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SEISMIC SAFETY MARGINS RESEARCH PROGRAM
(Phase I)
Project III - Soil-Structure Interaction

THE SEISMIC SOIL-STRUCTURE INTERACTION PROBLEM FOR NUCLEAR FACILITIES

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April 1980

Lawrence
Livermore
Laboratory

This work was supported by the United States Nuclear Regulatory Commission under a Memorandum of Understanding with the United States Department of Energy.

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FOREWORD

This report is one of a number of reports on the same topic, namely, a general review of soil structure interaction, as part of the scope of work on the Seismic Safety Margins Research Program (SSMRP). Because of the controversial nature of the subject, reports were solicited from a wide range of individuals familiar with the topic, and they were encouraged to present <u>their</u> views and positions. Accordingly, the material in this report is the responsibility of the authors and the reader should not imply that this report represents the views or positions of the SSMRP staff.

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by

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SEISMIC SAFETY MARGIN RESEARCH PROGRAM

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Part I

GENERAL CONSIDERATIONS

The basic problem of soil-structure interaction is illustrated in Fig. 1. It involves the determination of the motions of one or more structures at a given site from a knowledge of a given motion (the control motion) at a specified point (the control point) of the site prior to construction (the free field).

A complete soil-structure interaction analysis for any structure must necessarily consist of two distinct parts; a site response analysis and an interaction analysis. Unless the nature of the seismic wave field into which the structure is being placed is known with a reasonable degree of accuracy, there is no way in which the resulting interaction of the structure with the soil deposit and the wave field can be determined.

The site response analysis involves the determination of the temporal and spatial variations of the free-field motions. The interaction analysis involves the determination of the motions of a structure placed in the above seismic environment. These are different types of problems but each needs to be addressed to determine a solution to the soil-structure interaction problem.

Each of the above problem types can in principle be formulated in terms of continuum models or discretized models, and it is not possible here to describe all of the possible forms of equations of motion which have been proposed. It is, however, useful for a better understanding of the nature of and the connection between the two problem types to consider the equations of motion for the three linear models shown in Fig. 2. The models are

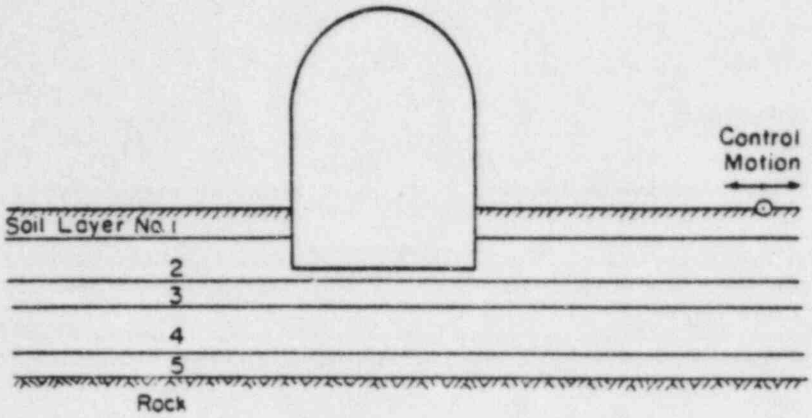


Fig. 1 SOIL STRUCTURE INTERACTION PROBLEM

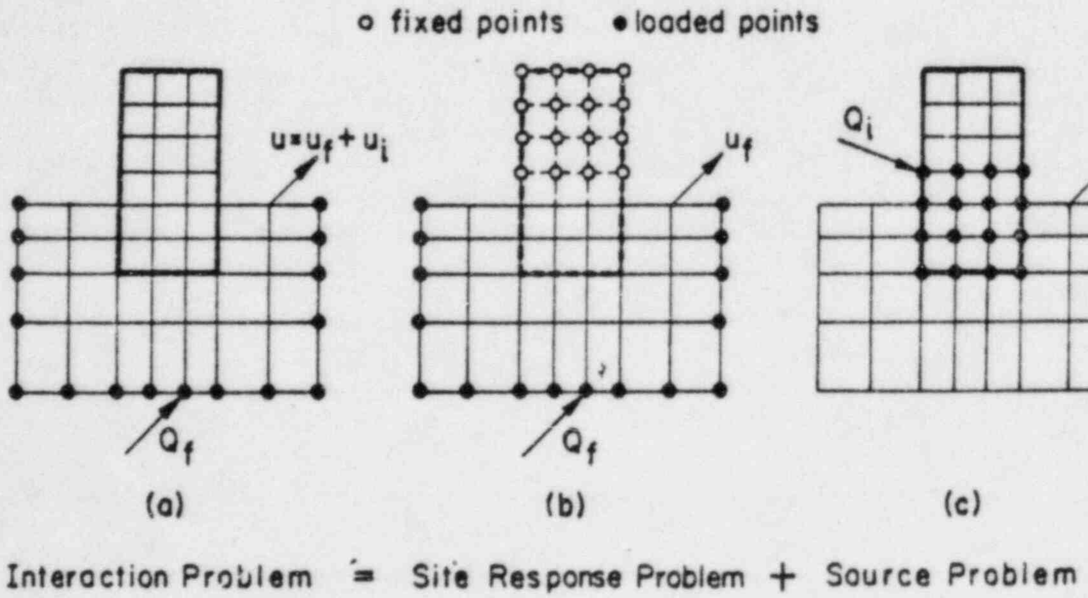


Fig. 2 SUPERPOSITION THEOREM FOR INTERACTION PROBLEMS

identical in the sense that all are of the finite element type and all are spanned by the same finite element mesh. Also, all masses and stiffnesses are the same, except that the structural part of the model shown in Fig. 2(b) has no stiffness and mass, and that for this model the structural nodes above ground level are assumed to be fixed in space (actually these points can be given any specified motion without loss of generality).

Since the fixed nodes have no influence on the motion of the ground Fig. 2(b) represents a free-field site response problem. It has the equation of motion

$$[M_f]\{\ddot{u}_f\} + [C_f]\{\dot{u}_f\} + [K_f]\{u_f\} = \{Q_f\} \tag{1}$$

where $[M_f]$, $[C_f]$, $[K_f]$ are the mass, damping, and stiffness matrix, respectively, for the free field, and $\{u_f\}$ is a vector containing the nodal point displacements. Since the source of excitation is outside the model the load vector $\{Q_f\}$ has non-zero elements on the external boundary only. Solutions to the equation of motion, Eq. (1), can be obtained by the methods described in Part II of this report and it will here be assumed that a free-field solution is available. Thus $\{u_f\}$ and $\{Q_f\}$ are known.

Figure 2(a) represents the corresponding interaction problem. The total displacements can be written

$$\{u\} = \{u_f\} + \{u_i\} \tag{2}$$

where $\{u_f\}$ are the known free-field displacements and $\{u_i\}$ are the interaction displacements. Assuming that the external boundary is very far away from the structure the equation of motion for the interaction problem is

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = \{Q_f\} \tag{3}$$

where $\{Q_f\}$ is the same load vector as in Eq. (1) and [M], [C], and [K] are

the total mass, damping, and stiffness matrices, respectively. Substitution of Eqs. (1) and (2) into Eq. (3) yields

$$[M]\{\ddot{u}_i\} + [C]\{\dot{u}_i\} + [K]\{u_i\} = \{Q_i\} \tag{4}$$

where

$$\{Q_i\} = ([M_f] - [M])\{\ddot{u}_f\} + ([C_f] - [C])\{\dot{u}_f\} + ([K_f] - [K])\{u_f\} \tag{5}$$

The load vector, $\{Q_i\}$, in Eq. (5), can be computed from the known free-field displacements. It depends only on the difference in properties between the structure and the excavated soil. Thus $\{Q_i\}$ has non-zero elements only at the structure and Eq. (4) is the equation of motion for the source problem illustrated by Fig. 2(c). This problem is well-posed and can be solved for the interaction displacements, $\{u_i\}$. The total displacements for the soil-structure interaction problem can be found by superposition as indicated by Eq. (2).

Equations (2) and (4) remain valid even as the distance to the boundary goes to infinity and the mesh size shrinks to infinitesimal dimensions. Hence, the above formulation can be extended to continuum mechanics and three dimensions. A similar substructure formulation for the interaction problem is given in Part III of this report, see Fig. 4.

The above formulations reveal three important characteristics of the soil-structure interaction phenomenon:

1. The only free field ground motions which are of importance for the interaction phenomena are those within the volume to be excavated for the embedded part of the structure.
2. For the embedded structures the amount of interaction depends on the difference in mass and stiffness between the structure and the volume of excavated soil, see Eqs. (4) and (5).

3. Soil-structure interaction analysis implies in many cases the use of superposition, see Eq. (2). Thus true nonlinear analyses may, for many types of motion specification, not be possible.

The first observation has far reaching consequences; especially for embedded structures on relatively soft sites since, for such site, both theory and observation indicate that the free-field motions vary significantly with depth. This implies that the site response analysis is an important, and in the opinion of the writers perhaps the most important, part of a soil-structure interaction analysis.

The second observation is perhaps self-evident. However, it highlights the importance of embedment which for compensated (or nearly compensated) structures will tend to reduce the interaction effects. In buildings where total compensation is achieved, soil-structure interaction effects are likely to be insignificant.

The third observation may not be too important for nuclear structures which are not usually designed to accept large soil deformations and thus nonlinear effects can be handled by approximate methods. Furthermore, as will be discussed below, site response problems cannot in general be solved by the nonlinear methods. Therefore, it is in many cases esoteric to even consider a true nonlinear interaction analysis.

The considerations outlined above are discussed in detail in the subsequent five parts of this report:

Part II The Site Response Problem

Part III The Interaction Problem

Part IV Interaction with Body Waves

Part V Interaction with Rayleigh Waves

Part VI The Humboldt Bay Power Plant Case Study

and our conclusions are presented in Part VII. It is hoped that the general understanding of soil-structure interaction provided by this report will be enhanced to the point where such studies can be made with greater confidence by engineers concerned with the design of nuclear power plants. However it is also hoped that the present report will also be useful in dealing with all types of soil-structure interaction effects during earthquakes.

Part II

THE SITE RESPONSE PROBLEM

Site response problems involve the determination of the temporal and spatial variation of motions within a site. In principle, these motions can be determined from a large model which includes the source of the earthquake. However, in practice the source parameters and the regional geology cannot be determined in sufficient detail to solve this problem with a high degree of accuracy in the frequency range of interest for design. Thus, current methods of site response analysis normally attempt to predict the above variation of motions from a single specified control motion at some control point within the site. This problem is mathematically ill-posed and unique solutions can be obtained only by the introduction of restrictive assumptions regarding the geometry of the site and the nature of the wave field causing the control motion. In practice, consistent solutions can be obtained only for horizontally layered sites. Possible wave patterns include: vertically propagating or inclined plane body waves, and horizontally propagating surface waves. Only the case of vertically propagating waves can currently be solved by truly nonlinear methods.

CONTROL MOTION AND CONTROL POINT

The inherent problem in site response analysis is the choice of wave field to be used in the analysis and it is therefore natural to classify and discuss the different available methods according to the type of wave field assumed. However, before doing so it should be mentioned that the choice of an appropriate control motion and control point is just as, and

in most cases, more important than the choice of wave field.

The control motion should be chosen with due respect to observed relations between earthquake magnitude, epicentral distance, maximum acceleration, duration, frequency content, see Idriss (1978) and Ref. 1, and the site-dependent characteristics established by Hayashi et al. (1971), Seed et al. (1976a, 1976b), Faccioli (1978), and Ref. 1.

Except for the obvious case in which the control motion is an observed record at the control point, the preferable control point is a point either at the ground surface or, as discussed below, at an assumed rock outcrop at the depth of which the motion is specified. This is so because most of our data base of strong motion earthquake records from which the control motion has to be estimated was obtained at surface stations and, even more important, because the frequency content of motions at points below the ground surface is strongly influenced by reflections at the free surface. Thus the specification that the control motion at depth should be a broad-band spectrum or a motion recorded at another site or depth may result in completely unrealistic computed motions for the site.

At the present time, the control motion for the design of nuclear power plants is usually specified as a broad-band spectrum at the ground surface in the free field as shown in Fig. 1 or Part I.

With the control motion and control point fixed, the solution to the site response problem depends entirely on the nature of the wave field producing the ground surface motions. This wave field may consist of many components including:

(1) some Rayleigh waves

(2) some Love waves

(3) some plane vertically propagating waves

(4) some plane body waves inclined at an angle to the vertical

and (5) some other wave types such as spherical and cylindrical waves which are usually not considered.

At the present time seismologists cannot advise engineers in sufficient detail on the relative contents of the different possible wave forms which make up the surface motion. Thus, even though soil-structure interaction analyses can be performed for an arbitrary wave field, in practice, such analyses cannot be made due to lack of data on the characteristics of the wave field involved.

Under these conditions, a typical engineering approach is to make analyses for extreme cases of possible wave fields, e.g., for a motion represented by all Rayleigh waves or for a motion represented entirely by a system of body waves, and to determine the influence of the motion specification on the results of the analysis. If the differences are small, then precise specification of the components of the wave field is considered unnecessary. If the differences are large, then increasing efforts must be made to determine even a crude assessment of the relative components of different wave types, or alternatively, conservative choices of wave components may have to be made for different parts of the analysis. It is important therefore to examine the characteristics of the different wave forms which might contribute to the surface control motion.

HORIZONTALLY PROPAGATING WAVES

For horizontally layered sites it is relatively easy to set up linear methods of analysis for horizontally propagating waves. However, many possible choices of wave patterns exist (inclined body waves at different

angles of incidence, different modes of surface waves, etc.) and it is currently impossible to determine from available seismological data the exact contributions of each wave type to earthquake motions near the surface in the frequency range of interest to earthquake engineers. However, some estimates have been made (Trifunac and Brune, 1970; Randall, 1971; Chandra, 1972; Nair and Emery, 1975; Liang and Duke, 1977; and Toki, 1977). These estimates involve considerable uncertainties however; in view of these uncertainties analysis of site response for motions represented only by horizontally propagating waves is mainly an academic exercise based on assumed data. Nevertheless, as will be discussed below, some practical conclusions can be drawn from such analyses.

The free-field motions caused by horizontally propagating waves will be discussed in three parts: Surface waves are discussed immediately below, inclined body waves are discussed after the section on vertically propagating waves and, finally, the three wave types are discussed together in a section on motions at shallow depths.

Surface Waves

Rayleigh waves in a perfect elastic half-space are well-known and the theory for these are given in standard textbooks, e.g., Richart et al., (1970). However, for obvious reasons soil dynamics analysts are much more interested in surface waves in multi-layered systems, Thomson (1950), Haskell (1953), Ewing et al. (1957). Two types of waves may occur in such systems: Love waves, in which the motions are horizontal and perpendicular to the direction, X, of wave propagation, and Rayleigh waves which involve both vertical and horizontal motions in the vertical XZ-plane. For plane harmonic waves the displacement fields are of the form:

Love waves: $$u_y = \sum_{s=1}^{\infty} L_s \cdot h_s(z) \cdot e^{i(\omega t - c_s x)} \tag{1}$$

Rayleigh waves: $$\left\{\begin{aligned} u_x &= \sum_{s=1}^{\infty} R_s \cdot f_s(z) \cdot e^{i(\omega t - k_s x)} \\ u_z &= \sum_{s=1}^{\infty} R_s \cdot g_s(z) \cdot e^{i(\omega t - k_s x)} \end{aligned}\right\} \tag{2}$$

where ω and t are the frequency and time, respectively, and L_s and R_s are unknown mode participation factors. The infinite sets of wave numbers, c_s and k_s, and mode shapes, $h_s(z)$, $f_s(z)$, and $g_s(z)$, may in principle be determined by methods developed by Thompson (1950) and Haskell (1953). The wave numbers are directly related to the phase velocities of the different wave modes through $V_L = \omega/c$ and $V_R = \omega/k$. Thus the fundamental problem of site response analysis with surface waves is to determine the infinite set of factors L_s and R_s from a single given amplitude of the control point. This is clearly an ill-posed problem and solutions can only be obtained by further assumptions, the most common of which is to assume that only the fundamental Rayleigh or Love mode, corresponding to s = 1, exists. For undamped systems the frequency-dependent phase velocity and mode shape can be found by the Thomson-Haskell method and the amplitude L_1 or R_1 may be determined from the control motion.

Continuum analyses are possible for the case of viscoelastic layers over an undamped half-space (Ewing et al. (1957), Boncheva (1977)). However, for this case it is more practical to first discretize the semi-finite system by the use of finite elements as proposed by Lysmer (1970) and Waas (1972) for Rayleigh waves and Waas (1972), Lysmer and Waas (1972) for Love waves. Only Rayleigh waves will be discussed here. The theoretical

model is shown in Fig. 1. It involves the assumption of a linear variation of displacements between layer interfaces and the existence of a stationary rigid base at some finite depth. If this depth is chosen to be considerably larger than the wave length of the Rayleigh waves of interest, a half-space is simulated by this model.

For an N-layer system these assumptions reduce the equation of motion for the layered system to a quadratic eigenvalue problem:

$$([A]k^2 + [B]k + [C] - \omega^2[M])\{v\} = \{0\} \tag{3}$$

where [A], [B], [C] and [M] are simple 2N x 2N matrices which can be formed from the stiffnesses, damping ratios and mass densities of the layered system, and $\{v\}$ is an eigenvector (mode shape) which contains the 2N displacement amplitudes of the layer interfaces. The mode shape represents the functions $f_s(z)$ and $g_s(z)$ in Eq. (2). For a given frequency, ω, Eq. (3) can be solved by methods developed by Waas (1972). The solution consists of 2N possible wave numbers, k_s, and associated mode shapes $\{v\}_s$ and, in analogy with Eq. (2), the general solution to the equation of motion may be expressed in the form:

$$\{u\} = \sum_{s=1}^{2N} R_s\{v\}_s \cdot e^{i(\omega t - k_s x)} \tag{4}$$

For a damped system all the wave numbers will be complex with negative imaginary parts. Hence, Eq. (4) can also be written

$$\{u\} = \sum_{s=1}^{2N} e^{im(k_s)} \cdot R_s\{v\}_s \cdot e^{i(\omega t - Re(k_s)x)} \tag{5}$$

which represents a system of generalized Rayleigh waves (modes) which propagate in the positive x-direction, each with its own mode shape, $\{v\}_s$,

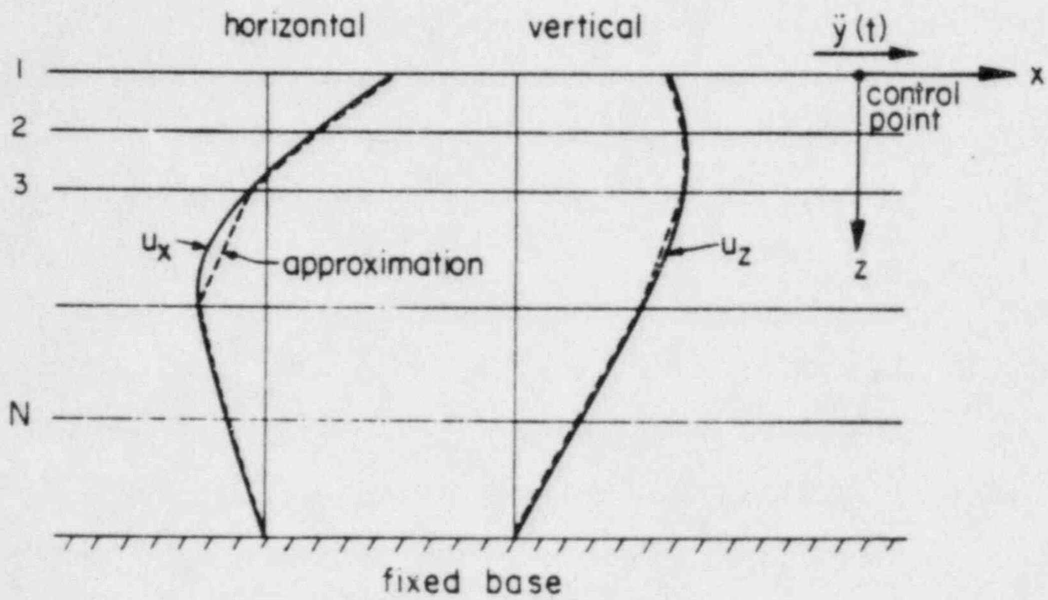


Fig. 1 TYPICAL SOIL PROFILE AND RAYLEIGH WAVE MODE SHAPE

phase velocity, $\omega/Re(k_s)$, and decay factor, $\exp[2\pi Im(k_s)/Re(k_s)]$, per wave length, $\lambda = 2\pi/Re(k_s)$. Experience with the method has shown that most of the Rayleigh modes decay extremely rapidly in the frequency range of interest to earthquake engineers and only a few terms of Eq. (4) need therefore be considered. If it is assumed that only the fundamental mode (defined as the mode with the largest value of $Re(k_s)$) is present, Eq. (4) reduces to

$$\{u\} = R_1 \cdot \{v\}\, e^{i(\omega t - k_1 x)} \tag{6}$$

and the mode participation factor, R_1, can be determined at each frequency from the amplitude of the control motion at say the surface at $x = 0$. This method has been used by Chen and Lysmer (1979) to determine possible Rayleigh motion fields for several sites.

Higher-order Surface Waves

The fundamental Rayleigh mode defined above is as shown by Lysmer (1970) identical to the fundamental mode considered by seismologists in layered systems overlying a deformable half space. The rest of the terms of Eq. (5) represent higher-order Rayleigh modes (in the terminology of seismologists) and body waves. These modes will have longer wavelength and will propagate faster than the fundamental mode which by definition has the shortest wavelength and thus the lowest phase velocity. While most of these higher modes can be neglected, since they decay rapidly in the direction of wave propagation, others may decay less rapidly than the fundamental mode. This phenomenon occurs only at relatively high frequencies on sites with a marked increase in stiffness with depth; say a sand profile over rock. These low-decay modes could conceivably contribute significantly to the motion at a surface control point. However, studies by Chen and Lysmer (1979)

have shown that such modes, when they occur, are associated with energy propagation in deeper high-velocity layers and that they cause near surface motions which are similar to those caused by vertical or slightly inclined body waves. That this is so is not surprising when one considers the propagation mechanism of these modes. The very facts that the waves travel at high velocities and decay slowly indicate that the major part of the energy propagation occurs in deeper layers with high body wave velocities and low damping. This immediately implies that insignificant amounts of energy are propagated horizontally in the softer surface layers or, in other words, that the higher frequency motions in the surface layers are maintained through nearly vertical energy propagation through a mechanism similar to that of slightly inclined body waves. The result is that the upper parts of the mode shapes, i.e. the variation of displacements with depth, are virtually identical to those found in analyzing vertically propagating or slightly inclined body waves.

Thus, in practical calculations the effects of higher-order surface wave components can be considered by assuming a certain content of slightly inclined or vertical body waves in the control motion. In view of this observation only Rayleigh wave fields consisting of fundamental modes will be considered in the following sections.

Effect of Layering

The importance of using layered system theory, rather than the simpler half-space theory, in dealing with structures on a layer of soil overlying rock is illustrated by the computed variations of accelerations and shear stresses with depth for the cases of (1) a homogeneous half-space and (2) a 128 ft layer of sand overlying a half-space shown in Figs. 2 and 3, respectively.

Figure 2 shows the computed results for a uniform half-space with properties similar to those of an average soil deposit. It may be seen that analyses for a typical control motion with a peak horizontal acceleration of 0.25g at the ground surface in such a half-space indicate that: (a) if the motion is assumed to consist entirely of Rayleigh waves, the vertical component of acceleration at any depth will be 2 to 4 times greater than the horizontal component of acceleration at the same depth; (b) if the motion is assumed to result solely from vertically propagating shear waves, the horizontal accelerations at any depth will be greater than those computed at the same depth for the Rayleigh wave assumption; and (c) the values of horizontal shear stress computed for a control motion represented entirely by Rayleigh waves will be about 50% greater than those computed on the assumption that the control motion is produced by vertically propagating shear waves.

