View metadata, citation and similar papers at core.ac.uk

brought to you by CORE

provided by Missouri University of Science and Technology (Missouri S&T): Scholars' Mine

Missouri University of Science and Technology

Scholars' Mine

International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

1991 - Second International Conference on Recent Advances in Geotechnical Earthquake Engineering & Soil Dynamics

13 Mar 1991, 1:30 pm - 3:30 pm

Seismic Response of Floating Piles to Obliquely Incident Waves

Shahid Ahmad
State University of New York at Buffalo, New York

S. M. Mamoon
Woodward-Clyde Consultants, St. Louis, Missouri

Follow this and additional works at: https://scholarsmine.mst.edu/icrageesd

Part of the Geotechnical Engineering Commons

Recommended Citation

Ahmad, Shahid and Mamoon, S. M., "Seismic Response of Floating Piles to Obliquely Incident Waves" (1991). *International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics*. 38.
https://scholarsmine.mst.edu/icrageesd/02icrageesd/session05/38

This Article - Conference proceedings is brought to you for free and open access by Scholars' Mine. It has been accepted for inclusion in International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics by an authorized administrator of Scholars' Mine. This work is protected by U. S. Copyright Law. Unauthorized use including reproduction for redistribution requires the permission of the copyright holder. For more information, please contact scholarsmine@mst.edu.

Proceedings: Second International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 5.30

Seismic Response of Floating Piles to Obliquely Incident Waves

Shahid Ahmad
Assistant Professor, Department of Civil Engineering, State University of New York at Buffalo, Buffalo, New York

S.M. Mamoon
Senior Staff Engineer, Woodward-Clyde Consultants, St. Louis, Missouri

SYNOPSIS: The response of single piles under vertically and obliquely incident SH, SV, and P waves is obtained using a hybrid boundary element (BEM) formulation. The piles are represented by compressible beam-column elements and the soil as a hysteretic viscoelastic half-space. A recently developed Green function corresponding to the dynamic Mindlin problem is implemented in the numerical formulation. Exact analytical solutions for the differential equations for the piles under distributed harmonic excitations are used. Treating the half-space as a three-dimensional elastic continuum, the interaction problem is formulated by satisfying equilibrium and displacement compatibility along the pile-soil interface. Solutions adopted for the seismic waves are obtained by direct integration of the differential equations in terms of amplitudes. Salient features of the seismic response are identified in several non-dimensional plots. Results of the analyses compare favourably with the limited data available in the literature.

INTRODUCTION

Over the years, significant progress has been made in developing procedures for evaluating the response of single piles and pile groups to both static (Banerjee 1978; Banerjee and Driscoll 1976; Banerjee et al. 1987; Poulos and Davis 1980) and dynamic loads (Banerjee and Sen 1987; Banerjee et al. 1987; Kaynia and Kausel 1982; Sen et al. 1985; Waas and Hartmann 1981; Wolf and Von Arx 1978). Most of the work is essentially restricted to pile foundations subjected to forced periodic excitation (Banerjee and Sen 1987; Banerjee et al. 1987; Sen et al. 1985; Waas and Hartmann 1981). The effect of vertically and obliquely incident seismic waves on surface and embedded shallow foundations has also been investigated in recent years (Dominguez and Roesset 1978); Wong and Luco 1978). Simplified rules have been suggested to estimate both the translational and rotational responses from the free-field motion. Although some work has been done to investigate the response of end-bearing piles to vertically incident waves (Flores-Berrones and Whitman 1982; Gazetas 1984; Kaynia and Kausel 1982; Kobori and Minai 1981), no study has yet been reported encompassing the effects of obliquely incident waves on piles.

Gazetas (1984) has presented results of a numerical study on the seismic response of end-bearing piles due to vertically incident SH waves. The one-dimensional soil amplification theory was used to evaluate the steady-state free-field displacements. The analysis was performed by expressing the response as a superposition of two effects: (1) A kinematic interaction effect; and (2) an inertial interaction effect.

Flores-Berrones and Whitman (1982) have used the Winkler soil model to study the dynamic soil-pile-supported mass interaction as an aid to understanding pile foundation behavior during earthquakes. This study also considers the case of end-bearing piles, driven through a soft stratum and resting on a hard rock bed. The seismic motion was modeled by assuming a simplified one-dimensional representation for the ground shaking at the pile end only. Damping was neglected and the coefficient of subgrade reaction was assumed constant with depth.

Kaynia and Kausel (1982) presented a more general formulation for the seismic response of single piles and pile groups in layered media. They presented absolute values of transfer function from horizontal displacement of ground surface to the pile head and pile cap displacement. However, they considered vertically incident shear waves only.

Wolf and Von Arx (1978) used an axisymmetric finite element formulation to obtain the impedance and transfer functions of a group of piles to vertically propagating waves. They later extended their work to incorporate horizontally traveling waves and studied the kinematic interaction in piles (Wolf and Von Arx 1982).

In the present work, the problem is modeled by a hybrid boundary element (Banerjee and Butterfield 1981) formulation. The piles are presented by compressible beam-column elements, and the soil is represented as a hysteretic viscoelastic half-space. The numerical scheme is based on the discretization into elements of the pile and the soil domain around the pile. Within each element the displacement and traction are assumed constant (or can be interpolated between nodal values in a higher order formulation). The boundary element method offers considerable advantage over other numerical methods, for this type of problem, primarily because of its ability to take into account the three-dimensional effects of soil continuity and boundaries at infinity.

This paper presents the results of a numerical study on the behavior of single piles subjected to vertically and obliquely incident SH, SV, and P waves (Figure 1). Soil-pile interaction effects are taken into account around the shaft interface as well as at the pile base. For soil domain, a new fundamental solution (Banerjee and Mamoon 1990; Mamoon et al. 1988), corresponding to a periodic dynamic point force in the interior of an elastic half-space, is implemented. The solution adopted for the free-field motions due to the seismic waves is obtained by direct integration of the differential equations of motion in terms of amplitudes (Michalopoulos 1976). The methodology presented also takes into account the spatial variation in the distribution of seismic excitation, which leads to a more realistic modeling of the problem. Both the free-field displacement and stress components are used as the seismic input. Finally, results are presented in the form of nondimensional plots, and the pile-soil interaction is studied. Further details of the study can be found in Mamoon and Ahmad (1990).

