EFFECT OF NONLINEARITY ON THE DYNAMIC BEHAVIOR OF PILE GROUPS

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ABSTRACT

The aim of this paper is to demonstrate the use of a hybrid method in studying the effect of nonlinearity on the seismic- behavior of the soil-pile system. The dynamic behavior of a 2*2 pile group, with a rigid pile cap is analyzed under strong ground motion. The nonlinear near-field is modeled using finite elements and the elastic far-field is modeled using a boundary finite element method – CIFECM (Consistent Infinitesimal Finite Element Cell Method). For the near-field, material-nonlinearity due to plasticity and strain hardening has been considered. The Hierarchical Single Surface (HiSS) model available in literature has been used for dealing with nonlinearity. The Hybrid Time-Frequency Domain (HTFD) method of iterative solution is used in conjunction with CIFECM to model the non-linearity and the unbounded nature of the medium respectively. The effect of material nonlinearity on seismic response of the soil-pile system is presented.

KEYWORDS: Pile Groups, Nonlinear Analysis, Hybrid Domain, CIFECM, HiSS Soil Model

1. INTRODUCTION

The several numerical and analytical methods have been reported in the literature for computing the dynamic stiffness and seismic response of pile groups. Most of them assumed linear visco-elastic response of the surrounding soil (e.g. Kaynia 1982). However, under strong ground excitation pile foundations may undergo large displacements and the behavior of the soil system can be strongly nonlinear. Realistic modeling of the nonlinear dynamic response of the pile-soil system is very important, especially for the dynamic analysis of highway bridges and off-shore structures. Studies on the nonlinear dynamic response of piles have been conducted with the Winkler foundation model (Nogami et al. 1992; Maheshwari and Watanabe 2006). Through such discrete models, it is difficult to represent the nonlinear stiffness, damping and inertia effects of continuous soil media. Rigorous nonlinear studies on pile foundations were reported by Wu and Finn 1997, Bentley and El Naggar 2000, Maheshwari et al. 2005. In these finite element models the nonlinearity has been modeled with increasing detail. However, the frequency dependent radiation boundary conditions are only approximately satisfied. Therefore it is recognized that the soil-pile system should be separated into two different zones; i.e. the near-field (plastic zone), where strong nonlinear soil-pile structure interactions occur, and the far-field where the behavior is primarily elastic and frequency dependent (Wang et al. 1998). Emani and Maheshwari (2008) presented the effect of nonlinearity on the pile head response due to pulse loading. This paper is an extension of that work for dynamic and seismic loading.

In the present study, the three-dimensional soil-pile system is sub-structured into nonlinear near-field and linear far-field. The nonlinear near field is modeled using finite elements and analyzed in the time-domain. The far-field is modeled using CIFECM (Consistent Infinitesimal Finite Element Cell Method). This is assumed linear and analyzed in the frequency domain. Both the systems are coupled through the Hybrid Time-Frequency Domain (HTFD) algorithm, Wolf (1988). The coupled dynamic system is analyzed for both external loading from the pile cap and seismic loading. The effect of nonlinearity of soil on the seismic response due to real earthquake time history is presented.

2. THE NUMERICAL MODELLING OF THE SYSTEM

As shown in Fig. 1, the three-dimensional numerical model of the physical system consists of two substructures, namely, an unbounded far-field and a bounded near-field. The dynamic stiffness of the unbounded far-field $[S^{\infty}]$ is added to the dynamic stiffness of near-field which is evaluated using finite element method (Wolf 1985). The bounded domain is minimally selected to accurately represent the near-field effects, like nonlinearities.

