SOIL-FOUNDATION INTERACTION OF REACTOR STRUCTURES SUBJECT TO SEISMIC EXCITATION

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ABSTRACT

A theoretical investigation has been conducted to study the soil-structure dynamic interaction effects on the seismic response of reactor structures. The analysis was made by considering a linear, damped, unsymmetric, three-dimensional flexible structure coupled with an elastic, homogeneous, isotropic half-space. The model is capable of handling multi-component ground excitation input and accounting for additional ground effects, such as the through-soil coupling between translation and rocking motion.

Numerical results for some example problems dealing with a prestressed concrete reactor vessel (PCRV) are presented in terms of both transient and steady-state responses. The seismic behavior of reactor structures was found to differ considerably from that exhibited by ordinary buildings. Significant interaction effects were observed for various cases which have not been previously analyzed.

1. INTRODUCTION

The dynamic interrelationship between an elastic structure and its foundation is known to have a significant influence on the structural response during earthquakes. These interaction effects are more pronounced for heavy structures, particularly when founded on a relatively soft medium, and it has become necessary to incorporate the soil-structure interaction in the seismic design considerations of today's large nuclear power stations.

The interaction phenomenon was initially studied by approximating the dynamic properties of the soil with discrete springs and masses. More rigorous solutions have recently been developed by coupling the structure with an elastic half-space. Among these recent contributions, one may cite the work of Parmelee [1,2], Luco [3], and Scavuzzo, et al. [4]. The use of an elastic half-space in an interaction problem has been shown to yield results which are in good agreement with the measured data from Duke, et al. [5].

Parmelee coupled the elastic half-space with a mathematical model representing a single-story building, and considered the translation and rocking degrees of freedom of the system. Luco studied the interaction of soil with an infinitely long shear wall. He assumed that the ground excitation is such that the movement of soil is parallel to the direction of the shear wall and hence, the motion of the system is purely translational. Scavuzzo, Bailey, and Raftopoulos obtained the time-history of the seismic response of nuclear reactors on an AECsponsored study. They considered the dynamic interrelationship between an elastic half-space and a conventional N-mass structure in a lateral translational motion. No damping in the structure can be considered to exist in their formulation. Tajimi [6] discussed Parmelee's paper on the higher mode aspects and derived the equations for steady-state response of an N-mass structure with lateral translation and rocking degrees of freedom. Tajimi's primary emphasis was to demonstrate the formulation for an N-mass building model through the use of a modal transformation technique, and no numerical results were presented. In these investigations utilizing an elastic half-space, the geometry and motion of the system are highly idealized so that the problems necessarily were treated in a rather restricted manner. Hence, the techniques are not adequate for general analysis of complex unsymmetric structures such as nuclear power plants.

In the present work, a more general approach to the problem is presented by considering a linear, damped, unsymmetric, three-dimensional flexible structure coupled with an elastic, homogeneous, isotropic half-space. The structure is simulated by a discrete system which can have, in addition to its modal coordinates, six rigid-body degrees of freedom. Additional considerations, such as through-soil coupling between translation and rocking, have been included in the analysis. The seismic excitation is defined by a free-field displacement column matrix which is allowed to have three translational and three rotational components, each having a prescribed time history. The three-dimensional interaction equations were formulated from the Lagrange's equations, and the time-history responses of the system were obtained through a Fourier synthesis method. In order to demonstrate the validity of the analysis, the general equations derived were subsequently specialized to a number of simplified models for obtaining numerical results of some example problems. Both transient and steady-state responses are presented here for a prestressed concrete reactor vessel (PCRV).

2. THEORETICAL FORMULATION

2.1 Derivation of Governing Equations

In the derivation of the governing equations, the Lagrangian formulation was adopted in contrast to direct application of Newton's Laws approach which has been used in the past for interaction studies. The indical notation and summation convention are used throughout this section except when matrix notation is preferable in order to enhance the clarity. The indices appear as subscripts and the superscripts used are for identification purposes.

Consider the theoretical model shown in Fig. 1 where two Cartesian coordinate reference systems have been introduced. The X_i (i=1,2,3) frame is a fixed reference system and the x_i (i=1,2,3) frame is a moving coordinate system. The base mat is assumed to be a rigid slab which supports a flexible superstructure. The x_i frame is assumed to be fixed in the base and its origin, 0, is located at the center of the base-medium contact area. The velocity vector of the origin of the moving frame is designated as $\dot{\underline{u}}^o$. The x_i frame also rotates with an angular velocity vector $\underline{\Omega}$ relative to the X_i frame.

Let the kinetic energy of the structure-base system be designated as T which is decomposed into

$$T = T^B + T^S \tag{1}$$

where T^B and T^S denote, respectively, the kinetic energy of the base and the kinetic energy of the remainder of the structure.

For a rigid base, it has been shown in [7] that

$$T^B = \frac{1}{2} m^B \dot{u}_i^o \dot{u}_i^o + \frac{1}{2} I_{ij}^B \Omega_i \Omega_j + m^B e_{ijk} \dot{u}_i^o \Omega_j r_k^B \quad (i,j,k = 1,2,3) \tag{2}$$

where $\dot{u}_i^o$ and Ω_j are, respectively, the components of $\underline{\dot{u}}^o$ and $\underline{\Omega}$, e_{ijk} is the permutation symbol [8], m^B is the total mass of the base, r_k^B are components of the position vector of the mass center of the base mat, and I_{ij}^B are the inertia tensors of the base with respect to the x_i coordinate system defined as

$$I_{ij}^B = \int_{\tau^B} \rho \; (\delta_{ij} r_k r_k - r_i r_j) d\tau \tag{3}$$

where ρ is the mass density, δ_{ij} is the Kronecker delta, r_k are the components of the position vector of a particle with origin at O, and the integral is over the volume of the base, τ^B.

Let the remainder of the structure (superstructure) be represented by a lumped-mass model. The kinetic energy T^S may then be expressed as a finite sum consisting of the following N terms

$$T^S = \frac{1}{2} m_{\ell n}^D \dot{u}_\ell^S \dot{u}_n^S \quad (\ell, n = 1,2...N) \tag{4}$$

where $m_{\ell n}^D$ is a diagonal inertia matrix of the discrete masses, and $\{u^S\}$ is an N-component displacement vector relative to the fixed reference frame.

