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SELECTION CRITERIA FOR PILE DIAMETER IN SEISMIC AREAS

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ABSTRACT

According to modern seismic codes such as Eurocode 8, pile foundations in earthquake-prone areas must resist two different, yet simultaneous bending actions resulting from kinematic and inertial interaction. Due to the different nature of the two demands, pile must resist seismic actions following different patters, thus leading to different design requirements. In this work, analytical solutions are presented to define maximum and a minimum pile diameters required to resist kinematic and inertial effects in an essentially elastic manner, respectively. It is shown that the range of admissible diameters decreases with decreasing soil stiffness and with increasing design acceleration, collapsing into a single admissible diameter for certain problem configurations. Regions where no pile diameter can guarantee elastic response during strong seismic shaking are identified.

Keywords: pile diameter, kinematic interaction, inertial interaction, seismic soil-structure interaction.

INTRODUCTION

During strong earthquake shaking, piles are subjected to two different, yet simultaneous bending actions: (1) forces and moments imposed onto the pile heads due to structural vibrations transmitted through the cap (inertial forces), (2) curvatures imposed along the piles by the surrounding soil which deforms under the passage of the impinging seismic waves (kinematic effect). These types of loading are presented schematically in Fig. 1. Whereas inertial interaction is a well-known phenomenon, as it can be fully described in terms of imposed forces, kinematic interaction is more difficult to interpret and quantify, thereby it is rarely accounted for in design. On the other hand, analytical works (Margason, 1975; Dobry and O'Rourke, 1983; Mylonakis, 2001) and post-earthquake investigations have demonstrated the importance of the phenomenon, by revealing damage at depths where inertial forces are typically negligible, in deposits that have not suffered a loss of strength such as that induced by soil liquefaction (Mizuno, 1987; Nikolaou et al., 2001). The accumulated evidence has been recognized by some seismic codes, which require evaluating kinematic effects under certain conditions. For example Eurocode 8 states that: "*piles shall be designed to resist the following two types of action effects: (a) inertia forces from the superstructure... ; (b) kinematic forces arising from the deformation of the surrounding soil due to the passage of seismic waves*", and: "*bending moments developing due to kinematic interaction shall be computed only when all of the following conditions occur simultaneously: (1) the ground profile is of type D, S1 or S2, and contains consecutive layers of sharply differing stiffness; (2) the zone is of moderate or high seismicity, i.e. the product (a_gS) exceeds 0.10g; (3) the supported structure is of class III or IV*". Notwithstanding the importance of kinematic bending moments at soil layer interfaces, a significant

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amount of published information (Kavvadas & Gazetas 1993, Nikolaou et al., 2001; Di Laora, 2009; Di Laora et al., 2011) has revealed the existence of significant kinematic bending at the pile head even in homogeneous, when shear soil wave propagation velocity is sufficiently low. Di Laora (2009) and de Sanctis et al (2010) showed that a long fixed-head pile in homogeneous soil experiences a curvature at the top, which is approximately equal to that of soil in free-field conditions (i.e., pile-soil interaction is negligible).
In one-dimensional conditions, curvature in homogeneous soil is:

$$\left(1/R\right)_s = \frac{a_s}{V_s^2} \tag{1}$$

where a_s and V_s are the acceleration and the shear wave velocity of the soil.
Assuming equal soil and pile curvatures, the kinematic moment at pile head can be determined as:

$$M_{kin} = E_p I_p \left(1/R\right)_p = E_p I_p \left(1/R\right)_s = E_p I_p \frac{a_s}{V_s^2} \tag{2}$$

where $(1/R)_p$, E_p and I_p are curvature, Young's modulus and cross-sectional moment of inertia of the pile, while a_s is the horizontal acceleration at soil surface. It is noted that head moment increases linearly with increasing design acceleration and parabolically with decreasing soil stiffness.