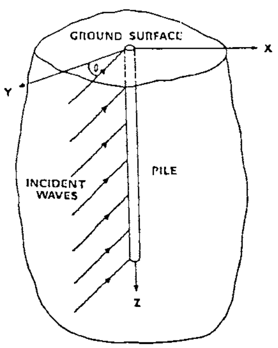


Fig. 1 - Model for Pile and System Coordinates

METHOD OF ANALYSIS

The hybrid boundary element formulation used herein involves the construction of an integral representation for the soil domain and coupling it with the equations of motion for the piles. The procedures embodies a number of assumption which are listed below.

1. The pile shaft-soil interface is discretized into arbitrary cylindrical segments and the base into one circular disk. The actual distribution of surface tractions is replaced by piecewise constant distributions over each segment.

2. The longitudinal and transverse pile-soil interface tractions are uniform along the circumferential direction and act respectively in the z- and x-directions.

3. Incident waves are assumed to be parallel to the y-z plane, producing little or no motion in the y-direction (Figure 1). Therefore, displacements and tractions in the x- and z-directions only are considered ; i.e., each node has only two degrees of freedom. For the purpose of simplicity these displacements and tractions are computed at the pile axis.

It should be noted that the assumptions listed above can all be relaxed if one is prepared to pay for the higher computational effort. The present analysis was developed for a desk-top computer.

Soil Equations

For semi-infinite soil media, the solution for the governing differential equation can be expressed as an integral equation, describing the motion of the soil as

$$u_j(\xi,\omega)=\int_S G_{ij}(x,\xi,\omega)\phi_i(x,d\omega)ds \qquad (1)$$

where u_j is the displacement in the soil, G_{ij} is the Green function for an elastic, homogeneous half-space, and ϕ_i is the traction at the pile-soil interface.

By discretizing the pile-shaft-soil interface into n cylindrical segments, the base of the pile being a circular disc, we can use equation (1) to write the displacements at the pile-soil interface as the following matrix equation:

$$\{u_s^s\}=[G]\ \{\phi_s^s\} \qquad (2)$$

where superscript 's' represents the scattered values and the subscript 's' indicates that the stress and displacement values are obtained from a consideration of the soil domain alone.

For the purpose of this work, a new Green function (G matrix) corresponding to the displacement field due to a dynamic point force in the interior of a semi-infinite solid is implemented. Diagonal terms of this matrix are weakly singular and therefore determined easily by numerical quadrature methods. Details of this are discussed in Mamoon et al (1988), Banerjee and Mamoon (1990).

Scattering problems dealing with semi infinite regions are usually formulated by decomposing the total displacement and stress fields (u^f,ϕ^f) into two parts: (i) a known free-field (u^f,ϕ^f), and (ii) the scattered field (u^s,ϕ^s), i.e.

$$\{u^t\}=\{u^f\}+\{u^s\}$$

$$\{\phi^t\}=\{\phi^f\}+\{\phi^s\} \qquad (3)$$

The free-field is the wave motion that would be present in the absence of scattering surfaces and the scattered part is the wave diverging from the scattering region. The scattered field $\{u^s\}$ satisfies the radiation condition at infinity, which guarantees the absence of reflected radiation.

Writing equation (2) in terms of equations (3),

$$\{u_s^t\}-\{u_s^f\}=[G]\ (\{\phi_s^t\}-\{\phi_s^f\})$$

or

$$\{u_s^t\}=[G]\{\phi_s^t\}\ -\ \{b_s\} \qquad (4)$$

where

$$\{b_s\}=[G]\{\phi_s^f\}\ -\ \{u_s^f\} \qquad (5)$$

Pile Equations

The set of equations represented by equations (4) has to be coupled with an equivalent set for the pile domain obtained by solving the governing differential equations (Sen et al 1985a) of the piles for axial and transverse harmonic excitations. These solutions have been explicitly given in (Sen et al 1985b). Writing separately the axial and lateral pile displacements, we have .

$$\{u_p^t\} = \begin{Bmatrix} \{u_z^t\} \\ \{u_x^t\} \end{Bmatrix} \tag{6}$$

In the above equation, superscript 't' indicates the total field values, and subscript 'p' indicates that the pile-soil interface displacements are obtained from a consideration of the pile domain alone. Since the distribution of tractions along the pile-soil interface is a complex function of nodal values of traction, a direct determination of the displacements is scarcely possible. For arbitrary pile head displacements and rotations, it is necessary to examine the behaviour of individual piles when subjected to axial and lateral dynamic loads. This can be best done by considering the effects of applied pile head displacements and rotations as the algebraic sum of the motion of an unsupported pile (i.e. no soil reaction) under arbitrary pile head boundary conditions and those of a fixed head pile.

If the pile head undergoes a vertical displacements equal to ω_0, a horizontal displacement equal to u_0 and a rotation amounting to θ_0 (Figure 2) superposition of all these solutions leads to the linear sets

$$\{u_z^t\} = [D_z]\ \{\phi_z^t\} + \omega_0\{b_d^z\} + \Theta_0\{b_\Theta^z\} \tag{7}$$

$$\{u_x^t\} = [D_x]\ \{\phi_x^t\} + u_0\{b_d^x\} + \Theta_0\{b_\Theta^x\} \tag{8}$$

where,

$[D_z]$ is the coefficient matrix derived for axial vibration;

$[D_x]$ is the coefficient matrix derived for transverse vibration;

$\{b_d^z\}$ is the displacement vector for unit pile head vertical displacement;

$\{b_d^x\}$ is the corresponding vector for unit pile head transverse displacement;

$\{b_\Theta^z\}$ and $\{b_\Theta^x\}$ are the vertical and lateral displacement vector for unit pile rotation, respectively;

$\{\Theta_z^t\}$ and $\{\Theta_z^t\}$ are the axial and lateral pile soil interface reactions, respectively.

