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DYNAMIC RESPONSE OF TWO RIGID FOUNDATIONS BY THE 3-D TIME DOMAIN BEM

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SUMMARY

This paper deals with the dynamic soil-foundation interaction problem by the three-dimensional time domain BEM. The soil-foundation model considered herein is two rigid massless foundations, that is, one is an embedded foundation and the other a surface one. Dynamic responses of these foundations subjected to plane SV waves are obtained directly in both transient and steady states. In particular, the effects of both incident and azimuth angles on the foundation input motion are investigated through the present method.

INTRODUCTION

In recent years, it has been often done to investigate dynamic interaction between rigid foundations by using the boundary integral equation method or the boundary element method. For instance, Yoshida et al.(Ref. 1), Sato et al.(Ref. 2) and Wong et al.(Ref. 3) obtained the impedance functions and the foundation input motion for such foundations. However, these analyses were carried out in frequency domain and therefore followed by inverse transform in order to obtain time history of response. On the other hand, recently, time domain BEM has drawn much attention by the reason that we are able to obtain directly results in time domain and more accurate transient response . Karabalis et al.(Ref. 4) first applied time domain BEM to dynamic interaction problem and their results were obtained by using relatively simplified procedure.

In this paper, the time domain BEM which the authors have already developed(Ref. 5) is presented in the accurate form. First of all, in order to show the accuracy of the present method, the obtained results are compared with those of Dominguez(Ref. 6) for an embedded rigid foundation. Secondly, two closely spaced foundations are considered for the soil-foundation model and the effects of incident and azimuth angles on the foundation input motion of these foundations are treated in both transient and steady state by using the present method.

METHOD OF SOLUTION

Initial-boundary value problem of elastodynamics is governed by the Navier equation,

$$\mu \nabla^2 \underset{\sim}{u} + (\lambda+\mu) \nabla \nabla \cdot \underset{\sim}{u} + \rho \underset{\sim}{f} = \rho \ddot{\underset{\sim}{u}} \qquad (\underset{\sim}{x}, t) \in R \times T^+ \tag{1}$$

The solution of the problem is given on the smooth boundary in the form of the following boundary integral equation (Ref. 7):

$$\frac{1}{2}u_k(\underset{\sim}{y},t)=\int_B\int_0^t\{U_{ik}(\underset{\sim}{x},\tau;\underset{\sim}{y})t_{(n)}(\underset{\sim}{x},t-\tau)$$
$$-T_{(n)ik}(\underset{\sim}{x},\tau;\underset{\sim}{y})u_i(\underset{\sim}{x},t-\tau)\}d\tau\ dB(\underset{\sim}{x}) \tag{2}$$

where λ and μ are Lamé's constants and, ρ is the density of the medium, and U_{ik} and $T_{(n)ik}$ are three-dimensional fundamental solution and the corresponding traction tensor in the infinite medium ,respectively.

Considering the discontinuity of stress along the wave front, we can construct a time-stepping scheme to solve Eq.(2). By discretizing the boundary integral equation (2) with constant elements with respect to space and linear ones for displacements and constant ones for tractions with respect to time, we can obtain the following matrix form :

$$\frac{1}{2}\begin{Bmatrix} u_e^N-u_{eg}^N \\ u_f^N-u_{fg}^N \end{Bmatrix}=\sum_{n=1}^{N}\left\{\begin{bmatrix} G_{ee}^n & G_{ef}^n \\ G_{fe}^n & G_{ff}^n \end{bmatrix}\begin{Bmatrix} t_e^{N-n+1}-t_{eg}^{N-n+1} \\ t_f^{N-n+1}-t_{fg}^{N-n+1} \end{Bmatrix}-\begin{bmatrix} H_{ee}^n & H_{ef}^n \\ H_{fe}^n & H_{ff}^n \end{bmatrix}\begin{Bmatrix} u_e^{N-n+1}-u_{eg}^{N-n+1} \\ u_f^{N-n+1}-u_{fg}^{N-n+1} \end{Bmatrix}-\begin{bmatrix} S_{ee}^n & S_{ef}^n \\ S_{fe}^n & S_{ff}^n \end{bmatrix}\begin{Bmatrix} u_e^{N-n}-u_{eg}^{N-n} \\ u_f^{N-n}-u_{fg}^{N-n} \end{Bmatrix}\right\} \tag{3}$$

where subscripts e and f denote interface between the rigid foundation and the soil , and free surface ,respectively. Subscript g represents the free field in a half-space and superscript denotes time step.

