SEISMIC RESPONSE OF INELASTIC PILE FOUNDATIONS: A NEW PERFORMANCE BASED DESIGN PHILOSOPHY

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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. This paper focuses on the influence of soil compliance, pile-to-pile interaction, intensity of seismic excitation, pile diameter, above–ground height of the pile, location of plastic hinges (above or below ground development), on the seismic response of pile supported bridge structures. Evaluation of the bridge pier behaviour is achieved through key performance measure indices, as is: the displacement (global) and curvature (local) ductility demands and the maximum drift ratio. It is shown that the ductility demand of a bridge pier decreases with both (a) increasing soil compliance, and (b) below-ground location of plastic hinges development. By exploiting the results, a new performance based design method is developed that allows for soil and pile yielding instead of over-designing the foundation to behave nearly elastically and forcing the potentially developed plastic hinges to occur in the pier (as with conventional capacity design).

INTRODUCTION

In geotechnical earthquake engineering performance based design has, until recently, received little attention. The main reason is the inherent difficulty of obtaining reliable estimates of the induced displacements, which is a prerequisite to a performance based design approach. The successfulness of the performance based design partially hinges on the appropriate choice of a reliable tool to predict the nonlinear behaviour of structures. However, given the inevitable uncertainties in estimating the various geotechnical parameters, the solution of the problem still remains a challenge.

On the other hand, capacity design principles mainly refer to the superstructure, usually underestimating the effect of soil and foundation. Even when foundation compliance is taken into account, little care is given to the nonlinearity of soil and foundation. In fact, current practice in seismic "foundation" design, particularly as entrenched in seismic codes (e.g. EC8), attempts to avoid the mobilization of "strength" in the foundation. In structural terminology: no "plastic hinging" is allowed in the foundation–soil system. In simple geotechnical terms, the designer must ensure that the foundation system will not even reach a number of "thresholds" that would conventionally imply failure.

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 may be strategically designed to remain structurally elastic while the pier is detailed for inelastic deformation and energy dissipation. Thus, the following states are prohibited:

- mobilization of the "bearing-capacity" failure mechanisms under cyclically-uplifting shallow foundations ;

- sliding at the soil–footing interface or excessive uplifting of a shallow foundation ;
- passive and shear failure along the sides and base of an embedded foundation ;
- yielding of below-ground structural members of a foundation (e.g. piles).

This is achieved by introducing overstrength factors plus factors of safety larger than 1 against each of the above failure modes. Although such a restriction may appear reasonable (the inspection and rehabilitation of foundation damage after a strong earthquake is not an easy task), it may lead to high-cost design solutions which are not necessarily associated with optimal performance of the structure in the case of occurrence of ground motions larger than design (Anastasopoulos et al., 2009). Moreover, 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.

The issue addressed in this paper, involves the parametric investigation of the nonlinear inelastic response of a single column bent on pile (Gerolymos et al, 2009). The influence of pile inelastic behavior and soil-structure interaction on structure ductility demand is identified, and 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, (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, and (c) the ductility demands on the superstructure decrease with increasing soil compliance.
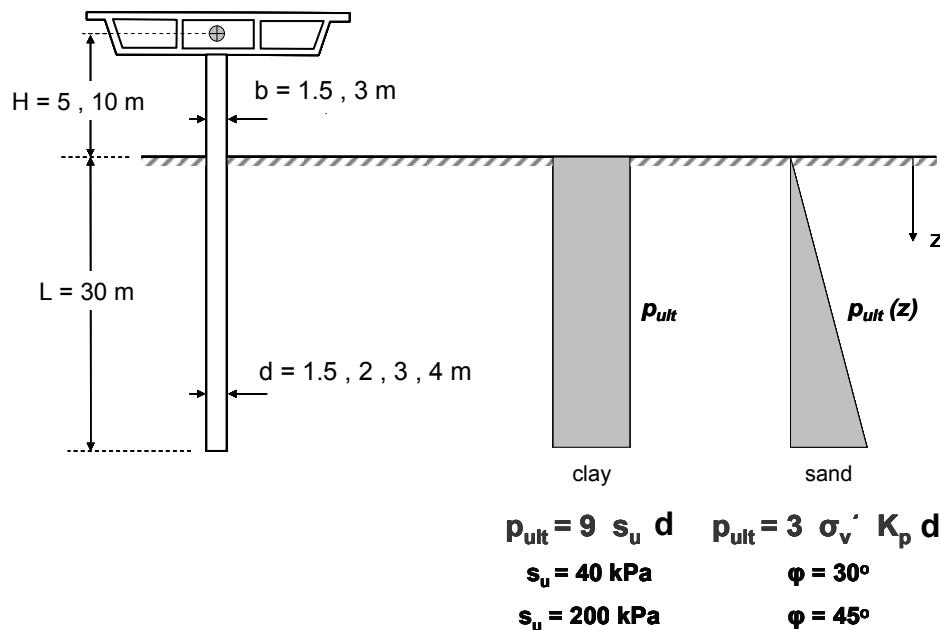

