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INFLUENCE OF THE SOIL-STRUCTURE INTERACTION ON SEISMIC RESPONSE OF A RAILWAY BRIDGE

Dezi F.[1], Dall'Asta A.[2], Leoni G.[3], Scarpelli G.[4]

ABSTRACT

The paper presents a numerical procedure to evaluate kinematic and inertial interactions effect for pile groups by using a finite element approach. The method is used to calibrate Lumped Parameter Models (LPMs) which can be implemented in commercial finite elements programs to perform time domain analyses. The procedure is employed to study the spatial dynamic response of the railway bridge on the Tronto river (Marche region, Italy) that, due to the particular stiffness of substructures and deck, represents an interesting case study. The seismic analysis is carried out in the time domain considering several real accelerograms. The results are compared with those obtained from the fixed base model by showing the importance of Soil-Structure Interaction (SSI) in the actual design of the bridge.

Keywords: Soil-Structure Interaction, impedance functions, pile groups, bridges, real accelerograms, time domain analysis.

INTRODUCTION

As well known, the Soil-Structure Interaction (SSI) plays an important role in the evaluation of the seismic response of stiff structures, like short piers and abutments of bridges, where longitudinal or transversal restraint devices are located (Dezi 2006; Dezi and Scarpelli 2006). In these cases the usual hypothesis of fixed base model generally leads to conservative design solutions which cannot always be accepted for economical reasons. Furthermore modern technical codes suggest to take into account the SSI. Several numerical analytical methods were developed to compute the dynamic stiffness and the seismic response of pile foundations accounting for pile-soil-pile interaction. Under the assumption of linear behaviour for the soil and the structure, the analysis is usually carried out in the frequency domain. To perform structural non linear analyses it is necessary to work in the time domain. In such analyses Lumped Parameter Models (LPMs) are usually used to take into account the frequency dependent impedance of the foundation (Wolf 1988).

In this paper the influence of the SSI in the seismic response of a continuous four-span railway bridge is presented. The major span of the bridge is characterised by a bow-string and the whole deck is connected to one of the abutments in the longitudinal direction and to all the substructures in the transverse direction. The bridge considered is an interesting case study because it is characterised by very stiff substructures and the deck is also characterised by a certain transverse stiffness that could introduce remarkable interactions among the substructures under transverse seismic shakings.

The first section shows a numerical model for the analysis of the kinematic interaction of pile groups based on a finite element approach and on available theories to compute the soil-foundation dynamic impedances. The Domain Decomposition technique is used and the soil is assumed to be a Winkler type medium. In the second part the application to the case study of a railway bridge is shown. First

[1] PhD, Dept. of Physics and Engrg for Materials and Environment, Technical University of Marche Region, Email: francesca.dezi@tin.it
[2] Full Professor, Dept. PROCAM, University of Camerino, Email: andrea.dallasta@unicam.it
[3] Associate Professor, Dept. PROCAM, University of Camerino, Email: graziano.leoni@unicam.it
[4] Full Professor, Dept. of Physics and Engrg for Materials and Environment, Technical University of Marche Region, Email: g.scarpelli@univpm.it

the free field motion is derived starting from seven real accelerograms through a local site response analysis. Then LPMs are defined for the foundations by calibrating the parameters with the impedance functions calculated with the previous model. Finally, the response in the time domain of a 3D finite element model of the bridge is presented by showing comparisons with the case of the fixed base model.

SOIL - PILE FOUNDATION - STRUCTURE INTERACTION ANALYSIS

The interaction in the soil-foundation domain is performed by considering the model proposed by Kaynia and Kausel (1982) where the piles are assumed to be beam elements resting on a Winkler type medium which allows to describe dynamic soil-pile interaction and the dynamic interaction among the various piles constituting the foundation. The main assumption is that each pile interacts with the soil and the other piles of the foundation by considering the soil constituted by independent layers.

Soil-pile group kinematic interaction

A group of n circular piles having length L and diameter ϕ is considered. Each pile is assumed to be a flexurally and axially deformable beam with negligible torsional stiffness. The equilibrium condition of the pile group (Figure 1c) can be expressed in weak form by the Lagrange D'Alembert principle that, in the frequency domain, provides the following equation:

$$\int_0^L \boldsymbol{K}_P \mathsf{D}\boldsymbol{u}(\omega; z)\cdot \mathsf{D}\hat{\boldsymbol{u}} dz - \int_0^L \boldsymbol{r}(\omega; z)\cdot \hat{\boldsymbol{u}} dz - \omega^2 \int_0^L \boldsymbol{M}\boldsymbol{u}(\omega; z)\cdot \hat{\boldsymbol{u}} dz = 0 \quad \forall \hat{\boldsymbol{u}} \neq \boldsymbol{0} \tag{1}$$

where ω is the circular frequency and $\boldsymbol{u}$ and $\boldsymbol{r}$ are the vectors obtained by grouping together the total displacements of piles and the resultants of the soil reactions

$$\boldsymbol{u}^T(\omega; z) = \begin{bmatrix} \boldsymbol{u}_1^T & \cdots & \boldsymbol{u}_p^T & \cdots & \boldsymbol{u}_n^T \end{bmatrix} \qquad \boldsymbol{r}^T(\omega; z) = \begin{bmatrix} \boldsymbol{r}_1^T & \cdots & \boldsymbol{r}_p^T & \cdots & \boldsymbol{r}_n^T \end{bmatrix} \tag{2}$$

Furthermore, denoting by A and I the area and the moment of inertia of the pile cross section, and by E and ρ the Young modulus and the density of the material constituting the piles, the local stiffness and mass matrixes are $\boldsymbol{K}_P$ and $\boldsymbol{M}$

$$\boldsymbol{K}_P = \begin{bmatrix} \boldsymbol{K}_{11} & \cdots & \boldsymbol{0} & \cdots & \boldsymbol{0} \\ \vdots & & \vdots & & \vdots \\ \boldsymbol{0} & \cdots & \boldsymbol{K}_{pp} & \cdots & \boldsymbol{0} \\ \vdots & & \vdots & & \vdots \\ \boldsymbol{0} & \cdots & \boldsymbol{0} & \cdots & \boldsymbol{K}_{nn} \end{bmatrix} \qquad \boldsymbol{K}_{pp} = E \begin{bmatrix} I & 0 & 0 \\ 0 & I & 0 \\ 0 & 0 & A \end{bmatrix} \tag{3}$$

$$\boldsymbol{M} = \begin{bmatrix} \boldsymbol{M}_{11} & \cdots & \boldsymbol{0} & \cdots & \boldsymbol{0} \\ \vdots & & \vdots & & \vdots \\ \boldsymbol{0} & \cdots & \boldsymbol{M}_{pp} & \cdots & \boldsymbol{0} \\ \vdots & & \vdots & & \vdots \\ \boldsymbol{0} & \cdots & \boldsymbol{0} & \cdots & \boldsymbol{M}_{nn} \end{bmatrix} \qquad \boldsymbol{M}_{pp} = \rho \begin{bmatrix} A & 0 & 0 \\ 0 & A & 0 \\ 0 & 0 & A \end{bmatrix} \tag{4}$$

