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SEISMIC RESPONSE OF PILE FOUNDATIONS - SOME IMPORTANT FACTORS

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ABSTRACT:

In this paper an analysis for the seismic response of single piles is presented in which it is assumed that the whole dynamic part is concentrated in the analysis of the free-field response. This response is then applied statically to the pile. The effect of pile inertia is therefore ignored. Based on the proposed method, the responses of piles subjected to several typical earthquakes are then calculated and compared with the results of other dynamic methods. It is shown that good agreement exists between the proposed method and a more involved numerical analysis. It is further shown that even when the soil medium is strongly layered the proposed method captures the salient seismic behavior of piles. This simple methodology is used to provide an insight into the seismic behavior of single piles. For typical earthquakes, the effects of pile diameter, length and stiffness, and soil stiffness, on the pile response are studied, with particular emphasis on the pile bending moments.

KEYWORDS:

Pile foundation; dynamics of piles; seismic analysis; seismic response; aseismic design

INTRODUCTION:

In recent years many methods of analysis have been proposed for the aseismic design of pile foundations. They range from complicated mathematical methods to simple subgrade reaction or Winkler-type analyses. For the low frequency range which is of the most importance in earthquakes, the simpler methods, although to a large extent approximate, have produced results similar to more involved rigorous mathematical models.

Gazetas et al (1993) recently proposed a method based on the Winkler hypothesis and found good agreement between their method and that of Kaynia (1982), which is a rigorous method within the assumption of a linear layered half space soil mass.

In the static analysis of piles, a number of researchers have taken advantage of Mindlin's fundamental closed form solution which gives the response of a point in an elastic half space due to the application of a unit concentrated force elsewhere in the medium. Aboundary element method, which models the pile as a beam and employs the finite difference method for the analysis, is well established and is described by Poulos and Davis (1980).

The dynamic equivalent of Mindlin's closed form solution does not yet appear to exist. A number of researchers have obtained numerical solutions for the response of a half space , or a layered half space, to an internal unit harmonic or pulse excitation, for example Apsel (1980), and Mamoon and Banerjee (1992). Some researchers, including Penzien (1970) have however used the static Mindlin theory in the seismic analysis of single piles and pile groups. They have justified such an approach by the fact that, when the characteristic wave length of the soil medium is long compared with the zone of major influence resulting from interaction, the elastic stress and displacement fields within the soil can be adequately defined by a static theory. Such an assumption is tantamount to ignoring the effect of inertia, which has been shown by Penzien to be small indeed compared with the elastic forces.

In this paper, a simple methodology for seismic response analysis of single piles is outlined and compared with other methods. It is shown that the proposed method, although static in nature, captures the salient characteristics of the seismic pile response. The proposed method is then used to obtain the response of an end bearing single pile of different diameters embedded in a clay layer of various thicknesses and stiffness, and subjected to 10 different earthquakes. The maximum pile moments in the 1440 cases studied are presented in a series of graphs which may be of practical use to foundation engineers.

THE PROPOSED APPROACH:

In the proposed approach, the problem of pile response to earthquake excitation is decomposed into two parts. In the first part, the free field response is calculated, and in the second part this response is applied to the pile statically, ignoring the effect of inertia. The pile response is then calculated using the Mindlin fundamental solution. In the analysis it is assumed that the earthquake consists of horizontal shear waves only, which has been found to be a conservative assumption in most cases. The free-field response could be calculated by the well-established methods such as the one used in development of the well-known SHAKE computer program (Schnabel et al, 1972). In the current analysis the program ERLS (Poulos, 1991) has been used. ERLS can take into account non-linear behavior of the soil layers, and models the horizontal soil layers on top of a bedrock as a system of mass- spring-damper. The earthquake is applied at the level of the rock below the soil layers, and the response of the multi-degree freedom soil layer system is obtained.

In a static analysis it can be shown that, by dividing the interface of pile-soil system into rectangular elements, the following compatibility condition holds:

$$\left[D + \frac{II}{K_R n^4}\right]\{\rho\} = \frac{[II]}{K_R n^4}\{\rho_e\} \tag{1}$$

in which D is the matrix of the finite-difference coefficients, $[II]$ is the inverted soil-displacement-factor matrix which is the inverse of a matrix whose elements are evaluated by integration, over a rectangular soil element, of the Mindlin equation for the horizontal displacement of a point within a semi-infinite mass caused by horizontal point-load within the mass. $K_R=E_pI_p/E_sL^4$ is pile-flexibility factor with its components being, modulus of elasticity of pile, moment of inertia of pile, modulus of elasticity of soil, and length of the pile respectively. n is the number of elements, $\{\rho\}$ is pile displacement vector and $\{\rho_e\}$ is the vector of external soil displacements which is obtained from the free field analysis. More details are given by Poulos and Davis (1980).