Fig 1. The problem investigated and the two types of presumed soil deposits

THE STUDIED PROBLEM

The studied problem is sketched in Fig 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.

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 analysed.
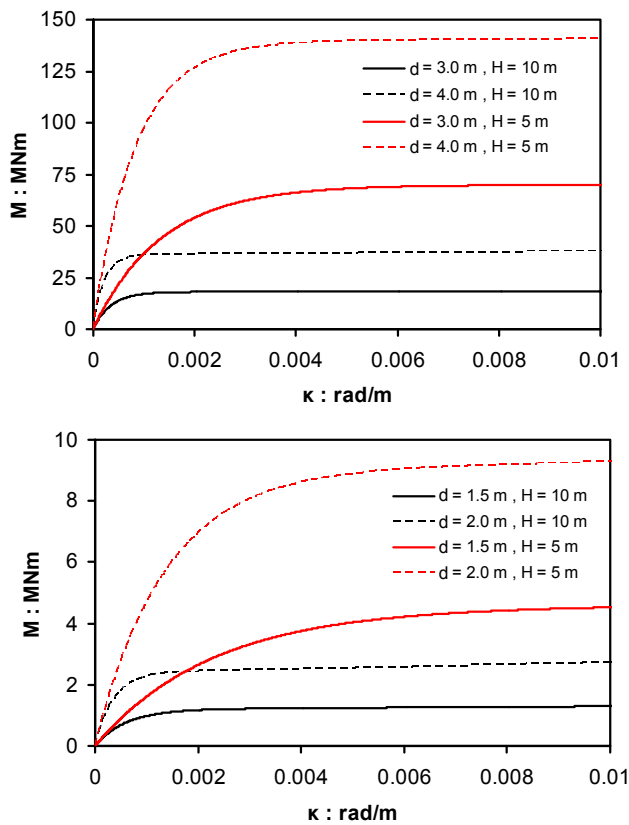

Fig 2. Predefined moment–curvature relations used in the analyses

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. This restriction for the fixed-base period leads to a mass of 45 Mg for the pile diameter of $d = 1.5$ m, and 720 Mg for that of $d = 3.0$ m, in the case of the tall pier. The nonlinear behavior of the pile-column is characterized through the predefined moment–curvature relations illustrated in Fig 2. These curves have been obtained with the BWGG model (Gerolymos and Gazetas, 2005), for $n = 1$, initial stiffness equal to the uncracked flexural stiffness EI of the pile-column, and ultimate strength equal to the conventionally calculated

moment at the ground surface considering that a critical acceleration of 0.2 g is applied on the deck mass. In the case of the variable-diameter piers, the bending moment capacity of the pile cross-sections is calculated to be proportional to the square power of the cross-section diameter d^2, which is a reasonable assumption for a given detailing of reinforcement. In that way, the potential development of a plastic is forced to occur in the above-ground portion of the pile-column.

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, (a) we are mainly concerned about achieving equivalence of the studied systems in the framework of nonlinear response analysis without considering soil-structure interaction effects, rather than about reinforcement details that correspond to the utilized moment–curvature curves. And (b) the critical acceleration was scaled to 0.2 g, to ensure that the system will enter the inelastic regime under the used seismic excitation.

