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EFFECT OF PLASTICITY OF SOIL ON SEISMIC RESPONSE OF PILE FOUNDATION: PARAMETRIC STUDY

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ABSTRACT

Much of the reported research on the dynamic analysis of pile foundations assumes linear behavior of soil that may not be valid at strong excitations. In this paper material nonlinearity of soil caused by plasticity and work hardening is considered for the dynamic analysis of pile foundations. An advanced plasticity based soil model, HiSS, is incorporated in a finite element technique. Analysis is carried out in the time domain. The algorithm is verified with available analytical results and then linear and nonlinear responses of a single end bearing pile are compared. Effects of different frequencies of excitation (harmonic) and ratio of rigidity of soil and pile have been investigated.

INTRODUCTION

Much of the reported research such as Kaynia & Kausel (1982), Gazetas (1984), for dynamic analysis of pile foundations assumes linear behavior of soil media. Complexity involved with nonlinear analysis and limited computer resources forced researchers to keep the analysis linear. But with the advent of new technology in computers it has become possible to consider nonlinear analysis.

There are a few researchers incorporating time domain analysis with pile foundation such as by Nogami & Konagai (1986). In much of the previously performed research soil is typically modeled using discrete systems of mass, spring and dashpots. Using such systems, it is difficult to properly represent damping and inertia effects of continuous semi-infinite soil media. Further full coupling in the axial and lateral direction is not considered. Only few researches (in this field) incorporated the finite element technique in the time domain because the computational effort involved is enormous compared to those required with the frequency domain. Inclusion of material nonlinearity such as that caused by plasticity of soil demands that an analysis should be performed in real time using finite elements to adequately represent possible inhomogeneous soil media. Cai et al. (2000) included the plasticity of soil using a finite element technique in the time domain. But a thorough verification of the developed algorithm using linear elastic analysis had not been performed.

In this paper plasticity and work hardening of the soil media are considered in the dynamic analysis of the pile foundation. An advanced plasticity based model for soil media, HiSS, as used by Cai et al., is incorporated into the finite element formulation. The soil-pile system is idealized as an assemblage of solid elements with the analysis performed in the time domain.

Verification of the rigorous approach used has been performed assuming elastic behavior of soil for harmonic excitation. Once this verification was completed, the linear response (assuming that soil behaves linearly) and nonlinear response (considering plasticity of soil) of a single end bearing pile was compared. Influence of frequency of excitation and ratio of rigidity of soil and pile is significant on soil-pile interaction. Effects of both of these parameters, on comparison of linear and nonlinear response of pile head, have been investigated. Using the proposed algorithm it is possible to investigate the effects of some other parameters such as slenderness ratio of pile and different soil profiles.

MODELING

A three-dimensional finite element model used for a single end bearing pile, completely embedded in the soil is shown in Fig. 1. This soil-pile system is idealized as an assemblage of eight-node hexahedral elements. Pile elements are assumed to be linear but can be nonlinear.

All the bottom nodes are taken as fixed as the foundation block is assumed to be resting on bedrock. Also for simplification, nodes on the side walls of the foundation are assumed fixed and may not be exact for satisfying the radiation condition.

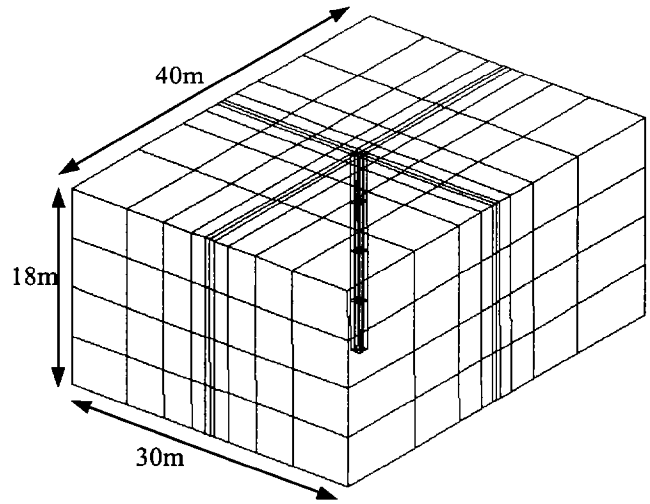

Fig. 1. Finite element model for soil-pile system.

