Soil-Pile-Structure Interaction Under Seismic Loads: Influence Of Ground Motion Intensity, Duration And Non Linearity

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SUMMARY:
Kinematic soil-pile interaction, arising from the wave propagation in the soil, may affect the seismic response of pile foundations and have an important role in the seismic design of structures. In this paper, initially, incremental dynamic analyses are performed to evaluate the effects of ground motion duration and soil non-linearity on the performance of single fixed-head piles in different soil profiles. A beam on a non-linear Winkler foundation is used in the analysis to investigate the significance of yielding, gapping, soil cave-in and cyclic hardening/degradation effects on piles performance.
Secondly, a pile-column supported bridge structure is considered and the soil–pile-bridge pier interaction to seismic loading is investigated. Results illustrate the potential for both kinematic and inertial response.

Keywords: Dynamic soil-structure interaction, soil and pile inelasticity, IDAs, inertial interaction, kinematic interaction

1. INTRODUCTION

The prediction of the performance of pile foundations during earthquakes is a fundamental task for the seismic design of structures. Most modern seismic codes, like Eurocode 8, recommend accounting for soil–structure interaction effects in the seismic design of both foundations and superstructures. The soil-pile-structure interaction problem has been extensively investigated by several researchers. Several methods have been developed for the assessment of seismic performance of soil-structure systems. Generally, medium dense or firm ground is assumed to behave as a linear or equivalent-linear material when subjected to moderate earthquake motions, and the entire soil-foundation-structure system is subdivided into two separate sub domains: the superstructure and the soil-foundation. On the other hand, when the ground is loose or soft or when the ground undergoes strong earthquake motions, soil non-linearity becomes predominant and could considerably modify the dynamic response of the entire system.

In this paper, Incremental Dynamic Analyses (IDAs) are performed to evaluate the effects of Ground Motion Duration (GMD) and soil non-linearity on the performance of single fixed-head piles in various homogeneous soil profiles including saturated clay and sand in either fully dry or saturated state, with different levels of compaction. The analyses are performed by means of a generalized dynamic normal force-displacement Beam on Non-linear Winkler Foundation (BNWF) model (Allotey and El Naggar, 2008), which accounts for cyclic soil degradation/hardening, soil and structural yielding, slack zone development and radiation damping. Finally, the fully-coupled behavior of a pile-column supported bridge structure is evaluated. The influence of soil nonlinearity and Soil-Structure Interaction (SSI) is discussed.

2. DYNAMIC INTERACTION ANALYSIS: KINEMATIC AND INERTIAL EFFECTS

During an earthquake, the interaction between soil and foundation-structure causes the motion applied at the base of the superstructure to deviate from the free-field motion, and the pile foundation to experience additional bending, axial and shearing stresses. Even though the bending moments due to kinematic interaction effects can be very large, they are often neglected in practical design.

In this study, the kinematic soil structure interaction of a fixed pile head is considered, followed by the analysis of the seismic response of a bridge pier supported by a single pile. The non-linear kinematic interaction analyses is performed considering two steps. In the 1st step, the free-field displacements within the deposit along the pile is defined by means of a linear-equivalent site response analysis starting from real accelerograms defined at the outcropping bedrock. In the 2nd step, the soil-pile interaction is evaluated using a BNWF model and the soil-pile interaction is approximated using non-linear springs (p-y curves) in parallel with stiffness proportional dampers. This allows estimating the relative displacements between soil and pile due to the free-field motion. In the BNWF model, the pile itself is modelled as a series of beam-column elements, each with discrete springs connecting the pile to the soil, and the free-field motion obtained within the deposit is applied to the p-y springs as excitation to the system. A slightly idealized version of an actual bridge is considered to perform fully-coupled SSI analysis. The analysis will account for both inertial and kinematic effects, in the spirit of the direct method. The results are reported in terms of bending moments along the pile. The role of kinematic interaction is evaluated comparing the obtained SSI results with those of the non-linear kinematic interaction analysis.

3. PILE-SOIL INTERACTION ANALYSIS WITH BNWF MODEL

The dynamic BNWF model by Allotey and El Naggar (2008) is a degrading polygonal hysteretic model encompassing multilinear backbone curve with defined rules for loading, reloading and unloading. This model is able to capture the dynamic nonlinear behaviour of soil through the following features. It accounts for cyclic soil degradation through simulating unloading-reloading behavior considering a set of rules such as those proposed by Pyke (1979). It can simulate gap formation and closing along the soil-pile interface for cohesive soils and reloading in the slack zone (by means of a strain-hardening curve) for cohesionless soils. In addition, the model can handle cyclic soil degradation/hardening as well as reduced radiation damping due to increased soil non-linearity. The initial confining pressure at zero pile displacement is modeled as a prestraining effect applied to the compression-only elements attached to both sides of the pile.

