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DYNAMIC SOIL-FOUNDATION INTERACTION IN OCEAN ENVIRONMENT

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ABSTRACT

A thin layer element method is formulated for Biot's equation describing the dynamic behavior of fluid-filled elasto-porous medium. Using the formulation, the dynamic response of a foundation deeply embedded in the seafloor is analyzed. The ocean environment is characterized by the fluid above and inside the seafloor sediments and its effects on the dynamic response of the foundation are examined. It is found that the ocean environment can considerably affect the dynamic response of the sediment deposits and foundation.

KEY WORDS: Foundation, dynamic response, seafloor sediments

INTRODUCTION

Biot (1962) has made a framework in the formulation of dynamic response of fluid-filled elasto-porous medium. This formulation has been generally used for dynamic response analysis of submerged soil and evaluated typically by either analytical solutions obtained by solving the differential equations or the numerical finite element method. Considerable difficulty exists in obtaining analytical solutions for Biot's equation in general and thus the solutions have been developed only for very simple conditions (e.g. Biot, 1956; Jones, 1961; Deresiewics, 1960; Foda and Mei, 1982). Those conditions are generally too simple compared with those commonly encountered in the real situation. The finite element method has been applied for the numerical evaluation of Biot's equation (e.g. Ghaboussi and Wilson, 1973; Prevost, 1982; Simon et al., 1986; Zienkiewicz et al., 1977). Contrary to the former approach, this approach can account for complex geometry and inhomogeneity without increasing the degree of difficulty and amount of computation. However, compared with the finite element scheme applied to a single-phase medium, the computation effort increases substantially due to the additional degrees of freedom associated with pore fluid. Various people (e.g. Kausel and Roesset , 1975; Lysmer and Waas, 1972; and Tajimi and Shimomura, 1976) have presented a thin layer element method, which combines the finite element scheme and analytical solution and uses the Rayleigh wave modes in the expression of the responses. This approach requires computation effort far less than the regular finite element approach and yet has a capacity of accommodating complex conditions far more than the approach with the analytical solution. It has been applied to the dynamic response computation of a single-phase medium but does not appear to have been applied to a fluid-saturated porous medium in published literature yet. For the dynamic response analysis of a two-phase mixture, this approach appears to be very attractive because a large computation effort is generally required in such analysis by the regular finite element method. This paper presents the thin layer element formulation for a fluid-saturated porous medium and applies it to the analysis of dynamic soil-foundation interaction in the ocean environment, characterized by the fluid above and inside the seafloor sediments.

FORMULATION

A soil medium is assumed to be an elastic porous medium saturated with pore fluid. The average displacement of the pore fluid relative to the displacement of the solid skeleton is defined as

$$\mathbf{w} = n(\mathbf{U} - \mathbf{u}) \tag{1}$$

where n = porosity; $\mathbf{w} = (w_x, w_z)^T$ in which w_j is relative displacement of fluid in the j direction; $\mathbf{u} = (u_x, u_z)^T$ in which u_j is displacements of solid skeleton in the j direction; $\mathbf{U} = (U_x, U_z)^T$ in which U_j is the absolute displacement of the fluid in the j direction; and x and z = Cartesian coordinates in horizontal and vertical directions, respectively. The total normal stresses acting on a unit area of mixture is

$$\sigma = (1-n)\sigma_s + \mathbf{m}n\pi = \sigma' + \mathbf{m}\pi \tag{2}$$

where $\sigma_s = (\sigma_{sx}, \sigma_{sz}, \tau_{xz})^T$ in which σ_{sj} is a normal stress in the j direction acting on the solid skeleton over the unit area; $\sigma = (\sigma_x, \sigma_y, \tau_{xz})^T$ in which σ_j is a total normal stress in the j direction; $\sigma' = (\sigma_x', \sigma_z', \tau_{xz})^T$ in which σ_j' is an effective normal stress in the j direction; τ_{xz} = shear stress; π = pore fluid pressure; and $\mathbf{m} = (1,1,0)^T$.