A totally different picture is obtained if similar analyses are made for a 128 ft layer of sand overlying a rock formation, for which computed response values are shown in Fig. 3. In this analysis the stiffness and damping of the sand was allowed to vary with depth and with the cyclic strain level developed due to the motion. For this more realistic representation of the case of a soil deposit overlying rock it may be seen that (a) if the control motion is represented by a system of Rayleigh waves, the vertical components of motion are about two thirds of the horizontal components at any depth--a result more consistent with the observed ratio of horizontal to vertical accelerations in a large number of earthquakes; (b) down to depth of about 60 ft, there is no great difference in the values of peak horizontal accelerations whether the computations are based either on the assumption that the control motion results only from Rayleigh waves or only

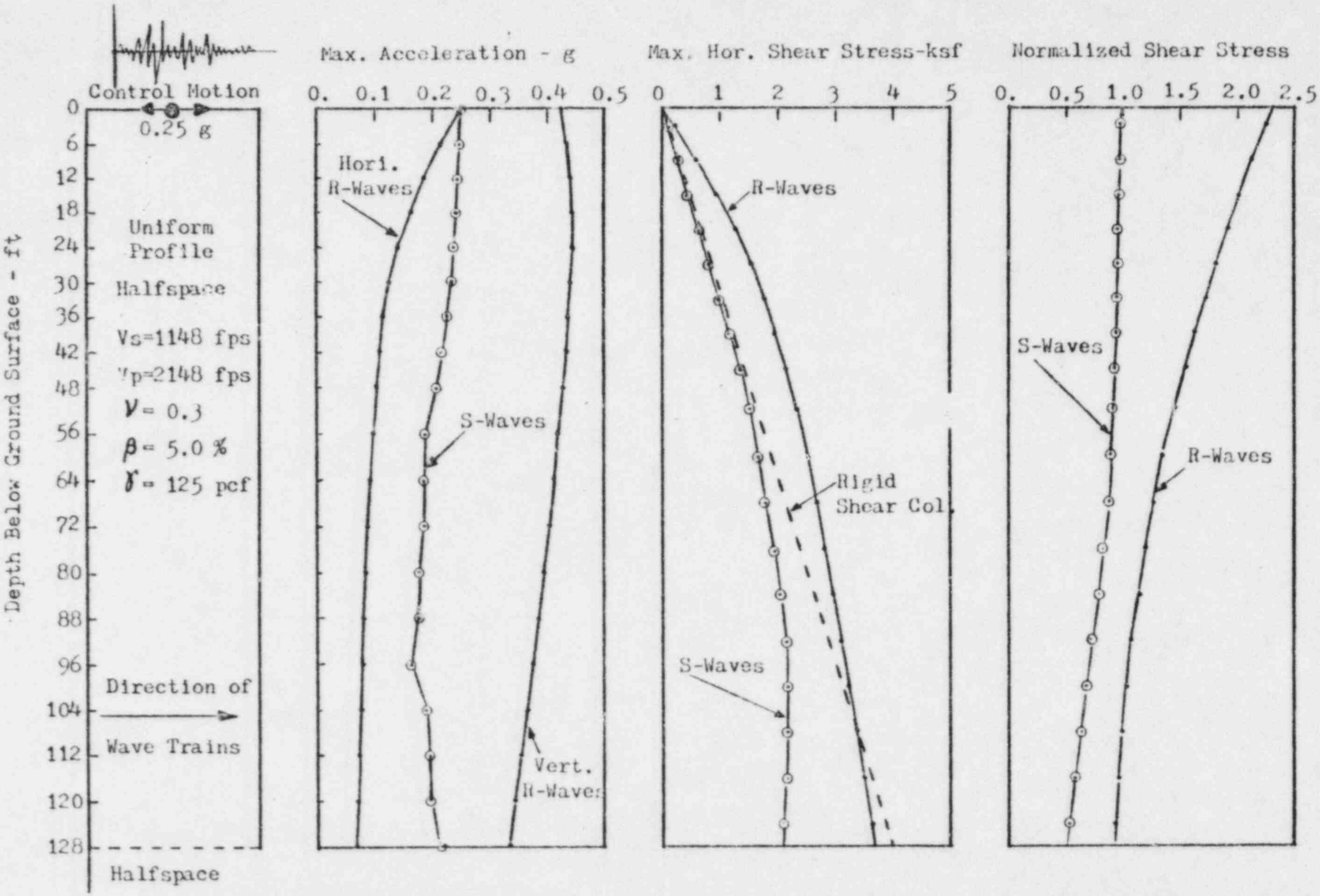


Fig. 2 SITE RESPONSE BY RAYLEIGH WAVES AND VERTICALLY PROPAGATING SHEAR WAVES IN UNIFORM HALF SPACE

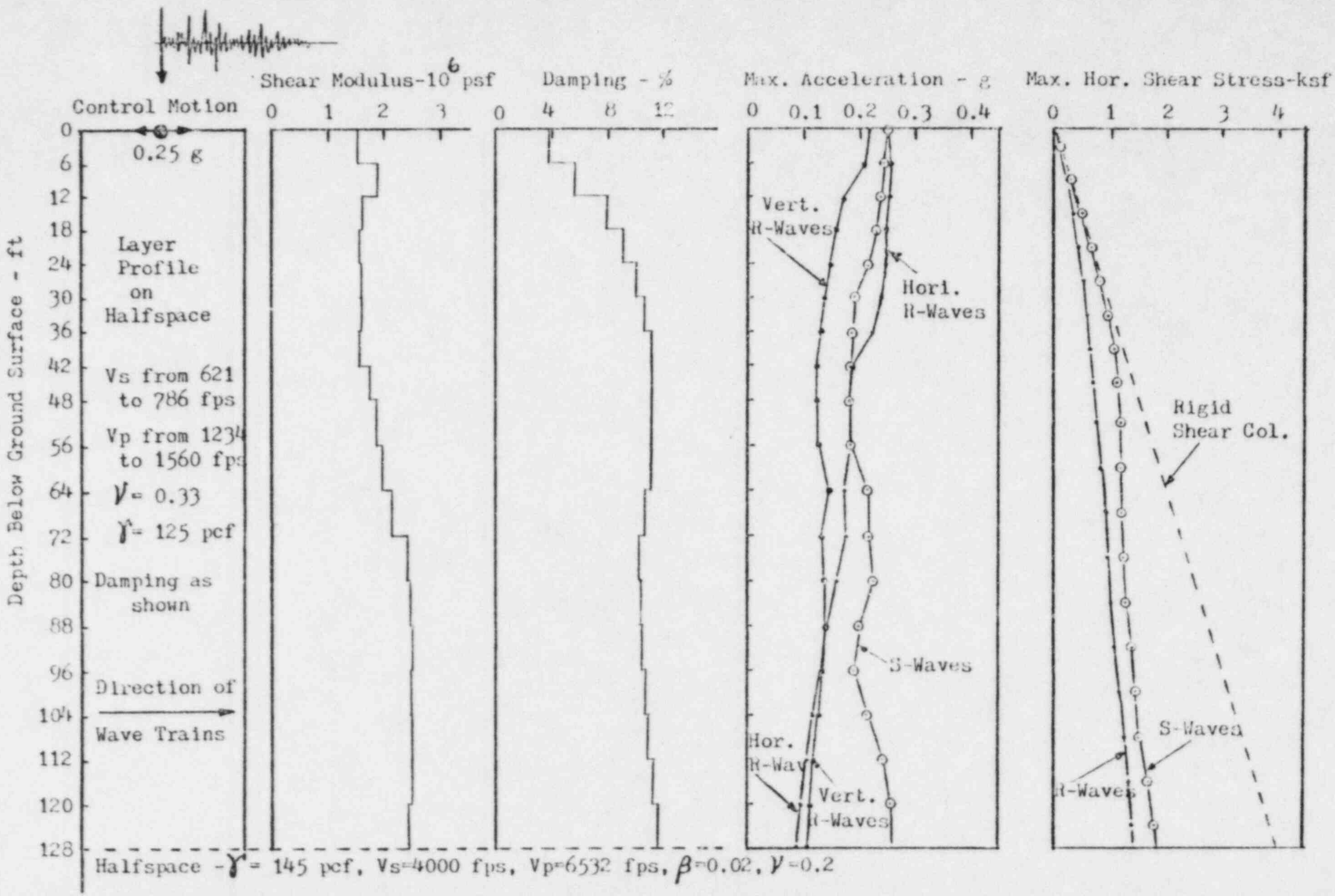


Fig. 3 SITE RESPONSE BY RAYLEIGH WAVES AND VERTICALLY PROPAGATING SHEAR WAVES - 128 FT LAYER OF SAND OVER UNIFORM HALF-SPACE

from vertically propagating shear waves; and (c) the values of maximum horizontal shear stress computed for a control motion represented entirely by Rayleigh waves are typically about 30% less than those computed on the assumption that the control motion is produced only by vertically propagating shear waves.

It is clear from the above examples, that for soil deposits, it is more conservative to compute maximum horizontal shear stresses based on the assumption that the motions result from vertically propagating waves than it is to assume a field consisting of Rayleigh waves--yet the opposite result would be obtained if the layering of the system were not considered. Similarly, erroneous conclusions concerning vertical motions could result from the use of a homogeneous half-space to represent a layered soil deposit. Finally, it may be seen that if the system layering is correctly considered, horizontal accelerations will vary with depth but they will not differ significantly whether the motions are represented entirely by Rayleigh waves or entirely by vertically propagating waves. This topic will be discussed at length in a later section of this part of the report.

Effect of Distance of Propagation

It is important to note that Figs. 2 and 3 show only the variation of response with depth directly under the point on the ground surface where the control motion is specified. In dealing with horizontally propagating Rayleigh waves, it is also important to consider the manner in which the motions will vary with distance of propagation. The results of such a study are shown in Fig. 4. In this analysis a typical NRC control motion with a peak acceleration of 0.25g was represented by a system of Rayleigh waves

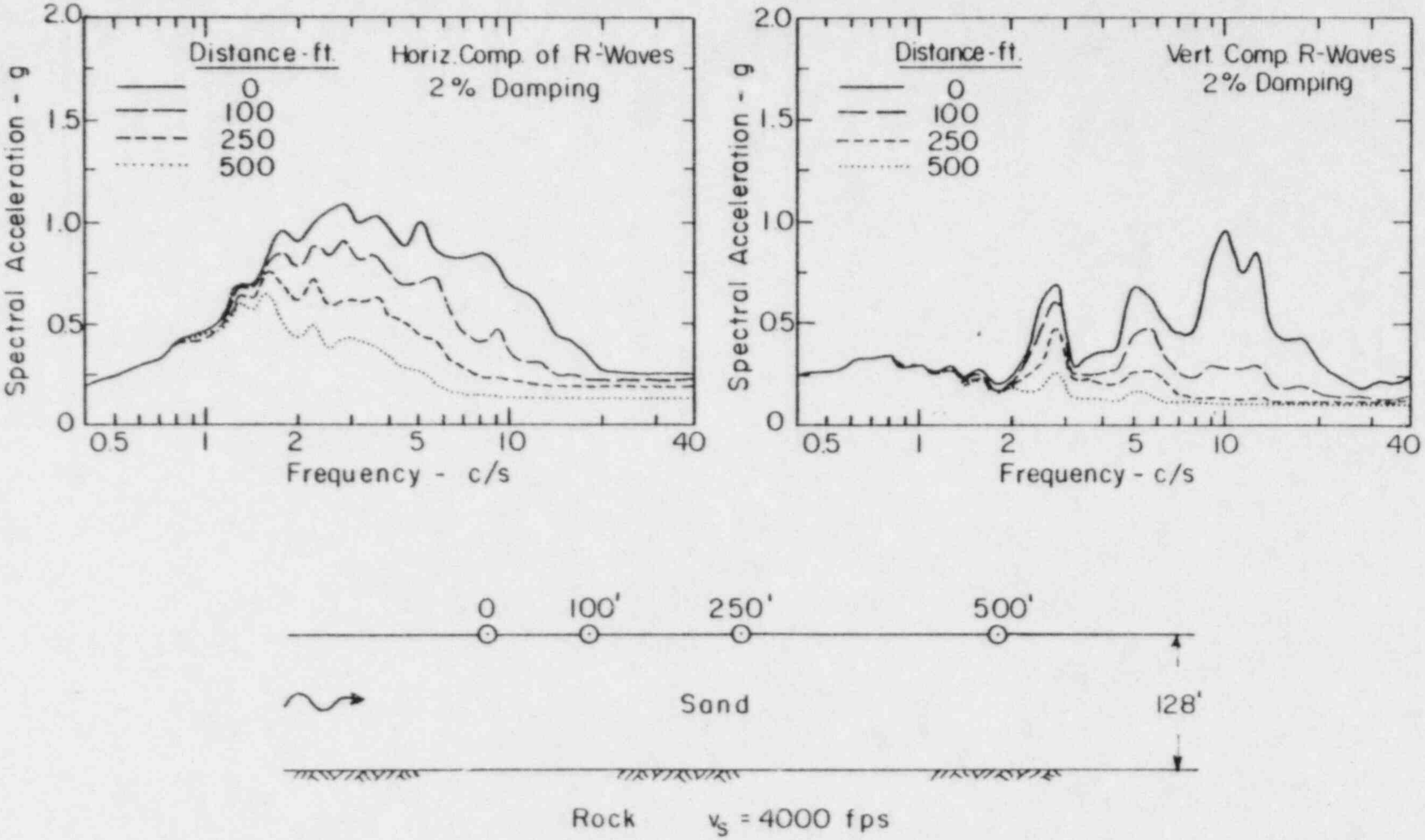


Fig. 4 ATTENUATION OF RAYLEIGH WAVE CHARACTERISTICS WITH DISTANCE TRAVELLED THROUGH SOIL DEPOSIT

which were then allowed to propagate horizontally from the control point location across a horizontal deposit of sand overlying rock. The figure shows the changes in surface motion characteristics with distance of propagation. It is readily apparent that in such a system, the high frequency components of the fundamental Rayleigh waves are rapidly damped out as a result of the relatively high damping characteristics of the soil and in fact, at a distance of a few hundred feet, virtually all motions with frequencies higher than 1 to 2 Hz have decayed to insignificant values. Since most soil deposits extend horizontally for thousands of feet, it is thus unrealistic to expect that the high frequency components of motions in such deposits could result from horizontal propagation of fundamental Rayleigh waves.

As to the higher-order surface waves discussed in a previous section, p. II-8, some of these modes may not decay as rapidly as the fundamental modes. However, as discussed, if such waves do occur, they will, at shallow depths, be similar to slightly inclined body waves and they can be treated as such. They will, because of their long wavelengths produce motions which are essentially in phase at any shallow depth within reasonable horizontal distances. Thus they are unlikely to produce significant rocking and/or torsion in nuclear structures.

Thus for structures on soil deposits and cases when the frequencies of concern are greater than 1 or 2 Hz, there is no realistic basis for considering that Rayleigh waves make any significant contribution to the site response. The same logic would also apply to Love waves.

However there are two cases where this argument would not necessarily apply:

(1) for surface motions propagating in rock where the much lower damping and much higher wave velocities would lead to very low rates of horizontal attenuation of motions

and (2) for structures whose response is primarily dependent on the long period components of motion (say below 1 Hz) since such components are apparently not damped out readily even in soil deposits.

The above observations are apparently consistent with seismological observations of earthquake ground motions which do not indicate any significant contributions of Rayleigh waves in the high frequency range.

With regard to the design of nuclear power plants, the above discussion leads to two important conclusions:

(1) For structures located on soil deposits, there is no need to consider Rayleigh or Love waves as making any significant contribution to the ground motions for the purpose of evaluating soil-structure interaction effects.

and (2) For structures located on rock, surface waves may contribute to the ground motions but not at the highest frequencies required to be considered in design. Never-the-less consideration of the possible effects of some components of surface waves may be warranted in design.

Additional evidence supporting these statements will be presented in Part V of this report.

VERTICALLY PROPAGATING WAVES

The great majority of methods for site response analysis use Kanai's (1952) assumption of vertically propagating shear waves. This assumption leads to simple one-dimensional mathematical models for horizontally layered

systems and has, partly because of the similarity between motions caused by different wave fields, led to remarkable success in predicting the major features of site response during earthquakes, especially since the analytical procedure was modified by introduction of the equivalent linear method by Seed and Idriss (1969).

Linear site response problems with vertically propagating waves can be solved by a multitude of numerical techniques which are described in texts on soil dynamics, e.g., Desai and Christian (1976). The most efficient method for computing free-field motions from a specified surface control motion appears to be the complex response method used in the program SHAKE, Schnabel et al. (1972). With these methods it is currently possible to analyze any layered viscoelastic soil system overlaying a viscoelastic half-space. The control motion can be specified at the ground surface, at any depth in the soil deposit or as an outcrop motion. Nonlinear effects can be approximated by the equivalent linear method.

Recent efforts have been directed towards the development of true nonlinear methods of analysis. Several methods have been proposed for performing nonlinear total stress analysis of site response problems with vertically propagating shear waves. The most important of these are: The method of characteristics, Streeter et al. (1974), Idriss et al. (1976), Taylor and Larkin (1978); the finite difference method, Joyner (1977); and implicit integration schemes, Martin (1975). In addition several methods of effective stress analysis have been proposed, Ghaboussi and Dikmen (1978), Zienkiewicz et al. (1978), Finn et al. (1977), Liou et al. (1977), and Martin and Seed (1979), which can predict the pore pressure build-up in saturated sands during seismic excitation.

Comparative studies of ground motion characteristics computed by the equivalent linear method and non-linear methods show relatively small differences except where motions are very strong and soils relatively weak--a situation not likely to occur at a nuclear plant site. Thus the development of non-linear analysis techniques has further confirmed the fact that equivalent linear methods are sufficiently accurate for virtually all practical purposes in evaluating the response of nuclear power plant sites.

A major problem of current methods of nonlinear site response analysis is a limitation on the location of the control point. It is not currently possible to specify the control motion at the surface, and specification at a deeper point within the profile is, for reasons to be discussed in the following section, not desirable. However, the motion may be specified at a deep outcrop. This problem has been solved by Joyner and Chen (1975) for the special case of a layered nonlinear system overlying a uniform linearly elastic half-space, see Fig. 5(a). Joyner and Chen specify the outcrop control motion, y(t), at the surface of the elastic half-space shown in Fig. 5(b). The horizontal motions in the half-space, Fig. 5(b), are by simple wave theory

$$u_b(z,t) = \frac{1}{2}\,y(t+z/V_s) + \frac{1}{2}\,y(t-z/V_s) \tag{7}$$

where V_s is the shear wave velocity for the half-space. In the combined system, Fig. 5(a), an additional downward propagating wave occurs due to reflections from the upper soil layer. Hence the motions in that system are:

$$u_a(z,t) = \frac{1}{2}\,y(t+z/V_s) + u(t-z/V_s) \tag{8}$$

The function u(t) must be such that: $u_a(o,t) = u_o(t)$, where $u_o(t)$ is the actual motion of z = 0. Hence, $u(t) = u_o(t) - \frac{1}{2}\,y(t)$ and

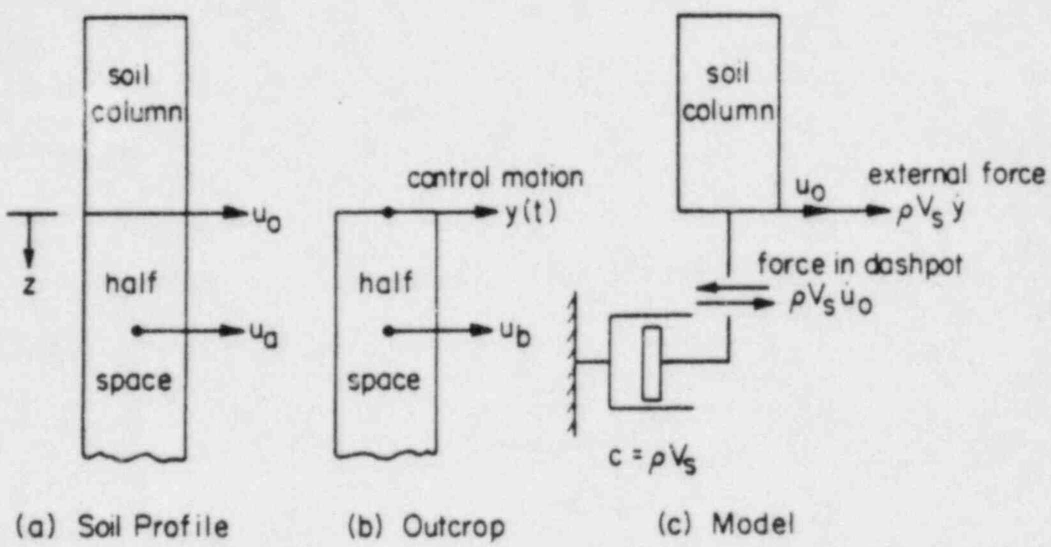


Fig. 5 CONTROL MOTION AT OUTCROP

$$u_a(z,t) = \frac{1}{2}\, y(t + z/V_s) - \frac{1}{2}\, y(t - z/V_s) + u_o(t - z/V_s) \qquad (9)$$

This leads to the following shear stress at z = 0

$$\tau_o(t) = \rho V_s\, \dot{y}(t) - \rho V_s\, \dot{u}_o(t) \qquad (10)$$

where ρ is the mass density of the half-space. Joyner and Chen apply this stress boundary condition at the base of the upper soil column and thus achieve a system which can be analyzed by nonlinear methods and which correctly accounts for the effects of the underlying half-space.

The boundary condition expressed by Eq. (10) may be achieved by the physical model shown in Fig. 5(c). In this model the upper soil column is supported on a Lysmer-Kuhlemeyer (1969) dashpot and excited by a horizontal force at the base proportional to the known outcrop velocity time history.

The Joyner-Chen model is an important contribution to the art of nonlinear site response analysis and may, as suggested by Joyner (1975), be extended to approximately two-and three-dimensional soil-structure interaction analyses of nonlinear regions overlying a linear half-space for the special case of vertically propagating waves arriving from the half-space (see discussion related to Fig. 10 of Part III of this report).

INCLINED BODY WAVES

Some energy may be arriving at the control point in the form of nonvertically propagating body waves. There is in fact evidence to suggest that most of the energy approaching the ground surface results from body waves inclined within about 30° of the vertical. This includes the effect of the high-order surface wave modes discussed in a previous section.

