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SEISMIC RESPONSE OF BRIDGE PIERS FOUNDED ON INCLINED PILE GROUPS

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ABSTRACT

This paper investigates the seismic response of bridge piers founded on inclined pile groups, evaluating effects of soil-structure interaction by considering different pile group geometries and piles inclinations. Analyses are performed by means of the direct approach taking advantage of a 3D numerical model developed by the authors for the analysis of inclined pile groups. In particular, both the superstructure and piles are modelled by means of beam elements and the soil is schematized as a visco-elastic medium constituted by independent infinite horizontal layers. The soil-pile and the pile-soil-pile interaction are captured in the frequency domain by means of elastodynamic Green's functions that also allow including the hysteretic and radiation damping. In order to investigate kinematic stress resultants in piles and to evaluate the significance of the soil-foundation system filtering effect, kinematic interaction analyses of the soil-foundation systems are also performed by removing the superstructure mass from the models. The influence of key parameters such as the piles inclination and the group configuration on the structural behaviour (displacements and stress resultants) and on the foundation response (maximum shears and bending moments in piles) is discussed.

INTRODUCTION

Inclined piles are able to resist higher lateral loads than vertical ones with the same diameter and length, as part of the horizontal force is axially transmitted. Inclined piles are thus less stressed by shear and bending, with the advantage of limiting the pile diameter, especially in the case of soft soils. Nevertheless, the use of inclined piles in seismic areas has been discouraged by many modern codes (NTC2008; EN1998-5) because of their poor performance in past earthquakes. Among others, some examples of such negative performances are the wharfs of the Port of Okland, during the 1989 Loma Prieta Earthquake, and the Port of Los Angeles, during the 1994 Northridge Earthquake. Post earthquake investigations revealed that in many cases the poor connection at the cap or the inappropriate pile design have been the major causes of failures (Mitchell et al., 1991). Indeed, inclined piles may induce large forces on the pile cap and permanent rotations may arise for strong earthquakes; furthermore, the high axial force may reduce piles bending moment capacity. These aspects should be carefully taken into account in the design to guarantee a good seismic performance of inclined pile groups.

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However, there are evidences that in certain cases inclined piles, if properly designed, may be beneficial for both the superstructure and the piles themselves (Gazetas and Mylonakis, 1998; Guin, 1997); some examples in this sense are represented by the Landing Road Bridge during the 1987 Edgecumbe earthquake and the Maya Warf during the 1995 Kobe earthquake. These observations, in conjunction with the awareness of the real causes of damages observed in past earthquakes, have contributed to renovate the confidence of practicing engineers in this type of foundation to such an extent that foundations on inclined piles have been recently employed in the construction of some big infrastructures (e.g. the Oakland Bay Bridge in San Francisco).

Many works have recently been published focusing on the seismic behaviour of inclined piles and important progress has been made thanks to the work of Sadek and Shahrour (2004), Gerolymos et al. (2008), Giannakou et al. (2010), Padrón et al. (2010) e Morici et al. (2013) (among the others). However, the beneficial or detrimental role of inclined piles on the dynamic behaviour of the supported structures and on the foundation itself has not yet been sufficiently investigated.

The effects of soil-structure interaction in the seismic response of single bridge piers founded on pile groups with inclined piles are investigated in this paper. By assuming a linear behaviour for the superstructure and a linear equivalent behaviour for the soil-foundation system, a procedure is developed in the frequency domain adopting a finite element direct approach. The soil-foundation system is modelled according to the procedure proposed by Morici et al. (2013) and assembled to the superstructure model to obtain the dynamic stiffness matrix of the whole soil-foundation-superstructure system. The procedure makes it possible to account for the actual deformability of the soil-foundation system, hysteretic and radiation damping and modification of the input motion due to the filtering effect of the embedded foundation. The soil-pile interaction is accounted for by considering the real pattern of the foundations. Furthermore, site amplification effects of the seismic motion are evaluated performing a local site response analysis that is necessary also to define the seismic input, represented by the free-field ground motion at depths corresponding to the nodes of pile finite elements.

The procedure is adopted to perform seismic soil-structure interaction analyses of some case studies constituted by single bridge piers founded on inclined pile groups with different piles inclinations. A soil deposit constituted by three horizontal layers, with dynamic properties increasing with depth, overlaying a seismic bedrock is considered. The modification of the input motion due to the filtering effect of the embedded foundation as well as the kinematic stress resultants in piles due to the passage of seismic waves are also addressed. Two different pile group layouts are analysed considering different piles inclinations. The seismic input, represented by real accelerograms defined at the outcropping bedrock guarantees, at the deposit surface, the compatibility with the specific code spectral acceleration evaluated at the bridge fundamental period, calculated with the fixed base assumption.

ANALYSIS PROCEDURE

The paper focuses on the seismic behaviour of single bridge piers founded on inclined pile groups (Fig. 1). Such a scheme may be representative of bridges with simply supported decks and of multispan continuous decks not connected at the abutments in the transverse direction. In the case of bridges with a dual load path mechanism, analyses may be representative of the transverse seismic behaviour of inner piers where effects of the boundary conditions are negligible.