Several parameters must be calibrated and provided as input in the model to assess the phenomenological model and the soil mechanical behaviour. In this analysis, different types of soil that feature typical cyclic hardening/degrading behaviour are considered. For saturated soils (sand or soft clay), the cyclic response of the soil along the upper portion of pile is generally considered unconfined and is characterized by an inverted S-shaped hysteresis curve due to slack zone development (Figure 1.b). On the other hand, the cyclic response of soil along the lower segment of pile is considered confined and is characterized by an oval-shape hysteresis curve (Figure 1.c). In the case of dry soils (loose sand in particular), soil cave-in is expected to occur, hence the soil cyclic response is characterized by an oval-shape hysteresis curve along the upper portion of the pile as well. Undergoing cyclic loading, soils may exhibit both stiffness and strength degradation depending on the maximum strain amplitude and number of loading cycles experienced. For saturated soft clay, stiffness degradation is usually more significant than strength degradation, while for dry sands a typical hardening response is expected (Figure 1.d).

4. PARAMETRIC INVESTIGATION

A comprehensive parametric study is carried out to analyse the effects of soil non-linearity and soil degradation on the performance of floating single piles with fixed head condition.

Three different types of soil, characterized by suitable geotechnical parameters, are investigated in order to evaluate their non linear behaviour under seismic loading. The seismic input is defined at the seismic bedrock considering four different real accelerograms selected to be representative of different duration scenarios. Incremental Dynamic Analyses (IDAs) are performed to better understand the soil-pile interaction phenomena as the intensity of ground motion increases.

4.1 Analysis Cases

Six different homogeneous soil deposits are considered, all with constant thickness of 20 m and resting upon a uniform linear visco-elastic bedrock (characterized by shear velocity V_s= 800 m/s and soil damping ratio ξ = 5%), as shown in Figure 1a. Table 1 presents the soil type and properties of the different soil deposits considered. Two shear wave velocity values, V_s= 100 m/s and V_s= 200 m/s, and three different soil types are considered: dry sand (DS), saturated sand (SS) and saturated clay (SC). The foundation consists of a single vertical fixed-head pile with a circular cross-section with diameter, d = 1 m and a total length, L_p = 20 m. The concrete pile has a Young modulus, E_p =3×10^7 kPa and density, ρ_p = 2.5Mg/m^3. The pile is modelled as a beam element and is discretized into 0.5 m long finite elements to achieve a suitable level of accuracy. Non-linear springs (spring–dashpot combinations) are attached to each pile node in both sides and are excited at their end with the free-field motion. The initial confining pressure is modeled by imposing a pre-straining displacement to the springs considering a coefficient of lateral earth pressure K_H equal to 1.0 since the pile is assumed to be driven.

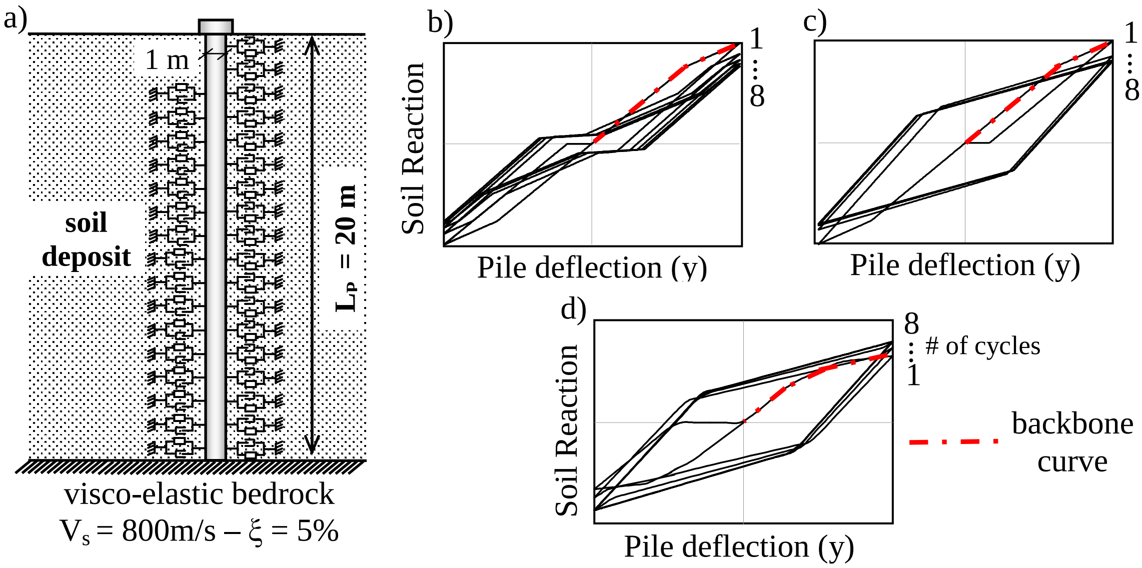

Figure 1. a) Soil profile and BNWF model; Hysteretic curves: b) S-shaped hysteresis curve; c) oval-shape hysteresis curve and d) hardening response

