite element model: (a) line elements and (b) extruded view

Inertial interaction analysis

The inertial interaction analysis is carried out in the time domain by means of a three-dimensional finite element model of the bridge is developed in SAP2000® (Figure 7). Piers are considered to be beam elements with variable cross section in order to account for the actual geometry of the substructures. Beam elements are also used to model the steel-concrete composite deck by suitably reducing the inertia of the cross section concrete components and accounting for the change of steel plate thickness along the deck. The real position of the LRBs and of the multidirectional sliding bearings is taken into account by adopting rigid body constrains. Accounting for the device thickness, isolators are modelled with two-joint link elements for which a viscoelastic behaviour is defined in the bridge longitudinal and transversal directions. The effective horizontal stiffness of the devices k_{ef} is used to define the stiffness coefficients of the links while the damping coefficients are calibrated starting from the well known expression

$$c = \xi \frac{2k_{ef}}{\omega_0} \tag{5}$$

where ω_0 is assumed to be the fundamental frequency of the isolated bridge. Additional structural damping for the superior modes is introduced by the Rayleigh method.

The frequency dependent dynamic behaviour of the foundation is taken into account introducing a suitable LPM able to approximate the diagonal and out-of-diagonal terms. In this work, a 6 degree of freedom LPM is constructed for each soil-foundation system by assembling different LPMs as shown in Figure 8. The components of the impedance matrix of the adopted LPM are

$$\tilde{\Im}_\alpha(\omega) = \left[k_\alpha + k_{t\alpha} - \omega^2 (m_\alpha + m_{t\alpha})\right] + i\,\omega (c_\alpha + c_{t\alpha}) \qquad \alpha = x, y \tag{6a}$$

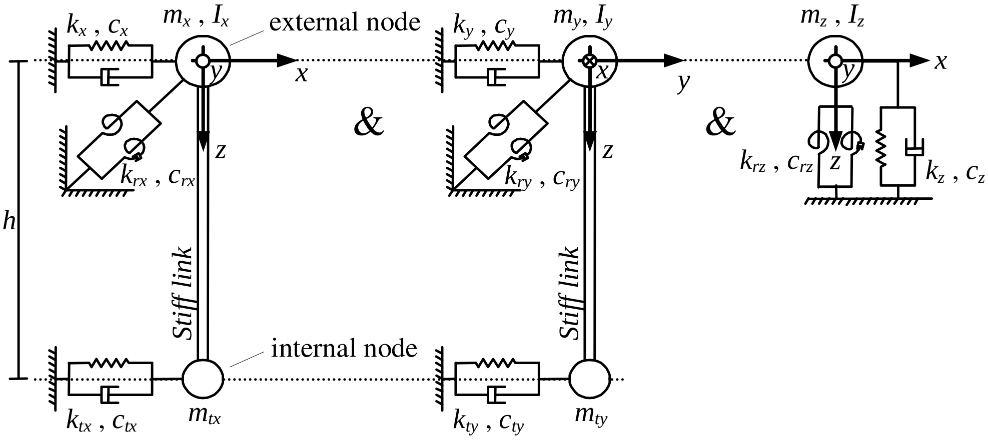


Figure 8. LPM for soil-foundation systems

$$\tilde{\Im}_{r\alpha}(\omega) = \left[k_{r\alpha} + k_{t\alpha}h^2 - \omega^2 I_\alpha\right] + i\,\omega\left(c_{r\alpha} + c_{t\alpha}h^2\right) \qquad \alpha = x, y \tag{6b}$$

$$\tilde{\Im}_z(\omega) = \left[k_z - \omega^2 m_z\right] + i\,\omega c_z \qquad \tilde{\Im}_{rz}(\omega) = \left[k_{rz} - \omega^2 I_z\right] + i\,\omega c_{rz} \tag{6c, d}$$

$$\tilde{\Im}_{x-ry}(\omega) = \left[k_{tx}h - \omega^2 m_{tx}h\right] + i\,\omega c_{tx}h \qquad \tilde{\Im}_{y-rx}(\omega) = \left[k_{ty}h - \omega^2 m_{ty}h\right] + i\,\omega c_{ty}h \tag{6e, f}$$

The 25 constants are calibrated with a least mean squares procedure in order to achieve the better approximation of the impedance functions previously evaluated, in the frequency range of 0÷10 Hz. Figure 5 shows, with dashed lines, the non-null components of the LPM impedance matrix.