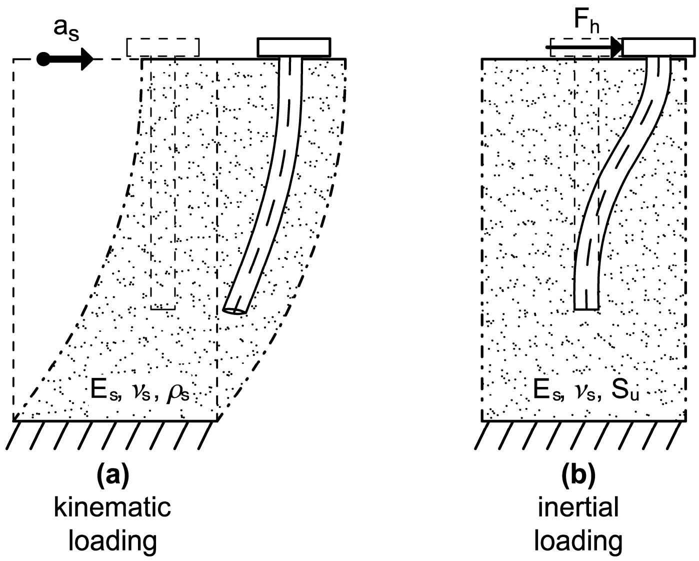

Figure 1. Kinematic and inertial loading

The above equation also indicates that kinematic moment is proportional to the fourth power of the pile diameter (i.e., proportional to d^4). As the moment capacity of a circular cross section M_y, for a homogeneous material, is proportional to the third power of pile diameter (i.e., proportional to d^3), it follows that kinematic action prevails over section capacity with increasing pile diameter. This suggests that there exists a *maximum diameter* beyond which the pile will not be able to withstand the kinematically imposed bending moments in an elastic manner. On the other hand, inertial actions provide a *minimum diameter*, as shown in the ensuing.

For the purposes of this investigation and focusing on fine-grained soils, it is reasonable to assume that the load carried by the pile under working conditions, W_p, is proportional to pile diameter (i.e., proportional to shaft resistance) and, therefore, that the inertial moment acting upon the pile is proportional to $W_p \times d$ (i.e., proportional to d^2). The above observations have two main consequences, as illustrated in Fig. 2:

1) Kinematic moments (M_{kin}) at the pile top tend to dominate over inertial ones (M_{in}) with increasing pile diameter;
2) Only a limited range of diameters allows a pile to undertake both kinematic and inertial moments ($M_{tot} < M_y$) and, therefore, is admissible from a seismic design viewpoint.

Whereas the first aspect has already been analyzed (Di Laora, 2009; Di Laora and Mandolini, 2011), the issue of selecting a proper diameter when designing a pile seems to be little explored (Di Laora et al., 2011; Saitoh, 2005), and not covered by seismic codes. The analytical study presented in this paper aims at exploring the range of admissible diameters for steel piles with reference to head bending. Design issues deriving from such investigations are also discussed.

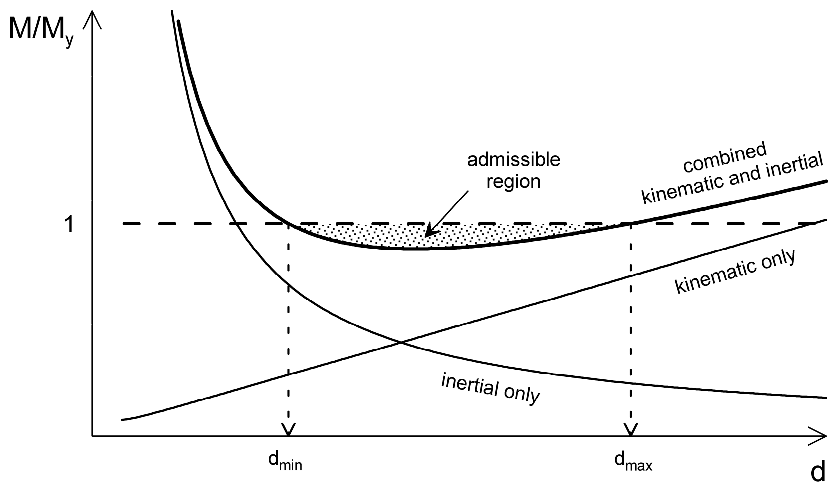

Figure 2. Kinematic, inertial and yield moment as function of pile diameter

ADMISSIBLE DIAMETERS FOR KINEMATIC AND INERTIAL LOADING

Yield moment

With reference to a cylindrical steel pile, the cross-sectional yield moment can be computed as:

$$M_y = E_p I_p \varepsilon_y \frac{2}{d}\left(1 - \frac{W_p}{f_y A}\right) \tag{3}$$

ε_y and f_y being the uni-axial yield strain and the corresponding stress of the material, A the cross-sectional area and W_p the axial load carried by the pile.

Considering a perfectly floating pile (i.e., a pile with bearing capacity controlled exclusively by shaft resistance) in cohesive soil, W_p can be expressed as:

$$W_p = \frac{\pi \alpha}{SF} S_u L\, d \qquad (4)$$

where S_u is the undrained shear strength of the soil material, α the pile-soil adhesion coefficient (typically ranging from 0.3 to 1), and *SF* a global safety factor against axial bearing capacity failure.