The bridge pier is founded on a pile group with a generic layout, characterized by piles with generic inclinations and rigidly connected at the head. The superstructure is modelled by means of beam elements while the soil-foundation system takes advantage of the Morici et al. (2013) approach. The dynamic stiffness matrices of the superstructure and the soil-foundation system are derived separately in the frequency domain, by assuming that soil and structural elements (superstructure and piles) behave linearly, and are then assembled to obtain the following complex-valued system of linear equations governing the dynamics of the whole soil-foundation-superstructure system:

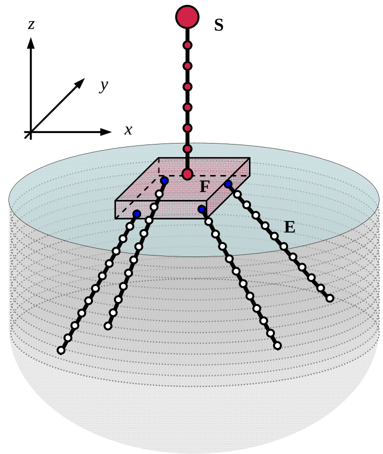

Figure 1. Soil-foundation-superstructure model

$$\begin{bmatrix} \mathbf{Z}_{SS} & \mathbf{Z}_{SF} & \mathbf{0} \\ \mathbf{Z}_{FS} & \mathbf{Z}_{FF,S}+\mathbf{Z}_{FF,F} & \mathbf{Z}_{FE} \\ \mathbf{0} & \mathbf{Z}_{EF} & \mathbf{Z}_{EE} \end{bmatrix} \begin{bmatrix} \mathbf{d}_S \\ \mathbf{d}_F \\ \mathbf{d}_E \end{bmatrix} = \begin{bmatrix} \mathbf{0} & \mathbf{0} & \mathbf{0} \\ \mathbf{0} & \Im_{FF,F} & \Im_{FE} \\ \mathbf{0} & \Im_{EF} & \Im_{EE} \end{bmatrix} \begin{bmatrix} \mathbf{0} \\ \mathbf{u}_{FF,F} \\ \mathbf{u}_{FF,E} \end{bmatrix} \tag{1}$$

In equation (1) the dynamic stiffness matrix **Z** is partitioned consistently with the unknown nodal displacement vector **d** in order to highlight components relevant to the superstructure (*S*), piles rigid cap at the top end of piles (*F*) and embedded piles (*E*), as reported in Figure 1.

The soil-pile interaction is accounted for in the definition of the dynamic stiffness matrices relevant to the rigid cap and to the piles embedded into the soil. The right-hand side of equation (1) is obtained by multiplying the complex impedance matrix of the soil by vector $\mathbf{u}_{FF}$, collecting the soil free-field motion, which constitutes the seismic input; the product furnishes the soil-pile interaction forces developing as a consequence of the seismic motion propagating thought the foundation soil. The free-field ground motions at different depths, corresponding to the nodes of pile finite element discretization, may be obtained by means of site response analysis while the complex impedance matrix of the soil, which reflects the overall dynamic characteristics of the soil medium relatively to the foundation displacements at the soil–foundation interfaces, is assembled according to the model of Morici et al. (2013). Consequently, the effects of local site conditions may be included in the evaluation of the free-field motions while the modification of the seismic input due to the filtering effect of the embedded foundation is automatically captured by the procedure.

Soil-foundation system modelling

The coupled soil-foundation system is modelled according to the analytical formulation proposed by Morici et al. (2013). Piles are assumed to be beam elements and the soil is schematized with infinite independent horizontal layers. The dynamics of each layer is described by means of elastodynamic Green's functions that allow accounting for the pile-soil-pile interaction and the soil hysteretic and radiation damping. The dynamic stiffness sub-matrix relevant to the soil-foundation system, appearing in equation (1) is formulated as follows:

$$\begin{bmatrix} \mathbf{Z}_{FF,F} & \mathbf{Z}_{FE} \\ \mathbf{Z}_{EF} & \mathbf{Z}_{EE} \end{bmatrix} = \begin{bmatrix} \mathbf{K}_{FF,F} & \mathbf{K}_{FE} \\ \mathbf{K}_{EF} & \mathbf{K}_{EE} \end{bmatrix} - \omega^2 \begin{bmatrix} \mathbf{M}_{FF,F} & \mathbf{M}_{FE} \\ \mathbf{M}_{EF} & \mathbf{M}_{EE} \end{bmatrix} + \begin{bmatrix} \Im_{FF,F} & \Im_{FE} \\ \Im_{EF} & \Im_{EE} \end{bmatrix} \tag{2}$$

In equation (2) **K** and **M** are the real stiffness and mass matrix of the pile groups, respectively, while $\Im$ is the complex impedance matrix of the soil, also appearing in equation (1). Matrix (2) is the same involved in the kinematic analysis of the soil-foundation system with which effects on the foundation due to the propagation of the seismic waves in the soil may be assessed. This problem is governed by the following complex-valued system of linear equations:

$$\begin{bmatrix} \mathbf{Z}_{FF,F} & \mathbf{Z}_{FE} \\ \mathbf{Z}_{EF} & \mathbf{Z}_{EE} \end{bmatrix} \begin{bmatrix} \mathbf{d}_{F,K} \\ \mathbf{d}_{E,K} \end{bmatrix} = \begin{bmatrix} \Im_{FF,F} & \Im_{FE} \\ \Im_{EF} & \Im_{EE} \end{bmatrix} \begin{bmatrix} \mathbf{u}_{FF,F} \\ \mathbf{u}_{FF,E} \end{bmatrix} \tag{3}$$

It is worth noting that the displacement and the known vectors in equation (3) are obtained as sub-vectors of the relevant quantities appearing in equation (1). Furthermore, $\mathbf{d}_{F,K}$ is the motion experimented by the foundation in absence of the superstructure (Foundation Input Motion), namely the actual motion transmitted to the superstructure by the foundation, accounting for the filtering action exerted by the embedded foundation, and $\mathbf{d}_{E,K}$ are the motions of the embedded piles resulting from the free-field input that may be used to evaluate the kinematic stress resultants along piles. The procedure may be applied once the free-field displacements at different depths, corresponding to the nodes of the finite element model of piles, are determined. The evaluation of the free-field motions generally requires site response analyses of the soil profile starting from a selected input motion. It is worth noting that the non-linear behaviour of the soil may be taken into account in the proposed method by considering degraded stiffness and enhanced damping, derived from the maximum strain level attained at different depths from the site response analysis.

