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SEISMIC RESPONSE OF BRIDGE PILE-COLUMNS

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ABSTRACT

While seismic codes do not allow plastic deformation of piles, the Kobe earthquake has shown that limited structural yielding and cracking of piles may not be always detrimental. As a first attempt to investigate the consequences of pile yielding in the response of a pile–column supported bridge structure, this paper explores the soil–pile–bridge pier interaction to seismic loading, with emphasis on structural nonlinearity. The pile-soil interaction is modeled through distributed nonlinear Winkler-type springs and dashpots. Numerical analysis is performed with a constitutive model (Gerolymos and Gazetas, 2005a; 2005b; 2006a) materialized in the OpenSees finite element code (Mazzoni et al. 2005) which can simulate: the nonlinear behaviour of both pile and soil; the possible separation and gapping between pile and soil; radiation damping; loss of stiffness and strength in pile and soil. The model is applied to the analysis of pile-column supported bridge structures, focusing on the influence of soil compliance, intensity of seismic excitation, pile diameter, above–ground height of the pile, and above or below ground development of plastic hinge, on key performance measures of the pier as is: the displacement (global) and curvature (local) ductility demands and the maximum drift ratio. It is shown that kinematic expressions for performance measure parameters may lead to erroneous results when soil–structure interaction is considered.

Keywords: pile, nonlinear, ductility, yielding, soil-structure interaction

INTRODUCTION

Current seismic design of bridge structures is based on a presumed ductile response. A capacity design methodology ensures that regions of inelastic deformation are carefully detailed to provide adequate structural ductility, without transforming the structure into a mechanism. Brittle failure modes are suppressed by providing a higher level of strength compared to the corresponding to ductile failure modes. For most bridges, the foundation system may be strategically designed to remain structurally elastic while the pier is detailed for inelastic deformation and energy dissipation. Essentially-elastic response of the foundation is usually ensured by increasing the strength of the foundation above that of the bridge pier base so that plastic hinging occurs in the pier instead of the foundation.

The concept of ductility design for foundation elements is still new in earthquake engineering practice. The potential development of a plastic hinge in the pile is forbidden in existing regulations, codes and specifications. The main reasons are: (i) the location of plastic hinges is not approachable for post-seismic inspection and repair, (ii) the high cost associated with repair of a severely damaged foundation, and (iii) failure due to yielding in the pile prior to exceeding soil capacity is an undesirable failure mechanism, by contrast to that in which soil capacity is mobilized first.

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However, several case-histories (especially from the Kobe 1995 earthquake) have shown that: (a) pile yielding under strong shaking cannot be avoided, especially for piles embedded in soft soils; and (b) pile integrity checking after an earthquake is a cumbersome, yet feasible task. Furthermore, there are structures where plastic hinging cannot be avoided in members of the foundation during a severe earthquake. A good example of such structure is the pile-column (also known in the American practice as extended pile-shaft), where the column is continued below the ground level as a pile of the same or somewhat larger diameter. Obviously, the design of such foundation requires careful consideration of the flexural strength and ductility capacity of the pile.

In this paper, a parametric investigation of the nonlinear inelastic response of pile-column bridge systems is conducted, and the influence of pile inelastic behavior and soil–structure interaction on structure ductility demand is identified. The role of various key parameters are examined, such as: (a) soil compliance, (b) above-ground height of the column shaft, (c) pile diameter, (d) intensity of the input seismic motion, and (e) location of the plastic hinge, on characteristic performance measures of the soil-structure system response, such as: the displacement (global), μ_δ , and curvature (local), μ_φ , ductility demands and the maximum drift ratio γ_{max}. It is shown that: (a) neglecting the consideration of the soil-structure interaction effects may lead to unconservative estimates of the actual seismic demand, and (b) the development of a plastic hinge along the pile (for instance for cases that the pile is designed with inferior or equal strength compared to that of the pier) is beneficial for the pier response.

PROBLEM PARAMETERS AND ANALYSIS METHODOLOGY

Definition of the problem

The studied problem is sketched in Figure 1: a pile-column embedded in clay or sand deposit, monolithically connected to the bridge deck is excited by a seismic motion. It is assumed that the transverse response of the bridge structure may be characterized by the response of a single bent, as would be the case for a regular bridge with coherent ground shaking applied to all bents.

