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Nonlinear response of bridge piers on inclined pile groups: the role of rocking foundation input motion

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Abstract

This paper presents first results of an on-going research focused on the effects of piles layout and inclination on the nonlinear seismic response of bridge piers. The analysis methodology, based on the substructure approach, is firstly presented. The soil-foundation system is studied in the frequency domain according to a numerical model developed by the authors while the inertial interaction analysis of the superstructures is carried out in the time domain to capture the nonlinear structural behaviour. A suitable lumped parameter model is used to approximate the frequency dependent behaviour of the soil-foundation impedances in the time domain analyses. The procedure is applied to some case studies constituted by single bridge piers founded on medium stiff and soft clayed soil deposits. Pile groups with piles of different inclinations are considered, as well as piers with different fundamental periods and yielding bending moments of the base cross sections, to simulate systems with different ductility capacity. Analyses results show the key role of the foundation rocking on the superstructure response and demonstrate that inclined pile foundations may have a significant impact on the superstructure response, reducing the pier head displacements and ductility demand.

Keywords: bridge piers, inclined pile groups, nonlinear seismic analyses, soil-foundation-structure interaction

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1. Introduction

The use of inclined pile foundations in seismic prone areas is discouraged by modern technical codes [1, 2], despite their recognized capability to resist higher lateral loads than vertical ones (with the same diameter and length). This

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suggestion is based on documented poor performances of inclined piles in past earthquakes [e.g. 3]. However, investigation on the actual causes of failure, carried out by many researchers in the last decades [e.g. 4-5], often revealed that observed damage may have been due to design inadequacies rather than to intrinsic drawbacks [e.g. 3]. In addition, some evidences of good seismic performances of inclined piles [6, 7] are also available demonstrating that the beneficial or detrimental role of inclined piles on the seismic response of the supported structures and the foundation itself is still not well established. Despite important progresses towards a better comprehension of the dynamics of inclined piles have been recently made [8-11], further investigations are needed to clarify the role of inclined piles on the superstructure seismic response.

This paper presents first results of an ongoing research focused on the effects of inclined piles foundations on the nonlinear seismic response of single bridge piers. The analysis methodology, based on the substructure approach, is firstly presented and then applied to some case studies. The soil-foundation system is studied in the frequency domain by means of a numerical model developed by the authors [11]. Inertial interaction analyses of bridge piers are performed in the time domain to capture the nonlinear structural behaviour and a lumped parameter model with frequency independent parameters is adopted to account for the frequency dependent behaviour of the soil-foundation impedances. The procedure is applied to several case studies consisting of bridge piers founded on vertical and inclined pile groups in medium stiff and soft clayed soil deposits. Piers with different yielding bending moments of the base cross sections are considered. Analyses results show the key role of the rocking foundation input motion on the structural response in terms of structural displacements and piers ductility demand.

2. Analysis Methodology

A single bridge pier supported by a deep foundation with generic piles inclination and layout is considered. By handling soil nonlinearites through a linear equivalent approach, the seismic Soil-Structure Interaction (SSI) problem is studied according to the substructure method, by dividing the soil-foundation-structure system (Fig. 1a) into the soil-foundation (Fig. 1b) and the superstructure subsystems (Fig. 1c) and introducing suitable compliant restraints at their interface. This approach is classically applied to linear systems but may be used to include effects of superstructure nonlinearity [12] by assuming piles to behave linearly and making the hypothesis that shear strains induced by inertial interaction are negligible compared to those due to the propagation of seismic waves.

2.1. Analysis of the soil-foundation system: impedances and foundation input motion

The dynamic analysis of the soil-foundation system provides the frequency-dependent impedance matrix, which defines the compliant restraint of the superstructure, and the Foundation Input Motion (FIM) (i.e. the actual motion of the foundation), which differs from the free-field motion as a consequence of the foundation filtering effect.

