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EARTHQUAKE ENGINEERING RESEARCH CENTER

SOIL STRUCTURE INTERACTION IN DIFFERENT SEISMIC ENVIRONMENTS

by

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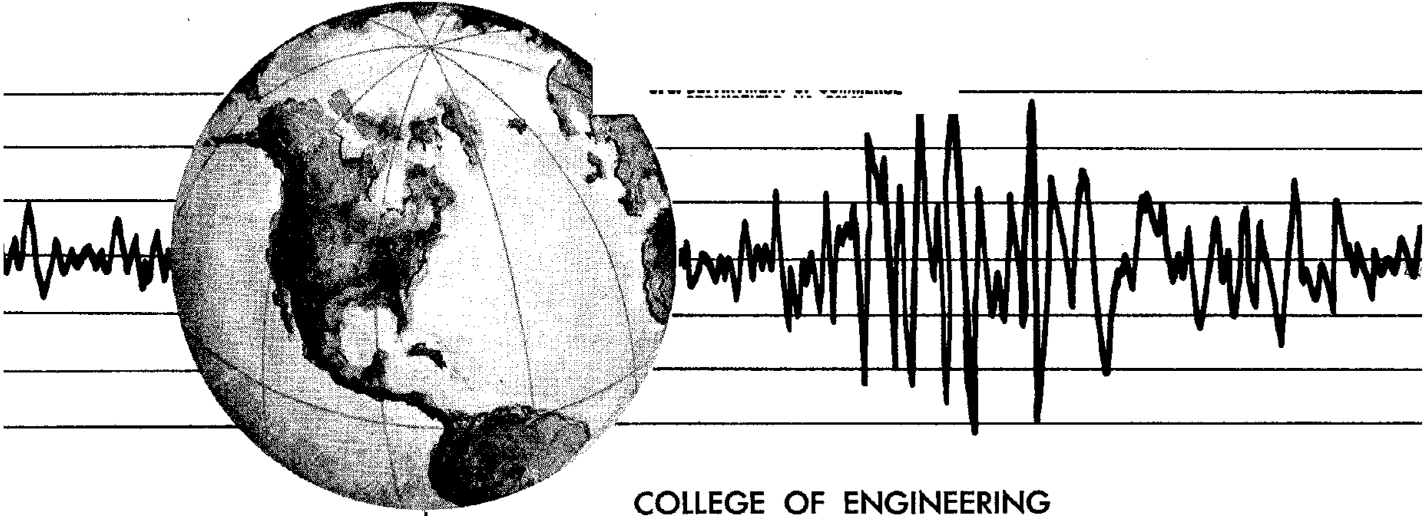

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Comparative analyses of the same structure are conducted for an input of R waves and for vertically propagating S and P waves in a rock site and sand site. In the rock site the R waves produce higher peak horizontal spectral acceleration up to 25% and a significant rocking effect at points away from the center of gravity of the structure. However, the S and P waves show a higher peak vertical spectral acceleration by up to 15% at the center of the structure. Very similar horizontal response, but higher vertical response only at the center of the structure for S and P waves, are obtained for the sand site.

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SOIL STRUCTURE INTERACTION IN DIFFERENT SEISMIC ENVIRONMENTS

By

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INTRODUCTION

The current strong interest in nuclear power and the concerns regarding the seismic safety of the facilities involved has generated the development of improved methods of seismic soil-structure interaction analysis. A complete analysis of this problem consists of several parts. First, the seismic environment must be defined. Second, an analytical model must be designed for the soil-structure systems, and, third, the model must be analyzed by some effective and accurate numerical technique. Direct solutions of the complete interaction approach carried out by the finite element method for simplified seismic environments have been applied to a wide range of problems with different geometries (Kausel and Roesset, 1974; Seed et al, 1975; Lysmer et al, 1975).

However, earthquake motions result from a complex pattern of body and surface waves whose nature and magnitude will depend on factors such as the fault rupture mechanism, the focal depth, the regional geology, the epicentral distance and the local soil conditions. The control motion for seismic

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analysis of nuclear power plants is usually specified by the U. S. Nuclear Regulatory Commission in the form of a broad-band acceleration spectrum. A time-history of ground accelerations is then developed which produces the desired spectral shape. No requirement is made concerning the nature of the wave systems producing these motions. Thus at the present time the assumption is often made that soil motions are primarily due to vertically propagating shear waves and compression waves. On the basis of this assumption, analytical techniques have been developed to calculate the motions everywhere in the free field based on one-dimensional wave propagation theory and the use of equivalent linear soil modeling techniques for a viscoelastic layered system. Results obtained by this approach have been found to be in good agreement with field observations of ground response (Schnabel, 1972), and soil-structure interaction (Valera, 1977), during actual earthquakes.

However, some authors, e.g., Wong and Trifunac (1974), Luco (1976), have argued that consideration of oblique body waves and surface waves may produce results significantly different from those obtained by assuming vertically propagating waves. In fact, low frequency surface waves have been observed in several earthquakes such as El Centro, 1940 (Trifunac, 1971), Parkfield, 1966 (Anderson, 1974), Koyna, 1967 (Singh et al, 1975) and San Fernando, 1971 (Hanks, 1975).

Methods for approximating the effects of horizontally propagating waves on structures and earth dams have been proposed by Scavuzzo (1967), Dezfulian and Seed (1969), Dibaj and Penzien (1969), Udaka (1975), Scanlan (1976), Werner et al (1977) and others. All of these methods assume either specified traveling base motions or use theories involving a uniform half space and extremely simple wave forms.

It appears, therefore, that there is a need for both a better determination of the seismic environment in layered soil systems for use in design studies and also for methods of soil-structure interaction analysis capable of handling a wider range of input motions. As a step in this direction an analysis method has been developed to accept any type of plane strain body waves or Rayleigh waves or a combination of these as input free field motions. This method of analysis makes use of viscoelastic finite elements, solves the quilibrium equations following the complex response method, and uses the equivalent linear method to approximate the non-linear material behavior. In addition, this method includes the use of transmitting boundaries to simulate the existence of semi-infinite multilayered free field deposits, the use of transmitting and/or viscous boundaries in the direction prependicular to the plane of analysis to simulate 3-dimensional effects in the ground and the use of viscous boundaries to model the half-space underlying the soil-structure system.

A method is also presented for calculating R-wave motions and oblique body wave motions in a horizontally layered free field. These different wave fields may be superimposed to produce a more generalized seismic environment.

Finally, two soil-structure interaction analyses are presented herein which assess the difference in response produced by an input consisting of Rayleigh waves and by a combination of vertically propagating shear and compression waves on the same given structure in a rock site and in a sand site, respectively.

METHOD OF ANALYSIS - A TWO STEP PROCEDURE

The proposed method of analysis computes the total motions by superimposing the free field motions and the soil-structure interaction motions. The superposition technique has been described by Clough and Penzien (1975), Gómez-Massó (1978), and Lysmer (1978) for the analysis of discretized systems and it has been used by Aydinoglu et al (1977) to approximate the modal behavior of buildings resting on an elastic soil layer over a half-space and subjected to harmonic excitations.

A schematic representation of the model with the input and radiated waves considered in the present method of analysis is shown in Fig. (1). The theory presented here refers to plane-strain finite-element models consisting of three regions. A central zone within which elements of irregular shapes can be used, and two adjacent free-field regions. Each one of the "blocks" next to the structure-and-soil model in Fig.(1) represents a numerical boundary condition to account for the dissipation of energy in the form of waves.

A diagram of the global method of solution is presented in Fig. (2). The system to be analyzed is represented by the structure and the surrounding soil (SSS). All materials are assumed to have viscoelastic properties and the analysis is carried out in two steps. The finite-element system, SSS, is decomposed into two finite-element models, namely the free-field system, FFS, and the incremental system, NET, as shown in Fig. (2). The NET system has material properties resulting from subtraction of those of the FFS system from those of the SSS system. The first step of analysis is Step A—The Free-Field Analysis— in which the temporal and spatial variation of the seismic motions, u_f, in the FFS model are determined. The second step is Step B—The Finite-Element Analysis—in which the interaction motions, u_i, caused by the presence of the structure are calculated. Once the interaction motions are obtained, the total displacements, u, for the complete interaction analysis of the SSS