Appropriate radiation boundary conditions on the side walls should be applied and is currently being developed. Seismic excitation is assumed to be acting on the fixed nodes. For simplification it is assumed to consist of vertically propagating shear waves. But the analysis being in the time domain, a real three-dimensional excitation can be considered.

FORMULATION AND COMPUTERIZATION

Governing Equation

Digitizing loading history at each time increment, the governing equation of motion at time $t+\Delta t$, Bathe (1982), is:

$$M\,{}^{t+\Delta t}\ddot{U} + C\,{}^{t+\Delta t}\dot{U} + K\,{}^{t+\Delta t}U = {}^{t+\Delta t}R \tag{1}$$

where ${}^{t+\Delta t}R$ is external load at this time step. Employing the Trapezoidal rule (linear acceleration method) of integration, first acceleration and velocity at time $t+\Delta t$ are expressed in terms of displacement ${}^{t+\Delta t}U$ and known quantities at time t, then equation 1 is solved for displacement ${}^{t+\Delta t}U$ as follows:

$$\left(\frac{4M}{\Delta t^2}+\frac{2C}{\Delta t}+K\right){}^{t+\Delta t}U = {}^{t+\Delta t}R + C\left(\frac{2}{\Delta t}{}^{t}U + {}^{t}\dot{U}\right) + M\left(\frac{4}{\Delta t^2}{}^{t}U + \frac{4}{\Delta t}{}^{t}\dot{U} + {}^{t}\ddot{U}\right) \tag{2}$$

In the following sections simplification of equation 2 has been performed according to the constitutive relation for the soil.

Elastic Model

Here it is assumed that the constitutive stiffness of the soil is that for the elastic case. Matrix M and K are developed as typical finite element analysis requires while C is taken as Rayleigh damping. It can be found using the global mass matrix M and stiffness matrix K with a specified damping ratio and frequency range. In this analysis, for determination of C a damping ratio equal to 5% is taken. The lowest and highest frequency for this purpose is taken as the first and fifth natural frequency (rad/sec) of the soil stratum, respectively. For the elastic case an iterative method of solution is not required but the analysis is carried out incrementally, Bathe (1982), using the expression:

$${}^{(t+\Delta t)}U = {}^{t}U + \Delta U \tag{3}$$

Using equation 3, 2 is simplified for the displacement increment leading to:

$$\left(\frac{4M}{\Delta t^2}+\frac{2C}{\Delta t}+K\right)\Delta U = {}^{t+\Delta t}R - K\,{}^{t}U + M\left(\frac{4}{\Delta t}{}^{t}\dot{U} + {}^{t}\ddot{U}\right) + C\,{}^{t}\dot{U} \tag{4}$$

At each step of time, the displacement is found using equation 4 and then 3.

HiSS soil model

When including soil plasticity the matrix K doesn't remain constant, changing at each step of time, requiring an iterative scheme for the solution, Bathe (1982), as:

$${}^{(t+\Delta t)}U^{(i)} = {}^{t+\Delta t}U^{(i-1)} + \Delta U^{(i)} \tag{5}$$

It is assumed that the dissipation of seismic energy through inelastic deformation overshadows the dissipation of energy through viscous damping. Therefore the velocity related damping term is neglected. With this and using 5, equation 2 simplifies to:

$$\left(\frac{4M}{\Delta t^2}+{}^{t}K\right)\Delta U^{(i)} = {}^{t+\Delta t}R - M\left(\frac{4}{\Delta t^2}\left({}^{t+\Delta t}U^{(i-1)} - {}^{t}U\right) - \frac{4}{\Delta t}{}^{t}\dot{U} - {}^{t}\ddot{U}\right) - {}^{t}K\,{}^{t+\Delta t}U^{(i-1)} \tag{6}$$

For determination of the matrix K, the δ_0 version of HiSS, as proposed by Wathugala and Desai (1993), is considered. This model is based on associative plasticity and assumes nonvirgin loading (loading or unloading), elastic.

