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SYNOPSIS While currently available methods of dynamic soil-foundation interaction idealize the soil as a continuum, this paper presents a general theory to obtain the dynamic response of offshore caissons resting on a saturated or nearly saturated poroelastic medium. The model, based on Biot's theory, considers the compressibility of both solid and fluid phase and assumes that the fluid flow is governed by Darcy's Law for an isotropic medium. Results are presented as plots of normalized amplitudes of displacement load or rotation-moment ratios for a rigid strip founded on a dense coarse sand. The results demonstrate that fluid compressibility, which is primarily a function of the degree of saturation, has an important effect on rocking motions. Soil premeability appears to have a rather minor effect on the response.

INTRODUCTION

It has long been recognized that shearing deformations in saturated sands occur with virtually no interference of the pore water and, thus, the shear wave velocity depends only on the stiffness of the soil skeleton, except for the minor influence of the added mass of water. Volumetric strains, on the other hand, caused primarily by the rearrangement of grains, have to overcome the elastic resistance of the pore water. Consequently, there exists a strong structural coupling between the solid skeleton and the water filling the pore space, as a result of which dilatational wave velocity depends strongly on the relative compresibilities of the (porous) solid and fluid constituents of the soil.

To mathematically describe the structural coupling of the two soil phases, Biot (1941) proposed that soil be modeled as a *poroelastic* medium rather than as a (one phase) continuum. Biot's poroelastic "soil" consists of two distinct and interacting phases: a continuous elastic (or viscoelastic) porous solid and a continuous elastic fluid, representing the soil skeleton and the pore water, respectively. The basic deviation of such a model from reality lies in the assumption of continuity of the solid phase which contrasts with the particulate nature of soils. Thus, for instance, no coupling between volumetric and shear strains ('dilatancy') can be reproduced with the model, unless the solid phase exhibits inelastic deformational characteristics, as was proposed by Bažant & Krizek (1975) and Prévost (1979). Nevertheless, the model has led to the development of a theory (Biot, 1955, 1956; Deresiewicz, 1960; Paria, 1958; DeJosselin deJong, 1963; etc.) which can successfully account for pore-water flow, volume changes and deformations that take place simultaneously in real soils.

Despite these capabilities of the poroelastic theory few attempts habe been made to use it in dynamic problems of interest to geotechnical and earthquake engineers, as for example in problems of dynamic soil-foundation interaction caused by machine, sea-wave and earthquake loading. Two are believed to be the reasons: (1) The mathematical difficulties arising from the adoption of the poroelastic model for soil. (2) The lack of a clear understanding of the soil parameters that the model introduces and the absence of well-established experimental methods to evaluate their magnitude. In his original formulations Biot only qualitatively described the material constant of his model, while later Biot & Willis (1975) presented expressions in terms of the so-called 'unjacketed' compressibilities of the two phases due to a pressure applied to the pore water; these parameters, however, are not particularly useful for practical applications. Currently, because of the work of Ishihara (1967, 1970) and Bažant & Krizek (1975), the poroelastic constants can be evaluated in terms of parameters readily obtainable in the laboratory, as is shown in this paper.

This paper demonstrates that it is possible to rigorously formulate and solve the problem of obtaining the dynamic response of long caisson-type structures founded on poroelastic soil and excited by sea-wave or earthquake loading. Preliminary results, presented in the form of normalized amplitudes of displacement-load or rotation-movement ratios as functions of frequency, reveal that fluid compressibility, which depends primarily on the degree of saturation of the porous soil, is the most important parameter that may influence the amplitude of rocking oscillations of a caisson. For the range of frequencies of interest (i.e., 0.1 Hz $\leq$ f $\leq$ 5 Hz), soil permeability has some effect only if it is very large (of the order of a few centimeters/second) and the frequency of vibration very low (less than 0.5 Hz). Finally, the porosity of the soil exerts its main influence through the shear modulus of the solid skeleton and the bulk density of the porous medium.

To give the reader some background information, before proceeding to our analysis, several published studies related in one way or another to the present problems are next briefly reviewed.