For detail on these coefficients see Sen et al (1985b).

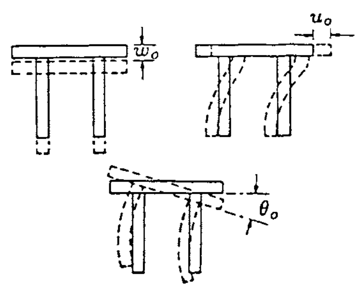


Fig. 2 - Pile Head Boundary Conditions:

(a) Vertical Displacement = ω_0,

(b) Horizontal Displacement = u_0, and

(c) Rotation = Θ_0

It is to be noted that, for a single pile, the value contributed by $\{b_\Theta^z\}$ is zero. These equations can now be rearranged into

$$[B_z]\ \{u_z^t\} = [D_z]\ \{\phi_z^t\} \tag{9}$$

$$[B_x]\ \{u_x^t\} = [D_x]\ \{\phi_x^t\} \tag{10}$$

where $[B_z]$ and $[B_x]$ are matrices incorporating the displacement and rotation vectors for unit pile head boundary conditions in equations (7) and (8). Combining equations (9) and (10) we have

$$[B]\ \{u_p^t\} = [D]\ \{\phi_p^t\} \tag{11}$$

Assembly and Solution

Now premultiplying both sides of equation (4) with [B] matrix we have

$$\begin{aligned}[B]\ \{u_s^t\} &= [B]\ [G]\ \{\phi_s^t\} - [B]\ \{b_s\} \\ &= [E]\ \{\phi_s^t\} - \{d_s\}\end{aligned} \tag{12}$$

where

$$[E] = [B]\ [G] \tag{13}$$

and

$$\{d_s\} = [B]\ \{b_s\} \tag{14}$$

By satisfying the equilibrium and compatibility conditions at the pile-soil interface, the total tractions acting on the piles may be determined from (11) and (12) as

$$\{\phi_p^t\} = -[[D] + [E]]^{-1}\ \{d_s\} \tag{15}$$

The total displacements are then obtained from equation (4).

In equation (5) information is needed about the known values of the free-field displacements $\{u_s^f\}$ and tractions $\{\phi_s^f\}$ which are discussed next.

Incident Waves

Consider the half-space $z \geq 0$ (figure 1) and a train of plane SH-waves, propagating parallel to the y-z plane. The waves are assumed to be inclined with respect to the y-axis in the y-z plane (Figure 1). Therefore, it will produce motions in the x-z plane only.

The solutions adopted for these waves are obtained by direct integration of the differential equations in terms of amplitudes (Michalpoulos 1976). The motion for an elastic medium with one-dimensional geometry in the case of SH-waves has the form (Dominguez and Roesset 1978),

$$u_x = [\, A_{SH} \exp(\frac{i\omega}{C_s} nz) + A'_{SH} \exp(-\frac{i\omega}{C_s} nz)] f(y,t) \quad (16)$$

where ω is the angular frequency, A_{SH} and A'_{SH} are amplitudes of the incident and reflected waves, C_s the shear wave velocity of the soil, ℓ and n are the direction cosines of the direction of propagation.

For the half-space, $A_{SH} = A'_{SH}$, and for unit amplitude of the motion on the surface, equation (16) reduces to (Dominguez and Roesset 1978):

$$u_x = \cos(\frac{\omega}{C_s} nz) f(y,t) \quad (17)$$

where

$$f(y,t) = \exp(-\frac{i\omega \ell y}{C_s}) \exp(i\omega t) = \exp[i\omega(t - \frac{\ell y}{C_s})] \quad (18)$$

The shear stresses are obtained from the displacements by differentiation,

$$\tau_{xy} = G(-\frac{i\omega}{C_s} \ell) \cos(\frac{\omega}{C_s} nz) f(y,t) \quad (19a)$$

$$\tau_{zy} = -G(\frac{\omega n}{C_s}) \sin(\frac{\omega}{C_s} nz) f(y,t) \quad (19b)$$

where G is the shear modulus. Normal stresses due to SH-waves are insignificant and therefore ignored. τ_{xz} is assumed to be constant at each element and directly furnished by equation (19b) at the mid-point of that element. For SH-waves, equations (17) and (19b) directly furnish $\{u_s^f\}$ and $\{\phi_s^f\}$ variations along the depth for equation (5).

COMPARISONS WITH PUBLISHED RESULTS

Comparison with Gazetas (1984)

Gazetas (1984) has presented results for seismic response of end-bearing piles due to vertically propagating SH-waves, in the form of displacement and rotation kinematic interaction factors

$$I_u = \frac{u_p}{u_0} \quad \text{and} \quad I_\phi = \frac{\phi_p r_0}{u_0} \quad (20)$$

where $r_0 = d/2$ is the radius of the pile; u_p and u_0 are the amplitudes of the relative horizontal displacement of the pile top and the free-field ground surface, respectively; and ϕ_p is the pile top rotation.

Because of damping in the system, the displacement components are not in phase with the excitation. The interaction factors are complex functions of frequency; only their absolute values (amplitudes) are studied here.

Gazetas' results show the variation of I_u and I_ϕ with the frequency ratio f/f_1, where f is the excitation frequency and f_1 is the fundamental natural frequency of the unperturbed soil deposit. For a homogeneous soil stratum f_1 (in Hertz) is approximately calculated from (Dobry 1976)

$$f_1 = \frac{C_s}{4H} \quad (21)$$

where C_s is the shear wave velocity and H the stratum thickness.