A simplified formulation related to virgin loading in HiSS is described. When in this model material parameter $\beta=0$, the dimensionless yield surface F can be defined as:

$$F = \left(\frac{J_{2D}}{p_a^2}\right) + \alpha_{ps}\left(\frac{J_1}{p_a}\right)^n - \gamma\left(\frac{J_1}{p_a}\right)^2 \tag{7}$$

where J_1 is the first invariant of the stress tensor σ_{ij}; J_{2D} is the second invariant of the deviatoric stress tensor; p_a is the atmospheric pressure; γ and n are material parameters; α_{ps} is the hardening function defined in terms of plastic strain trajectory ξ_v, as:

$$\alpha_{ps} = h_1 / \xi_v^{h_2} \quad (8)$$

where h_1 and h_2 are material parameters. The increment of trajectory of volumetric plastic strain $d\xi_v$ is found using increment in volumetric plastic strain $d\varepsilon_v^p$. Detailed formulation for this and for the determination of constitutive stiffness tensor for the virgin loading can be found in Wathugala and Desai (1993).

Seismic Loading

The seismic loading is applied at the bedrock and thus the external force in the equation of motion is found by

$$^{t+\Delta t}R = -MP_F \, ^{t+\Delta t}\ddot{V}_b \quad (9)$$

where P_F is the pseudostatic response influence coefficient vector and $\ddot{V}_b$ is the bedrock acceleration at time $t+\Delta t$, due to vertically propagating shear waves.

Computerization

An algorithm was developed to perform the analysis. To check for convergence in nonlinear analysis, three different criteria (displacement, out of balance load and internal energy) are simultaneously used as suggested by Bathe (1982). Special procedures (Wathugala, 1990) have been used to ensure the robustness of the HiSS iterative solution.

VERIFICATION OF THE ALGORITHM

Much computational effort is involved in the finite element dynamic analysis of pile foundation using the time domain, therefore, it is necessary to thoroughly check the developed algorithm. This can best be done for the elastic case considering harmonic excitation. This has been done in the following two sections with bedrock motion consisting of a sinusoidal wave of amplitude 1 m/s^2 and frequency of excitation equal to 2 Hz. Following material properties have been used for all the results in this paper, if not specifically revised:

For Soil $E_s = 4147kPa; \upsilon = 0.42; \gamma = 0.047; n = 2.4;$
$\rho = 1610Kg/m^3; h_1 = 0.0034; h_2 = 0.78;$

For Pile $E_p = 25GPa; \upsilon = 0.15; \rho = 2400Kg/m^3;$

Free-Field Motion

Here it is observed that the bedrock motion is modified due to the presence of the soil. Fig. 2 shows the motion at the ground surface (in the absence of pile) due to bedrock motion. It can be seen that after a few cycles, the amplitude of free field motion becomes constant due to damping of the soil. Also it is noted that this constant amplitude is approximately four times the amplitude of the bedrock motion.

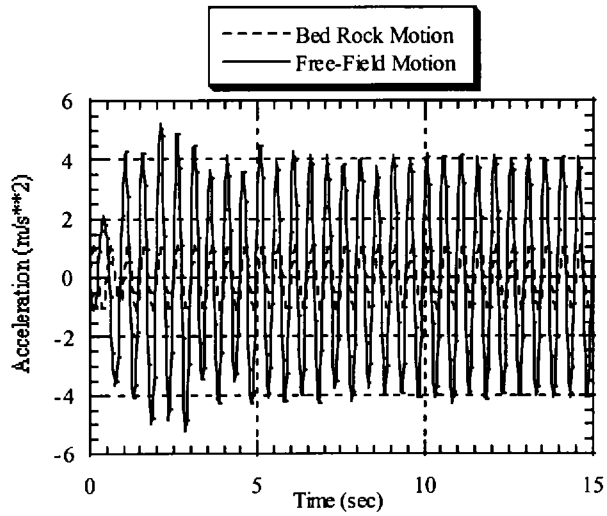

Fig. 2. Response of the ground due to harmonic excitation at the bed rock (elastic case).