Superstructure modelling

The dynamic stiffness matrix of the superstructure is obtained according to the finite element formulation of the problem. Taking into account equation (1) the following expression holds:

$$\left[\begin{array}{c|c} \mathbf{Z}_{SS} & \mathbf{Z}_{SF} \\ \hline \mathbf{Z}_{FS} & \mathbf{Z}_{FF,S} \end{array}\right] = \left[\begin{array}{c|c} \mathbf{K}_{SS} & \mathbf{K}_{SF} \\ \hline \mathbf{K}_{FS} & \mathbf{K}_{FF,S} \end{array}\right] - \omega^2 \left[\begin{array}{c|c} \mathbf{M}_{SS} & \mathbf{0} \\ \hline \mathbf{0} & \mathbf{M}_{FF,S} \end{array}\right] + i\omega \left[\begin{array}{c|c} \mathbf{C}_{SS} & \mathbf{C}_{SF} \\ \hline \mathbf{C}_{FS} & \mathbf{C}_{FF,S} \end{array}\right] \tag{4}$$

where **K**, **M** and **C** are the classical frequency independent stiffness, mass and damping matrices of the superstructure, respectively. With reference to masses, the relevant deck mass is lumped at the pier head while the pier mass is distributed along the height, consistently with the finite element discretization. The damping matrix **C** is conveniently represented in the form

$$\left[\begin{array}{c|c} \mathbf{C}_{SS} & \mathbf{C}_{SF} \\ \hline \mathbf{C}_{FS} & \mathbf{C}_{FF,S} \end{array}\right] = \alpha \left[\begin{array}{c|c} \mathbf{K}_{SS} & \mathbf{K}_{SF} \\ \hline \mathbf{K}_{FS} & \mathbf{K}_{FF,S} \end{array}\right] \tag{5}$$

where α may be calibrated to achieve a predefined structural damping in correspondence of the fundamental period of the pier.

CASE STUDIES

Case studies consist of single bridge piers, relevant to a multi-span bridge characterized by a steel-concrete composite deck. With reference to the deck cross section geometry reported in Fig. 2a, translational and rotational masses of 11.37 t/m and 94.72 t/m^2, respectively, have been computed and the mass relevant to the single pier has been evaluated by considering a span length of 25 m. The pier has a solid square cross section of edge 2.00 m and its mass is consistently distributed along the column. The piers' heights are equal to 15 m, including the pile cap thickness. The concrete is of grade C30/37 and is considered to be linearly elastic with Young's modulus E_c = 3.5 x 107 kN/m². Cracking effects are accounted for by considering an effective modulus of elasticity $E_{c,eff} = 0.75E_c$. The pier is discretized by 1 m long beam finite elements. The fundamental period of the fixed base system is 0.537s.

Foundations are constituted by 2x2 pile groups with diameter D = 1 m, spacing s = 5D and length L = 30 m. The piles cap is considered to be rigid with a master node placed in correspondence of its centroid. Two pile group layouts have been considered (C1 e C2), as depicted in Fig. 2b and four piles inclinations (θ = 0°, 10°, 20°, 30°) have been investigated.

A 40 m thick three layered soil deposit of class C (NTC2008) overlaying a seismic bedrock is considered in the applications (Fig. 2c). The bedrock is characterized by a shear wave velocity of 800 m/s while the shear wave velocity profile of the soil deposit is reported in Fig. 2c. A visco-elastic behaviour is assumed for the soil, calibrating stiffness and damping consistently with the maximum shear strains attained during the seismic shacking in order to account for its non linear hysteretic behaviour. In the applications the shear modulus degrading and damping evolution curves proposed by Vucetic and Dobry (1991) are adopted.

Seismic action

The seismic action is defined at the outcropping bedrock by means of real accelerograms. In particular, seven accelerograms with magnitude ranging from 5.2 e 7.2 and epicentral distances within the range 9÷136 km are selected (Table 1). The free-field motions within the soil deposit in correspondence of the nodes of pile finite element discretization are obtained by means of 1D local response analyses, accounting for the nonlinear soil behaviour in a linear equivalent manner, as specified in the previous section.