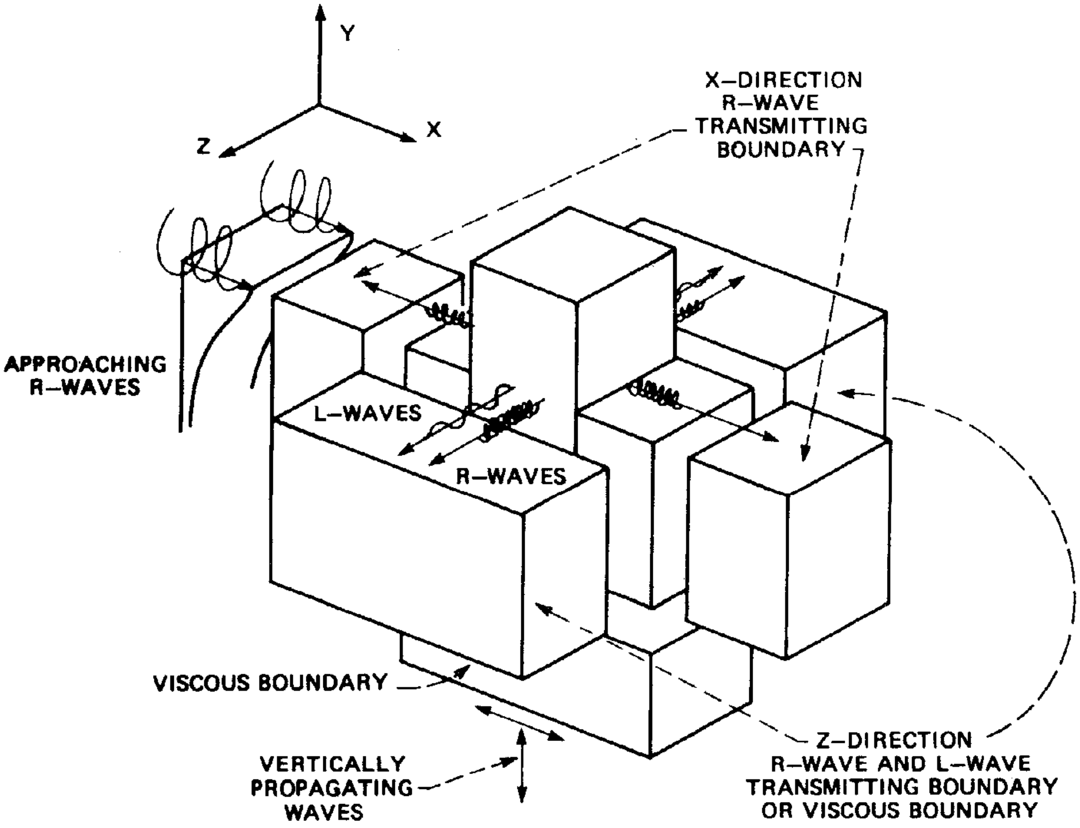

FIG. 1 MODEL FOR INCIDENT AND REFLECTED SEISMIC WAVES

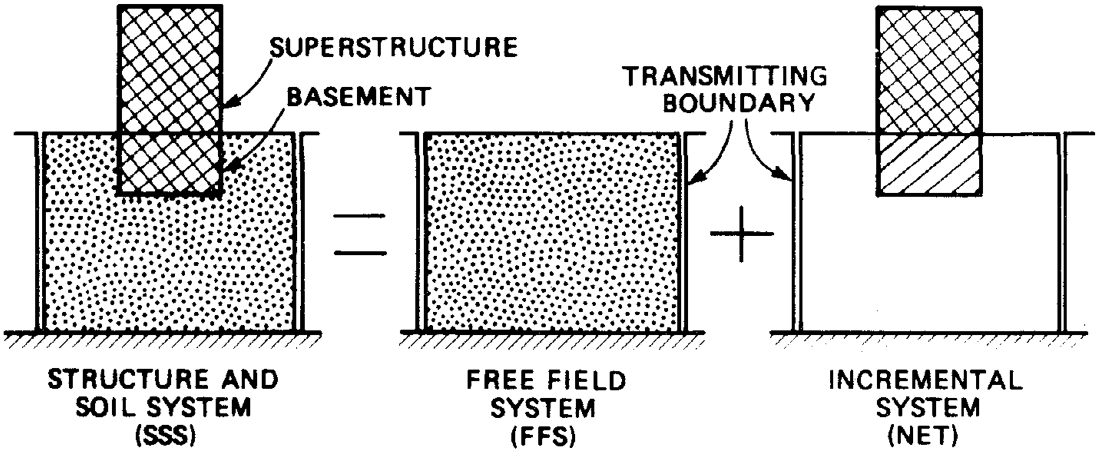

FIG. 2 SUPERPOSITION STAGES FOR COMPLETE ANALYSIS

model are obtained by the following superposition:

$$u(x, y, t) = u_f(x, y, t) + u_i(x, y, t) \qquad (1)$$

where all displacements in Eq. (1) are absolute in the sense that they refer to the same fixed set of coordinate axes.

The governing equations in Steps A and B are solved by means of the method of complex response extended to transient motions, together with complex stiffness expressions to allow for frequency independent material damping. Material nonlinearities are modeled using the equivalent linear method (Seed and Idriss, 1969) in both Steps A and B.

COMPUTATION OF FREE FIELD MOTIONS

The types of waves considered are inclined or vertically-propagating S and P waves and horizontally traveling R waves. The free field consists of a plane-strain system of homogeneous linearly viscoelastic layers overlying a homogeneous viscoelastic halfspace. Because of limitations in the present state of the art the computations are developed only for horizontally layered sites.

Let the control motion $\ddot{d}(t)$ consist of a seismic accelerogram recorded on top of the ℓ^{th} layer. If the record has N points digitized at equal time intervals, Δt, i.e. $d(j\Delta t)$, $j = 0,1,2,\ldots N-1$, then it can be expressed in the form of a finite Fourier series of N/2 + 1 harmonics as follows:

$$\ddot{d}_j = \mathrm{Re} \sum_{s=0}^{N/2} \ddot{D}_s \exp(i\omega_s t) \qquad j=0,1\ldots N-1 \qquad (2)$$

where ω_s is the circular frequency of each harmonic, $\omega_s = \frac{2\Pi s}{T}$, for $s = 0,1,2...N/2$; $T = N\Delta t$ is the duration of the earthquake and $\ddot{D}_s$ are the complex amplitudes.

The free field displacements u_f can be expressed by superimposing the individual harmonic components as a finite Fourier series of the form

$$\{u_f\} = \mathrm{Re} \sum_{s=0}^{N/2} \{U_f\}_s \exp(i\omega_s t) \tag{3}$$

where the free-field complex amplitudes, U_f, are calculated in the frequency domain and are given by:

$$\{U_f\}_s = \{P\}_s \ddot{D}_s \tag{4}$$

$\{P\}_s$ is the vector of the free-field amplification functions also known as transfer functions for each frequency.

Equation (4) expresses free field motions determined by one control motion. However, in cases where both S and P waves propagate simultaneously in the free-field or both horizontal and vertical components of the Rayleigh wave motions at the control point are known, then the control motions will consist of a horizontal acceleration time history, $\ddot{d}^h(t)$, and a vertical acceleration time history $\ddot{d}^v(t)$. If both of these time histories have the same duration and time interval Δt, then Eq. (4) will be replaced by:

$$\{U_f\}_s = \{P_f^h\}_s \ddot{D}_s^h + \{P_f^v\}_s \ddot{D}_s^v \tag{5}$$

The computation of the free-field transfer functions is different for S and P waves than for R waves, but once this step is completed, the rest of the computations are identical in all cases. The computation of transfer functions for the different types of waves is presented in the following sections.

Site Transfer Functions for Body Waves - The equation of motion for a discretized n-layer system assuming linear variation of displacement within each layer, is as follows:

$$\left([K] - \omega^2 [M]\right) \{\phi\} = \{{}^{Q}_{F}\} \qquad (6)$$

in which [K] and [M] are the global stiffness and mass matrices of order 2n + 2. The vector $\{\phi\}$ contains 2n + 2 normalized complex displacement amplitudes for the layers and the boundary, F is the load vector consisting of the last two boundary forces between the layered system and the half-space. For the case of inclined body waves the stiffness matrix can be decomposed into three parts:

$$[K] = [A]\ k^2 + [B]k + [G] \qquad (7)$$

where k is the complex wave number defining the phase velocity and attenuation factor of horizontal propagation. For the case of vertical incidence, i.e. $k = 0$, the matrices [A] and [B] drop out of this equation, and the S wave and P wave are completely uncoupled.

The normalized boundary forces and the boundary displacement amplitudes are calculated by using the theory for waves obliquely incident to a boundary between two media. Then, by solving Eq (6) for each frequency the rest of the complex amplitudes ϕ are determined, and from these, the transfer functions are readily obtained (Chen, 1979). The numerical examples presented herein are restricted, however, to the case of vertically incident S and P waves.

Site Transfer Function for R Waves - The free field is treated as a continuum in the horizontal direction but discretized into a finite number, n, of semi-infinite layers underlain by a rigid base, as shown in Fig. (3). Each layer is modeled with two nodal points each having two degrees of freedom, namely the horizontal and vertical displacements. The equilibrium equation for a harmonic R wave is written as the following complex eigenvalue problem (Waas, 1972):

$$\left([A]k^2 + i[B]k + [G] - \omega^2[M]\right)\{V\} = \{0\} \tag{8}$$

in which $i = \sqrt{-1}$, matrices [A], [B] and [G] are formed with the damping and stiffness properties of the layers, [M] is the global mass matrix, and {V} contains the 2n complex displacement amplitudes at the layer interfaces. For a given ω , Eq. (8) can be solved for the 2n possible eigenvalues, (wave numbers) k_s, and the corresponding eigenvectors (mode shapes), $\{V\}_s$, s = 1,2,...2n.

The nodal point displacements can be expressed as the sum of the contributions of the different mode shapes as follows:

$$\{u\} = \sum_{j=1}^{2n} \alpha_j \{V\}_j \exp(i\omega t - ik_j x) \tag{9}$$

where α_j are unknown mode participation factors. Equation (9) represents a superposition of generalized R-waves each with its own mode shape and wave number. For a viscoelastic system the wave numbers are complex with negative imaginary parts, since these waves decay as they travel in the positive X-direction. In general, it is not possible to find these mode participation factors. However, if it is assumed that the fundamental mode dominates the response at all frequencies, Eq. (9) reduces to:

$$\{u\} = \alpha_1 \{V\}_1 \exp(i\omega t - ik_1 x) \qquad (10)$$

where α_1, $\{V\}_1$ and k_1 correspond to the fundamental mode. If the control motion is specified at the top of the j-th layer, α is given for each frequency by:

$$\alpha_1 = \ddot{D}_1 / V_{1j} \qquad (11)$$

and hence

$$\{P\} = \{V\}_1 \exp(-ik_1 x)/V_{1j} \qquad (12)$$

The computation of free field R or body wave motions is carried out by the computer program SITE (Chen, 1979).