SUMMARY OF PREVIOUS WORK

Biot (1956) considered the three dimensional wave propagation of shear and dilatational waves in a poroelastic medium and showed the existence of two types of dilatational waves and of one rotational wave. The first dilatational wave ('fast' or P_f - wave) is transmitted through the fluid and solid phase and is controlled by the bulk compressibility of the medium; the second ('slow' or P_s-wave) involves large volumetric changes of porous solid phase and at low frequencies of vibration loses the nature of a wave reducing to a diffusion-type (consolidation) process. The practical implications of Biot's findings were evaluated by Ishihara (1967, 1970) who related the poroelastic parameters to the porosity, n, and the compressibilities of the soil skeleton, C_p, the solid particles, C_s, and the fluid, C_w. Using realistic ranges of these parameters, he found that in the frequency range of interest to engineers, the P_f - wave travels without drainage of the pore-water, even if soil permeability is large ($\simeq$ 0.10 cm/sec), and, consequently, suggested that a continuum undrained soil model (having the same shear modulus, G, and a Poisson's ratio $\nu \simeq 0.5$ $[1 - GnC_w]$) can replace with very good accuracy the two-phase poroelastic medium. To explain the absence of relative motion between water and solid he observed that at low frequencies the wave-lengths are large and thus the pore-water pressuregradients (e.q., $\partial p_w/\partial x$) are small to cause any noticeable water flow, despite the large time intervals involved. On the other hand, he also noticed the strong dependence of P_s-wave velocity on frequency and showed that at very low frequencies the deformation can progress only as a consolidation process.

Deresiewicz (1960) and Deresiewicz & Rice (1962) were the first to present solutions to simple boundary value problems involving wave progagation through poroelastic layers. General two-dimensional solutions of the governing field equations were obtained through a Helmholtz resolution of the displacement vectors of the two phases and, then, some phenomena occuring during reflection and refraction of body waves in poroelastic media were studied. This work has served as the starting point for our formulation that is presented in this paper. Recently, Halpern & Christiano (1979) implemented Deresiewicz's solution to obtain dynamic impedance functions for a poroelastic halfspace due to a vertical oscillatory normal point force at the surface (i.e., Lamb's problem for a poroelastic medium). Results, however, were presented only for a case of one-dimensional wave propagation.

Allen, Richart & Woods (1980) reported an experimental investigation of compressional wave propagation in saturated and nearly saturated sands. They found P_f-wave velocity to depend very strongly on the degree of saturation, S: a decrease in S from 100% to 99.7% reduces the velocity by a factor of two, while in a completely saturated medium void ratio is the most influential parameter. Ishihara's equation predicted with good accuracy the observed variation of P_f-wave velocity with changing soil parameters.

In recent years the search for oil and gas in deep water has stimulated a significant research effort aimed at predicting the behavior of marine soils during ocean wave storms as well as earthquakes. Several studies have appeared on the generation and dissipation of pore water pressures and the related phenomenon of liquefaction of sand deposits (Lee et., al, 1975; Moshagen et al, 1975; Prevost et al, 1975; Rahman et al, 1977; Madesen, 1978; Wijesinghe et al, 1980; Yamamoto et al, 1978; Martin et al, 1980; Nataraja et al, 1980). In all of these studies only ocean wave loading was investigated and only Rahman et al, (1977) studied the effect of of soil-tank interaction on the development pore pressures. The work of Madsen (1978) and Yamamoto et al, (1978) is of greatest interest in connection with our analysis, since they employed Boit's poroelastic model to determine the effect of (sinusoidal) water waves propagating over a porous deep sand layer. In contrast with the formulation of Boit (1956) and Deresiewicz et al (1962), however, these authors ignored the inertia effects in the soil and treated the loading as a quasi-static one, presumably because the large periods of oscillation associated with strong ocean wave storms make the dynamic effects secondary. The main conclusions of these studies can be summarized as follows: (1) for fully saturated soils consisting of silt or finer material permeability has no effect on soil response and no phase lag occurs between applied and generated pore pressures; permeability is, however, significant in medium and coarse sands. (2) The behavior of partially saturated sands depends primarily on the relative compressibility of the soil skeleton and the pore fluid. These results are in qualitative agreement with those of Ishihara (1970).

One of the objectives of the research whose first results are reported herein is to bridge the gap between the quasi-static methods employed in the studies of ocean-wave induced pore pressures and effective stresses and the dynamic formulations of Biot, Deresiewicz and Ishihara. The analysis that is presented here is general in that the response can be obtained not only due to progressive sea waves, but also to any prescribed harmonically varying with time surface excitation.