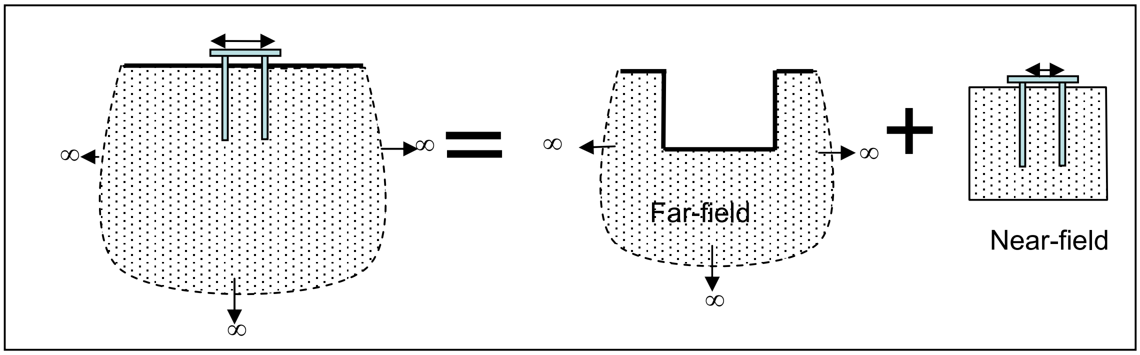

Figure 1 Sub-structuring the pile soil system into far-field and near-field

The physical system (Figure 2) consists of a 2 x 2 pile group, square in plan, with a pile-cap of width *2B* and thickness *t*. The centre to centre spacing between the piles is *s*, length of each pile is *L*, Young's modulus of the pile material is E_p, and Poisson's ratio of the pile material is ν_p. The piles are embedded in soil which is considered homogeneous with Young's modulus E_s, Poisson ratio ν_s, mass density ρ_s and damping ratio ζ. In the present study, the bounds of near field are fixed by trial and error, starting from the edge of pile cap. In all the three directions, the size of this bounded domain, is extended by a length B (Figure 2a).

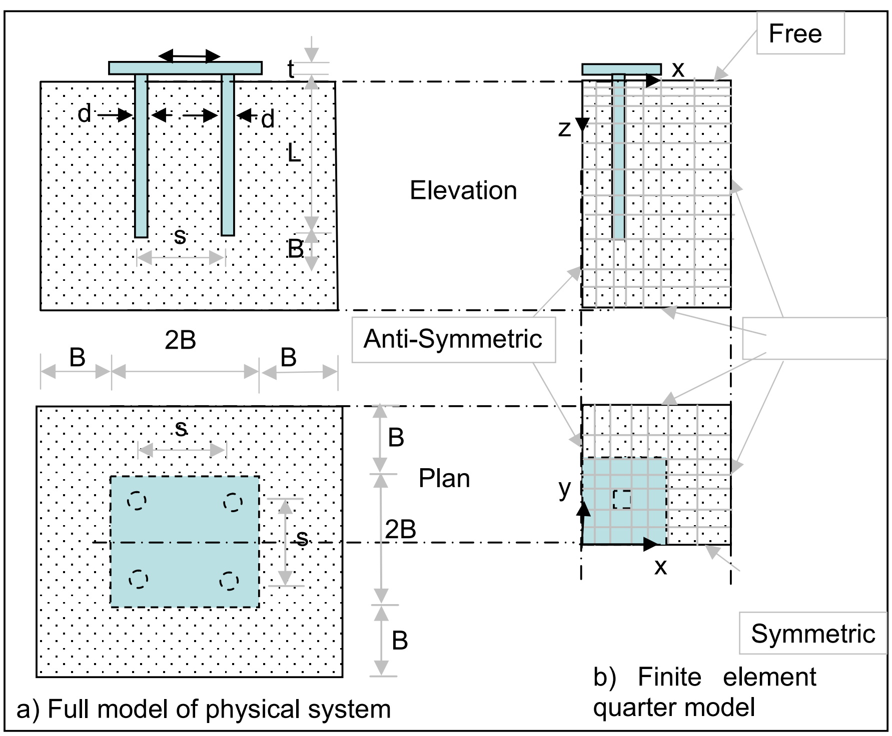

Figure 2. The model of 2 x 2 pile group embedded in a homogeneous near field
a) Full model b) Finite element quarter model used in linear analysis

Under symmetric loading conditions, the symmetry of the system can be used. Taking advantage of this, for linear analyses, only one-fourth of the actual bounded domain was built and corresponding symmetric and anti-symmetric boundary conditions were applied to the respective nodes on the planes of symmetry and anti-symmetry, respectively (Figure 2b). For nonlinear analyses, advantage of only symmetry can be exploited therefore in that case a half model is used in the analyses. At the interface of the bounded and unbounded domains, the radiation condition is enforced by adding the dynamic stiffness of the unbounded far-field. In the bounded domain, both soil and the pile are modeled with 8-noded 3D solid (finite) elements with three degrees of freedom at each node. Distributed inertia is modeled using consistent mass matrices. The circular piles are modeled as equivalent square piles. The geometric and material properties used in the analysis are: (a) For piles: d = 0.6 m, L = 15 m, E_p = 25 GPa, ρ_p = 2500 kg/m^3, ν_p = 0.25; (b) For soil: E_s = 25 MPa, ρ_s = 1800 kg/m^3, ν_s = 0.4.

