ANALYSIS OF PILE FOUNDATIONS FOR ASEISMIC DESIGN

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SUMMARY

This paper presents a simple approach for the analysis of pile foundations for seismic loads. The linear plane strain soil reactions are used in the analysis. This approach is tested by comparing the method with shaking table test results. A closed form solution is presented for the harmonic response of piles to shear and Rayleigh waves in the horizontal direction, and compression and Rayleigh waves in the vertical direction. The axial stress produced in the pile is shown to be significant.

INTRODUCTION

The analysis of pile foundations for seismic loading is customarily performed by applying the horizontal forces and overturning moments from the superstructure to the pile foundation at the ground level. These forces result from the analysis of the structure under support excitement conditions. A theoretical study by the authors (Ref. 1) showed that the stresses in pile foundations due to the soil motion can be very significant. To consider the stresses produced in piles, a simple model is used for the soil-pile-structure system as shown in Fig. 1. This model, which corresponds to a rigid massless base at the pile head, allows the study of pile behavior under soil motion only. To study the effect of the structure, another model is used which incorporates the effect of the structure at the pile head. Substructuring could be used to analyze the soil-pile-structure system (Ref. 2).

Soil Free Field Motion

The soil motion is the result of a complex process of energy release and wave propagation through different media. The resulting motion at any given site depends in part on the fault mechanism, the wave's path and the local site conditions. Since determination of the component waves of an earthquake record is not practical, assumptions regarding actual free-field motion must be made. The most commonly used assumption is that the shear wave (S-wave) propogates vertically, yielding only the horizontal component of motion. A parallel assumption of a compression wave (P-wave) propogating vertically can

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be made yielding the vertical component of motion. These assumptions apply independently to the horizontal and vertical records.

A surface wave assumption can also be used in which one of the earthquake components, e.g., the horizontal, can be assumed to be the result of Rayleigh wave (R-wave) propogation with the corresponding vertical component calculated based upon the horizontal component. This vertical component will not in general match the recorded motion.

Additionally, inclined body waves can be used to produce the horizontal and vertical records (Ref. 3).

After the wave pattern is determined, a closed form solution for the soil motion can be determined for some cases. For a two-layered viscoelastic soil system the solution for shear waves is well known (Ref. 4). For Rayleigh waves the solution is available (Ref. 1). For more than two layers the solution for Rayleigh waves becomes cumbersome and the finite element solution is more attractive.

In this paper a two layered soil system with frequency independent soil damping is employed.

Horizontal Pile Response

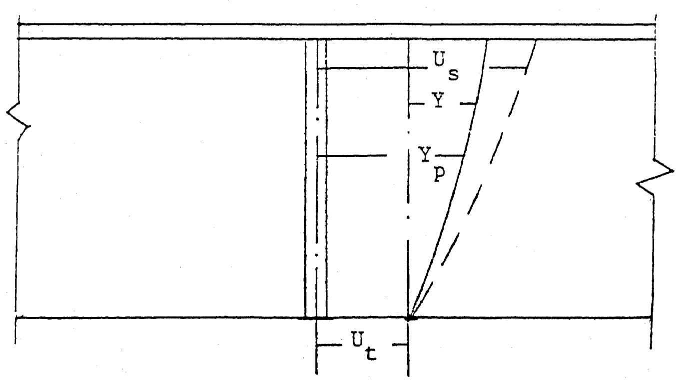

Fig. 1. Soil and Pile Displacements

For the model shown in Fig. 1, the equation of motion of a pile element can be written

$$EI \frac{\partial^4 y}{\partial z^4} + m \frac{\partial^2 y_p}{\partial t^2} + K_u (y_p - u_s) = 0. \quad (1)$$

in which E is the pile's modulus of elasticity, I is the moment of inertia, and m is mass per unit length; y(z,t) is the relative pile displacement, $y_p(z,t)$ and $u_s(z,t)$ are the total pile and soil displacements, respectively. The modulus K_u^s is the horizontal soil stiffness. Using a plane strain model (Ref. 5),

$$K_u = G(S_{u1} + i\ S_{u2}) \quad (2)$$

in which G is the soil shear modulus, $i = \sqrt{-1}$, S_{u1} and S_{u2} are dimensionless functions of the frequency factor a_o, $a_o = r_o \omega / v_s$ where ω is the frequency, r_o is the pile radius, and v_s is the shear wave velocity of the soil layer.