Finally, D is the formal differential operator that provides the pile overall deformations. Under the assumption of preservation of the plane cross section and of negligible shear strains, it is defined as

$$\mathsf{D}\boldsymbol{u}^T = \begin{bmatrix} \tilde{\mathsf{D}}\boldsymbol{u}_1^T & \cdots & \tilde{\mathsf{D}}\boldsymbol{u}_p^T & \cdots & \tilde{\mathsf{D}}\boldsymbol{u}_n^T \end{bmatrix} \qquad \tilde{\mathsf{D}}\boldsymbol{u}_p^T = \begin{bmatrix} \dfrac{\partial^2 u_1}{\partial z^2} & -\dfrac{\partial^2 u_2}{\partial z^2} & \dfrac{\partial u_3}{\partial z} \end{bmatrix}_p \tag{5}$$

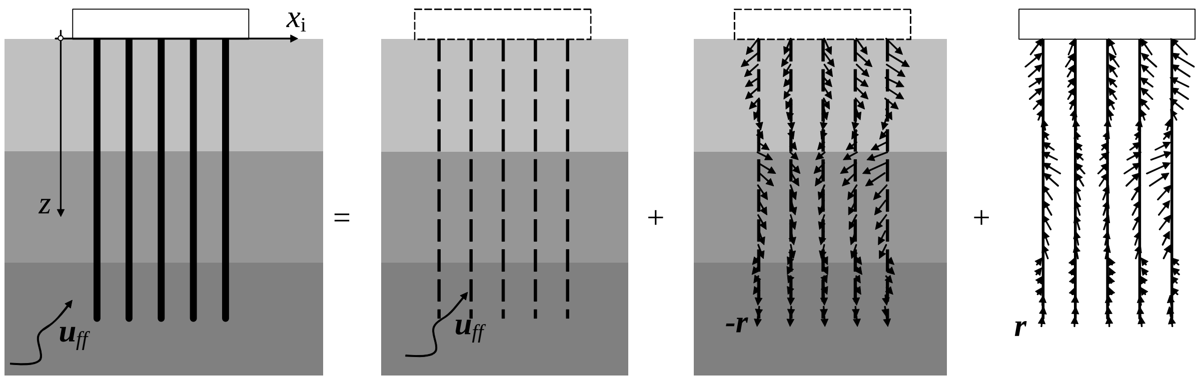

Figure 1. Decomposition of the domain: a) free field; b) soil subject to the interaction forces; c) foundation subject to the interaction forces

Equation (1) is of general validity since no assumption was made on the soil reaction forces. According to the Domain Decomposition procedure, the reaction forces can be expressed as functions of the unknown displacements and of the incident free field motion of the soil through the definition of the soil impedance operator (Wolf 1987).

By assuming the soil as a Winkler type media, namely represented through independent horizontal layers, and under the assumption of linear elastic behaviour, the soil impedance can be obtained starting from the relationship

$$\boldsymbol{u}(\omega;z)=\boldsymbol{u}_{ff}(\omega;z)-\boldsymbol{D}(\omega;z)\boldsymbol{r}(\omega;z) \tag{6}$$

where

$$\boldsymbol{u}_{ff}(\omega;z)=\begin{bmatrix}\boldsymbol{u}_{1\,ff}\\ \vdots\\ \boldsymbol{u}_{p\,ff}\\ \vdots\\ \boldsymbol{u}_{n\,ff}\end{bmatrix} \quad \text{and} \quad \boldsymbol{D}(\omega;z)=\begin{bmatrix}\boldsymbol{D}_{11} & \cdots & \boldsymbol{D}_{1q} & \cdots & \boldsymbol{D}_{1n}\\ \vdots & & \vdots & & \vdots\\ \boldsymbol{D}_{p1} & \cdots & \boldsymbol{D}_{pq} & \cdots & \boldsymbol{D}_{pn}\\ \vdots & & \vdots & & \vdots\\ \boldsymbol{D}_{n1} & \cdots & \boldsymbol{D}_{nq} & \cdots & \boldsymbol{D}_{nn}\end{bmatrix} \tag{7}$$

are the vector of the incident free field motion and the matrix of the elastodynamic Green's functions depending on the soil profile, respectively. Equation (6) expresses the soil displacements along each pile alignment by superimposing the free field motion and the displacements induced by the soil-pile interaction forces (Figure 1a and b). This approach was adopted by many authors: Fan at al (1991), Gazetas (1984), Kaynia and Kausel (1982). Here the Green's functions are taken from the simplified wave propagation theory proposed by Dobry and Gazetas (1988) and by Makris and Gazetas (1992; 1993). Since matrix $\boldsymbol{D}$ is not singular, the soil impedance matrix results $\boldsymbol{K}_S(\omega;z)=\boldsymbol{D}(\omega;z)^{-1}$, and consequently the reactions from the soil can be expressed from

$$\boldsymbol{r}(\omega;z)=-\boldsymbol{K}_S(\omega;z)\left[\boldsymbol{u}(\omega;z)-\boldsymbol{u}_{ff}(\omega;z)\right] \tag{8}$$

By substituting (8) into (1), the following solving equation is obtained

$$\int_0^L \boldsymbol{K}_P \mathrm{D}\boldsymbol{u}\cdot\mathrm{D}\hat{\boldsymbol{u}}dz+\int_0^L \boldsymbol{K}_S\boldsymbol{u}\cdot\hat{\boldsymbol{u}}dz-\omega^2\int_0^L \boldsymbol{M}\boldsymbol{u}\cdot\hat{\boldsymbol{u}}dz=\int_0^L \boldsymbol{K}_S\boldsymbol{u}_{ff}\cdot\hat{\boldsymbol{u}}dz \quad \forall\hat{\boldsymbol{u}}\neq\boldsymbol{0} \tag{9}$$

which permits calculating the response $\boldsymbol{u}$ of the pile group subjected to the free field motion $\boldsymbol{u}_{ff}$.