In this analysis, the earthquake motion is applied at the rock level and at each time step the free field response is calculated and is used to satisfy the compatibility conditions of Eqs. (1) at the center of each element. This, along with four pile boundary conditions, enables the solution of the problem to be obtained including calculation of the pile displacements and internal forces.

The mass acting on the pile cap mass can be directly included into analysis by setting the shear at the pile top equal to the cap inertia at each time step.

COMPARISON WITH OTHER METHODS:

Almost all of the proposed methods for seismic analysis of pile foundations operate in the frequency domain, giving the pile response for an incident harmonic excitation. An earthquake therefore should first be decomposed into its harmonic constituents by the application of a Fourier transform, then for each harmonic component the response is calculated, and finally the superposition of the responses so calculated is transformed to the time domain by an inverse Fourier transformation. In such an analysis, with only horizontal SH waves involved, the free field response U_{ff} of the soil is:

$$U_{ff} = f(z,\omega)\exp(i\omega t) \tag{2}$$

in which $f(z,\omega)$ is a soil profile which is harmonically vibrating with an angular velocity ω. In order to compare the proposed method with the published results of harmonic based analysis, for a number of values of ω, the $f(z,\omega)$ is calculated based on the wave propagation principles and then applied as soil free-field profiles to the pile. The pile response calculated in this way is compared with the published results of harmonic excitation with the same ω. This means that the maximum amplitude of a real harmonic excitation is being compared in the two analyses. Ke Fan (1992) and Gazetas et al (1993) used Kaynia's rigorous solution and studied the performance of an end bearing pile embedded in a two layer soil profile, in which the contrast of the shear wave velocity of the upper layer to the lower layer was 1/1, 1/2, 1/3, and 1/6; the thickness of the upper layer was 1/4 that of the lower layer.

Figure 1 compares the ratio $Re[U_p]/U_{ff}$ of the pile and free field displacement at the top of the pile from the two analyses, for various ratios of shear wave velocity contrast.