The equilibrium condition of forces acting on the soil skeleton in a unit soil volume is described as

$$(1-n)\mathbf{L}^T\sigma_s + (1-n)\rho_s\mathbf{b} + nk^{-1}\dot{\mathbf{w}} = (1-n)\rho_s\ddot{\mathbf{u}} \tag{3}$$

where $\dot{\mathbf{w}} = \partial\mathbf{w}/\partial t$; $\ddot{\mathbf{u}} = \partial^2\mathbf{u}/\partial t^2$; $\mathbf{b} = (b_x, b_z)^T$ in which b_j is body force in the j direction per unit mass; ρ_s = density of a unit volume of the solid material in the skeleton; and

$$\mathbf{L}^T = \begin{bmatrix} \frac{\partial}{\partial x} & 0 & \frac{\partial}{\partial z} \\ 0 & \frac{\partial}{\partial z} & \frac{\partial}{\partial x} \end{bmatrix} \tag{4}$$

The equilibrium condition of the forces acting on the pore fluid domain in a unit volume of soil is given by

$$n\nabla\pi - k^{-1}n\dot{\mathbf{w}} + n\rho_f\mathbf{b} = \rho_f\ddot{\mathbf{w}} + n\rho_f\ddot{\mathbf{u}} \tag{5}$$

where ρ_f = density of unit volume of pore fluid; and $\nabla = (\partial/\partial x, \partial/\partial z)^T$. Combining Eqs. 3 and 5, the equilibrium condition of the pore fluid and solid skeleton mixture is expressed as

$$\mathbf{L}^T\sigma + \rho\mathbf{b} = \rho\ddot{\mathbf{u}} + \rho_f\ddot{\mathbf{w}} \tag{6}$$

where ρ = density of unit volume of mixture = $(1-n)\rho_s + n\rho_f$. Since linear elastic conditions are considered, body forces will be neglected hereafter.

The fluid stored in a unit volume mixture is equal to the summation of the compression of the solid grain by all round fluid pressure, the compression of the solid frame by the effective stresses, and the compression of the fluid. Therefor, according to Simon et al. (1984), the rate of the fluid stored is expressed as

$$\nabla^T\mathbf{w} = -\alpha\mathbf{m}^T\varepsilon + Q^{-1}\pi \tag{7}$$

where ε = strains = $(\varepsilon_x, \varepsilon_z, \gamma_{xz})^T$; and α and Q are related with material properties through

$$\alpha = 1 - \frac{K_d}{K_s} \qquad \text{and} \qquad Q^{-1} = \frac{1}{K_f} + \frac{\alpha - n}{K_s} \tag{8}$$

where K_s = elastic volumetric modulus of solid; K_f = volumetric modulus of fluid; and K_d = elastic volumetric modulus of solid skeleton. Substituting π in Eq. 7 into Eq. 2 and using the stress-strain relationship, $\sigma' = \mathbf{D}\varepsilon$, the total stresses, σ, can be correlated with ε and $\mathbf{w}$. With $\varepsilon = \mathbf{Lu}$, this expression and pore fluid pressure given in Eq. 7 can be written in a matrix form such that

$$\begin{Bmatrix}\sigma \\ \pi\end{Bmatrix} = \begin{bmatrix} (\mathbf{D} + \alpha^2 Q\mathbf{mm}^T)\mathbf{L} & \alpha Q\mathbf{m}\nabla^T \\ \alpha Q\mathbf{m}^T\mathbf{L} & Q\nabla^T \end{bmatrix}\begin{Bmatrix}\mathbf{u} \\ \mathbf{w}\end{Bmatrix} \tag{9}$$

Using π and σ defined in Eq. 9, Eqs. 5 and 6 can be rewritten in the following matrix form after using the relationship $\varepsilon = \mathbf{Lu}$:

$$-\mathbf{M}\begin{Bmatrix}\ddot{\mathbf{u}} \\ \ddot{\mathbf{w}}\end{Bmatrix} - \mathbf{C}\begin{Bmatrix}\dot{\mathbf{u}} \\ \dot{\mathbf{w}}\end{Bmatrix} + \mathbf{K}\begin{Bmatrix}\mathbf{u} \\ \mathbf{w}\end{Bmatrix} = \begin{Bmatrix}0 \\ 0\end{Bmatrix} \tag{10}$$

where

$$\mathbf{M} = \begin{bmatrix} \rho & \rho_f \\ \rho_f & \rho_f/n \end{bmatrix} \qquad \mathbf{C} = \begin{bmatrix} 0 & 0 \\ 0 & k^{-1} \end{bmatrix}$$

$$\mathbf{K} = \begin{bmatrix} \mathbf{L}^T(\mathbf{D} + \alpha^2 Q\mathbf{mm}^T)\mathbf{L} & \alpha Q\mathbf{L}^T\mathbf{m}\nabla^T \\ \alpha Q\nabla\mathbf{m}^T\mathbf{L} & Q\nabla\nabla^T \end{bmatrix} \tag{11}$$

It is noted that , when the layer is made of fluid only, Eq. 10 and all other formulations can be rewritten with $\mathbf{u} = \mathbf{0}$, $k = \infty$, $n = 1$ and $Q = 1/K_f$. The undrained condition corresponds to $\mathbf{w} = 0$ and $k = 0$ in the above formulations.