Figures 3 and 4 compare the variation of I_u and I_ϕ versus f/f_1 for a pile having L/d = 40, soil density ratio equal to 1·60 embedded in a homogeneous soil layer with Poisson ratio equal to 0.40 and material damping β=0·05. The ratio E_p/E_s is set equal to 50, 000. It is to be noted that in the present study u_0 is directly obtained from equation (17), with the angle of incidence set equal to 90° (vertically incident wave). $\{u_p\}$ is obtained from the solution of equation (4) and $\{\phi_p\}$ from the transverse displacements of the top two nodes. It may be seen that there is a good agreement between the two solutions. The slight variations may be attributed to the differences in the assumptions between the two models.

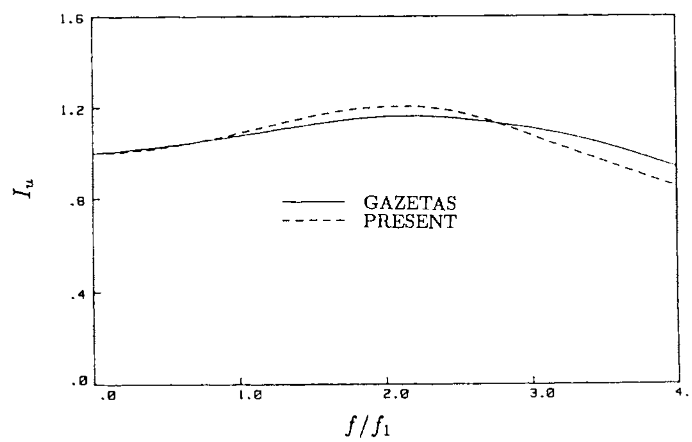


Fig. 3 - Comparison of Displacement Kinematic Interaction Factors, Layer Over Rigid Base. (L/d = 40, E_p/E_s = 50,000)

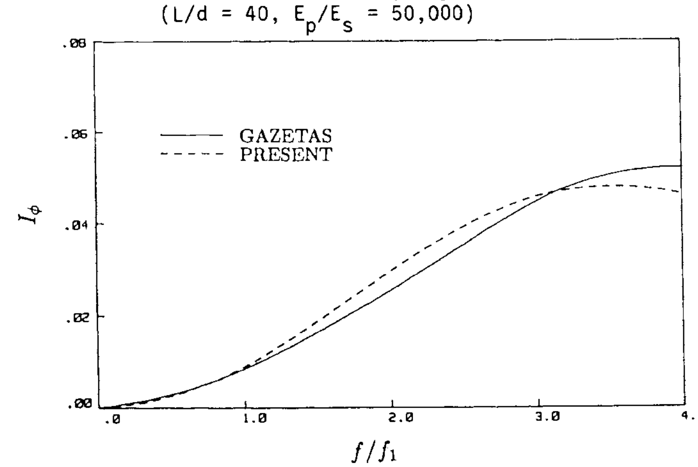


Fig. 4 - Comparison of Rotation Kinematic Interaction Factors, Layer Over Rigid Base (L/d = 40, E_p/E_s = 50,000)

Comparison with Kaynia (1982)

Figure 5 displays the absolute value of the transfer function from the ground surface horizontal displacement, u_0, to the pile head displacement, u_p, for a single pile, obtained by Kaynia (1982). In his study the seismic excitation corresponds to the case of shear waves propagating vertically through the soil (one dimensional amplification). In this problem, L/d = 15, E_p/E_s = 1000 and ρ_p/ρ_s = 0•70. Results obtained by the present analysis are also plotted in Figure 5.

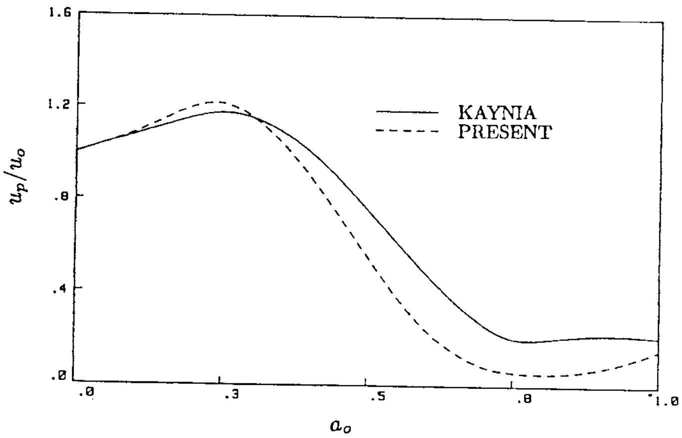


Fig. 5 - Comparison of Transfer Function for Horizontal Displacement of Pile Head (L/d = 15, E_p/E_s = 1000).

PRESENTATION AND ANALYSIS OF RESULTS

The results of the present study on the seismic behaviour of single piles are presented here in a dimensionless form. Among the problem parameters influencing the response, the most important are: the stiffness ratio E_p/E_s of the pile Young's modulus over a characteristic Young's modulus of the soil deposit; the slenderness ratio L/d of the length over the diameter of the pile, excitation frequency ω and angle of incidence of the seismic waves. The dimensionless frequency parameter is defined as

$$a_0 = \frac{\omega d}{C_s} \tag{22}$$

The Young's modulus of the soil was assumed to be constant with depth, typical of stiff overconsolidated clay deposits. The Poisson's ratio of the soil deposit was assumed to be 0•4. The ratio of the density of pile material to that of the soil was taken as 1 6, typical of concrete piles. Material damping (β) was set equal to 0 05.

Response Due to SH Waves

The SH-waves were assumed to produce unit displacement on the free-field. The pile shaft-soil interface was modelled with eleven cylindrical elements and the base as one circular disk.

The influence of the E_p/E_s ratio is portrayed in Figures 6(a) and (b) for a pile having L/d = 15. The analysis is performed at a dimensionless frequency a_0=0 35. This is justified because the spectra of earthquake records concentrate mostly on a low frequency range. Figure 6(a) shows the variation along the pile depth of the real part of the ratio between the total transverse displacement, u_p^t, and the free-field ground surface displacement, u_0. Figure 6(b) depicts the corresponding imaginary parts. The responses are shown for a vertically incident SH-wave (angle of incidence = 90°). It is observed that the E_p/E_s ratio has a profound effect on the responses, at all depths. Also, significant bending seems to occur for a very flexible pile (E_p/E_s=100 and 1000) and almost low, uniform rigid body motion for a very rigid one. Flexible piles are more susceptible to greater motions than rigid piles. It is also apparent that, for piles of lesser flexibility, bending moments are severe around the bottom half of the pile, which may be particularly susceptible to damage.