Let $\{u\}$ designate the displacement vector of the discrete structural masses relative to the x_i frame, and let the rigid-body displacement vector of the superstructure-base system be defined as a 6X1 column matrix

$$\{U^R\} = \begin{Bmatrix} u_i^o \\ \hline \theta_i \end{Bmatrix} \quad (i = 1,2,3) \tag{5}$$

where θ_i are components of small rotation vector of the x_i coordinate frame and $\dot{\theta}_i \equiv \Omega_i$. The following kinematic relation relates $\{u^S\}$ to the rigid-body displacements $\{U^R\}$ and the relative displacements $\{u\}$ in a linear transformation

$$\dot{u}_n^S = A_{nr} \dot{U}_r^R + B_{n\rho} \dot{u}_\rho \quad \begin{matrix} (n=1,2,...N) \\ (\rho=1,2,...L) \\ (r=1,2,...6) \end{matrix} \tag{6}$$

where A and B are rectangular transformation matrices. The displacement vector $\{u\}$ introduced here is relative to a rigid-body reference position of the system. In seismic analyses, the modal data associated with $\{u\}$ are usually known and they are often referred to as the "fixed-base natural frequencies and mode shapes."

Let the number of modes be assumed to be the same as the number of components in $\{u\}$, and let $\{u\}$ be expressed in terms of the fixed-base mode shapes of the discrete structure as

$$u_\rho = \Psi_{\rho s} q_s(t) \quad (\rho, s = 1,2,...L) \tag{7}$$

where $q_s(t)$ are the generalized coordinates and $\Psi_{\rho s}$ is the modal matrix of the discrete structure. Substitution of Eq. (6) into (4) in conjunction with Eq. (7) yields

$$T^S = \frac{1}{2} M_{pr}^S \dot{U}_p^R \dot{U}_r^R + \frac{1}{2} \mathcal{M}_{\rho s} \dot{q}_\rho \dot{q}_s + M_{rs}^{SB} \dot{U}_r^R \dot{q}_s \tag{8}$$

where

$$M^{S}_{pr} = m^{D}_{\ell n} A_{\ell p} A_{nr} \tag{9a}$$

$$\mathcal{M}_{\rho s} = m^{D}_{\ell n} B_{\ell q} B_{nt} \Psi_{q\rho} \Psi_{ts} \tag{9b}$$

$$M^{SB}_{rs} = m^{D}_{\ell n} A_{\ell r} B_{n\rho} \Psi_{\rho s} \quad \begin{array}{l} (\ell, n = 1,2,\ldots N) \\ (p, r = 1,2,\ldots 6) \\ (q, \rho, s, t = 1,2,\ldots L) \end{array} \tag{9c}$$

The strain energy of a lumped-mass structure may be written as

$$U = \frac{1}{2} k_{qs} u_q u_s \tag{10}$$

or, in terms of the generalized coordinates, as

$$U = \frac{1}{2} K^{S}_{\rho t} q_\rho q_t \tag{11}$$

where

$$K^{S}_{\rho t} = k_{qs} \Psi_{q\rho} \Psi_{st} \ .$$

Equations (2), (8), and (11) will be used in conjunction with the virtual work expression for substitution into the Lagrange's equations.

In the present formulation, the forces exerted on the base by the medium are treated as external surface forces on the structure-base system. The virtual work associated with the contact forces at the structure-medium interface may be written as [7]

$$\delta W^{M} = Q^{M}_{r} \delta U^{R}_{r} \tag{12}$$

where Q^{M}_{r} are components of the generalized forces associated with the interface stresses and the partitioned form of Q^{M}_{r} vector is consistent with Eq. (5), namely:

$$\left\{Q^{M}_{r}\right\} = \left\{\begin{array}{c} P_i \\ \hline M_i \end{array}\right\} \quad \begin{array}{l} (i = 1,2,3) \\ (r = 1,2,\ldots 6) \end{array} \tag{13}$$

in which

$$P_i = \int_{\Sigma^B} \tau_{i2} dA \qquad \text{and} \qquad M_i = \int_{\Sigma^B} e_{ijk} r_j \tau_{k2} dA \tag{14}$$

and τ_{ij} are the components of contact stress and the integrals are over the entire contact area Σ^B.

Using the principle of superposition, let $\{U^R\}$ be written as

$$U^{R}_{r} = u^{G}_{r} + u^{I}_{r} \quad (r = 1,2,\ldots 6) \tag{15}$$

where u^{G}_{r} is the prescribed free-field ground displacement vector and u^{I}_{r} is the additional ground displacement vector due to the presence of the structure. In the general model developed here, both $\{u^G\}$ and $\{u^I\}$ are allowed to have six components; three translations and three rotations. Each element in $\{u^G\}$ can have an independent time-history input.

Treating q_n and u^{I}_{r} as coordinate variables and applying the Lagrange's equations, the following two systems of equations are obtained:

$$M_{nm}\ddot{q}_m + C_{nm}\dot{q}_m + K^S_{nm}q_m = - M^{SB'}_{nr}\ddot{U}^R_r \tag{16a}$$

$$M^B_{pr}\ddot{U}^R_r + M^S_{pr}\ddot{U}^R_r + M^{SB}_{pn}\ddot{q}_n = Q^M_p \quad \begin{matrix}(n,m = 1,2,...L)\\(p,r = 1,2,...6)\end{matrix} \tag{16b}$$

where M^B can be put in partitioned form as

$$\left[M^B\right] = \left[\begin{array}{c|c} M^{BB} & M^{BC} \\ \hline M^{BC'} & I^B \end{array}\right] \tag{17}$$

with all the submatrices being 3X3. The elements in I^B have already been defined in Eq.(3) and

$$M^{BB}_{ij} = \delta_{ij}m^B \tag{18}$$

$$M^{BC}_{ik} = - e_{ijk}r^B_j m^B \tag{19}$$

In Eq. (16a), C_{nm} is known as the generalized damping matrix of the superstructure, and it may be derived from the dissipation term in the Lagrange's equations. The prime denotes the transpose of a matrix.

2.2 Solution of the Dynamic Equations

The solution to the system of differential equations given by Eqs. (16a) and (16b) was made tractable by first determining the steady-state responses of the interaction system and then obtaining its time-history solution through a Fourier synthesis method. The techniques require that the complex frequency responses of the system be determined and the seismic excitations be transformed into its frequency domain. The time histories of the response were then found by inverting the response transforms. The steady-state solution of the problem is obtained as follows:

For harmonic response,

$$q_n(t) = \bar{q}_n e^{i\omega t} \quad , \quad U^R_r = \bar{U}^R_r\, e^{i\omega t} \tag{20}$$

where ω is the circular frequency of the excitation, the bar over a vector designates the complex amplitude, and $i = \sqrt{-1}$ when it is not used as a subscript.