Finite element solution

The problem is solved by the Finite Element Method dividing each pile into e elements and approximating the pile motion and the free field motion within the elements by interpolating the displacements at the end nodes with third order polynomials for transverse displacements and second order polynomials for axial displacements. Five degrees of freedom are thus associated to each node of the mesh, namely three translations and two rotations, which are grouped in the nodal vector $\boldsymbol{d}_i^{\,T} = [u_1, u_2, u_3, \varphi_1, \varphi_2]_i$. The twisting rotation is not considered, consistently with the assumption of null torsional stiffness, to avoid singular problems. The displacement fields can be expressed as

$$\boldsymbol{u}_p(z;\omega) \cong \boldsymbol{N}(z)\boldsymbol{d}_p^e(\omega) \qquad \boldsymbol{u}_{p\,ff}(z;\omega) \cong \boldsymbol{N}(z)\boldsymbol{d}_{ff}^e(\omega) \tag{10}$$

where $\boldsymbol{N}$ is the matrix of the interpolating polynomials and $\boldsymbol{d}_p^e$ and $\boldsymbol{d}_{ff}^e$ are the vectors of the pile and free field element nodal displacements respectively

$$\boldsymbol{d}_{p,ff}^{e\;\;T} = [u_{1h}, u_{2h}, u_{3h}, \varphi_{1h}, \varphi_{2h}, u_{1k}, u_{2k}, u_{3k}, \varphi_{1k}, \varphi_{2k}]_{p,ff} \tag{11}$$

By substituting the previous equations into (9) and by suitably assembling the node displacements in a unique displacement vector, standard considerations allow obtaining

$$\left(\overline{\boldsymbol{K}}_P - \omega^2\overline{\boldsymbol{M}} + \overline{\boldsymbol{K}}_S\right)\boldsymbol{d} = \overline{\boldsymbol{K}}_S\boldsymbol{d}_{ff} \tag{12}$$

where $\boldsymbol{d}$ and $\boldsymbol{d}_{ff}$ are the vectors grouping the nodal total displacements and the nodal free field displacements, respectively. Furthermore,

$$\overline{\boldsymbol{K}}_P = \sum_{e=1}^{E}\int_0^{L_e}(\mathrm{D}\boldsymbol{N})^T\boldsymbol{K}_P(\mathrm{D}\boldsymbol{N})dz \qquad \overline{\boldsymbol{M}} = \sum_{e=1}^{E}\int_0^{L_e}\boldsymbol{N}^T\boldsymbol{M}\boldsymbol{N}dz \qquad \overline{\boldsymbol{K}}_S = \sum_{e=1}^{E}\int_0^{L_e}\boldsymbol{N}^T\boldsymbol{K}_S\boldsymbol{N}dz \tag{13}$$

are the global stiffness matrix of the piles, the global mass matrix of the piles, and the global impedance matrix of the soil, obtained by assembling the relevant contributions of all the elements.

When piles are connected at the head to a rigid cap, a constraint has to be imposed to the *dof* of the pile head. This is obtained by introducing a master node to which associate the cap rigid motion described by the vector $\boldsymbol{U}_F^{\,T} = [U_1, U_2, U_3, \Phi_1, \Phi_2, \Phi_3]$ where U_i are displacements components and Φ_i are rotations, is assigned. The constraint can be expressed analytically as $\boldsymbol{d} = \boldsymbol{A}\tilde{\boldsymbol{d}}$ where $\tilde{\boldsymbol{d}}$ is the vector of the displacements necessary to describe the foundation motion constituted by the six generalised displacements of the rigid cap and by the displacements at the nodes of the embedded pile sections, and $\boldsymbol{A}$ is the geometric matrix of the rigid constraint. The application of the constraint results in the equation

$$\boldsymbol{A}^T\left(\overline{\boldsymbol{K}}_P - \omega^2\overline{\boldsymbol{M}} + \overline{\boldsymbol{K}}_S\right)\boldsymbol{A}\tilde{\boldsymbol{d}} = \boldsymbol{A}^T\overline{\boldsymbol{K}}_S\boldsymbol{d}_{ff} \tag{14}$$

that can be written in the concise form

$$\tilde{\boldsymbol{K}}\tilde{\boldsymbol{d}} = \tilde{\boldsymbol{f}} \tag{15}$$

where $\tilde{\boldsymbol{K}}$ is the dynamic stiffness matrix of the foundation and $\tilde{\boldsymbol{f}}$ is the vector of generalized applied forces due to the free field motion.

Kinematic interaction of the foundation can be easily studied by partitioning equation (15) as follows:

$$\left[\begin{array}{c|c}\tilde{\boldsymbol{K}}_{FF} & \tilde{\boldsymbol{K}}_{FE}\\ \hline \tilde{\boldsymbol{K}}_{EF} & \tilde{\boldsymbol{K}}_{EE}\end{array}\right]\left[\begin{array}{c}\boldsymbol{U}_F^k\\ \hline \tilde{\boldsymbol{d}}_E^k\end{array}\right] = \left[\begin{array}{c}\tilde{\boldsymbol{f}}_F\\ \hline \tilde{\boldsymbol{f}}_E\end{array}\right] \tag{16}$$

By calculating the displacements of the embedded part of the foundation $\tilde{\boldsymbol{d}}_E^k$ from the second row and by substituting into the first, the following equation is obtained:

$$\left(\tilde{\boldsymbol{K}}_{FF} - \tilde{\boldsymbol{K}}_{FE}\tilde{\boldsymbol{K}}_{EE}^{-1}\tilde{\boldsymbol{K}}_{EF}\right)\boldsymbol{U}_F^k = \tilde{\boldsymbol{f}}_F - \tilde{\boldsymbol{K}}_{FE}\tilde{\boldsymbol{K}}_{EE}^{-1}\left(\tilde{\boldsymbol{f}}_E\right) \tag{17}$$

from which

$$\Re = \left(\tilde{\boldsymbol{K}}_{FF} - \tilde{\boldsymbol{K}}_{FE}\tilde{\boldsymbol{K}}_{EE}^{-1}\tilde{\boldsymbol{K}}_{EF}\right) \tag{18}$$

is the foundation impedance matrix which synthetically represents the entire foundation system behaviour and can be used for the analysis of any complex superstructures on pile foundations.
The Foundation Input Motion (FIM), namely the motion filtered by the specific foundation which has to be imposed to the structure, can also be obtained by solving equation (17)

$$\boldsymbol{U}_F^k = \Re^{-1}\left[\tilde{\boldsymbol{f}}_F - \tilde{\boldsymbol{K}}_{FE}\tilde{\boldsymbol{K}}_{EE}^{-1}\left(\tilde{\boldsymbol{f}}_E\right)\right] \tag{19}$$

and the stresses in the piles accounting for kinematic interaction can be calculated from the constitutive relationship $\boldsymbol{S}^k = \boldsymbol{K}_P \mathrm{D}\boldsymbol{u}^k$, once the displacements are approximated by equation (10).