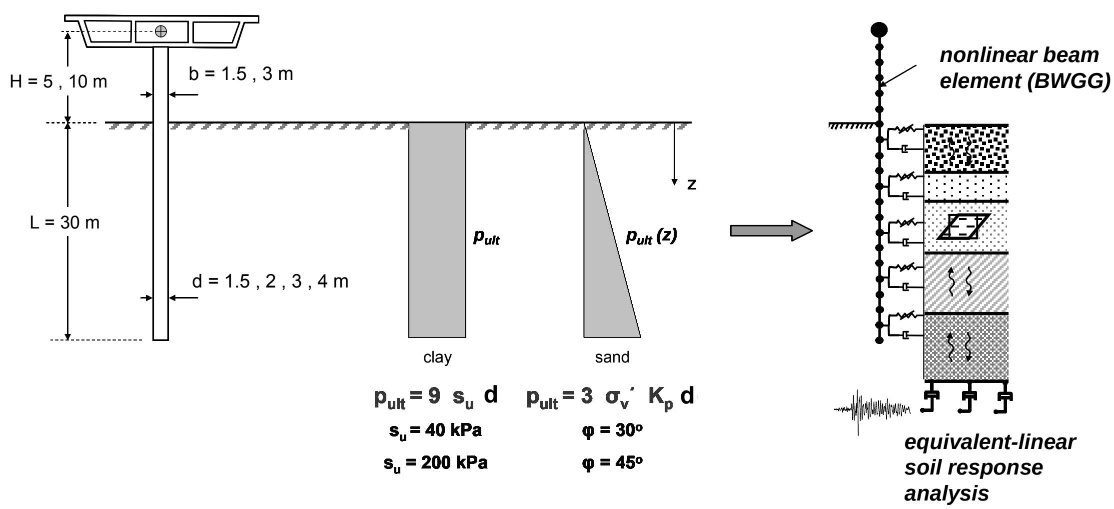

Figure 1. Schematic illustration of the problem investigated and the model used for the analyses

The height of the pier H is given parametrically the values of 5 and 10 m, so that a typical urban bridge and a rather short viaduct, in respect, are examined. The diameter b of the pile-column above-ground takes values of 1.5 and 3.0 m. However, to investigate the influence of the plastic hinge position on the system response, two more cases are examined: the below-ground pile-column diameter d is increased by 33 %

relatively to the above-ground diameter *b*. So, for pile diameters d = 1.5, 2.0, 3.0, and 4.0 m, pier diameter equals to b = 1.5, 1.5, 3.0, and 3.0 m, respectively. For sake of simplicity, the term diameter will refer from this point on, to the below-ground diameter *d*. The embedment length of the pile *L* is considered in every case equal to 30 m. In total, a set of four structural configurations are analyzed.

The mass of the deck is calculated so that the fundamental period of the fixed-base pier would be T = 0.3 sec for all cases studied. The nonlinear behavior of the pile-column is characterized through the predefined moment–curvature relations illustrated in Figure 2. The curves have been obtained with the BWGG model (Gerolymos & Gazetas 2005b) and are representative of a column with uncracked flexural stiffness EI and ultimate strength equal to the conventionally calculated moment at the ground surface considering a critical acceleration of 0.2 g applied on the deck mass (Drosos, 2008).