STATEMENT OF THE PROBLEM

Fig. 1a shows in cross-section a long gravity caisson during a wave storm. The caisson rests on the surface of the soil and has its axis perpendicular to the line of the wave profile. Being an obstruction in the free wave field, the caisson is subjected to wave forces (diffraction) thereby, undergoing swaying and rocking oscillations. Moreover, the soil surface (sea floor) is also subjected to wave induced pore pressures which are in-phase with the oscillations of the sea surface and have an amplitude

$$p_o = \frac{1}{2}\ \rho_w gH/\cosh(2\pi d/L) \tag{1}$$

As a result, fluid flow and deformations occur in the porous medium and cause additional vibrations of the caisson. Thus, the whole problem can be reduced in the two 'key' subproblems illustrated schematically in Figs. 1b and 1c. The paper presents a rigorous formulation and solution to these two plane-strain problems, namely, the determinitation of displacement and stress fields in a poroelastic medium subjected to harmonic surface pressures, $p_o \exp(i\omega t)$, or carrying a rigid strip (massless) plate that undergoes rocking and swaying vibrations due to external harmonic forces, e.g., $F_o \exp(i\omega t)$. The formulation can be readily extended to treat soil profiles consisting of any number of horizontal layers, using the technique described by Gazetas & Roesset (1976) and Gazetas (1980). However, only the case of a very deep soil deposit (halfspace) is studied herein.

POROELASTIC SOIL PARAMETERS

The following soil parameters are needed to describe the behavior of a poroelastic medium:

(1) The porosity $n = V_u/V$, where V_u = the pore volume and V is the total (bulk) volume of the soil.

(2) The shear modulus G which is independent of the presence of the fluid and characterizes

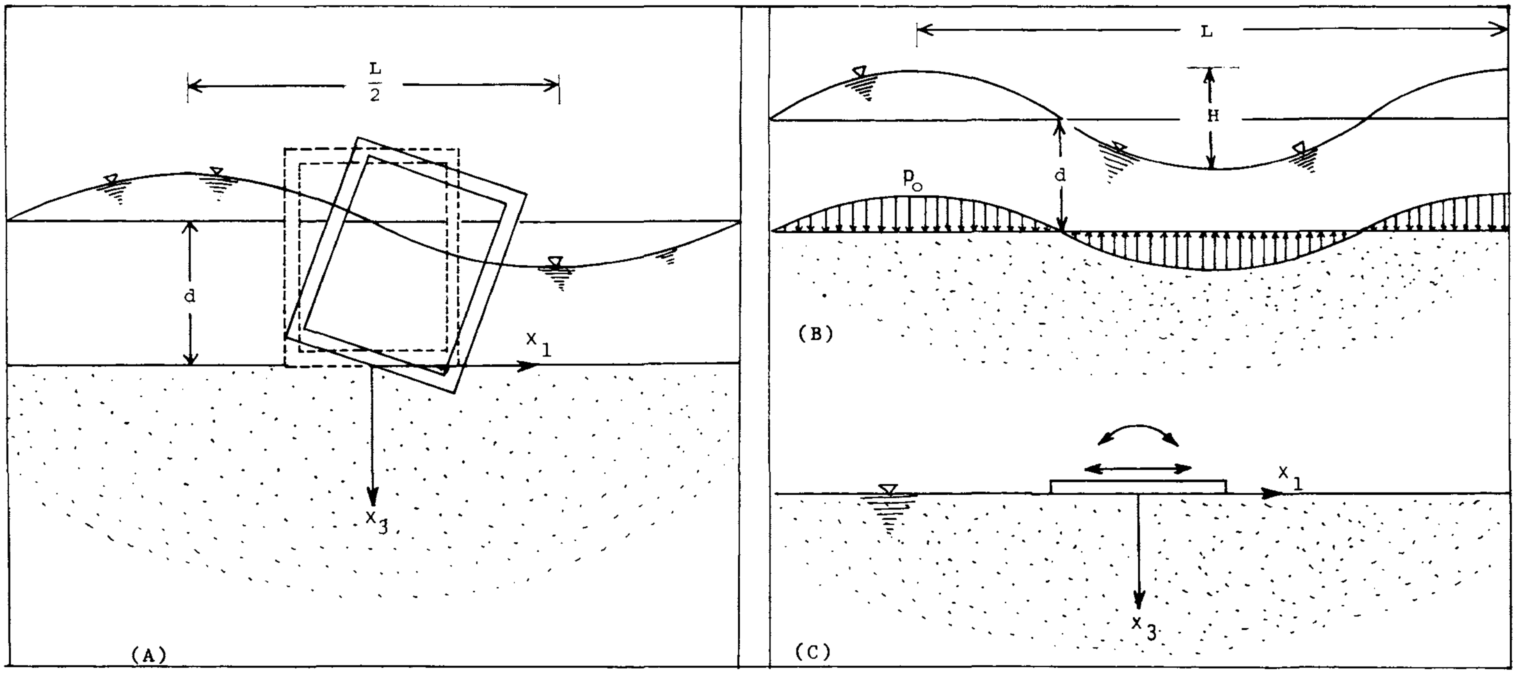

FIG. 1. (A) Offshore caisson during ocean wave storm; (B) and (C) The two key sub-problems.

the stiffness of the porous solid structure; G is chiefly a function of porosity, effective confining pressure and amplitude of shear strain.