Solving Eq. (16a) for the $\bar{q}_n$ results in

$$\bar{q}_n = - T^{-1}_{nm}\, M^{SB'}_{mr}\bar{\ddot{U}}^R_r \tag{21}$$

where $\bar{\ddot{U}}^R = - \omega^2\bar{U}^R$ and T^{-1} is the inverse of the complex matrix $T(i\omega)$ defined by

$$T_{nm}(i\omega) = - \omega^2 M_{nm} + i\omega C_{nm} + K^S_{nm} \; . \tag{22}$$

The two systems of equations (16a) and (16b) may be combined into a single set by substituting Eq. (21) into Eq. (16b). This gives

$$\left(M^B_{pr} + M^S_{pr}\right)\bar{\ddot{U}}^R_r + M^{SB}_{pn}\, D_{nm}M^{SB'}_{mr}\bar{\ddot{U}}^R_r - K^M_{pr}\bar{u}^I_r = 0 \tag{23}$$

where the generalized forces have been replaced by

$$\bar{Q}^M_p = K^M_{pr}(i\omega)\bar{u}^I_r \tag{24}$$

and the matrix D_{nm} is defined as

$$D_{nm} = \omega^2 T^{-1}_{nm} \tag{25}$$

The response of the structure-base system is thus coupled with the displacements of the elastic half-space.

In Eq. (24), the matrix $K^M_{pr}(i\omega)$ is the dynamic stiffness matrix of the half-space medium. Its elements are complex functions taken from the solutions of the dynamic response of elastic semi-infinite solids under harmonic surface loading or harmonic incident waves. The imaginary part of an element in K^M matrix accounts for the energy dissipation due to radiation of the wave (radiation damping).

In view of Eq. (15), Eq. (23) reduces to

$$\left(\omega^2 M^I_{pr} + K^M_{pr}\right) \bar{u}^I_r = - \omega^2 M^I_{pr} \bar{u}^G_r \tag{26}$$

where

$$M^I_{pr} = M^B_{pr} + M^S_{pr} + M^{SB}_{pn} D_{nm} M^{SB'}_{mr}$$

Equation (26) represents a system of six algebraic equations with complex coefficients. When the free-field excitation components, $\bar{u}^G_r$, are prescribed, Eq. (26) can be solved for the amplitudes and phase angles of $\bar{u}^I_r$. The base steady-state motion can then be determined by the second equation in Eq. (20) with

$$\bar{U}^R_r = \bar{u}^G_r + \bar{u}^I_r \tag{28}$$

Using Eq. (26), one can show that $\{\bar{u}^I\}$ is related to the free-field acceleration amplitude vector by the following relation

$$\{\bar{u}^I\} = [E(i\omega)]^{-1}[M^I]\{\bar{\ddot{u}}^G\} \tag{29}$$

where

$$[E(i\omega)] = \omega^2[M^I] + [K^M(i\omega)]. \tag{30}$$

In accordance with Eq. (28), the base acceleration amplitudes can be expressed in terms of the ground excitations by the equation

$$\{\bar{\ddot{U}}^R\} = [H^R(i\omega)]\{\bar{\ddot{u}}^G\} \tag{31}$$

where

$$[H^R(i\omega)] = [I] - \omega^2[E(i\omega)]^{-1}[M^I] . \tag{32}$$

The amplitudes of the absolute accelerations of the structural masses can be put in the same form as Eq. (31). This gives

$$\{\bar{\ddot{u}}^S\} = [H^S(i\omega)]\{\bar{\ddot{u}}^G\} \tag{33}$$

where

$$[H^S(i\omega)] = \left([A] + [B][\Psi][D][M^{SB}]'\right)[H^R(i\omega)] . \tag{34}$$

The H^R and H^S matrices relate the steady-state amplitudes of the output quantities to the amplitudes of the input excitations and are often referred to as the complex frequency response matrices.

With the exception of certain physical phenomena such as faulting, the time history of an earthquake disturbance is sufficiently well-behaved to guarantee the existence of a Fourier transform. Let

$$\left\{\tilde{\ddot{u}}^G(\omega)\right\} = \int_{-\infty}^{\infty} \left\{\ddot{u}^G(t)\right\} e^{-i\omega t} dt \tag{35}$$

be the Fourier transform column matrix of the ground accelerations so that the response transforms of the absolute accelerations of the structural masses are

$$\left\{\tilde{\ddot{u}}^S(\omega)\right\} = \left[H^S(i\omega)\right] \left\{\tilde{\ddot{u}}^G(\omega)\right\} \tag{36}$$

Upon inverting Eq. (36), the time history of the structural response is obtained as

$$\left\{\ddot{u}^S(t)\right\} = \frac{1}{2\pi} \int_{-\infty}^{\infty} \left[H^S(i\omega)\right] \left\{\tilde{\ddot{u}}^G(\omega)\right\} e^{i\omega t} d\omega \tag{37}$$

2.3 <u>Through-Soil Coupling Effects</u>

The medium stiffness matrix K^M in the interaction equations represents the influence of the foundation flexibility. The elements on its diagonal are the most significant ones since the off-diagonal elements account for the stiffness coupling effects through the medium. The dynamic stiffness properties of a semi-infinite elastic solid under forced vibrations have been determined by many investigators [9-14] based on a partial bonding theory. In the partial bonding conditions, it is assumed that no shear stresses will be induced for vertical or rocking motion and, conversely, the normal stresses vanish for lateral translational motion. These assumptions enable us to find the medium stiffness parameters for each type of motion separately and to analyze the interaction problem by considering a diagonal medium stiffness matrix.

The complete bonding condition has recently been investigated by Oien [15] who studied the harmonic wave diffraction problem in the steady-state motion of a rigid strip bonded to the surface of an elastic half-space. In his analysis, it has been shown that the horizontal translation and rocking of the rigid strip is coupled through the medium when the complete bonding condition is assumed. Using a different method, Karasudhi, et al. [16] also estimated the numerical values of such coupling effects. When the through-soil coupling terms are included, the resulting K^M matrix becomes nondiagonal. The ability to readily adjust the complex frequency response matrix of the system so as to account for these through-soil coupling effects is one of the important advantages of the present approach.

3. NUMERICAL COMPUTATIONS

As examples illustrating the applicability of the analysis, numerical results were obtained for the seismic response of an 1100 MW(e) PCRV based on several simplified lumped-mass models. The parameters representing the foundation medium properties are based on the research results readily available in the literature. The models used may have either circular or square bases although a comparison between a circular base with a square base having the same area has been shown to exhibit good agreement for the case of a vertical vibrating load [13].