Inertial interaction in the time domain
The foundation impedance $\Re$ is a 6x6 matrix which can be split in its real and imaginary parts

$$\Re(\omega) = Re[\Re](\omega) + i\, Im[\Re](\omega) \tag{20}$$

As its components are functions of ω it cannot be used in the usual time domain analysis performed by commercial computer codes. To this purpose, the effects of the structure-foundation interaction can be introduced in the structural scheme though a lumped parameter model, constituted by springs, dashpots and masses with constant parameters, having dynamic impedances which well approximate those of the real foundation, although in a limited range of frequencies. The foundation input motion to the system is thus constituted by the Inverse Fourier Transform of $\boldsymbol{U}_F^k$ determined in (19). After the structural analysis is performed, the effects of the inertial interaction on the foundation can be computed by substituting the Fourier Transform of the obtained foundation displacements $\boldsymbol{U}_F$ into the following equation:

$$\tilde{\boldsymbol{d}}_E^i = \tilde{\boldsymbol{K}}_{EE}^{-1}\tilde{\boldsymbol{K}}_{EF}\left(\boldsymbol{U}_F^k - \boldsymbol{U}_F\right) \tag{21}$$

where $\tilde{\boldsymbol{d}}_E^i$ are the displacements of the embedded parts of the piles due to the inertial interaction. Subsequently, by approximating the displacements within each element by (10), the stress resultants in the piles can be obtained from the constitutive relationship $\boldsymbol{S}^i = \boldsymbol{K}_P \mathrm{D}\boldsymbol{u}^i$. In order to obtain the complete stress state of the piles, the stress resultants evaluated in this step have to be added to $\boldsymbol{S}^k$ previously determined.

A CASE STUDY

The railway bridge on the Tronto river is located in the Marche region, central Italy, few km away from the town of San Benedetto del Tronto. The structure has a total length of 175 m divided into four spans of 30, 75, 40 and 30 m. The deck is 14.5 m wide and it is composed of two parallel steel I-shaped girders 2600 mm high connected by cross girders with a spacing of 0.55 m and a 0.7 m thick concrete slab (Figure 2). The steel tubular arches of the bow-string span have a box cross-section with dimensions of 1550 mm x 1830 mm, 30 mm thick. All the supports are fixed in the transverse

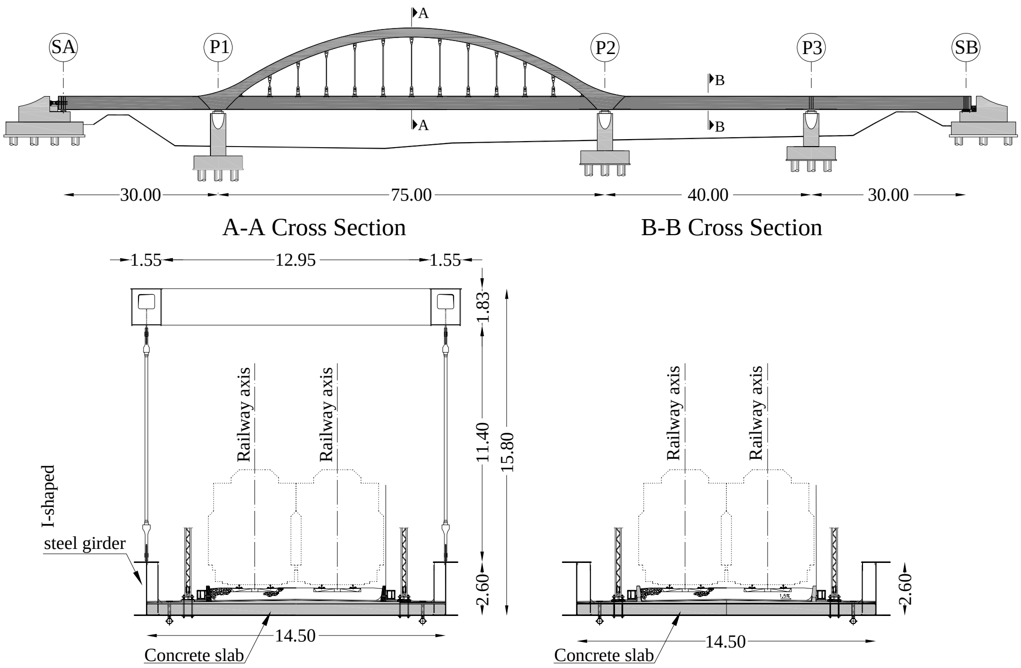

Figure 2. Lateral view and sections of the bridge

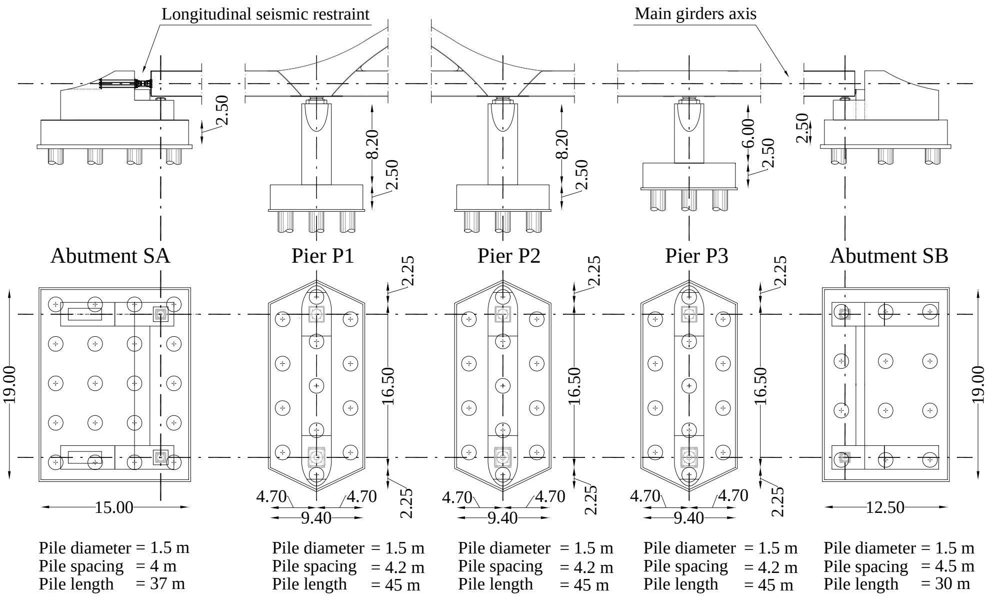

Figure 3. Details of the substructures and layouts of the pile groups

direction while they allow longitudinal displacements so that all substructures contribute equilibrating transverse seismic action. In the longitudinal direction the full seismic action is entrusted to abutment SA by seismic restraints. The substructures are founded on driven concrete piles according to schemes reported in Figure 3.