Assuming harmonic motion, in which $u_t(t) = U_t e^{i\omega t}$ is the pile tip displacement, $u_s = U_s(z)e^{i\omega t}$, $y_p = y + u_t$ and $y = Y(z)e^{i\omega t}$, Eq. 1 becomes

$$EI \frac{d^4 Y}{dz^4} + (K - m\omega^2)\, Y = K_u U_s - (K_u - m\omega^2)\, U_t \qquad (3)$$

The solution of Eq. 3 may be written

$$Y = C_1 \cosh \lambda z + C_2 \sinh \lambda z + C_3 \cos \lambda z + C_4 \sin \lambda z - U_t + F(z) \qquad (4)$$

in which C_1 through C_4 are integration constants and

$$\lambda^4 = \frac{m\omega^2 - K_u}{EI}$$

The function F(z) is a wave dependent soil-pile interaction relationship. For shear waves F(z) can be written

$$F(z) = U_o\, q' \cos \alpha z \qquad (5)$$

in which $\alpha = \omega / v_s$, $U_o(t)$ is the soil displacement at the surface, and

$$q' = \frac{K_u}{EI\, \alpha^4 + K_u - m\omega^2}$$

For Rayleigh waves F(z) may be written

$$F(z) = q(A_1 e^{-u_1 z} + B_1 e^{u_1 z}) + s(D_1 e^{-u_1' z} + E_1 e^{u_1' z}) \qquad (6)$$

in which A_1, B_1, D_1, and E_1 are constants describing the fundamental Rayleigh wave displacement (Ref.1),

$$u_1 = \omega \left[\left(\frac{1}{v_r}\right)^2 - \left(\frac{1}{v_p}\right)^2 \right]^{1/2} \; ; \; u_1' = \omega \left[\left(\frac{1}{v_r}\right)^2 - \left(\frac{1}{v_s}\right)^2 \right]^{1/2}$$

v_r is the Rayleigh wave velocity, v_p is the compression wave velocity,

$$q = \frac{K_u}{EI\, u_1^4 + K_u - m\omega^2} \quad \text{and} \quad s = \frac{K_u}{EI\, u_1'^4 + K_u - m\omega^2}$$

<u>Vertical Pile Response</u>

The equation of motion in the vertical direction is

$$m \frac{\partial^2 w_p}{\partial t^2} - EA \frac{\partial^2 w}{\partial t^2} + K_w (w_p - w_s) = 0 \quad (7)$$

in which A is pile cross-sectional area, w is the pile relative displacement, w_p and w_s are the total pile and soil displacements, respectively. The modulus K_w is the vertical soil stiffness. Again using the plane strain model (Ref.5)

$$K_w = (S_{w1} + i\, S_{w2}) \quad (8)$$

The solution of Eq. 7 may be expressed

$$W = C_5 \cos \lambda_1 z + C_6 \sin \lambda_1 z - W_t + F'(z) \quad (9)$$

in which C_5 and C_6 are integration constants, W_t is the soil-pile displacement at the tip and

$$\lambda_1^2 = \frac{m\omega^2 - K_w}{EA}$$

Again, F'(z) is a wave dependent soil-pile interaction relationship. For compression waves F'(z) can be written

$$F'_z = W_o q'_1 \cos \beta z \quad (10)$$

in which $\beta = \frac{\omega}{v_p}$, $W_o(t)$ is the soil displacement at the soil surface

and

$$q'_1 = \frac{K_w}{K_w - m\omega^2 + EA\, \beta^2}$$

For Rayleigh waves

$$F'(z) = q_1 (A_2 e^{-u_1 z} + B_2 e^{u_1 z}) + s_1 (D_2 e^{-u'_1 z} + E_2 e^{u_1 z}) \quad (11)$$

where

$$q_1 = \frac{K_w}{K_w - m\omega^2 - EA\, u_1^2} \quad ; \quad s_1 = \frac{K_w}{K_w - m\omega^2 - EA\, u_1'^2}$$

and A_2, B_2, D_2 and E_2 are constants related to A_1, B_1, D_1, and E_1 in Eq. 6 to describe the Rayleigh wave displacements in the vertical direction.