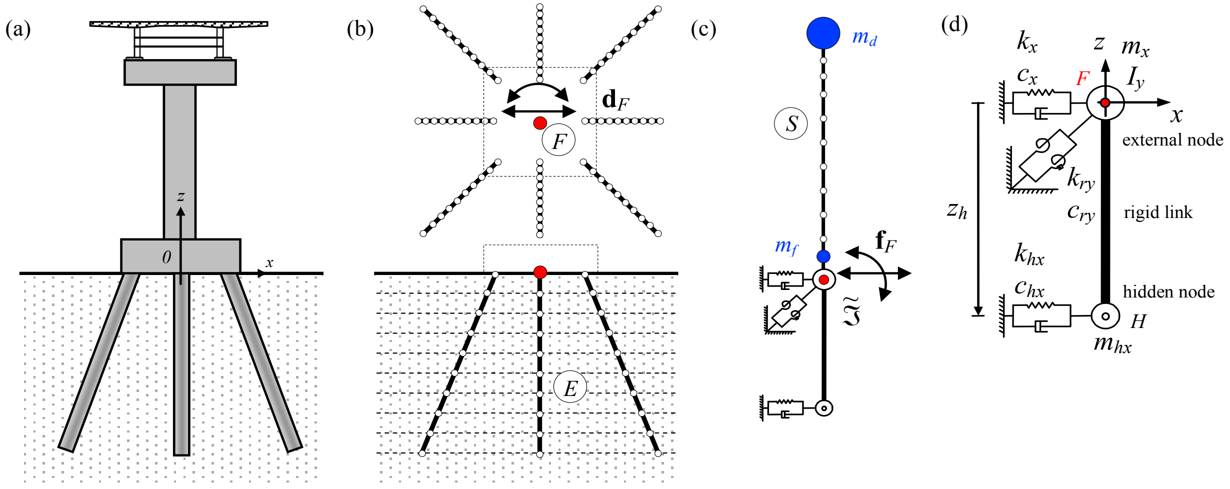

Fig. 1. (a) Bridge pier founded on a generic pile foundation, (b) soil-foundation model, (c) superstructure model; (d) adopted LPM

The numerical finite element model proposed by Dezi et al. [11] is adopted in this paper (Fig. 1b); the model allows studying the soil-pile interaction problem of deep foundations characterised by generic piles layout and inclinations, subjected to the propagation of seismic waves in layered soil deposits. The dynamic problem is solved in the frequency domain by modelling piles with beam finite elements and the soil with independent horizontal infinite layers. The dynamics of each layer accounts for the soil hysteretic and radiation damping, as well as for the pile-soil-pile interaction. The presence of a rigid cap is simulated by introducing a rigid constraint at the piles head (*F*). In the frequency domain, the discrete problem is governed by the following system of complex linear equations:

$$\left[\begin{array}{c:c}\mathbf{Z}_{FF} & \mathbf{Z}_{FE}\\ \hdashline \mathbf{Z}_{EF} & \mathbf{Z}_{EE}\end{array}\right]\left[\begin{array}{c}\mathbf{d}_{F}\\ \hdashline \mathbf{d}_{E}\end{array}\right]=\left[\begin{array}{c:c}\mathcal{I}_{FF} & \mathcal{I}_{FE}\\ \hdashline \mathcal{I}_{EF} & \mathcal{I}_{EE}\end{array}\right]\left[\begin{array}{c}\mathbf{u}_{F}\\ \hdashline \mathbf{u}_{E}\end{array}\right]_{ff} \tag{1}$$

where **Z** is the dynamic frequency dependent stiffness matrix of the system and **d** is the vector of nodal displacements, partitioned to highlight components of the embedded piles (*E*) and of the rigid cap (*F*) (Fig. 1b). The right-hand side of equation (1) represents the vector of nodal forces resulting from the soil-pile and pile-soil-pile interaction; this is obtained multiplying the global impedance matrix of the soil $\mathcal{I}$ by the soil free-field motions, collected in vector $\mathbf{u}_{ff}$, evaluated within the soil deposits in correspondence of nodes of the pile mesh through specific site response analyses. The complex-valued foundation impedance matrix and the FIM, obtained by condensing problem (1), result