Consider a horizontally layered submerged soil. The displacements of the medium in the wave field is expressed in the form of $(\mathbf{u}(x,z,t), \mathbf{w}(x,z,t)) = (\mathbf{u}(x), \mathbf{w}(z))e^{i(\omega t - hx)}$ in which ω = circular frequency and h = wave number. Using a shape function in the z direction and omitting the time factor, the displacements of the jth layer in the wave field are approximately expressed by using the displacements at the upper and lower ends of the jth layer as

$$\begin{Bmatrix}\mathbf{u}_j(x,z) \\ \mathbf{w}_j(x,z)\end{Bmatrix} = e^{-ihx}\ \mathbf{Z}(z)\begin{Bmatrix}\mathbf{U}_j \\ \mathbf{W}_j\end{Bmatrix} \tag{12}$$

where $e^{-ihx}\ \mathbf{U}_j{}^T = (u_{xj}(x,0)^T, u_{xj}(x,H_j)^T, u_{zj}(x,0)^T, u_{zj}(x,H_j)^T)$ and $e^{-ihx}\mathbf{W}_j{}^T = (w_{xj}(x,0)^T, w_{xj}(x,H_j)^T, w_{zj}(x,0)^T, w_{zj}(x,H_j)^T)$, in which z=0 and H_j indicate respectively the upper end and lower ends of the jth layer; ; and $\mathbf{Z}(z)$ = matrix containing the shape function . When linear variations of the displacements are assumed along z, the shape function matrix $\mathbf{Z}$ is

$$\mathbf{Z}(z) = \begin{bmatrix} \mathbf{a}(z)_j & \mathbf{0} \\ \mathbf{0} & \mathbf{a}(z)_j \end{bmatrix} \tag{13}$$

where $\mathbf{0}$ = matrix containing zeros of size 2 rows and 4 columns; and

$$\mathbf{a}(z)_j = \begin{bmatrix} 1-z/H_j & z/H_j & 0 & 0 \\ 0 & 0 & 1-z/H_j & z/H_j \end{bmatrix}$$

After substituting Eq. 12 into Eq. 10, application of Galarkin's procedure to Eq. 10 results in

$$\sum_{j=1}^{J}\int_V \mathbf{Z}^T\left(\mathbf{K}_j - i\omega\mathbf{C}_j + \omega^2\mathbf{M}_j\right)\mathbf{Z}\begin{Bmatrix}\mathbf{U}_j \\ \mathbf{W}_j\end{Bmatrix}e^{-ihx}dV - \sum_{j=1}^{J}\int_S \mathbf{Z}^T\mathbf{K}'_j\mathbf{Z}\begin{Bmatrix}\mathbf{U}_j \\ \mathbf{W}_j\end{Bmatrix}e^{-ihx}dS = 0 \tag{14}$$

where J = numbers of layers; V = volume of the layer j; S = surface area of the layer j; and $\mathbf{K}' = \int\mathbf{K}dz$. Transforming the surface integration in Eq. 14 into the volume integration and integrating over x result in

$$\sum_{j=1}^{J}\int_0^{H_j}\left\{\mathbf{Z}^T\left(\mathbf{K}_j - i\omega\mathbf{C}_j + \omega^2\mathbf{M}_j - \frac{\partial\mathbf{K}'_j}{\partial z}\right) - \frac{\partial\mathbf{Z}^T}{\partial z}\mathbf{K}'_j\right\}\mathbf{Z}\begin{Bmatrix}\mathbf{U}_j \\ \mathbf{W}_j\end{Bmatrix}dz = 0 \tag{15}$$