Boundary Conditions

To investigate the behavior of piles under the effect of soil motion, boundary conditions which correspond to a rigid massless base at the pile head are used. The axial stress at the top of the pile vanishes, i.e.,

$EA \left. \frac{\partial w}{\partial z} \right|_{z=0} = 0$, and the tip of the pile is fixed $w(z=H) = 0$, where H is the pile embedment depth.

In flexure, the pile is assumed fixed at the head and pinned at the tip which leads to the following conditions:

$$\text{at } z = 0 \qquad EI\frac{\partial^3 y}{\partial z^3} = 0 \qquad \text{and} \qquad \frac{\partial y}{\partial z} = 0$$

$$\text{at } z = H \qquad \frac{\partial^2 y}{\partial z^2} = 0 \qquad \text{and} \qquad y = 0$$

Discussion

An analysis was applied to a single pile with the following properties: $EI = 3.391 \times 10^8$ lb-ft^2, $r_o = 1$ ft , $H = 50$ ft, $m = 14.4$ lb-sec^2/ft^2. The properties of the first and second layers are: $G_1 = 10^6$ psf , $G_2 = 10^7$ psf, $\rho_1 = 3.0$ lb-sec^2/ft^4, $\rho_2 = 4.5$ lb-sec^2/ft^4, $\nu_1 = 0.33$ and $\nu_2 = 0.22$ where ρ is mass density and ν is Poisson's ratio. A damping ratio of 5.0% was used.

Two frequencies in the range of interest for Rayleigh waves (6 rad/s and 18 rad/s) were used. The results are shown in Figures 2 through 4.

Based on this example, the following observations can be made:

1. Rayleigh waves produce horizontal and vertical soil displacements similar to those produced by shear and compressive waves, as shown in Figure 2. This observation is verified along with a more detailed study in Ref. (3). As the frequency increases, the soil displacements due to Rayleigh waves begin to differ from those due to body waves.
2. The pile tends to follow the soil movement due to both Rayleigh and body waves. No significant reduction of soil movement is observed in this frequency range (Figure 3) due to the presence of the pile.
3. The axial stresses due to Rayleigh and compression waves can be very significant even when the vertical soil motion is only about 50% of the horizontal motion. Figure 4 shows the distribution of the axial and bending stresses in pile with depth. It can be seen for this example, that the axial stress is dominant in the lower part of the pile. These calculated stress trends seem to agree with field observations (Ref. 7). Rayleigh waves produce higher maximum bending stress and lower axial stress when compared with shear and compression waves, respectively.

COMPARISON WITH TEST RESULTS

A shaking table test was performed on a model of a tank supported by piles (Ref. 8). The axial and bending stresses in a single pile were calculated using the shear wave equations and compared with measured values from the experimental model. The axial stresses for the model were calculated by considering the strain in the piles due to the rigid rotation of the superstructure about the top of the middle pile.