SSI analyses of the Isolated (I) bridge are performed and the results are compared with those obtained from a conventional fixed base approach. Furthermore, in order to better appreciate SSI effects on the isolated structure, SSI analyses of the Non-Isolated (NI) bridge are also performed. A Fixed Base (FB) model, fully restrained at the base of the piers, and a Compliance Base (CB) model are developed for the I and NI bridges. The seismic input for the CB models is obtained starting from the FIMs given from the kinematic interaction analysis. For the FB models the seven triplets of real accelerograms, obtained at the ground surface from the local response analyses, are used as input motions.

Main results

The SSI effects on the overall bridge flexibility, on the maximum displacements and shear strains of the isolators as well as on the seismic base shear of piers are presented and discussed. SSI effects on the I and NI bridges are evaluated comparing results of the CB with those of the FB model; when possible the behaviours of the two different structures are also compared.

Structural deformability

The SSI effects on the overall structural deformability are investigated by means of steady-state analyses in which unit point loads are applied to the middle of the deck as depicted in Figure 9. Such analyses allow evaluating the fundamental frequencies of the structures consistently with their non-classically damped nature. Figure 9 shows the amplitude of the displacement response in the longitudinal, transverse and vertical directions, measured at the loaded point, and normalized with respect to the maximum values. With reference to the I bridge, SSI has no effect on the fundamental frequencies of the structure. On the contrary, in the case of NI bridge some important differences may be observed with reference to the horizontal response of the bridge; in particular, as a consequence of the high transverse stiffness of the piers, SSI modifies sensibly the fundamental frequencies in the transverse direction. This was not observed for the I bridge because of the isolation unit stiffnesses which are significantly lower than those of the foundations and substructures.

Isolators shear strains

The total shear strain γ in the isolators is computed summating the contributions of the total seismic displacement γ_s, the angular rotation γ_α and the axial compression γ_c, evaluated according to expressions

$$\gamma_s = d_E / t_e \tag{7a}$$

$$\gamma_\alpha = a^2 / 2 t_i t_e \tag{7b}$$

$$\gamma_c = 1.5 N_E / \left(S_1 G_{dyn} A_r \right) \tag{7c}$$

in which d_E is the maximum displacement of the isolator at the Ultimate Limit State (ULS), t_e is the total thickness of the elastomeric layers, t_i is the thickness of the single layer, N_E is the seismic axial force in the device and G_{dyn} is the equivalent dynamic shear modulus. Furthermore, a is an equivalent dimension which depends on the angular rotation ϕ, and S_1 and A_r are shape factors.

Figure 10a shows the isolator maximum displacements and rotations obtained from the seven SSI analyses (denoted by Ri, $i = 1\ldots7$) normalized with respect to the relevant values obtained from the FB model. Results refer to the edge isolators belonging to piers P1 and P2 that have different highs. SSI slightly influences the maximum relative displacements of the devices while significantly increases the maximum rotations as a consequence of the foundation rocking.