Continuity conditions of displacements and tractions on the interface between the rigid foundation and the soil are given as ,respectively,

$$\{u_e^N\}=[A]\{U^N\} \tag{4}$$

$$\{P^N\}=[T]\{t_e^N\} \tag{5}$$

where matrices [A] and [T] are transformation matrices depending on the shape of foundation and mesh size employed, and vectors $\{U^N\}$ and $\{P^N\}$ are displacement at the bottom of foundation and forces applied to the bottom of foundation respectively. For the problem of foundation input motion, vector $\{P^N\}$ reduces to zero vector $\{0\}$.

By substituting of Eqs.(4) and (5) into Eq.(3), the resulting set of linear equations may be written in matrix form as

$$\begin{bmatrix} \frac{1}{2}[A]+[H_{ee}^1][A] & [H_{ef}^1] & -[G_{ee}^1] \\ [H_{fe}^1] & \frac{1}{2}[I]+[H_{ff}^1] & -[G_{fe}^1] \\ 0 & 0 & [T] \end{bmatrix}\begin{Bmatrix} U^N \\ u_f^N \\ t_e^N \end{Bmatrix}=\begin{Bmatrix} A_0^N \\ B_0^N \\ P^N \end{Bmatrix} \tag{6}$$

where the right-hand side in Eq.(6) contains the contributions of both previous steps and the free field which consists of the known values. Eq.(6) can be solved easily by using the ordinary step-by-step scheme.

NUMERICAL RESULTS

The free-field displacements used in this paper and the response amplitude are represented as

$$\underset{\sim}{u}_f=\sum_{n=0}A_n\underset{\sim}{d}^{(n)}\exp[ik_n(\underset{\sim}{x}\cdot\underset{\sim}{p}^{(n)}-C_n\cdot t)]\cdot H(C_n\cdot t-\underset{\sim}{x}\cdot\underset{\sim}{p}^{(n)}) \tag{7}$$

$$|u^N|=\sqrt{\{Re(u^N)\}^2+\{Im(u^N)\}^2} \tag{8}$$

where A_n is the amplitude, $\underset{\sim}{d}^{(n)}$ and $\underset{\sim}{p}^{(n)}$ are unit vectors defining the directions of motion and propagation respectively, k_n , $\underset{\sim}{x}$ and C_n are the wave number, the position vector and the propagation speed of wave ,respectively, and H is the Heaviside step function. According to Eq.(8), we are able to distinguish steady state from transient one for the response amplitude.

Firstly, in order to show the accuracy of the present method, let us consider the dynamic response of an embedded rigid foundation subjected to incident SH waves as shown in Fig.1. Fig.2 represents the comparison between the results obtained by the present method and those of Dominguez . Two results are in good agreements with each other for two incident angles,i.e., $\theta=0^\circ$ and 45°.

Secondly, let us consider the dynamic response of two rigid foundations which are an embedded rigid massless foundation (=BASE-1) and a surface one (=BASE-2) subjected to incident SV waves as shown in Fig.3. Fig.3 shows also material constants and shape of foundations. Symbols θ and ϕ in this figure denote incident and azimuth angles ,respectively.

Figs.4-6 show time history of response amplitude for $\alpha_0=2.0$ and $\theta=15^\circ$,and each figure corresponds to azimuth angles $\phi=0^\circ$, 180° and 90° respectively. In these figures, horizontal axis represents dimensionless time, C_2T/B and α_0 denotes dimensionless frequency, $\omega B/C_2$, where C_2 is the shear wave velocity of the medium. It is made clear from the obtained results that transient state moves to steady one around the dimensionless time 7.5, and components of all directions for the azimuth angle $\phi=90^\circ$ have non-zero values and complicated behaviours appear in rocking motions at earlier stage .