The response of horizontally layered sites to plane harmonic body waves arriving at a specified incident angle through an underlying elastic half-space

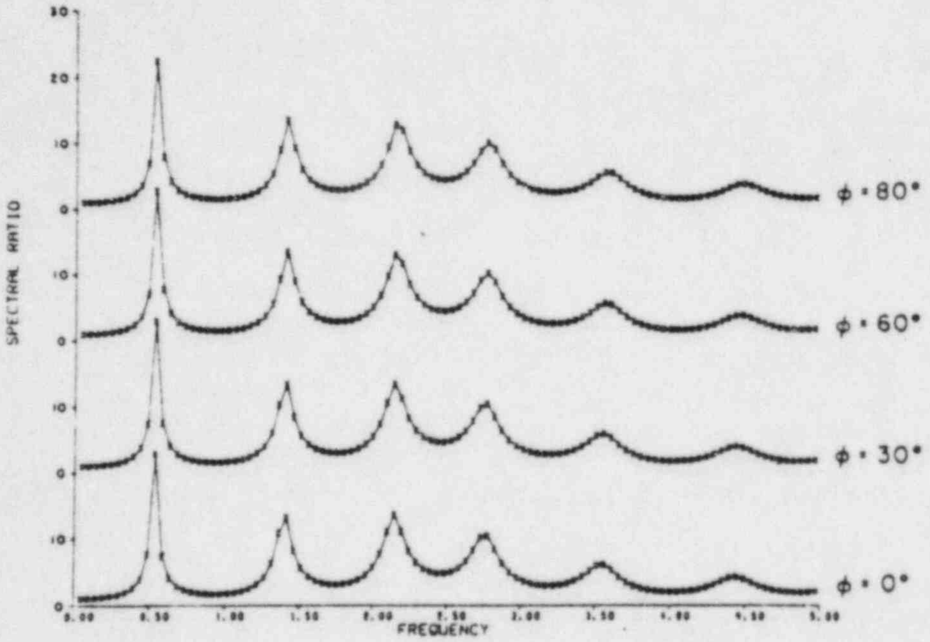
has been investigated by several researchers. The fundamental work was done by Thomson (1950) and Haskell (1960,1962) who developed an efficient matrix method for computing the frequency-dependent transmission coefficients in a layered continuum for incident SH, SV and P-waves. Efficient computer codes for the Thomson-Haskell method were developed by Hannon (1964) and Teng (1967). Silva (1976) extended the Thomson-Haskell method to include damping in the soil layers. With this method it is possible to solve linear or equivalent linear site response problems with inclined body waves for systems consisting of viscoelastic soil layers overlying a uniform undamped half-space, provided the incident angle in the half-space is known. Since no damping is included in the half-space the resulting surface motions do not decay in the horizontal direction. More recently, Chen and Lysmer (1979), have developed a method which includes damping in the underlying half-space.

Analyses of surface response to inclined body waves have also been made by Joyner et al. (1976), who determined the transfer functions from bedrock to the soil surface for a soil deposit 186 m in depth for shear waves propagating at various angles to the vertical. The results of this study are shown in Fig. 6, and it is apparent that for angles of incidence up to 45°, there is a negligible difference between the motions computed for inclined waves and for vertically propagating waves.

It is reasonable to conclude therefore that the variation of horizontal motions with depth within a soil deposit are for all practical purposes the same, whether they are computed for vertical or inclined directions of propagation within the depth range of interest to engineers. On this basis it is appropriate to use analyses for vertically propagating waves because of their greater simplicity and availability of solutions.

MOTIONS OF SHALLOW DEPTH

As discussed in Part I of this report only the motions within a relatively shallow depth (the projected depth of embedment) of the free-field



(c) Computed Horizontal Spectral Ratios Between Surface and 186 m

Fig. 6 INFLUENCE OF ANGLE OF SHEAR WAVE INCIDENCE ON COMPUTED SURFACE RESPONSE

(after Joyner et al., 1976)

will influence the motions of structures. The same discussion also indicates that both the spatial and temporal variation of the free-field motions within this depth are of importance in evaluating soil-structure interaction effects. It is therefore appropriate to discuss in more detail how the amplitude and frequency content of free-field motions vary with depth and in particular how they vary near the free ground surface.

Not to consider this variation would be equivalent to assuming that the soil mass behaves like a rigid body in which case no interaction would take place and an analysis of the problem would be unnecessary. The belief that it is necessary to analyze soil-structure interaction therefore implies the tacit assumption that motions will vary with depth and appropriate consideration of this fact must be included in any analytical procedure.

As will be shown in the following, the presence of the strong discontinuity represented by the free ground surface imposes predictable and observable limitations on how horizontal amplitudes and the frequency content of motions vary with depth near the ground surface.

Theoretical Considerations

The potential effects of the free ground surface on the amplitude and frequency content of waves at various depths in a uniform deposit is shown in Figs. 7 and 8. Both figures show the variation of amplitude with the dimensionless depth z/λ_s in a perfect half-space, where $\lambda_s = V_s/f$ is the wavelength of shear waves at the frequency, f[Hz], considered.

Figure 7 corresponds to the case of vertically propagating shear waves for which the horizontal amplitude is

$$U = U_o \cos 2\pi \frac{z}{\lambda_s} \qquad (11)$$

and Fig. 8 corresponds to the case of horizontally propagating Rayleigh waves.

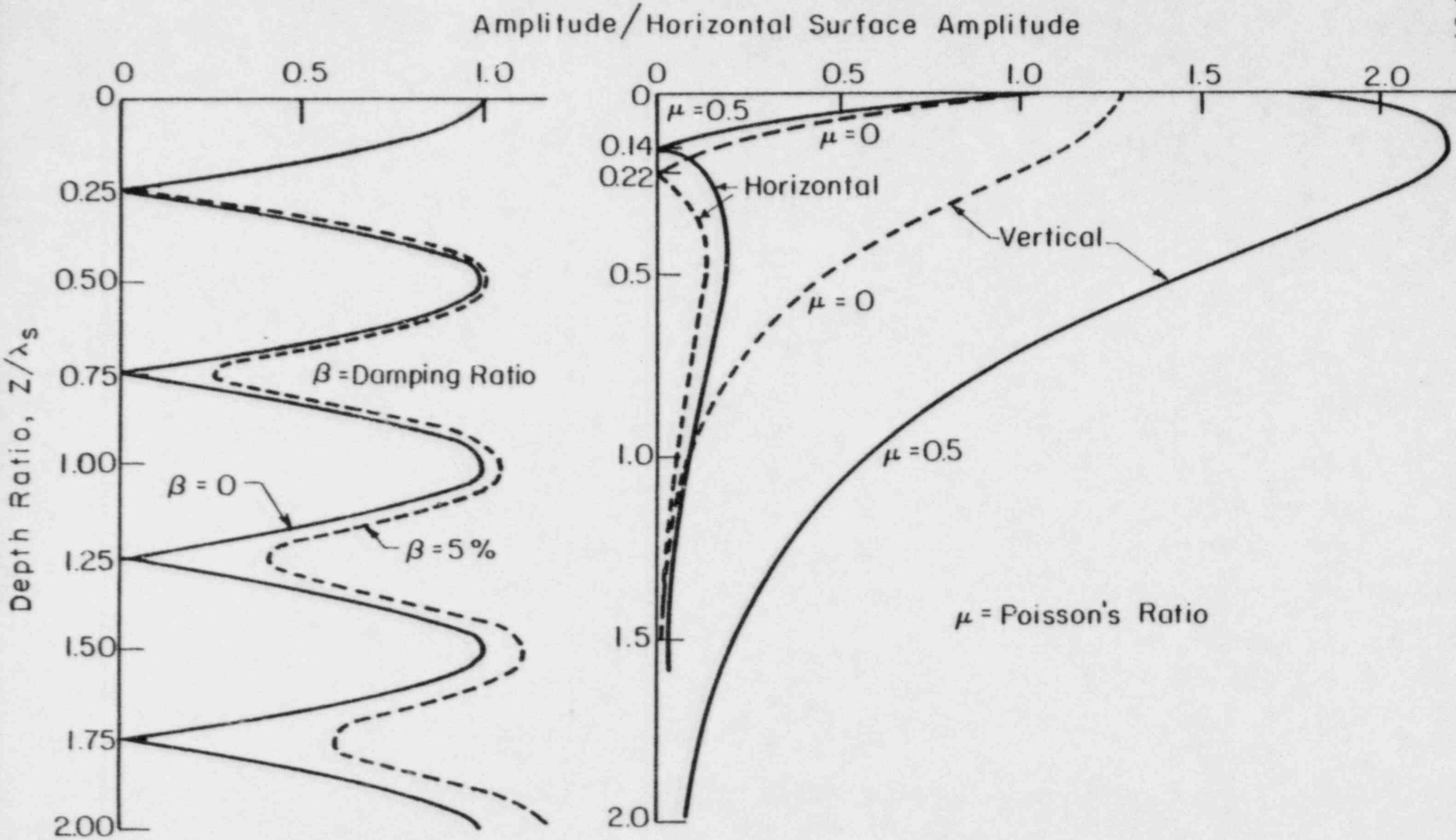


Fig. 7 SHEAR WAVES

Fig. 8 RAYLEIGH WAVES

The two types of wave fields are obviously quite different. It is remarkable, however, that both the shear wave field and the Rayleigh wave field produce monotonically decreasing horizontal displacements within the approximate depth

$$
\begin{aligned}
z &\simeq \frac{1}{4}\lambda_s \quad \text{to} \quad \frac{1}{5}\lambda_s \\
&\simeq \frac{V_s}{4f} \quad \text{to} \quad \frac{V_s}{5f}
\end{aligned}
\tag{12}
$$

and that all horizontal displacements vanish at this depth. A similar phenomenon occurs for inclined shear waves and for layered soil systems where V_s in Eq. (12) can be replaced by the average shear wave velocity, $\overline{V}_s$ above the depth z. As can be seen from the dotted curve shown in Fig. 7 the existence of material damping does not change the substance of these observations.

Two important conclusions can be drawn from these analyses:

(1) Any horizontal motion computed (or observed) at the depth z must be deficient of components of the frequency

$$
f \simeq \frac{\overline{V}_s}{4z} \quad \text{to} \quad \frac{\overline{V}_s}{5z} \tag{13}
$$

i.e., its response spectrum will have a dip at the approximate frequency f, which incidentally is equal to the fixed base natural frequency of the soil column above the depth z. Thus, the only level at which a smooth spectrum can exist is at a free surface, and specifying a control motion with a smooth spectrum at any other depth will, as experience has shown, lead to completely unrealistic results. This free surface can be the actual ground surface or a real or imaginary rock outcrop; however a smooth

spectrum cannot exist within a soil deposit, whether the motions be due to near-vertically propagating body waves or to horizontally propagating Rayleigh waves.

(2) In a deposit with uniform properties, seismic motions will decrease with depth below the ground surface at least down to the depth

$$z \simeq \frac{\overline{V}_s}{4f_{max}} \text{ to } \frac{\overline{V}_s}{5f_{max}} \tag{14}$$

where f_{max} is the highest frequency present in the motion.

This follows directly from Eq. (11) which shows that all components decrease in amplitude within the above depth. Because of variations in soil characteristics with depth this predicted reduction will often extend to depths greater than those indicated by Eq. (14). For a typical soil site, with say $\overline{V}_s$ = 1000 fps, and a seismic environment, with say f_{max} = 20 Hz, the above formula shows that a significant reduction in the free field motion may occur within the upper 10 ft (or deeper if the predominant frequency is lower) of the site. Thus in view of the discussion in connection with Eq. (4) of Part I, even relatively shallow embedment may significantly influence the seismic response of structures on soft sites and both the embedment and the reduction in the amplitude of the seismic environment with depth should be considered in a rational interaction analysis.

Substitution of realistic values of V_s and f into Eq. (12) will show that z is typically larger than 20 ft for soil sites and 60 ft for rock sites. Thus typical structures experience only the upper part ($z/\lambda_s < 0.2$) of the motions shown in Figs. 7 and 8. In this "shallow" depth range horizontal motions produced by any seismic environment with the same horizontal surface

control motion are quite similar. It is therefore to be expected that the horizontal motions produced at points below the ground surface during earthquakes will be relatively independent of the type of wave field producing the motion.

The above observations were made for motions in a uniform half-space. For layered systems, the stiffness of which usually increases with depth, calculations have shown that the similarity between motions produced at shallow depth by different types of wave fields is even more pronounced.

An interesting example of these effects in a 600 feet deep soil deposit overlying a rigid half-space is shown by the analytical results presented in Figs. 9, 10, and 11. To study the response at different depths in this deposit, analyses were made using vertically propagating shear wave theory for 15 different excitation records. In eight of the analyses, existing records obtained on deep soil deposits were scaled to have a peak acceleration of 0.20g and considered to the developed at the ground surface. The distribution of acceleration with depth and the frequency characteristics of the motions developed at depths of 40 and 76 ft were then determined by deconvolution analyses.

For the same soil deposit, a second study was made in which seven records representative of rock motions were used as base excitation and the base motions were scaled in each case to produce a peak acceleration of 0.20g at the ground surface.

There was surprisingly little difference in the computed distribution of motions whether the excitation was applied at the ground surface or whether it was applied at the base of the soil deposit. The results of the two sets of studies were analyzed statistically to determine the mean

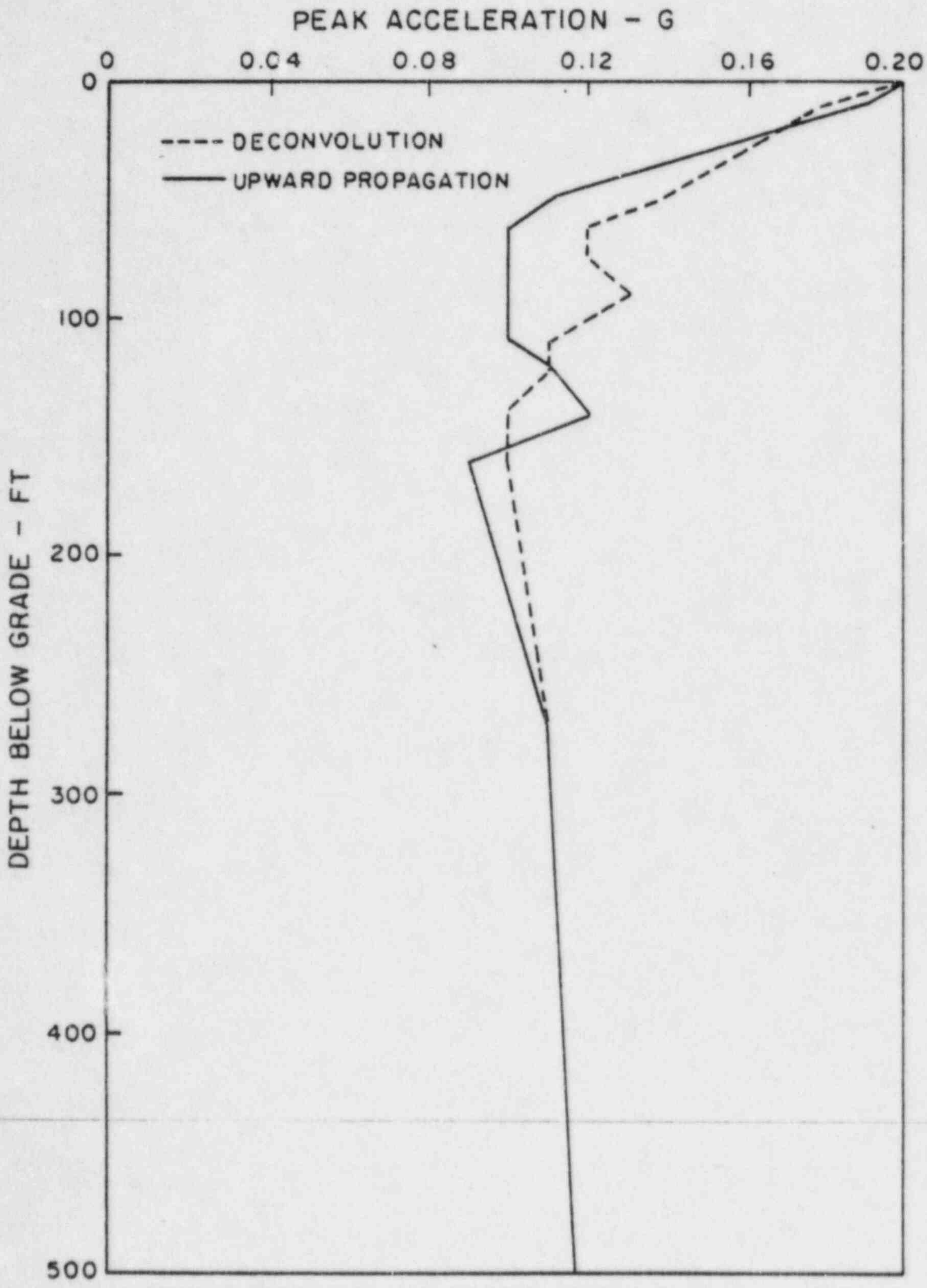


Fig. 9 VARIATION OF MEAN PEAK ACCELERATION WITH DEPTH OF DECONVOLUTION AND UPWARD PROPAGATION ANALYSES

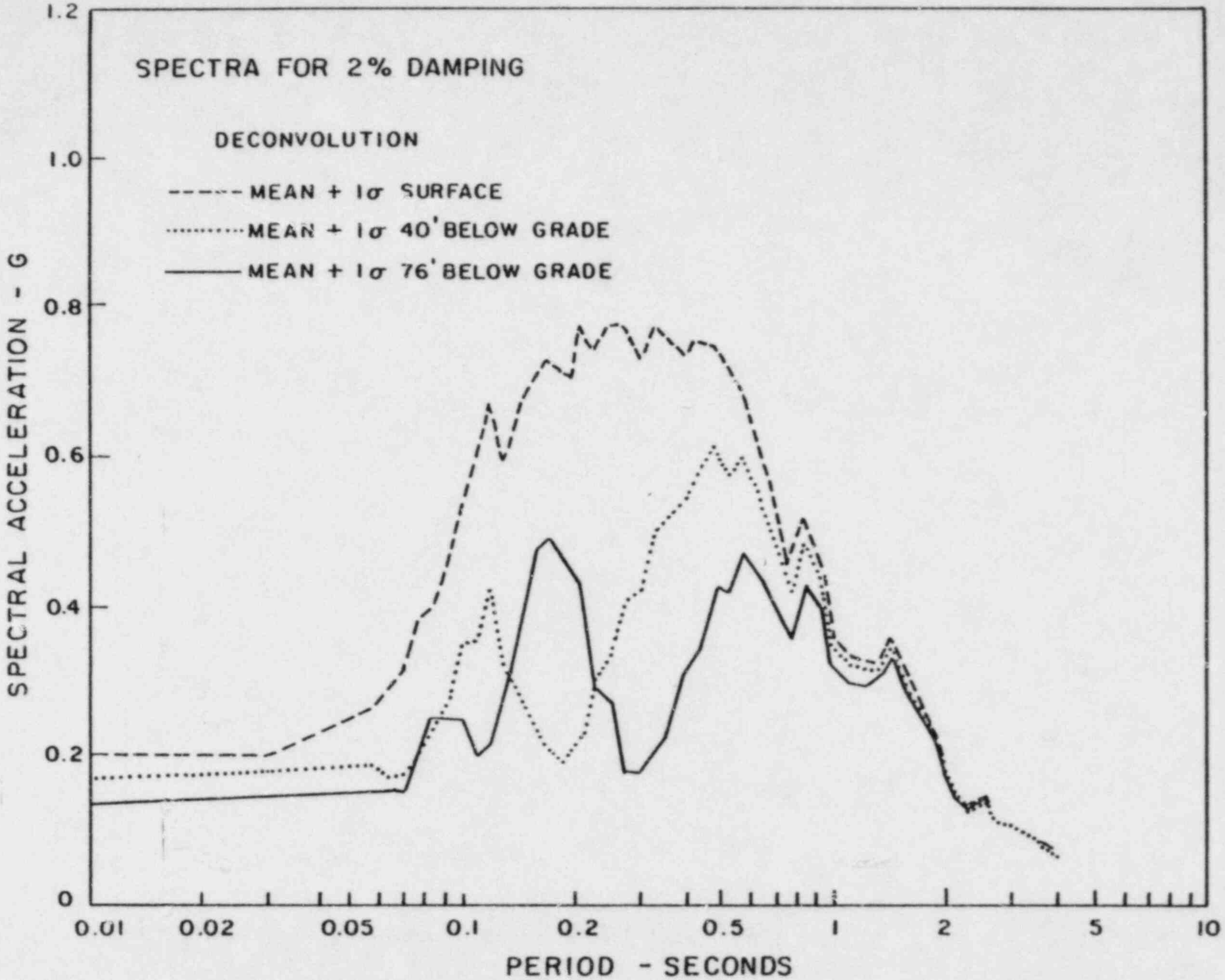


Fig. 10 STATISTICAL ANALYSIS OF COMPUTED SPECTRAL SHAPES AT DIFFERENT DEPTHS IN SOIL DEPOSIT

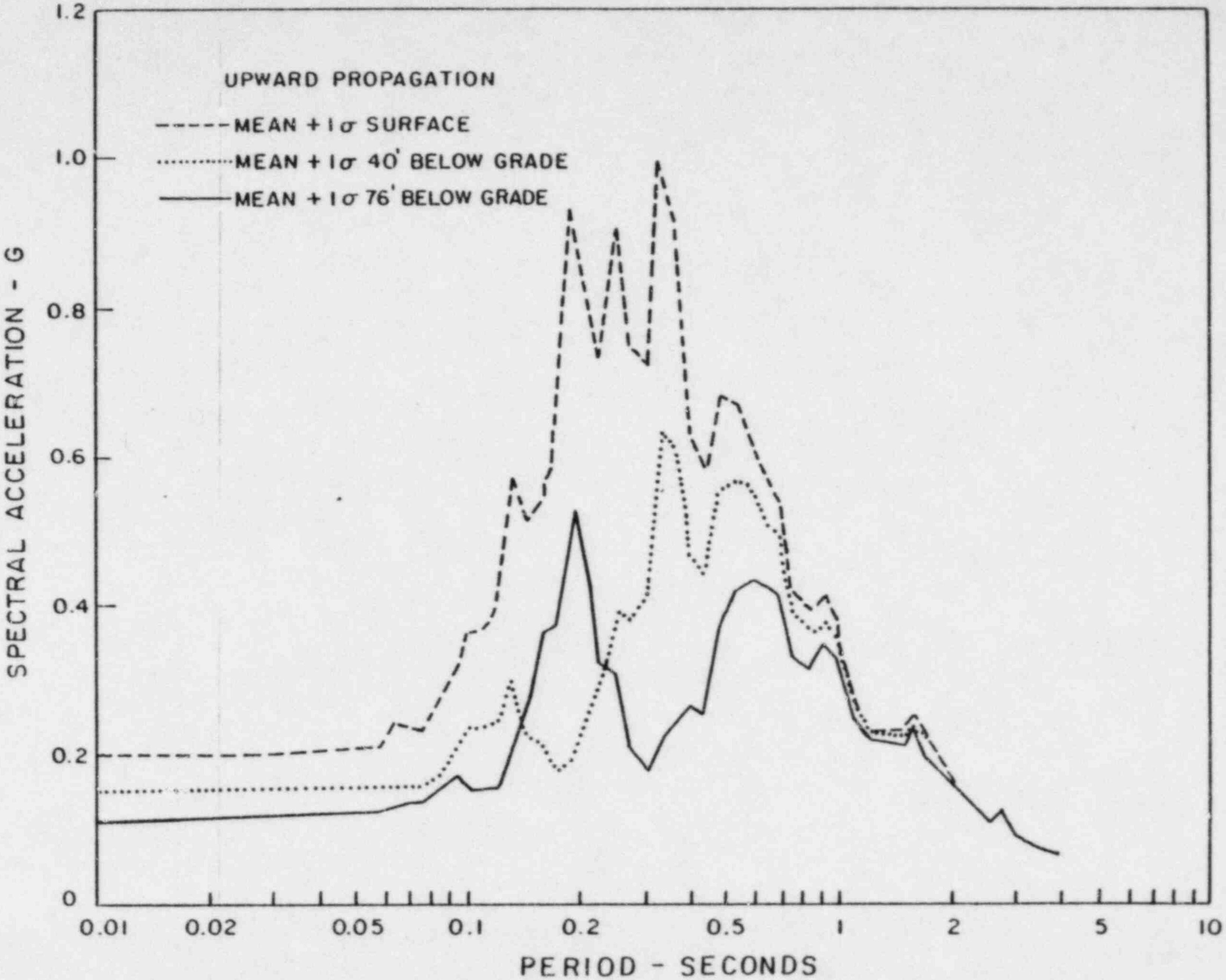


Fig. 11 STATISTICAL ANALYSIS OF SPECTRAL SHAPES AT DIFFERENT DEPTHS IN SOIL DEPOSIT

acceleration distribution separately for the deconvolution analyses and for the base input analyses. The results of this analysis are shown in Fig. 9. On the whole the results are remarkably similar, all showing a marked drop in peak acceleration within the upper 100 ft.