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 $\varphi = 30^{\circ}$, (b) sand with friction angle $\varphi = 40^{\circ}$, (c) clay with undrained shear strength $S_u = 40$ kPa, and (d) clay with undrained shear strength $S_u = 200$ kPa.
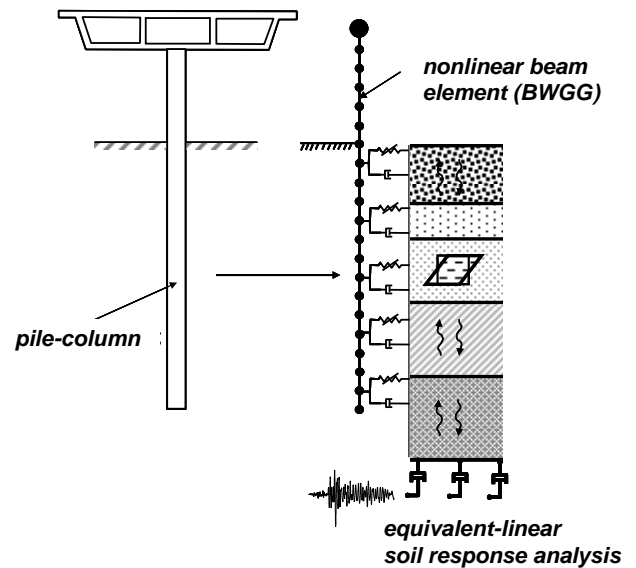

Fig 3. Schematic illustration of the model used for the analyses

SOIL PROFILES AND SEISMIC EXCITATIONS

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). Bedrock was assumed 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
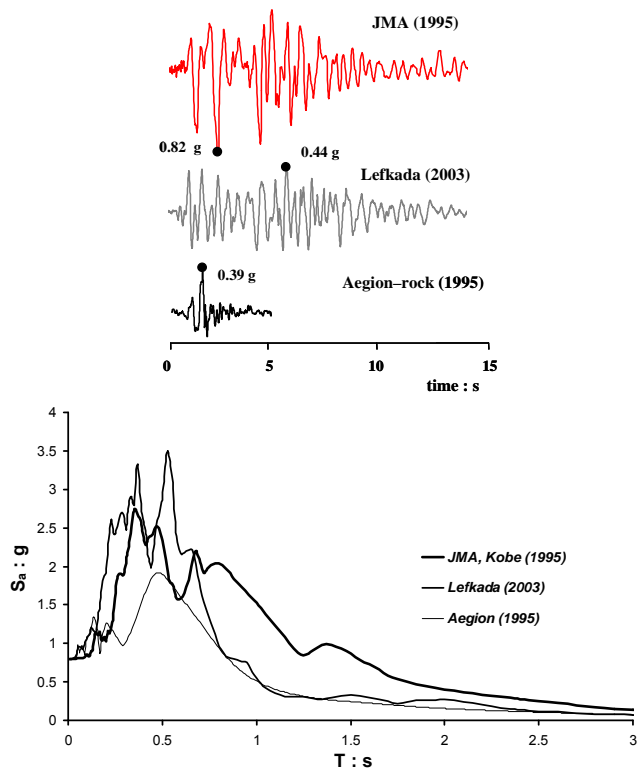- the JMA record from Kobe earthquake (1995).

Fig 4. 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 ($\xi = 5$ %) response spectra scaled to Sa (T = 0 s) = 0.8g.

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 (Fig 3). The acceleration time histories at the surface and the corresponding elastic response spectra scaled to a SA (T = 0 s) = 0.8 g for 5 % damping, are presented in Fig 4.
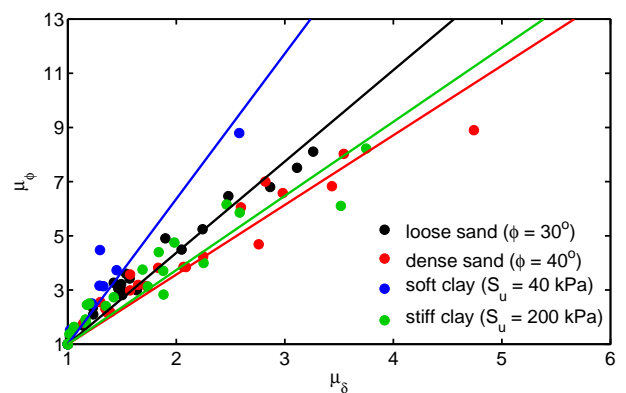