COMPUTATION OF THE INTERACTION MOTIONS

Assuming that the energy dissipation in the 3rd direction (the Z-direction) or through the system base occurs only for non-zero interaction displacements, u_I, the finite element equilibrium equation of the SSS system in Fig. (2) can be written as follows:

$$[M]\{\ddot{u}\} + [E]\ \{u_i\} + [K]\ \{u\} = \{Q\} \qquad (13)$$

where [M] and [K] are the global mass and stiffness matrix, respectively. The vector {Q} represents the loading forces on the boundaries of the system, and [E] is the generalized matrix accounting for the energy dissipation in the Z-direction and through the system base, and will be discussed later. Likewise, the dynamic equilibrium equation of the discretized FFS model, shown in Fig. (2) is the following:

$$[M_f]\{\ddot{u}_f\} + [K_f]\{u_f\} = \{Q\} \qquad (14)$$

where $[M_f]$ and $[K_f]$ are the global mass and stiffness matrix, respectively, and the load vector is the same as in the SSS model.

Substitution of Eqs. (1) and (14) into Eq. (13) leads to the following expression in u_i:

$$[M]\{\ddot{u}_i\} + [E]\{u_i\} + [K]\{u_i\} = \{f\} \qquad (15)$$

where

$$\{f\} = -\left([M_n]\ \{\ddot{u}_f\} + [K_n]\ \{u_f\}\right) \qquad (16)$$

and

$$[M_n] = [M] - [M_f] \qquad (17)$$

$$[K_n] = [K] - [K_f] \qquad (18)$$

in which the n subscript refers to the NET system. The magnitude of the interaction load vector {f} depends on the free field motions and the properties of the NET system.

The so-called "transmitting boundaries" are dynamic stiffness matrices which have been successfully used to represent mathematically the semi-infinite free-field layers and allow a size reduction of the mesh for analysis with the subsequent savings in computation time (Lysmer and Waas, 1972; Kausel et al, 1974; Lysmer et al, 1975). Therefore, this technique is also used in this study and transmitting boundary matrices are assembled with the stiffness matrix for the central block in Fig. (1) to form the complete global stiffness matrix.

The complex response method together with complex stiffness expressions is used to solve Eq. (15) for any given transient free field motions. The interaction displacements can be expressed in the following form:

$$\{u_i\} = \mathrm{Re} \sum_{s=0}^{N/2} \{u_i\}_s \exp(i\omega_s t) \tag{19}$$

By expressing Eq. (15) in terms of finite Fourier series and substituting the above equation one obtains:

$$\mathrm{Re} \sum_{s=0}^{N/2} \left(-\omega_s^2 \, [M] + [E^*] + [K^*] \right) \{U_i\}_s \exp(i\omega_s t) =$$

$$= \mathrm{Re} \sum_{s=0}^{N/2} \{F^*\}_s \exp(i\omega_s t) \tag{20}$$

where

$$\{F^*\}_s = \left(\omega_s^2\ [M_n] - [K^*]\right)\{U_f\}_s \tag{21}$$

and the asterisk,*, indicates a matrix, with complex elements.

Expression (20) can be written in terms of the complex amplitudes U_i for one particular frequency

$$\left(-\omega_s^2\ [M] + [E^*] + [K^*]\right)\{U_i\}_s = \{F^*\}_s \tag{22}$$

Upon solution of Eq. (22) for the amplitudes, U_i, the total motions can be calculated as follows:

$$\{u\} = \mathrm{Re} \sum_{s=0}^{N/2} \left(\{U_f\}_s + \{U_i\}_s\right) \exp(i\omega_s t) \tag{23}$$

The computer program CREAM (Gómez-Massó, 1978) has been developed to accept an arbitrary seismic input, calculate the interaction motions and obtain the total motions by superposition according to this equation.

<u>3-D Simulation - Viscous Boundaries vs Transmitting Boundaries</u> - As shown in Fig. (1), the input R-waves travel in the X-direction and the radiated surface waves travel in the X- and Z-directions, whereas the incident and reflected body waves are contained in the X-Y plane. If a flexible base in considered, body waves will also propagate through the underlying half-space. The wave energy reaching the free field boundaries in the X-direction can be absorbed by R-wave transmitting boundaries.

At the base the viscous boundaries developed by Lysmer and Kuhlemeyer (1969) can be used to absorb the energy reaching the halfspace.

Viscous boundaries may also be used to absorb the radiated energy in the Z-direction assuming that this energy is dissipated in the form of plane shear waves (Lysmer et al 1975). The dynamic stiffness matrix for the boundary can be expressed as follows:

$$[E^*] = \frac{i\omega}{H} \quad [C] \qquad (24)$$

where $i = \sqrt{-1}$, ω is the frequency, [C] is the diagonal matrix of damping coefficients and H is the thickness of the structure in the Z-direction.

A theoretically more attractive boundary condition is the use of L- and R-wave transmitting boundaries to absorb also the energy propagated in the Z-directions. For a given frequency such boundaries can be expressed as follows:

$$[E^*] = \frac{2}{H} \sum_{j=1}^{m} \Delta X_j \left([L]_j + [R]_j \right) \qquad (25)$$

where $[L]_j$ and $[R]_j$ are the L and R-wave boundary matrices developed by Waas and Lysmer (1972) for a soil profile corresponding to the j-th vertical column of nodal points. All of the matrices $[L]_j$ and $[R]_j$ will be similar except for the position of the terms in the $[E^*]$ matrix. ΔX_j is the average width of the elements adjoining the j-th column of nodal points.

It is interesting to compare the viscous and the transmitting boundaries used for 3-D simulation. For this purpose two comparative analyses were carried out using the model shown in Fig. (4) and a synthetic accelerogram with $a_{max} = 0.25g$. First, the L-wave boundaries were compared with the horizontal viscous boundaries using a seismic input consisting of vertically propagating shear waves. Second, the R-wave boundaries were compared with the vertical viscous boundaries using an input of vertical P waves. The results obtained using the viscous and the transmitting boundaries show peak accelerations within 5% to 7% and spectral acceleration curves within 5% to 15% in all cases as shown in Fig. (3). It thus appears that the viscous boundaries are a good approximation to the more realistic L- and R-wave transmitting boundaries. Viscous boundaries have the advantages of being more economical, not requiring vertical nodal point columns and allowing a viscous base to be considered in the model. Therefore, the viscous boundaries were used for 3-D simulation in all subsequent analysis presented herein.

SURFACE WAVES VS. VERTICALLY PROPAGATING WAVES IN A ROCK SITE

The results of a soil-structure interaction analysis using a combined excitation of S and P waves were compared to the results of an analysis obtained for an input of consisting only of Rayleigh waves. This latter case may be considered an extreme case, since no strong motion seismic environment is likely to consist entirely of a Rayleigh wave field, but it provides a limiting bound in assessing the significance of this type of motion.

<u>Computational Model</u> - The soil-structure finite element model used is shown in Fig. (4). The material properties of the structure were typically those of steel and reinforced concrete. The free field consisted of two rock layers of 50 ft and 320 ft in thickness with shear wave velocities of about 3600 fps and 5600 fps, respectively and with a damping ratio of 2%.

The control motion used in this analysis was a synthetic record with a peak acceleration scaled to 0.25 g, and having a spectrum similar to that specified by the Nuclear Regulatory Commission. This control motion was also used to calculate the free-field S and P waves. The input of combined in-

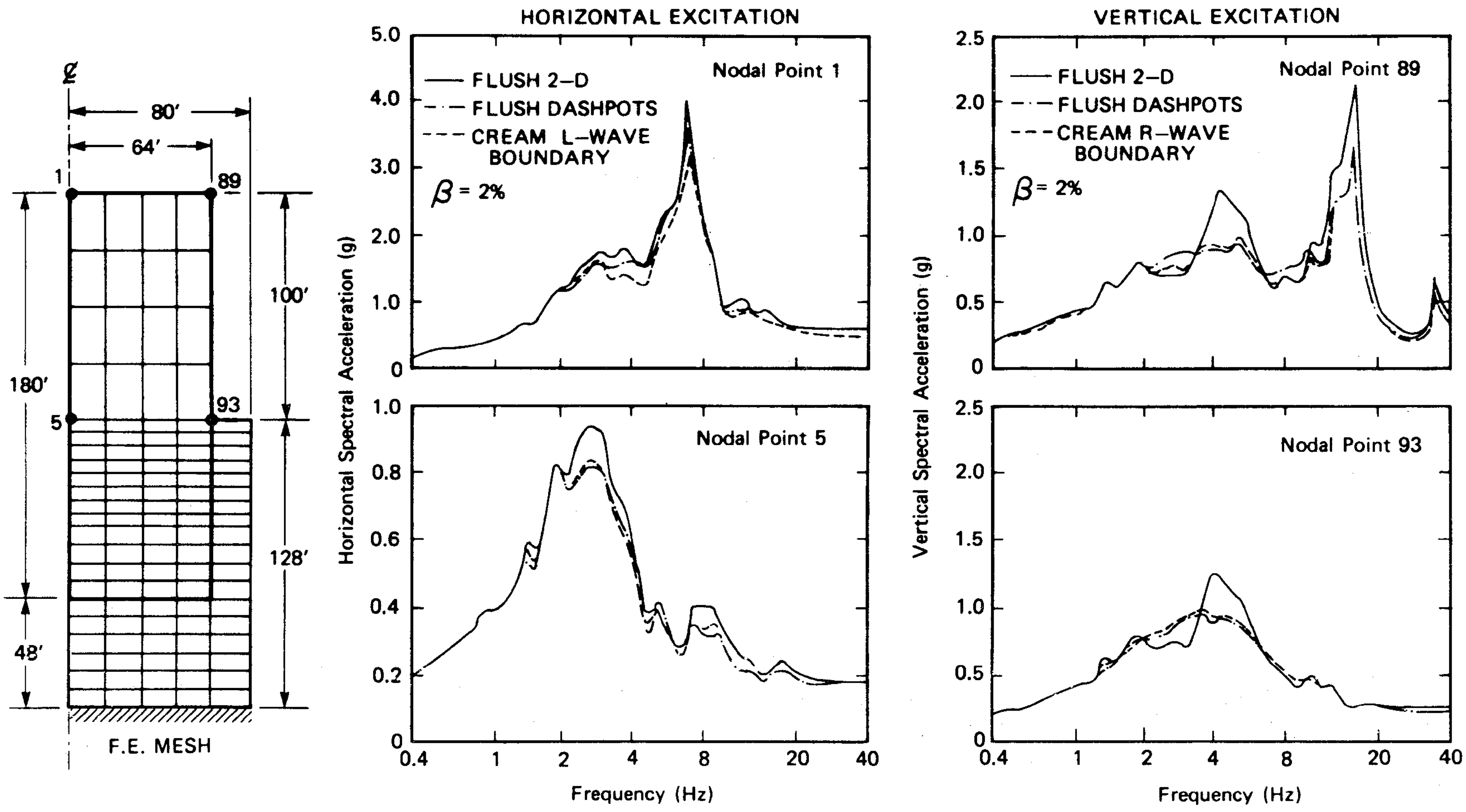

FIG. 3 FINITE ELEMENT MODEL AND COMPARISON OF RESPONSE SPECTRA FOR DIFFERENT METHODS OF 3-D SIMULATION

phase S and P waves represents the upper limit of body wave excitations and will be referred to as the S + P wave case.