The response spectra for the motions developed at depths of 40 and 76 ft were also computed and analyzed statistically for the two different groups. The 84 percentile spectra for surface motions, motions at 40 ft depth and motions at 76 ft depth for the deconvolution analyses are shown in Fig. 10. It may be seen that while the spectrum for the surface motions is of the broad band type, the spectrum for motions at a depth of 40 ft contains a marked suppression of frequencies corresponding to a period of about 0.18 second while that for motions at a depth of 76 ft shows a marked suppression of frequencies corresponding to a period of about 0.3 second. The fixed base natural periods of this deposit for 40 ft of soil and 76 ft of soil were about 0.18 and 0.3 seconds respectively. Similar results are shown in Fig. 11 for the base excitation analyses. Thus it may be seen that the frequency suppression effect, as predicted by Eq. (13), is mainly a feature of the geometry and material characteristics of the deposit and depends only slightly on the type of wave motions involved.

In a deposit 600 ft deep extending to substantial distances in all directions there would not be expected to be any substantial contribution of surface waves to the motions in the frequency range of 1 to 20 Hz, the types of motions primarily investigated in this study, and thus the use of vertically propagating shear waves as the primary wave field is appropriate. Never-the-less the effect of the discontinuity provided by the ground surface on the amplitudes of motions and the frequency characteristics of motions at different depths is clearly illustrated by this example.

Field Evidence

Despite the fact that no concerted effort has been made to date to obtain field data to confirm the above theoretical predictions, a substantial body of field data does in fact exist.

Variation of peak acceleration with depth

The best data to show the variation of peak accelerations with depth is that obtained from vertical arrays of instruments, which only in recent years have been installed at a number of locations to record earthquake motions. Probably the most successful array has been that installed by the U.S. Geological Survey near Menlo Park, California, Joyner et al. (1976). Details of the instrument locations at depths of 0, 12 m, 40 m, and 186 m in relation to the soil profile are shown in Fig. 12 and the characteristics of the deposit in which they were installed are shown in Fig. 13. A number of records of small earthquakes were obtained from the instruments in this array during the period 1972 to 1977. Three typical records are shown in Figs. 14, 15 and 16. The marked decrease in amplitude of the recorded motions with depth is readily apparent.

Similar decreases of motion amplitude with depth have been observed in four earthquakes recorded in a similar array at Richmond, California by the University of California Seismological Laboratory.

For somewhat stronger motions, an excellent set of data was obtained by records obtained in the basements of buildings in Tokyo in the Tokyo-Higashi-Matsuyama earthquake of July 1, 1968 Ohsaki and Higawara (1970). The recorded values of maximum acceleration for different basement depths are shown in Fig. 17. Although there is considerable scatter in the data, peak accelerations at a depth of 70 ft are typically only about 25% of those recorded at the ground surface. It may be argued that these results are influenced by soil-structure interaction effects, but such effects are likely

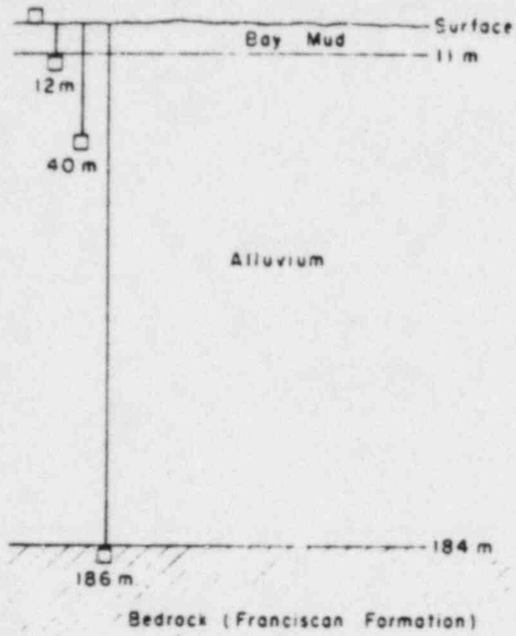


Fig. 12 SOIL PROFILE

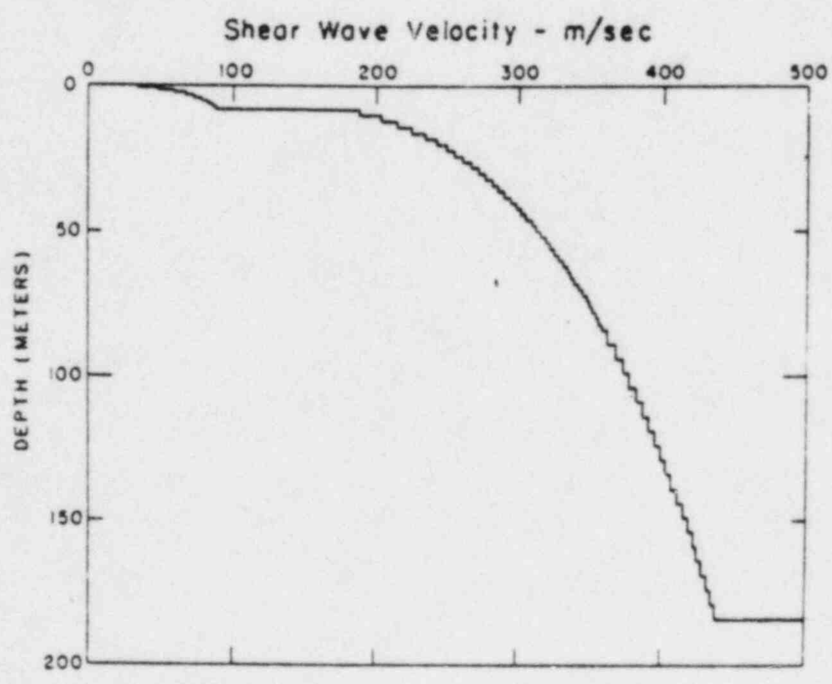


Fig. 13 SHEAR WAVE VELOCITY VS. DEPTH

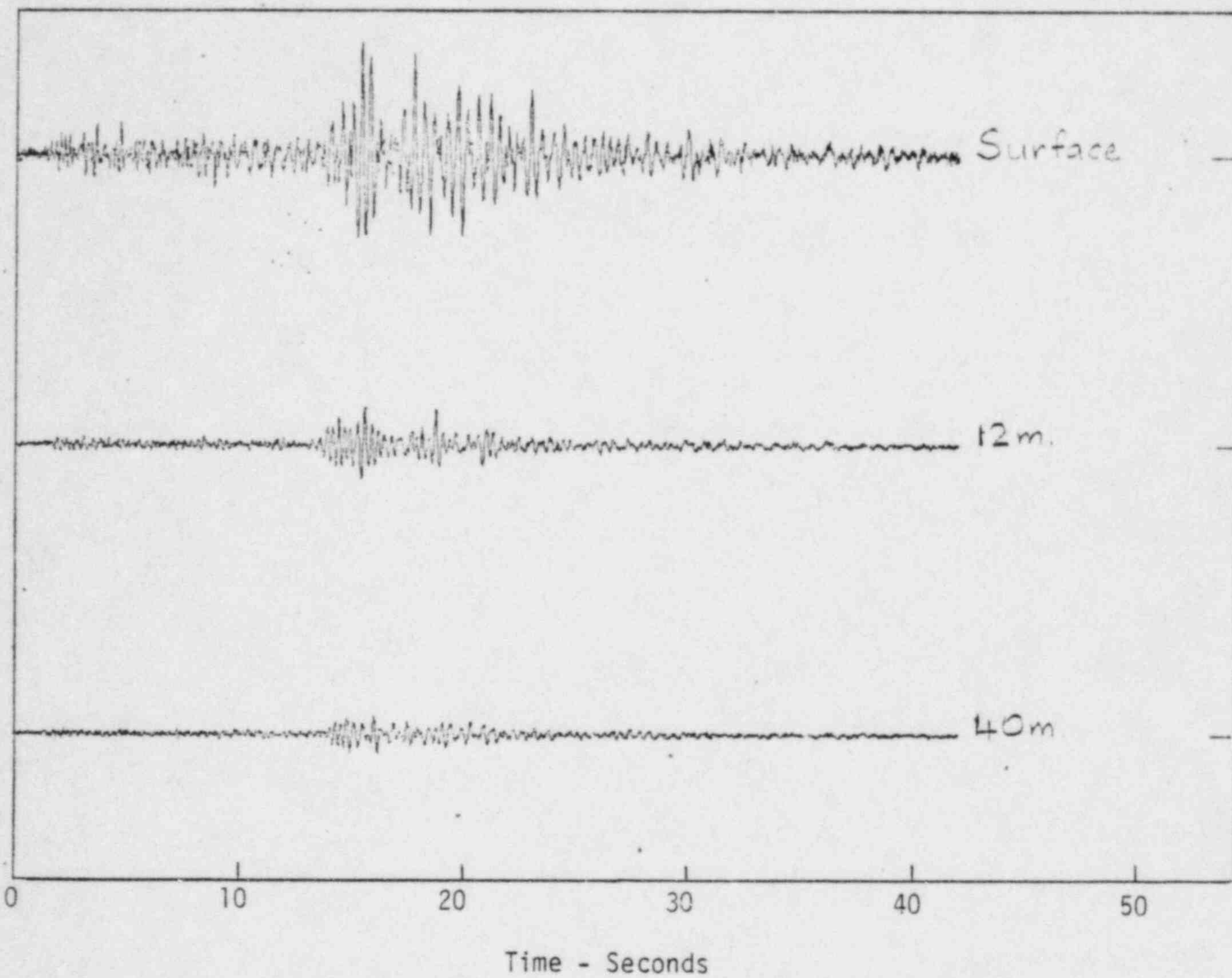


Fig. 14 VARIATION OF GROUND MOTIONS WITH DEPTH IN U.S.G.S. ARRAY DURING SALINAS (CALIFORNIA) EARTHQUAKE OF MARCH 10, 1972

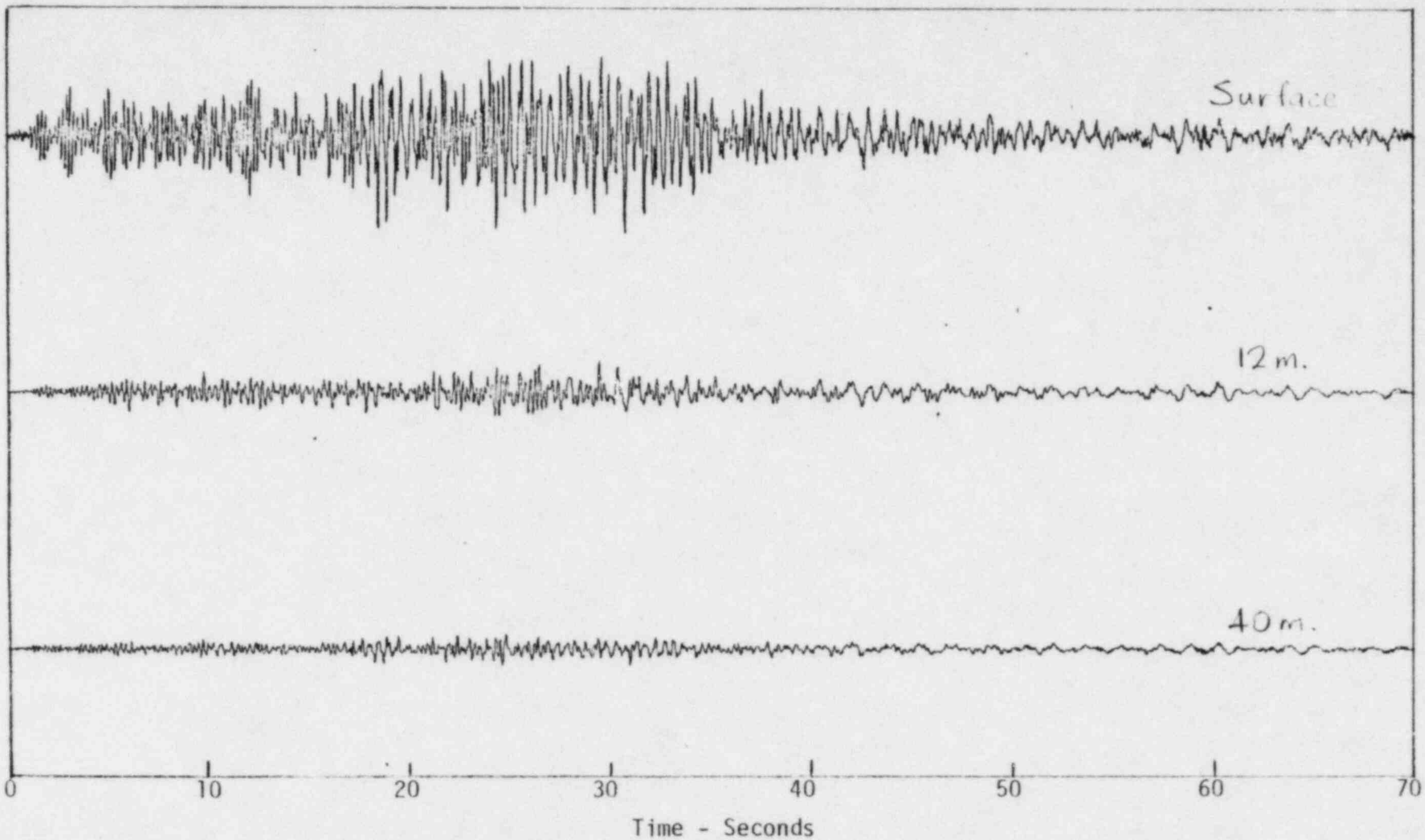


Fig. 15 VARIATION OF GROUND MOTIONS WITH DEPTH IN U.S.G.S. ARRAY DURING BEAR VALLEY (CALIFORNIA) EARTHQUAKE OF APRIL 9, 1972

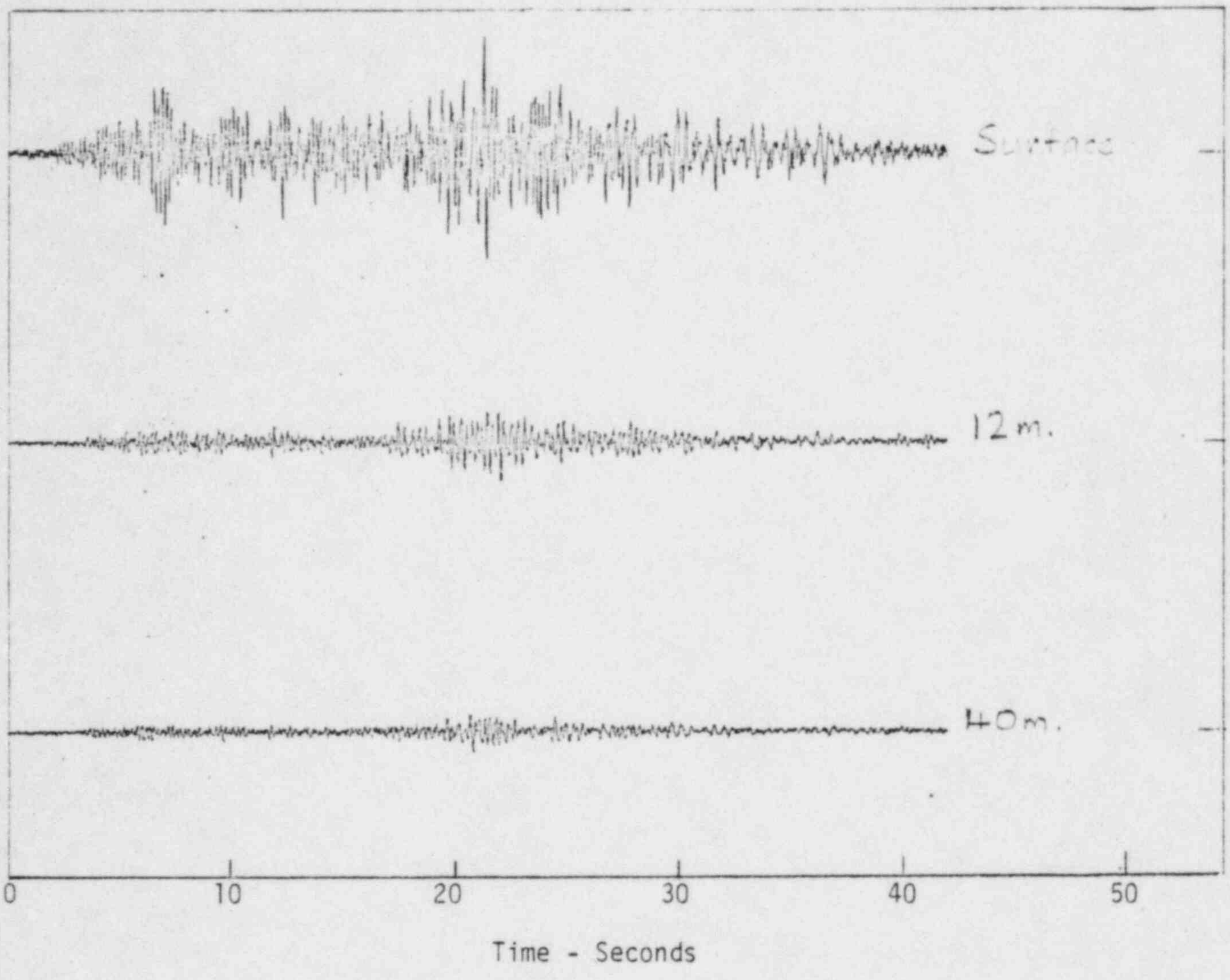


Fig. 16 VARIATION OF GROUND MOTIONS WITH DEPTH IN U.S.G.S. ARRAY DURING BEAR VALLEY (CALIFORNIA) EARTHQUAKE OF JANUARY 15, 1973

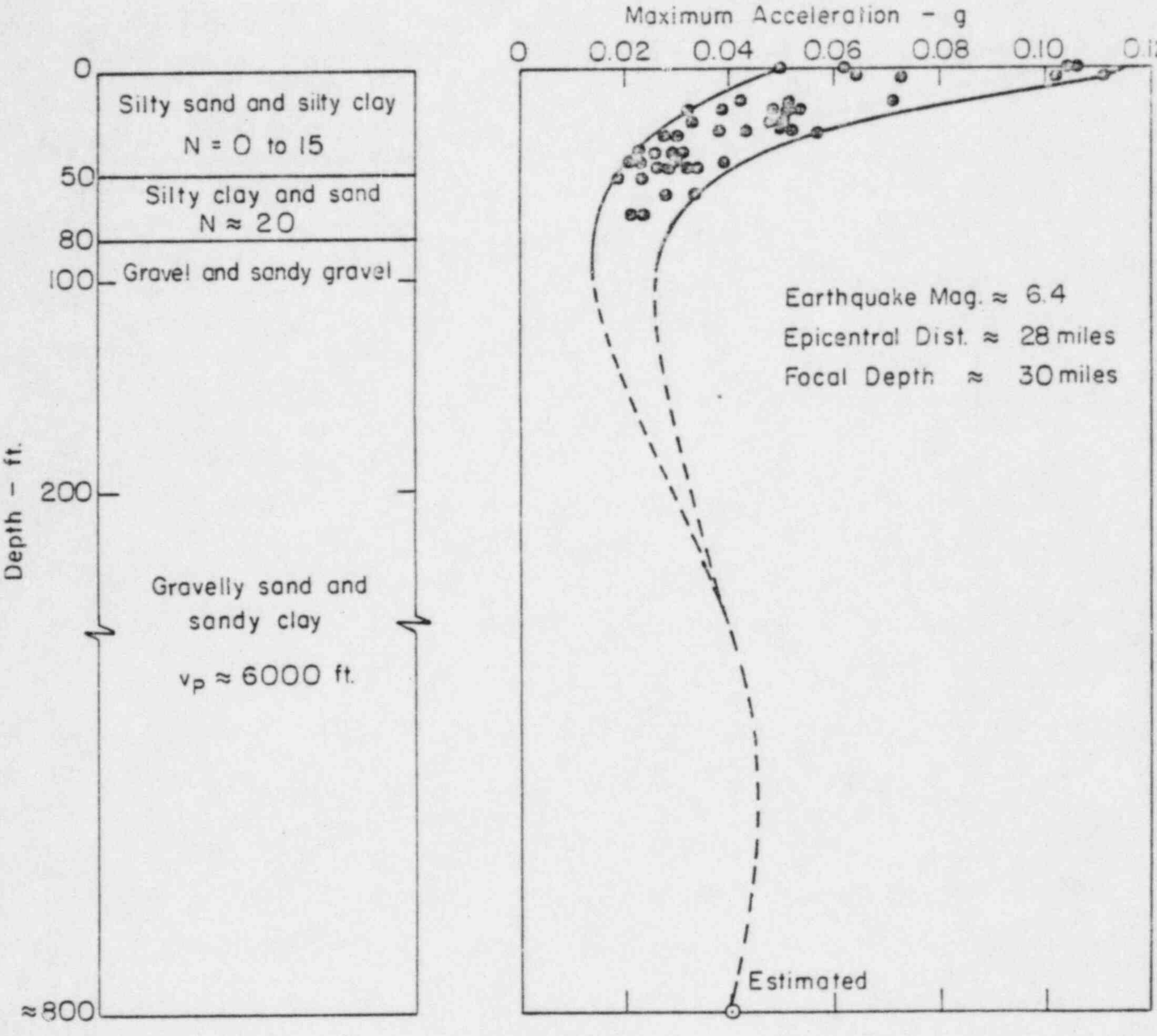


Fig. 17 VARIATION OF RECORDED MAXIMUM ACCELERATION WITH DEPTH FOR BUILDING IN TOKYO-HIGASHI-MATSUYAMA EARTHQUAKE, JULY 1, 1968

to be small and in any case would tend to minimize the variation of peak acceleration with depth rather than amplify the effect.

Yet another source of data can be obtained from records obtained in nearby pairs of buildings, each pair involving one constructed at the ground surface and the other at a depth of about 15 ft below the ground surface, in the San Fernando, California earthquake of 1971. Such records for eight sets of buildings are listed in Table 1. It may be seen that in 7 of the 8 cases listed, the peak acceleration recorded in the building with a basement was substantially less than that in the building constructed on the ground surface. While some variation of motions would be due to different spatial locations of these buildings, a statistical study of this data clearly shows that the substantial decrease in acceleration with depth is not a chance phenomenon but a pattern attributable to deterministic effects.

Finally, for very strong motions, an excellent set of records was obtained at the Humboldt Bay Power Station in the 1975 Ferndale earthquake. One of these records was obtained at a free-field ground surface location and another at the base of a caisson structure at a depth of 80 ft. The full set of records is shown in Fig. 18. The average maximum acceleration at the ground surface was 0.30g while the average at a depth of 80 ft was 0.13g. Clearly this difference needs to be taken into account if the effects of soil-structure interaction are to be analyzed in a meaningful way in this case.