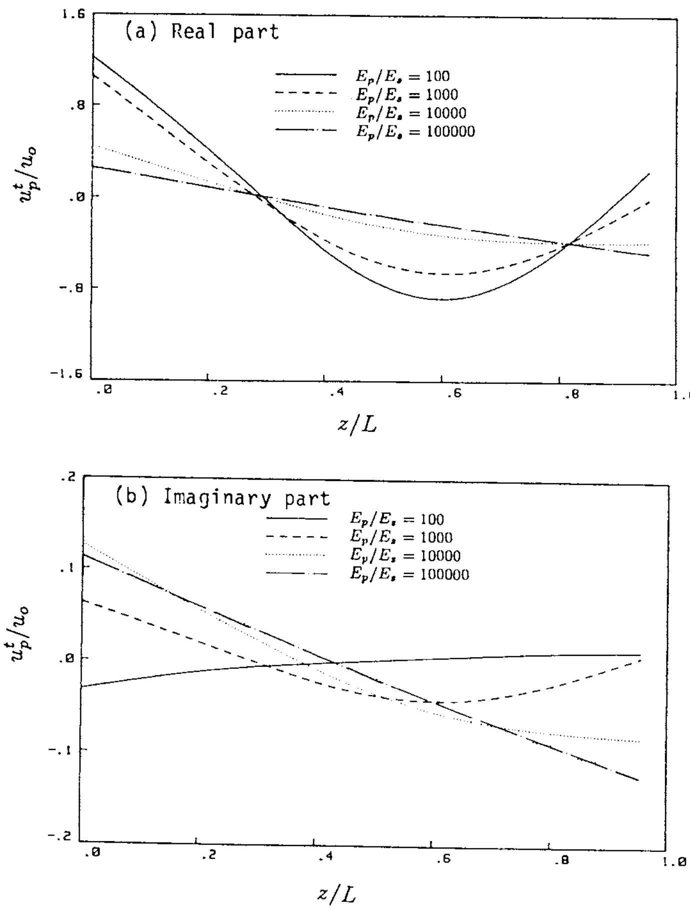


Fig. 6 - Transverse Displacment Ratios for Vertically Incident SH-Waves (L/d = 15, a_0=0 35):

(a) Real part, (b) Imaginary part

Figures 7(a) and (b) show the response of a pile having L/d = 15, E_p/E_s = 1000 to varying angle of incidence (Θ=45°, 60°, 75° and 90°) and at frequency a_0=0 35. These figures show the variation of the real and imaginary parts, respectively, of the ratio of u_p^t and u_0. It is observed that obliquely incident waves produce higher displacement (in the real part) than a vertically incident one, particularly in the upper two-third portion of the pile. But for the imaginary part, an opposite trend is observed. Also, bending along the pile decreases as the waves become more oblique.

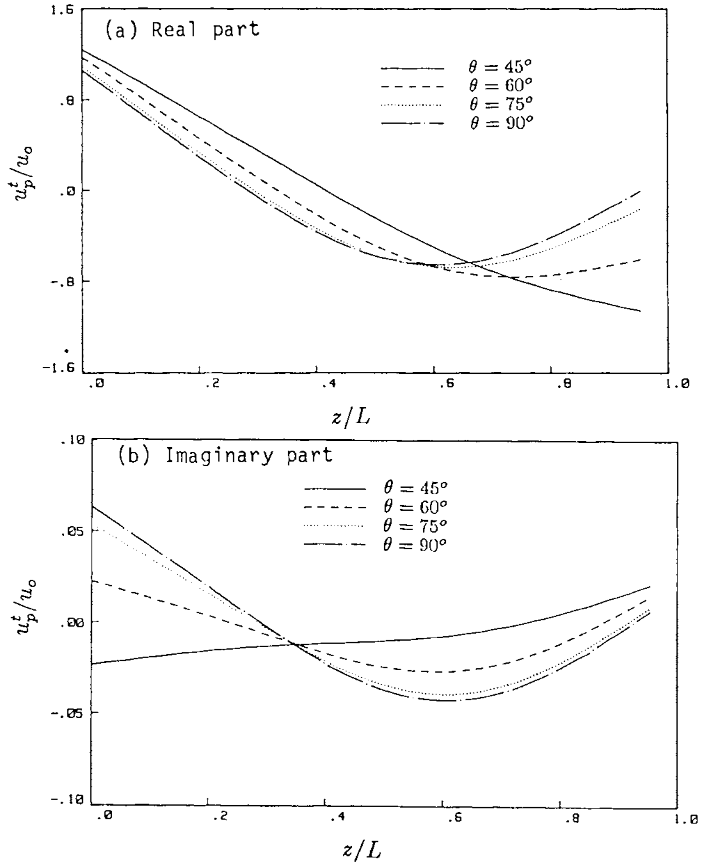


Fig. 7 - Influence of Incident Angles on Transverse Displacement Ratios, (L/d = 15, E_p/E_s = 1000, a_0 = 0 35): (a) Real part, (b) Imaginary part

Figures 8 and 9 show the variation of the displacement (I_u) and rotation (I_ϕ) kinematic interaction factors, respectively, for the same pile (i.e. L/d = 15 and E_p/E_s = 1000) versus the dimensionless frequency parameter a_0. The plots show the influence of the incident angles on the seismic response. It is apparent from Figure 8 that, at lower frequencies, piles appear to essentially follow the movement of the ground; hence, their presence has no practical effect on the seismic motion at the ground surface level. On the other hand, at higher frequencies, piles may not be able to follow the wavy movements of the free-fields and may thereby experience considerably reduced deformations. This is in agreement with the actual earthquake observations of the response of a pile foundation, as reported by Tajimi (1977). Moreover, at lower frequencies the angle of incidence seems to have little influence on the relative pile motion. But at higher frequencies, piles subjected to more obliquely incident waves continue to almost follow the ground motions.