$$\Im(\omega)=\left(\mathbf{Z}_{FF}-\mathbf{Z}_{FE}\mathbf{Z}_{EE}^{\;-1}\mathbf{Z}_{EF}\right) \qquad \mathbf{d}_{F}(\omega)=\Im^{-1}\left[\mathbf{f}_{F}-\mathbf{Z}_{FE}\mathbf{Z}_{EE}^{\;-1}\mathbf{f}_{E}\right] \tag{2a, b}$$

Since the in-plane seismic response of bridge piers is considered, impedance matrix (2) assumes the form

$$\Im(\omega)=\begin{bmatrix}\Im_{x} & 0 & \Im_{x-ry}\\ 0 & \Im_{z} & 0\\ \Im_{x-ry} & 0 & \Im_{ry}\end{bmatrix} \tag{3}$$

where subscripts *x*, *z* and *ry* are adopted to indicate dynamic stiffness components relevant to the horizontal, vertical and rotational degrees of freedom of the foundation cap (*F*), consistently with the reference system reported in Fig. 1a. Subscript *x-ry* refers to the roto-translational coupling term.

2.2. Superstructure analysis: definition of the LPM and the seismic action

The inertial interaction analysis of the superstructure on compliant restraint and subjected to the FIM is carried out in the time domain. For this purpose, LPMs constituted by frequency independent parameters [13] are adopted to approximate the frequency dependent impedance matrix of the soil-foundation system. In particular, the LPM reported in Fig. 1d is adopted [14-15] to capture the in-plane dynamic behaviour of the soil-foundation systems. The model is characterized by 3 *dofs* (2-D analysis) and the impedance matrix, which depends on 13 parameters, assumes the form

$$\tilde{\Im}(\omega)=\left(\tilde{\mathbf{K}}-\omega^{2}\tilde{\mathbf{M}}+i\omega\tilde{\mathbf{C}}\right) \tag{4}$$

Parameters are calibrated with a least squares procedure to provide the best fit of the foundation impedances in the frequency range 0÷10 Hz. The superstructure is modelled exploiting potentials of structural analysis software and the LPM is implemented at the base of the superstructure. The seismic actions are applied to the compliant base models at node *F* by considering time histories of forces computed by means of the inverse Fourier transform

$$\mathbf{f}_{F}(t)=\frac{1}{2\pi}\int_{-\infty}^{\infty}\tilde{\Im}\mathbf{d}_{F}e^{i\omega t}d\omega \tag{5}$$

3. Case studies

The nonlinear seismic response of single bridge piers extracted from a multi-span continuous steel-concrete composite viaduct is investigated (Fig. 2a). Piers with a solid square cross section of side B and height H_p characterised by different fundamental periods and yielding bending moments of the base cross sections are considered. Geometric parameters and masses m_d (Tab. 1) are calibrated to obtain systems characterized by fundamental periods T_1 of the fixed base systems equal to 0.6 and 1.2 s. The concrete is of grade C35/45 with Young's modulus $E_c = 34077\text{x}10^3$ kN/m², reduced to 80% to account for cracking effects. The foundation is constituted by a 3x3 group of concrete piles of diameter $d = 1$ m, spacing $s = 3d$ and z-projected length $L_z = 20$ m (Fig. 2c). Different pile inclinations are considered as reported in Fig.2a. Two homogeneous clayed soil deposits, representative of medium stiff and soft soil conditions (soil types C and D of EN1998-1 [1]), are assumed for the applications. Soil deposit C and D have shear wave velocity V_s equal to 240.5 m/s and 71.7 m/s, respectively, and density ρ equal to 1.56 t/m^3 and 1.39 t/m^3, respectively. Both soils are characterized by a Poisson's ration $\nu = 0.4$ and hysteretic damping ratio $\xi = 0.05$.