Table 1. Soil type and soil proprieties

Soil Deposit	Soil Type	Soil Consistency	D_r [%]	I_p	V_s [m/s]	γ [kN/m^3]	ν –	φ [°]	C_u [kPa]
100DS	Dry Sand	loose	35	/	100	14.22	0.3	30	/
100SS	Saturated Sand	loose	42	/	100	19.65	0.3	33	/
100SC	Satured Clay	soft	/	10	100	15.45	0.45	/	30
200DS	Dry Sand	medium dense	55	/	200	18.86	0.3	35	/
200SS	Satured Sand	medium dense	60	/	200	20.12	0.3	35	/
200SC	Satured Clay	medium	/	10	200	19.00	0.45	/	70

4.2 Model description and parameter estimation

The reference backbone curves used in this work are the API-recommended p-y curves for sands and soft clay (API 2007). Figure 2 shows the API curves, developed using the soil properties summarized in Table 1, and the four-segments curves used to fit the API curves.

Table 2 shows the different parameters of the cyclic p-y model used for each soil type. Gapping is assumed to occur within the top third of the pile. However, in sand, any developed gap will be simultaneously filled with backfilled soil again (cave-in soil) and no permanent gap will be developed. The soil cave-in parameters are assumed to vary linearly with depth and to increase with the lateral confining pressure.

Stiffness and strength degradation parameters are based on physical quantities deduced from the literature: centrifuge tests for saturated sand (Popescu and Prevost, 1993) and undrained cyclic triaxial compression tests for clay (Hyodo et al., 1994). For dry sand, a typical hardening response is considered (Lo Presti et al., 2000).

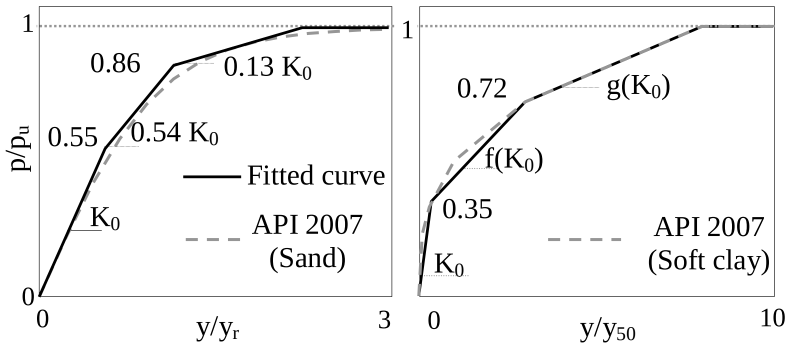

Figure 2. Typical 4-sement curve fit to the hyperbolic curve

Table 2. Cyclic and degradation model parameters for each analysis case

		Parameter	100DS	100SS	100SC	200DS	200SS	200SC
Cyclic curve parameter	< L/3	soil cave-in	5	0:lin:5	0:lin:5	5	0:lin:5	0:lin:5
	> L/3		5	5	5	5	5	5
	< L/3	DRC stiffness ratio	1	0:lin:1	0:lin:1	1	0:lin:1	0:lin:1
	> L/3		1	1	1	1	1	1
		gap force	1	1	1	1	1	1
Degradation parameter		stiffness hardening/degradation	1.2	0.1	0.7	1.2	0.1	0.7
		strength hardening/degradation	1.2	0.1	1	1.2	0.1	1
		stiffness curve shape	2	0.9	2.5	2	0.9	2.5
		strength curve shape	2	0.9	1	2	0.9	1
		slope of the S-N curve	0.1	0.32	0.12	0.1	0.32	0.12
		cyclic stress ratio at N=1	0.8	0.3	1	0.8	0.6	1

4.3 Definition of ground motion records and free-field displacements

To investigate the Ground Motion Duration (GMD) effects on the non-linear seismic response of the soil-pile system, 4 real ground motion records, defined at the outcropping bedrock, are selected from the Pacific Earthquake Engineering Research Center (PEER) database (http://peer.berkeley.edu/smcat/), to be representative of three duration scenarios defined by means of a damage factor I_D (Cosenza and Manfredi (2000)): ‘small duration’($I_D < 5$), ‘moderate duration’ (I_D <