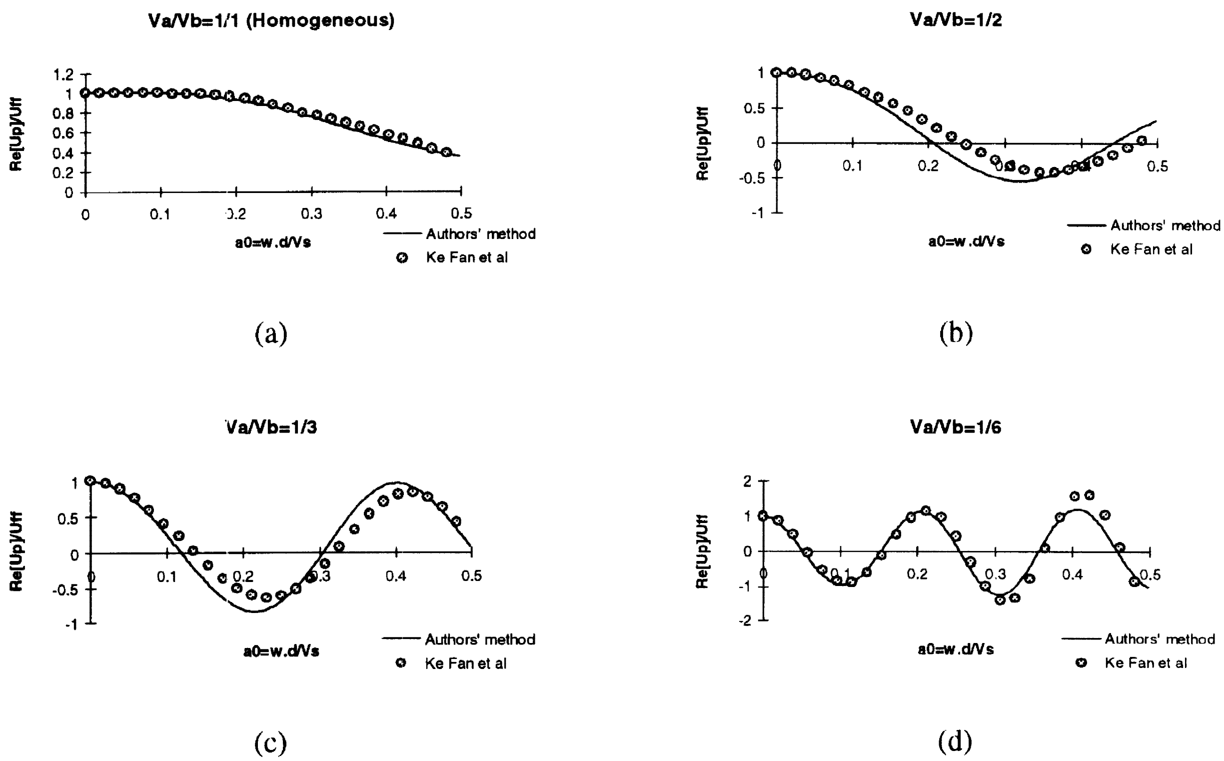


Fig. 1. A comparison between Authors' and Ke Fan et al methods, L/d=20, Ep/Es=1000, v=0.4

The agreement between the two methods is very good for the homogeneous case and quite good for the non-homogeneous cases. It should be noted that the Mindlin equation is strictly for the homogeneous case and is applied to non-homogeneous soil in an approximate way.

In order to validate the method further , several real earthquakes were considered. A Winkler method proposed by Kavvadas and Gazetas (1993) was employed to obtain the response of a single pile of different diameters and lengths subjected to the earthquake records. Figures 2 and 3 show the comparison of the results between the proposed method and the Winkler method for the homogeneous case when subjected to the Newcastle 1994 record. The agreement is good both for a free head and a fixed head pile. Figure 4 compares the maximum moment obtained by the two methods for a pile length of 12 meters and different diameters. For all the diameters, the agreement is very good.

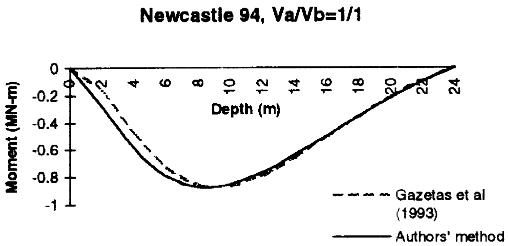


Fig. 2. The Moment profile at the time step where maximum moment happened. L/d=20, Ep/Es=1000

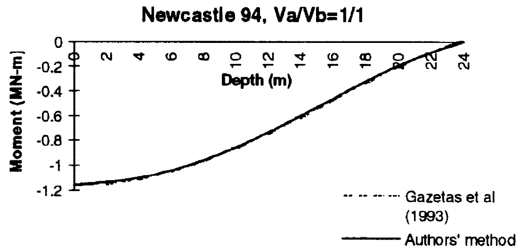


Fig. 3: The Moment profile at the time step where maximum moment happened. L/d=20, Ep/Es=1000

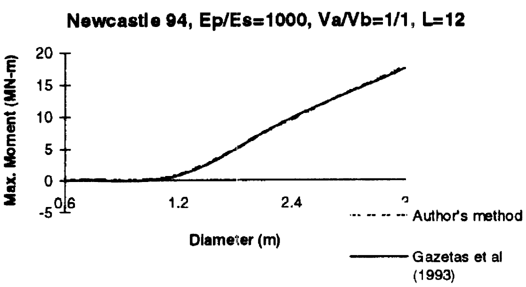


Fig. 4. Comparison of authors' method and Gazetas method for different values of pile diameter, for the homogeneous case, L=12 meters.

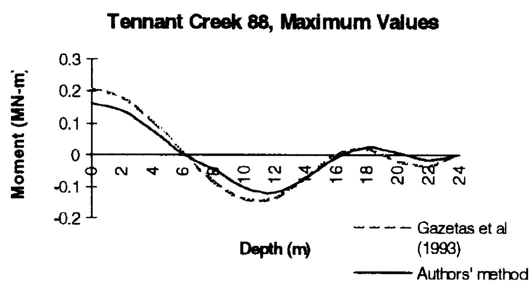


Fig. 5. The moment profile at the time step where maximum moment occurred. L/d=20, Ep/Es=1000

Figure 5 compares the results of the two methods for a non-homogeneous soil with a strong stiffness contrast between the upper and lower layers. The proposed method tends to underestimate the response, but is in fair agreement with the other method.