(3) The Poisson's ratio ν_d, measured under conditions of complete drainage.

(4) The fluid compressibility C_w obtained from the change in fluid volume due to an increase in fluid pressure:

$$C_w = \frac{\bar{\varepsilon}}{dp_w} \tag{2}$$

in which $\bar{\varepsilon} = d\rho_w/\rho_w$ is the fluid volumetric strain ρ_w the mass density of the fluid and p_w the pore fluid pressure. C_w depends solely (for a given temperature and pressure) on the amount of air contained in the fluid. This amount is measured with the degree of saturation S of the porous medium. For water with S = 100%, $C_w \approx 4.9 \times 10^{-7}$ 1/kPa, while for S>98%

$$C_w(S) = C_w(100) + \frac{100 - S}{p_w} \tag{3}$$

(5) The soil permeability k (m/sec) relating the velocity of fluid flow to the existing gradient of fluid pressure.

(6) The mass densities ρ_s and ρ_w of the solid and fluid phases. The bulk mass density is given by

$$\rho = \rho_s(1 - n) + \rho_w n \tag{4}$$

DEFINITION OF KINEMATIC AND STRESS VARIABLES

The kinematic variables needed to describe the behavior of a poroelastic medium are the macroscopic (statistical average) displacements of the solid, u_i, and fluid, U_i, where the subscript i(i = 1,2) refers to the x_i axis (Fig.1). The volumetric strains of the solid and fluid phases are, respectively:

$$\varepsilon = \varepsilon_{ii} = \frac{\partial u_i}{\partial x_i} \qquad (i = 1,2) \qquad \bar{\varepsilon} = \bar{\varepsilon}_{ii} = \frac{\partial U_i}{\partial x_i} \tag{5}$$

where a repeated index denotes a summation with respect to that index over its range.

The stress variables used in the subsequent theory are not the conventional effective stress, σ'_{ij}, and pore fluid pressures, p_w, but the macroscopic components introduced by Biot (1941, 1956). Considering a perfectly planar unit cross-section passing through the grains of the porous medium (Fig. 2), the total transmitted force can be resolved into a force component acting on the solid phase and one acting on the fluid phase. Denoting by σ_{ij} and s the average values of the corresponding stresses (force/unit cross-sectional area), one can write for the total stress:

$$\sigma_{ij}^t = \sigma_{ij} + s\,\delta_{ij} \tag{6}$$

in which δ_{ij} is the Kronecker delta ($\delta_{ii} = 1$, $\delta_{ij} = 0$ if $i \neq j$) and possitive are the tensile stresses. The pore fluid pressure p_w is related to s through the porosity, n:

$$p_w = - s/n \tag{7}$$

with the minus sign indicating that p_w is positive if compressive. Recalling that

$$\sigma_{ij}^t = \sigma'_{ij} + (-p_w)\,\delta_{ij} \tag{8}$$

leads to the following relation between effective stresses and the macroscopic stresses of Biot's poroelastic theory:

$$\sigma'_{ij} = \sigma_{ij} - \frac{1-n}{n}\,s \tag{9}$$

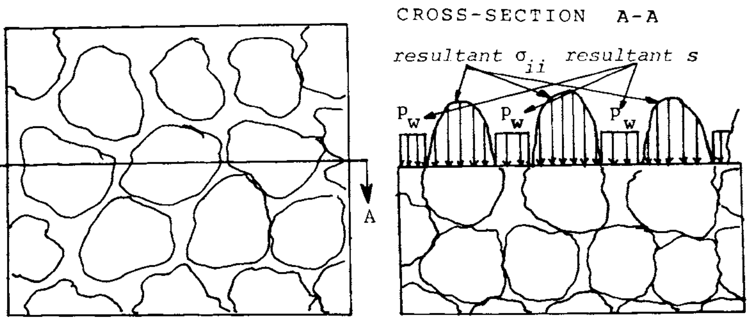

FIG. 2. Illustration of macroscopic stress components.