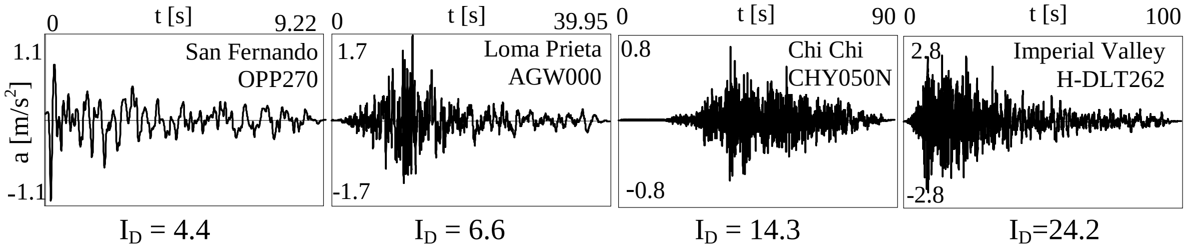

Figure 3. Earthquake records adopted in the analyses

16) and ‘large duration’ (I_D > 22). Figure 6 shows the seismic acceleration time histories of the selected records and other related data. Each record has been scaled to 4 increasing levels of intensity using an iterative procedure. This procedure involves: firstly, applying a scale factor to the selected outcropping motion; performing a 1D linear-equivalent site response analysis; iteratively, adjust input motion until the spectral acceleration of the surface motion, in correspondence of the fundamental natural period of the soil deposit (evaluated by considering low-strain mechanical parameters), converges to the following values: 0.2g, 0.4g, 0.6g and 0.8g. 1D site response analyses are performed considering different degradation and damping ratio curves for clayey and sandy soils (Figure 4a). A total number of 16 ground motions, defined at the outcropping bedrock, for each soil profile are obtained. In step 2 of the analysis, he calculated motion at each elevation is employed as input motion to the soil spring along the pile length, and the pile response is evaluated.

5. RESULTS

The results of the site response analysis for the definition of free-field displacements and nonlinear kinematic interaction analyses are reported. The influences of ground motion intensity and its duration on the kinematic soil-pile interaction for different soil types are discussed. Finally, the results obtained from a case study for a pile-column supported bridge structure are presented. The role of kinematic interaction is evaluated comparing the fully-coupled SSI results with those of the non-linear kinematic interaction analysis. The results are reported in terms of bending moments along the pile.

5.1 Nonlinear kinematic interaction

Figure 4a shows the variation of shear modulus and damping ratio with shear strain considered in the site response analysis. Figure 4b presents the calculated acceleration response spectra of the ground surface motion for all considered soil profiles subjected to the Imperial Valley earthquake. In Figure 4b, 100DS, 100SS, 200SS, and 200DS denote sandy deposits, whereas 100SC and 200SC denote clayey soils. Saturated and dry sands, with same shear wave velocity Vs (100DS/100SS or 200DS/200SS), exhibit mostly the same acceleration response spectrum at the ground surface, owing to using the same shear modulus degradation and damping curves. However, saturated clays have different free-field response, being largely in the range of linear elastic behaviour. The scale factors applied to reach the target level of intensity, are evaluated corresponding to the fundamental elastic period of the soil deposit with reference to low-strain mechanical parameters (equal to 0.8 s for Vs = 100m/s soil profiles and to 0.4 s for Vs = 200m/s soil profiles) but, as can be observed from the acceleration elastic response spectra of Figure 4b, a significant shift in the site's fundamental periods is observed after the site response analysis due to the non-linear soil behaviour. Analogous results are also obtained for the other ground motion records.