3. METHODOLOGY AND FORMULATIONS

The unbounded soil is initially modeled using frequency independent springs, dashpots and masses using HTFD method (Bernal and Youssef 1998). In the present work, this is implemented by assembling the mass, stiffness and damping matrices of the unbounded far-field at a reference frequency, to the interface nodes of bounded near field. This reference frequency can be different for stiffness, damping and mass matrices. For rapid convergence the reference system is generated from static stiffness matrix, high-frequency damping matrix and zero mass matrix. Using the assembled system matrices, the equations of motion are solved in the time domain. Because of using frequency independent properties for the unbounded domain, interaction forces obtained will be different from those corresponding to the actual impedances. For accounting for this difference, pseudo-forces are evaluated in the frequency domain wherein the actual impedances are taken into consideration. This analysis makes use of dynamic stiffness of the unbounded domain, which is evaluated using the Consistent Infinitesimal Finite Element Cell Method (CIFECM). The pseudo forces are used in the next iteration in the time domain analysis. Thus, the time and frequency domain analyses alternate in each iteration, until a convergence is obtained for the interaction forces at the interface.

3.1 *Formulation of Hybrid Time Frequency Domain (HTFD) method*

In the time domain analysis, the governing equation of motion is (Bernal and Youssef 1998):

$$\begin{bmatrix}[M_{ss}] & [M_{sb}] \\ [M_{bs}] & [M_{bb}]+[M_{ref}]\end{bmatrix}\begin{Bmatrix}\{\ddot{u}_s(t)\} \\ \{\ddot{u}_b(t)\}\end{Bmatrix}+\begin{bmatrix}[C_{ss}] & [C_{sb}] \\ [C_{bs}] & [C_{bb}]+[C_{ref}]\end{bmatrix}\begin{Bmatrix}\{\dot{u}_s(t)\} \\ \{\dot{u}_b(t)\}\end{Bmatrix}+\begin{Bmatrix}\{P_s(t)\} \\ \{P_b(t)\}+[K_{ref}]\{u_b(t)\}\end{Bmatrix}=\begin{Bmatrix}\{0\} \\ \{L(t)\}\end{Bmatrix}-\begin{Bmatrix}\{0\} \\ \{PSF(t)\}\end{Bmatrix} \quad (1)$$

Here $[M]$ and $[C]$ (with subscripts *ss*, *sb*, *bs* and *bb*) are the mass and damping matrices of the bounded near-field respectively, $[M_{ref}]$, $[C_{ref}]$ and $[K_{ref}]$ are the mass, damping and stiffness matrices, respectively, of the unbounded far-field at the reference frequency. $\{P\}$ (with subscripts *s* and *b*) is the restoring force vector in the near-field nodes, $\{PSF\}$ is pseudo force vector of interface nodes, $\{\ddot{u}\}$, $\{\dot{u}\}$ and $\{u\}$ are the acceleration vector, velocity vector and total displacement vectors respectively, and $\{L\}$ is vector of driving loads (seismic loading). The subscripts *s* and *b* refer to the partitioned matrices. The subscript *ss* is associated with the nodes of the bounded near field, the subscript *bb* is associated with the nodes of the unbounded far-field and *sb*, *bs* are associated with the interface nodes. Because the unbounded system's property matrices are condensed to the interface nodes, the subscript *bb* is also associated to the interface nodes only.

A consistent mass matrix $[M]$ formulation is adopted for the near-field consisting of the soil-pile system. The damping $[C]$ in the near-field is represented by stiffness-proportional-damping. The restoring forces are obtained from the nonlinear stress analysis of the near-field finite elements. For this, a nonlinear soil model HiSS is adopted for stiffness of soil elements and elastic stiffness matrix based on Hooke's law is adopted for pile elements. The mass, stiffness and damping matrices of the unbounded domain are evaluated using the CIFECM.