To study the bridge response to earthquake loading, geotechnical investigations were carried out throughout the foundations soils. Specifically, standard penetration tests were carried out and laboratory tests were performed in order to obtain the soil profile (Figure 4) and the geotechnical characterisation of the site: the alluvial soil deposits located over the bedrock formation reach the maximum thickness of 30 m and are made of gravel, sand and cobbles of clear fluvial origin alternate to clay and sandy clay of colluvial origins. The seismic characterization of the layers was established on the basis of tests performed in boreholes (standard penetration, vane shear) as well as cross-hole and surface seismic measurements performed in the proximity of the site. In particular a shear wave velocity of 200-250 m/s was obtained for all the alluvial deposits except for the first 2-3 m where

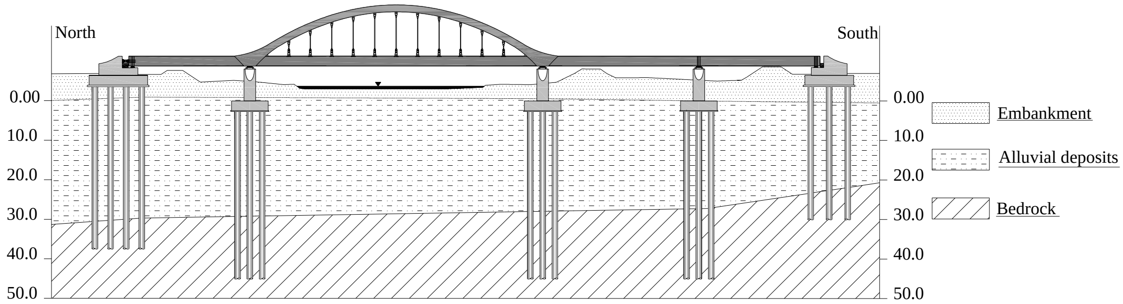

Figure 4: Soil profile

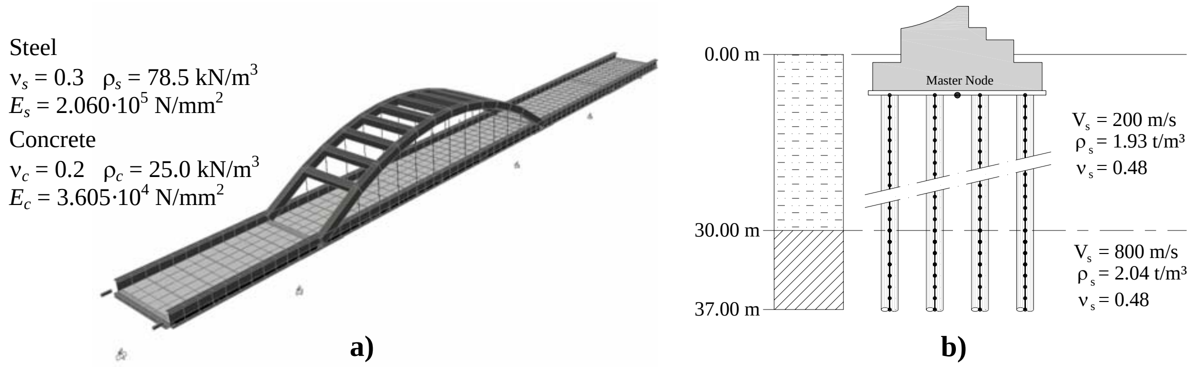

Figure 5: Structural models: a) superstructure; b) foundation and soil profile

values less than 180 m/s were found. The bedrock formation has an average shear wave velocity of 800 m/s.

Superstructure model

A three-dimensional finite element model of the bridge is developed in *SAP2000*® (CSi – Computers and Structures inc.). Arch members, main steel girders, cross girders, and suspenders are modelled by two-node beam elements. Shell elements are used to simulate the concrete slab of the deck. The composite action between the longitudinal girders and the concrete slab is taken into account by using rigid links connecting the concrete slab nodes with those of the steel girders so as to reproduce the effect of the real position of the slab. Uniformly distributed masses are added to the slab to simulate the presence of the trains and other masses not considered in the geometrical model. All piers and abutments are assumed to be rigid because of their dimensions and are simulated concentrating translational and rotational masses in their relevant centroids. The materials are considered to be linear elastic with the constants reported in Figure 5a.

Foundation model

The foundations are modelled according to the method proposed by discretising each pile into 1 m long finite elements. Master nodes are introduced at the centroid of the pile groups at the level of pile head. The soil profile shown in Figure 5b is based on the geotechnical characterization of the site.

Definition of the Free-Field Motion

During earthquakes the local soils act as a filter and modify the ground motion characteristics, the phenomenon is known as ‘soil amplification problem’. Physically, the problem is to predict the characteristics of the seismic motions that can be expected at the surface (or at any depth) of a soil stratum. To get a prediction of the surface vibration, a simulation of soil site response was performed with the computer program NERA (Bardet J.P., Tobita T. 2001) that uses a pseudo-linear treatment, and applies an iterative procedure in order to account for the strain dependence of modulus and damping. The strain dependence of the shear modulus is assumed to correspond to that presented by Sun and Seed (1988), for clays with plasticity index in the range 5-10, while the damping ratio is assumed to vary with shear strain according to the mean pattern presented for clays by Idriss (1990). The shear modulus G' of granular soils has been proposed by Seed and Idriss (1986) by the expression

$G' = 1000 K_2 (\sigma'_m)^{0.5}$ where G' and σ'_m (the mean effective stress) are in psf, and K_2 is a strain-dependent factor. The strain dependence of equivalent damping ratio is assumed to be in accordance with the average curve found by Idriss (1986) for granular and gravely materials.
The seismic action at the bedrock level is represented trough seven real accelerograms matching the elastic response spectrum of the Italian OPCM 3431 for soil type A and hazard level 3. These accelerograms are the same adopted by the ReLuis (network of Italian academic laboratories for seismic engineering) research project. Using NERA a one-dimensional ground response analysis for the specific site was performed. The time histories obtained have been used directly to represent the *Free-Field Motion* at the ground surface and at different depths (Figure 6b). As it can be easily checked in Figure 6a, the response of the soil deposit is largely dominated by the fundamental natural periods of the thick alluvial layer.