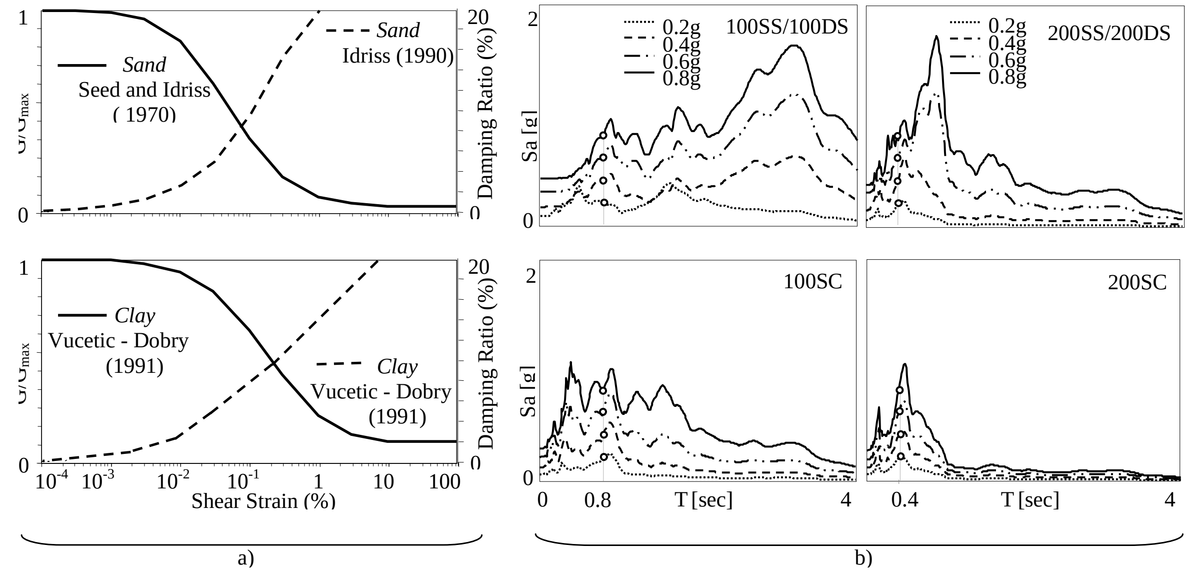

Figure 4. a) Variation of shear modulus and damping ratio with shear strain; b) Imperial Valley earthquake: acceleration response spectra of the ground surface motion

The graphs in Figure 5 show the envelops of maximum and minimum bending moments within the pile obtained from the IDAs for the soil profiles with shear wave velocity equal to 100 m/s (i.e., 100DS, 100SS and 100SC) for the 4 selected records. The responses are generally characterized by a peak value that can occur at the pile head or at a certain depth along the pile. It is observed that the bending moment for piles installed in saturated and dry sands (100SS and 100DS) are comparable, while the ones installed in the saturated clays (100SC) experience almost an order of magnitude smaller moments and are characterized by a different shape.

The difference of moments for piles in sand or clay deposits are principally due to the different behaviour obtained from the site response analysis in the 1st step of the analyses. For the 100DS profile, at level of intensity equal to 0.2 g, the maximum bending moment is localized along the pile at a depth of about 2/3 L_p below the pile head for all selected records and the bending moment at the head is generally much smaller. With increasing levels of seismic intensity, the maximum bending moment, at about 2/3 L_p below the pile head, increases and the bending moment distributions become more severe in the upper part of the pile. In the case of the 100SS profile, the bending moments attain the maximum value at the pile head for both high and low seismic intensities and gradually decrease along the pile. Finally, the 100SC profile is always characterized by smaller values of bending moments with respect to the other profiles. Similarly to the 100SS profile, in this profile, the bending moments achieve the maximum value at the head of the pile and gradually decrease along the pile.

With increasing seismic intensity, the soils exhibit significant non-linear behaviour (cyclic degradation of soil stiffness and strength, soil-pile gap formation with or without cave-in and recompression, soil yielding and radiation damping). The amount of non-linearity associated with different intensity levels affects the way in which maximum bending moments increase. Even for soil profiles with V_s = 200 m/s, the bending moment for piles in saturated and dry sands (200SS and 200DS) are comparable, while the ones relevant to the satured clays (200SC) are almost an order of magnitude smaller. Furthermore, increasing the shear wave velocity of the deposit, the kinematic effects are less evident. The results lead also to the conclusion that the GMD does not significantly affect the non-linear seismic response of the pile. In fact, the different ground motion records, representing three duration scenarios (I_D < 5, I_D < 16 and I_D > 22), do not lead to any particular trend in the results. In some cases, non-linearities are evident in the early part of the response time-history, even at 0.2 g.