Figs.7-8 show steady state of response amplitudes at BASE-2, and horizontal axis represents dimensionless frequency α_0. It is shown that the responses of incident angle $\theta=15^\circ$ are considerably different from those of single foundation and especially horizontal displacement for azimuth angle $\phi=180^\circ$ is much larger than those of free field.

CONCLUSION

In this paper, we presented the accurate form of three-dimensional time domain BEM and applied to the foundation input motion of two rigid foundations. We were able to obtain not only transient response but also steady response directly by using the time domain BEM. In particular, we could make clear the complicated behaviours between two foundations corresponding to azimuth angles.

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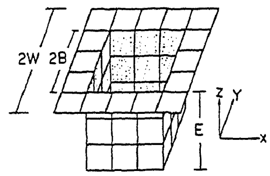


Fig. 1 Embedded Rigid Foundation Model Subjected to Incident SH Waves

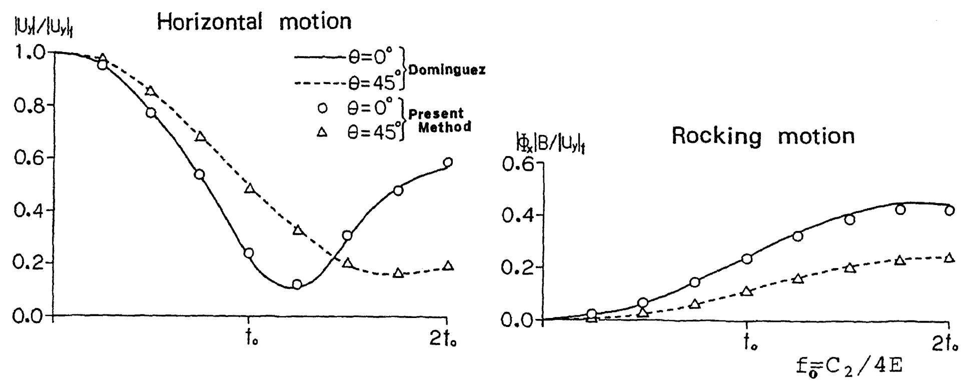
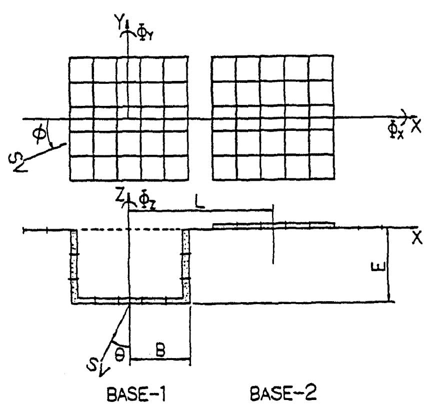


Fig. 2 Comparison of Results of Dominguez and those of Present Method



$L = 2.4B$, $E/B = 1.2$
$B = 40m$
$\mu = 2.5x10^5\ t/m^2$
$\nu = 1/3$
$\rho = 0.25t \cdot sec^2/m^4$

Fig. 3 Two Rigid Foundations Model Subjected to Incident SV Waves

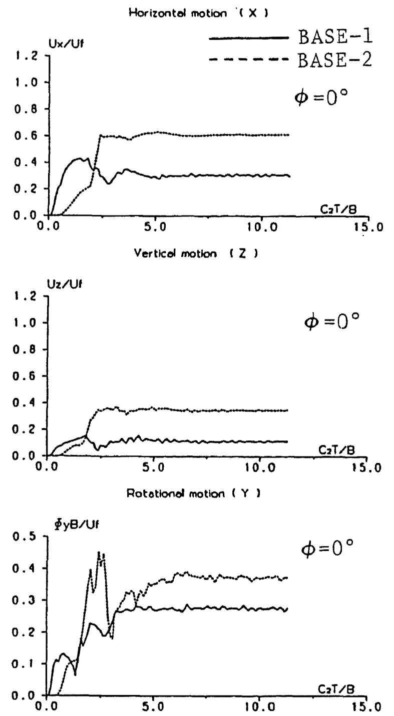


Fig.4 Time History of Amplitudes (α_0=2.0, θ=15°, ϕ=0°)