Foundation Parameters

Suitably Lumped Parameter Models have to be used to catch the frequency dependent behaviour of foundations in the time domain analysis. The simplest of these models consists, for each degree of freedom of the rigid cap, of one spring, one damper and one mass having the relevant constants calibrated so that both the real part and the imaginary part of the foundation impedances are well approximated. The dynamic stiffness relevant to the ***j***-th dof of the foundation is thus approximated by

$$\Re_j(\omega) = \left[\left(K_j - \omega^2 M_j\right) + i\omega C_j\right] \tag{22}$$

where the real part varies with frequency as a second order parabola while the imaginary part varies linearly with frequency. Use of LPMs leads to some loss of precision, but this is compensated by the advantage of using commercial structural analysis software and performing non-linear analyses when required. The frequency-independent coefficients of springs, viscous dampers and masses are calibrated to reproduce, in the low and medium frequency ranges (0 ÷ 10 Hz) only, the trend of the frequency dependent impedances obtained from the soil–pile–foundation interaction analysis previously described. Figure 7 shows the fluctuation of the real and imaginary parts of the dynamic

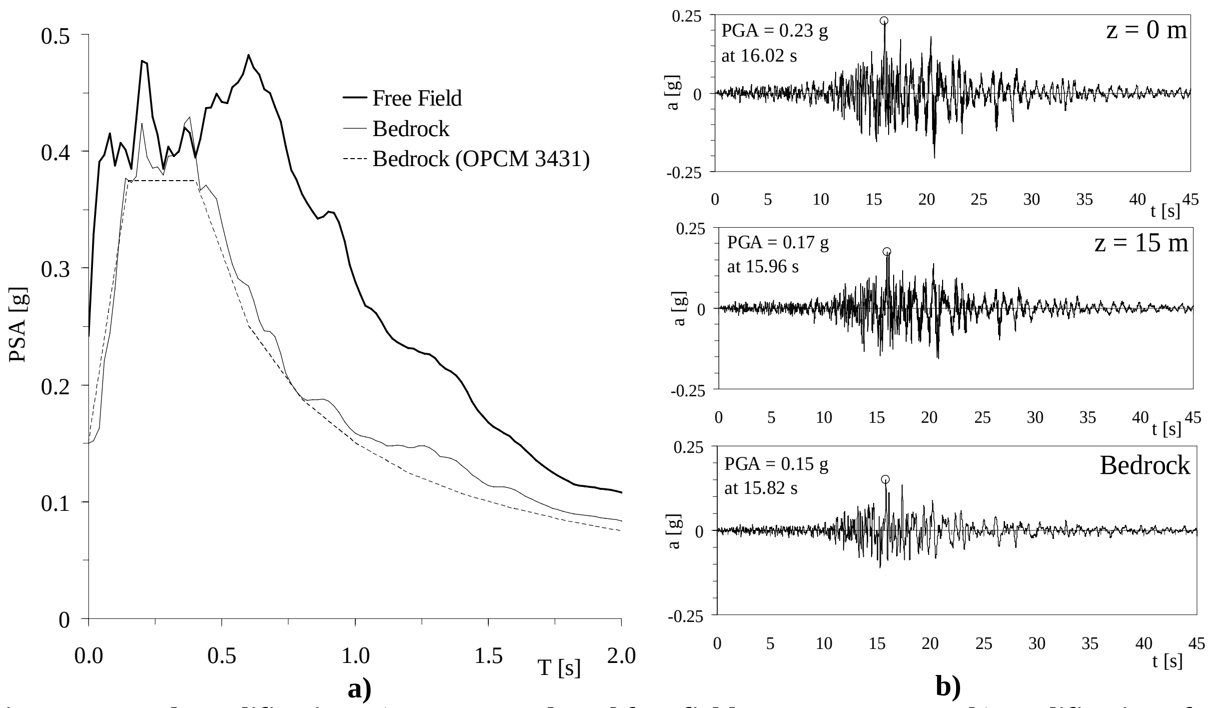

Figure 6: Local amplification: a) average rock and free field response spectra; b) modification of ground motion with depth

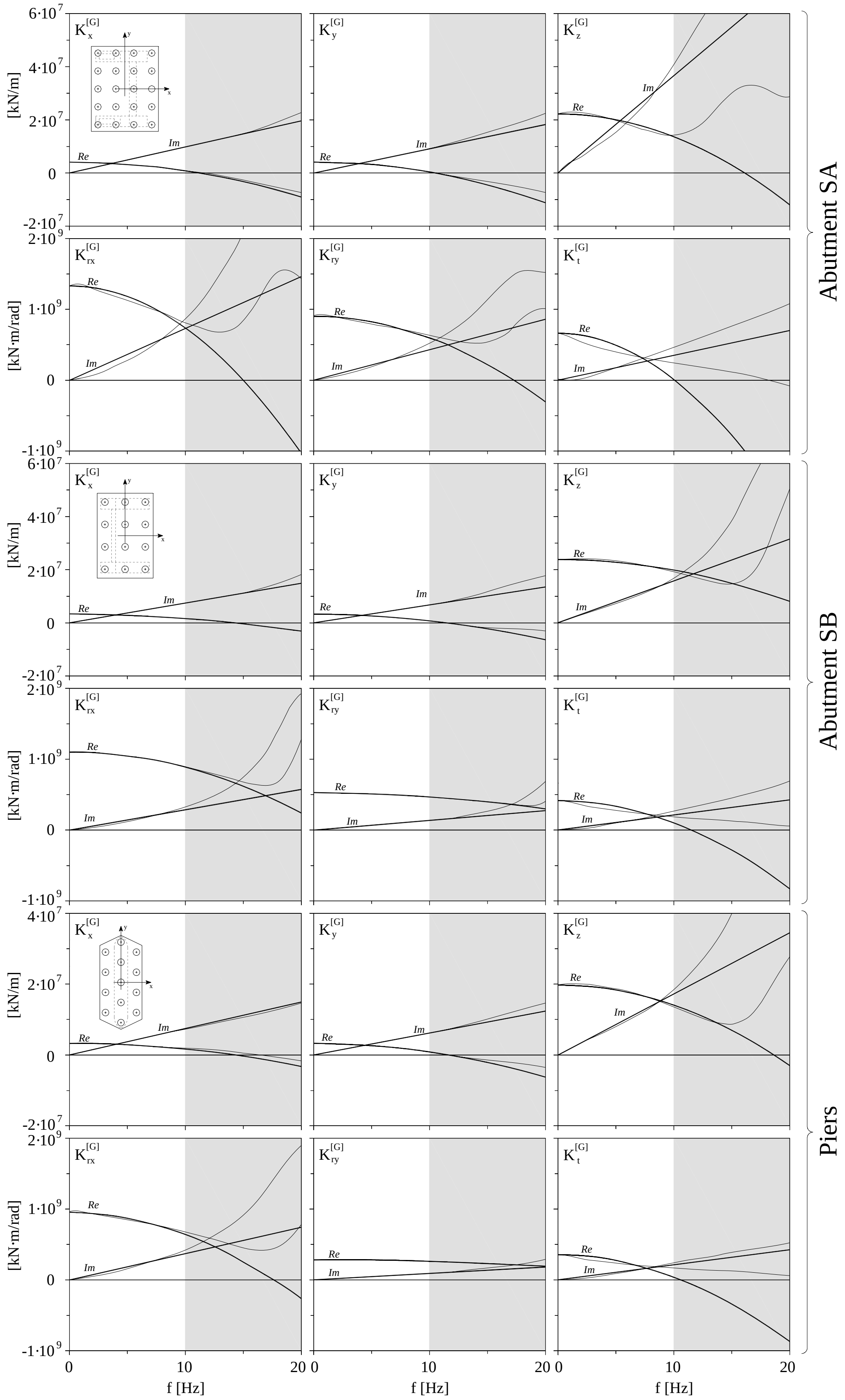

Figure 7: Impedance functions of pile group of abutment SA, abutment SB and piers