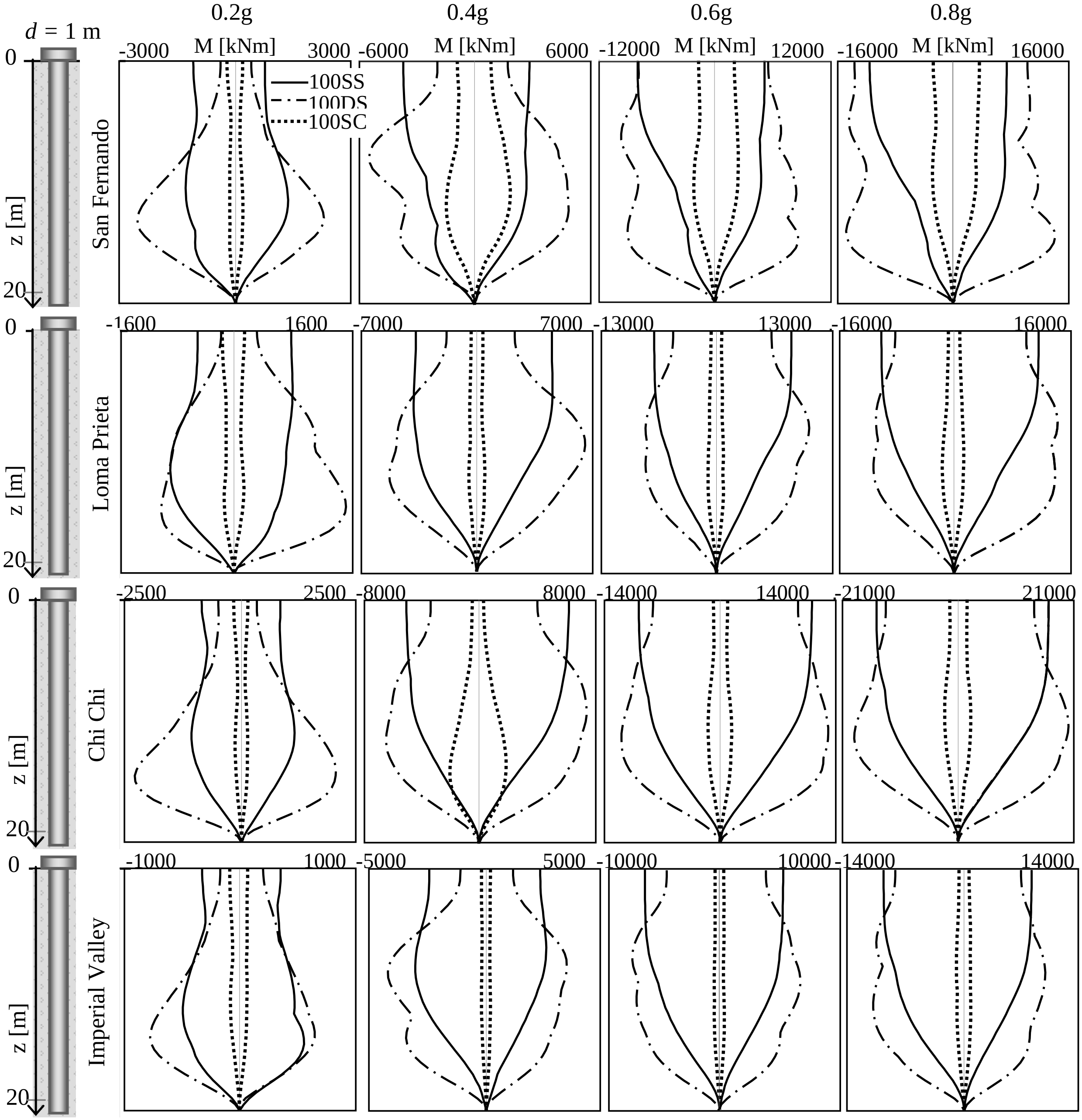

Figure 5. Envelope of bending moments obtained performing IDAs for soil profiles with V_s = 100 m/s.

5.2 Nonlinear soil-pile-structure interaction

Using the method applied in the previous section together with Table 1 and 2, the non-linear dynamic response of a multi-column bridge pier is evaluated. The studied problem is depicted in Figure 6: a pile-column embedded in different homogeneous soil profiles, rotationally restrained at the pile head to simulate the presence of a pile cap. It is assumed that the transverse response of the bridge may be described by the response of a single pier, as would be the case for a multi-spam bridge with coherent ground shaking applied to all piers. The pier height, H = 6 m, its diameter, d =*1* m. The deck mass at the top of the pier is 115 t (calculated by assuming that the 3 columns carry equal loads) and the fundamental period of the fixed-base pier is T=0.55 sec.

The profiles of the bending moments along the pile obtained from the fully coupled SSI analyses are compared with those previously obtained form the kinematic interaction analyses in absence of the superstructure. Figures 7 and 8 show the envelops of maximum and minimum bending moments

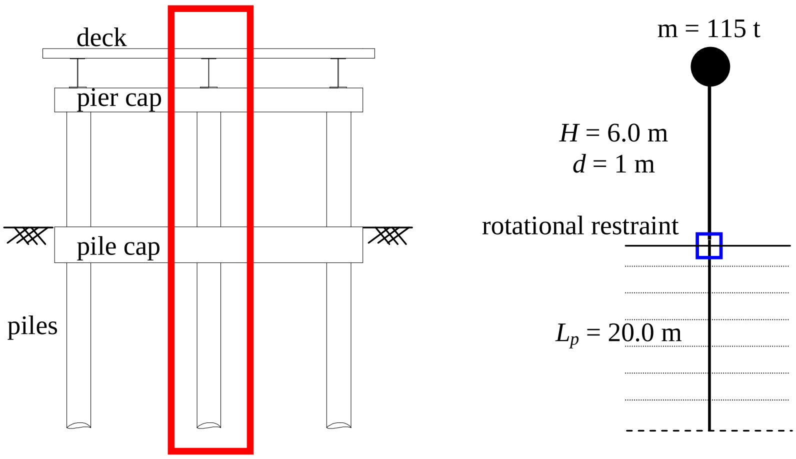

Figure 6. a)Multi-column bent; b) Schematic illustration of the analysed problem.

within the pile obtained from the IDAs for all soil profiles for the Imperial Valley earthquake. The responses are generally characterized by a maximum moment at the pile head since the inertial effects arising from the superstructure have a significant influence at the pile head and attenuate rapidly with depth. For soil profiles with shear wave velocity equal to 100 m/s, it is observed that the kinematic interaction has a strong effect on the pile response both at the head and at greater depth. In the particular case of 100DS soil profile, kinematic bending moments along the shaft are greater than those obtained at the pile head from the non-linear SSI analyses. Furthermore, for the soil profiles with shear wave velocity equal to 200 m/s, kinematic bending moments are less important but are still predominant in the lower portion of the pile.