Performing integration with respect to z in Eq. 15 result in the characteristics equation in the discretized form such that

$$\sum_{j=1}^{J}\left(h^2\alpha_j + ih\beta_j + \gamma_j\right)\begin{Bmatrix}\mathbf{U}_j \\ \mathbf{W}_j\end{Bmatrix} = \mathbf{0}$$

or

$$\left(h^2\alpha + ih\beta + \gamma\right)\begin{Bmatrix}\mathbf{U} \\ \mathbf{W}\end{Bmatrix} = \mathbf{0} \tag{16}$$

where

$$\alpha_j = \frac{H_j}{6}\begin{bmatrix} Aa & 0a & \alpha Qa & 0a \\ 0a & Ga & 0a & 0a \\ \alpha Qa & 0a & Qa & 0a \\ 0a & 0a & 0a & 0a \end{bmatrix} \; ; \; \beta_j = \frac{1}{2}\begin{bmatrix} 0b & (A\text{-}2G)b & 0b & \alpha Qb \\ Gb & 0b & 0b & 0b \\ 0b & \alpha Qb & 0b & Qb \\ 0b & 0b & 0b & 0b \end{bmatrix}^T -$$

$$\frac{1}{2}\begin{bmatrix} 0b & (A\text{-}2G)b & 0b & \alpha Qb \\ Gb & 0b & 0b & 0b \\ 0b & \alpha Qb & 0b & Qb \\ 0b & 0b & 0b & 0b \end{bmatrix} \; ; \; \gamma_j = \frac{1}{H_j}\begin{bmatrix} Gc & 0c & 0c & 0c \\ 0c & Ac & 0c & \alpha Qc \\ 0c & 0c & 0c & 0c \\ 0c & \alpha Qc & 0c & Qc \end{bmatrix} +$$

$$i\omega\frac{H_j}{6}\begin{bmatrix} 0a & 0a & 0a & 0a \\ 0a & 0a & 0a & 0a \\ 0a & 0a & k^{-1}a & 0a \\ 0a & 0a & 0a & k^{-1}a \end{bmatrix} - \omega^2\frac{H_j}{6}\begin{bmatrix} \rho a & 0a & \rho_f a & 0a \\ 0a & \rho a & 0a & \rho_f a \\ \rho_f a & 0a & \rho_f/na & 0a \\ 0a & \rho_f a & 0a & \rho_f/na \end{bmatrix} \tag{17}$$

in which $A = \lambda + 2G + \alpha^2 Q$; and

$$a = \begin{bmatrix} 2 & 1 \\ 1 & 2 \end{bmatrix} \quad b = \begin{bmatrix} 1 & -1 \\ 1 & -1 \end{bmatrix} \quad c = \begin{bmatrix} 1 & -1 \\ -1 & 1 \end{bmatrix} \tag{18}$$

Wave numbers, h, and their associated mode shape vectors, $(\phi_u^T, \phi_w^T)^T$, can be determined by solving the characteristic equation Eq. 16. Since the fluid pressure is an all around equal pressure, there is a constraint between the freedoms associated with w_x and w_z and thus the total degree of freedoms for J layers is 3J instead of 4J. This results in 3J non-zero conjugate pairs and J zero conjugate pairs in eigenvalues computed from Eq. 16. In order to satisfy the wave scattering conditions, only those with the minus sign in the imaginary part are selected among the conjugate pairs. Then, the displacements of the submerged layered soil along x is expressed at the nodal points as

$$\begin{Bmatrix} U(x) \\ W(x) \end{Bmatrix} = \sum_{s=1}^{3J} e^{-ih_s x}\alpha_s \begin{Bmatrix} \phi_u \\ \phi_w \end{Bmatrix}_s \tag{19}$$

and thus the displacements within the jth layer as

$$\begin{Bmatrix} u_j(x,z) \\ w_j(x,z) \end{Bmatrix} = Z(z)\sum_{s=1}^{3J} e^{-ih_s x}\,\alpha_s \begin{Bmatrix} \phi_{uj} \\ \phi_{wj} \end{Bmatrix}_s \tag{20}$$

where $(\phi_u^T, \phi_w^T)_s^T$ = sth eigenvector in which ϕ_u and ϕ_w = vectors of size 2J ; ϕ_{uj} and ϕ_{wj} = vectors containing the values at the locations corresponding the jth layer in ϕ_u and ϕ_w, respectively; h_s = sth eigenvalue; and α_s = sth mode participation factor.

A vertical cut is considered at x=0 in a layered soil. The pressures acting along the surface of the cut are σ_x, τ_{xz} and π, and those distributed pressures at the jth layer can be replaced with nodal forces acting at the top and bottom of the layer through

$$\begin{Bmatrix} P_{xj} \\ P_{zj} \\ P_{\pi j} \end{Bmatrix} = -\int_0^{H_j} Z^T \begin{Bmatrix} \sigma_{xj}(0,z) \\ \tau_{xzj}(0,z) \\ \pi_j(0,z) \end{Bmatrix} dz \tag{21}$$