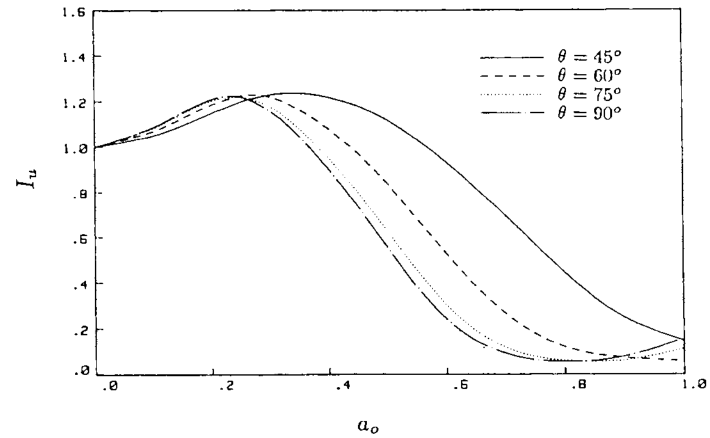


Fig. 8 - Displacement Interaction Factors for Varying Incident Angles (L/d = 15, E_p/E_s = 1000)

As already mentioned, a rotational component of motion is developed at the head of a pile in addition to the translational one (this component is not present in the free-field surface motion). The influence of varying angles of incidence on I_ϕ is depicted in Figure 9. It is apparent that, in the low frequency range, piles subjected to vertically incident waves show higher rotations of the pile head. But in the higher frequency range the opposite trend is observed; as the incident angle becomes more oblique, the pile heads exhibit higher rotations.

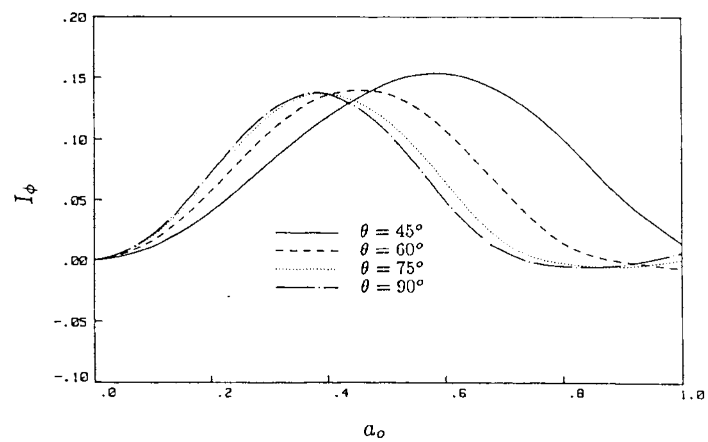
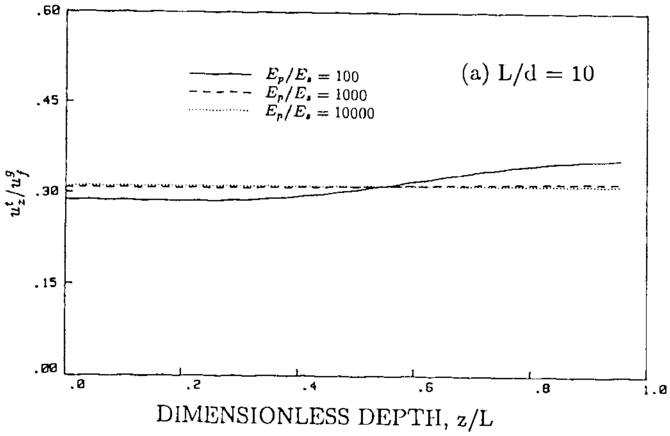


Fig. 9 - Rotation Interaction Factors for Varying Incident Angles. (L/d = 15, E_p/E_s = 1000)

Response Due to P and SV-Waves

The motion of single piles under the influence of a combination of P and SV waves is considered next. As in the case of SH waves, the results are presented as a function of the normalized depth z/L and dimensionless frequency parameter a_0. Incident waves are assumed to produce unit displacement on the free-field ground surface.

Figs. 10(a) and b) show the variation along the depth of the ratio of total axial displacement at the pile head u_z^t and the free-field axial ground motion u_f^g due to an obliquely incident combination of P and SV waves ($\Theta = 75^o$) having a frequency $a_o = 0.5$, for a short and long pile, respectively. Corresponding transverse displacement ratios are portrayed in Figs. 11(a) and (b). As in the case of SH waves, the stiffness ratio and the embedment length have significant influence on the responses. Whereas, a rigid pile shows almost a uniform rigid body motion, flexible piles exhibit oscillatory movements, resulting in unacceptable amounts of moments that could cause yielding and fracture in the pile.



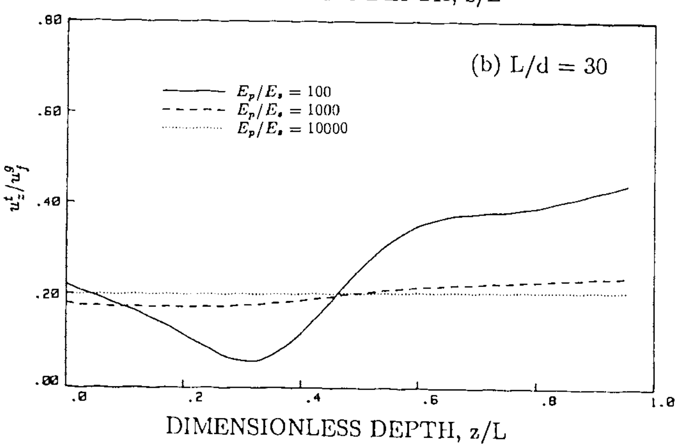
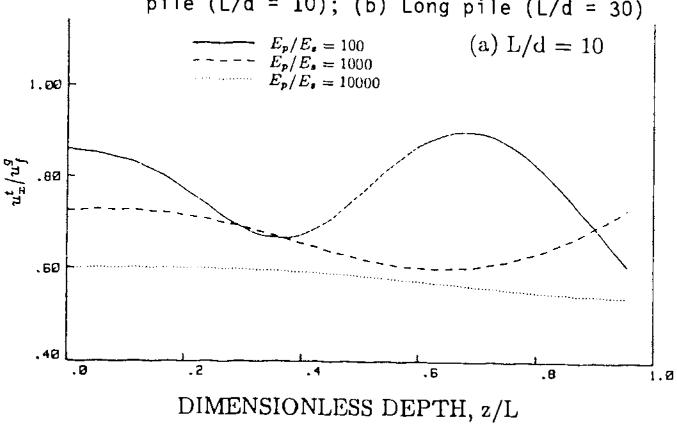