The boundary conditions assumed for a model with an attached rigid mass are:

$$\text{at } z = 0 \qquad M\omega^2 y_m = EI\frac{\partial^3 y}{\partial z^3} \qquad \text{and} \qquad M\omega^2 h y_m = EI\frac{\partial^2 y}{\partial z^2}$$

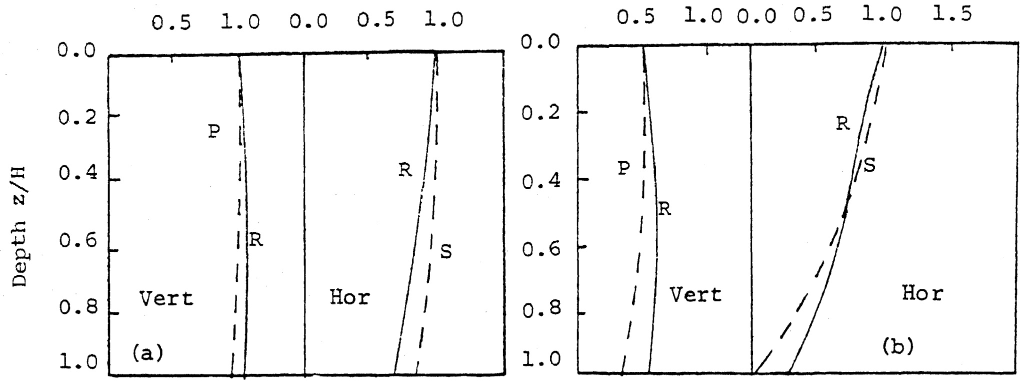

Fig. 2. Soil Displacements a) ω = 6 rad/sec b) 18 rad/sec

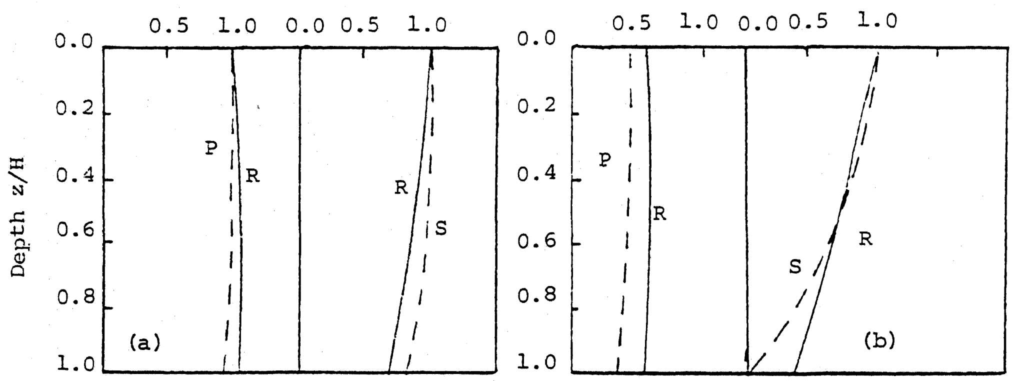

Fig. 3. Pile Displacement a) ω = 6 rad/sec b)]8 rad/sec

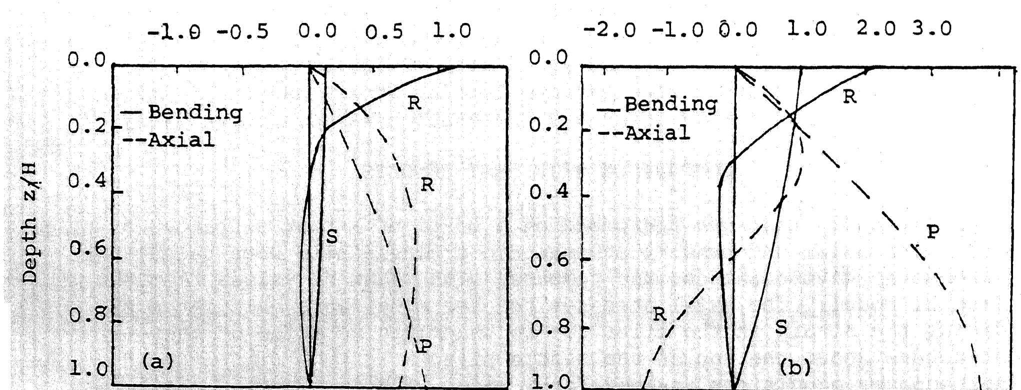

Fig. 4. Stress/(10^{-3}E) in Pile a) ω = 6 rad/sec b)]8 rad/sec

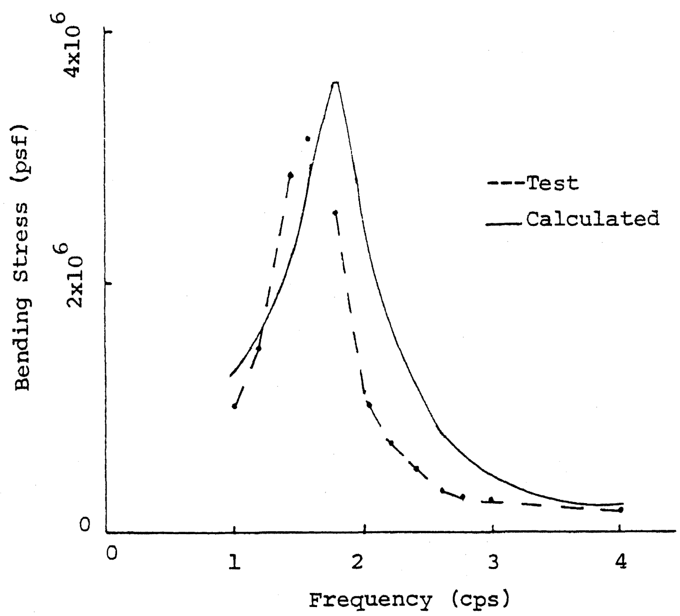

Fig. 5. Maximum Bending Stress in Pile

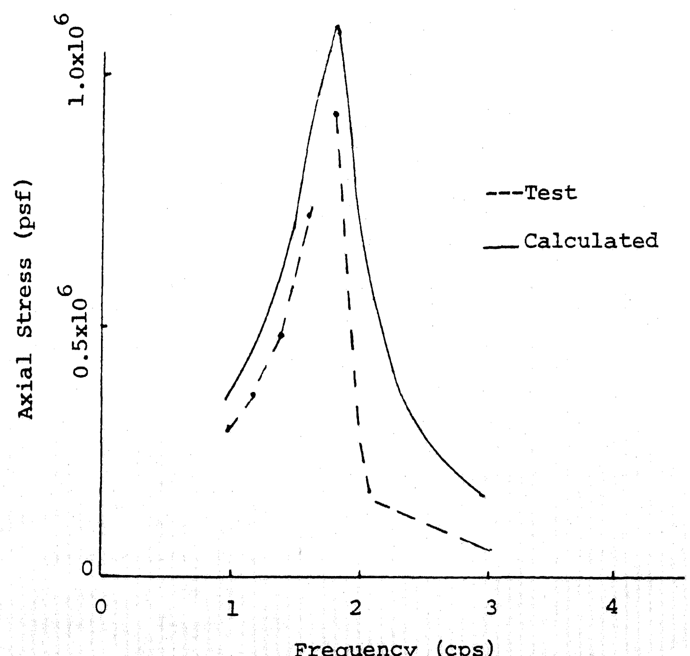

Fig. 6. Maximum Axial STress in Pile

where y_m is the horizontal displacement of the center of gravity of the mass of the superstructure, M, as given by $y_m = y_p(z=0) - h\Theta$

where $\Theta = \left.\frac{\partial y}{\partial z}\right|_{z=0}$, and h is the height of the center of gravity above the pile head. The pile tip was assumed fixed.

A comparison between the computed and experimental maximum bending and axial stress is shown in Figs. 5 & 6, respectively. These values are in good agreement. The differences are due to: the difficulty in assigning the proper soil properties to the theoretical model, the difference between the soil accelerations observed and those calculated by the shear wave propogation theory (Ref. 8), and the exclusion of the rotational mass moment of inertia of the superstructure.

CONCLUSIONS

A method is presented for the analysis of pile response to seismic waves. It is shown that the axial stresses can be an important factor in design in a seismic environment. Rayleigh and body waves produced similar response in the low frequency range for the included example. Comparison with experimental test results were favorable. Further improvements on the model can be made by using a multilayered soil system.

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