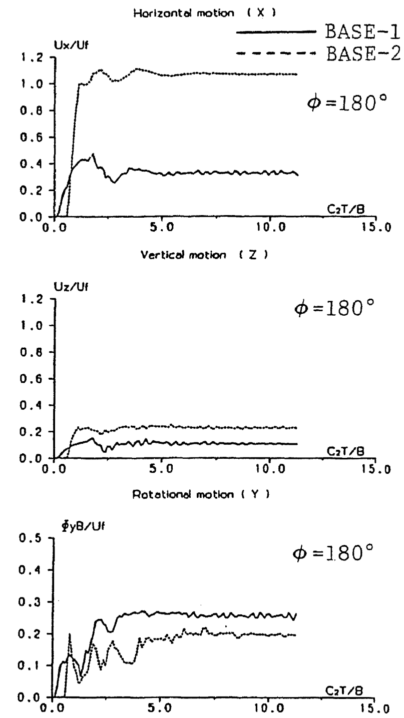


Fig.5 Time History of Amplitudes (α_0=2.0, θ=15°, ϕ=180°)

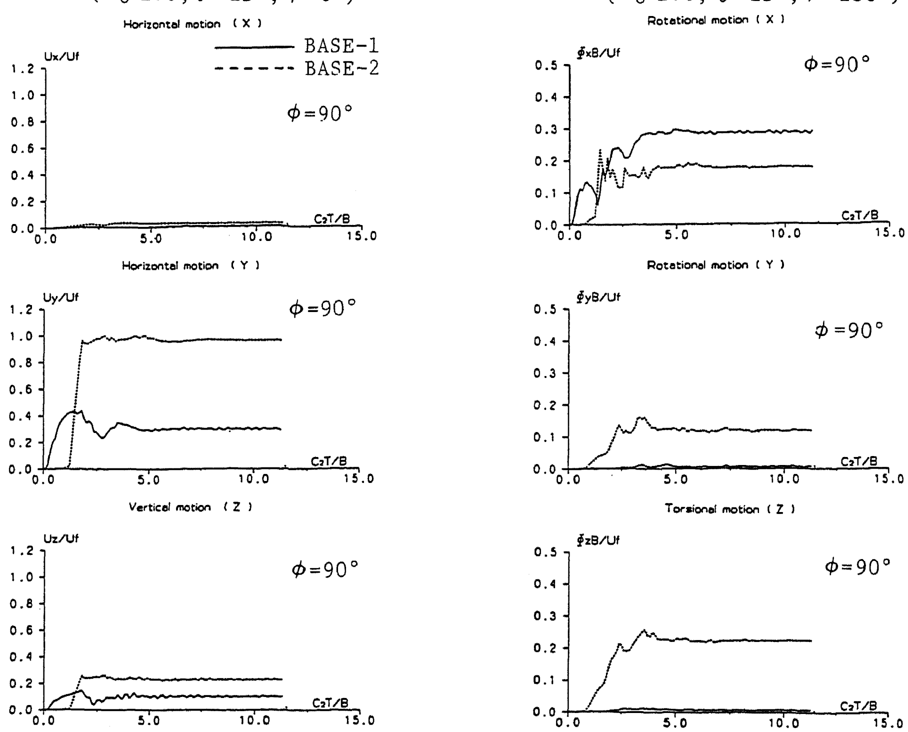


Fig.6 Time History of Amplitudes (α_0=2.0, θ=15°, ϕ=90°)