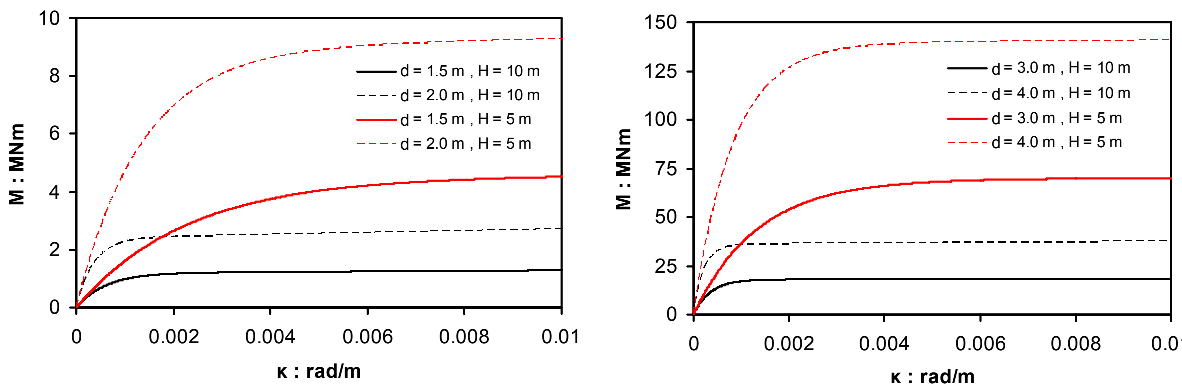

Figure 2. Predefined moment–curvature relations used in the analyses

It is noted that the objective of the parametric study described herein is to investigate the seismic response of the system in the inelastic regime and not to design the structure. Therefore, we are mainly concerned about achieving equivalence of the studied systems in the framework of nonlinear response analysis. The critical acceleration was scaled to 0.2 g, to ensure that the system will enter the inelastic regime under the used seismic excitation.

Soil parameters

The influence of near-field soil compliance on the seismic response of the soil–pile–structure system is investigated parametrically considering four different homogeneous soil profiles (Fig. 1): (a) sand with friction angle $\varphi = 30^o$, (b) sand with friction angle $\varphi = 40^o$, (c) clay with undrained shear strength S_u = 40 kPa, and (d) clay with undrained shear strength S_u = 200 kPa.

The small-amplitude stiffness k (= p_y / y_0) was obtained from the available beam-on-dynamic-Winkler-Foundation solutions (e.g. Gazetas & Dobry 1984, Makris & Gazetas 1992) in terms of the Young's modulus of the soil.

For piles of diameter *d* in cohesive soils the ultimate soil reaction per unit length of pile can be approximated by the well known expression

$$P_y = \lambda_1 S_u d \tag{1}$$

where S_u is the soil undrained shear strength, and λ_1 varies from 9 to 12, depending on the friction ratio f_s / S_u at the pile–soil interface. A value of λ_1 = 9 is often used for a soft clay, while λ_1 = 11 is more appropriate for a stiff clay. For piles embedded in cohesionless soils, Broms (1964) proposed an analytical expression for the ultimate soil reaction :

$$P_y = 3\gamma_s' \, d \tan^2\left(45^\circ + \frac{\phi}{2}\right) z \tag{2}$$

where φ is the angle of friction. Equation 2 is very often preferred in practice among other more rigorous expressions for its simplicity and compatibility with experimental results.

For the description of the nonlinear behavior of the near-field soil the well-known p–y relations of Reese et al. (1974) and Matlock (1970) are used for sand and clay, respectively.

Seismic Excitations and Site Response Analysis

The influence of soil amplification on the seismic response of the soil–pile–structure system is not examined, mainly for two reasons: (a) a thorough investigation of seismic ground response is out of scope of this paper, and (b) the unavoidable differences in free-field motions from the soil response analysis of the four different soil profiles, would complicate the comprehension of the related phenomena. Therefore, a single soil profile was selected for ground response analysis: a category C profile, according to NEHRP (1994) with the bedrock considered to be at 50 m depth.

The influence of shaking on the seismic response is investigated by selecting three real acceleration records as seismic excitations:

- the record from Aegion earthquake (1995),
- the record from Lefkada earthquake (2003), and
- the JMA record from Kobe earthquake (1995).

The first two records are representative strong motions of the seismic environment of Greece, with one and many cycles, respectively. JMA record is used to investigate the dynamic response of the soil–pile–structure system to a quite unfavorable incident. The dominant periods of the acceleration time histories for the aforementioned three earthquake records range from 0.2 to 0.8 s, resulting in a fixed base fundamental period ratio (designated as the fixed base fundamental period of the superstructure divided by the predominant period of the free-field surface acceleration time history) which ranges from 0.66 to 2.67. This is a wide range of values which ensures generalization of the results presented herein. Near-fault effects such as “rupture–directivity” and “fling” (Gerolymos et al. 2005) are also captured by the utilized accelerograms.

All the records were first scaled to a PGA of 0.5 g and 0.8 g at the ground surface; then through deconvolution analyses conducted with SHAKE (Schnabel et al. 1972), the bedrock motion as well as the motion at various depths along the pile, were estimated. The ground motion profiles obtained from SHAKE analyses are then used as input motion in the developed BNWF model. The acceleration time histories at the surface and the corresponding elastic response spectra scaled to a SA = 0.8 g (T = 0 s) for 5 % damping, are presented in Figure 3.