The pseudo force vector is evaluated by:

$$\{PSF(t)\} = \{R_b(t)\} - [M_{ref}]\{\ddot{u}_b(t)\} - [C_{ref}]\{\dot{u}_b(t)\} - [K_{ref}]\{u_b(t)\} \tag{2}$$

Where $\{R_b(t)\}$ is an interaction force vector. In the HTFD prodedure, Eq. (1) and Eq. (2) are solved iteratively. In the first iteration the pseudo-force vector is taken as null vector and Eq. (1) is solved in the time domain. The response history of the interface nodes is then used to evaluate the pseudo forces from Eq. (2).

3.2 Consistent Infinitesimal Finite Element Cell Method (CIFECM)

In the present work, CIFECM (Wolf and Song 1996) is used to evaluate the dynamic impedances of the ground subsystem. A relationship between the dynamic stiffness matrices of any two similar structure-medium interfaces of the same unbounded medium can be formulated through dimensional similitude. Another relationship follows from the finite element assemblage of the finite element cell that is enclosed by the two interfaces. The dynamic stiffness matrix of the finite element cell can be evaluated from the finite element cell equation obtained from these two relationships.

3.3 Hierarchical Single Surface (HiSS) Model

A nonlinear soil model HiSS (Hierarchical Single Surface) has been used to introduce the effect of plasticity. There is a series of these models, as presented by Wathugala and Desai (1993). In the present work, the δ_0^* version of HiSS is considered. Here, the δ_0 model denotes the basic model for initially isotropic material, hardening isotropically with associative plasticity that involves zero deviation from normality δ_0 of the increment of plastic strain to the yield surface F. Superscript * is used to denote a modified series of models specially developed to capture the behavior of cohesive soils. Some of the important basic characteristics of the model are described below.

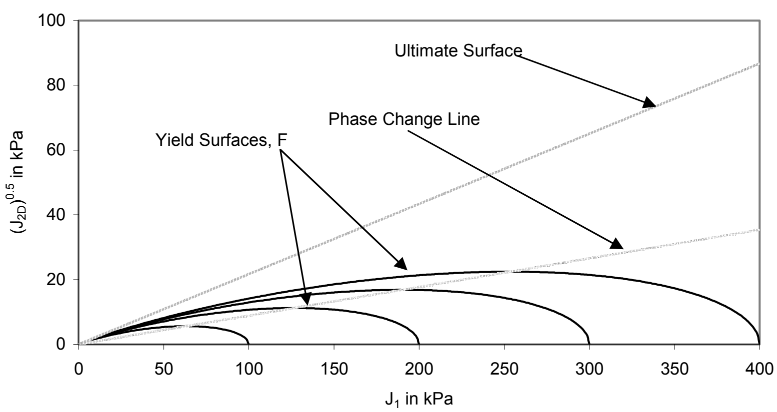

Figure 3 Shape of yield surfaces for HiSS model

Both plasticity and work hardening of the soil are considered in the model that is characterized as a cap model, Chen and Baladi (1985). The model is based on an incremental stress-strain relationship and assumes associative plasticity. For associative plasticity, the yield function (F) becomes the potential function (Q). Further, this model assumes the constitutive relationship for nonvirgin loading (i.e. loading or unloading) to be elastic. In this model, a material parameter β is used to define the shape of the yield surface in the octahedral plane. Assuming β=0, the dimensionless yield surface F can be simplified as:

$$F = \left(\frac{J_{2D}}{p_a^2}\right) + \alpha_{ps}\left(\frac{J_1}{p_a}\right)^{\eta} - \gamma\left(\frac{J_1}{p_a}\right)^2 = 0 \tag{3}$$

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 η are material parameters that influence the shape of F in J_1-$\sqrt{J_{2D}}$ space; parameter η is related to the phase change point that is defined as the point where material changes from contractive to dilative behavior (Figure 4). α_{ps} is the hardening function defined in terms of plastic strain trajectory ξ_v, as:

$$\alpha_{ps} = h_1 / \xi_v^{h_2} \tag{4}$$

where h_1 and h_2 are material parameters. ξ_v denotes trajectory of the volumetric plastic strain. Typical yield surfaces in J_1-$\sqrt{J_{2D}}$ space for this model are shown in Figure 3.