Other data are available to show similar affects to those discussed above but it is believed that the cases presented provide sufficient validation that variations in ground motion with depth are not randomly variable, but characteristically decrease in the range of engineering interest, except

Table 1

Change in Maximum Acceleration Between Ground Level and Basement Level

Location	Maximum Acceleration		Percent Change in Ground Surface Accel. at Basement Level
	Ground Surface	Basement	
8244 Orion Blvd. 15107 Vanowen Blvd.	0.26g	0.12g	-54%
14724 Ventura Blvd. 15250 Ventura Blvd.	0.26g	0.23g	-12%
Hollywood Storage Bldg.	0.22g	0.15g	-32%
6430 Sunset Blvd. 6466 Sunset Blvd.	0.19g	0.12g	-37%
1880 Century Park East 1800 Century Park East	0.13g	0.10g	-23%
222 South Figueroa 234 South Figueroa 445 South Figueroa	0.15g 0.20g	0.14g	-20%
3407 West Sixth 616 S. Normandie 3470 Wilshire 3550 Wilshire	0.18g	0.12g 0.14g 0.17g	-33% -22% - 6%

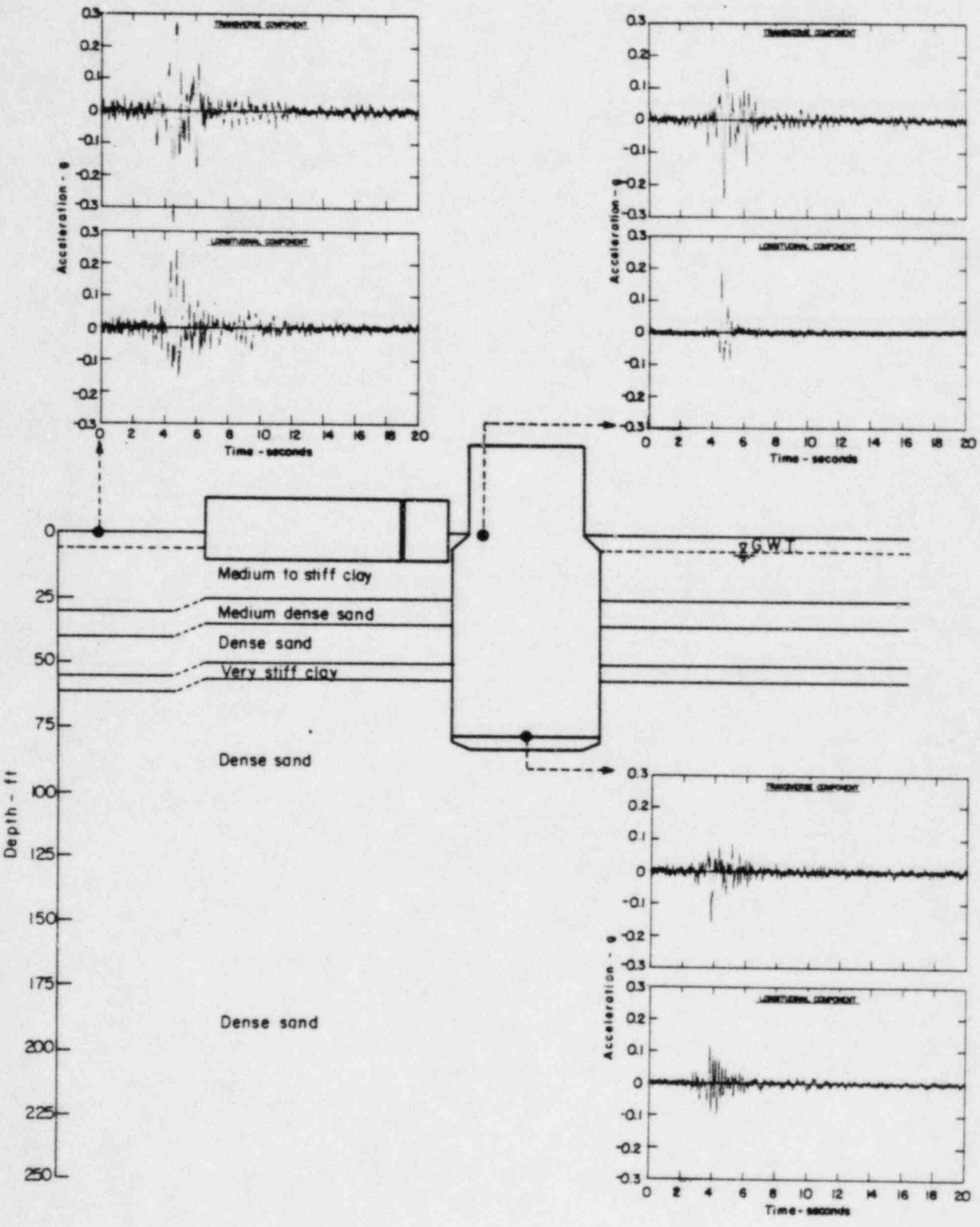


Fig. 18 GROUND MOTION RECORDS AT HUMBOLDT BAY NPS

for sites with unusual variations in soil characteristics with depth.

Variation of frequency characteristics with depth

Analytical considerations show not only a variation of peak horizontal accelerations with depth but also a deterministic variation of frequency content, and therefore of spectral shape at different depths below the ground surface. Specifically it is to be expected that at any depth z below the ground surface, frequencies of the order of $f = \overline{V}_s/4z$ [Hz] will be suppressed due to ground surface reflection effects. (For Rayleigh waves the suppressed frequency would be approximately $\overline{V}_s/5z$ but it has already been shown that Rayleigh waves with frequencies above about 1 Hz could not persist in an extensive soil deposit due to the rapid attenuation of high frequencies in relatively short horizontal distances).

Corroborative evidence of this effect is provided by the data obtained from the Menlo Park array for the recorded motions shown in Figs. 14, 15, and 16. Acceleration response spectra for the motions recorded in these events are plotted in Figs. 19 to 21, and normalized spectra, obtained by dividing the spectral ordinates for any period by the spectral ordinate for the surface motions at that frequency are shown in Figs. 22 to 24. It may be seen that using this technique, the normalized surface spectrum becomes a broad band spectrum and the spectra at other depths are scaled proportionally. It may also be seen that for all three earthquakes, the normalized spectra for motions at a depth of 12 m show a marked suppression of frequencies (evidenced by a dip in the spectrum) corresponding to a period of 0.5 sec, which corresponds to the value $4z/\overline{V}_s$ for this deposit. Similarly the normalized spectra for a depth of 40 m show a suppression of frequencies corresponding to

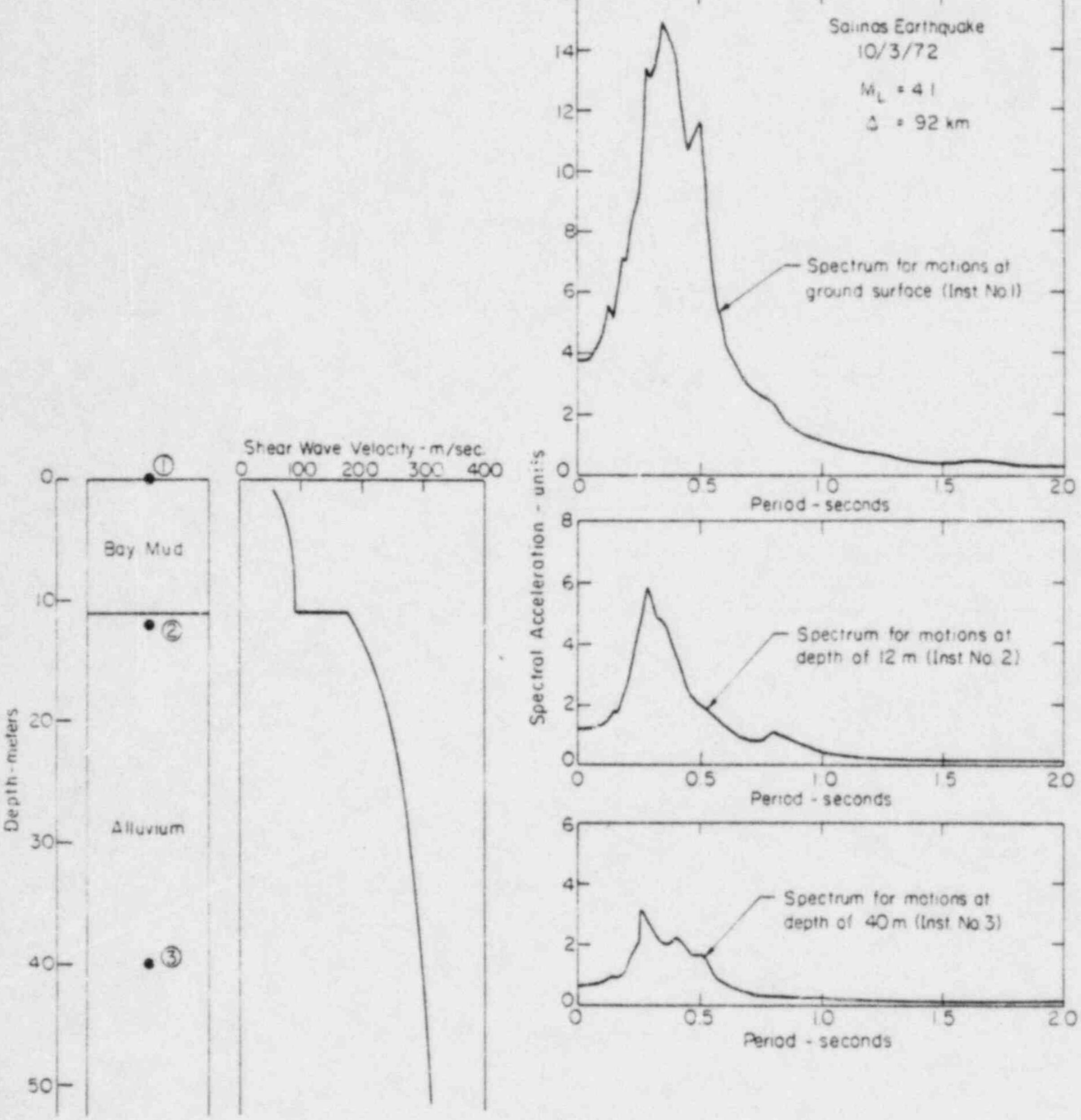


Fig. 19 SPECTRA FOR MOTIONS RECORDED IN SALINAS EARTHQUAKE, 1972

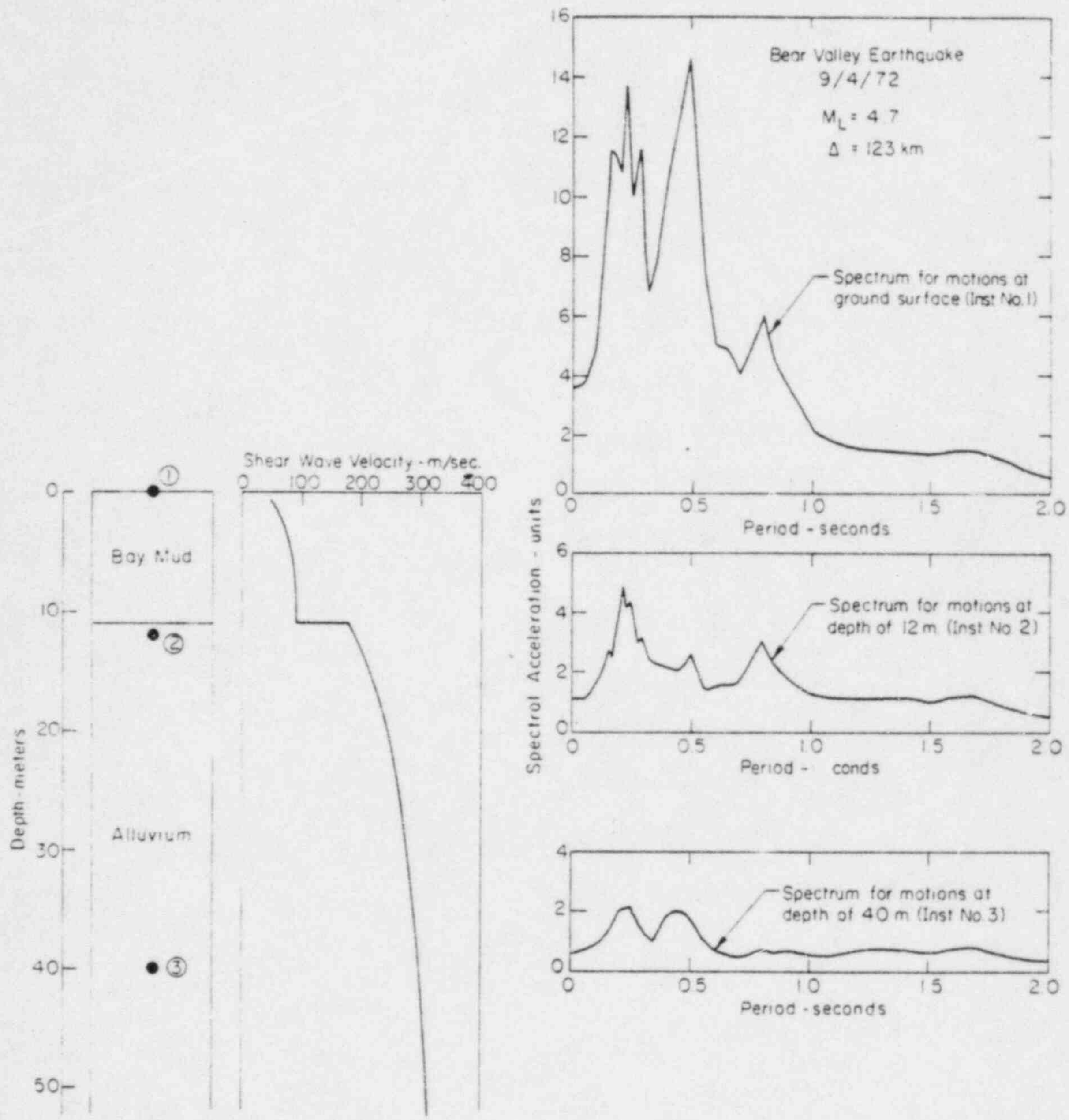


Fig. 20 SPECTRA FOR MOTIONS RECORDED IN BEAR VALLEY EARTHQUAKE, 1972

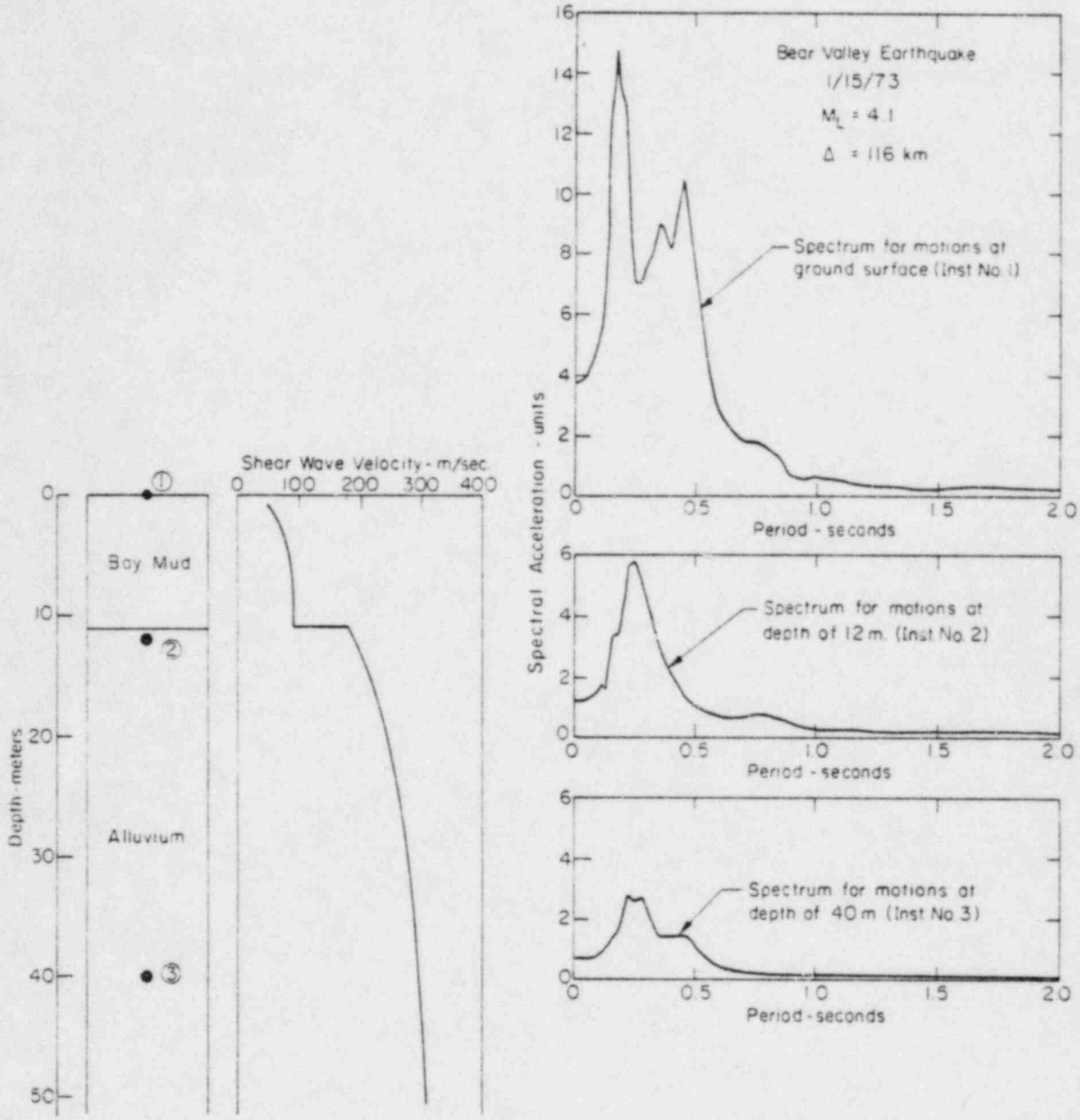


Fig. 21 SPECTRA FOR MOTIONS RECORDED IN BEAR VALLEY EARTHQUAKE, 1973

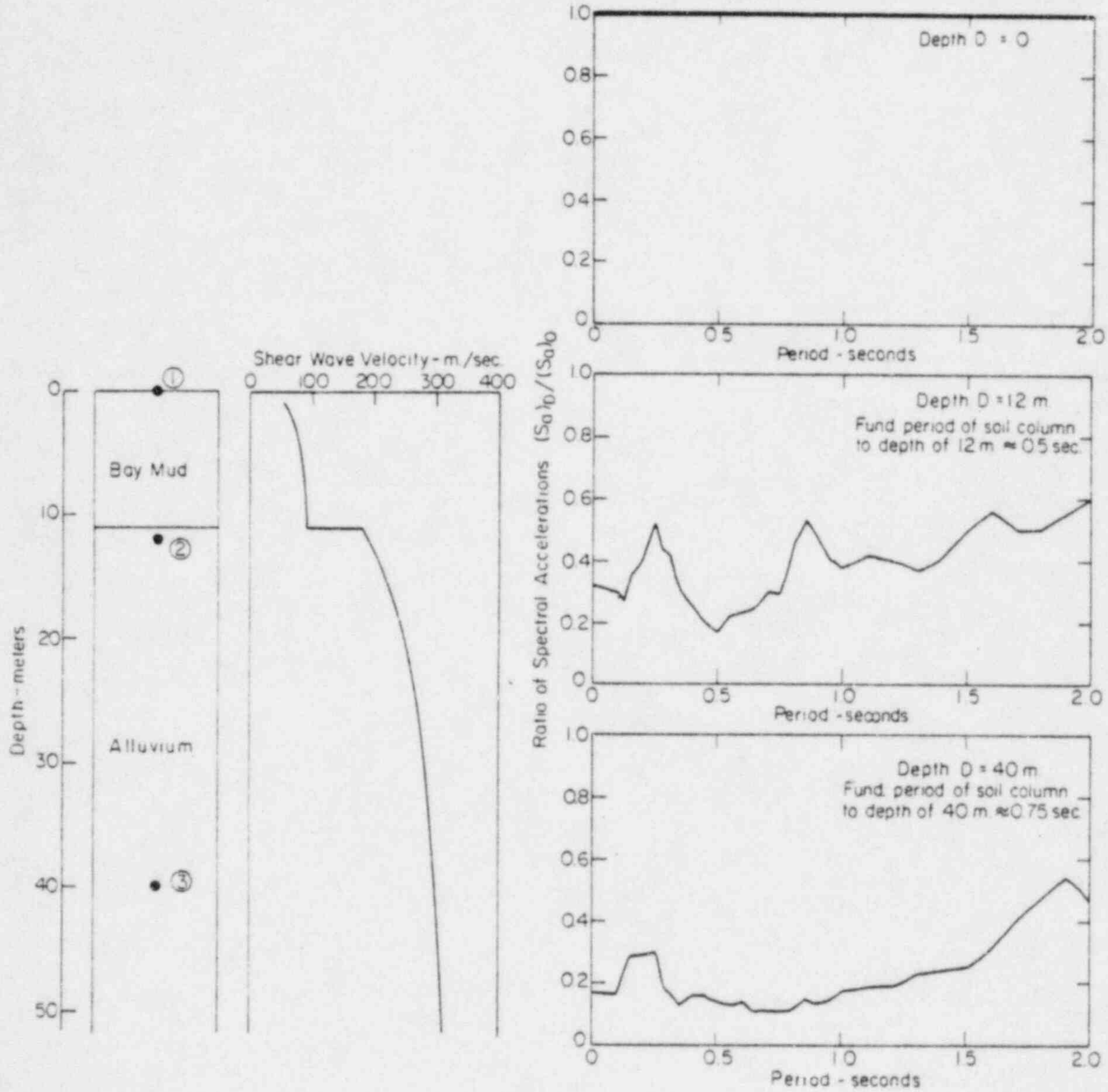


Fig. 22 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS - SALINAS EARTHQUAKE, 1972

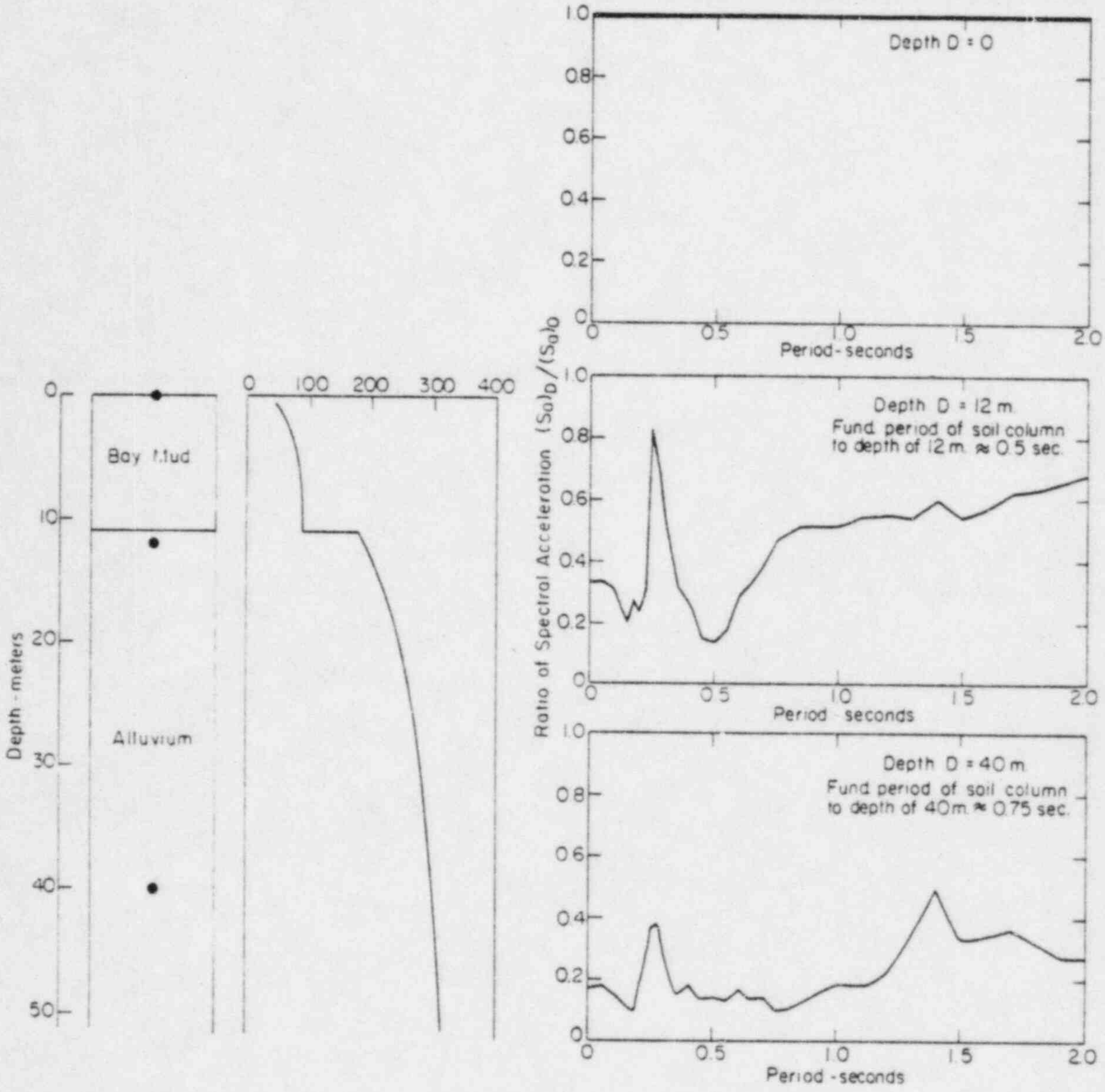


Fig. 23 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS - BEAR VALLEY EARTHQUAKE, 1972