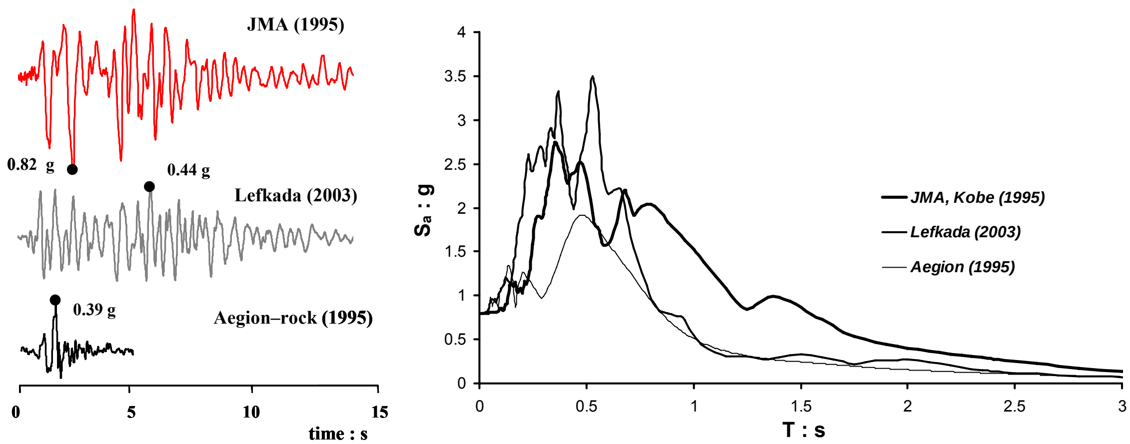

Figure 3. Real acceleration time histories used as seismic excitation, after scaling to a peak ground acceleration of a_g = 0.5 and 0.8 g, and corresponding (ξ = 5 %) response spectra scaled to $S_a^{(T = 0\ s)}$ = 0.8 g.

Constitutive equations and numerical modeling

The developed BWGG model is a versatile one-dimensional action–reaction relationship, capable of reproducing an almost endless variety of stress–strain or force–displacement or moment–rotation relations, monotonic as well as cyclic. It is being applied here to model the monotonic and cyclic response of piles, expressing both the p–y and the moment–curvature relationships. A simple version of the model is outlined below.

The lateral soil reaction against a deflecting pile is expressed as the sum of an elastic and an inelastic component according to:

$$p_x = \alpha_s k_s y + (1-\alpha_s) p_y \zeta_s \tag{3}$$

where p_x is the resultant (in the direction of loading) of the normal and shear stresses along the perimeter of a pile segment of unit length and it includes both "in-phase" and "out-of-phase" components ; the latter reflects radiation and hysteretic damping in the soil. y is the pile deflection at the location of the spring; k_s is a reference spring stiffness ; α_s is a parameter governing the post yielding stiffness ; p_y is a characteristic value of the soil reaction related to the initiation of significant inelasticity (yielding) ; ζ_s is a dimensionless inelastic soil parameter.

The inelastic behavior of the pile is similarly expressed in terms of a strength-of-materials-type bending moment–pile curvature relation, which includes an elastic and an inelastic component:

$$M = \alpha_p E_p I_p \frac{\partial^2 y}{\partial z^2} + (1-a_p)\, M_y\, \zeta_p \tag{4}$$

where E_p I_p is the initial (elastic) bending stiffness (also called flexural rigidity), α_p is a parameter controlling the post yielding bending stiffness, M_y is the value of bending moment that initiates structural yielding in the pile, and ζ_p is the hysteretic dimensionless parameter which controls the nonlinear structural response of the pile.

The expression of variable ζ as well as more details can be found in Gerolymos & Gazetas (2005a, 2005b, 2006a, 2006b), although the model utilized here is a slightly improved/simplified version of the model.

The seismic response of the soil–pile–structure system is simulated herein via a beam-on-nonlinear-Winkler-foundation (BNWF) finite element model developed in OpenSees (Fig. 1).