The accuracy of the computer results has been verified in several ways. The steady-state results were obtained both by applying Cramer's rule and by using a complex matrix inversion subroutine. The two computing methods were found to give the same answers. As another independent verification of the steady-state results, the data for a typical conventional building were also used as input constants for the single-mode model and the response curves are plotted in Fig. 2. These curves are in close agreement with those given earlier by Parmelee [1]. The computation work associated with the Fourier synthesis method was done with the aid of the fast Fourier transform algorithm. Verification of time-history results was also made by comparing the responses computed for rigid ground cases with the values given by other computer programs.

Using conventional methods, the fixed-base natural frequencies, mode shapes, and modal characteristics of the PCRV were obtained. The terms "two-mode PCRV" as used here refers to the idealized PCRV structure having two fixed-base natural frequencies. Some explanation is necessary concerning the geometry and coordinates of the models used in the example problems.

The superstructure in Model I (Fig. 3a) is represented by N masses in series, each having one degree of freedom relative to the rigid-body displacements of the system. The earthquake motion in this model was assumed to have only a single component, lateral translation, and the base of the model is allowed to translate and rotate (rocking) about the x_3-axis, which is perpendicular to the direction of the ground motion. The rotations of the lumped structural masses relative to the x_i frame were ignored. When the rocking degree of freedom of the base is omitted, Model I reduces to the system studied by Scavuzzo, et al. [4].

Model II (Fig. 3b) represents an idealized unsymmetric system with an eccentric mass (or appendage) with multicomponent ground displacement input. The seismic motion was assumed to be a horizontal motion combined with vertical ground movement. The two displacement components can have a prescribed phase relation. The top mass, m_1, is supported by a "beam-type" member so that the translation and rotation of the top mass are elastically coupled. The structure has two natural modes associated with the two degrees of freedom, u_{m_1} and ϕ_{m_1}. The appendage mass is assumed to be attached rigidly to the top mass as shown in Fig. 3b. The geometry has been arranged in such a way that the system will vibrate in a coupled lateral-rocking-vertical type of motion under the excitation produced by a combined horizontal-vertical ground movement.

Model III (Fig. 3c) represents an idealized three-dimensional lumped-mass model simulating a nuclear reactor structure. The appendage mass, m_a, is again assumed to be attached rigidly to the main mass, m_1. The main mass is assumed to have four degrees of freedom relative to the rigid-body reference position of the system. These include the three rotational displacements and one translational displacement (North-South). Relative translations of main mass in the other directions were assumed negligible so that only four structural modes (fixed-base) are considered. The appendage mass is located at a different elevation from that of the main mass and, furthermore, the mass center of m_a is not in line with the principal axis of the main mass. The eccentric location of the appendage mass thus gives rise to coupling effects through the structure such that a ground translational movement in the North-South direction will excite all the four modes of the structures.

4. RESULTS AND DISCUSSIONS

4.1 Steady-State Response

The steady-state results are presented in the form of frequency-response curves with the amplitude ratios of the displacements considered to be of primary interest. The phase angles were not plotted. Since the complex amplitudes are used in the equations, it is understood that the ordinates in the response diagrams are the ratios of the absolute values of two complex amplitudes.

The ground flexibility is represented by the parameter V_S, which is the shear wave velocity of the half-space medium defined as

$$V_S = \sqrt{\mu/\rho} \tag{38}$$

where μ and ρ are the modulus of rigidity and the mass density of the half-space medium, respectively. The condition $V_S = \infty$ is the limiting case where the ground is rigid. The results obtained with rigid ground represent the responses of the system without considering the interaction effects.

For three-dimensional analysis, the model is assumed to have a circular base with radius r_o. The steady-state results supplied by Bycroft [11,12] for a three-dimensional elastic half-space were utilized. Bycroft's results are valid within the frequency range of $\omega r_o/V_s <$ 1.5. However, the extension of a half-space analysis to higher frequency factors has subsequently been made by Olen [15] and by Awojobi [17].

Figures 4 and 5 present the response amplitudes of the 1100 MW(e) PCRV simulated by Model I. Both the single-mode (soft-mounted) and two-mode (hard-mounted) cases were considered. These results clearly indicate that a large nuclear power station will behave differently from the conventional high-rise buildings during earthquakes. The interaction effects tend to reduce the response of a low, heavy structure due to the presence of relatively large amounts of radiation damping.

The response determined for a system simulated by Model II, where a two-component ground input was used, is given in Fig. 6. The free-field displacement vector was assumed to have two translational components; a lateral component u_g and a vertical component v_g. The curves in Fig. 6 were computed with v_g being in phase with u_g and the amplitude of v_g was taken to be 2/3 of that for u_g. It is interesting to note that as the medium shear wave velocity decreases, the response at the second peak is being reduced more rapidly than that at the first resonance. It has also been shown by the present authors [7] that the response curves can be affected by a change in the phase relationship between u_g and v_g.

The influence of structure-ground interaction upon the response of higher modes has been a subject of some international controversy. Parmelee [1] and Luco [3] have both shown that the higher mode response will be affected appreciably by the interaction phenomenon while Tajimi [6] claimed that most investigators in Japan believe that the effect is negligible. Since the results of the present investigation have again shown a rather pronounced influence on the higher mode response, it may be desirable to re-examine the opinion of the analysts in Japan on this particular issue.

The model for the ground medium in Model II was subsequently replaced by a two-dimensional half-space in order to study the influence of through-soil coupling between horizontal translation and rocking. In this study, the half-space response data given by Olen [15] and Karasudhi, et al. [16] were utilized and the steady-state results for the case with V_S = 1500 FPS are plotted in Fig. 7 against those obtained without consideration of coupling. Curves computed for other values of V_S show the same changes, differing primarily in their magnitude. The comparison indicates that the through-soil coupling effects not only tend to modify the natural frequencies of the system as expected, but also to reduce the structural response.

4.2 Time-History Response

The response of an interaction system under an arbitrary time-history input was determined for an example problem based on Model III which was subjected to a ground translational excitation in the X_3 direction. The time history of the N-S component of the El Centro, California earthquake (May 1940) was used as input and a modal damping factor of .05 was used for all the structural modes. When the ground is treated as a deformable medium, the base displacement vector has six non-zero components, and the system responds with three-dimensional motion to seismic excitation. Time histories of the six components of $\{u^I(t)\}$ are shown in Fig. 8 for the case with the structure on firm soil (V_S = 1000 FPS). As expected, the X_3 (North-South) base translation and rocking about the X_1-axis are seen to have much larger magnitudes than the other four components.

Figure 9 shows the comparison of the total base accelerations in X_3 direction with the applied free-field excitation input. When the reactor is on soil, the maximum values for the base acceleration tend to be reduced by the interaction effects. For a rock foundation medium, the peak base accelerations do not differ appreciably from the free-field maximums when damping is present in the structure. This is mentioned because the base accelerations can be significantly amplified by the interaction effects if the structure is undamped (see, for example, the results in Scavuzzo, et al. [18]).