GOVERNING FIELD EQUATIONS

The constitutive relations between kinematic and stress variables of a poroelastic medium are (Biot, 1941, 1956; Ishihara, 1967, 1970; Richart et al., 1970):

$$\sigma_{ij} = (D\,\varepsilon + Q\,\bar{\varepsilon})\delta_{ij} + G\left(\frac{\partial u_1}{\partial x_2} + \frac{\partial u_2}{\partial x_1}\right) \tag{10a}$$

$$s = Q\,\varepsilon + R\,\bar{\varepsilon} \tag{10b}$$

in which the moduli D, Q and R are obtained from the previously introduced basic soil parameters as follows:

$$D = \frac{2\nu_d}{1 - 2\nu_d}\,G + \frac{Q^2}{R} \tag{11a}$$

$$Q \simeq \frac{1 - n}{C_w} \tag{11b}$$

$$R \simeq \frac{n}{C_w} \tag{11c}$$

in which the approximation refers to neglecting the compressibility of the solid grains C_s. (Typically $C_s \simeq C_w/20$) Eqs. 10 should be complemented with the equations of dynamic equilibrium. Assuming that the fluid flow through the solid structure obeys Darcy's Law:

$$\dot{u}_i - \dot{U}_i = \frac{k}{\gamma_w}\frac{\partial p_w}{\partial x_i} \tag{12}$$

where $\gamma_w = \rho_w g$ and the dot denotes derivative with respect to time, the dynamic equilibrium of the solid and fluid parts can be written:

$$\rho_{11}\,\ddot{u}_i - \rho_{12}\,\ddot{U}_i = \frac{\partial \sigma_{ij}}{\partial x_j} + b(\dot{u}_i - \dot{U}_i) \tag{13a}$$

$$-\rho_{12}\,\ddot{u}_i + \rho_{22}\,\ddot{U}_i = \frac{\partial s}{\partial x_i} - b(u_i - U_i) \tag{13b}$$

in which $b = n\gamma_w/k$ and the corresponding terms apparently represent the seepage forces; $\rho_{11} = \rho_s(1 - n) + \rho_{12}$, $\rho_{22} = n\rho_w + \rho_{12}$ and ρ_{12} density of an additional apparent mass which relates to the coupling between the fluid and the solid structures. By requiring that the strain energy in a poroelastic medium be non-negative for any possible combination of applied stresses, one arrives at the inequality $\rho_{12}^2 < \rho_{11}\rho_{22}$ (Biot, 1956). For the relatively small frequencies of ocean wave and earthquake induced loading, $\rho_{12} \simeq 0$*.

*The reader is cautioned at this point of a sign discrepancy involving the seepage terms of the equilibrium equations in Biot(1956) and Deseriewicz & Rice (1962).

Eqs. 10 and 13 govern the propagation of time varying (dynamic) disturbances in a poroelastic medium.

GENERAL SOLUTION

Eqs. 10 and 13 can be uncoupled into three wave equations in terms of the potential functions $\Phi_1(x,z)$, $\Phi_2(x,z)$ and $H(x,z)$ (Deresiewicz & Rice, 1962) related to the solid and fluid displacements vectors $\mathbf{u}(x,z)$ and $U(x,z)$ by

$$\mathbf{u} = \text{grad}(\Phi_1 + \Phi_2) + \text{curl } H \tag{14a}$$

$$\mathbf{U} = \text{grad}(\mu_1\Phi_1 + \mu_2\Phi_2) + \text{curl } H \tag{14b}$$

in which μ_1, μ_2 and μ are given in Appendix. Considering propagation of harmonic disturbances, only, the wave equations are:

$$(\nabla^2 + \omega^2/c_i)\,\Phi_i = 0, \quad i = 1,2 \quad \text{and} \quad (\nabla^2 + \omega^2/c_3^2)H = 0 \tag{15}$$

in which ω = the frequency of oscillation; ∇^2 = the Laplace operator; and c_1, c_2 and c_3 = the wave velocities of the two dilatational and the rotational wave, respectively. The velocities c_1 and c_2 (of the P_f and P_s waves) are obtained from the 'undrained' wave velocity c, i.e. the velocity that corresponds to zero fluid flow through the solid structure, from the relation

$$c_i^2 = \frac{c_p^2}{q_i} \quad , \; i = 1,2 \tag{16}$$

in which

$$c_p^2 = Z/\rho \tag{17}$$

where $Z = P + R + 2Q$, $P = D + 2G$, and q_i are the roots of the quadratic equation

$$(PR - Q^2)q^2 - \left(\rho_{11}R + \rho_{22}P + \frac{ib}{\omega}Z\right)q + \rho_{11}\rho_{22} + \frac{ib}{\omega}\rho = 0 \tag{18}$$