The pile-column is discretized into nonlinear beam elements with length 0.5 to 1.0 m, whose bending behavior is governed by the macroscopic constitutive BWGG model (Eq. 4). The mass of the deck is simulated as a concentrated mass at the top node of the pile-column, whereas the distributed mass of the extended pile is simulated by lumped masses on beam-element nodes.

The near-field soil–pile interface is simulated with nonlinear p–y spring elements, the behavior of which is described also by the BWGG model (Eq. 3). Model parameters were appropriately calibrated to match the p–y curves of Reese et al. (1974) and Matlock (1970). The free extremities of the soil springs were excited by the acceleration time histories obtained at each depth from the free-field seismic response analysis.

Although the developed finite element model has the capability to reproduce higher order phenomena (e.g. P–Δ effects), such phenomena were ignored, considering that their strong dependence on the mass of the structure and the geometry would obscure the role of other parameters (e.g. structural inelasticity and soil compliance).

Analysis Methodology and Performance Measure Parameters

Besides the fundamental response amounts (acceleration, displacement, moments, etc.) that describe the behavior of a structure under dynamic loading, other important seismic performance measures are the local and global ductility demand μ_φ and μ_δ, and the maximum drift ratio γ_{max}.

The local (curvature) ductility demand μ_φ is defined as the maximum curvature κ_{max} imposed on the structure by an earthquake, divided by the yield curvature κ_y, which is a property of the pile-column cross-section.

$$\mu_\phi = \frac{\kappa_{\max}}{\kappa_y} \tag{5}$$

For bridge structures supported on extended piles, the local ductility demand imposed on the pile shaft might govern the design of the system, because damage to the pile (such as spalling of cover concrete, crack widths, potential for buckling or fracture of longitudinal reinforcement) is related to the local curvature ductility.

The following procedure is followed for the assessment of local curvature ductility demand in the analyses conducted. The moment–curvature curve of each pile-column cross-section is approximated by a bilinear elastic–perfectly plastic relation, in which the first (linear) section is defined as the secant stiffness through the first-yield point κ_{fy} (yielding of first longitudinal reinforcement bar) and the second section by the tangent line on the post-yielding section of the actual moment–curvature curve. The intersection of these two lines defines the cross-section yield curvature κ_y (Fig. 4).

Similarly, the global (displacement) ductility demand μ_δ is the ratio of the maximum displacement of the system u_{max}, imposed by an earthquake, to the yield displacement u_y, which is a soil–pile–structure system property.

$$\mu_\delta = \frac{u_{\max}}{u_y} \tag{6}$$

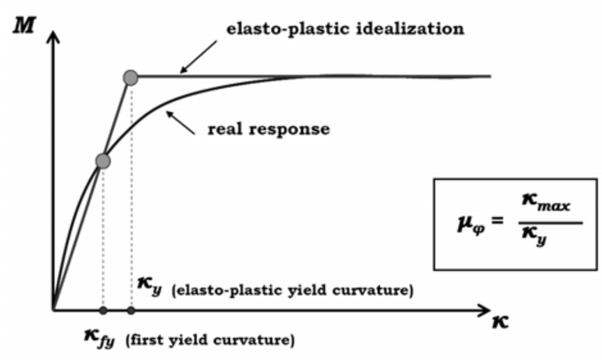

Figure 4. Definition of yield curvature of the soil-pile-structure system

The yield displacement u_y is assessed through static nonlinear analyses (push-over analyses) according to the following procedure: At the center of mass of the superstructure, a horizontal force is gradually applied. The maximum displacement and the curvature along the pile-column are continuously monitored. The displacement measured, when the pile curvature reaches the first-yield point κ_{fy}, is defined as the first-yield displacement u_{fy}. Then, similarly to the procedure followed for the determination of yield curvature, the load–displacement curve is approximated by an equivalent bilinear elastic–perfectly plastic curve, in which the first (linear) section is defined as the secant stiffness through the first-yield point u_{fy} and the second section by the tangent line on the post-yielding section of the load–displacement curve. The intersection of these two lines defines the yield displacement u_y.