Fig. 10 - Axial Displacement Ratios for Obliquely Incident SV and P Waves (Θ=75^o): (a) Short pile (L/d = 10); (b) Long pile (L/d = 30)



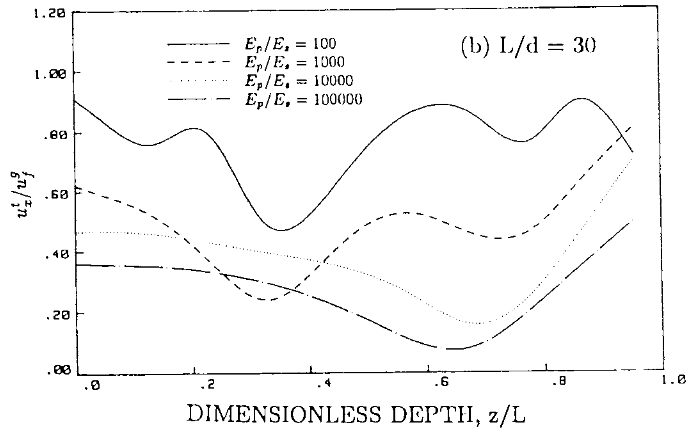
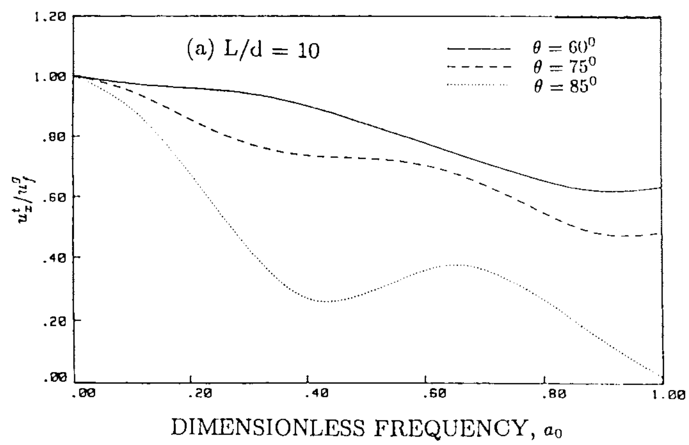


Fig. 11 - Transverse Displacement Ratios for Obliquely Incident SV and P Waves (Θ=75^o): (a) Short pile (L/d = 10); (b) Long pile (L/d = 30).

The transverse displacement ratios u_x^t/u_f^g for the top of the pile are depicted in Figs. 12(a) and (b) for a short and long pile, respectively, with E_p/E_s equal to 1,000. Again, the increase in the filtering effects with the increasing angle of incidence is evident.



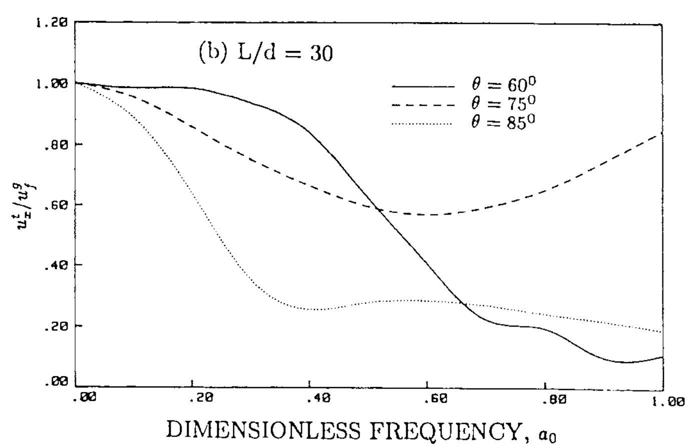


Fig. 12 - Transverse Displacement Ratios for Obliquely Incident SV and P Waves (E_p/E_s = 1000); (a) Short pile (L/d = 10); (b) Long pile, (L/d=30).

PRACTICAL APPLICATIONS

The practical significance of all such curves is apparent: By multiplying a given free-field design response spectrum with the appropriate interaction curve, the design response spectrum can be obtained and used as input motion at the base of a structure on pile foundations.

Such results would be useful not only for developing an improved understanding of the mechanics of the problem, but also for deriving simple preliminary design rules for foundation engineering practice. This study was well worth undertaking, since the majority of important structures are supported on pile foundations and knowledge of the subject is inadequate. Also, the results would interest many engineers who have built heavy structures on soft soils in seismic-prone areas.

CONCLUSIONS

Responses of piles to vertically and obliquely incident seismic waves have been analysed. It is found that the stiffness ratio, angle of incidence and the excitation frequency have significant influence on the seismic responses of piles. While the number of cases studied is not sufficiently large to derive approximate formulae or general conclusions, it appears that at the low frequency range piles essentially follow the ground motion. On the other hand, at higher frequencies they seem to remain relatively still, while the free-field soil mass moves considerably. This filtering effect is found to be severe for a vertically incident wave, gradually diminishing for a more obliquely incident one.