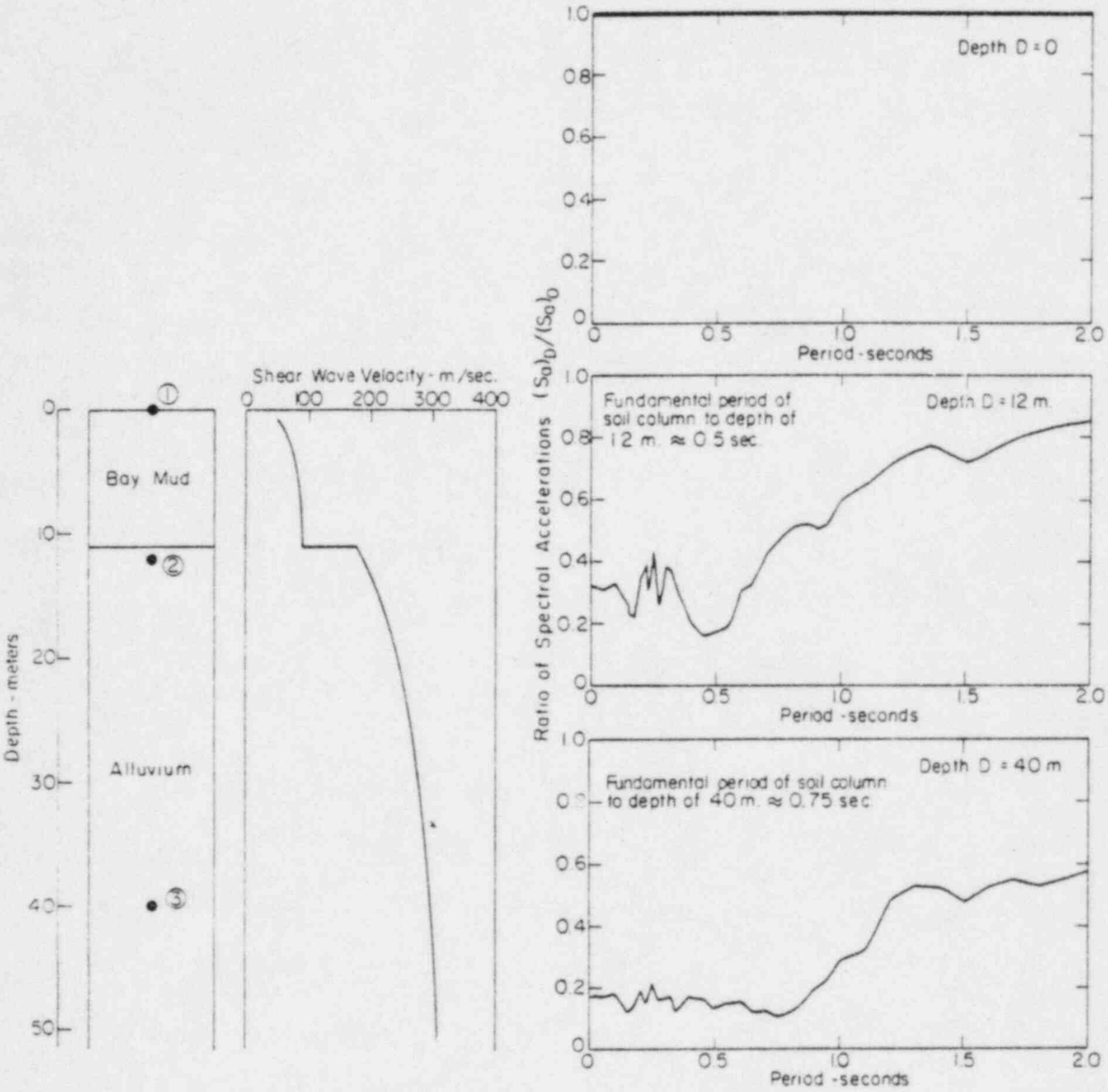


Fig. 24 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS - BEAR VALLEY EARTHQUAKE, 1973

a period of 0.75 seconds, which corresponds to the value of $4z/\overline{V}_s$ for the same deposit. The deterministic value of the frequency suppression effect is clearly evident from this data.

Similar results are also obtained from an analysis of spectra for the motions recorded at Humboldt Bay Power Station (Fig. 18). The spectra for the transverse and longitudinal records of horizontal ground motions at the ground surface and at a depth of 80 ft are shown in Figs. 25 and 26 and the normalized spectra for the same motions are shown in Figs. 27 and 28. Again it is apparent that there is a strong frequency suppression for both transverse and longitudinal motions at a period of about 0.5 sec, which corresponds closely to the computed value of $4z/\overline{V}_s$ for the soil deposit at this site. The great similarity in normalized spectra for transverse and longitudinal motions is shown more clearly in Fig. 29, where the spectra are plotted together and show almost identical characteristics, again illustrating the deterministic nature of this effect.

Hays et al. (1979) and Gazetas and Bianchini (1979) have recently reported data showing similar effects for motions recorded at depths below the ground surface. The data presented by Murphy and West is the average recorded at a distant site for eight nuclear detonations, which closely simulate earthquake effects at such distances. The average normalized spectra for the eight events are shown in Fig. 30, and the frequency suppression corresponding to the period $4z/\overline{V}_s$ is readily apparent.

The same is also true of the motions recorded at Ohgishima Station, Japan and analyzed by Gazetas and Bianchini. The site conditions are shown in Fig. 31 and the normalized spectra in Fig. 32. The frequency suppression effect at a period of 0.3 second at a depth of 15 m is readily apparent.

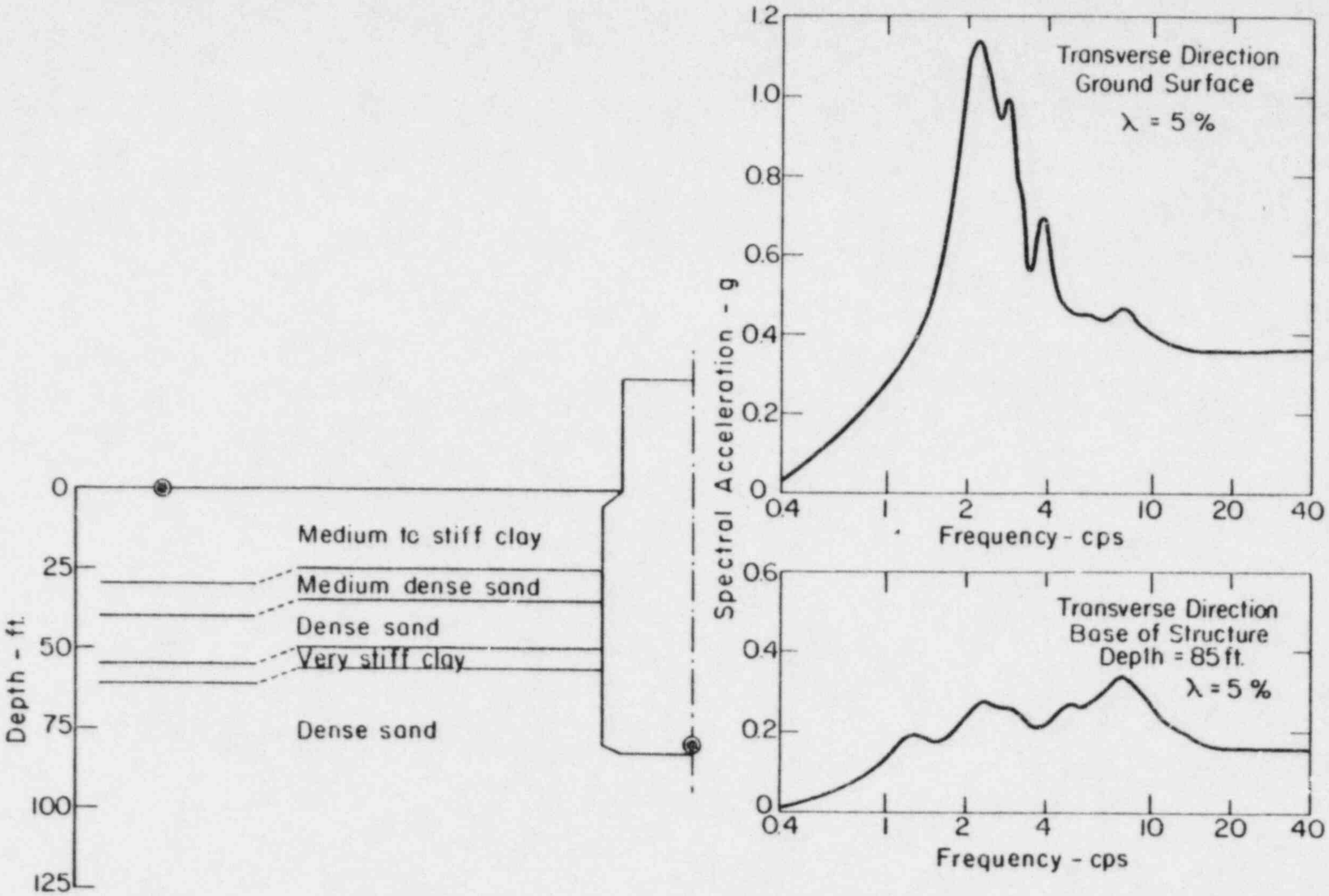


Fig. 25 SPECTRA FOR TRANSVERSE MOTIONS RECORDED AT HUMBOLDT BAY IN 1975 FERNDALE EARTHQUAKE

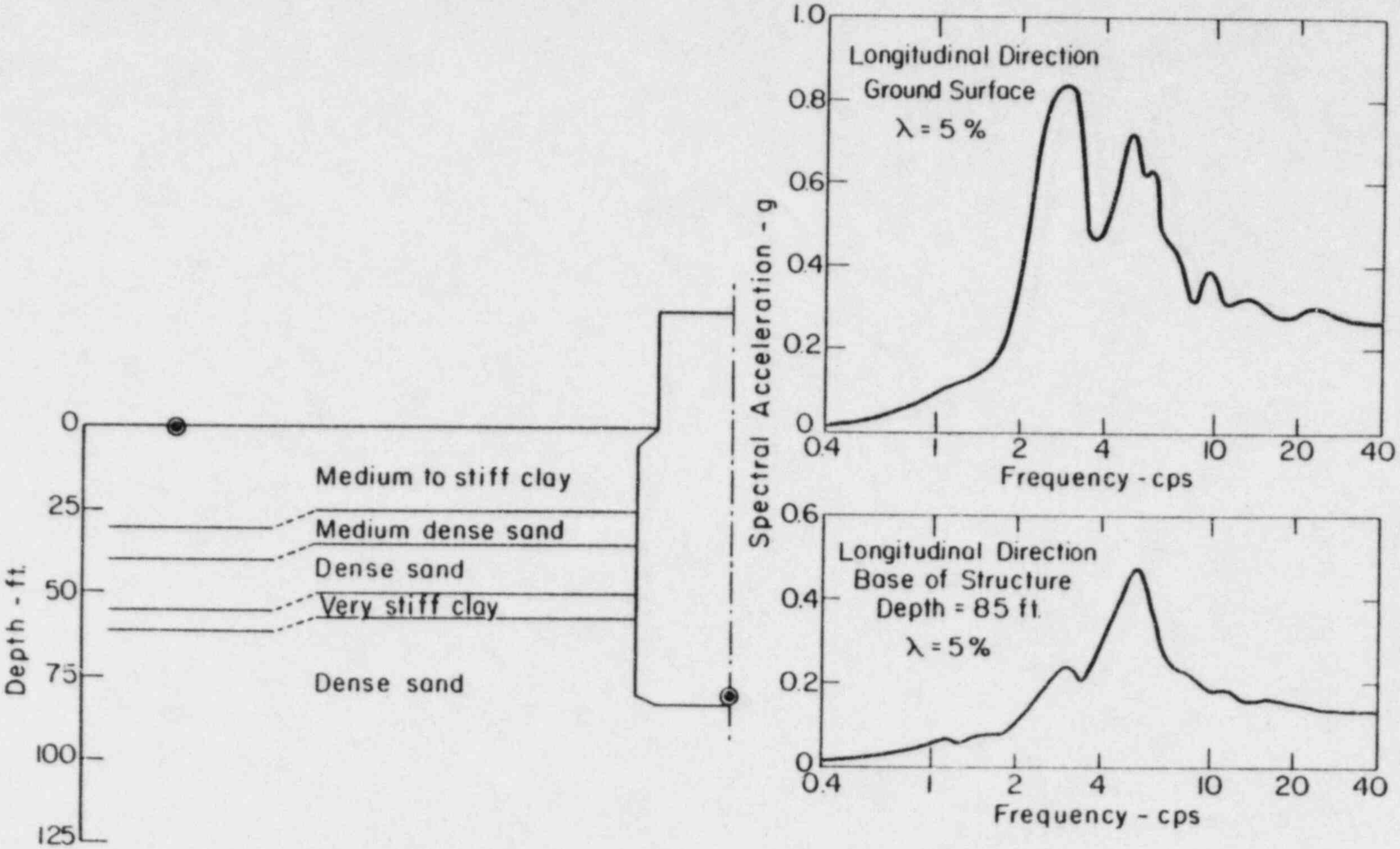


Fig. 26 SPECTRA FOR LONGITUDINAL MOTIONS RECORDED AT HUMBOLDT BAY IN 1975 FERNDALE EARTHQUAKE

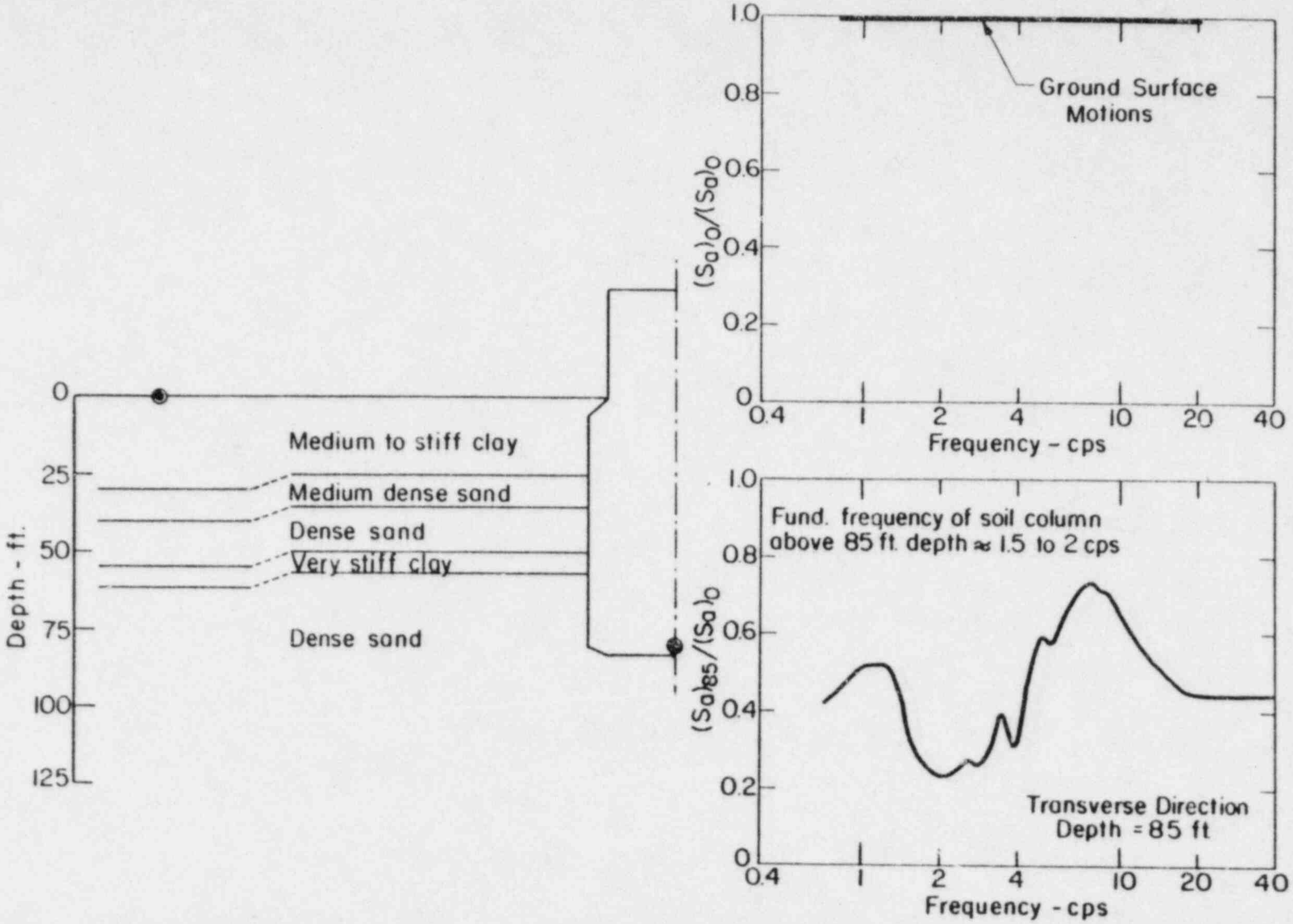


Fig. 27 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS AT HUMBOLDT BAY (FERNDALE EARTHQUAKE, 1975 - TRANSVERSE DIRECTION)

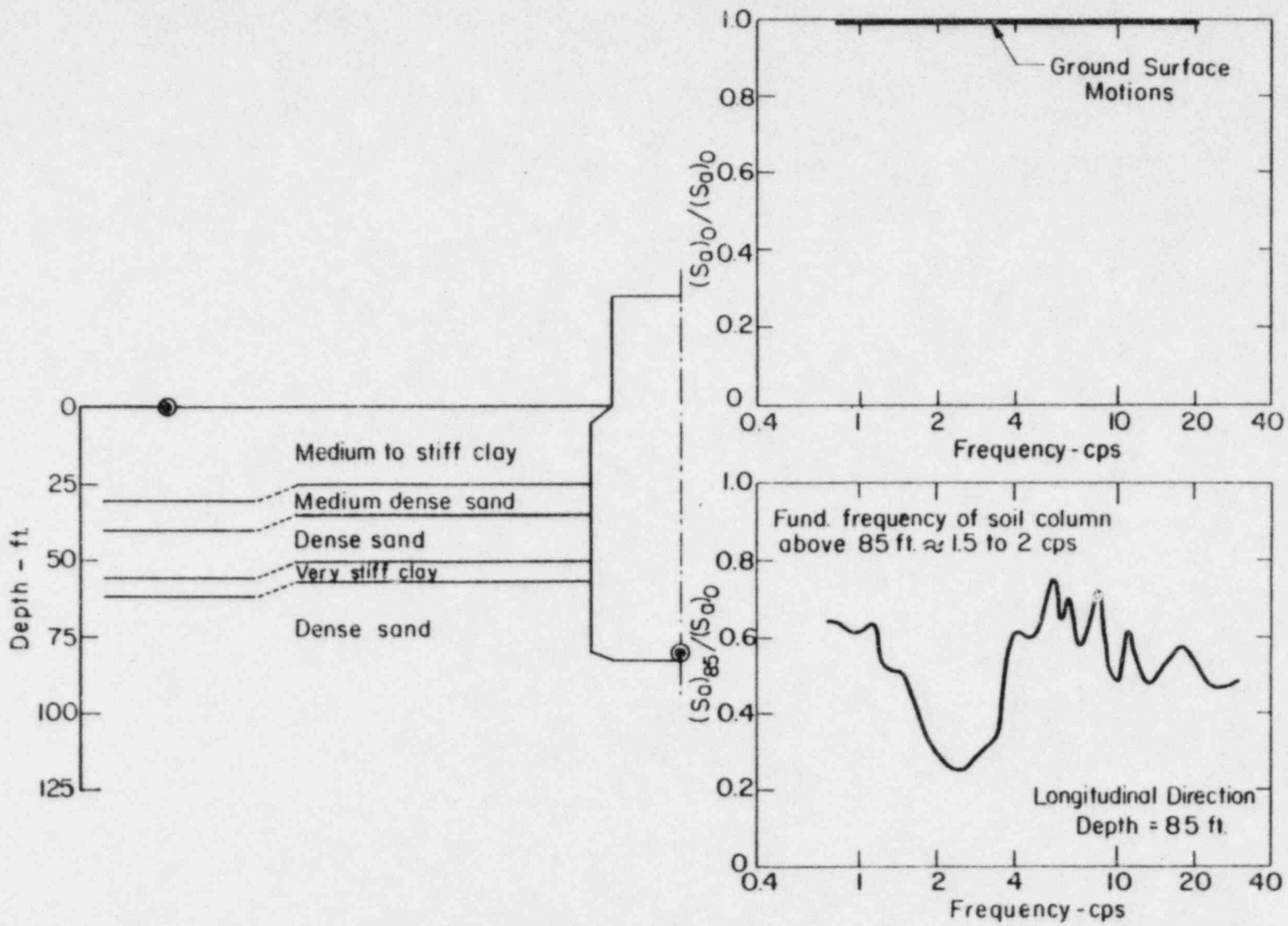


Fig. 28 RELATIVE SPECTRAL ACCELERATIONS AT DIFFERENT DEPTHS AT HUMBOLDT BAY (FERNDALE EARTHQUAKE, 1975 - LONGITUDINAL DIRECTION)