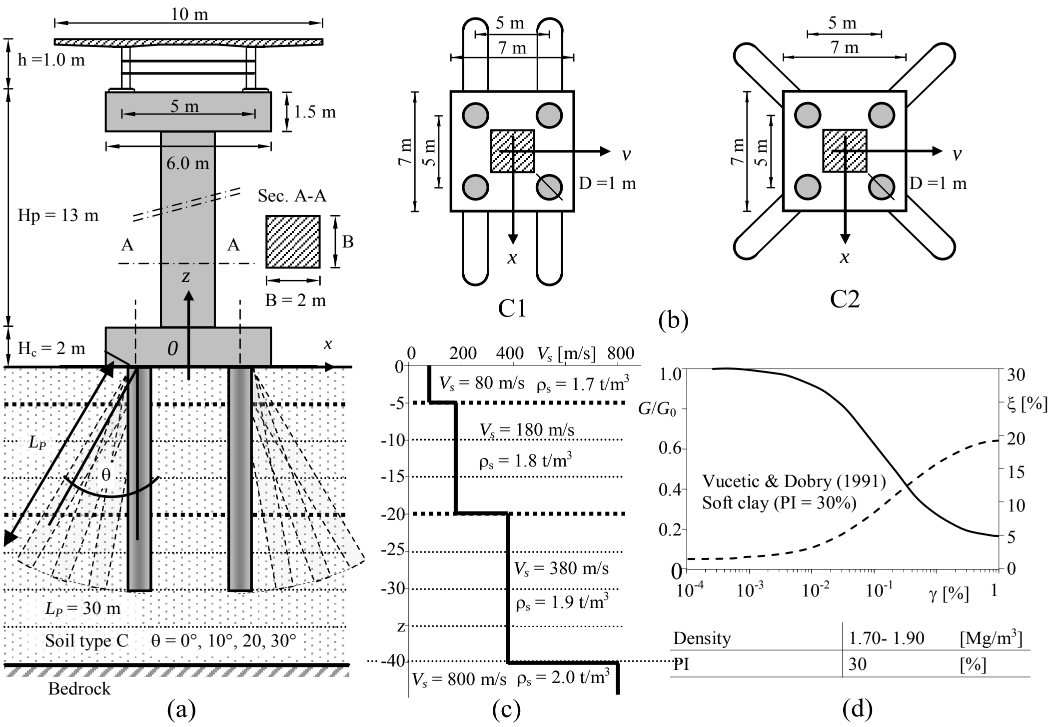

Figure 2. (a) Cross section of the soil-foundation-superstructure system; (b) foundations layouts; (c) shear wave velocity profile of the soil deposit and (d) shear modulus degrading and damping evolution curves

Table 1. Selected real accelerograms

Earthquake	Date [dd/mm/yy]	M_w	Δ [km]
Friuly, Tolmezzo (aftershock)	17/06/1976	5.2	27
Tabas, Dayhook	16/09/1978	7.2	12
Campono-Lucano; Arienzo	23/11/1980	6.9	78
Campono-Lucano; Auletta	23/11/1980	6.9	25
Friuly, Scomplago Centrale (aftershock)	16/09/1977	5.4	9
Duzce 1, Izmit	12/11/1999	7.2	105
Strofades, Koroni-Town Hall	18/11/1997	6.6	136

Seismic actions are scaled to comply with the selected seismic hazard (PGA = 0.25g on soil type A) and ensure the compatibility with the code reference acceleration response spectrum for soil type C (NTC2008). In particular, to avoid excessive scattering of soil-structure interaction results due to the local response of the deposit a specific strategy has been adopted: accelerograms, recorded on rock outcropping sites (soil type A), are iteratively scaled to achieve, at the deposit surface, the design spectral acceleration at the fundamental period of the fixed base structure.

Fig. 3 shows the elastic acceleration response spectra obtained for the selected earthquakes at the deposit surface (grey lines) and their relevant mean spectrum (red line); the reference spectrum for soil type C is also reported (black line) for comparison. As a result of the adopted iterative procedure, it can be observed that all spectra present the same ordinate in correspondence of the pier fundamental period.

Soil-structure interaction analyses

Soil-structure interaction analyses are performed in the frequency domain according to the methodology presented in previous sections by means of a specific computer code running in the MATLAB programming environment; the pier is modelled with beam finite elements of length 1 m, according to recommendations included in Priestley et al. (2007). At the pier head translational and rotational masses of 320 t and 3902 tm^2 (both inclusive of the bent contribution), respectively, are considered; the pier mass is instead distributed along the pier stem. Translational and rotational masses of the pile rigid cap are lumped at the pier base; these are estimated in 200 t and 859 tm^2, respectively.

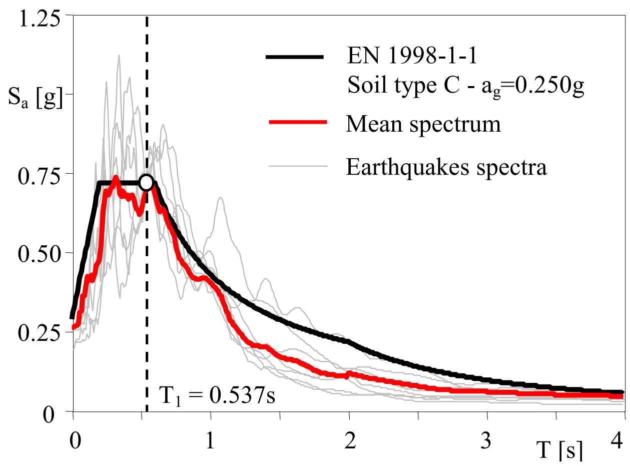

Figure 3. Acceleration elastic response spectra at the deposit surface: spectra of the selected earthquakes, mean and reference spectra

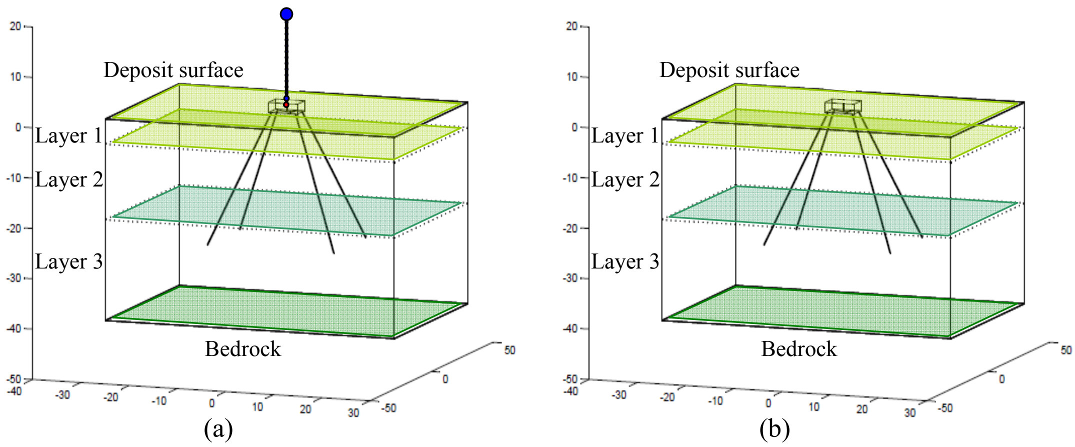

Figure 4. Developed numerical models: (a) soil-foundation superstructure model and (b) soil-foundation model for the evaluation of kinematic interaction effects

The soil foundation system is included in the model according to the analytical formulation proposed by Morici et al. (2013). A 5% stiffness proportional damping for the first mode is introduced in terms of Rayleigh damping. A pictorial view of the developed model is presented in Fig. 4a.