R-Wave Free Field Motions for the Rock Site - The control motion for the Rayleigh wave analysis was the same as used above. It represents the horizontal surface component of R waves which travel from left to right. The location of the control point along the X-axis is of no practical significance because the surface waves attenuate slowly due to the low material damping of the rock. The control point was therefore placed at the free field location corresponding to nodal point G. Spectral acceleration curves for the free field motions at nodal points E, G, and I are shown in Fig. (4). The small attenuation in the X-direction observed is due to the very low damping ratio values of 2% used throughout the rock material. Vertical spectral curves are similar in frequency content to the corresponding horizontal curves but about 15% higher in magnitude.

Comparison of Response Using S + P Waves and R Waves - A comparison of the response of the structure subjected to combined S + P waves and R waves is shown in Figs (5) through (8). The maximum horizontal accelerations computed at points on the slab for both cases of analysis were identical. The strong similarity observed in the horizontal response spectra at points on the slab for the S + P wave and the R-wave cases is shown in Fig. (5).

The vertical response of the slab was, however, somewhat higher for the Rayleigh wave excitation and decreased in the direction of wave propagation. Maximum accelerations were about 20% higher for R-wave case. Vertical response spectra computed at the two ends of the slab, points E and I, and at

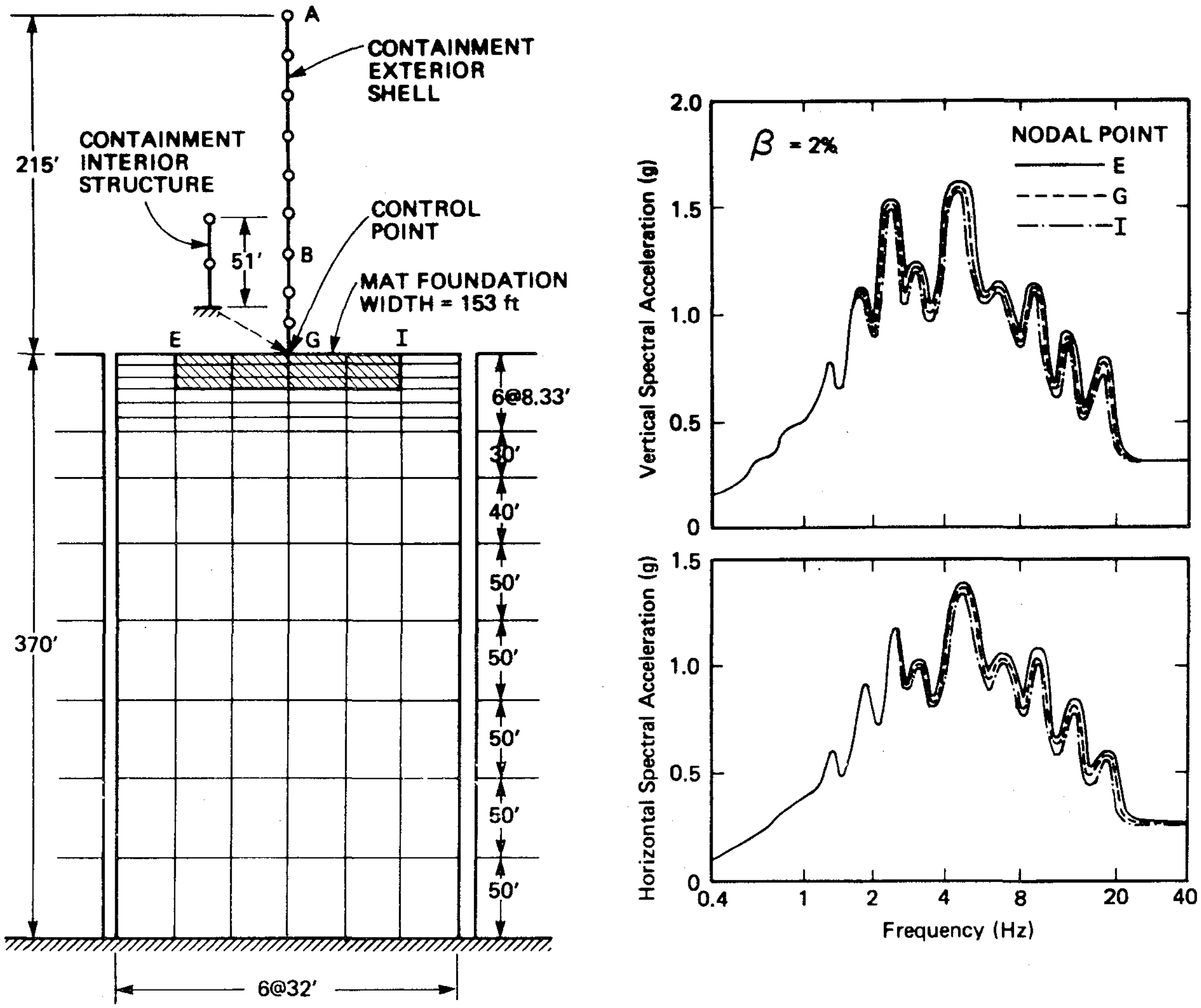

FIG. 4 FINITE ELEMENT MESH AND FREE FIELD R-WAVE MOTIONS FOR A ROCK SITE

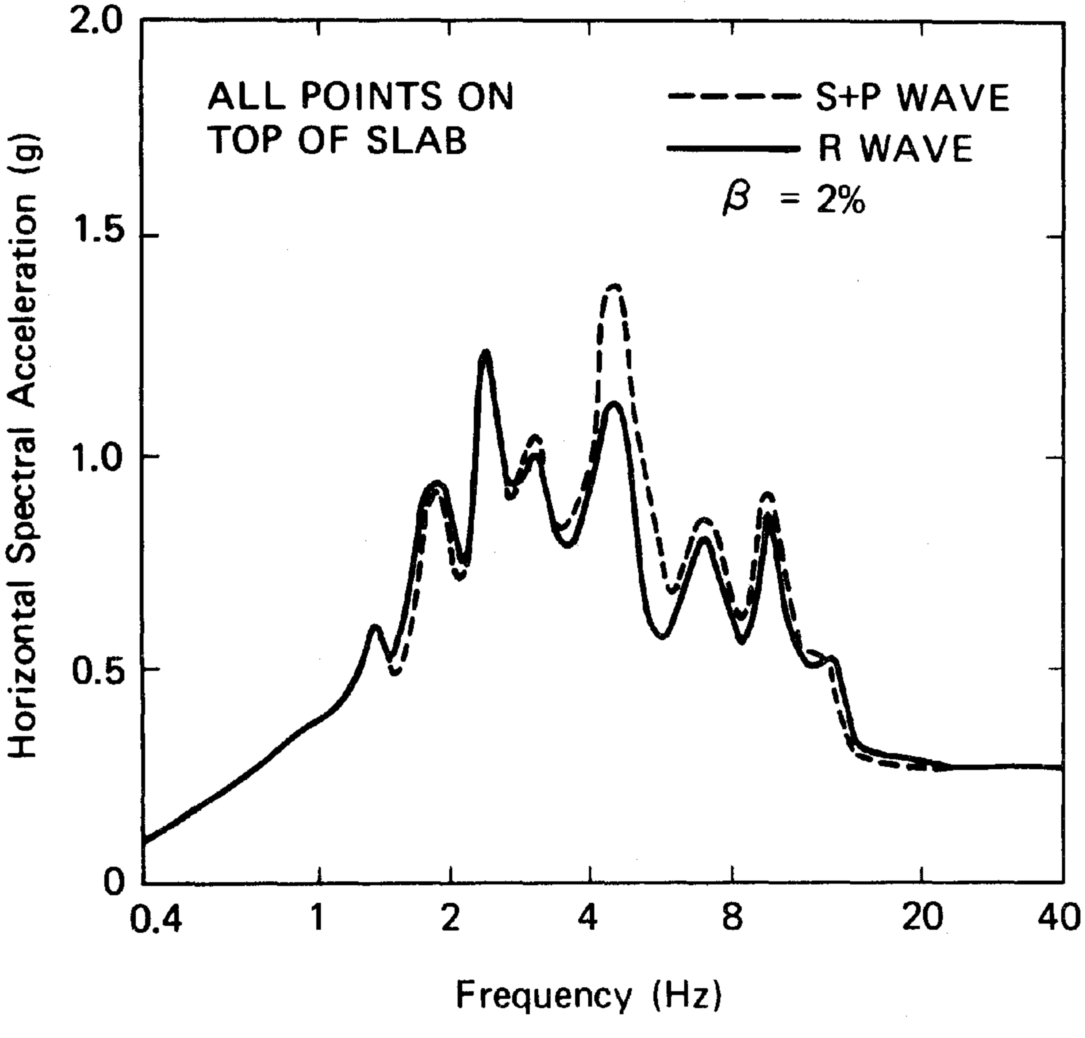

FIG. 5 HORIZONTAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (ROCK SITE)