Thus due to the soil stratum, the bedrock motion was amplified. It can be seen using the frequency domain analysis for a linear case, Wolf (1985), Gazetas (1984) etc. that, for this frequency and given properties of soil, the motion will be amplified approximately by the same amount. This verification can be done for different frequencies of excitation.

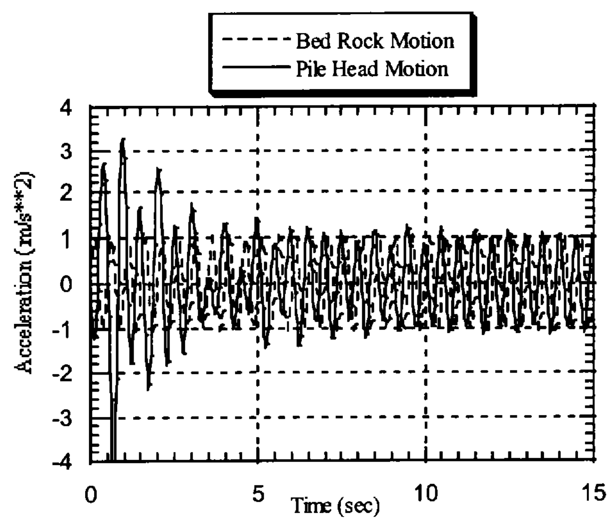

Fig. 3. Response of the pile head due to harmonic excitation at the bed rock (elastic case).

Pile Head Motion

In addition to free-field motion above verification has also been performed in the presence of a single pile. It can be seen from Fig. 3 that pile head motion (steady state) for this frequency is about the same as the bedrock motion. An interesting point is that presence of a single pile decreased the amplitude of the motion and it is quite different then the free-field motion. This clearly shows the effect of soil-pile interaction, which beside other factors also depends on relative rigidity between pile and soil. In this case the ratio of rigidity of pile and soil is quite high hence influence of soil-pile interaction is significant.

Further if we compare this outcome with Gazetas (1984) we find that it is almost identical. This process can be repeated for different frequencies of excitation.

These examples have been used to validate the current algorithm developed for the elastic case. For the nonlinear case (HiSS model) analysis was performed by changing the constitutive relation and iterating to convergence.

RESULTS AND DISCUSSION

Using the algorithm developed linear (elastic) and nonlinear (HiSS) motion at the pile head is compared for harmonic excitations. Results are grouped in two sections to investigate the effect of frequency and rigidity ratio. These are presented in the first two sections. A condition of numerical instability in the algorithm has been discussed in the third section. To derive these results input motion at the bedrock consists of a sinusoidal wave of amplitude 1 m/s^2 and a specified frequency.

Effects of Variation in Frequency

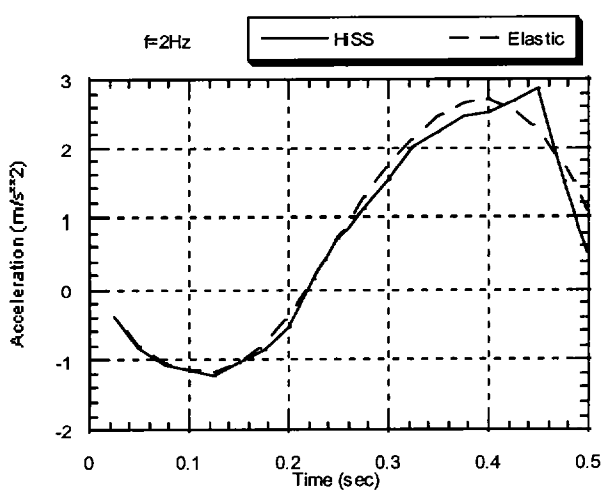

Fig. 4a. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (f=2Hz).