Piers are modelled as rigid body elements, coupled with end rotational springs (Fig. 2d). Each case study of Tab. 1 is analyzed considering a linear and nonlinear behavior of the pier and assuming a foundation characterized by vertical and inclined piles, founded on both soil deposits. The lumped elastic stiffness k of the flexural spring is calibrated to reproduce the fundamental period of the fixed base system. Nonlinear analyses are performed considering an elastic (almost) perfectly plastic model characterised by yielding bending moments M_{y1} and M_{y2} (Fig. 2b) equal to 1/3 and 2/3 the maximum bending moment M_u, respectively, acting on the elastic systems for the design seismic input.

The seismic input is defined at the ground surface of each deposit through a set of 3 artificial accelerograms, individually matching the elastic response spectrum for ground type C and D. Fig. 2e shows, for soil deposit D, the pseudo-acceleration elastic response spectra of the 3 accelerograms, together with the code spectrum. Since the design seismic input differs for the two soil types, different values of M_u (and consequently of M_{y1} and M_{y2}) are defined for each case study of Tab. 2; these are reported in Tab. 1 with the elastic stiffness k of the flexural springs.

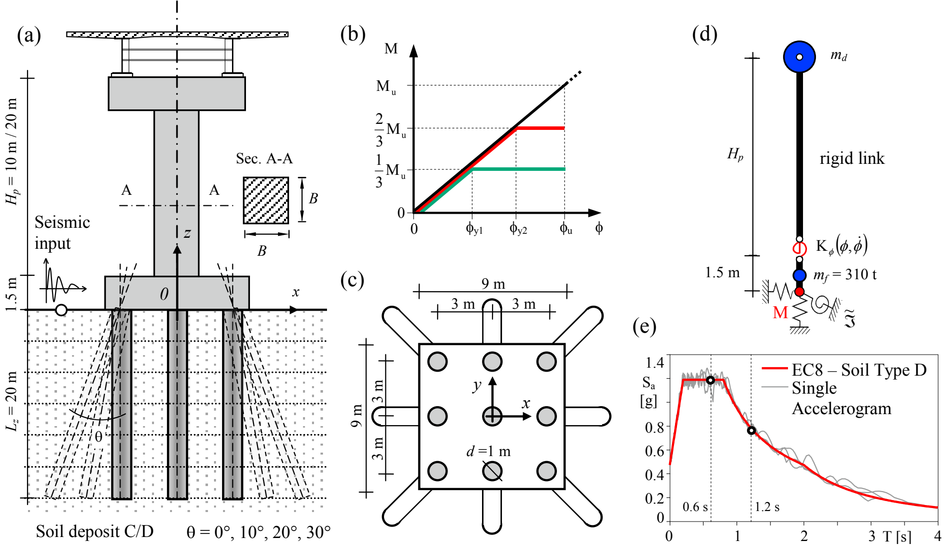

Fig. 2. (a) Pier elevation; (b) M-ϕ constitutive relationships; (c) foundation layout; (d) structural scheme; (e) acceleration elastic response spectra

Table 1. Superstructure parameters.

Case study	T (s)	H_p (m)	B (m)	m_d (t)	k (kNm/rad)	M_u (kNm) Deposit C	M_u (kNm) Deposit D
T06H10	0.6	10	1.5	314.6	3.46E+04	4.39E+04	5.16E+04
T06H20	0.6	20	2.5	303.5	3.33E+04	8.47E+04	9.95E+04
T12H10	1.2	10	1.1	314.6	9.99E+03	2.20E+04	3.44E+04
T12H20	1.2	20	1.8	303.5	8.96E+03	4.24E+04	6.63E+04