Kinematic Loading

For a flexible pile in homogeneous soil, setting the kinematic demand moment in Eq. (2) equal to the yield moment in Eq. (3) and considering an axial load W_p given by Eq. (4), one obtains the following second-order algebraic equation:

$$\frac{1}{2\varepsilon_y}\frac{a_s L}{V_s^2}\left(\frac{d}{L}\right)^2 - \left(\frac{d}{L}\right) + \frac{4\alpha}{q_A\, SF}\frac{S_u}{f_y} = 0 \qquad (5)$$

Equation (5) admits the solutions (roots)

$$d_{kin} = 2\varepsilon_y \frac{V_s^2}{a_s}\left[\frac{1}{2} \pm \sqrt{\frac{1}{4} - \frac{1}{\varepsilon_y}\frac{2\alpha}{q_A\, SF}\left(\frac{V_{s1}^2}{a_s L}\right)^{-1}\left(\frac{S_u}{f_y}\right)}\,\right] \qquad (6)$$

the larger of which corresponds to the critical (maximum) pile diameter to withstand kinematic action. In the above expressions, $q_A = 1-(1-2t/d)^2$ is a dimensionless factor accounting for wall thickness, *t*, of a hollow pile. It should be noticed that in the realm of the above derivation, W_p was assumed not to generate lateral inertial loads on the pile (kinematic problem).

The above expression can be cast in the alternative form:

$$d_{kin} = 2\varepsilon_y \frac{V_s^2}{a_s}\left[\frac{1}{2} \pm \sqrt{\frac{1}{4} - \frac{1}{\varepsilon_y}\frac{6\,\rho_s\,\alpha\, a_s L}{q_A\, SF}\left(\frac{E_s}{S_u} f_y\right)^{-1}}\,\right] \qquad (7)$$

which has the advantage that the term in brackets does not depend on soil stiffness and strength separately, but only on their ratio.

In the extreme case of a pile carrying zero axial load ($W_p = 0$, which implies infinite safety against bearing capacity failure; $SF \to \infty$), the term in brackets in Eqs. (6) and (7) tends to unity and the solution reduces to the simple expression:

$$d_{kin} = 2\varepsilon_y \frac{V_s^2}{a_s} \qquad (8)$$

which can be obtained directly from Eqs. (2) and (3). As the vertical load carried by the pile is much smaller than its axial structural capacity, Eq. (8) can be used instead of Eq. (7), with an error of less than 2-3%.

Inertial Loading

Considering exclusively inertial action and assuming, as a first approximation, that the lateral load imposed on the pile is proportional to the axial gravitational load W_p, it is straightforward to show that the maximum moment at the pile head is

$$M_{in} = \frac{1}{4}\left(\frac{\pi q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} S_a\, W_p\, d \tag{9}$$

δ being the Winkler stiffness parameter (which of the order of 1 to 2 for inertial loading, as indicated by Roesset, 1980), $q_I = 1-(1-2t/d)^4$ a dimensionless factor accounting for the wall thickness t of a hollow pile, S_a a dimensionless spectral amplification parameter, and g the acceleration of gravity. Setting the right sides of Eqs (3) and (9) equal, and employing Eq. (4), the following solution is possible:

$$d_{in} = \frac{8\alpha}{SF} L \left[\frac{S_a}{\varepsilon_y}\left(\frac{\pi}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(q_I \frac{E_p}{E_s}\right)^{-\frac{3}{4}}\left(\frac{S_u}{E_s}\right) + \frac{1}{2q_A}\left(\frac{S_u}{f_y}\right)\right] \tag{10}$$

which defines a critical (minimum) pile diameter to withstand inertial action. In the limit case of zero ground acceleration ($a_s = 0$), Eq. (10) reduces to

$$d_{in} = \frac{4\alpha}{SF\, q_A}\left(\frac{S_u}{f_y}\right) L \tag{11}$$

which corresponds to the minimum diameter to resist the gravitational load W_p. The same result can be obtained by setting $a_s = 0$ in Eq. (5).

Combined Kinematic & Inertial Loading

For the more realistic case of combined kinematic and inertial loading, Eqs. (2) and (9) can be added for the overall flexural earthquake demand at the pile head through the simplified expression

$$M_{tot} = M_{kin} + M_{in} \tag{12}$$

where subscript *tot* stands for "total". In the above equations, M_{kin} and M_{in} should be interpreted as statistical quantities (i.e., quantities multiplied by pertinent load combination factors), to account for possible phase differences between maximum kinematic and inertial actions.
Setting the above moment equal to the yield moment in Eq. (3), one obtains the second-order dimensionless equation

$$\frac{1}{2}\frac{a_s L}{V_s^2}\left(\frac{d}{L}\right)^2 - \varepsilon_y\left(\frac{d}{L}\right) + \frac{4\alpha}{q_A SF}\left(\frac{S_u}{E_p}\right)\left[1 + 2\frac{q_A}{q_I}\left(\frac{\pi q_I}{\delta}\right)^{\frac{1}{4}}\left(\frac{a_s}{g}\right)\left(\frac{E_p}{E_s}\right)^{\frac{1}{4}} S_a\right] = 0 \tag{13}$$