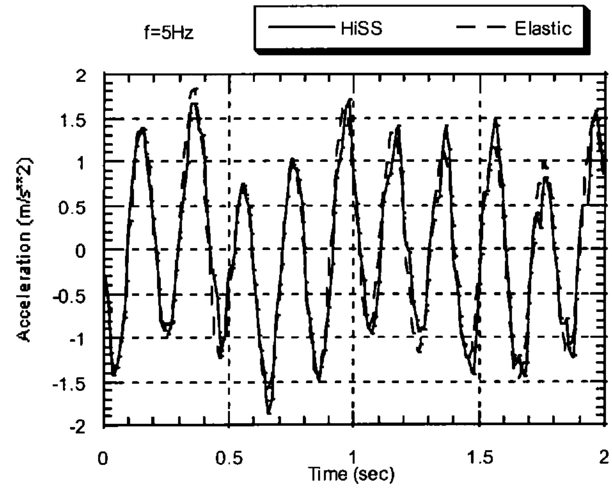

Fig. 4b. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (f=5Hz).

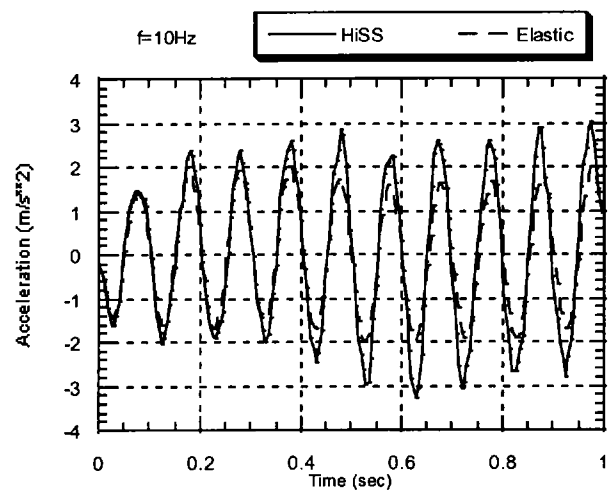

Fig. 4c. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (f=10Hz).

Here frequency of excitation is changed while other factors are kept constant. Fig. 4a, 4b & 4c shows the comparison of linear and nonlinear response for frequencies equal to 2Hz, 5Hz & 10Hz respectively. It can be seen that there is little increase in motion due to plasticity and phase of the motion is same as for the elastic case. As the frequency increases the difference between linear and nonlinear response increases. This may suggest that effects of plasticity are more significant at higher frequencies of excitation.

Effects of Variation in Ratio E_p/E_s

Here the ratio of rigidity between soil and pile (E_p/E_s) is changed while other factors are kept constant. Results are derived for a frequency of 5Hz and three values of E_p/E_s as shown in Fig. 5a to 5c.

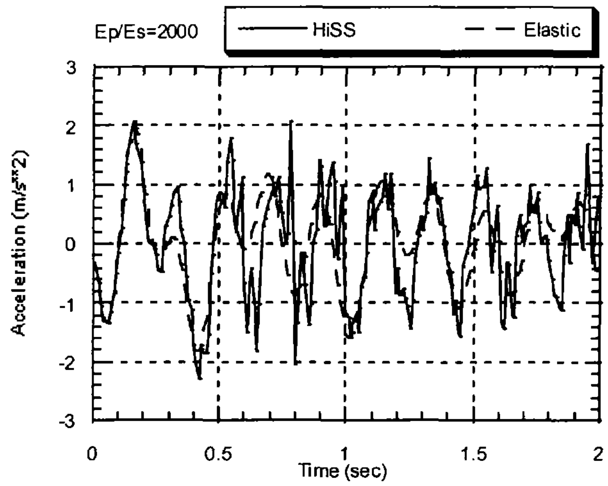

Fig. 5a. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (Ep/Es=2000).