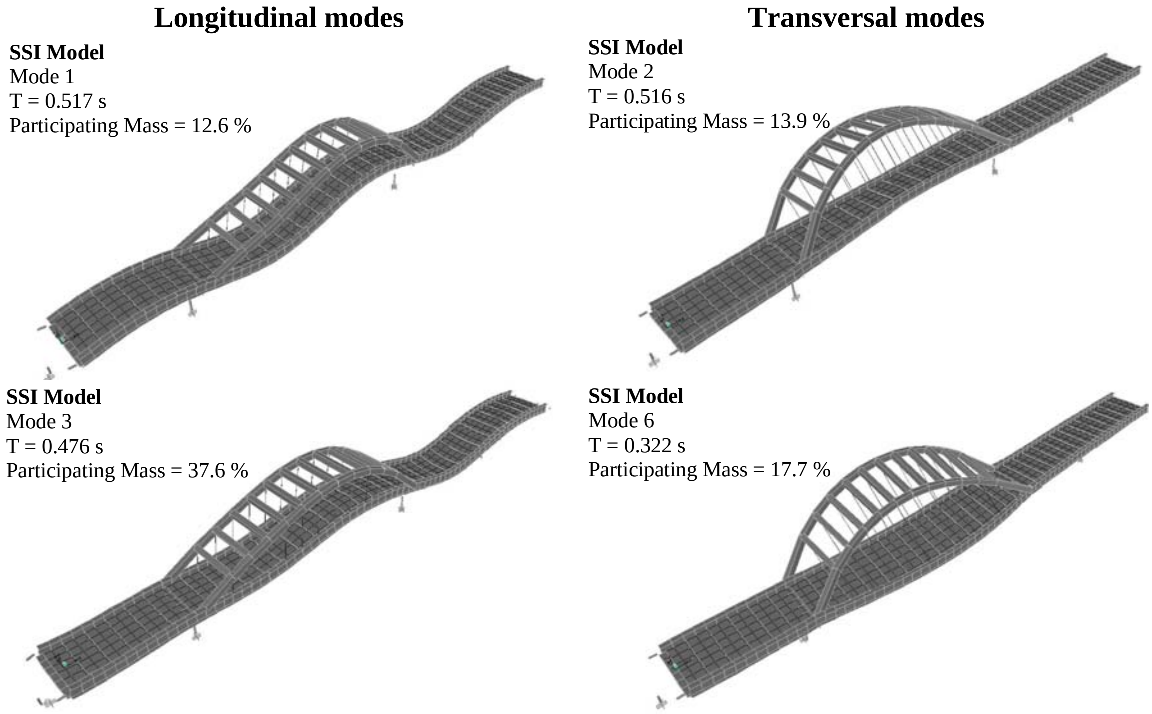

Figura 8: Foundamental longitudinal and transversal mode shapes

impedances for high frequency values. However, during high frequency shaking additional phenomena associated with nonlinear soil behaviour are present that are not represented in the soil–pile foundation interaction analysis.

This is because nonlinear behaviour has the tendency to suppress fluctuations with frequency (Badoni and Makris 1997), so that impedance functions need to be accurately described in the range of low and medium frequency, only. In *SAP2000*® the LPMs are introduced as six linear visco-elastic restraints and six additional masses applied to the master node of each foundation. The restraints are one-joint grounded spring *SAP2000*® library elements having stiffness and damping matrixes built up with the frequency independent parameters previously calibrated.

Eigenvalue Analysis

Preliminary eigensolutions analyses were performed by means of the three-dimensional finite element model in order to check the effects of the SSI on the overall system flexibility. The main longitudinal and transversal mode shapes are plotted in Figure 8. As expected, the foundation flexibility increases the fundamental periods and alters the mode shapes of the fixed base structure. As a consequence SSI is expected to influence actions at the base of substructures.

Seismic response

The seismic response of the bridge is investigated under seven recorded earthquake ground motions derived from the local response analysis applied alternatively along the longitudinal and transverse directions. The response quantities of interest are the base shear at the head of the pile group for piers and abutments and the axial force at the longitudinal seismic restraints (abutment SA).

Figure 9a shows the Fourier amplitude spectra of the axial force on one of the longitudinal seismic restraints resulting from fixed base and SSI models. The fixed base response exhibits two main peaks corresponding to the dominant frequency of the structure ($\approx$ 4 Hz) and to the dominant frequency of the whole soil deposit ($\approx$ 2 Hz) whereas the SSI response is characterised by only one peak since the dominant frequencies of the structure (Figure 8) are very close to the dominant frequency of the whole soil deposit. It is worth to notice that in both cases the dominant frequencies fall within the plateau of the response spectrum (Figure 6a) and thus no strong reductions of the response forces is attended as a consequence of the SSI. The different frequency contents of the responses obtained in the case of fixed

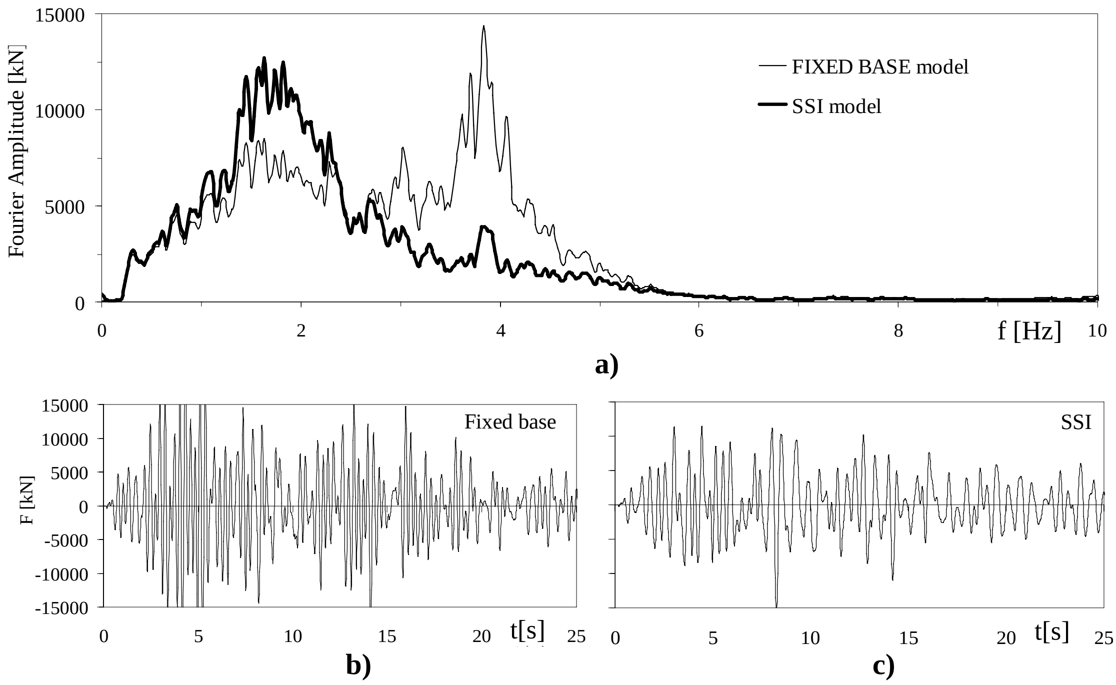

Figure 9: Axial force at one of the seismic restraints: a) Time history for the fixed base model; b) Time history for the SSI model; c) Average Fourier amplitude spectra