Many previous investigators analyzed the interaction effects on the response of the superstructure through a response spectrum approach. In the present investigation, this is studied by examining the exact time history of the structural responses. Figure 10 presents the total translational acceleration (in North-South direction) of the appendage mass for a range of ground flexibility values. For this example problem, the interaction effects tend to reduce the inertia loads on the appendage mass as the ground medium becomes softer.

5. CONCLUSION

The approach described in this report suggests a method of combining the elastic half-space dynamic solutions to computer programs for elastic structures developed by finite-element methods. Although the primary emphasis in the present investigation has been placed upon the solution of dynamic response of a nuclear power station, the general formulation developed here is applicable to a wide class of structure-medium interaction problems. With appropriate modifications, the equations presented here can be used in the analysis of more involved systems in which the modal equations of the superstructure are not decoupled.

Numerical results demonstrating the usefulness of the Fourier synthesis method for solving coupled, three-dimensional interaction problems are presented. Significant modifications of both the response frequencies and magnitudes for both higher as well as fundamental frequencies are indicated. Further variations for low, massive structures such as nuclear power stations are obtained as compared to results for more conventional buildings.

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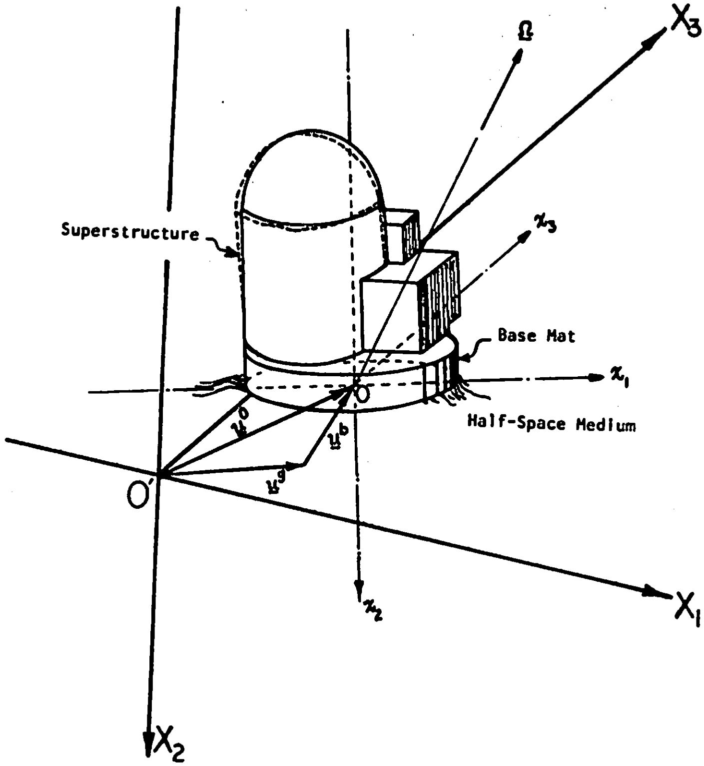


Fig. 1. Coordinate systems for theoretical model.

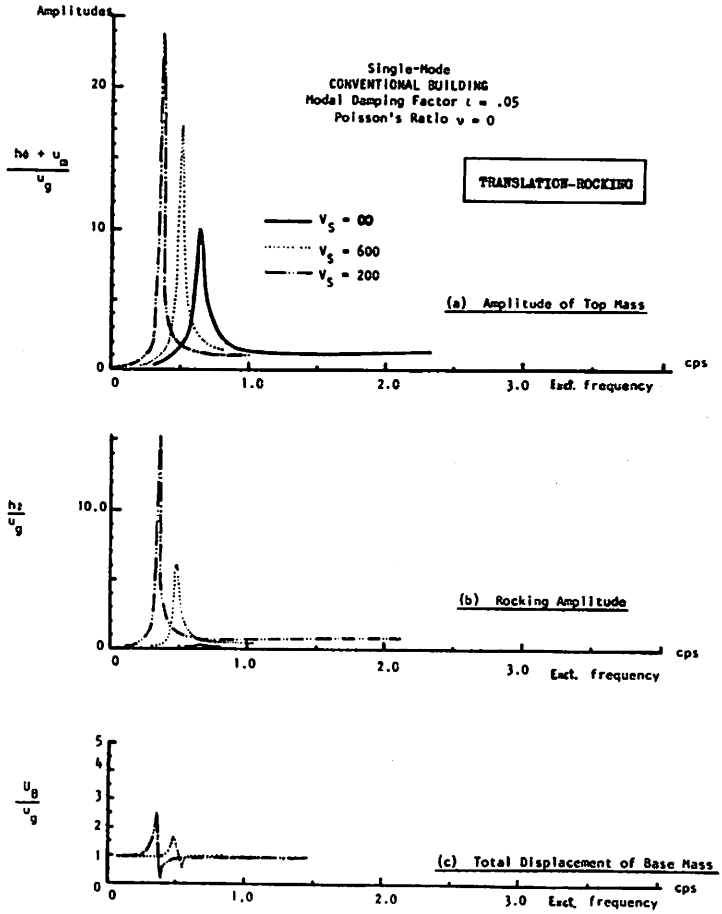
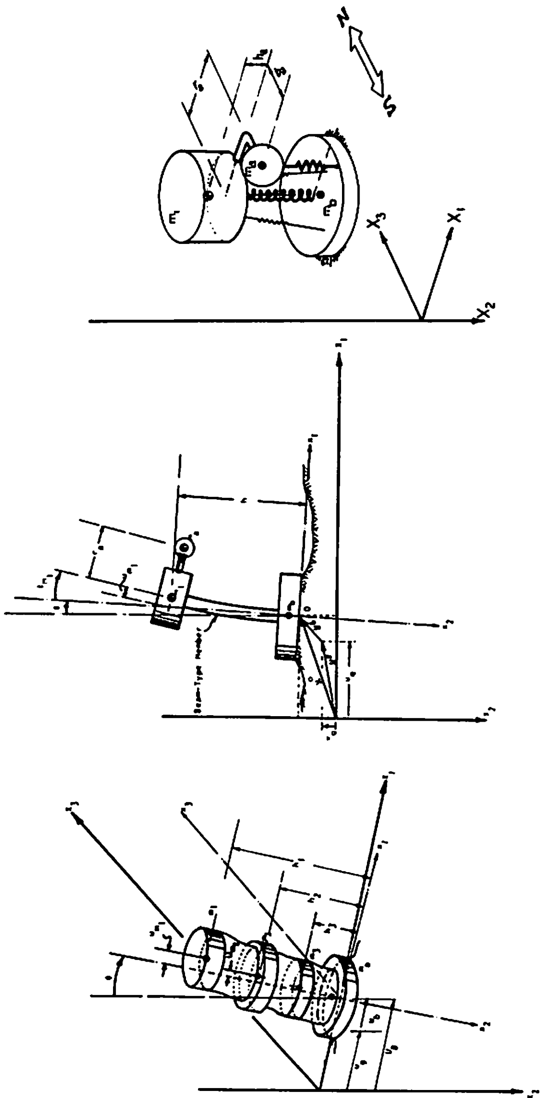


Fig. 2. Response amplitudes for buildings.