4. Results of the applications

For the sake of brevity, phenomena governing the piers dynamic response are firstly discussed qualitatively presenting results to few analyses and focusing on the key role of the rocking foundation input motion; then results of the whole set of analyses are shown, considering the mean value (obtained from the set of accelerogram) of the maximum piers relative displacement, with respect to the foundation. Fig. 3 plots the time histories of displacements and rotations of the pile cap (F) (i.e. the FIM) registered in soil deposits D, for one of the selected accelerograms. It can be noted that positive displacements of the cap are associated with positive rotations, in the case of vertical piles, and negative rotations in the case of inclined piles; furthermore, foundation rotations increase by increasing the pile inclination. While rotation of vertical pile foundations only arises as consequence of axial forces in piles developing to equilibrate the kinematic pile head bending moments, rotation of inclined pile foundations is also due directly to soil movements that produce uplifting and sinking of piles. Thus, for pile groups with inclined piles the rotation component of the FIM induces inertia forces opposite in sign with respect to that produced by horizontal displacements. This generally reduces inertia forces on the superstructure that undergoes minor displacements, as can be observed in Fig. 4a where the time histories of the relative displacements of the deck with respect to the foundation are reported for case studies T06H20 in soil deposit C for one of the selected accelerogram. Fig. 4b shows the moment-chord rotation diagrams relevant to the plastic hinges at the base of piers T06H20 (nonlinear models with capacity M_{y1}) founded on vertical and 30° inclined piles in soil deposit C; consistently with displacements, a reduction of the ductility demand of the plastic hinge at the pier base can be observed.

Fig. 5 shows the mean values of the maximum relative displacements of the deck obtained from the set of accelerograms for all the case studies. Contributions to displacements due to the foundation rocking, the elastic deflection of the piers and the plastic rotation of the hinge at the piers base are shown. For the nonlinear systems characterized by different yielding bending moment of the pier cross section, an overall reduction of the contribution due to the plastic hinge rotation (i.e. a reduction of the ductility demand) is evident by increasing the piles inclination, as well as an overall increment of the contribution due to the foundation rocking. This trend is so marked for soil deposit D that a significant number of nonlinear systems founded on piles with high inclinations (20° and 30°) perform elastically. The effects of the foundation rocking appear negligible for slender structures in soil deposits C and very pronounced for stiff structures in soft soils; this observation comply with general considerations from the literature concerning the significance of SSI effects on the dynamic response of structures.

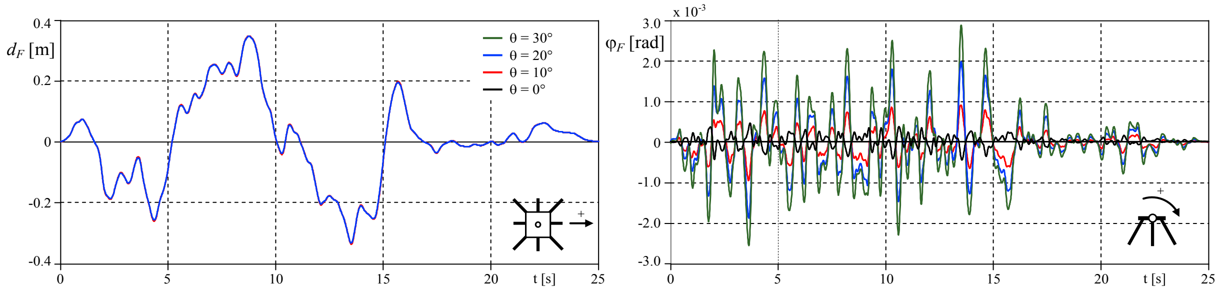

Fig. 3. Translational and rotational components of the FIM for one of the selected accelerograms for soil deposit D.