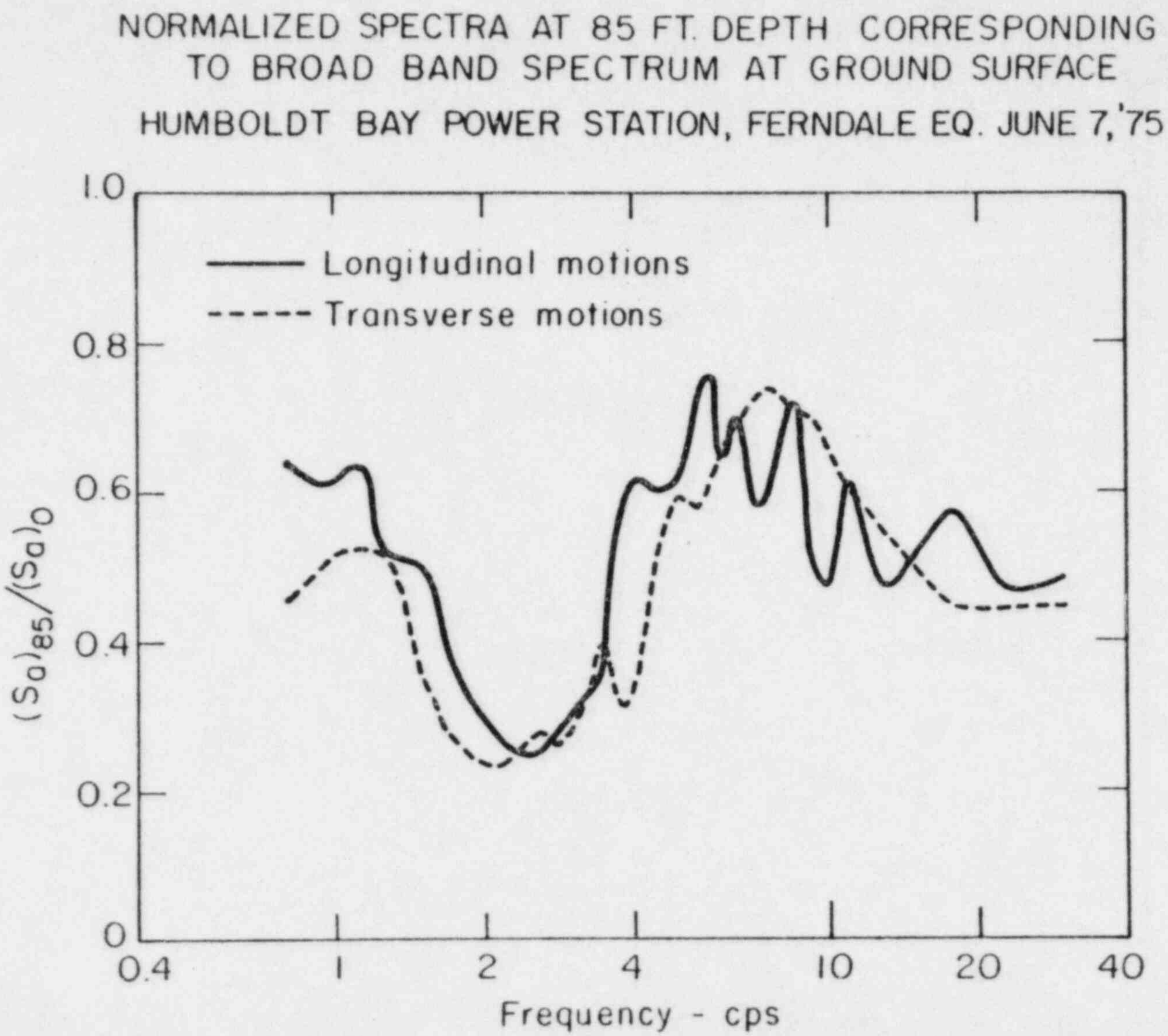


Fig. 29 COMPARISON OF NORMALIZED SPECTRA AT 85 FT DEPTH FOR TRANSVERSE AND LONGITUDINAL MOTIONS

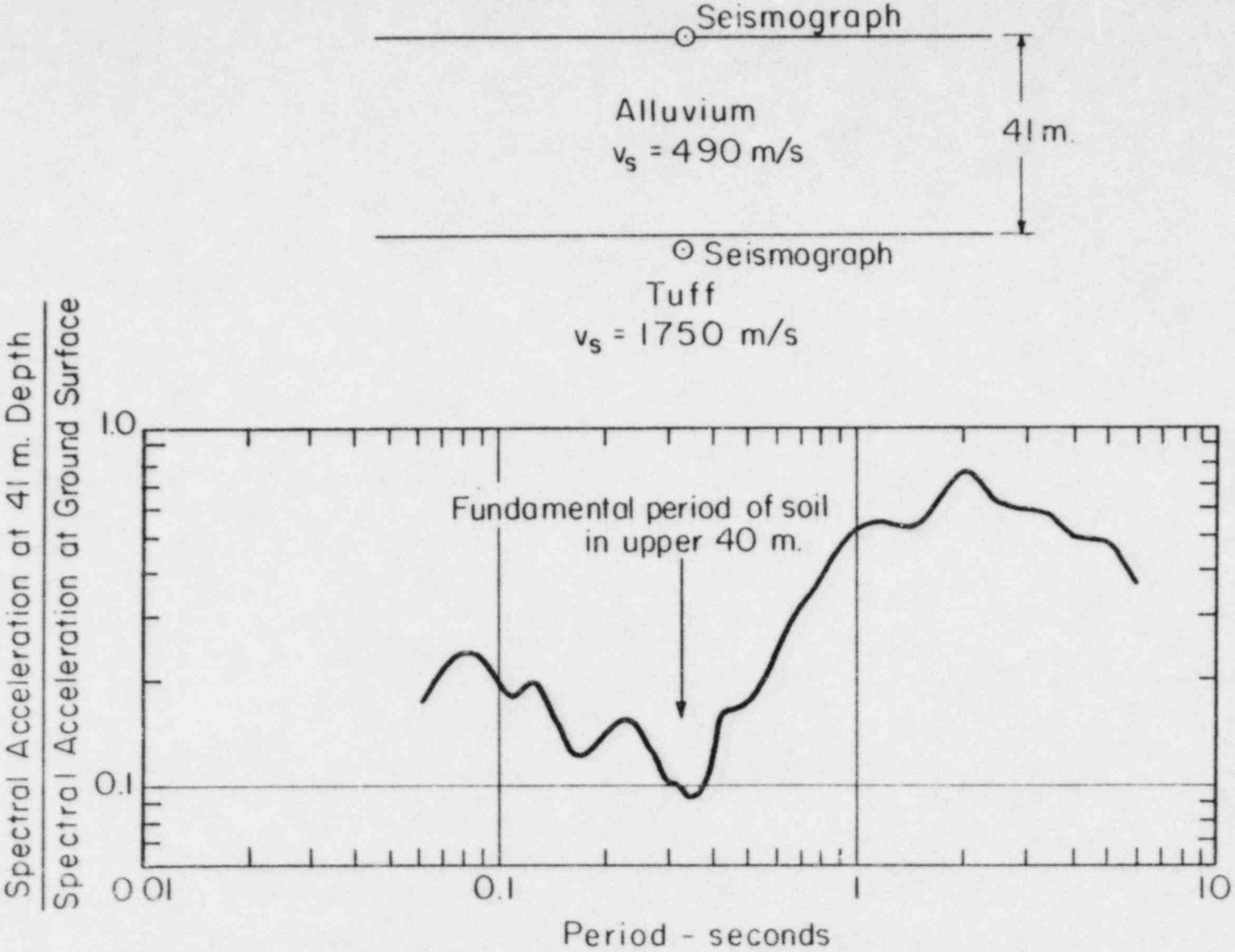


Fig. 30 VARIATION OF HORIZONTAL GROUND RESPONSE WITH DEPTH AT BEATTY, NEVADA -- AVERAGE OF 10 RECORDS

(After Murphy and West, 1974)

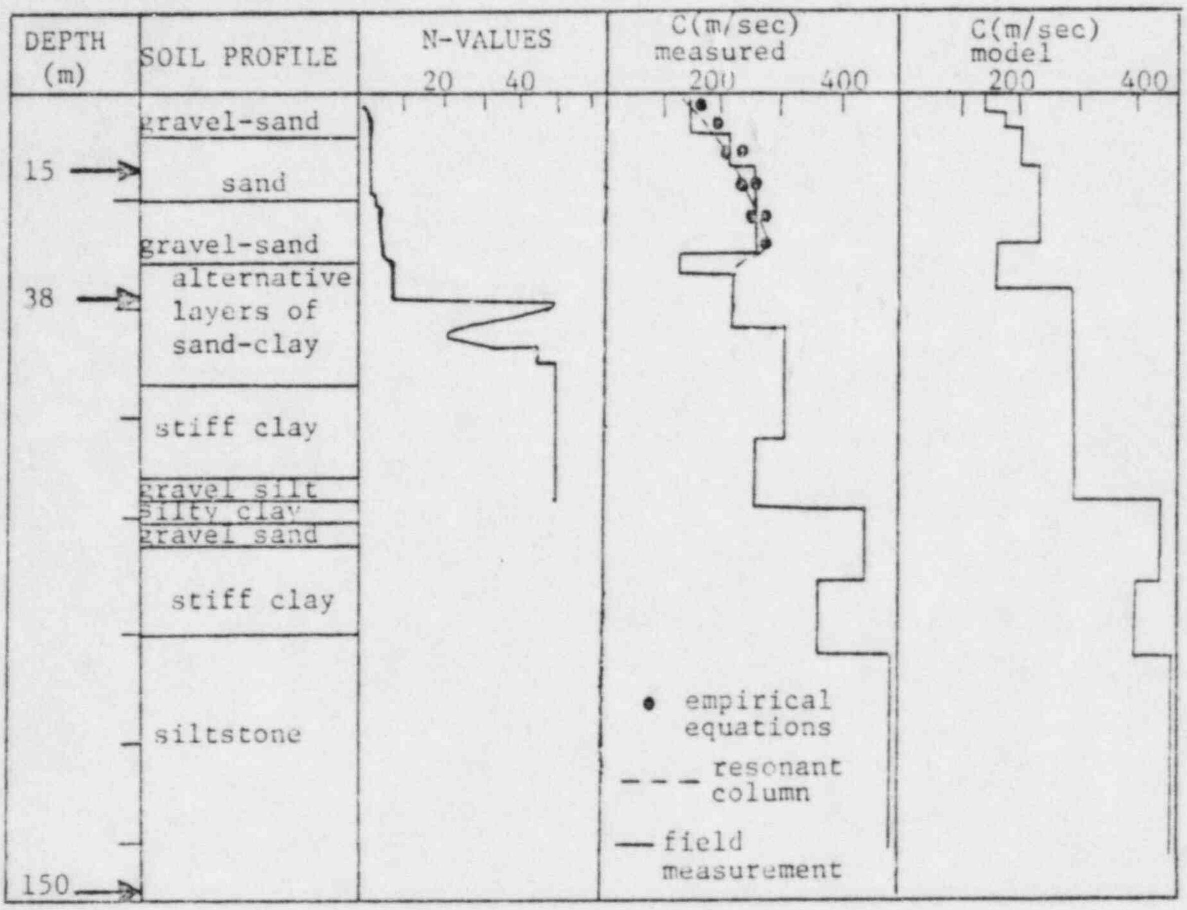


Fig. 31 SITE CONDITIONS AT OHGISHIMA STATION, TOKYO, JAPAN
(After Gazetas and Bianchini, 1979)

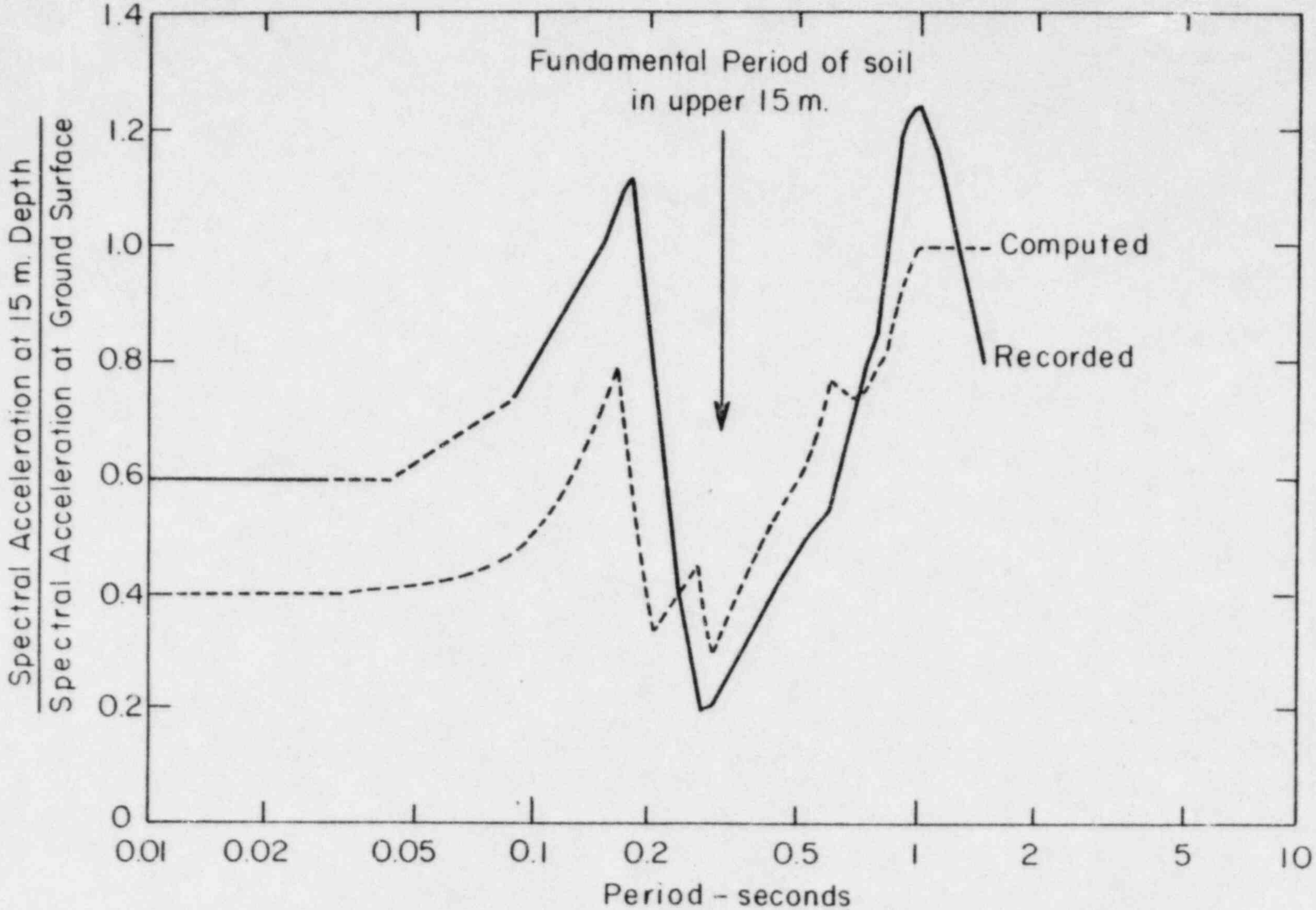


Fig. 32 VARIATION OF HORIZONTAL GROUND RESPONSE WITH DEPTH AT OHGISHIMA STATION, JAPAN
(after Gazetas and Bianchini, 1979)

Gazetas and Bianchini also made ground response analyses for the Ohgishima site using vertically propagating shear wave procedures and computed a similar suppression of frequencies at a period of 0.3 sec. by this procedure. They concluded that analyses of this type tend to underestimate the magnitude of the frequency suppression effect but that in general they provide a reasonable basis for evaluation.

In summary it seems apparent that the frequency suppression effect in soil deposits is not merely an analytical concept but that it is also apparent in recorded data. This agreement between analytical concepts and observational data clearly indicates the need to consider this phenomenon in evaluating site response or soil-structure interaction effects for embedded structures.

Summary and Conclusions

At the outset of this section it was shown that any analyses of soil-structure interaction must necessarily be based on a knowledge of the seismic environment to which the structure will be subjected. This requires an understanding of the spatial distribution of motions in the ground surrounding and underlying the structure.

In the light of the discussion of this subject presented in the preceding pages it seems reasonable to draw the following conclusions concerning the role of the seismic environment in soil-structure interaction analyses.

1. On rock sites structures are likely to be founded near the surface. For such sites, earthquake motions may consist of an unknown mixture of Rayleigh waves, Love waves and near-vertically propagating body waves. Because of the low damping in the rock, attenuation of Rayleigh and Love waves will be small within the general area of the site but the contribution of these types of waves to the total ground

motion, within the frequencies of interest for nuclear power plants, will never-the-less be small. The presence of such waves will tend to increase the rocking and torsional excitation on the base of the structure due to out of phase effects as the waves pass across the base. Thus structures located on rock should be analyzed for these motions to determine the potential severity of their contributions to the total response of the structure. However, the greater part of the response can be considered to result from vertically propagating body waves.

In analyses using vertically propagating waves, however, it should be noted that because of the fact that these waves will in reality be inclined at different angles to the vertical and will be out-of-phase at different points on the base of the structure due to non-homogeneities in the rock through which they must travel, some allowance could be made for the "base-slab averaging effect" which will cause the average motions developed in a stiff base slab to be somewhat less than those developed at individual points on the rock surface.

2. For soil sites, structures are likely to be embedded at some depth (say 20 to 80 ft) below the ground surface. The effects of fundamental Rayleigh and Love waves need not be considered at such sites in the design of nuclear plants because the high frequency components of these waves (greater than 1 Hz) will have been damped out by the soil if it extends to any significant distance (say 1000 ft) around the location of the plant. Higher order Rayleigh modes can be simulated by inclined body waves. Thus the main source of excitation will be inclined body waves and for all practical purposes, these can be analyzed as if they propagated in a vertical direction. However in

soil deposits there will be an important variation in motion characteristics with depth and this should be considered in the analysis if meaningful results are to be obtained. The assumption of uniform motions in the upper layers of a soil deposit is inconsistent with the physical nature of wave mechanics and observations in the field and can only lead to misleading results unless the specified control motion is intended to take the natural variations in motion characteristics into account in some way. Without knowing something about the variations in motion it is difficult to see how this can be done realistically without introducing an unwarranted degree of conservatism into the soil-structure analysis procedure.

It should not be construed from the above statements that the assumption of vertically propagating waves at soil sites is appropriate for all types of structures. The long-period components of horizontally propagating waves may be extremely important for the design of buried pipelines, tunnel linings and earth retaining structures. However, except for increased stresses in the walls of buried or embedded structures the change in the stress field due to these waves appear to have little effect on the overall horizontal motions of such structures.

The propagating nature of the displacement field may also induce additional displacements and stresses in long above-ground structures such as bridges, Bogdanoff et al. (1965), Johnson and Galletly (1972), Abdel - Ghaffar and Trifunac (1976); and rocking and torsional motions in long period single structures, Wong (1975), Scanlan (1976) and Wong and Luco (1976).

Finally, and perhaps most important, control motions should be chosen with due respect to site conditions and, if a broad band design spectrum is used the control point should be located at the ground surface or, alternatively,

at an imaginary outcrop, where it could conceivably exist, and not at some arbitrary depth below the ground surface where the boundary conditions resulting simply from the existence of a ground surface preclude this possibility.

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Part III

METHODS OF SOIL-STRUCTURE INTERACTION ANALYSIS

The topic of soil-structure interaction analysis is extremely broad and it is not possible herein to report on all the types of soil-structure interaction problems which may occur in practice and all of the sophisticated methods of analysis which have been proposed in the literature. Rather, the authors will continue the generic discussion started in Part I, highlighting some important developments and considerations. Readers interested in further details are referred to several recent state-of-the-art reports on the topic, Refs. 1 and 2, Yoshimi et al. (1977), and Lysmer (1978).

As discussed in Part I most true interaction problems can only be solved by linear or equivalent linear methods. When this fact is considered together with the significant mathematical advantages of the assumption of linearity it is not surprising that the greater part of the literature on soil-structure interaction refers to linear systems and so will most of this chapter.

SUBSTRUCTURE METHODS

The mathematical problems involved in modeling the essentially semi-infinite soil mass and a desire to break complicated soil-structure interaction problems into more manageable parts have led to a large number of methods in which the soil mass is treated as a continuum and the structure as a discretized model. The half-space is analyzed first, usually in the frequency domain, and the impedance and scattering properties at the soil-structure interface are established. In the second step these properties are used as boundary conditions in a dynamic analysis of the structure with

a loading which depends on the free-field motions. In recent years several substructure methods have appeared in which the half-space solution is obtained by finite element analysis with transmitting boundaries.

Surface Structures

For the case of structures founded at the surface, substructuring becomes extremely simple. According to the theory given in Part I, the interaction displacements may be found from the simple model shown in Fig. 1(a) where all forces, Q_i, act at the base and the first story of the structure. These forces can be computed from Eq. (5) (Part I) which for this case reduces to

$$\{Q_i\} = - [M_s]\{\ddot{u}_f\} - [C_s]\{\dot{u}_f\} - [K_s]\{u_f\} \tag{1}$$

where the matrices $[M_s]$, $[C_s]$, and $[K_s]$ depend on the properties of the structure only, and $\{u_f\}$ need to be known only at the ground surface. This model is valid for any wave pattern in the free field, but has, as far as the writers can ascertain, rarely been used in engineering practice.

For the special case of vertically propagating body waves (only shear waves will be considered here) the model shown in Fig. 1(a) can be further simplified. As mentioned in connection with the discussion of Eq. (1) in Part I, the free field displacements for the upper nodes of the structure can be chosen completely arbitrarily. Thus, if all horizontal free-field displacements of the structure are chosen equal to the free-field horizontal surface control motion, y(t), and the vertical displacements are set equal to zero, the structure does not deform when subjected to the free-field motions and Eq. (5) of Part I reduces to:

$$\{Q_i\} = - [M_s]\{\ddot{u}_f\} \tag{2}$$

Thus the relative displacements can be computed by the well-known inertial

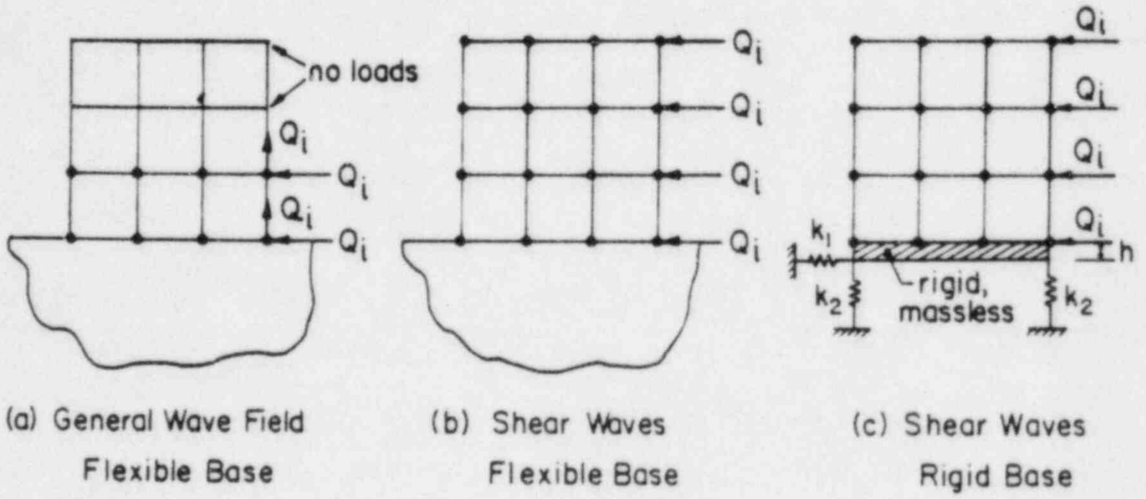


Fig. 1 ANALYSIS OF SURFACE STRUCTURES

interaction model shown in Fig. 1(b) where all forces are the product of the local mass and the free field surface acceleration.