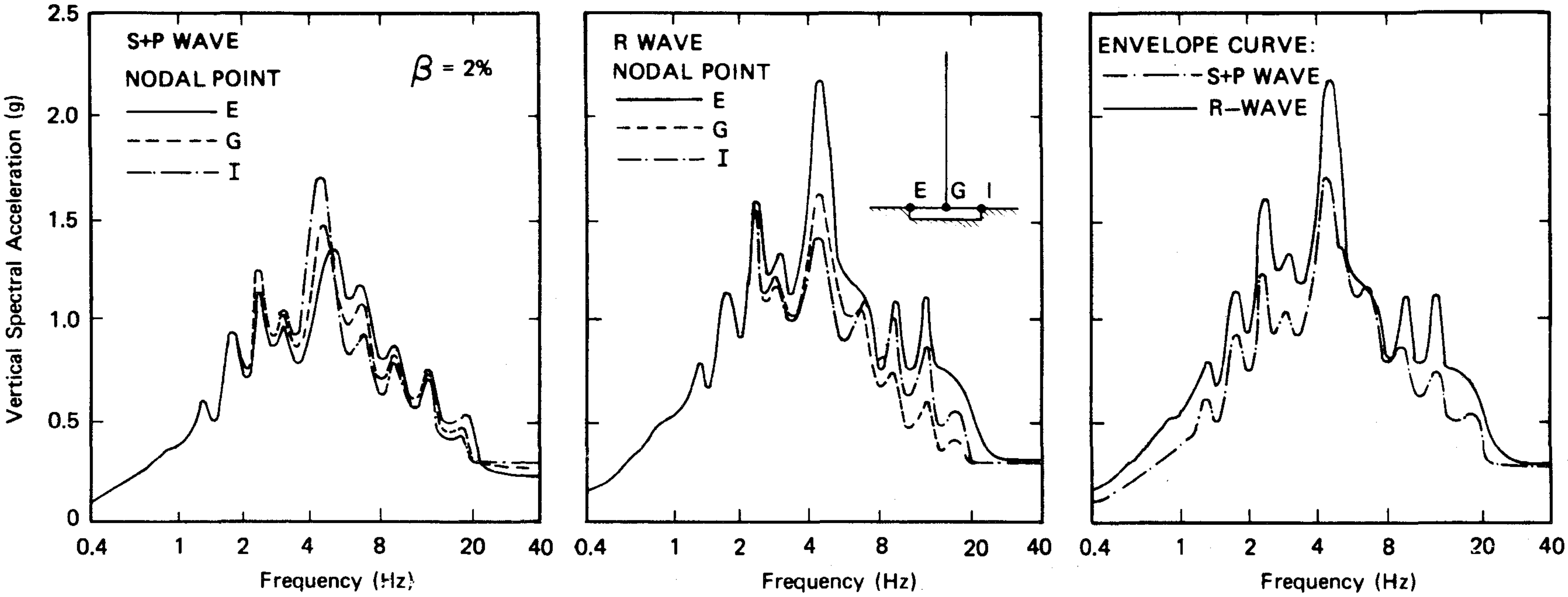

FIG. 6 VERTICAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (ROCK SITE)

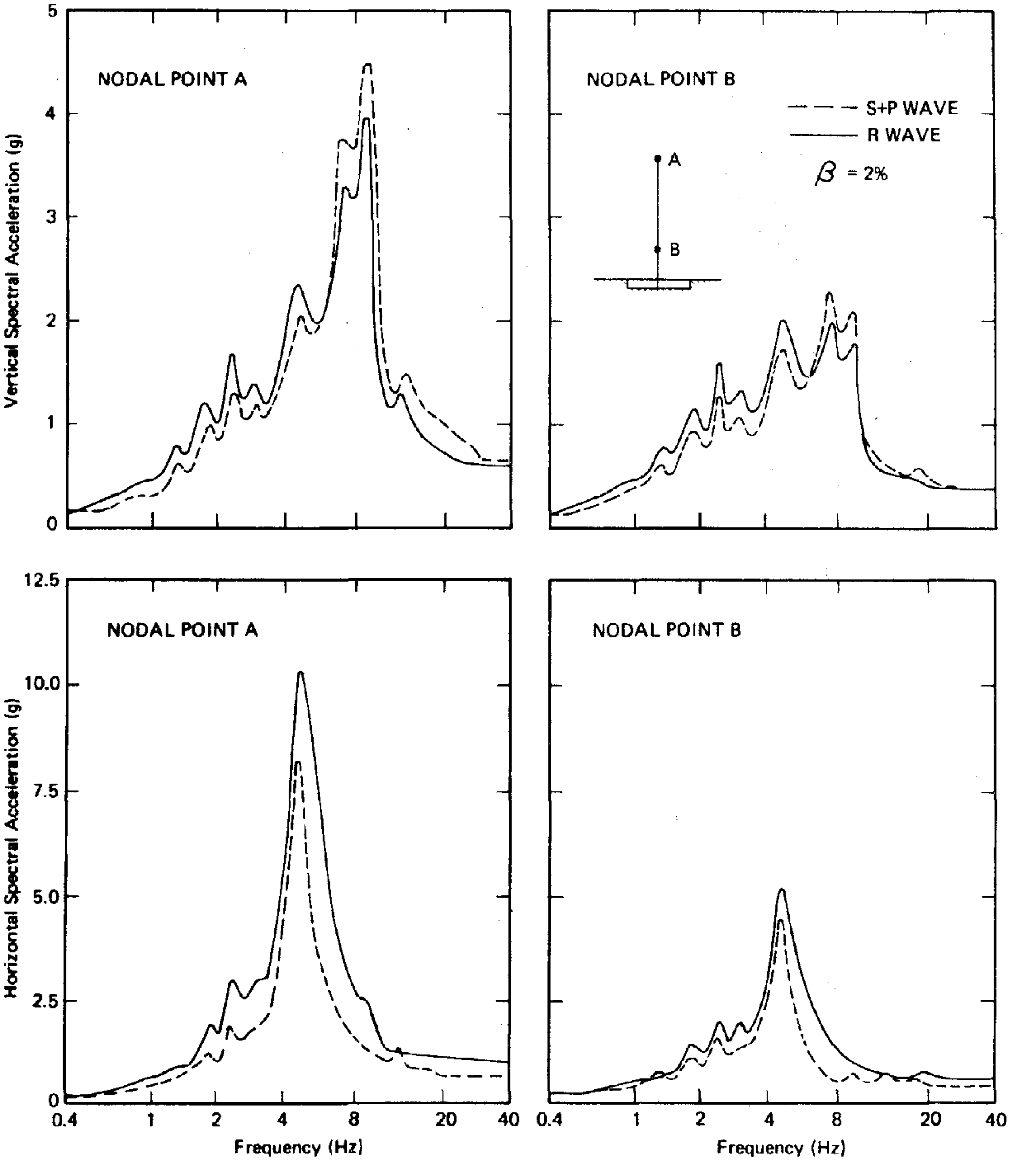

FIG. 7 COMPARISON OF RESPONSE SPECTRA AT NODAL POINTS A AND B (ROCK SITE)

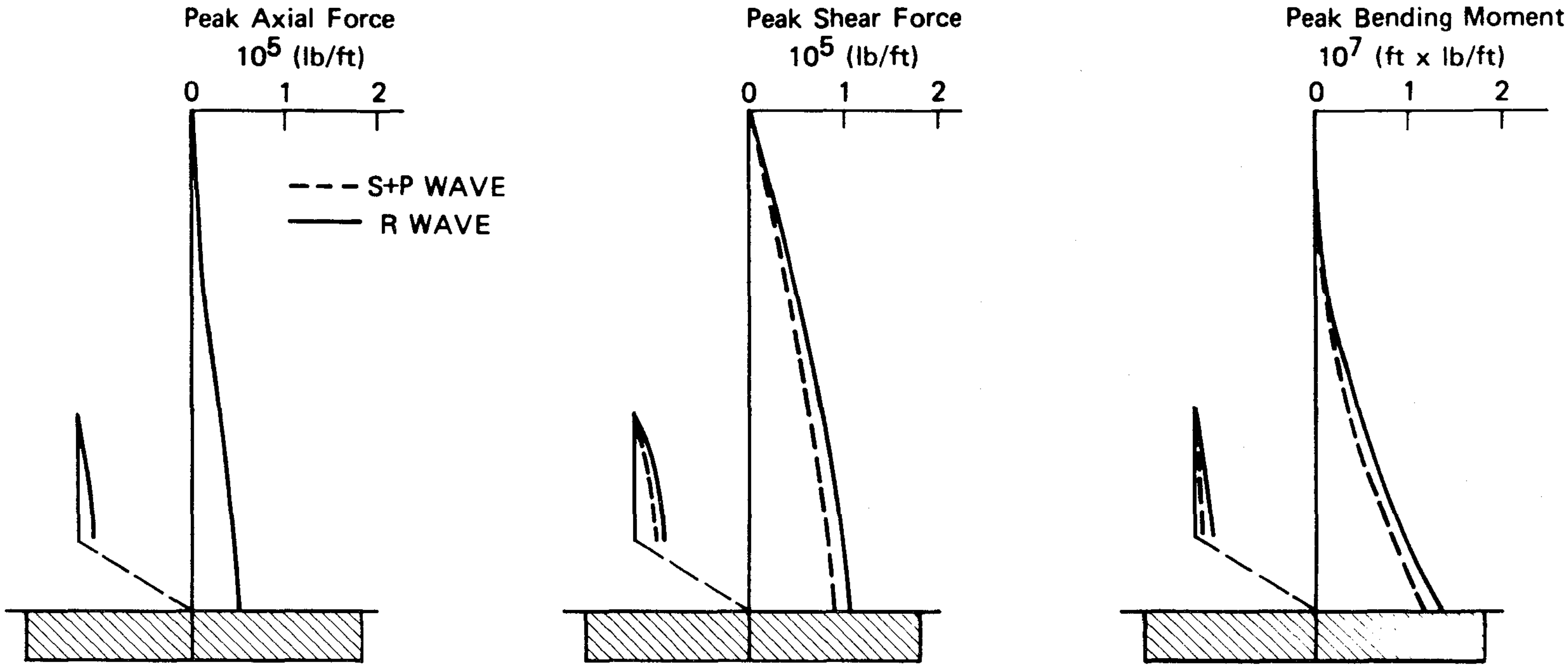

FIG. 8 COMPARISON OF BEAM ELEMENT MOMENTS AND FORCES (ROCK SITE)