In order to assess kinematic interaction effects on the seismic response of the soil-foundation system, the superstructure mass is set equal to zero (e.g. neglecting the presence of the superstructure) in the previously developed model (Fig.4b). These analyses are aimed at studying the filtering effect exerted by the soil-foundation system which modify the seismic input to the superstructure and at evaluating the kinematic stress resultants in piles, namely stresses deriving from the propagation of seismic waves through the soil.

For both model the seismic actions are applied in the *x* direction, according to the reference system depicted in Fig. 2.

RESULTS

In this section the main results of analyses performed on both the soil-foundation-superstructure and soil-foundation models are addressed. Results of steady-state analyses carried out by considering harmonic excitations applied at the deposit surface are also reported to shows the frequency response of both systems and investigate the systems compliance. Analyses aim at pointing out effects of the pile group layout and piles inclination on the seismic input motion transmitted to the superstructure (FIM), as well as on the seismic response of the superstructure and the foundation, in terms of displacements and stress resultants. Unless otherwise specified, results refer to the average values obtained from the analyses performed by considering the whole set of accelerograms.

Systems compliance

In order to study the frequency response of both the soil-foundation-superstructure and soil-foundation systems, steady-state analyses are performed by applying unit harmonic displacements at the surface of the soil deposit. Analyses allow investigating the fundamental periods of both systems which are non-classically damped.

Fig. 5a shows the absolute values of the horizontal displacements and rotations (multiplied by the pile diameter) of the rigid pile cap, obtained from the soil-foundation system analyses, for both the C1 and C2 configurations. The pile group layout and the piles inclination does not affect sensibly the fundamental periods of the system (response peaks are almost at the same frequencies) which are governed by the soil deposit response. However a reduction of the horizontal displacement and a significant increment of the pile cap rotation are evident by increasing the piles inclination. These results provide a first insight on the effects of the kinematic interaction on the definition of the superstructure seismic input.

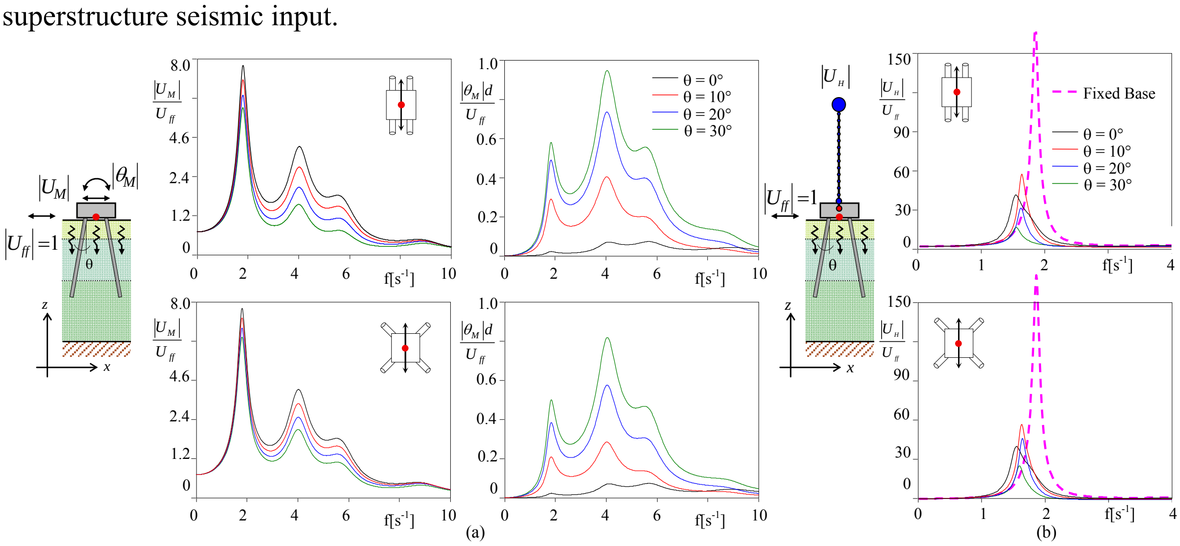

Figure 5. Frequency response: (a) displacement and rotation of the pile cap (“kinematic interaction”); (b) displacements of the pier head (“kinematic and inertial interaction”)

Fig. 5b shows the absolute values of the horizontal displacements of the pier head obtained by applying the previous harmonic excitation to the whole soil-foundation-superstructure model for both the foundation configuration C1 and C2 and all the investigated piles inclinations; the response of the fixed-base system is also reported for comparison. The structural response is plotted in the range 0÷4 Hz within which the fundamental frequency of the system falls. A significant reduction of the pier fundamental frequency is observed by diminishing the piles inclination as a consequence of the increasing compliance of the soil-foundation system; in the case of vertical piles the fundamental frequency reduces of about 8% with respect to the fixed base model. Furthermore, by increasing the piles inclination a reduction of the peak response is observed; the most important effect in this sense is evident passing from the fixed-base to the compliance base system.