In general it can be seen that the proposed method, despite all the approximations involved, gives results comparable with the method proposed by Kavvadas and Gazetas (1993). This, among other things, means that the pile response to earthquakes is very much determined by the free-field soil movements, and other effects such as pile inertia and the dispersion of waves by the pile are of secondary importance. This occurs mainly because the predominant frequencies present in earthquakes are usually less than 10 Hz.

EFFECTS OF DIFFERENT FACTORS:

The aim of this study is to investigate certain trends in the seismic response of pile foundations, and the effects on this response of important parameters such as pile and soil elasticity, pile length, pile diameter, and cap mass.

The following earthquakes were considered and applied to a clay layer of constant Young's modulus, embedding an end-bearing concrete pile whose length is equal to that of the layer: Meckering 1968, Newcastle 1989, Newcastle 1994, Cadoux, Tennant Creek, Pasadena, San Fernando, Taft, and Whittier. The first five earthquakes are Australian and the remaining four are North American. In order to make the results obtained from different earthquakes comparable, they were all scaled to produce a peak bedrock velocity of 75 mm/s.

In all the analyses the pile modulus is equal to 30000 MPa. It is further assumed that the bedrock is strong enough for axial failure to occur via yielding of the concrete pile itself. A mass associated with the maximum load which the pile can carry with an allowable concrete stress of 7.5 MPa was considered as the cap-mass. Eight different lengths of 5,7,10,12,15,20,25, and 30 meters, four different soil modulus values of 25,50,100, and 200 MPa., and five diameters of 0.3, 0.6, 0.9, 1.2, and 1.5 meters were considered. The total number of cases analysed was therefore 1440.

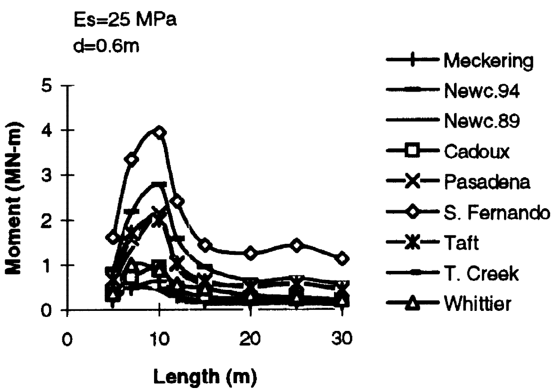


Fig. 6. Maximum Bending Moment versus length for E_s=25 MPa

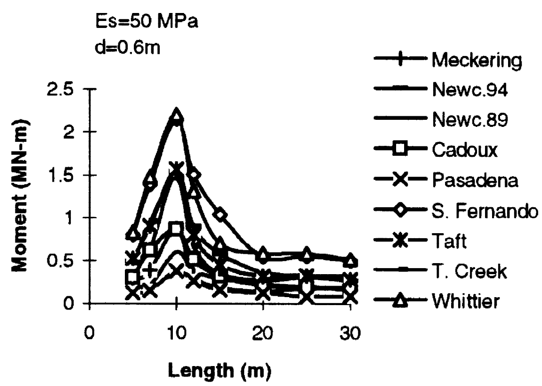


Fig. 7. Maximum Bending Moment versus length for E_s=50 Mpa

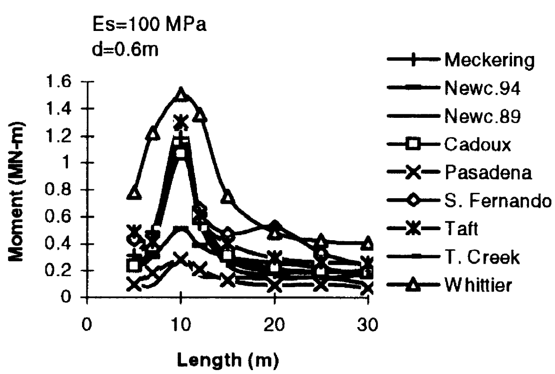


Fig. 8. Maximum Bending Moment versus length for E_s=100 MPa

Figures 6-8 represent the maximum moment developed in a pile with a diameter of 0.6 meters with different lengths, and soil modulus values, subjected to the earthquakes. Similar results were obtained for other diameters, but are not presented here due to the space limitations.

Although the earthquakes are all scaled to produce 75 mm/s maximum bedrock velocity, significant differences are seen in the maximum moment that they have produced for each case. The earthquake which causes the largest response differs from case to case. One of the reasons appears to be the frequency content; in each case, the earthquake with a closer dominant frequency to the natural frequency of the pile-soil system, has a more profound effect.