the center of the slab, point G, for both excitations are shown in Fig. (6). The first and second plots in Fig. (6) show the comparison of the three spectra for the S + P waves and the R-waves, respectively. The difference in response spectra between the two ends at the peak frequencies was 25% for the S + P wave analysis vs. 60% for the R-wave case. This indicates a stronger rocking effect in the latter case. Further, since both R-wave and S + P wave motions can be input in two different directions in the soil-structure interaction analysis, the envelope of the response spectra for these two cases should be used rather than a single curve for the comparison of the response of nodal points away from the center of gravity of the structure. Differences between the envelope curves for vertical response of the slab shown in the third plot in Fig. (6) indicate that the Rayleigh wave analysis produced responses about 30% higher at the peak frequency than the S + P wave analysis.

Results obtained from the R-wave analysis for the vertical beam elements showed higher peak horizontal accelerations, by up to 50%, and higher horizontal response spectra. A comparison of the horizontal response spectra at the highest point in the beam, point A, and at a point at about one-third of the height, point B, is shown in Fig. (7). The R-wave spectra at points A and B were respectively 25% and 15% higher at the peak frequency. However, the vertical peak accelerations computed at the beam elements were higher for the S + P wave case by up to 22%. A comparison of the vertical response spectra at points A and B,also plotted in Fig. (7),shows values from the S + P wave analysis to be higher at frequencies above 6 Hz and by 15% at the peak frequency, whereas the opposite occurred at low frequencies.

The maximum beam bending moment and shear and axial forces are plotted in Fig. (8) where the results for the R-wave case are shown to be higher than those of the S + P wave case by up to 35%.

The effect of a wave field consisting solely of traveling R waves in a rock site would therefore appear to be more severe on some parts of the structures, than the effect of simultaneous S and P waves. In addition, the low attenuation observed in the spectral curves for the R-wave free field motion indicates that the location of the surface control point is unimportant for structures founded on materials with relatively high stiffness characteristics. Hence, this example analysis illustrates a case in which the results of design computations using a pure R-wave input motion are, for some parts of the structure, significantly different from those of an S + P wave analysis. Clearly, however, the significance of this result depends on the validity of the assumption that R waves constitute the primary component of the seismic environment.

Effect of Rigid Base of Finite Element Model - All of the above calculations were obtained using a finite element model with a rigid base at a depth of 370 ft below the ground surface. However, the R-wave analysis was repeated using a viscous boundary at this depth in order to study the effects of possible reflections at the rigid boundary on the interaction motions. The structural responses computed by the two methods were virtually identical and it may thus be concluded that reflections from the rigid base of the finite element model are unimportant for practical calculations.

SURFACE WAVES VS. VERTICALLY PROPAGATING WAVES IN A SAND SITE

Soil structure interaction analyses were also conducted for S + P wave input and for R-wave input for the purpose of assessing the influence of the traveling waves in a shallow, relatively soft soil layer resting on a very stiff rock halfspace.

<u>Computational Model</u> - The idealized geometry of the site is shown in Fig. (9). The structure had the same dimensions and material properties as the model previously used for the rock site. The soil consisted of a 128 ft thick layer of homogeneous dense sand overlying the bedrock. The soil properties were those typical of dense sand to a depth of 48 ft and of a very dense sand for the remaining soil.

The control motion was a time history with a peak acceleration of 0.25g and a spectral form similar to that of the NRC Regulatory Guide. Contrary to the analysis of the rock site previously studied, at this site the location of the control point was found to be of crucial importance for the the frequency dispersion and significant material damping of the sand layer. Therefore, in order to make a meaningful comparison between the R-wave and S + P wave effects the control point was located at a distance of 200 ft., away from the center of the structure to allow for some motion decay in the sand deposit and the free field horizontal and vertical R-wave motions were calculated at the center of the slab and then used as control motions for the S + P wave analysis.

<u>R-Wave Free Field Motions for Sand Site</u> - The particular configuration of this site, which consists of a relatively soft soil layer underlain by a much stiffer rock mass, implies R-wave modes which at low frequencies show much higher horizontal components than vertical components. The characteristics of the horizontal and vertical free field motions of the R-wave field which

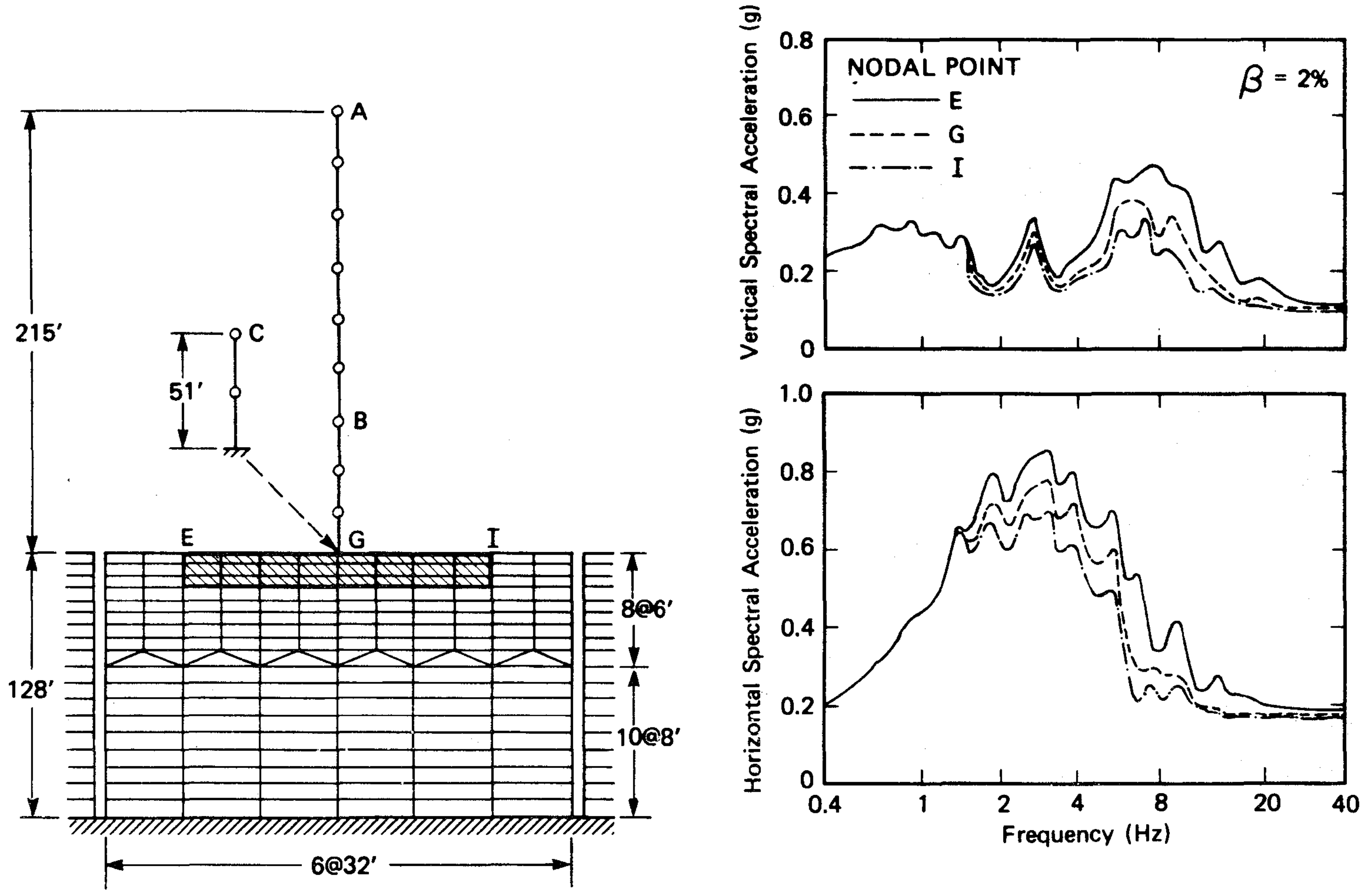

FIG. 9 FINITE ELEMENT MESH AND R-WAVE FREE FIELD MOTIONS FOR A SAND SITE

propagates from the left to the right are shown in Fig. (9) for nodal points E, G and I. As can be seen the higher damping and relative softness of the site as compared to the rock site previously studied produces a remarkable motion attenuation with distance in the direction of wave propagation.

Comparison of Response Using S + P Waves and R Waves - A comparison of the response of the structure under the effects of S + P waves and R waves is presented in Figs. (10) through Fig. (13). All peak horizontal and vertical accelerations were within 5% or 10% in both cases. The horizontal response spectra at points on the slab were very similar for both cases of excitation as is shown in Fig. (10).

Some rocking oscillations of the concrete slab were observed in both analyses. The difference between the peak vertical accelerations at points on the slab was within 20%. The vertical response spectra at the ends and the center of the slab are shown in Fig. (11). The first and second plots in Fig. (11) show the response spectra at these three points on the slab as obtained for the S + P wave case and the R-wave case, respectively.