which can be solved analytically for the pair of pile diameters

$$d_{1,2} = \frac{\varepsilon_y V_s^2}{a_s} \left\{ 1 \mp \sqrt{1 - \frac{24\alpha \rho_s a_s L}{q_A f_y \varepsilon_y SF} \left(\frac{E_s}{S_u} \right) \left[1 + 2 \frac{q_A}{q_I} \left(\frac{\pi q_I}{\delta} \right)^{\frac{1}{4}} \left(\frac{a_s}{g} \right) \left(\frac{E_p}{E_s} \right)^{\frac{1}{4}} S_a \right]} \right\} \tag{14}$$

that correspond to a minimum (d_1) and a maximum (d_2) obtained for the upper (−) and lower sign (+), respectively. Values between these two extremes define the range of admissible diameters for the conditions specified by the dimensionless ratios in the right side of Eq. (14).

RESULTS

With reference to a hollow steel pile, the range of admissible diameters is plotted in Fig. 3, as function of soil shear wave velocity V_s, for different values of surface seismic acceleration (a_s/g) and pile lengths L. Evidently, critical values are increasing functions of V_s leading progressively to a wider range of admissible diameters. The curves for purely kinematic (narrow dash) and inertial action (large dash) in Eqs. (7) and (10) bound the admissible range from above and below, respectively, which suggests that kinematic and inertial moments interact detrimentally for pile safety. The interaction is more pronounced with increasing pile length and seismic acceleration. Note that for piles in very soft soil such as peat (V_s less than 50 m/s), maximum pile diameter may be of the order of 1 m, as evident in the graphs.
It is worth mentioning that there is always a minimum soil shear wave velocity for which the admissible range collapses to a single point corresponding to a unique admissible pile diameter (i.e., $d_1 = d_2$). This diameter can be obtained by setting the term in square root in Eq. (14) equal to zero, to get

$$d_1 = d_2 = \frac{\varepsilon_y V_s^2}{a_s} \tag{15}$$

Remarkably, this value is exactly one half that obtained for kinematic action alone under zero axial force (Eq. 8). Note that this diameter is independent of pile Young's modulus, E_p, and wall thickness t.
Such diameter, and the correspondent (minimum) stiffness, will be referred in the ensuing as "critical". In this context, the shear wave velocity in the above equation assumes the particular meaning of critical stiffness. Its value may be derived setting the square root in Eq. (14) equal to zero. Following some trivial mathematical manipulation, the critical velocity is obtained as:

$$V_{s,crit} = \frac{4}{\sqrt{3\rho_s}} \left[\frac{\frac{q_A}{q_I} \left(\frac{\pi q_I}{\delta} \right)^{\frac{1}{4}} \frac{a_s}{g} E_p^{\frac{1}{4}} S_a}{\frac{q_A \varepsilon_y^2 SF E_p}{24 \alpha a_s L \rho_s} \frac{E_s}{S_u} - 1} \right]^2 \tag{16}$$

Evidently, for shear wave velocities less than critical, no real-valued pile diameter can be predicted from Eq. (14), which suggests that it is impossible for the pile head to stay elastic under fixed-against-rotation conditions.