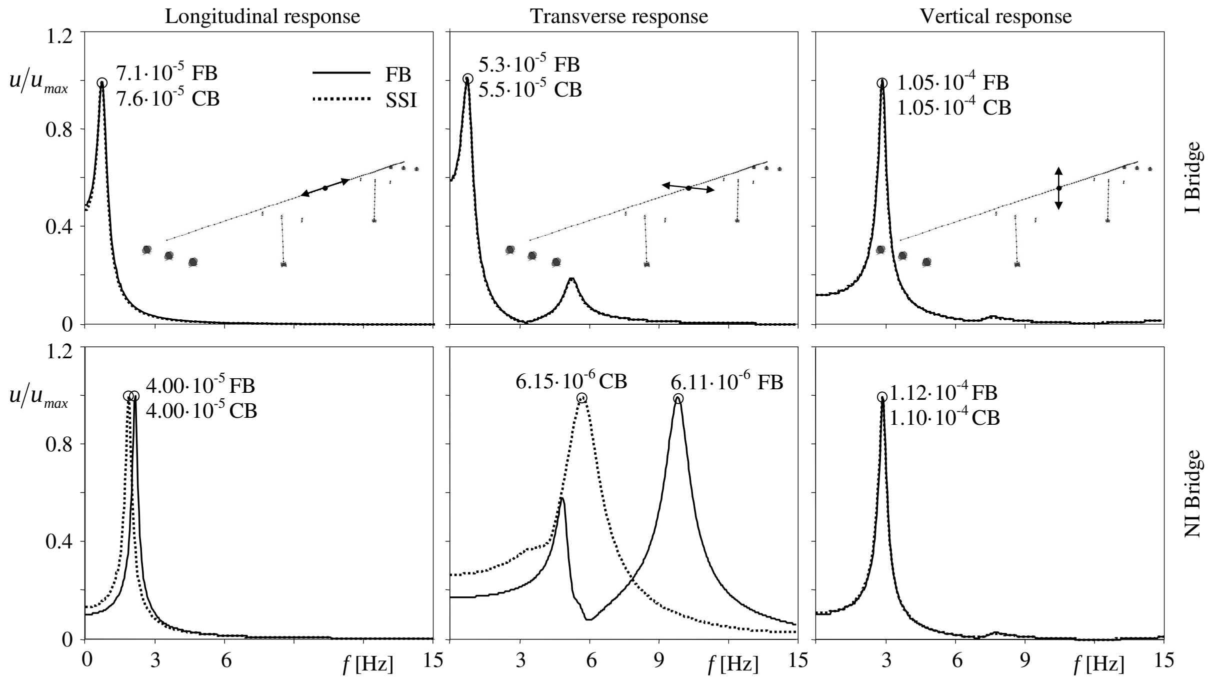


Figure 9. Steady-state displacement response

The mean value of the computed ratios is reported with dashed line for both displacements and rotations: concerning rotations a mean increase of about 75÷100% may be observed. Figure 10b shows the contributions to the shear strains of the same isolators due to displacements, rotations and axial force obtained from the FB and CB models. As expected, the contribution of the displacements is almost the same while an increase of the shear strains due to the device rotations is observed for the CB model.

Nevertheless, the contribution to the shear strain due to the axial force decreases in the case of the CB model so that the total shear strain obtained from the CB model are only slightly greater than those obtained from the FB structure.