(a) Model I
Conventional N-Mass Building

(b) Model II
Unsymmetric System

(c) Model III
Idealized Three-Dimensional
Four-Mode Structure

Fig. 3. Structural systems.

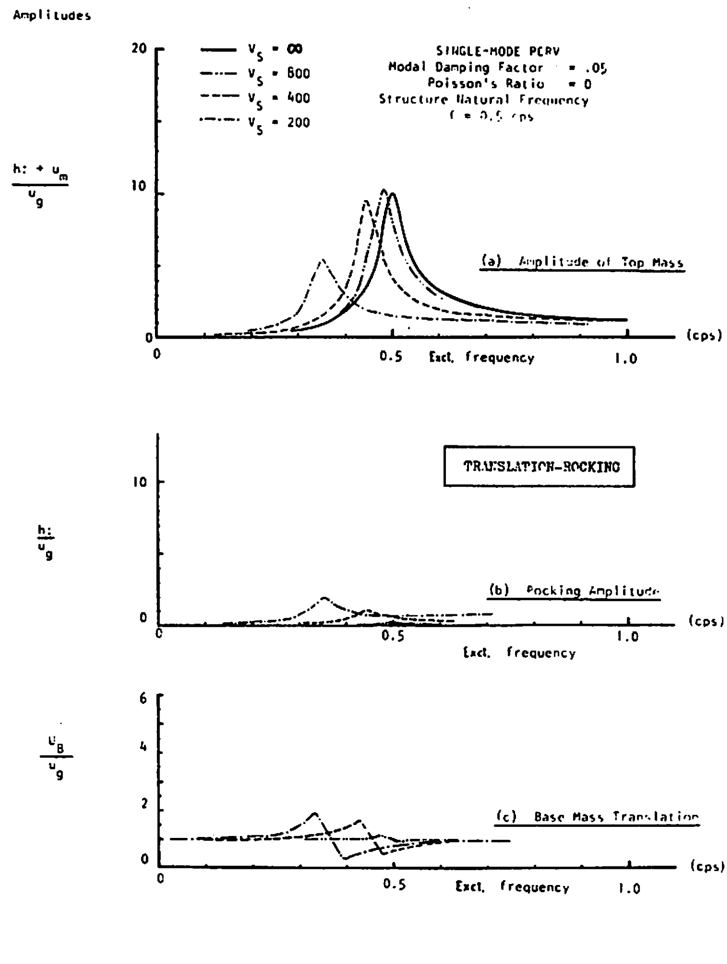


Fig. 4. Response amplitudes for soft-mounted PCRV.

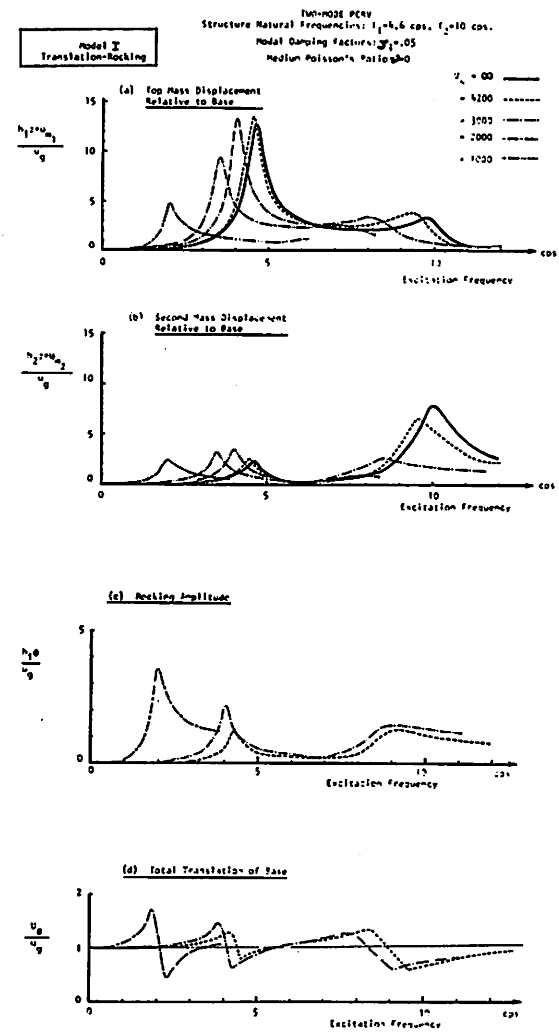


Fig. 5. Response amplitudes for hard-mounted PCRV.

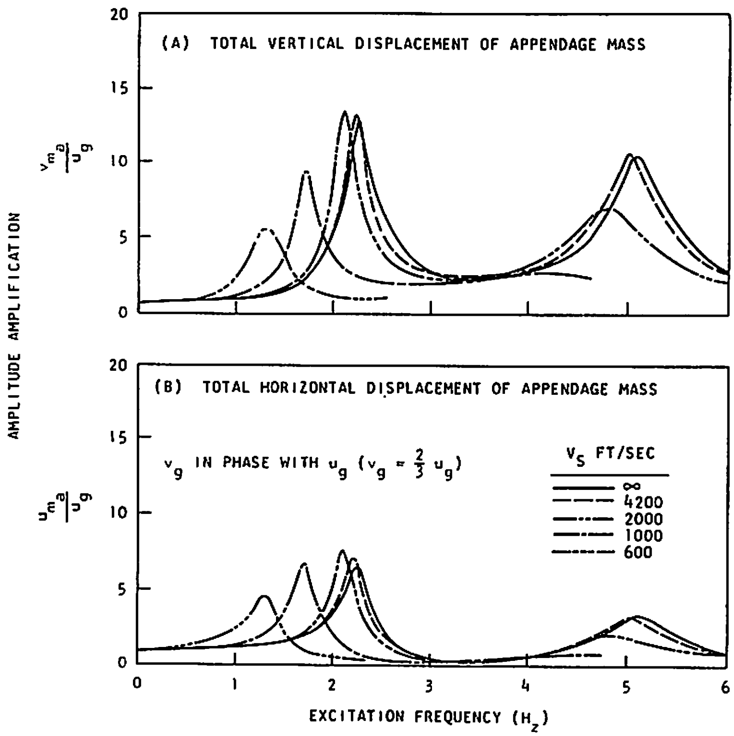


Fig. 6. Response amplitudes of unsymmetric PCRV subjected to combined lateral-vertical ground motion.