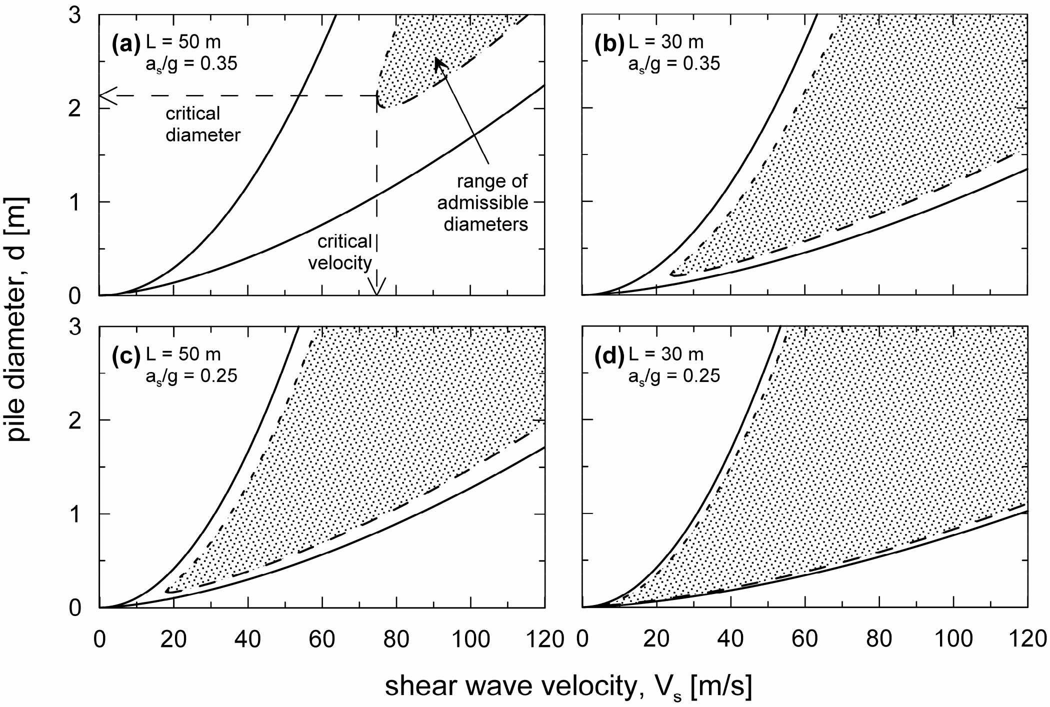

Figure 3. Admissible pile diameters against soil shear wave velocity (E_s/S_u = 500, f_y = 275 MPa, E_p = 210 GPa, ν_s = 0.5, S_a = 2.5, FS = 3, t/d = 0.015, α = 0.7, δ = 1.2). Continuous lines represent pure kinematic and inertial actions whereas dashed lines refer to the combined action.

The influence of design acceleration is explored in Fig. 4, where the region of admissible diameters is plotted for different values of a_s/g and two different pile lengths (L = 30, 50m). The first noteworthy observation is that although pile length does not affect pure kinematic limit diameter (Eq. 7), it plays a major role in determining the maximum admissible diameter, which also depends on inertial interaction. Second, for long piles in soft soil, low values of design acceleration are admissible.

Figure 5 depicts the values of critical wave velocity as function of design acceleration, for different values of pile length and wall thickness. It is observed that the minimum admissible soil stiffness increases with a_s/g and L, due to the progressively detrimental effect of inertial moments. On the contrary, larger wall thicknesses lead to progressively smaller critical velocities. Note that for high critical velocities, the corresponding diameter (not shown) may be very large and, thereby, may be not meaningful for design.

The influence of wall thickness t/d on pile diameter is explored in Fig. 6, where the regions of admissible diameters are plotted against soil stiffness for different values of wall thickness and design acceleration. Such regions decrease in size and translate with decreasing thickness, due to the decreasing moment capacity of the pile section. Figure 6a indicates that in active seismic zones, piles need thick walls to withstand earthquake action. In d:V_s space, for constant acceleration and pile ultimate strain, critical diameters are located upon a parabola whose equation straightforward to be derived from Eq. (15).
The aforementioned observations refer to piles designed to carry axial loads with a specific safety factor (*SF*). It is evident that a wider range of admissible diameters may be obtained by increasing *SF*, thereby leading to a pile foundation which over-satisfies axial bearing capacity requirements.

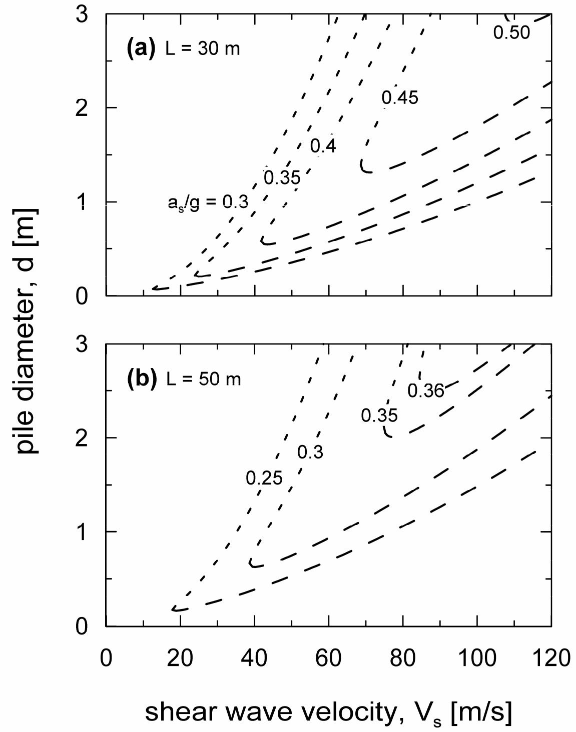

Figure 4. Admissible pile diameters against soil shear wave velocity (E_s/S_u = 500, f_y = 275 MPa, E_p = 210 GPa, ν_s = 0.5, S_a = 2.5, FS = 3, t/d = 0.015, α = 0.7, δ = 1.2)