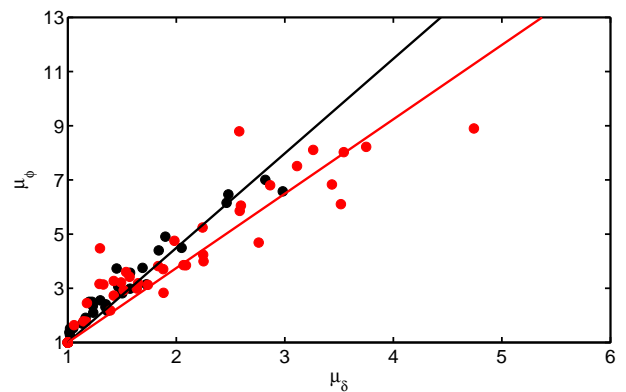*Fig 5. Correlation of local and global ductility demands for different soil types*

Fig 6. Correlation of local and global ductility demands for different plastic hinge locations

It should be stated here in that from a seismological point of view, simply scaling an acceleration time history to a large PGA value for representing the severity of an earthquake might not be always correct. It is well known from the literature that high peak ground accelerations are usually

accompanied by a large number of predominant cycles. Obviously, this is not the case for Aegion record which can be satisfactorily approximated by a single sinusoidal pulse.

RESULTS OF ANALYSIS

In Fig 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.
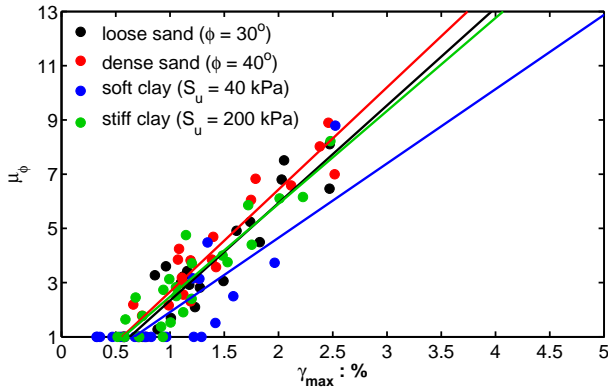

Fig 7. Correlation of local ductility demand and maximum drift ratio for different soil types
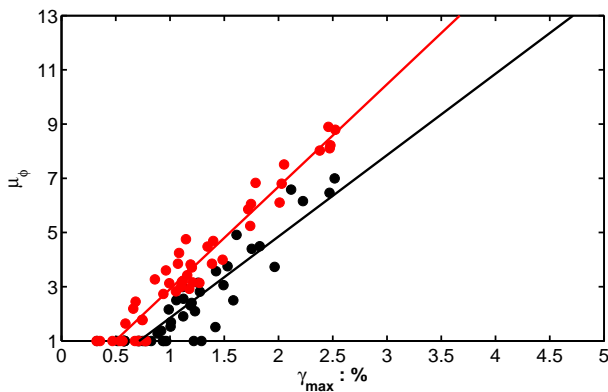

Fig 8. Correlation of local ductility demand and maximum drift ratio for different plastic hinge locations