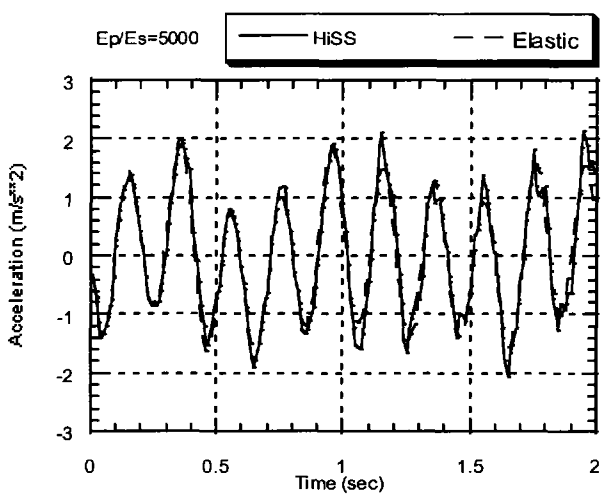

Fig. 5b. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (Ep/Es=5000).

It can be seen that as the ratio E_p/E_s increases the difference between linear and nonlinear response increases. Suggesting that the effect of plasticity is high when there is a large difference between the rigidity of soil and pile. Also there is a small increase in the amplitude at the higher values of E_p/E_s.

Numerical Instability

In Fig. 4a comparison of linear and nonlinear motion had been done for one cycle of loading. This comparison should be for a longer time history. Presently the algorithm is showing some numerical instability when using virgin loading in HiSS for lower frequencies. These problems are under consideration and once eliminated, the algorithm will be more robust.

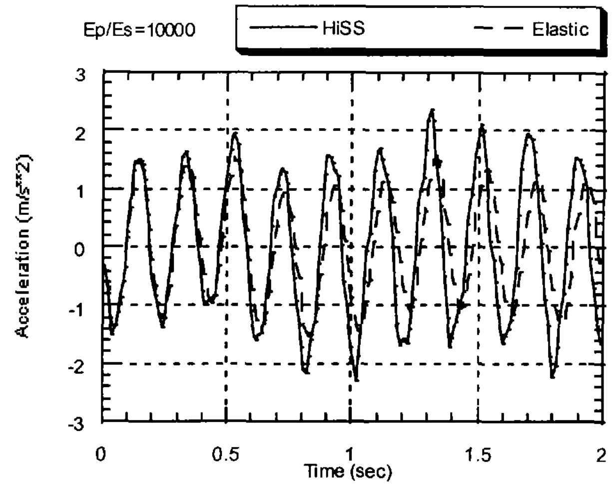

Fig. 5c. Comparison of linear and nonlinear response due to harmonic excitation at the bed rock (Ep/Es=10000).

Wathugala and Pal (1996) have presented some special procedures to improve the robustness but additional improvements may be required.

CONCLUSIONS

A methodology for inclusion of soil plasticity and work hardening in the dynamic analysis of pile foundations has been presented. First verification of the proposed algorithm, by computing the free-field response of a soil stratum and pile head response for the linear case, has been performed. Then linear and nonlinear response of a single end bearing pile is compared.

Effects of two important parameters, frequency and rigidity ratio (E_p/E_s), on this comparison, have been discussed. In general it was found that differences between linear and nonlinear response increases as these two factors increase. More results from future work will be used to verify this general conclusion. With the proposed algorithm it is possible to investigate the effect of other factors such as slenderness ratio of pile and soil profile.

Being based on finite elements the algorithm is quite versatile. Since analyses are in the time domain the algorithm can deal with multiple supports, non-uniform excitation. It is easily extendable to pile groups and inclusive of structures as well.

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