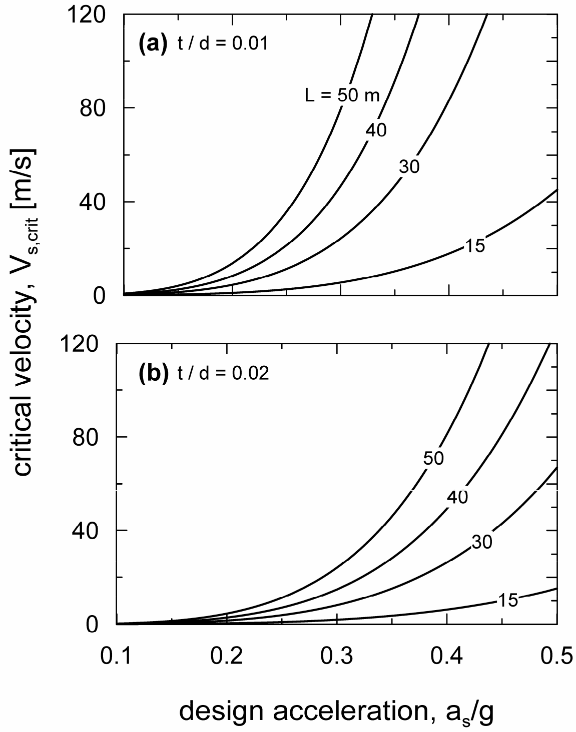

Figure 5. Critical shear wave velocity against design acceleration (E_s/S_u = 500, f_y = 275 MPa, E_p = 210 GPa, ν_s = 0.5, S_a = 2.5, FS = 3, α = 0.7, δ = 1.2)

To investigate this behavior, Figs 7a and 7b depict the admissible range by varying *SF* for two different values of a_s/g. The regions are bounded on the left by the curve of pure kinematic action, as it corresponds to an infinite value of *SF* (zero mass carried by the pile). The regions translate to the right by increasing design acceleration; for a_s/g = 0.45 (Fig. 7b) piles in soft soil must carry very small axial loads to guarantee elastic behavior for seismic action.

The role of ratio E_s/S_u is similar to that exerted by *SF*. Specifically, an increase in the ratio between stiffness and strength of the soil material leads, for a given value of V_s, to a reduction in strength and, hence, a lower mass carried by the pile. To highlight this issue, Fig. 7c and 7d depict the range of admissible diameters by varying E_s/S_u. The analogous behaviour with the upper graphs is evident in the results.

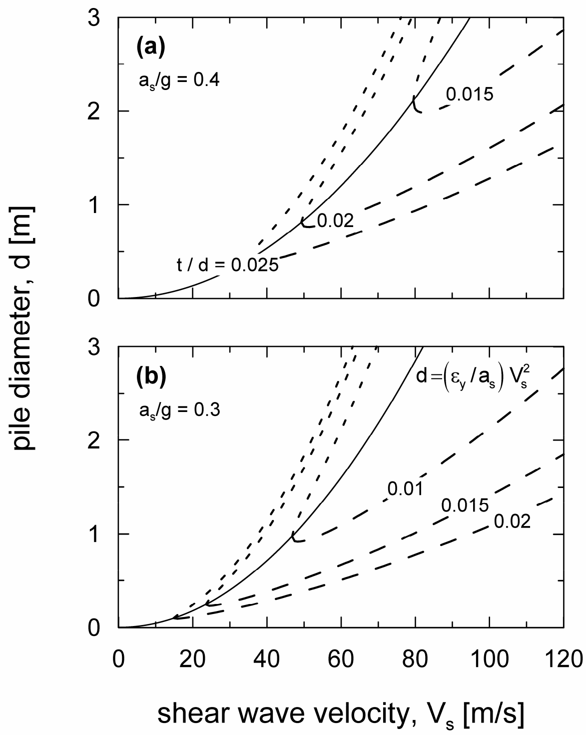

Figure 6. Admissible pile diameters against soil shear wave velocity (E_s/S_u = 500, f_y = 275 MPa, E_p = 210 GPa, ν_s = 0.5, S_a = 2.5, FS = 3, L = 40 m, α = 0.7, δ = 1.2)

STEEL VS CONCRETE PILES

The practical significance of the present work becomes more evident for the more common case of concrete piles. A concrete section of equal moment capacity to that of a steel section requires higher pile flexural stiffness, which, thereby, attracts a larger kinematic moment. Due to space limitations, the complete analytical study for concrete piles is not provided here. Instead, a limited set of results is reported in Fig. 8, comparing critical diameters against soil stiffness for steel and concrete piles and two different values of design acceleration. In the graph, the percentage of reinforcement in the concrete section A_s/A_c is taken as equal to the wall thickness ratio *t*/*d* in the steel pile, as they typically assume values within the same range.