where P_{xj}, P_{zj} and $P_{\pi j}$ are nodal force vectors of the jth layer and each of them contains the forces at the top and bottom. After substituting Eqs. 9, 20 and 13 into Eq. 21, the nodal forces acting on the vertical cut of the jth layer for x≥0 can be obtained as

$$\begin{Bmatrix} P_j \\ P_{\pi j} \end{Bmatrix} = i\,E_j\sum_{s=1}^{3J} h_s\alpha_s \begin{Bmatrix} \phi_{uj} \\ \phi_{wxj} \end{Bmatrix}_s + F_j\sum_{s=1}^{3J}\alpha_s \begin{Bmatrix} \phi_{uj} \\ \phi_{wzj} \end{Bmatrix}_s \tag{22}$$

where $P_j^T = (P_{xj}^T, P_{zj}^T)$; $\phi_{wj}^T = (\phi_{wxj}^T, \phi_{wzj}^T)$ and

$$E_j = \frac{H_j}{6}\begin{bmatrix} Aa & 0a & \alpha Qa \\ 0a & Ga & 0a \\ \alpha Qa & 0a & Qa \end{bmatrix} \quad \text{with} \quad a = \begin{bmatrix} 2 & 1 \\ 1 & 2 \end{bmatrix}$$

$$F_j = \frac{1}{2}\begin{bmatrix} 0b & (A\text{-}2G)b & \alpha Qb \\ Gb & 0b & 0b \\ 0b & \alpha b & Qb \end{bmatrix} \quad \text{with} \quad b = \begin{bmatrix} 1 & -1 \\ 1 & -1 \end{bmatrix} \tag{23}$$

Therefore those of the entire layered system at x=0 for the medium extending -∞≤x≤∞ are

$$\begin{Bmatrix} P \\ P_\pi \end{Bmatrix} = i2E\sum_{s=1}^{3J} h_s\alpha_s \begin{Bmatrix} \phi_u \\ \phi_{wx} \end{Bmatrix}_s + 2F\sum_{s=1}^{3J}\alpha_s \begin{Bmatrix} \phi_u \\ \phi_{wz} \end{Bmatrix}_s \tag{24}$$

Eq. 19 at x=0 can be rewritten as

$$\begin{Bmatrix} U \\ W_x \\ \hline W_z \end{Bmatrix} = \begin{bmatrix} \gamma_a \\ \hline \gamma_b \end{bmatrix} \{\alpha\}$$

or

$$\begin{Bmatrix} U \\ W_x \end{Bmatrix} = [\gamma_a]\{\alpha\} \quad \text{and} \quad \{W_z\} = [\gamma_b]\{\alpha\} \tag{25}$$

where $W^T = (W_x^T, W_z^T)$; and $\{\alpha\}$ = vector contain α_s at the sth location. Similarly, Eq. 24 can be rewritten as

$$\begin{Bmatrix} P \\ P_\pi \end{Bmatrix} = [\delta]\{\alpha\} \tag{26}$$

Eliminating $\{\alpha\}$, Eqs. 25 and 26 result in

$$\begin{Bmatrix} P \\ P_\pi \end{Bmatrix} = \begin{bmatrix} S_u & S_{uw} \\ S_{uw} & S_w \end{bmatrix} \begin{Bmatrix} U \\ W_x \end{Bmatrix} \tag{27}$$

where

$$\begin{bmatrix} S_u & S_{uw} \\ S_{uw} & S_w \end{bmatrix} = [\delta][\gamma_a]^{-1} \tag{28}$$

When a vertical long structure is located along x=0 and it is assumed to be impervious at the soil-structure interface (i.g. W_x=0), Eq. 27 results in

$$P = S_u\, U \tag{29}$$

S_u is the stiffness matrix of a fluid-saturated soil. The stiffness matrix of the soil-foundation system can be obtained by superimposing this matrix into the foundation stiffness matrix.

Using Eqs. 9 and 20 together with Eq. 13, the stresses and pore fluid pressure at the middle of the jth layer are

$$\begin{Bmatrix} \sigma_j(x, 0.5H_j) \\ \pi_j(x, 0.5H_j) \end{Bmatrix} = -\,i\,A_j\sum_{s=1}^{3J} h_s e^{-ih_s x}\alpha_s \begin{Bmatrix} \phi_{uj} \\ \phi_{wj} \end{Bmatrix}_s + B_j\sum_{s=1}^{3J} e^{-ih_s x}\alpha_s \begin{Bmatrix} \phi_{uj} \\ \phi_{wj} \end{Bmatrix}_s \tag{30}$$