It has to be noticed, that for the estimation of pile curvature, we did not use the FEM original curvature results as these showed mesh sensitivity. Instead, plastic rotation results, which are mesh insensitive, were used and divided by the plastic hinge length L_p to derive pile curvature. The length of plastic hinge L_p for the pile-columns was estimated according to Budek et al. (2000) approximation:

$$L_p = d + 0.06 \cdot H \tag{7}$$

where d is the pile diameter and H the above-ground height. Similar expressions, based however on different assumptions, have also been provided in Caltrans (1986,1990), Dowrick (1987), Priestley et al. (1996), Chai (2002), and Chai & Hutchinson (2002).
The drift ratio γ is defined as the maximum displacement of the deck imposed by an earthquake relative to pier base displacement divided by the height of the pier:

$$\gamma = \frac{u_{\max}^{deck} - u_{\max}^{pier-base}}{H} \tag{8}$$

ANALYSIS RESULTS AND DISCUSSION

Due to space restrictions, the results of the conducted nonlinear analyses are presented in terms of performance measure parameters, such as μ_φ, μ_δ, and γ. Results in terms of acceleration time-histories, peak bending moment, curvature and displacement distributions are presented and discussed in details in Gerolymos et al. (2009).

In Figure 5, the correlation of the local curvature ductility demand to the global displacement ductility demand is presented. All the analyses resulted to nonlinear behavior of the extended pile shaft ($\mu_\delta > 1$) are depicted categorized according to the foundation soil. The mean ratio $(\mu_\varphi - 1) / (\mu_\delta - 1)$ equals to 5.4 for soft clay, 3.4 for loose sand, 2.6 for dense sand, and 2.7 for stiff clay. Similar results have been also obtained by Hutchinson et al. (2004). At first sight, it seems that founding pile-columns in soft soils is unfavorable: for a given earthquake imposed global displacement ductility, the local curvature ductility demand is higher than the one corresponds to stiffer soils. This impression, as will be revealed later on, may be deceptive.

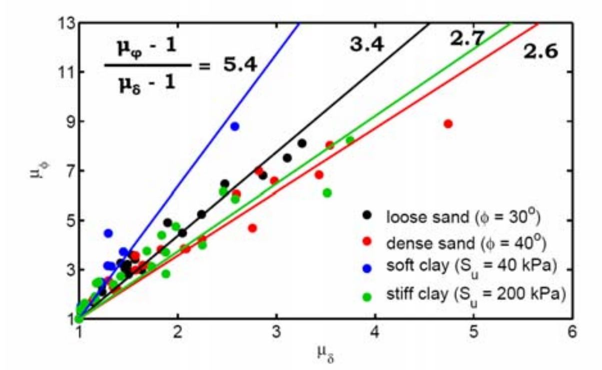

Figure 5. Correlation of local and global ductility demands for different soil types

A similar trend appears in Figure 6 where analyses results have been categorized according to the potential location of plastic hinge. For constant-diameter pile-columns the plastic hinge is likely developed below the ground surface (on pile) whereas for variable-diameter pile–columns, plastic hinges are developed at the base of pier. The average ratio ($\mu_\varphi - 1$) / ($\mu_\delta - 1$) takes a value of 3.5 for plastic hinge on the pile, and 2.7 for plastic hinge on the pier. The results discourage the inelastic design of pile; however, the picture is yet to be cleared.

In the same figure (Fig. 6), analyses results have been grouped according to pier diameter. A slight predominance of the larger pier (d = 3.0 m) is observed as the average value of ($\mu_\varphi - 1$) / ($\mu_\delta - 1$) ratio is 3.3 instead of 3.7 in case of smaller pier (d = 1.5 m).

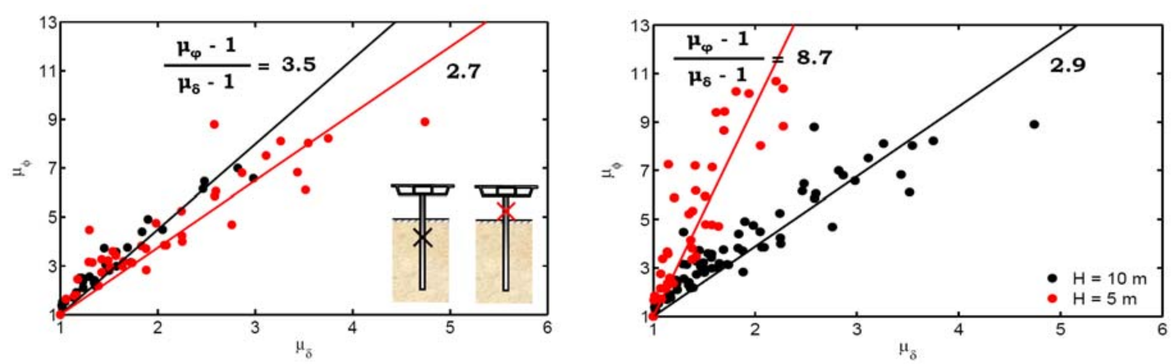

Figure 6. Correlation of local and global ductility demands for different potential location of plastic hinge and above-ground heights