Flexible piles undergo significant bending under seismic excitation, whereas rigid piles tend to show almost low, uniform rigid body motion. Obliquely incident waves produce higher displacement than a vertically incident one throughout the pile depth. In the low frequency range vertically incident waves produce higher rotations of the pile head; but in the higher frequency range the opposite trend is observed.

The interaction curves presented in this paper have significant practical utility. A design response spectrum for a structure resting on a pile foundation may be obtained readily by multiplying a given free-field design response spectrum with the value of transfer function from the appropriate interaction curve.

REFERENCES

Apsel, R. "Dynamic Green's Functions for Layered Media and Applications to Boundary Value Problems," Ph.D. Thesis, University of California at San Diego, CA, 1979.

Banerjee, P.K. "Analysis of Axially and Laterally Loaded Pile Groups." Developments in Soil Mechanics, C.R. Scott, ed., Applied Science Publishers, London, England, 1978, 317-346.

Banerjee, P.K. and Butterfied, R. Boundary Element Methods in Engineering Sciences. Mc-Graw Hill, London, England.

Banerjee, P.K. and Driscoll, R.M. "Three-dimensional Analysis of Raked Pile Groups." Proc. Inst. of Civ. Engrs., part 2, 1976, v. 61, 653-671.

Banerjee, P.K. and Mamoon, S.M. "A Fundamental Solution Due to a Periodic Point Force in the Interior of an Elastic Half-space." J. of Earthquake Engrg. and Struct. Dynamics, 1990, vol. 19, 91-105.

Banerjee, P.K. and Sen, R. "Dynamic Behavior of Axially and Laterally Loaded Piles and Pile Groups." Dynamic Behavior of Foundations and Buried Structures, P.K. Banerjee and R. Butterfield, eds., Elsevier Applied Science, London, England, 1987, 95-133.

Banerjee, P.K., Sen, R., and Davies, T.G. "Static and Dynamic Analyses of Axially and Laterally Loaded Piles and Pile Groups." Geotechnical Modeling and Applications, S.M. Sayed, ed., Gulf Publishing Co., Houston, Tx, 1987, 322-354.

Dobry, R., Oweis, I. and Urzua, A. "Simplified Procedures for Estimating the Fundamental Period of a Soil Profile." Bull. Seismological Soc. of Am., 1976, vol. 66, no. 4, 1293.

Dominguez, J. and Rosesset, J.M. "Response of Embedded Foundations to Traveling Waves." Report No. R78-24, Dept. of Civ. Engrg., Mass. Inst. of Techn., Cambridge, Mass, 1976.

Flores-Berrones, R. and Whitman, R.V. "Seismic Response of End Bearing Piles." J. Geotech. Engrg. Div., ASCE, 1982, vol. 108, no. 4, 554-569.

Gazetas, G. "Seismic Responses of End Bearing Single Piles." Soil Dynamics and Earthquake Engrg., 1984, vol. 3, no. 2, 82-93.

Kaynia, A.M. and Kausel, E. "Dynamic Behavior of Pile Groups." Second Int. Conf. on Numerical Methods in Offshore Piling, Austin, TX., 1982.

Mamoon, S.M. "Dynamic and Seismic Behavior of Deep Foundations," Thesis presented to the State Univ. of New York at Buffalo, NY, in partial fulfillment of the requirements of the degree of Doctor of Philosophy, 1990.

Mamoon, S.M. and Ahmad, S. "Seismic response of pile to obliquely incident SH, SV, and P waves," J. of Geotech. Engrg. Div., ASCE, 1990, vol. 116, no. , 186-204.

Mamoon, S.M., Banerjee, P.K. and Ahmad, S. "Seismic Response of Pile Foundations." Tech. Report NCEER-88-0034, Nat. Center for Earthquake Engrg. Res., State University of New York at Buffalo, Buffalo, NY, .988.

Michaipoulos, E. "Amplification of Generalized Su.face Waves," Thesis presented to the Massachusetts Institue of Technology, at Cambridge, MA, in partial fulfillment of the requirement for the degree of Master of Science, 1976.

Poulos, H.G. and Davies, E.H. Pile Foundation Analysis and Design. John Wiley and Sons, Inc., New York, NY, 1980.

Roesset, J.M. and Angelides, D. "Dynamic Stiffness of Piles: Numerical Methods in Offshore Piling." Inst. of Civ. Engrs., London, England, 1979, 22-23.

Roesset, J.M. "Soil Amplification of Earthquakes." Numerical Methods in Geotechnical Engineering, C.S. Desai and J.T. Christian, eds., McGraw-Hill, New York, NY, 1979.

Sen, R., Davies, T.G., and Banerjee, P.K. "Dynamic Analysis of Piles and Pile Groups Embedded in Homogeneous Soils." J. Earthquake Engrg. and Struct. Dynamics, 1985, vol. 13, no.1, 53-65.

Tajimi, H. "Seismic Effects on Piles." State of the Art Report No. 2, Ninth Int. Conf. on Soil Mech. and Foundation Engrg., Specialty Session, 1977, vol. 10, 15-26.

Waas, G. and Hartmann, G.H. "Pile Foundations Subjected to Dynamic Horizontal Loads," Conf. Struct. Mech. in Reactor Tech., Capri, Italy, 1981.

Wolf, J.P. and Von Arx, G.A. "Impedance Functions of a Group of Vertical Piles." Proc. Geotech. Engrg. Div., ASCE Specialty Conf. on Earthquake Engrg. and Soil Dynamics, Pasadena, CA, 1978, vol. 5, no. 2, 1024-1041.

Wolf, J.P. and Von Arx, G.A. "Horizontally Traveling Waves in a Group of Piles Taking Pile-soil-pile Interaction into Account." Earthquake Engrg. and Struct. Dynamics, 1978, vol. 10, no. 2, 225-237.

Wong, H.L. and Luco, J.E. "Dynamic Response of Rectangular Foundations to Obliquely Incident Seismic Waves." J. Earthquake Engrg. and Struct. Dynamics, 1978, vol. 6, no. 1, 3-11.