Soil-foundation filtering effect

As for the FIM, Fig. 6 shows for both configurations C1 and C2 the mean absolute values of the maximum displacements and rotations obtained from the analyses of the soil-foundation systems. The scattering of results is highlighted by means of segments connecting the maximum and minimum absolute values obtained from the different analyses. By increasing the piles inclination a slight reduction of the mean maximum horizontal displacement is evident for both the pile layouts (with reference to vertical piles the maximum reduction of about 10% is registered for piles inclination of 30°). On the other hand, a significant increment of the rotational component of the motion is observed by increasing the piles inclination; for piles inclination of 30° the rotation is about 20 times higher than that relevant to vertical piles.

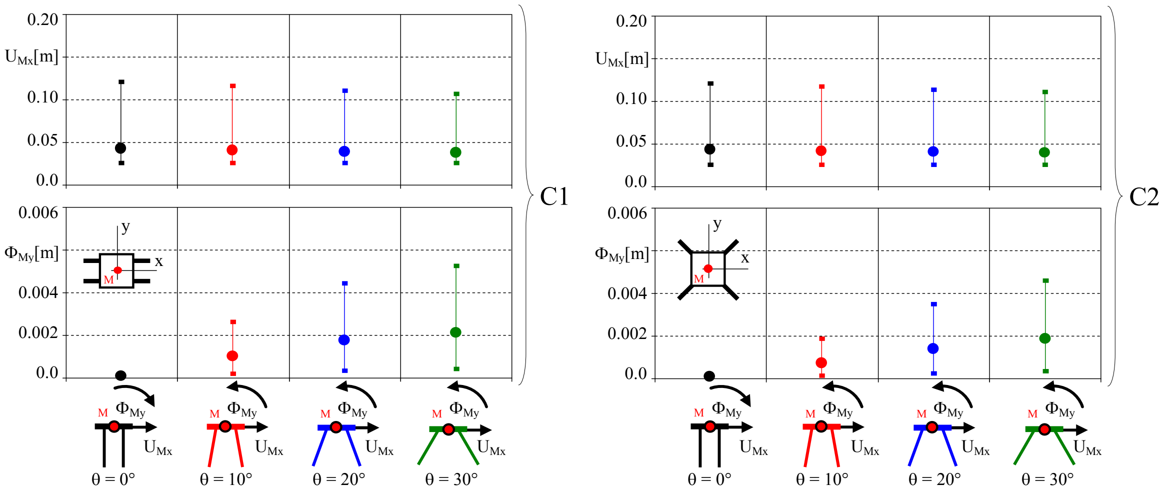

Figure 6. Motion transmitted to the superstructure (FIM)

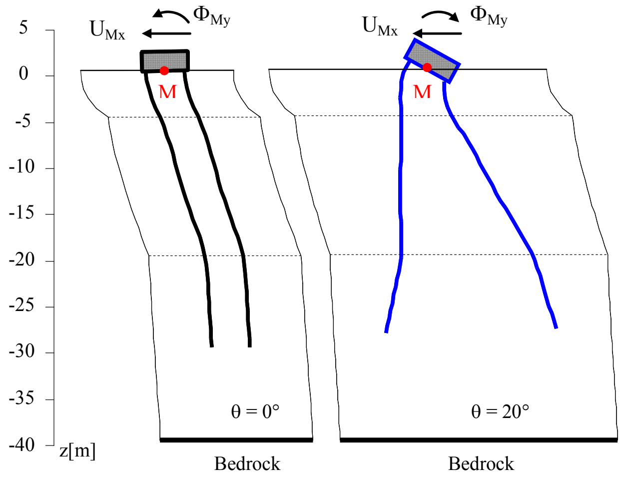

Figure 7. Deformed shape of the soil-foundation system (C1, θ = 0° e 20°) subjected to the Campano-Lucano earthquake (Arienzo)

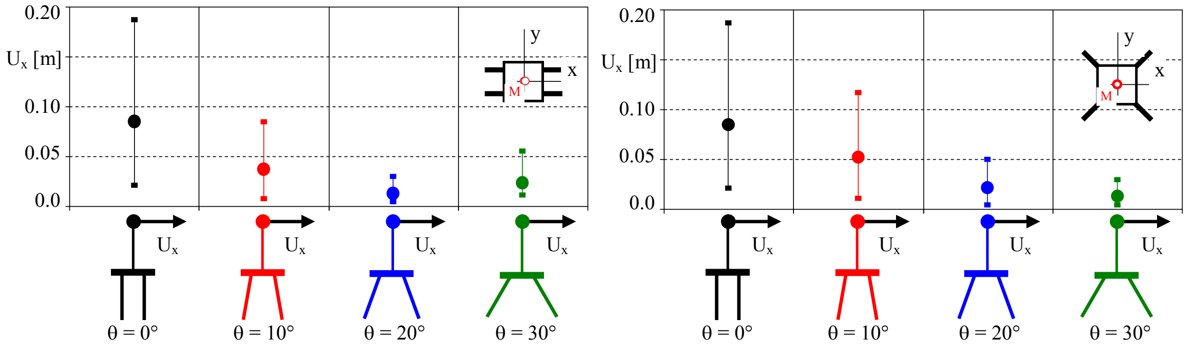

Figure 8. Mean absolute values of the superstructure displacements

Furthermore, it should be noted that for pile groups with inclined piles the rotation due to a positive horizontal translation of the system is opposite in sign with respect to that relevant to pile groups with vertical piles; this is clearly highlighted in Fig. 7 in which the deformed shape of the soil-foundation systems with vertical piles and with piles inclined of 20° are depicted; these refer to the foundation configuration C2 subjected to the Campano-Lucano earthquake (Arienzo) and are evaluated for the time step at which the maximum horizontal displacement of the pile cap is registered. Finally, as expected, few differences are observed between the response of pile group configurations C1 and C2, as a consequence of the direction of loading.