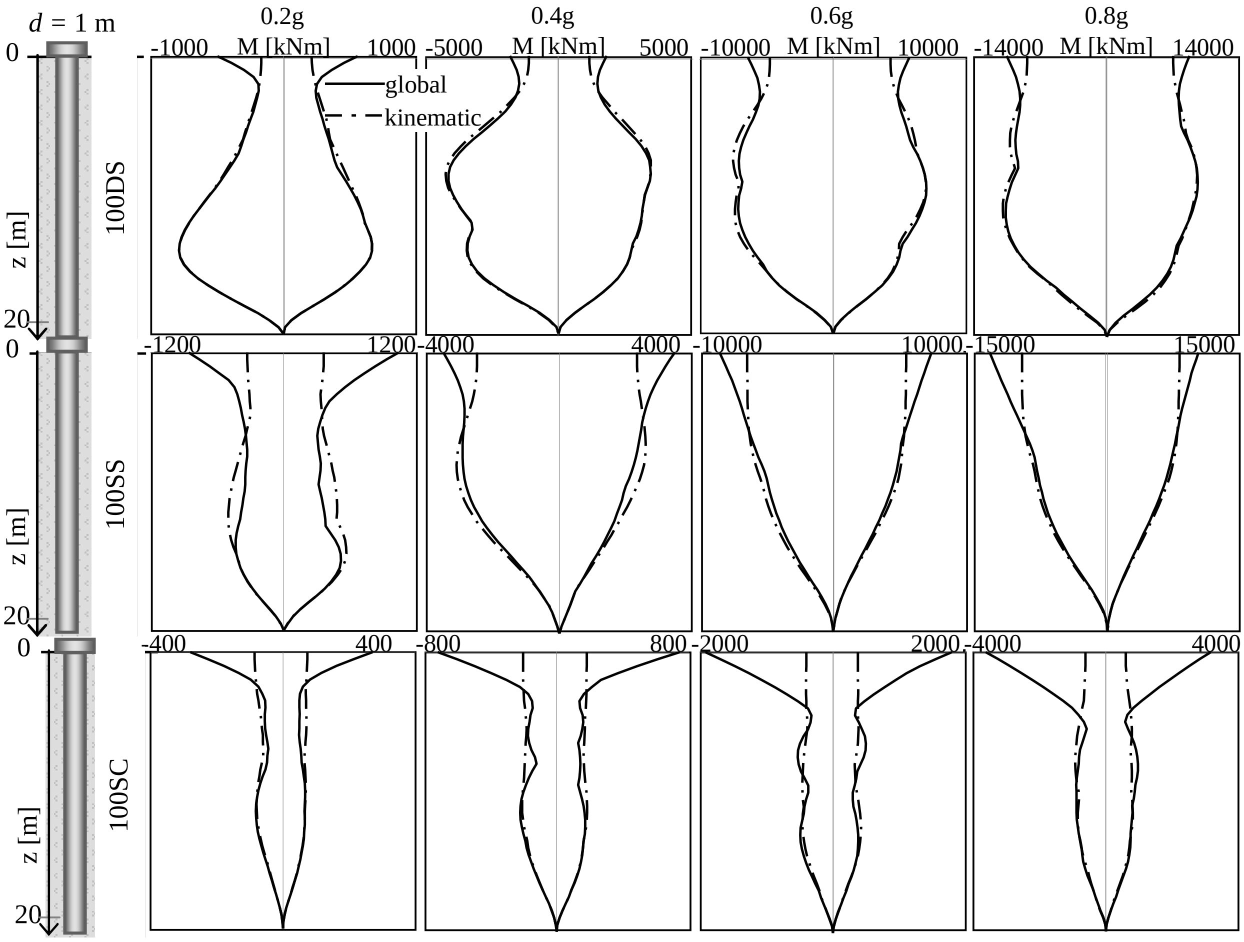

Figure 7. Envelope of bending moments obtained performing IDAs analyses for V_s = 100 m/s.