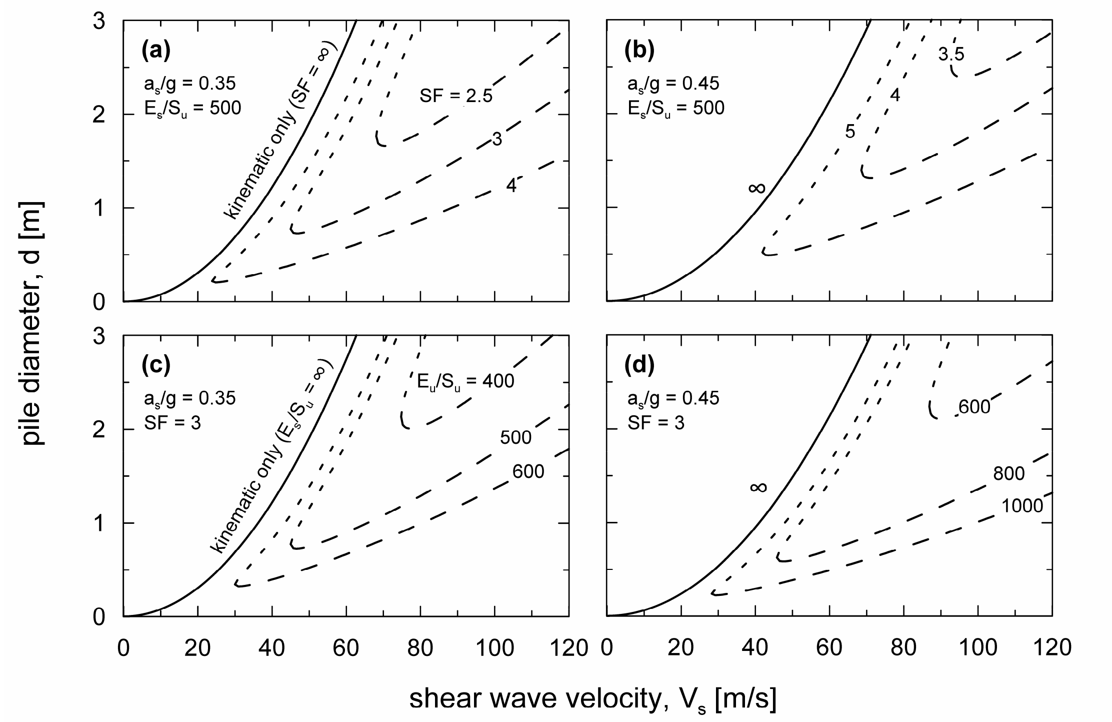

Figure 7. Admissible pile diameters against soil shear wave velocity (f_y = 275 MPa, E_p = 210 GPa, ν_s = 0.5, S_a = 2.5, L = 40 m, α = 0.7, δ = 1.2)

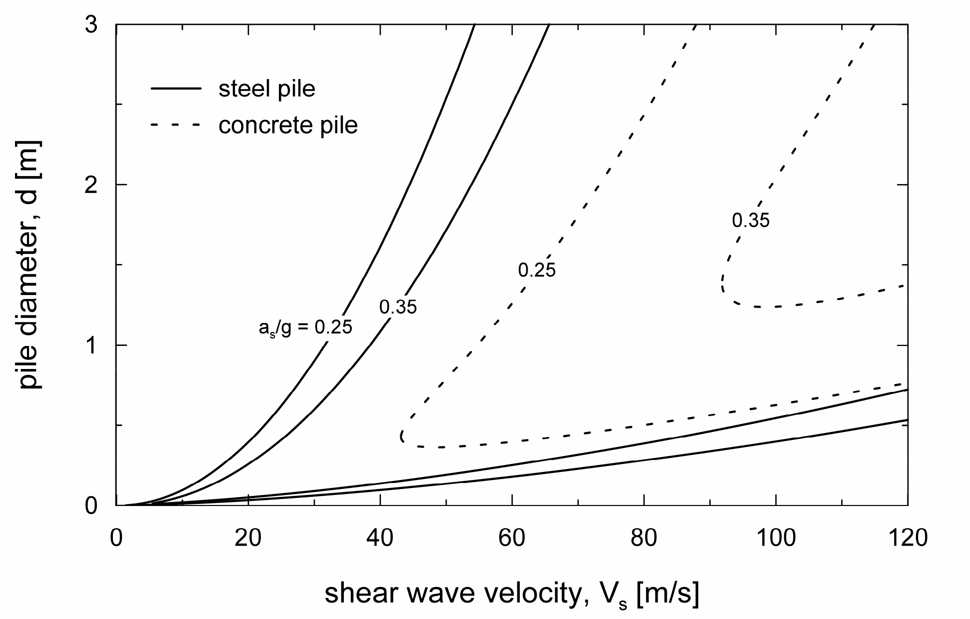

Figure 8. Admissible pile diameters against soil shear wave velocity (E_s/S_u = 500, f_{cd} = 14.2MPa, f_y = 275 MPa, $f_{y,reinf}$ = 373 MPa, $E_{p,s}$ = 210 GPa, $E_{p,c}$ = 30 GPa, ν_s = 0.5, S_a = 2.5, FS = 3, t/d = A_s/A_c = 0.015, c = 5 cm, L = 15 m, α = 0.7, δ = 1.2).