An increase in the soil modulus has a tendency to reduce the response; this may be explained by the fact that the stiffer the soil, the less is its (and consequently the pile's) deflection. This is not, however, always true as other parameters are also involved.

Figures 9-12 and Fig. 7 show the maximum moment developed in the pile for a constant soil modulus of 50 MPa and different diameters.

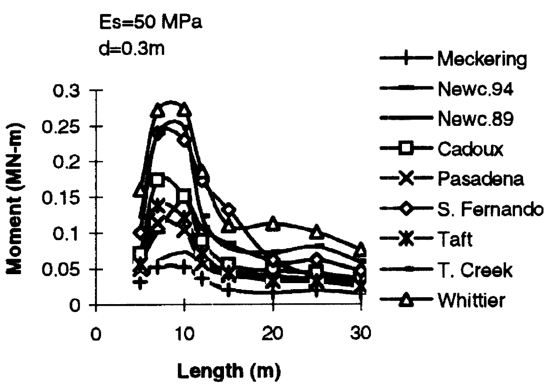


Fig. 9. Maximum Bending Moment versus length for d=0.3m

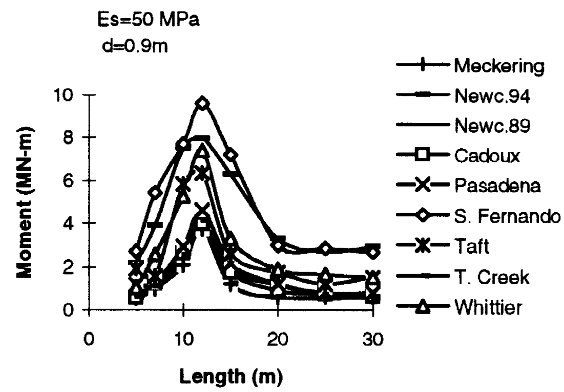


Fig. 10. Maximum Bending Moment versus length for d=0.9m

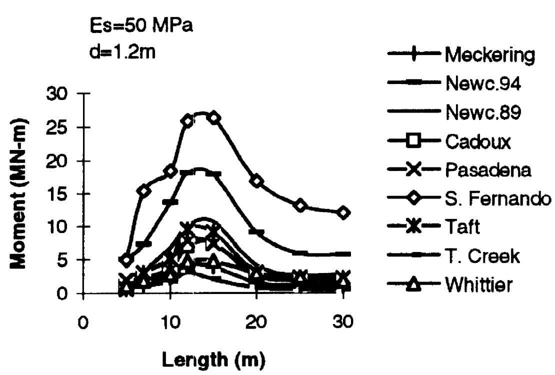


Fig. 11: Maximum Bending Moment versus length for d=1.2m

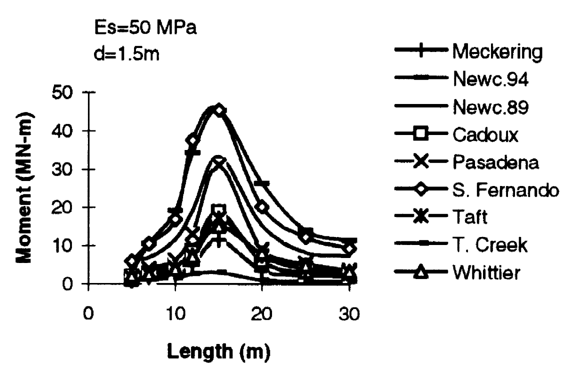


Fig. 12. Maximum Bending Moment versus length for d=1.5m

In Figs. 9-15, it can be seen that a specific pile length has produced a peak moment, and all the earthquakes have had their peak effect at about the same length. This length is apparently the one which produces identical fundamental frequencies for both the pile and the soil causing resonance. Figure 13 shows the fundamental frequency of the layer and that of the pile for different diameters and cap-masses. The fundamental frequencies of the pile shown in this figure are only approximate and based on simplified formulas presented by Gazetas. It is seen that an increase in diameter along with an increase in cap-mass has reduced the fundamental frequency of the pile, and the intersection point of pile and soil fundamental frequencies has been transferred from a point associated to a length of 10 meters for a 0.3m diameter pile to about 20 meters for a 1.5m diameter. This approximate analysis clearly show the same trend which observed in Figures 9-12.