where

$$A_j = \frac{1}{2}\begin{bmatrix} A & A & 0 & 0 & \alpha Q & \alpha Q & 0 & 0 \\ A\text{-}2G & A\text{-}2G & 0 & 0 & \alpha Q & \alpha Q & 0 & 0 \\ 0 & 0 & G & G & 0 & 0 & 0 & 0 \\ \alpha Q & \alpha Q & 0 & 0 & Q & Q & 0 & 0 \end{bmatrix}$$

$$B_j = \frac{1}{H_j}\begin{bmatrix} 0 & 0 & -(A\text{-}2G) & (A\text{-}2G) & 0 & 0 & -\alpha Q & \alpha Q \\ 0 & 0 & -A & A & 0 & 0 & -\alpha Q & \alpha Q \\ -G & G & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & -\alpha Q & \alpha Q & 0 & 0 & -Q & Q \end{bmatrix} \quad (31)$$

COMPUTED RESULTS AND REMARKS

A 10 m deep homogeneous horizontal sediment stratum is assumed to lie under the ocean water. Water depths 2m, 10m and 20m are considered. Physical properties of the sediments are $K_s = 3.7 \times 10^6$ tf/m^2, $K_f = 2.08 \times 10^5$ tf/m^2, $\nu = 0.25$, n (porosity) = 0.375, γ (unit weight) = 2.6 tf/m^3, and D (material damping) = 2 %. A massless vertical foundation is assumed to penetrate through the sediments and its head is located at the mudline (Fig. 1). Thus the motions of the water above the seafloor do not directly act on the foundation. A harmonic excitation is applied in the horizontal direction at the foundation head.

First, a single layer is used for each of the water and sediments for simplicity. Only four wave numbers and their associated wave modes exist in this case. Fig. 2 shows the wave dispersion curves computed from the characteristic equation. The real and imaginary parts of the wave number are related to respectively the wave length and the decay rate of the motion with x: as those values increase, the wave length is shorter and the decay is faster. The curves B and C for 2m depth are very similar to those for no water above the sediments and curve A is split into the curves A and D when 2m water exists. A horizontal excitation of frequency $\omega H/v_s = 2$ (where H = total thichness of the sediments) is applied at the head of the stiffnessless foundation. The complex displacement amplitudes along the mudline, induced by the lateral foundation response, are computed using the above computed wave numbers and mode shapes and are shown in Fig. 3. As is seen, the difference in the water depth affects the sediment response.

Refined analyses are performed by dividing the water and sediments into 10 layers as shown in Fig. 4. Flexural stiffness, EI = 2×10^4 tf·m^2, is considered for the foundation. The soil-foundation stiffnesses are computed by dividing the applied force by the displacement at the head. Fig. 5 shows the variations of foundation stiffnesses with frequency for various water depths: the stiffnesses are normalized by the static soil-foundation stiffness for dry sediments. The real part decreases as frequency increases and reaches to the minimum at the fundamental resonance of the system ($\omega H/v_s \cong \pi/2$ for no water above the sediments). As the water depth increases the fundamental resonant frequency decreases in Fig. 5. Even though no noticeable differences are observed under the static conditions (ω=0), the differences under dynamic conditions are significant among the stiffnesses for three different water depths.

CONCLUSIONS

A semi-analytical method is developed for the dynamic response analysis of a fluid-saturated porous medium. The method uses the finite element discretization only along depth and analytical form in the lateral direction. The method is found to be numerically very efficient, particularly for two-phase mixture problems. The water above the sediments couples with the sediments through the pore fluid and affects the dynamic response of the sediments when the dynamic disturbance is applied in the sediments. This in turn affects the dynamic response of the foundation in the seafloor.