Contrary to steel piles, the design of concrete piles is strongly affected by the level of earthquake acceleration: even for moderate values of a_s/g, piles in soft soil do not withstand the overall seismic demand. To overcome the problem, a design strategy could be to decrease the inertial demand may be to increase the safety factor for axial bearing capacity failure, leading to piles that operate under low values of axial load.

CONCLUSIONS

Kinematic and inertial forces arising in a piled foundation during earthquakes are of different nature and, thereby, are affected by pile size in a different manner. It was shown that kinematic bending at the pile head is associated with a maximum tolerable diameter, whereas inertial forces with a minimum tolerable diameter. The contemporary presence of both types of forces implies the existence of a limited range of diameters, which may resist the overall seismic demand. Exploring this range has been the subject of this work. A number of conclusions may be derived based on these results:

1) For very soft soils (V_s <100m/s), kinematic interaction dominates pile response. In this case, the maximum diameter may be very small and, thereby, it may strongly affect design. Increasing the piles number and/or length has no effect against seismic action.
2) For stiffer soils, inertial interaction might be dominant. This implies a minimum diameter, which for moderate-to-high design acceleration, may be quite large
3) The range of admissible diameters narrows with increasing acceleration, soil strength and pile length, whereas it widens with increasing soil stiffness, pile safety factor and wall thickness.
4) There is always a critical soil shear wave velocity below which no pile diameter is admissible. This suggests that below the particular soil wave propagation velocity, a fixed-head pile cannot stay elastic regardless of pile diameter. In the limit case V_s = 0 (a pile embedded in water), no diameter is admissible, which makes no sense, as the pile cannot be stressed kinematically along its length. This should not be viewed as paradoxical, as the strong interplay between V_s and a_s, (a_s may strongly decrease at low V_s's) was not considered in this study.
5) Concrete piles generally have a more limited range of admissible diameters as compared to steel piles.

Despite the simplified approach adopted in this study, issues of practical importance related to pile design in seismic areas have been highlighted. Exploring the topic using more sophisticated models and less restrictive assumptions may represent a goal for future research.

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REFERENCES

de Sanctis, L., Maiorano R.M.S., Aversa, S. (2010), "A method for assessing bending moments at the pile head", Earthquake Engng Struct. Dyn., vol. 39; pp. 375 – 397.

Di Laora, R. (2009). Seismic soil-structure interaction for pile supported systems. Ph.D. Thesis, University of Napoli "Federico II".

Di Laora, R. and Mandolini, A. (2011). "Some Remarks about Eurocode and Italian code about piled foundations in seismic area". ERTC-12 Workshop on Evaluation of EC8, Athens.

Di Laora, R., Mandolini, A. and Mylonakis, G. (2011). "Kinematic bending moments at pile head in layered soil". 5th International Conference on Earthquake Geotechnical Engineering, Santiago, Chile.

Dobry, R. & O'Rourke, M. J. (1983). Discussion on `Seismic response of end-bearing piles' by Flores-Berrones, R. & Whitman, R. V. J. Geotech. Engng Div., ASCE p. 109.

Kavvadas, M. & Gazetas, G. (1993), "Kinematic seismic response and bending of free-head piles in layered soil", Geotechnique, Vol. 43, No. 2, pp. 207 – 222.

Margason, E. (1975). Pile Bending During Earthquakes. Lecture ASCE/UC-Berkeley seminar on design construction & performance of deep foundations.

Mylonakis, G. (2001). "Simplified model for seismic pile bending at soil layer interfaces". Soils and Foundations, Vol. 41, No. 4, pp. 47 – 58.

Nikolaou, A. S., Mylonakis, G., Gazetas, G. and Tazoh, T. (2001). "Kinematic pile bending during earthquakes analysis and field measurements". Géotechnique, Vol. 51, No. 5, pp. 425 – 440.

Mizuno, H. (1987). Pile damage during earthquakes in Japan. In Dynamic response of pile foundations (ed. T. Nogami). ASCE Special Publication, pp. 53 – 78.

Roesset, J. M. (1980). The use of simple models in soil-structure interaction. ASCE specialty conference, Knoxville, TN, Civil Engineering and Nuclear Power, vol. 2.

Saitoh, M. (2005). "Fixed-head pile bending by kinematic interaction and criteria for its minimization at optimal pile radius". Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, No. 10, pp. 1243 – 1251.