4. VALIDATION OF THE MODEL AND ALGORITHIM

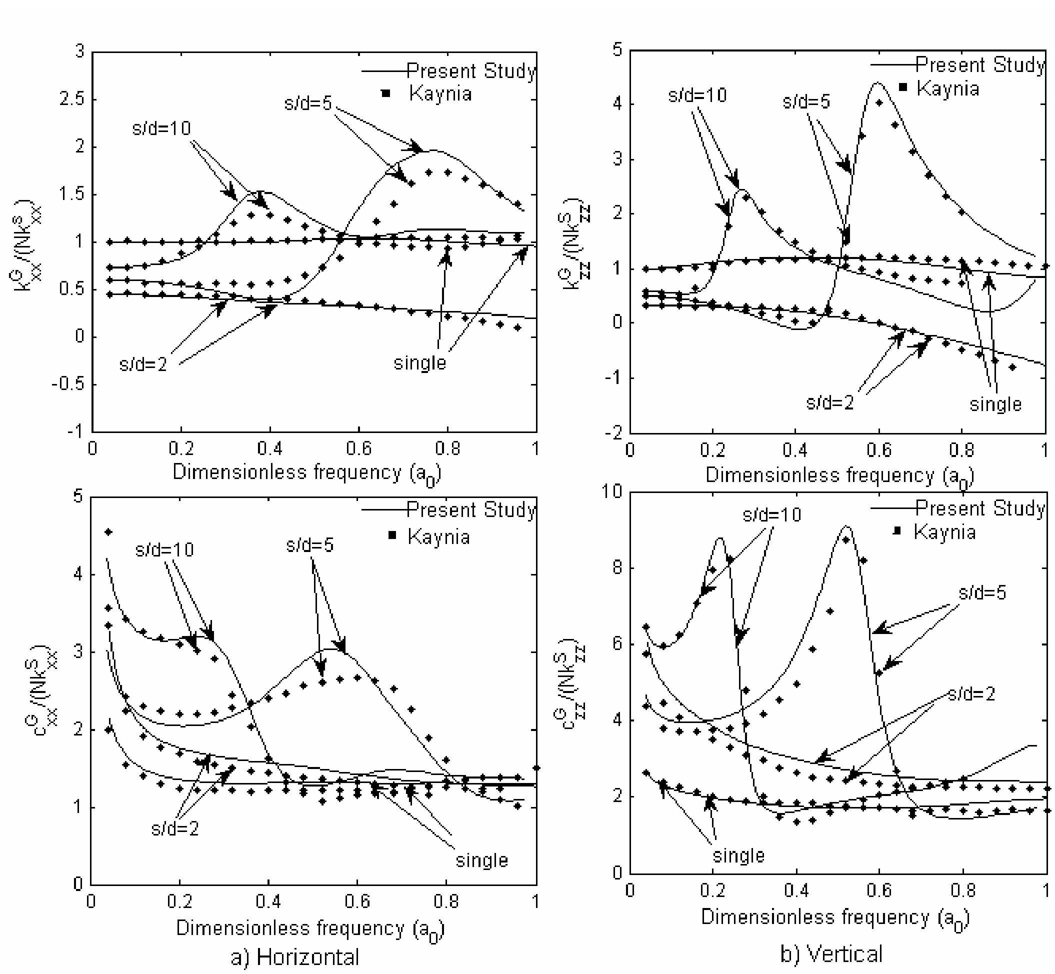

Figure 4 Dynamic Stiffness of a Single Pile and 2*2 Pile Group for different Spacing Ratios

The complex impedances $S(a_0)$ are expressed in the form given by Kaynia (1982) as

$$S(a_0) = k^G(a_0) + i a_0 c^G(a_0) \tag{5}$$

Where $k^G(a_0)$ is the stiffness coefficient including the effect of inertia and $c^G(a_0)$ is the damping coefficient of the pile group and a_0 is the dimensionless frequency defined as $a_0 = \omega d / V_s$, where ω is the angular frequency of excitation and V_s is the shear wave velocity of the soil. In the results presented, the impedances of the pile groups are normalized with respect to N times the static stiffness $k^S=S(a_0=0)$ of a single floating pile in the corresponding direction, where N is the number of piles in the group (i.e. N = 4 in this case). Fig. 4 shows the dynamic impedances using the present approach and that shown by Kaynia (1982) in horizontal and vertical mode of vibration. It can be observed that the results are in very good agreement for single pile as well as for 2*2 pile group for all three spacings considered. The minor difference, particularly in the horizontal direction is due to different methodology of modeling of far-field in two cases. In the present case CIFECM is used while Green's functions are employed by Kaynia (1982) to model the unbounded domain. Nonetheless, agreement among the results is excellent to validate the present methodology for dynamic analysis.