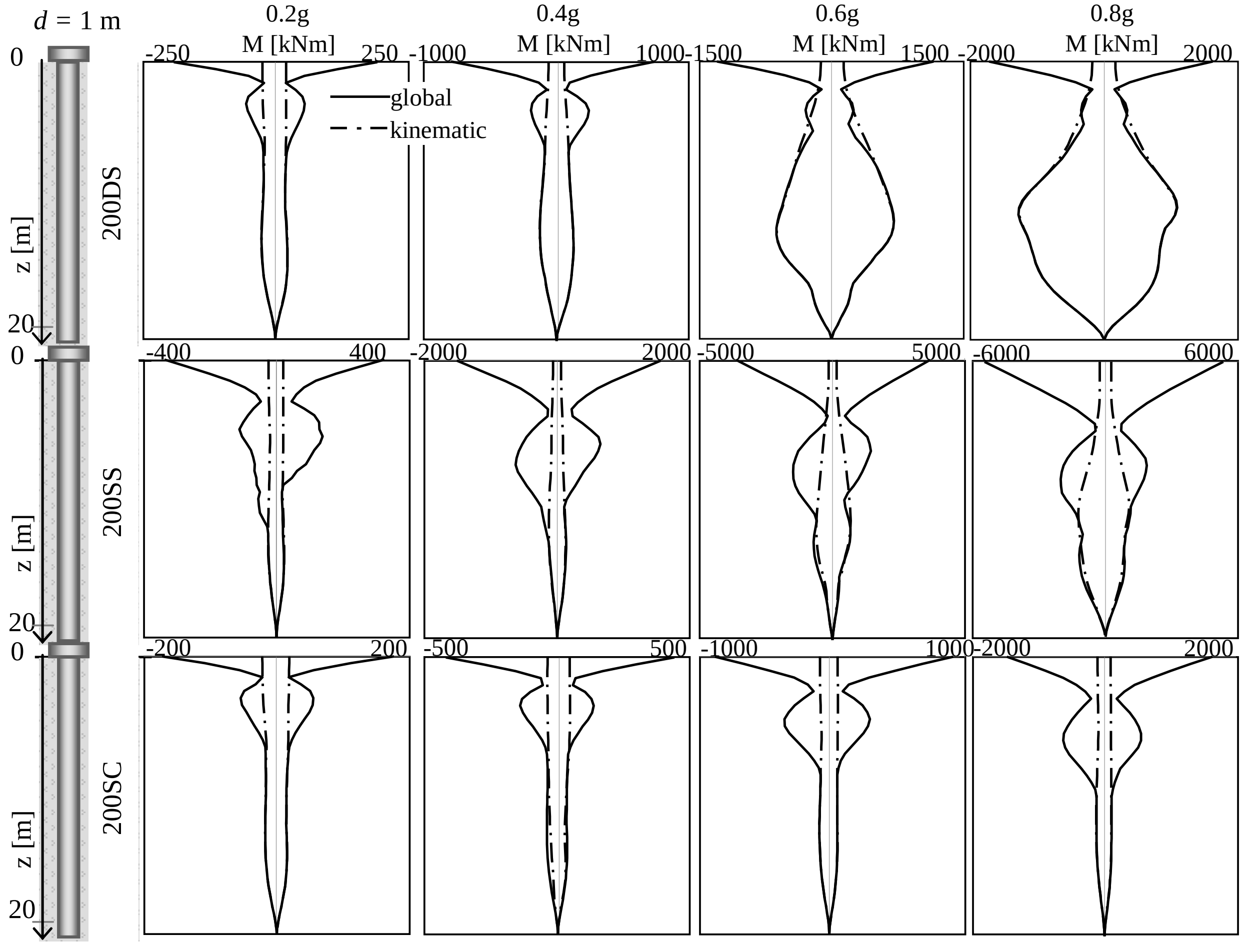

Figure 8. Envelope of bending moments obtained performing IDAs analyses for V_s = 200 m/s.

6 CONCLUSIONS

Incremental Dynamic Analyses have been performed to evaluate the effects of GMD and soil non-linearity on the kinematic interaction of single fixed-head piles in homogeneous soil profiles such as dry sand, saturated sand and saturated clay. A two step uncoupled procedure has been followed in the analysis: firstly, step, the free field motion is evaluated considering an equivalent site response analysis; secondly, the stress resultants in the pile were evaluated using a BNWF model, which is able to account for cyclic soil degradation/hardening, soil and structural yielding, slack zone development and radiation damping. The results have been compared with those obtained using a linear soil-pile model.

The non-linear dynamic response of a multi-column bridge pier has been evaluated to assess the potential for both kinematic and inertial response. The following conclusions may be drawn:

• Pile bending moments are strongly influenced by the type of analysis: linear or linear equivalent site response analysis and linear or non-linear kinematic interaction analysis.
• The maximum pile bending moment and moment variation along the pile shaft are considerably affected by the amount of non-linearity associated with different seismic events.
• Ground Motion Duration (GMD) does not significantly affect the non-linear seismic response of the pile.
• The kinematic effects strongly influence the pile response both at the head and at greater depth especially in soft soil deposits (Vs = 100 m/s).
• The inertial effects are important only in the upper part of the pile.

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