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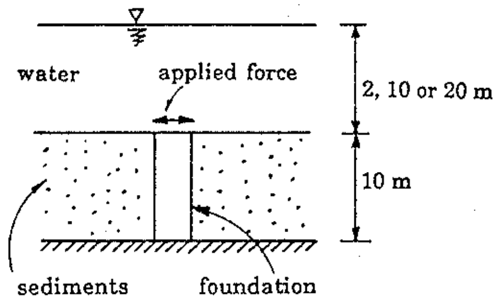
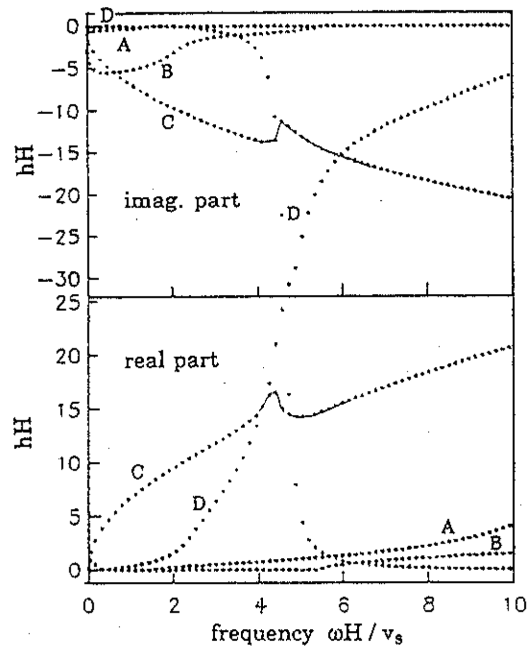
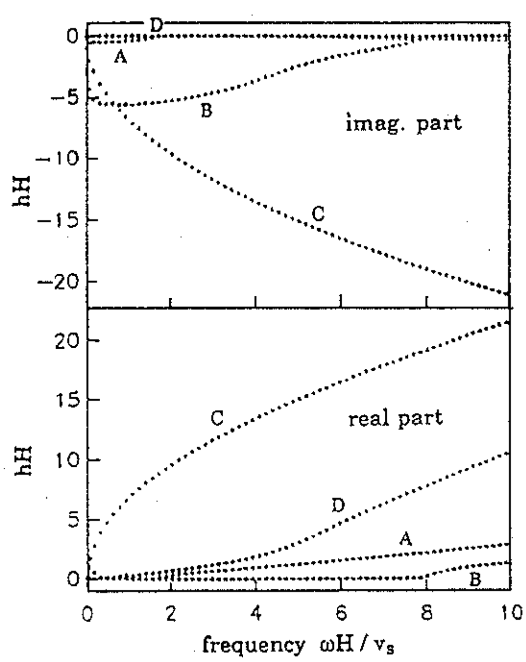


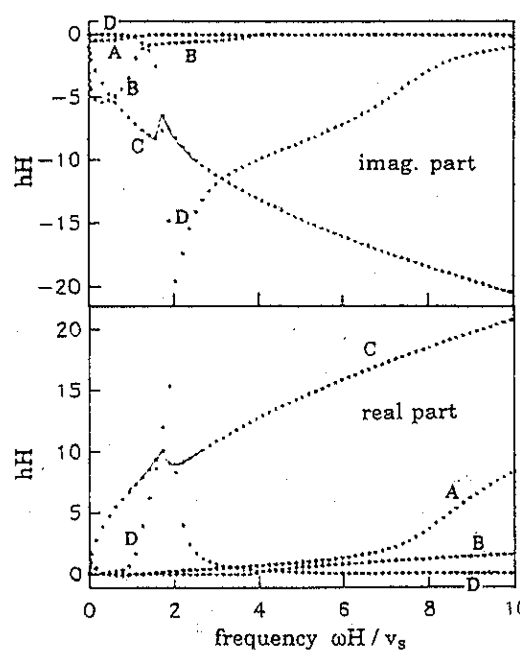
Fig. 1 Foundation deeply embedded in seafloor sediments