In Figures 7 and 8, the mean and peak values of the factors μ_φ, μ_δ, and γ_{max} are illustrated for various parameters examined. It is clearly observed that the mean and maximum values of both μ_φ and μ_δ factors are lower for soft soils and plasticized piles. This phenomenon discredits the trend appeared in Figures 5 and 6 and reveals the beneficial influence of soil compliance and pile inelasticity on the response of the structure examined. The apparent paradox stems from the fact that kinematic expressions do not distinguish between capacity and demand, as also stated in Mylonakis et al. (2000). For example, according to Figure 5, for a given displacement ductility demand the curvature ductility capacity of a pile-column embedded in soft soil needs to be larger than that of a pile-column embedded in stiff soil. However, this does not mean that for a given seismic excitation both pile-columns would exhibit the same displacement ductility.

Although the ratio $(\mu_\varphi - 1) / (\mu_\delta - 1)$ may take higher values for soft soils, the absolute values of μ_δ are small and so are the values of μ_φ. The maximum drift ratio γ_{max} seems to stay insensitive to parameters like soil stiffness and location of plastic hinges (Fig. 8). On the contrary, it depends strongly on the intensity of the seismic excitation.

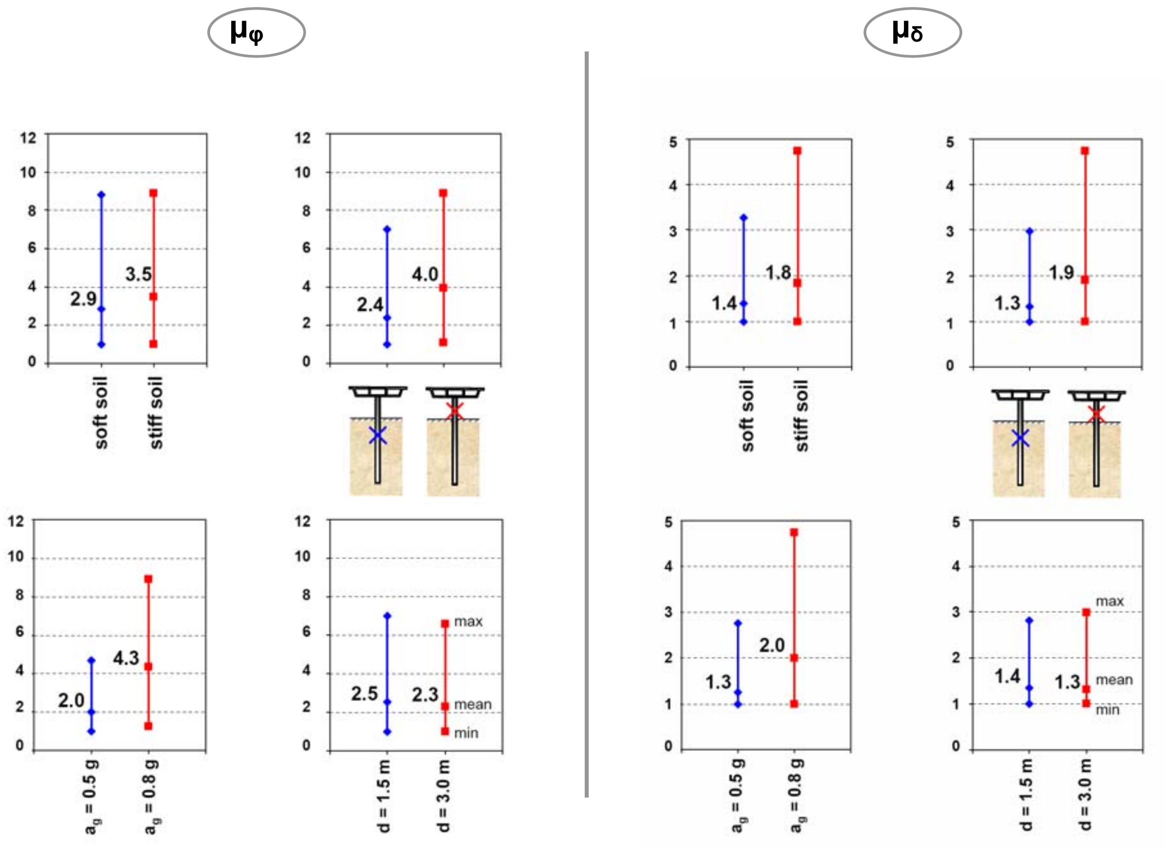

Figure 7. Variation of local (μ_φ) and global (μ_δ) ductility demand for different parameters examined

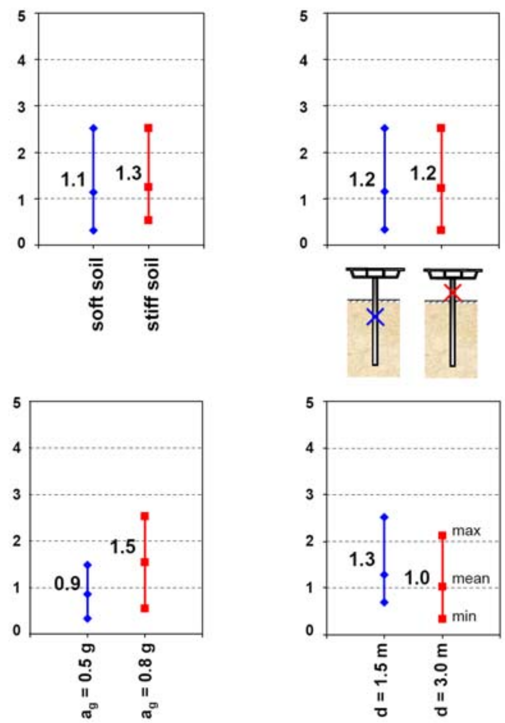

Figure 8. Variation of maximum drift ratio (γ_{max} : %) for different parameters examined

CONJECTURES

From the analysis of the results of the exploratory parametric analyses conducted herein, the following conclusions could be drawn:

For a given global (displacement) ductility demand μ_δ (M–u),

- the local (curvature) ductility demand μ_φ increases for increased soil compliance.
- the potential formation of plastic hinge below ground surface also increases the local (curvature) ductility demand μ_φ (*M–κ*).
- the curvature ductility demand slightly decreases with increasing pile diameter.