A similar trend appears in Fig 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.
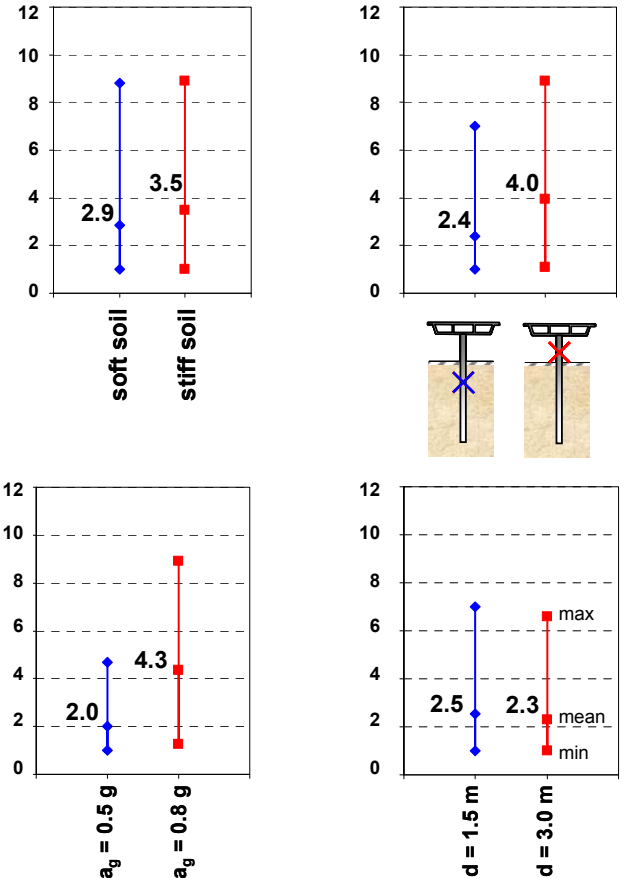

Fig 9. Variation of local curvature ductility (μ_φ) demand for different parameters examined

In Fig 7, the correlation of local curvature ductility demand to the maximum drift ratio is presented for all the soil profiles examined. For a given maximum drift ratio, the required curvature ductility is greater for stiffer soils. The depth of the plastic hinge location increases with decreasing soil stiffness resulting in larger rigid body displacement, which however is not associated with strain in the pier. An inversion in the trend observed earlier is evident.

The same trend is observed in Fig 8, where the effect of plastic hinge location is examined: for a given maximum drift ratio, the required curvature ductility is greater when the pier is plasticized. Indeed, the rigid body motion component of the displacement which increases with increasing depth of plastic

hinge location, does not produce any structural damage and hence does not affect the ductility demand on the pier.
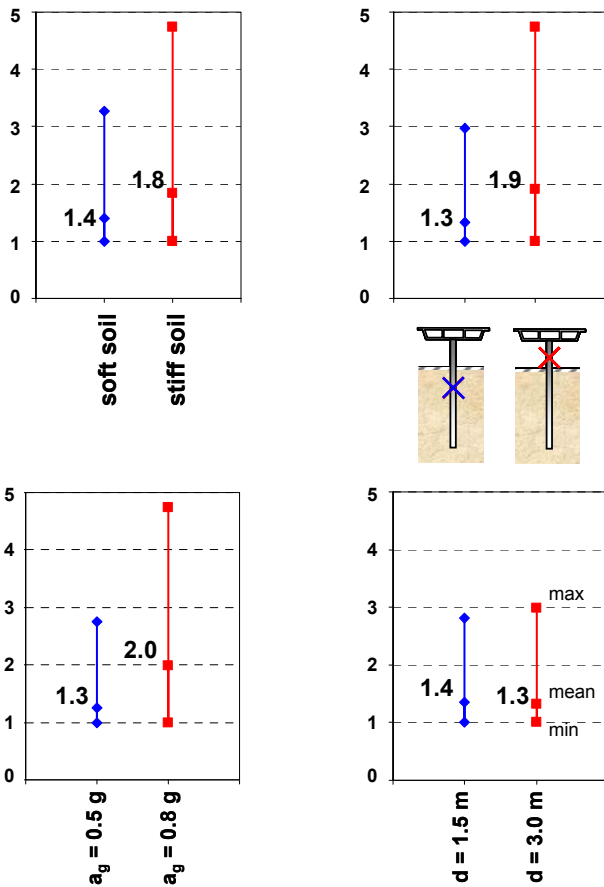

Fig 10. Variation of global curvature ductility (μ$_\delta$) demand for different parameters examined

In Figs 9 and 10, the mean and peak values of the factors μ_φ, μ_δ, 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 Figs 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 Fig 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.

CONCLUSIONS

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

For a given global (displacement) ductility demand μ$_\delta$ (*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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