The shear wave velocity c_3 is related to the shear wave velocity c_s of the solid phase:

$$c_3^2 = \frac{c_s^2}{q_3} \tag{19}$$

in which

$$c_s^2 = G/\rho_{11} \quad \text{and} \quad q_3 = \frac{\rho_{11}}{\rho}\,\frac{\rho_{11}\,\rho_{22} + ib/\omega}{\rho_{22} - ib/\omega} \tag{20}$$

Calling $h_j = \omega/c_j$ the wave-number corresponding to the j-th velocity, the general harmonic (plane-strain) solution of Eqs. 15 can be cast in the form

$$\Phi_1 = \{A_1 \exp(-ih_1m_1x_3) + B_1\exp(ih_1m_1x_3)\}\cdot \exp[i(\omega t - h_1\ell_1x_1)] \tag{21a}$$

$$\Phi_2 = \{A_2\exp(-h_2m_2x_3) + B\exp(ih_2m_2x_1)\}\cdot \exp[i(\omega t - h_2\ell_2x_1)] \tag{21b}$$

$$H = \{A_3\exp(-ih_3m_3x_3) + B\exp(ih_3m_3x_3)\}\cdot \exp[i(\omega t - h_3\ell_3x_1)] \tag{21c}$$

with $m_j^2 + \ell_j^2 = 1$, $j = 1,2,3$, and $i = \sqrt{-1}$. Using Eqs. 21 and Eqs. 14 the displacement components of the solid and fluid phase can be obtained; the constitutive relations (Eqs. 10) then yield the (macroscopic) stresses in the two phases. Realizing that there are six unknown constants of integration (A_j, B_j, $j = 1,2,3$), six equations must be derived from the boundary conditions of the problem.

BOUNDARY VALUE PROBLEMS: HALFSPACE

In a poroelastic halfspace (idealizing a very deep soil deposit) stresses and displacements must vanish as $z \to \infty$. Thus $B_j = 0$, $j = 1,2,3$, and the expressions for stresses and displacements reduce to:

$$\sigma_{33}^t(x_1, x_3) = -\{h_1^2 [D + Q + (R + Q) \mu_1 + 2G m_1^2] \exp(-ih_1 m_1 x_3) A_1$$
$$+h_2^2 [D + Q + (R + Q)\mu_2 + 2G m_2^2] \exp(-ih_2 m_2 x_3) A_2$$
$$+2h_3^2 m_3 \ell_3 \exp(-ih_3 m_3 x_3) A_3\} \cdot \exp[-i(\omega t - h\ell x_1)] \quad (22a)$$

$$\sigma_{13}(x_1, x_3) = - \{2Gh_1^2 m_1 \ell_1 \exp(-ih_1 m_1 x_3) A_1 + 2Gh_2^2 m_2 \ell_2 \exp(-ih_2 m_2 x_3) A_2$$
$$-Gh_3^2 (m_3^2 - \ell_3^2) \exp(-ih_3 m_3 x_3) A_3\} \cdot \exp[-i(\omega t - h\ell x_1)] \quad (22b)$$

$$p_w(x_1, x_3) = \{h_1^2 (Q + R\mu_1) \exp(-ih_1 m_1 x_3) A_1$$
$$+h_2^2 (Q + R\mu_2) \exp(-ih_2 m_2 x_3) A_2\} \cdot \exp[-i(\omega t - h\ell x_1)] \quad (22c)$$

$$u_1(x_1, x_3) = -i\{h_1 \ell_1 \exp(-ih_1 m_1 x_3) A_1 + h_2 \ell_2 \exp(-ih_2 m_2 x_3) A_2$$
$$-h_3 m_3 \exp(-ih_3 m_3 x_3) A_3\} \cdot \exp[-i(\omega t - h\ell x_1)] \quad (22d)$$

$$u_3(x_1, x_3) = - i\{h_1 m_1 \exp(-h_1 m_1 x_3) A_1 + h_2 m_2 \exp(-ih_2 m_2 x) A_2$$
$$+ h_3 \ell_3 \exp(-ih_3 m_3 x_3) A_3\} \cdot \exp[-i(\omega t - h \ell x_1)] \quad (22e)$$

in which, since the boundary is horizontal (i.e. parallel to x_1), all variables depend on x_1 and t in the form given by the function $\exp[i(h\ell x_1 - \omega t)]$ where $h\ell = h_j \ell_j$, $j = 1,2,3$. The boundary conditions needed to evaluate A_1, A_2 and A_3 depend on the nature of the excitation

(a) OCEAN WAVE LOADING (FIG. 1b)