The validation has been also carried out for the seismic response. Fig. 5 shows the seismic response of a 2*2 pile group with (s/d = 5) at different frequencies. It can be the observed that the results obtained from the present study are in good agreement with those shown by Kaynia (1982). Both these results (Figs. 4 and 5) validate the model and algorithm.

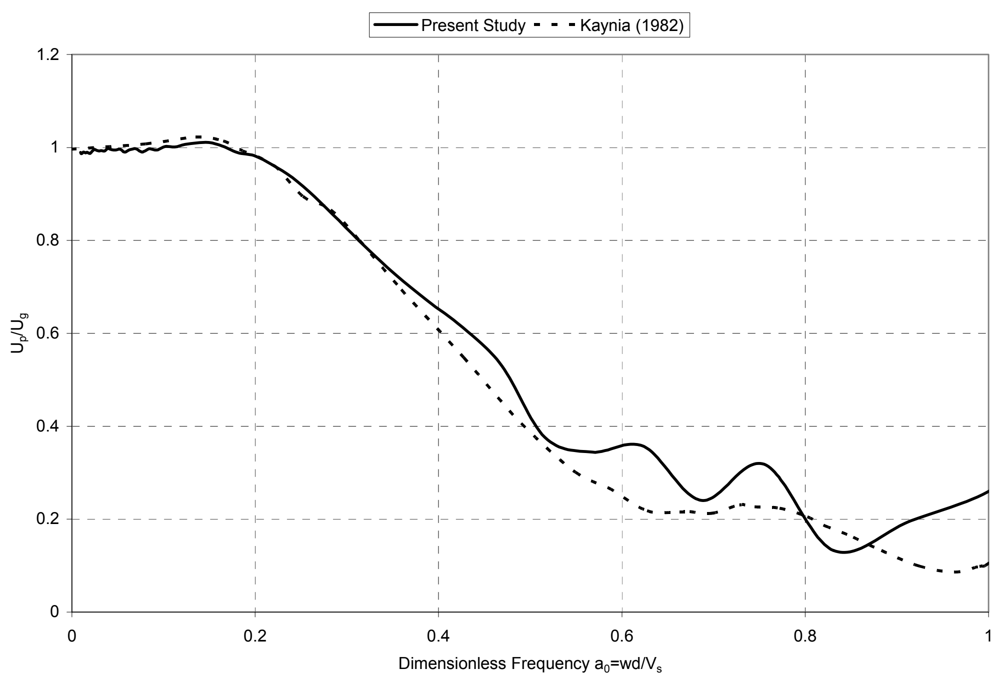

Figure 5 Seismic Response a 2*2 Pile Group with s/d = 5

5. EFFECT OF NONLINEARITY ON THE SEISMIC RESPONSE

The pile cap displacement time history has been computed for a 2*2 pile group for s/d =5 for linear and nonlinear cases. Thus effect of nonlinearity on response is evaluated. Results are shown in Figure 6 and Figure 7 for El Centro Earthquake and Kobe earthquake, respectively. It can be observed that there is significant effect of nonlinearity on the response particularly for the El Centro earthquake where the peak displacement is almost doubled due to nonlinearity. Hence effect of soil plasticity on the seismic response is significant and it shall be considered in the analysis and design.