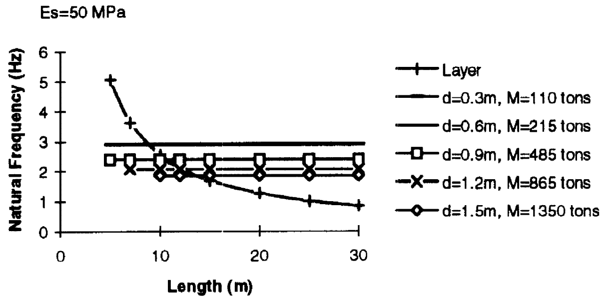


Fig. 13. Natural Frequency of soil layers and piles.

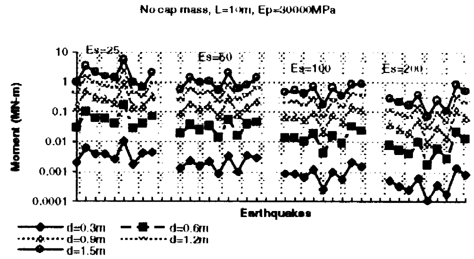


Fig. 14.Effect of Pile Diameter, Soil Modulus and Earthquakes on Maximum Bending Moment. Zero cap mass

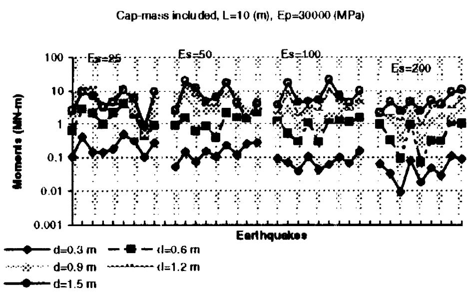
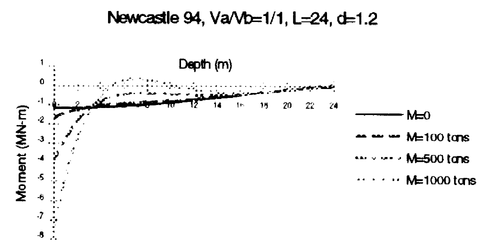


Fig. 15. Effect of Pile Diameter, Soil Modulus and Earthquakes on Maximum Bending Moment. Maximum cap mass



.Fig. 16. Effect of Cap Mass on Maximum Moment

Figures 9-12 also show unequivocally the increase in maximum moments as a result of diameter and cap mass increases. The effect of diameter alone may be better seen in Fig. 14,in which the maximum moments developed in a 10m long end-bearing pile, for 5 various diameters and 4 different values of soil modulus, subjected to the 9 earthquakes, are shown. The cap mass is assumed to be zero. Each line in this figure has 9 points, which are the response of following earthquakes from left to right; Meckering, Newcastle 94, Newcastle 89, Cadoux, Pasadena, San Fernando, Taft, Tennant Creek, and Whittier. The increase in the moment due to an increase in diameter is clearly seen. Some of other trends discussed earlier, such as the tendency of a decrease in the response with an increase in soil modulus, can also be observed in this figure. Similar graphs can be obtained for other pile lengths.

Figure 15 is similar to Fig. 14, except for the case of cap mass which here is the maximum value that the pile can carry for a safety factor of 2 against axial yield of the pile.

A comparison of these two figures reveals the significant effect of cap mass, which has increased the moments over ten-fold. It is however interesting to note that, for the mass to have a significant effect on the response it must be rather large. Figure 16 shows the response of a 24m long end bearing and fixed head pile to the Newcastle 94 earthquake. It can be seen that when the cap mass is less than 100 tons the response of the 1.2m diameter pile is hardly affected, and is mainly governed by the response of the free-field.

CONCLUSIONS:

In this paper a relatively simple methodology for the seismic analysis of single piles has been presented and compared with other methods. It has been shown that this method gives reasonably good results, despite the approximations involved.One of the main advantages of the proposed method is that it is in the time domain which makes a non-linear analysis feasible and straightforward within the simplifying assumptions.

A series of computations on the response of a single pile embedded in a clay layer to 9 earthquakes were performed, and the effects of pile diameter, length, cap mass, and soil modulus and thickness were investigated.The so-called kinematic pile seismic response, which is the response of a pile with no cap mass, is to a large extent dependent on the free-field response for most practical cases, and this free-field influence appears to be much more important than other factors such as pile inertia. The cap mass will have a dominant effect if it goes beyond a certain limit. This limit is well within the practical range of design load values for piles with different diameters.

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