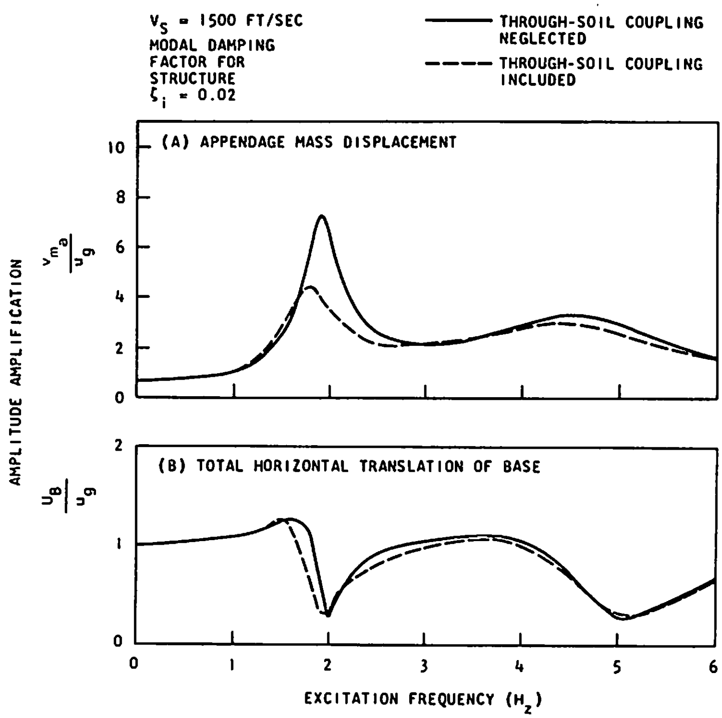


Fig. 7. Influence of through-soil coupling on the steady-state seismic response of reactor structure.

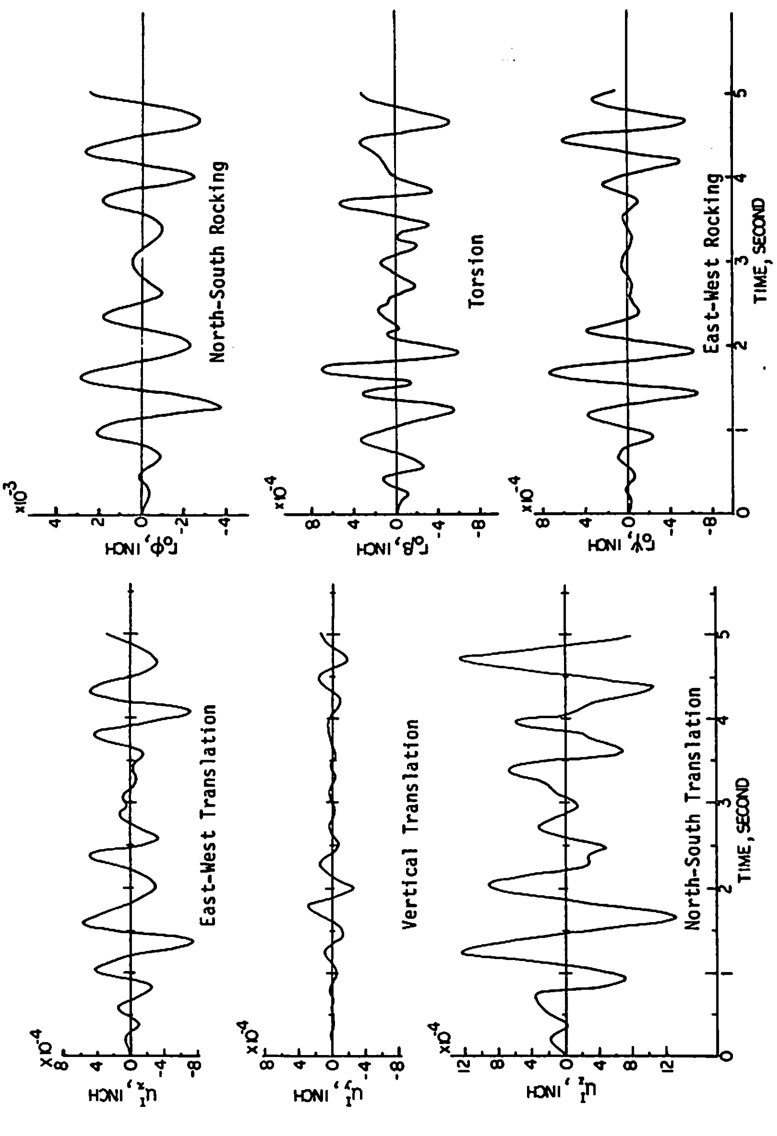


Fig. 8. Components of base displacement vector.