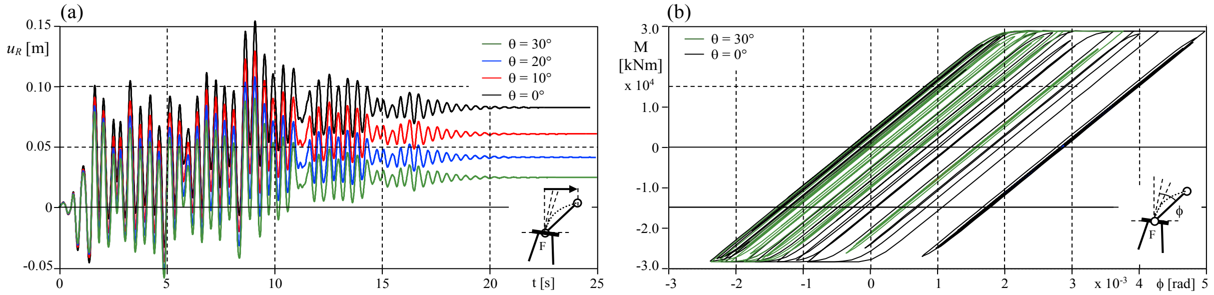

Fig. 4. (a) Time histories of the deck relative displacements for T06H20; (b) M-ϕ diagram for T06H20

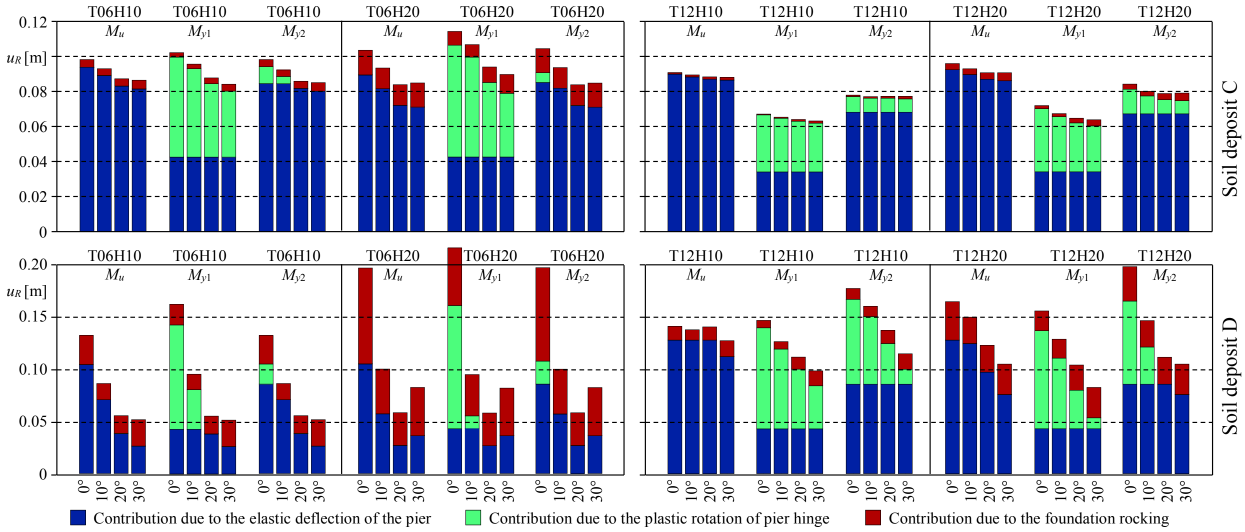

Fig. 5. Relative displacements of the deck for all case studies.

5. Conclusions

First results of an on-going research focused on the effects of piles layout and inclination on the nonlinear seismic response of bridge piers has been presented and discussed. In particular, piers resting on a medium stiff and soft clayed soil deposit and characterized by different fundamental periods, height and ductility capacity are analysed.

Results pointed out that for inclined pile foundations, rotations due to kinematic interaction are opposite in sign with respect to those developing in vertical pile foundations. Inertia forces on structural masses due to the foundation rotations reduces the ones due to horizontal translations so that for piers with inclined pile foundations a reduction of the structural displacements and of the ductility demand is observed, especially for stiff structures in soft soils.

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