- the curvature ductility demand increases in case of column-piles with smaller above-ground height ratios (*d* / *H*).

The opposite trends for the local ductility demand μ_φ are observed, when the maximum drift ratio γ_{max} is kept constant.

However, the conclusions above do not reveal the true nature of the problem and the following remarks should be considered:

- For a given earthquake, the global displacement ductility demand μ_δ decreases as the soil compliance increases. Thus, while $(\mu_\varphi - 1) / (\mu_\delta - 1)$ ratio has a higher value for a soft soil, the small μ_δ demand may refrain the local ductility demand μ_φ at levels lower than what corresponds to a stiffer soil.

- The same comment holds for the location of plastic hinge. The potential of plastic hinge development on the pile (i.e. below ground surface) reduces μ_δ demand, with consequent reduction of local ductility demand.

Most of the available relations for the performance measures in literature are functions of structure geometry and reinforcement details only. However, from the results presented in this paper, the need for modification of these expressions in order to include soil-compliance and pile-plastification effects on structure dynamic response is demonstrated. Some very early, improved $\mu_\varphi - \mu_\delta$ correlations are proposed herein.

Nevertheless, it has to be noted that ductility capacity required in a structure does not always coincides with ductility demand which depends on the characteristics of the seismic loading and inelasticity of soil-pile-structure system. Thus, a structure with higher required ductility capacity may experience lower developed ductility than another structure with lower ductility capacity requirements. The actual ductility demands of a structure can be assessed "accurately" exclusively within the framework of a nonlinear dynamic analysis, in which the influence of soil properties and excitation characteristics are parametrically investigated.

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