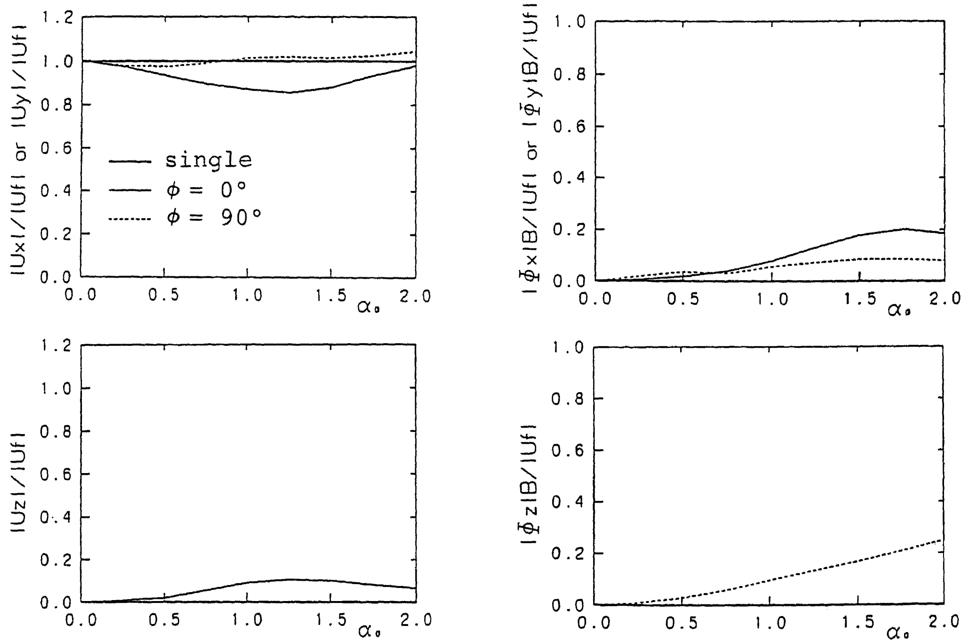


Fig.7 Steady Solutions of Amplitudes for BASE-2 (θ =0°)

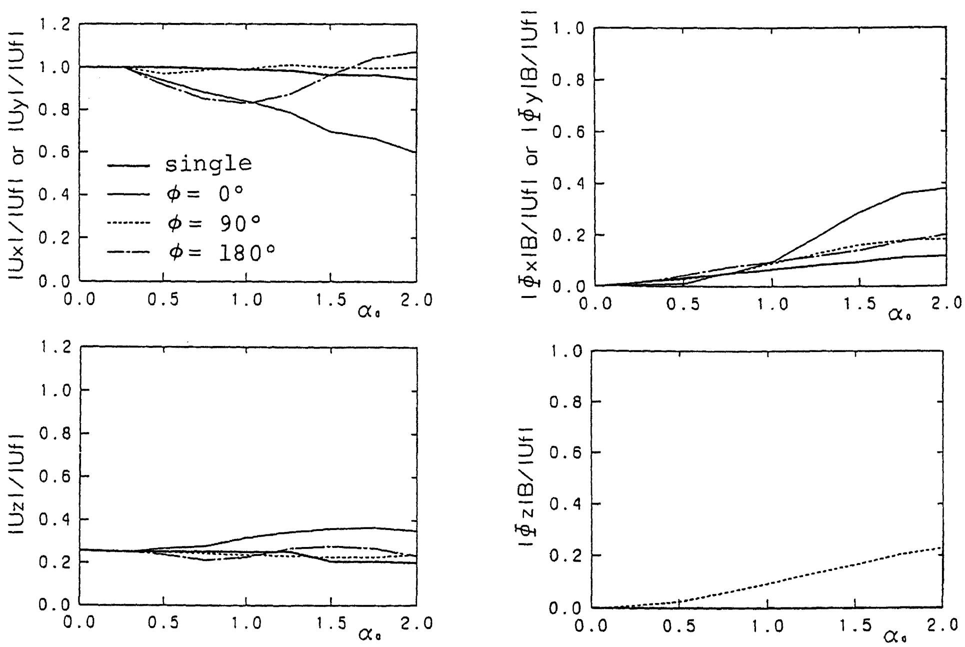


Fig.8 Steady Solutions of Amplitudes for BASE-2 (θ =15°)