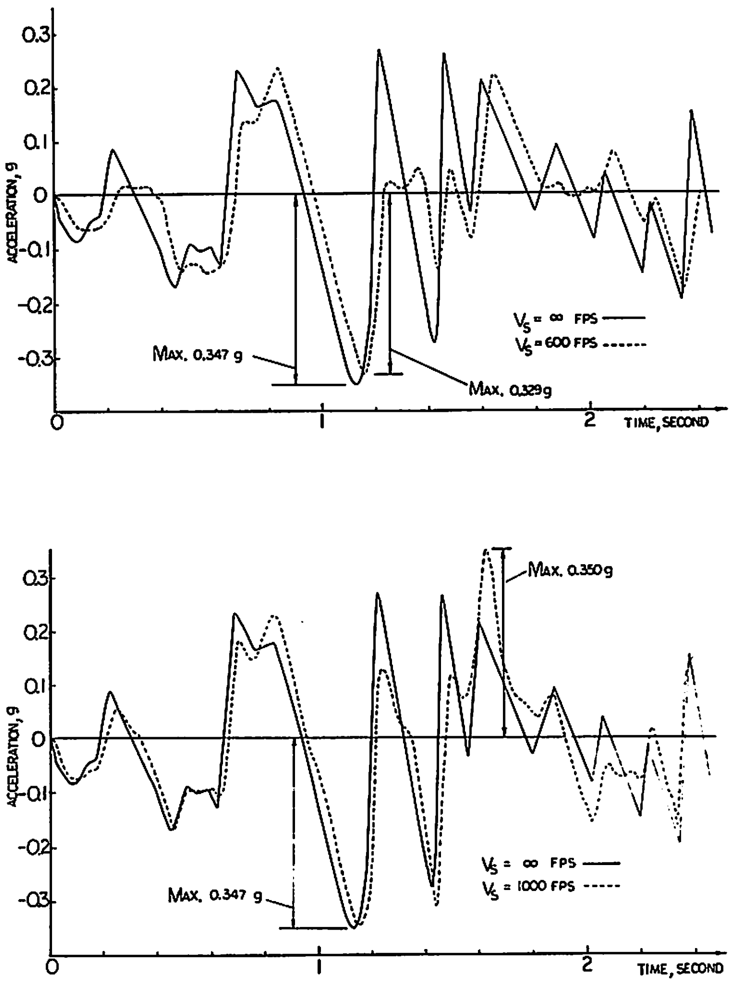


Fig. 9. Comparison between base acceleration and free-field excitation.

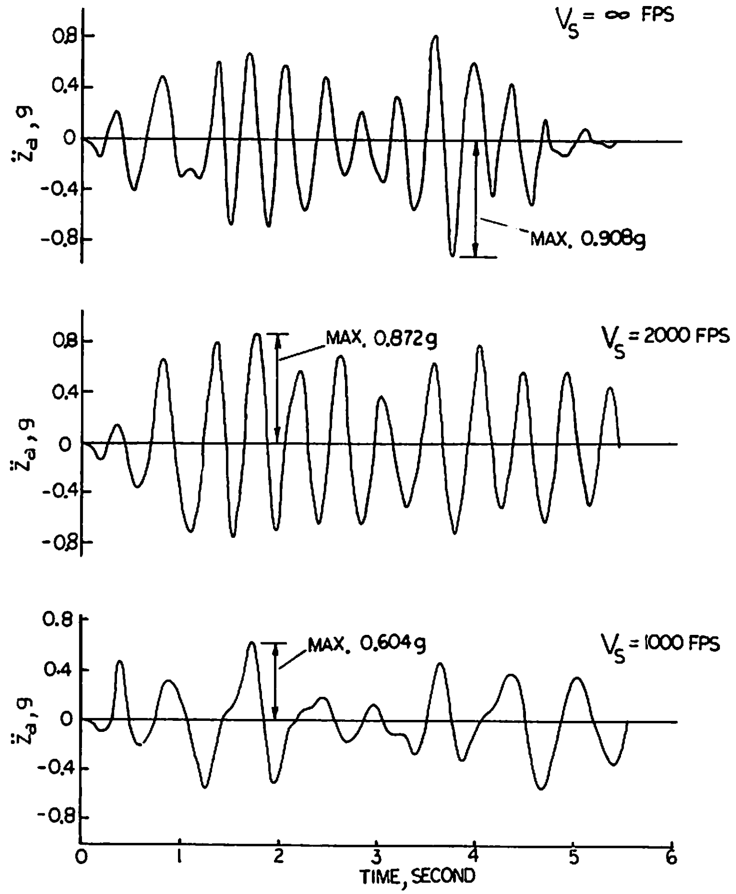


Fig. 10. Interaction effects on seismic response of appendage mass.

DISCUSSION

Q A. HADJIAN, U. S. A.

Contrary to Fig. 5 and more in line with Tajimi's thinking, Bielek's (1971) Ph. D. Thesis at Caltech shows that the 2nd mode of a two mass model of a containment structure does not show any variation of the frequency as compared to a fixed based model. This was parametrically shown to be true for a wide variation of soil properties.

A T. H. LEE, U. S. A.

The interaction effects on higher modes, in my opinion, will depend on the system parameters, at this stage I don't know what case Bielek has studied and what approach he has used. I will be glad to have a copy of his thesis for further studies.

Q H. SATO, Japan

The transfer function used in the equation of Fourier inversion would have interaction term between each ground component. I understand that these can be theoretically estimated.

Did you measure the interaction term which is supposed to appear in transfer function of multi-degree-of-freedom system ?

What sort of method of calculation did you use to obtain the Fourier spectrum for the analysis ?

A T. H. LEE, U. S. A.

The measured data for the Hollywood Storage Building, Los Angeles (Kern Country Earthquake, 1952) are for the horizontal translation only. To compare with these measured data, I only considered the single horizontal displacement of ground motion in my computer analysis. I agree with you on the point that the actual building response may be affected by a multiple-component ground motion during that earthquake. However, I did not make any attempt to incorporate other components in that particular comparison study.

Q K. UCHIDA, Japan

I think the influence of soil interaction of higher mode may depend upon both the type of vibration model and the soil stiffness used in the model. For example in case of complicated vibration model like nuclear power plants composed of various vibration elements, the natural modes with respect to soil motion may often be computed in the higher modes as you mentioned in your paper. So in these cases the higher modes may be important. However in case of the high rise buildings, tall chimneys like cantilever type, I consider only the first few modes may be important for the response calculation. What do you think of my opinion ?

A T. H. LEE, U. S. A.

Prof. Tajimi's opinion was on the analysis of a cantilever-type structure. It is quite possible that for this particular type of system, interaction effects on the higher modes are truly negligible. The consideration of the first few modes for analysis of high-rise buildings appears to be justifiable.

Q K. AKINO, Japan

With respect to flexible structures, especially tall buildings, we have experimental data in Japan for their natural periods. If we suppose that the first period is unity, the second is approximately 1/3 and the third is approximately 1/5, and those numbers correspond to natural periods of shear mode vibration modes of a cantilever beam. Therefore, it can be said that vibration modes of tall buildings are independent of the soil interaction. How do you think whether Japanese have to reconsider the influence of structure-ground interaction upon the response of higher modes, as you pointed out in your paper, (the second paragraph from the bottom in page 9)? However, we agree with your opinion for rigid structures such as the reactor buildings. (This question is turned to me from Dr. Tajimi).

A T. H. LEE, U. S. A.

The interaction effects depend, in a complicated way, on the parameters in the system. Naturally, they will be influenced by the inertia property of the structure. The experimental data in Japan are for cantilever-type systems whose masses are relatively small in magnitude as compared to those used in a reactor model. I must emphasize here that the interaction effects on higher modes can be significant in unsymmetric three-dimensional systems. Where the response of structural masses depends upon the rotations as well as translations of the system. The mode shapes of such unsymmetric systems differ appreciably from those used by Prof. Tajimi in his paper.

As I remember, I have never given any opinion for rigid structures because the model developed at Gulf General Atomic is for flexible structures. This must be a misunderstanding.