(b) Water Depth = 10m

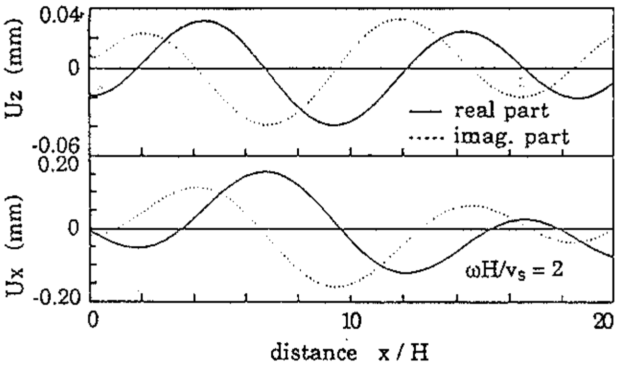


(a) Water Depth = 2m

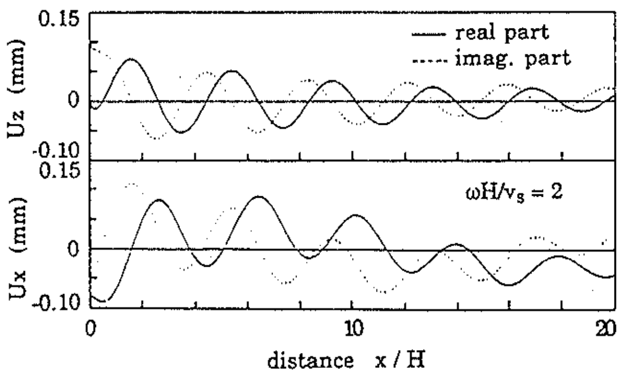


(c) Water Depth = 20m

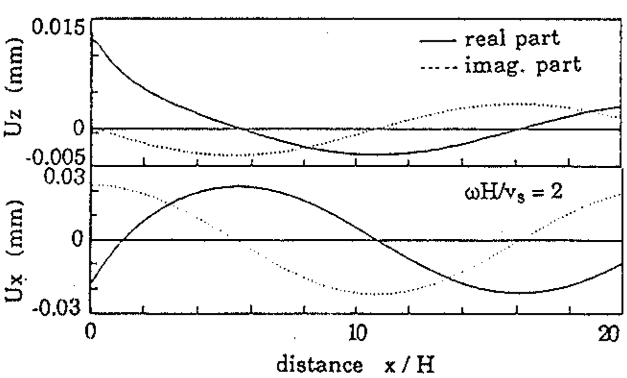
Fig. 2 Wave dispersion curves for water depths 2m, 10m and 20m



(a) Water Depth = 2m



(b) Water Depth = 10m



(c) Water Depth = 20m

Fig. 3 Seafloor displacement amplitudes along the surface for water depths 2m, 10m, and 20m

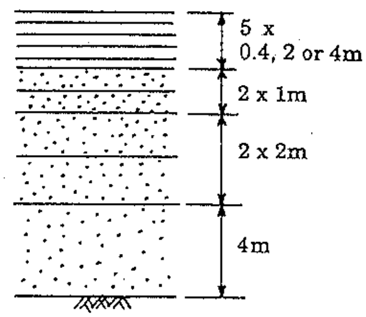


Fig. 4 Water-sediments-foundation system divided into 10 horizontal layers

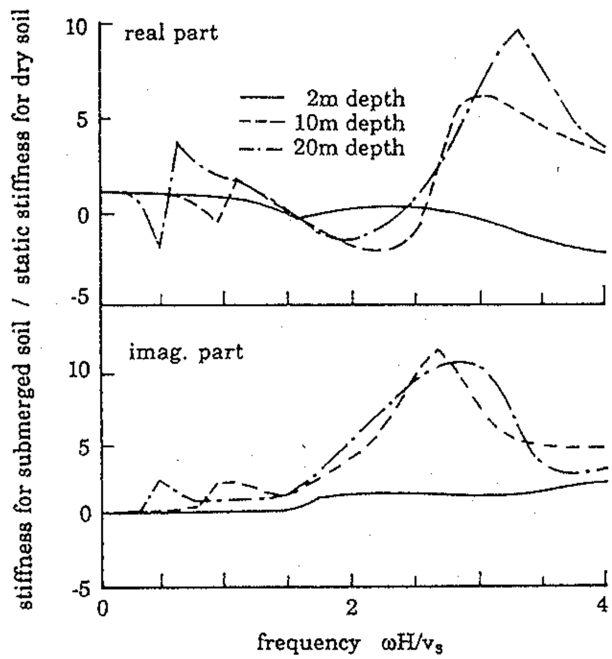


Fig. 5 Variations of soil-foundation stiffnesses with frequency for water depths 2m, 10m and 20m

Mail Code A-031, SIO
University of California
La Jolla, CA 92093

April 10, 1992

Mr. Charles E. Smith
Research Program Manager
Minerals Management Service
381 Elden Street. (MS647)
Herndon, Virginia 22070-4817

Dear Charlie:

Enclosed is a copy of my new publication which has come out of the last project funded by your organization in addition to the publications previously sent to you. As previously explained, the response of offshore foundation can significantly be affected by the water-seafloor interaction. This is a very significant finding, since such effects are not recognized and therefore is not taken account in the analysis at the present time. I am hoping that your funding for this research is restored and can continue to achieve our original objective. In the continuation of the research I am going to take into account aforementioned important finding.

I went to Japan for three weeks and just came back. During my stay in Japan, I met Mr. Noda of the Port and Harbor Research Institute. He is anxious to collaborate to this research (I am planning to use their facilities for this research). He asked me to send his best regard to you.

I hope that your funding will be restore very shortly for the continuation of this important research. Best regards.

Sincerely,

T. Nogami

Toyoaki Nogami