The first plot indicates that the highest peak in the response spectra for the two ends of the slab were of about the same magnitude for the S + P wave analysis, while the second plot shows that the R-wave analysis produced differences in those peaks of about 35%, indicating some attenuation effect. However, comparison of the envelope curves of the vertical spectra at the slab ends presented in the third plot shows the R-wave envelope to be higher by about 10% at frequencies less than 1.5 Hz. The S + P envelope was higher by about 20% at frequencies between 1.5 and 8 Hz.

Horizontal and vertical spectral curves obtained in both analyses for the

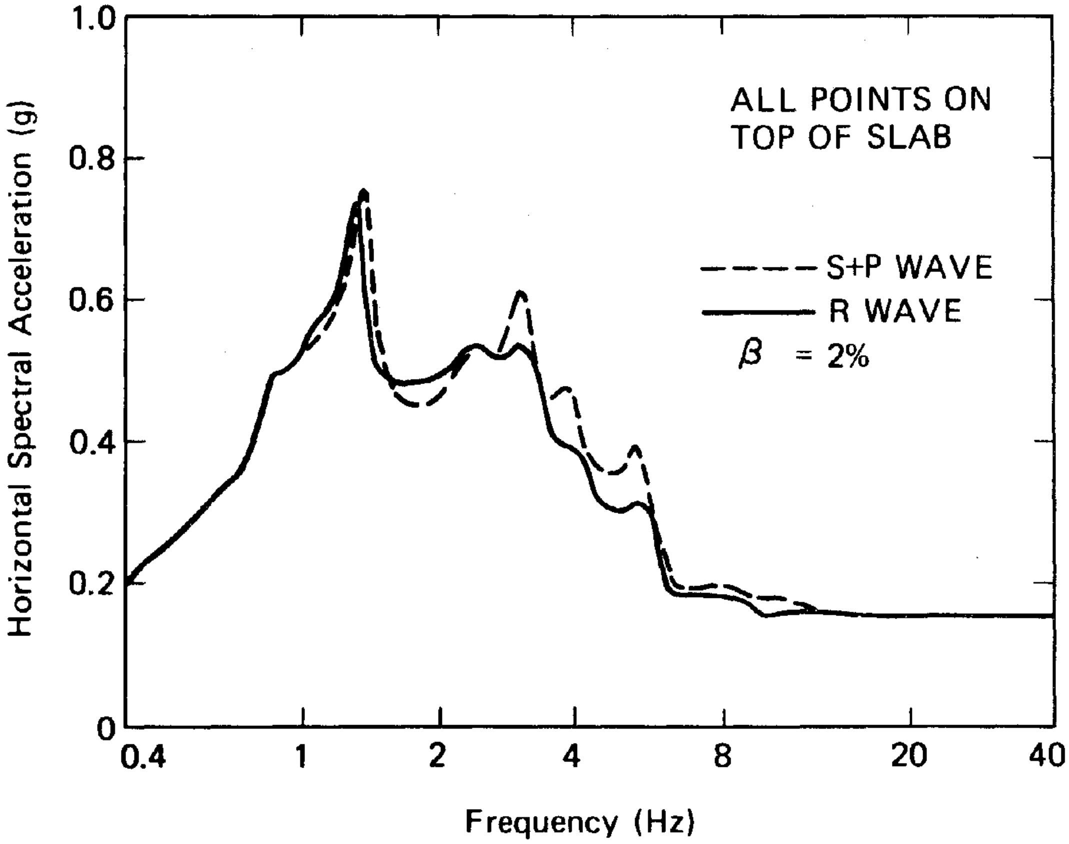

FIG. 10 HORIZONTAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (SAND SITE)

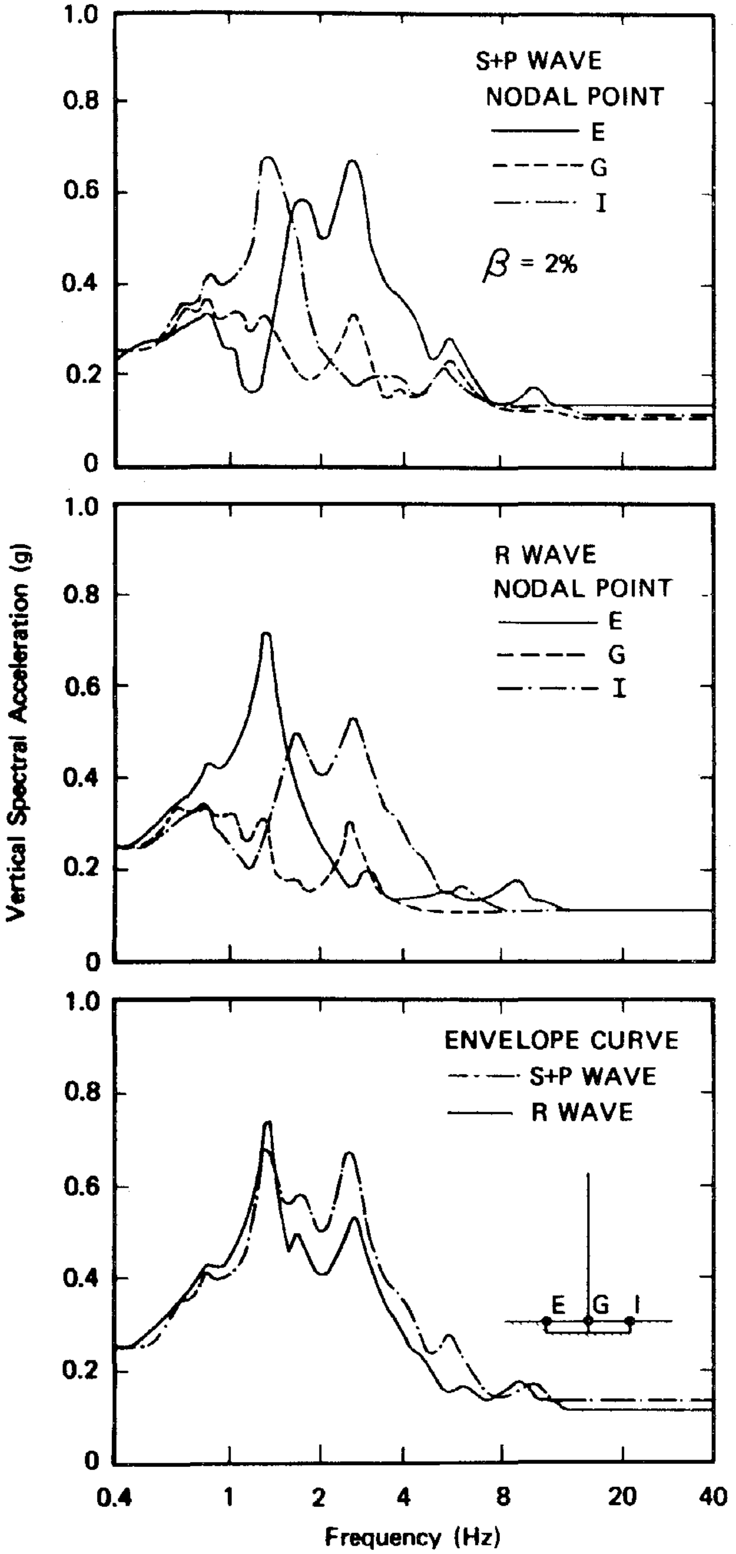

FIG. 11 VERTICAL RESPONSE SPECTRA ALONG THE TOP OF THE SLAB (SAND SITE)

vertical beam element are plotted in Fig. (12). The horizontal response spectra differ by less than 15% at peak frequencies. The vertical response spectra showed very similar shapes in the low frequency range. At frequencies higher than 4 Hz, the R-wave analysis yielded practically no response. However, in the 4 Hz to 15 Hz range, the S + P wave analysis showed the high response peaks which were nonexistent in the R-wave results.

The maximum beam bending moments and shear and axial forces calculated for the R-wave and S + P wave analyses are shown in Fig. (13). These results indicate that the differences between the two cases were within 5% or 10%.

The main characteristics of the free field R-wave motions in the sand site were, first that the particular soil profile configuration consisting of a shallow sand layer overlaying bedrock prevents the propagation of low frequency vertical motions which seem to be a primary factor in the overall rocking motion of structures. Second, the material properties of the sand produce a remarkable motion attenuation in the direction of wave propagation. The results of analyses using S + P and R wave fields are not significantly different. Therefore, this case illustrates a situation in which the assumption of vertically propagating waves would be entirely adequate for design purposes of stiff heavy structures with shallow embedment even if Rayleigh waves were in fact the primary source of excitation.