Superstructure displacements

Fig. 8 shows the mean absolute values of the maximum displacements of the deck (pier head) evaluated taking into account the contributions of the foundation translation and rotation. Again, beside the mean values, segments connecting maximum and minimum absolute values obtained from all the analyses are reported. The superstructure displacement diminishes sensibly by increasing the piles inclination; with reference to pile groups with vertical piles, displacements reduce of about 4 times for piles inclinations of 20° and 30°, depending on the group layout. This is mainly due to the displacement contribution of the foundation rocking which is, in the case of inclined piles, opposite in sign with respect to that produced by inertia forces.

Pile stress resultants

Fig. 9 shows for one pile of the investigated groups (C1 and C2) the envelopes of maximum and minimum stress resultants obtained from the kinematic interaction analyses (analyses performed on the soil-foundation system) and the complete soil-foundation-superstructure interaction analyses (analyses performed on the whole soil-foundation-superstructure system). The first (kinematic stress resultants) are induced in piles by the propagation of seismic waves through the soil, the latter (total stress resultants) result from the combined kinematic and inertial effects. In addiction, envelopes of inertial stress resultants due to forces transmitted by the superstructure are reported in Fig. 9; these are obtained, in view of the problem linearity, by the subtraction of the kinematic contribution from the total stress resultants (performed step by step).

Stress resultants are derived with reference to the local coordinate system (0, 1, 2, 3) of the pile. As for pile group configuration C1 the local reference system has the 3-axis oriented as the *y*-axis of the global coordinate system so that the only meaningful stress resultants are the shear force in the 2-axis direction and the bending moment around the 3-axis (since seismic action is applied in the *x* direction). On the other hand, in the case of pile group configuration C2 the local reference system is also rotated with respect to the global *z*-axis; thus both shear forces in the 2-axis and 3-axis directions, as well as bending moments around the same axes, are non-null.

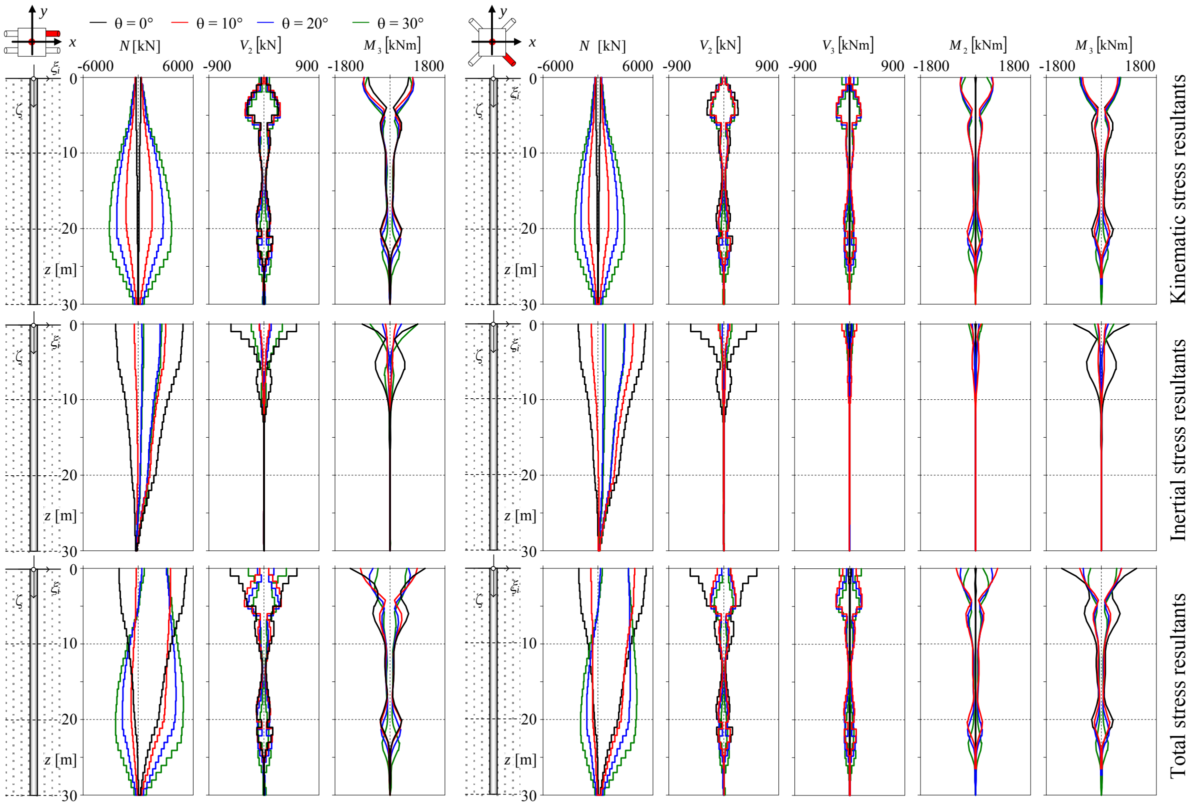

Figure 9. Pile stress resultants: kinematic, inertial and total components

With reference to kinematic stress resultants it should be observed that the soil horizontal free-field displacements, due to the seismic wave propagation, mainly induce shear forces and bending moments in vertical piles; small axial forces arise as a consequence of the system roto-translational coupling. In inclined piles horizontal soil displacements induce significant axial forces which increase by increasing the piles inclination; furthermore, shear forces and bending moments are comparable with those arising in vertical piles since only slightly increments have been observed by increasing the piles inclination (of about 10% for piles inclination of 30°). For both vertical and inclined piles, maximum values of shear forces and bending moments localise at the pile head, as a consequence of the kinematic constraint exerted by the rigid cap, or at the interface between layers characterised by a high impedance contrast. Concerning axial forces, the maximum value is attained at the pile head in the case of vertical piles and at a depth ranging from 10÷15 times the pile diameter in the case of inclined piles.