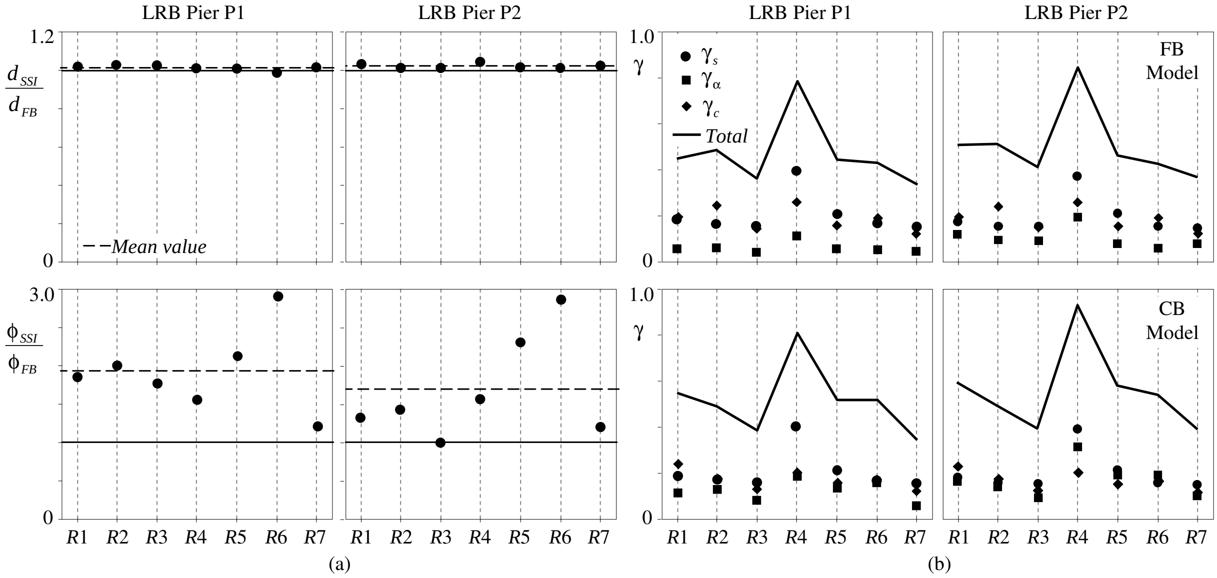


Figure 10. a) Isolator displacements and rotations; b) isolator shear strains

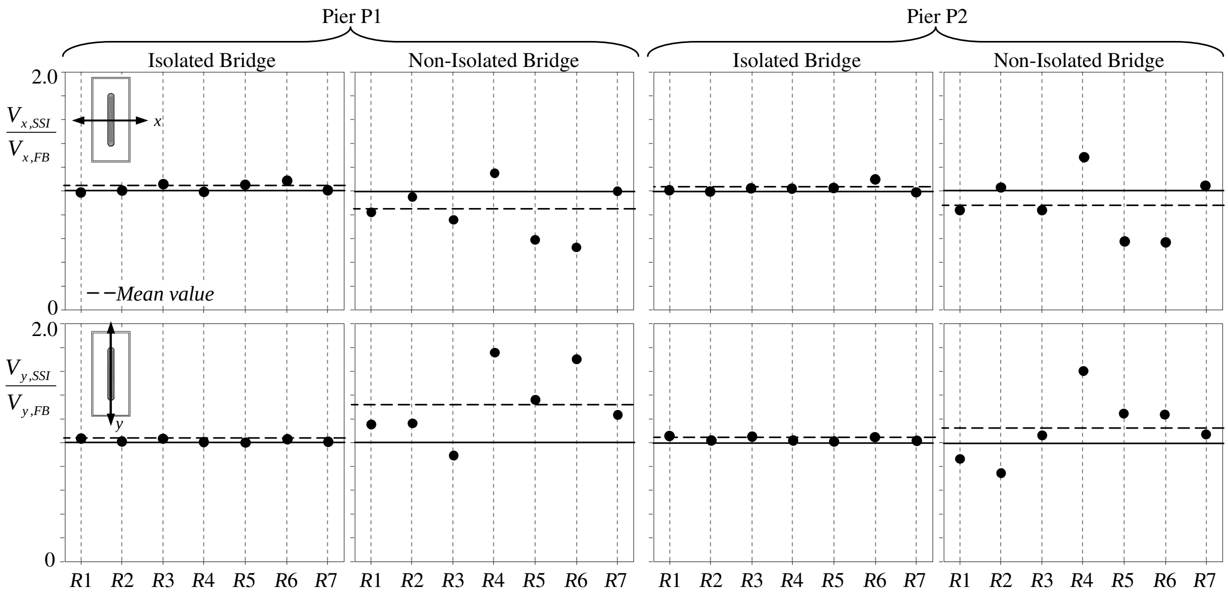


Figure 11. Pier base shears

Base shears

Figure 11 shows the pier maximum base shear, in the longitudinal and transverse directions, obtained from the CB model for the I and NI bridges, normalized with respect to the values obtained form the relevant FB models. The mean value of the ratios is reported in each graph with a dashed line. It is worth noticing that in the case of the I bridge the base shears obtained from the CB model are almost coincident with those obtained from the FB model for both the longitudinal and transverse directions; namely SSI slightly influences the dynamics of the bridge, as previously discussed. In the case of the NI bridge, SSI produces a significant modification of the pier shear. In particular, in the longitudinal direction a reduction of the base shear is observed in both piers while in the transverse direction an increase is evident (about 30% for pier P1); the increment of the fundamental periods due to SSI produces modification of the spectral accelerations that justify the pier base shear increments and decrements. Furthermore, it is worth noticing that results relevant to the NI bridge are more scattered than those relevant to the I bridge. For the NI bridge, differently from the I one, SSI modifies the bridge fundamental periods and consequently, depending on the seismic action, different spectral accelerations are responsible of the scattered values of the pier base shear ratio (Figure 11).

CONCLUSIONS

A numerical method for the Soil-Structure Interaction analysis of bridges has been presented taking advantage of the substructure technique and adopting a finite element approach. The kinematic interaction analysis of the soil-foundation system is performed in the frequency domain by means of the numerical method proposed by Dezi et al. (2009) while the inertial interaction analysis of the bridge is performed in the time domain by adopting suitable lumped parameter models to account for the frequency dependent soil-foundation impedances.

The procedure is adopted to investigate the effects of SSI on the seismic response of a real highway isolated overcrossing. A comprehensive geotechnical characterization of the site is available from which dynamic and mechanical properties of the soil profile are evaluated. 1D local response analyses are performed to include seismic site effects in the analyses.

SSI effects on the seismic response of the bridge are evaluated comparing the results with those derived from a conventional fixed base model. Furthermore, in order to highlight differences with non-isolated bridges, responses of fixed base and compliance base models of the non-isolated bridge are analyzed. Results demonstrate that the non-isolated bridge is more sensitive to SSI effects than the isolated one. The following observations have been drawn:

- foundation flexibility does not affect the fundamental frequencies of the isolated bridge but sensibly influences the overall dynamic behaviour of the non-isolated bridge, particularly in the transverse direction characterized by a high stiffness of the substructures;
- SSI slightly increases the maximum relative displacements in the isolators while significantly augments the angular rotations due to the foundation rocking; consequently, the related shear strains of the devices increase considerably;
- with reference to the isolated bridge, SSI does not affect sensibly the pier base shear in both the longitudinal and transverse directions; for the non-isolated bridge major effects are evident: an increase of the transverse base shear (about 30% for the pier P1) is observed while a decrease of the base shear is evident in the longitudinal direction, as a consequence of the foundation flexibility and the modification of the fundamental structural frequencies.

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