At the surface of the porous soil, $x_3 = 0$, the sea wave motion imposes that the fluid pore pressure, p_w. be equal to

$$p_w = p_o \exp[i(kx_1 - \omega t)] \quad , k = \frac{2\pi}{L} \quad (23)$$

where the amplitude p_o is given by Eq. 1 and the ocean wave length L is shown in Fig. 1. In addition the vertical effective normal stress must vanish $\sigma'_{33} = 0$. In relatively shallow waters (i.e. with d small), shear stresses are imposed from the oscillating sea water on the soil. Since they are usually small and decrease fast as d increases, they can safely be neglected (Madsen, 1978; Yamamoto et al, 1978). Thus, besides Eq. 23, the following two boundary conditions apply:

$$\sigma_{33}^t = - p_w \, , \quad \sigma_{13} = 0 \; ; \text{ at } x_3 = 0 \quad (24)$$

Introducing Eqs. 22a, 22b and 22c in Eqs. 23, 24 leads to a system of three linear albegraic equations from which A_1, A_2 and A_3 are readily evaluated.

(b) RIGID STRIP LOADING (FIG. 1c)

The boundary conditions in this case are of a mixed nature, i.e., while zero normal and shear stresses are sustained at all points beyond the loading area, rigid body horizontal and rotational displacements are imposed on the soil-strip interface. To overcome this difficulty, a formulation in terms of Green's function for the halfspace is used. Note that a similar approach was followed for a circular rigid disk on poroelastic halfspace by Halpern & Christiano, 1979, who performed a numerical evaluation of the resulting zero-to-infinity type integrals. Herein a numerical technique has been used, similar to the one described by Gazetas & Roesset (1976) and Gazetas (1980) for a homogeneous and a linearly heterogeneous elastic semi-infinite continuum, respectively. Only a bare outline of the method is offered here.

Due to the two-dimensional nature of the problem, the whole surface is discretized into a number of uniformly spaced nodal points. The dynamic displacements ('flexibility influence coefficients') of the nodal points are obtained from the solution of two simple boundary value problems associated with harmonically time-varying normal or shear stress pulses, uniformly distributed around a nodal point. Each of these pulses can be expanded in Fourier series along the horizontal axis, x_1, and for each harmonic $\sigma(\xi)\exp(i\xi x_1)$ the boundary conditions can be expressed in terms of stresses only. For example, considering an imposed normal stress, the boundary conditions are:

$$\sigma_{33}^t = \sigma(\xi) \exp(i\xi x_1); \quad (25a)$$

$$\sigma_{13} = p_w = 0; \quad \text{at } x_3 = 0 \quad (25b)$$

provided that the soil-strip inrerface is freely draining.

Having obtained stresses and displacements for each harmonic, a discrete inverse Fast Fourier Transform algorithm is employed to evaluate horizontal and vertical displacements at each nodal point (dynamic influence coefficients). It is then a simple problem to impose the conditions of rigid body motion to the nodal points at the soil-caisson interface and thus compute the dynamic compliances in horizontal and rocking motion. In matrix form

$$\begin{Bmatrix} \delta_H \\ \phi_o B \end{Bmatrix} = \frac{1}{G} \begin{bmatrix} S_{HH} & S_{HM} \\ S_{HM} & S_{MM} \end{bmatrix} \begin{Bmatrix} F_H \\ \frac{M_o}{B} \end{Bmatrix} \quad (26)$$

where δ_H and ϕ_o are the amplitudes of horizontal displacement and rotation of the rigid strip due to a harmonic force and moment of amplitude F_H and M_o. Each of the normalized compliances is a function of frequency can be written in the form:

$$S(\omega) = S'(\omega) \cos\omega t + S''(\omega) \sin\omega t \quad (27)$$

in which $S'(\omega)$ and $S''(\omega)$ are the amplitudes of the in-phase and $90°$-out-of-phase components of motion.