As expected inertial stress resultants attain their maximum values at the pile head. With respect to results obtained for vertical piles, a reduction of all stress resultants may be observed by increasing the piles inclination. In particular, axial force reduces to such an extend that for inclinations greater than 20° piles do not undergo tractions; shear forces and bending moments show significant reductions even for small inclinations ($\theta = 10°$).

Finally, concerning the total components, a reduction of stress resultants may be observed by increasing the piles inclination, especially at the pile head.

Superstructure stress resultants

Fig.10a-b shows the envelopes of maximum and minimum stress resultants along the pier, obtained from the complete soil-foundation superstructure analyses by considering both pile group configurations (C1 and C2); results obtained by considering the fixed-base hypothesis are also reported to highlight contribution of soil-structure interaction.

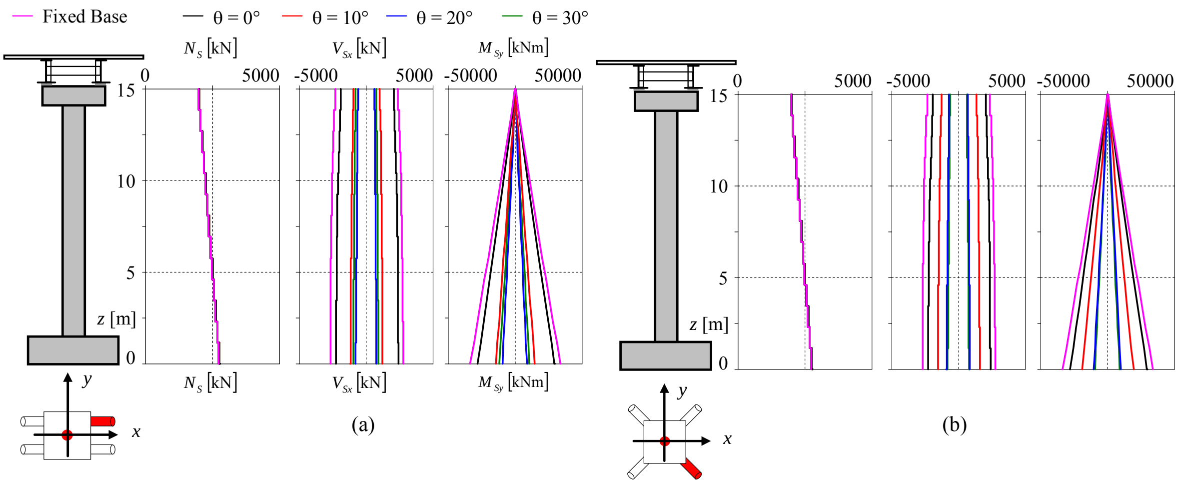

Figure 10. Superstructure stress resultants: (a) pile group configuration C1 and (b) C2

For the considered case studies the soil-structure interaction generally determines a reduction of the superstructure shear force and bending moment; passing from pile group configurations with vertical piles to pile group configurations with inclined piles, reductions become more and more evident with the increment of the piles inclination. In particular, for piles inclined of 30° shear force and bending moment reduce of about 60% with respect to the relevant values obtained from the fixed-base analysis. Finally, axial force is only due to vertical loads as the vertical component of the seismic shaking is neglected.

CONCLUSIONS

In this paper the seismic response of bridge piers founded on inclined pile groups has been investigated, evaluating effects of soil-structure interaction. Different pile group layouts and piles inclinations have been considered and the soil-structure interaction analyses are performed by means of a direct approach taking advantage of a 3D numerical model developed by the authors for the analysis of inclined pile groups. In particular, both the superstructure and piles are modelled by means of beam elements and the soil is schematized as a visco-elastic unbounded medium constituted by independent infinite horizontal layers. The soil-pile and the pile-soil-pile interaction are captured in the frequency domain by means of elastodynamic Green's functions that also allow including automatically the hysteretic and radiation damping. In order to investigate kinematic stress resultants in piles and to evaluate the significance of the soil-foundation system filtering effect, kinematic interaction analyses of the soil-foundation systems are also performed. By assuming a linear behaviour for both the soil and the superstructure, all the analyses are performed in the frequency domain by means of dedicated computer codes developed by the authors in MATLAB environment; the soil nonlinearities are taken into account in a linear equivalent manner calibrating stiffness and damping consistently with the maximum soil shear strains. In the applications, a soil deposit constituted by three horizontal layers having dynamic properties increasing with depth is considered; furthermore, the seismic action is represented by suitably scaled real accelerograms.

The main results of the investigation may be summarized as follows:

- the seismic motion transmitted to the superstructure by the soil-foundation system (FIM) depends sensibly on the piles inclination; by increasing the piles inclination a reduction of the translational component and a significant increment of the rotational component of the FIM may be observed;
- the superstructure displacements, measured at the pier top, reduce by increasing the piles inclination;
- inclined piles undergo kinematic shear forces and bending moments comparable with those of vertical piles; on the other hand inclined piles are subjected to greater axial forces, which increase by increasing the piles inclination;

- inertial stress resultants in inclined piles attain their maximum values at the pile heads and diminish with depth; globally, they progressively reduce by increasing the piles inclination;
- soil-structure interaction produces a reduction of the superstructure shear force and bending moment; passing from vertical to inclined piles reductions become more and more evident with the increase of the piles inclination

The performed investigations support evidences that inclined piles, at least in certain cases and if properly designed, can be beneficial rather than detrimental for both the structure they support and the piles themselves. Despite they are not sufficient to clarify the role of inclined piles on the seismic response of structures (bridges in particular), they demonstrate the need of further researches aimed at providing useful guidance to professional engineers.

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