SUMMARY AND CONCLUSIONS

Presented herein is a method for soil-structure-interaction analysis valid for a completely arbitrary seismic excitation in a plane-strain geometry with an approximation for 3-dimensional effects. This method is carried out in two steps. In the first step, the free field motions are calculated. In

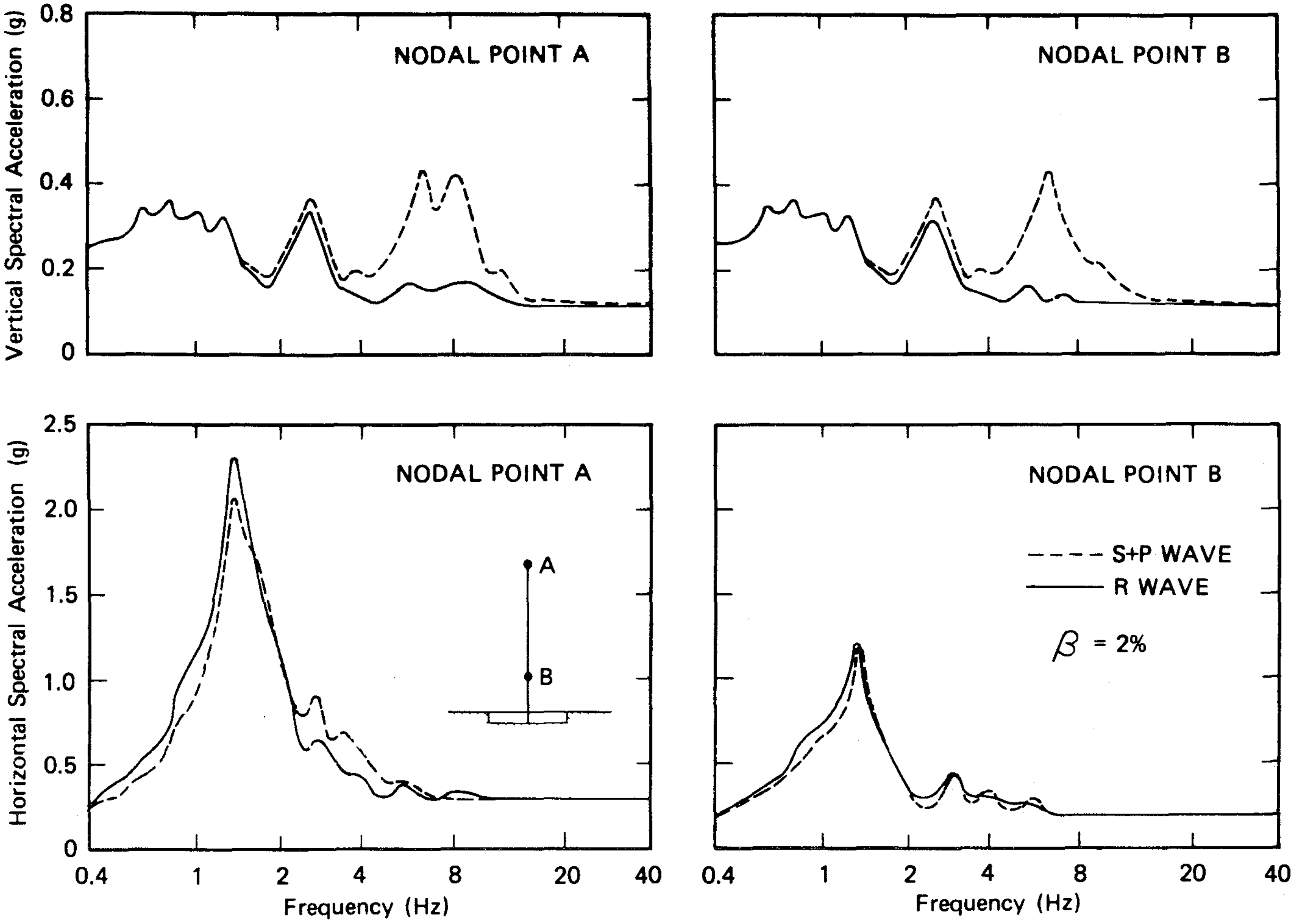

FIG. 12 COMPARISON OF RESPONSE SPECTRA AT NODAL POINTS A AND B (SAND SITE)

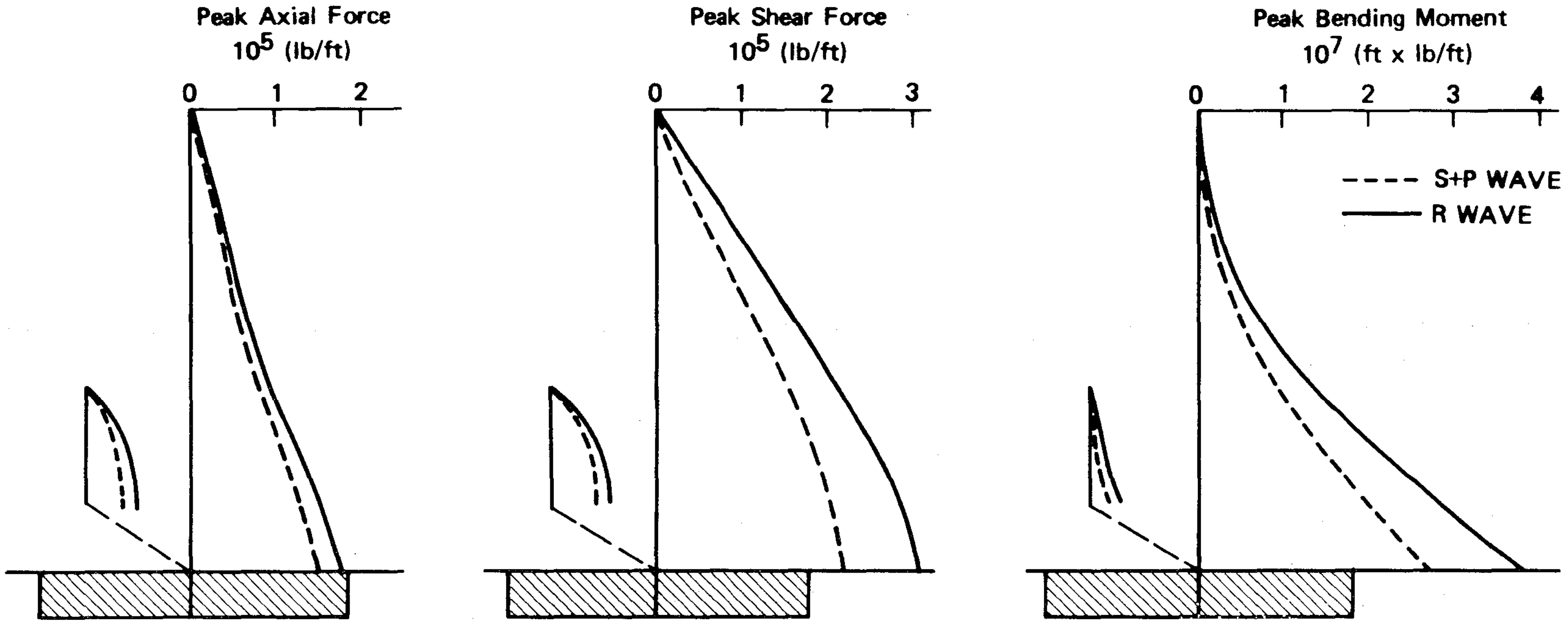

FIG. 13 COMPARISON OF BEAM ELEMEMT MOMENTS AND FORCES (SAND SITE)

the second step, the interaction motions are calculated and superimposed on the free field motions in order to obtain the total motions. Strain compatibility is achieved by using the equivalent linear method.

The method of solution described above uses some of the most efficient techniques currently available to produce a high quality and reasonably economic soil-structure interaction analysis. This method can be easily extended to other geometries such as inclined free field layers, once the required theories to determine free field motions become part of the state-of-the-art, and also to a truly 3-dimensional geometry.

The main conclusions of this study are the following:

1. Soil structure interaction problems with an arbitrary seismic environment can be solved by use of the complex response method and the superposition principle developed in this paper.
2. An important aspect of any analysis is the selection of a realistic seismic environment. This environment must satisfy the equations of motion for the free field, and may consist of both vertically propagating and horizontally propagating seismic waves.
3. A seismic environment which is composed only of Rayleigh waves may produce higher response of a shallow-embedment structure built in rock, than a seismic environement formed only be vertically propagating S and P waves.
4. Rayleigh wave effects are relatively unimportant for the design of rigid, shallow-embedment structures built in a shallow layer of sand overlying rock.
5. The 3-dimensional ground simulation by a 2-dimensional model may be

achieved by use of viscous boundaries or the more exact transmitting boundaries. However, the two methods give nearly identical results and the simpler viscous boundaries are therefore adequate for practical analyses.

6. The energy reflections from the bottom rigid boundary of a soil-structure finite element model have only a minor effect on the computed seismic response of the structure and need not be considered in most cases. In any event their effects can be eliminated by incorporating a transmitting boundary at the base of the finite element mesh.

All of the above conclusions are based on a comparison of two extreme load cases: a system composed entirely of vertically propagating body waves and a system composed entirely of horizontally propagating Rayleigh waves. Neither of these cases are likely to occur in nature. In reality, Rayleigh waves have not been observed in the frequency range above 1 or 2 Hz. On the other hand, calculations have shown that the seismic environments produced by slightly inclined body waves are very similar to those produced by vertically propagating waves. It, therefore, seems reasonable to conclude that soil-structure interaction response analyses based on the assumption of vertically propagating body waves provides an appropriate design procedure for most practical purposes.

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APPENDIX II
NOTATION

The following symbols are used in this paper:

A, B = stiffness components of the free field;

C = dashpot coefficients;

D = complex amplitude of the control motion in the frequency domain;

d = control motion in the time domain;

E = matrix accounting for energy dissipation in the Z-direction and through the system base;

F = equivalent load vector in the soil-structure system;

G = stiffness component of the free field;

H = thickness of the structure in the Z-direction;

K = stiffness matrix;

k = wave number;

L = L-wave transmitting boundary;

M = mass matrix;

m = number of vertical columns of nodal points in the finite element mesh;

N = number of points in a digitized accelerogram ;

n = number of layers in the free field;

P = free field transfer functions;

Q = load vector in the soil-structure system;

R = Rayleigh wave trnasmitting boundary;

T = duration of the earthquake;

t = time;

u = displacements;

U = complex amplitude of seismic displacements;

V = free field eigenvectors for Rayleigh waves;

x, y, z = space coordinates;

α = free field mode participation factor for R-waves;

ϕ = free field normalized displacement amplitudes for body waves;

ν = earthquake frequency (Hz);

ω = earthquake frequency (rad/sec)

Subscripts

f = free field;

i = interaction

n = net properties;

s = corresponds to the s^{th} frequency

Superscripts

h = horizontal;

v = vertical;

$*$ = complex finite element matrix;

$\ddot{}$ = acceleration

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