NUMERICAL RESULTS AND CONCLUSIONS

Preliminary conclusions are offered here regarding the influence of some key soil parameters on the dynamic response of a rigid strip resting on a water-saturated and a nearly saturated poroelastic halfspace. Fig. 3 portrays the in-phase and $90°$-out-of phase components of the normalized swaying and rocking compliances, S_{HH} and S_{MM}, as functions of the dimensionless frequency factor $a = \omega B\sqrt{\rho/G}$ where 2B is the width of the rigid strip and ρ the bulk density of the porous medium (Eq. 4). The following soil parameters, approximate for a dense and quite permeable sand, were used as input:

porosity: $n = 0.30$

shear modulus: $G = 10^5$ kPa

drained Poisson's ratio: $\nu_d = 0.25$

coefficient of permeability: $k = 1$ cm/sec

mass density of solid: $\rho_s = 2700$ kg/m^3

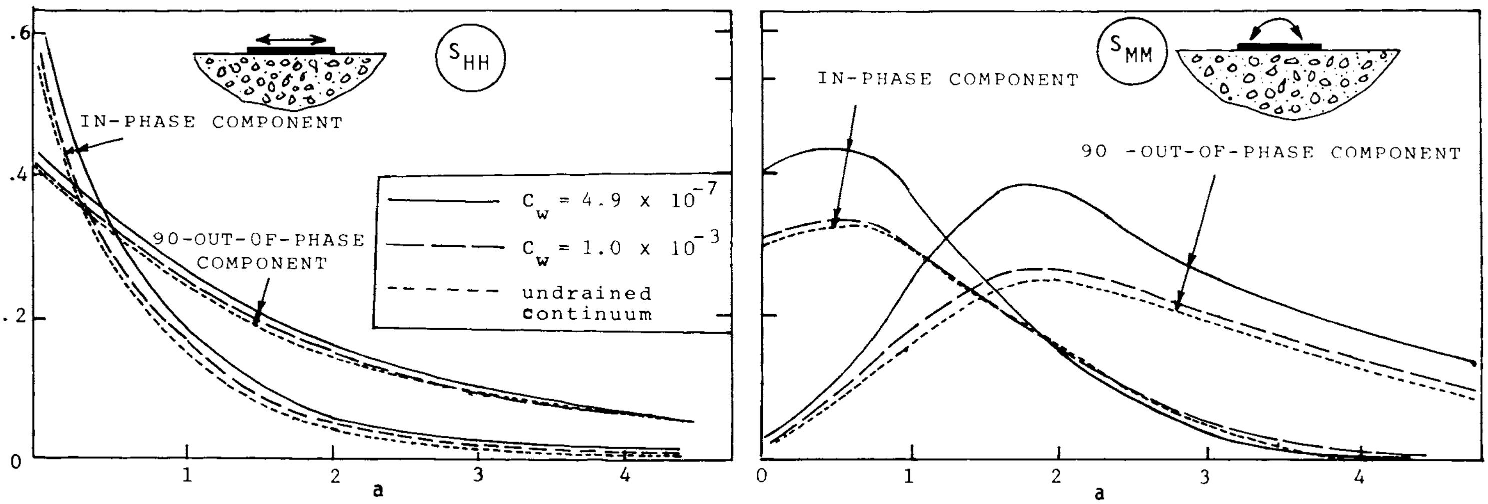

FIG. 3. Effect of fluid compressibility on swaying and rocking compliance functions.

mass density of fluid: $\rho_w = 1000\ kg/m^3$

while the fluid compressibility, C_w, varied from the fully saturated value of $4.9 \times 10^{-7} m^2/kN$ to $10^{-3} m^2/kN$ which corresponds approximately to $S \simeq 99.8\%$ for water depth of ≈ 20 meters. Also plotted in this figure (dotted lines) are the curves for an undrained halfspace continum having the same G and ρ with the poroelastic medium and $\nu = 0.495$.

The following are evident from the figure:

(1) the fluid compressibility has a small overall effect on swaying oscillations. The effect is noticeable primarily in the relatively low frequency range. The saturated porous medium exhibits horizontal displacements only slightly different from those of an undrained continuum, presumably because there is very little fluid flow through the porous solid. This may appear somewhat surprising, given the high permeability of the medium, at least for the very low frequency range. As Ishihara noticed, however, the large wave lengths at such frequencies imply that the generated pore-water pressure gradients are very small to produce any flow.

(2) Rocking oscillations are strongly influenced by the fluid compressibility, which is understandable in view of the mostly dilatational deformations imposed by the footing. As C_w decreases the medium becomes more compressible and displacements increase. Again, the fully saturated medium exhibits a response very similar to the one of the undrained continuum, indirectly confirming the small fluid flow during the motion.

Parametric analyses are currently conducted at Case Western Reserve University to evaluate the effect of the other soil parameters on the response of poroelastic media to ocean wave or strip loading, while the formulation is extended to multi-layered soil deposits.

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APPENDIX

$$\mu_j = \frac{g - ieb/\rho\omega - q_j}{h - ieb/\rho\omega}$$

$$\mu = \frac{\rho_{12} - ieb/\omega}{\rho_{22} + ieb/\omega}$$