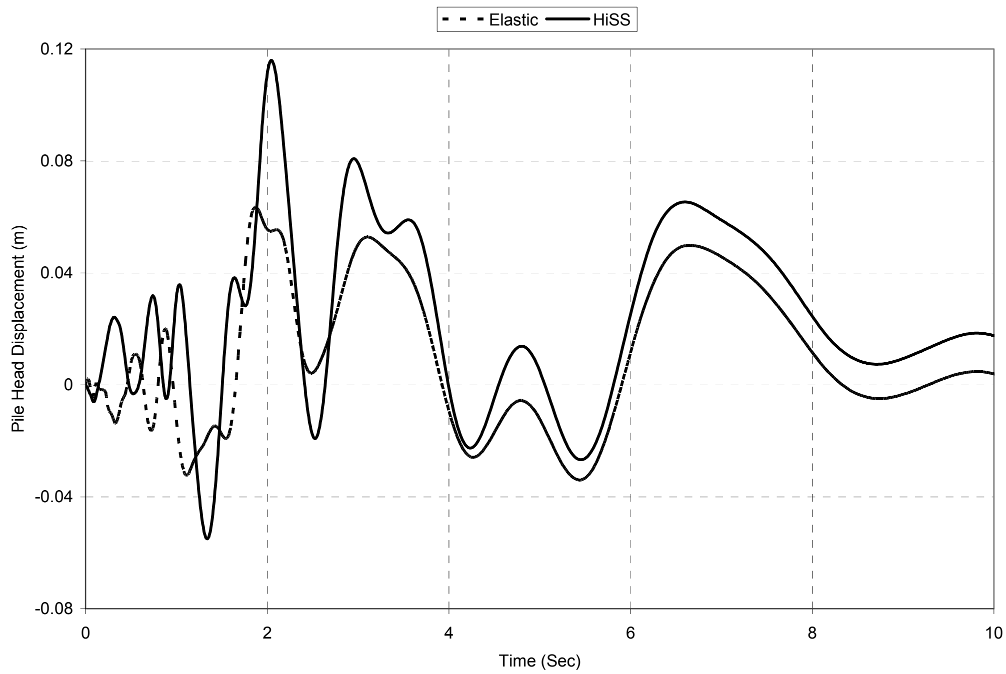

Figure 6 Pile Cap Displacement Time History for a 2*2 Pile Group with S/d = 5 due to El Centro EQ

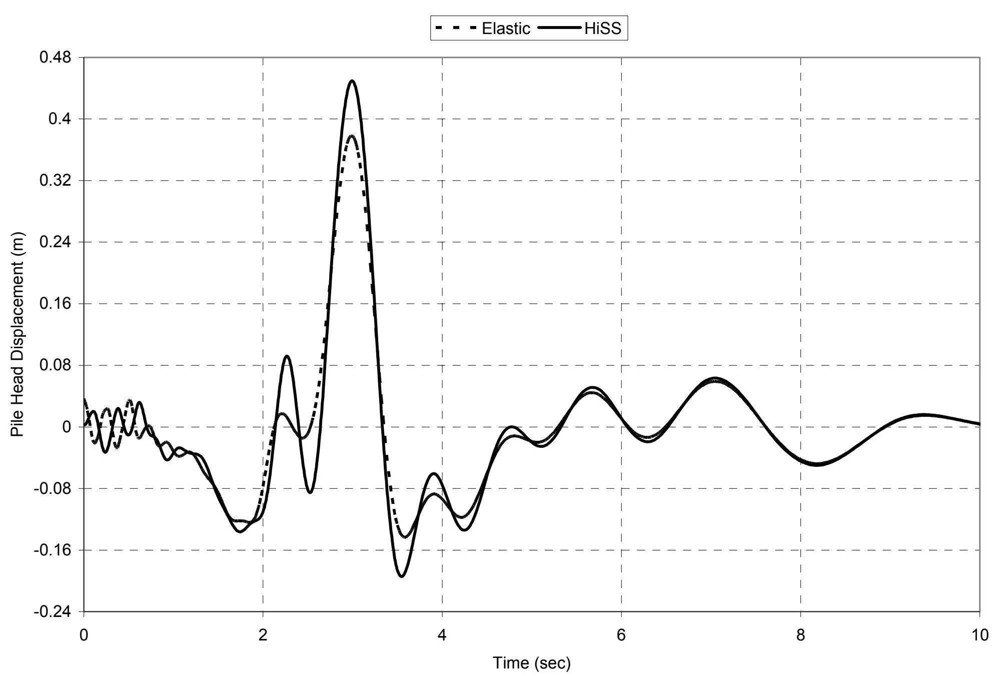

Figure 7 Pile Cap Displacement Time History for a 2*2 Pile Group with S/d = 5 due to Kobe EQ

6. SUMMARY AND CONCLUSIONS

In the present paper, a new methodology is proposed for nonlinear seismic analysis of pile foundations. It involves CIFECM to model the unbounded domain and Hybrid Frequency Time Domain (HFTD) method to deal with the nonlinearity of soil. The model and algorithm has been verified comparing dynamic stiffness and seismic response obtained from present study with those published in literature. It was observed that nonlinearity of soil significantly affects the seismic response of pile groups and therefore it is necessary to consider it for analysis and design of pile foundations. The research is continuing to further investigate the effect of nonlinearity.

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