Table 1. shears at the pile groups head

		LONGITUDINAL DIRECTION		TRANSVERSE DIRECTION	
		FIXED V [kN]	SSI V [kN]	FIXED V [kN]	SSI V [kN]
Abutment	SA	35092.29	31879.12	6932.97	7848.05
Pier	P1	4806.57	4381.18	13943.80	13753.45
Pier	P2	4806.57	4381.18	13998.64	15590.44
Pier	P3	4091.55	3804.04	9630.71	8755.83
Abutment	SB	4539.68	4194.22	6146.77	7574.45

base model and in the case of SSI model can also be observed from Figures 9b, c where the time histories of the axial forces at each seismic restraint of abutment SA are plotted for one of the accelerograms used.

Table 1 shows the values of shears at the head of the pile group for piers and abutments in the longitudinal and transverse direction for the cases of fixed and flexible foundation. All the response quantities are presented in terms of mean of the peak values obtained from the time history analyses. It results that SSI remarkably influences the longitudinal response of the bridge reducing the seismic demand in terms of shear forces acting on the substructures. On the other hand, in the transverse direction both increased and decreased values of base shear are obtained from SSI analyses with respect to the fixed base ones. This is due to the complex interaction between the stiff deck and the substructures that applies a redistribution of the seismic action among piers and abutments.

CONCLUSIONS

In this paper the influence of the SSI on the seismic response of the railway bridge on the Tronto river is studied. First a numerical model for the evaluation of soil-foundation dynamic impedances is presented; then the Domain Decomposition technique is adopted to perform time domain seismic analyses of the railway bridge introducing LPMs to model frequency dependent impedances behaviours of pile foundations. Comparisons of the response with respect to a conventional fixed base model show the importance of considering SSI effects in the seismic design of the bridge. The main conclusions are:

- the foundation flexibility increases the fundamental periods of the system and alters the shape of the vibration modes with respect to the fixed base structure;
- SSI analysis allows the design optimization of the longitudinal seismic restraints;
- the 3D modelling of the bridge was found critical to account for the complex effects of the mutual interaction between the deck and the substructures and to consider their influences on the transversal seismic response.

REFERENCES

Badoni, D., Makris, N. (1997) Analysis of the non linear response of structures supported on pile foundations. *Report No. UCB/EERC-97/07. Berkeley, Calif.: University of California.*

Dezi F. (2006). Soil-Structure Interaction Modelling, *Tesi di dottorato*, Università Politecnica delle Marche.

Dezi F., Scarpelli G. (2006) L'interazione terreno-fondazione-struttura nella risposta sismica delle pile da ponte. *V Congresso Nazionale dei Ricercatori di Ingegneria Geotecnica, Bari.*

Dezi F., Scarpelli G. (2006) L'influenza dell'interazione terreno-palo-struttura nel progetto sismico della spalla fissa di un ponte ferroviario. *V Congresso Nazionale dei Ricercatori di Ingegneria Geotecnica, Bari.*

Dobry R., Gazetas G., (1988). Simple Method for Dynamic Stiffness and Damping of Floating Pile Groups, *Geotechnique*, 38(4), pp.557-574.

Fan K., Gazetas G., A., Kausel E., Ahmad. S. (1991). Kinematic seismic response of single piles and pile groups. *J. of Geotechnical Engineering* 117(12), pp.1860–79.

Gazetas, G. Dobry, R. (1984). Horizontal Response of Piles in Layered Soils, *Journal of Geotechnical Engineering,* ASCE, 110(1), pp.20-40.

Gazetas G., Dobry. R. (1984a). Horizontal response of piles in layered soils. *Journal of Geotechnical Engineering.* ASCE 110, pp.20–40.

Gazetas G., Dobry. R. (1984b). Single radiation damping model for piles and footings. *Journal of Engineering Mechanics*. ASCE 110, pp.937–956.

Idriss, I. M. (1990) "Response of Soft Soil Sites during Earthquakes", *Proceedings Memorial Symposium to honor Professor Harry Bolton Seed*, , California, Vol. II, May.

Kaynia A. M., Kausel E., (1982) Dynamic stiffness and seismic response of sleeved piles. *Rep. R80-12*, MIT, Cambridge, Mass.

Kramer S. L., (1996) Geotechnical Earthquake Engineering. Prentice *Hall, Upper Saddle River, RJ*.

Makris, N., Gazetas, G. (1992). Dynamic Pile-Soil-Pile Interaction - Part II: Lateral and Seismic Response. *Earthquake Engineering Structural Dynamics*, 21(2), pp.145-162.

Makris, N., Gazetas G. (1993). Displacement phase differences in a harmonically oscillating pile. *Geotechnique* 43(1), pp.135–50.

NERA (Bardet J.P., Tobita T. 2001). Nonlinear Earthquake site Response Analyses of Layered Soil Deposit. *University of Southern California*.

OPCM n. 3431, Further modifications and integrations to OPCM n. 3274, 2005.

SAP2000 (October 2005). "CSI analysis reference manual". *Computer and Structures, Inc.*, , California, U.S.A.

Seed, H. B., Wong, R. T., Idriss, I. M. and Tokimatsu, K. (1986) "Moduli and Damping factors for Dynamic Analyses of Cohesionless Soils," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 11 2, No. GTI 1, November, pp.1016-1032.

Sun, J. I., Golesorkhi, R. and Seed, H. B. (1988) "Dynamic Moduli and Damping Ratios for Cohesive Soils," *Report No. UCB/EERC-88/15, Earthquake Engineering Research Center*, University of California, , 42p.

Wolf, J. P., Dynamic Soil-structure Interaction, Prentice-Hall, Englewood Cliffs, N.J., 1987

Wolf, J. P., Soil-structure Interaction Analysis in Time Domain, Prentice-Hall, Englewood Cliffs, N.J., 1988.