With a finite element representation of the soil mass the flexibility of the base slab can be easily considered with both of the above models. However, by assuming a rigid base slab the last model may be reduced to the even simpler model shown in Fig. 1(c). The frequency dependent impedances k_1 and k_2 can be determined from the impedance coefficients in the corresponding foundation vibration problem, and so can the thickness h of the massless base slab. The parameter h represents the interaction between sliding and rocking and is in general a complex-valued function of frequency. In practice, it is common to neglect this interaction, i.e., to set $h = 0$. This is often justified by the work of Veletsos and Wei (1971), who have shown that the cross-interaction is relatively weak for surface structures. Also, it is common to use constant springs, i.e., to neglect the frequency dependency of k_1 and k_2, see Ref. (1).

Recently, several continuum solutions have been presented for the response of rigid surface foundations to horizontally propagating waves in a half-space:

Strip footings:	Flitman (1962), Oien (1971)
Circular footings:	Kobori et al. (1973, 1976), Luco (1976)
Rectangular footings:	Savidis and Richter (1977), Luco and Wong (1977), Wong and Luco (1978)
Arbitrary shape:	Wong (1975), Wong and Luco (1976)

The latter work has resulted in the development of a computer code CLASSI which can handle structures with rigid foundations of arbitrary shape on the surface of a layered viscoelastic half-space. However, none of the above methods can be used to analyze embedded structures which therefore merit special discussion.

Embedded Structures

Most real structures are embedded in the ground. The effects of embedment can be quite strong and are not easily considered by substructure methods. The basic difficulty in solving the embedded soil-structure interaction problem is that, because the free-field motions vary considerably with depth, especially in softer materials, it is difficult to specify the distribution of the free-field forces on the embedded part of the structure. This observation has led Hall and Kissenpfenning (1975) to modify the model shown in Fig. 1(c) to the model shown in Fig. 2. In this model, which assumes the control motion, $\ddot{y}$, at the base level of the structure and vertically propagating shear waves, the masses m_1 and m_2 represent a lumped mass model of the free field. These masses are chosen so large that they are not influenced by the motion of the structure and they will therefore have the correct relative free-field displacements. The impedances, k_i, are usually assumed constant and are obtained from approximate theories, say Johnson et al. (1975) or Novak and Beredugo (1972). While this method cannot be justified rigorously it has been shown to work reasonably well for a number of cases when checked against more complete finite element analysis. Rigorous substructure methods for embedded foundations have been proposed by several authors. They fall into three distinct groups according to the number of degrees-of-freedom at which the half-space and the structure interacts. A comparison of the methods will be given at the end of this section.

Rigid boundary methods

Kausel and Roesset (1974) have proposed a rigorous 3-step method for the case of rigid embedded foundations. This procedure is illustrated in Fig. 3. The first step is a site response (scattering) problem in which the site includes a rigid massless foundation with the same shape as the actual

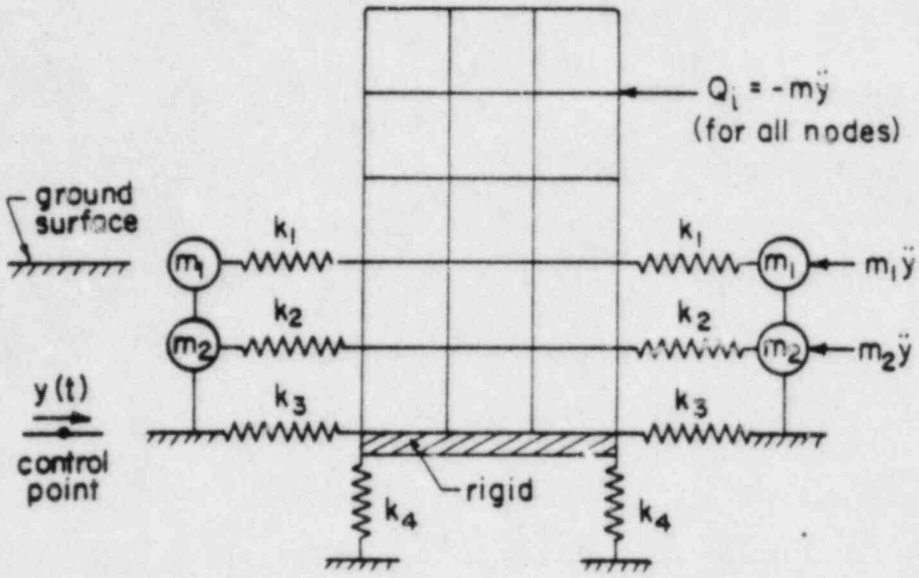


Fig. 2 THE HALL-KISSENPFENNIG MODEL

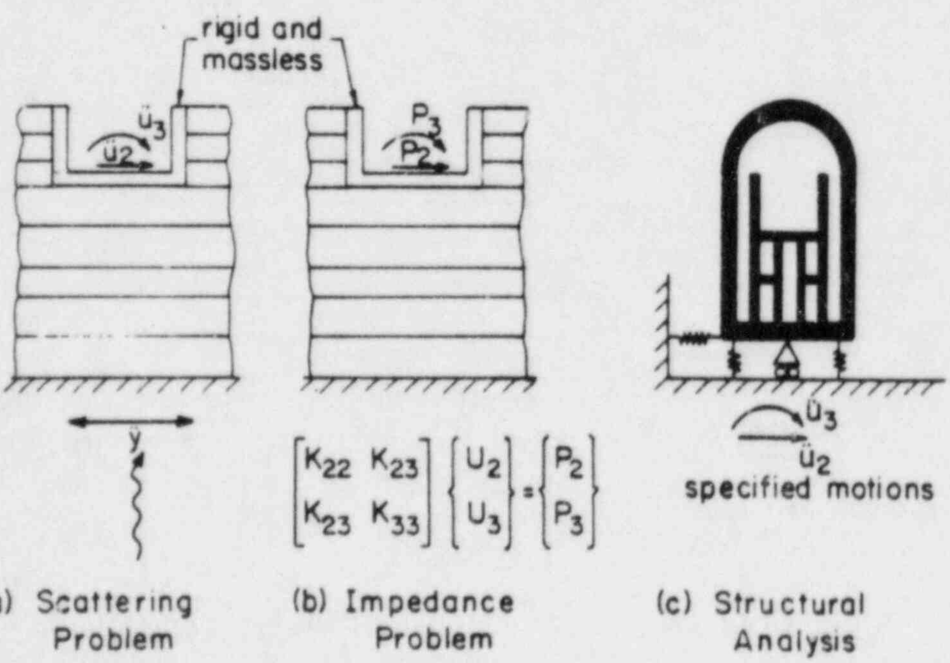


Fig. 3 3-STEP METHOD FOR INTERACTION ANALYSIS

foundation. The solution to this problem produces a set of rigid body free-field accelerations, $\ddot{u}_i$, for points in the structure. The second step is a foundation vibration (impedance) problem the solution to which produces the impedance matrix for the foundation and with this the springs and dashpots to be used in the last step of the analysis where the loading on the structure is computed from the free-field motions obtained from Step 1 using Eq. (2). The total displacements follow from Eq. (2) (Part I). While the Kausel-Roesset method has been presented here only for the case of vertically propagating shear waves it is in fact applicable with other wave fields. Unfortunately, rigorous solutions to the first step are difficult to obtain except by finite element methods and since the second step also requires a finite element analysis the complete 3-step method is hardly competitive with other finite element methods discussed below for obtaining rigorous solutions. However, with approximate solutions to Step 1, Kausel et al. (1978), and Step 2, Novak and Beredugo (1972), Novak (1974), Kausel and Roesset (1975), the method does lead to very economical analysis for the case of vertically propagating waves.

A different, but similar, formulation of the rigid embedded foundation interaction problem has been proposed by Luco et al. (1975) and used by Day (1977, 1978) to determine the response of structures on cylindrical embedded foundations subjected to inclined SH-waves. The method is similar to the 3-step method discussed above in that it usually requires two finite element analyses of the models shown in Fig. 3(a) and Fig. 3(b). The first analysis (the scattering problem) involves determining the forces which the free-field motions exert on the footing when this is held fixed in space. The second analysis (the impedance problem) is identical to that of the above procedure. The third step, which involves matrix algebra is as simple to

solve as Step 3 in Fig. 3(c). Structural engineers may recognize this method as a sophisticated version of the slope-deflection method. The method can, in principle, consider structure-soil-structure interaction.

A few continuum solutions have been obtained to the above scattering and impedance problems for embedded shapes in a perfect half-space. Thus, Thau and Umek (1973, 1974), Thau et al. (1974), Dravinski and Thau (1976a, 1976b), Luco et al. (1975), and Trifunac (1972) have developed solutions for a number of plane-strain problems involving rigid footings of rectangular and semi-circular cross sections. The most common excitation is SH-waves. The only three-dimensional problems which have been solved by continuum methods are: rigid semi-spherical and -ellipsoidal foundations, Luco (1976b), Apsel and Luco (1976) and embedded cylinders, Apsel (1979). The latter provides a solution for the impedance problem only. Considering the mathematical difficulties in obtaining continuum solutions for more complicated shapes embedded in layered systems and excited by different seismic environments, it must realistically be assumed that for practical problems both the scattering problem and the impedance problem have to be solved by finite element or finite difference methods.

The major disadvantage of the rigid boundary methods is of course that they assume a rigid basement. Thus, unless it can be shown in each case that this assumption is valid, these methods provide no means of evaluating the actual deformations and stresses in the embedded part of the structure.

Flexible boundary methods

If the flexibility of the embedded part of the structure is to be considered a flexible boundary method must be used. The basic elements of a theory for a rigorous finite element flexible boundary method has been developed by Gutierrez (1976) and Gutierrez and Chopra (1977, 1978).

As with the methods discussed above, the complete solution of a soil-structure interaction problem by the flexible boundary method requires first the elevation of a site response (scattering) problem similar to that shown in Fig. 3(a) to determine the motion of the now flexible boundary, and second the solution of an impedance problem similar to the one stated in Fig. 3(b). The latter problem now involves more degrees-of-freedom and leads to a larger impedance matrix. The third step involves an analysis of the structure alone and is only slightly more complicated than the problem shown in Fig. 3(c).

Unfortunately, as of this time, no satisfactory method has emerged for the solution of the scattering part of the problem. Thus the method has only been used for the analysis of surface structures, excited by vertically propagating body waves, Chopra and Gutierrez (1973) and Aydinoglu and Cakiroglu (1977).

Flexible volume methods

Both the scattering and impedance problems for the above methods can be greatly simplified if more common degrees-of-freedom between the half-space and the structure are included in the interaction problem. Let the soil and the structure be partitioned not at the interface but according to the partitioning shown in Fig. 4. In this partitioning the structure, Fig. 4(c), consists of the superstructure plus the basement minus the excavated soil, and the foundation consists of the original site, Fig. 4(b), i.e., the soil to be excavated is retained with the foundation. Interaction between the structure and the foundation occurs at all basement nodes.

The equation of motion for the complete problem, Fig. 4(a) is:

$$[M]\{\ddot{\hat{u}}\} + [K]\{\hat{u}\} = \{\hat{Q}_b\} \tag{3}$$

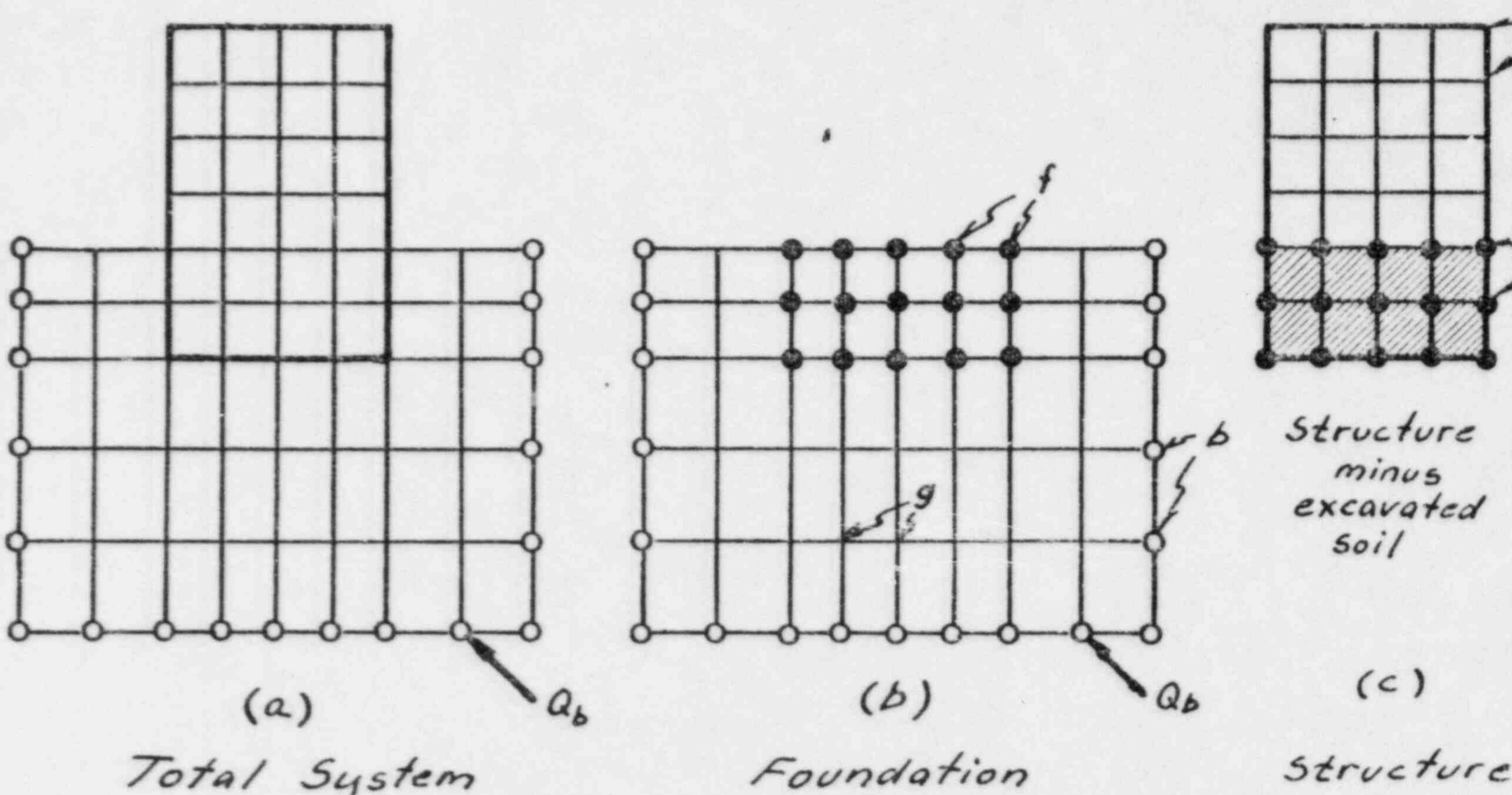


Fig. 4 SUBSTRUCTURING FOR THE FLEXIBLE VOLUME METHOD

where [M] and [K] are the total mass and stiffness matrices, respectively. $\{\hat{u}\}$ is the vector of nodal point displacements and $\{\hat{Q}_b\}$ are the external forces. Since the source of excitement is outside the model, $\{\hat{Q}_b\}$ has non-zero elements only at the degrees of freedom corresponding to boundary nodes, which are assumed to be far away from the structure.

For harmonic excitation of the frequency ω the load and displacement vectors can be written

$$\{\hat{Q}_b\} = \{Q_b\}e^{i\omega t} \tag{4}$$

and

$$\{\hat{u}\} = \{u\}e^{i\omega t} \tag{5}$$

where $\{Q_b\}$ and $\{u\}$ now contain complex force and displacement amplitudes. Hence, for each frequency the equation of motion takes the form

$$[C]\{u\} = \{Q_b\} \tag{6}$$

where [C] is a complex, frequency dependent stiffness matrix:

$$[C] = [K] - \omega^2[M] \tag{7}$$

Similar equations of motion can be written for each of the substructures in Fig. 4. Introducing the following subscripts which refer to degrees of freedom (DOF) associated with different nodes:

Subscript	Nodes	
s	superstructure	
i	basement	See also Fig. 4
f	excavated soil	
b	external boundary	
g	remaining soil	

the equation of motion for the foundation in Fig. 4(b) can be written

$$\begin{bmatrix} C_{ff} & C_{fg} & C_{fb} \\ C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{bmatrix} \begin{Bmatrix} u_f \\ u_g \\ u_b \end{Bmatrix} = \begin{Bmatrix} Q_f \\ 0 \\ Q_b \end{Bmatrix} \tag{8}$$

where $\{Q_f\}$ are the interaction forces from the structure in Fig. 4(c). Similarly, the equation of motion for the structure in Fig. 4(c) can be written:

$$\begin{bmatrix} C_{ss} & C_{si} \\ C_{is} & (C_{ii} - C_{ff}) \end{bmatrix} \begin{Bmatrix} u_s \\ u_f \end{Bmatrix} = \begin{Bmatrix} 0 \\ -Q_f \end{Bmatrix} \tag{9}$$

where compatibility of displacements $(u_i = u_f)$ and equilibrium $(Q_i + Q_f = 0)$ has been enforced. The term $(C_{ii} - C_{ff})$ simply indicates the stated partitioning according to which the stiffness and mass of the excavated soil is subtracted from the stiffness of the structure.

Assuming that the external boundary is very far away from the structure the equation of motion for the free field (Fig. 4(b) can be written:

$$\begin{bmatrix} C_{ff} & C_{fg} & C_{fb} \\ C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{bmatrix} \begin{Bmatrix} u'_f \\ u'_g \\ u'_b \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ Q_b \end{Bmatrix} \tag{10}$$

where $\{u'\}$ are the free-field motions (the solution to the site response problem). Subtraction of Eq. (10) from Eq. (8) yields:

$$\left[\begin{array}{cc|c} C_{ff} & C_{fg} & C_{fb} \\ \hline C_{gf} & C_{gg} & C_{gb} \\ C_{bf} & C_{bg} & C_{bb} \end{array}\right] \left\{\begin{array}{c} r_f \\ \hline r_g \\ r_b \end{array}\right\} = \left\{\begin{array}{c} Q_f \\ \hline 0 \\ 0 \end{array}\right\} \tag{11}$$

where $\{r\} = \{u\} - \{u'\}$ are the interaction displacements. By partitioning Eq. (11) as indicated $\{r_g\}$ and $\{r_b\}$ can be eliminated and $\{Q_f\}$ can be expressed in the form:

$$\{Q_f\} = [X_f]\{r_f\} = [X_f](\{u_f\} - \{u_f'\}) \tag{12}$$

The frequency dependent matrix $[X_f]$ is the impedance matrix corresponding to the nodal points indicated by heavy dots in the foundation (free field) model, Fig. 4(b). It can be determined as the inverse of the flexibility matrix for this model, i.e. from the solution for point loads in a uniformly layered system.

Substitution of Eq. (12) into Eq. (9) yields:

$$\begin{bmatrix} C_{ss} & C_{si} \\ C_{is} & (C_{ii} - C_{ff} + X_f) \end{bmatrix} \begin{Bmatrix} u_s \\ u_f \end{Bmatrix} = \begin{Bmatrix} 0 \\ [X_f]\{u_f'\} \end{Bmatrix} \tag{13}$$

from which the final motions of the structure can be determined. Except for the term C_{ff} which corresponds to the excavated soil this equation is similar to that used in the general substructure method proposed by Gutierrez (1976).

Thus the solution of the soil-structure interaction problem has been reduced to three steps:

1. <u>Solve the site response problem</u>

   to determine the free field motions $\{u_f'\}$ within the embedded part of the structure, see Fig. 5(a).

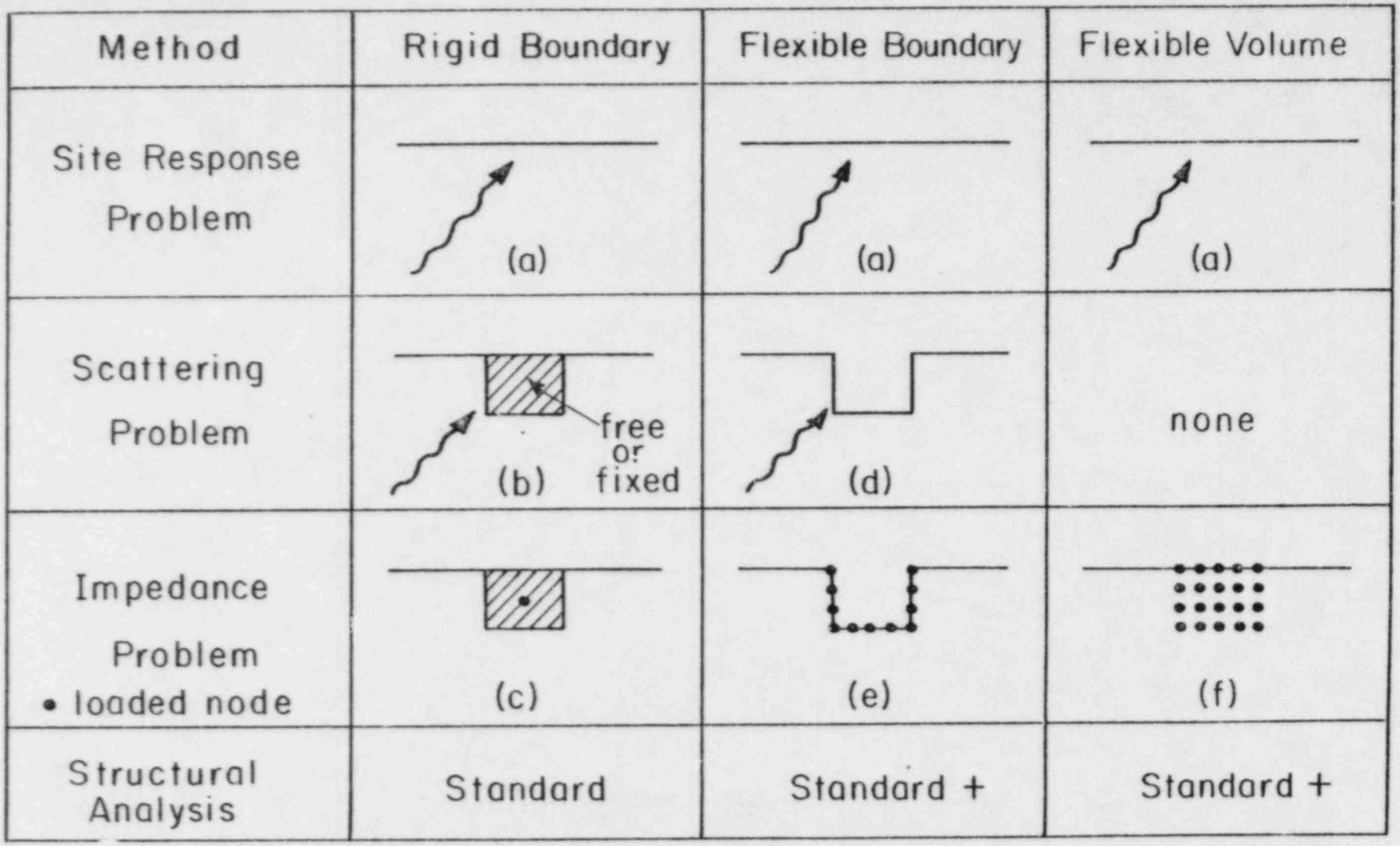
2. <u>Solve the impedance problem</u>

   to determine the matrix $[X_f]$, see Fig. 5(f).

3. <u>Solve the structural problem</u>

   This involves forming the complex stiffness matrices and load vector shown in Eq. (13) and solving this equation for the final displacements.

For horizontally layered sites the site response problem can be solved by the methods described in Part I. Current state-of-the-art would allow



Method	Rigid Boundary	Flexible Boundary	Flexible Volume
Site Response Problem	(a)	(a)	(a)
Scattering Problem	(b) free or fixed	(d)	none
Impedance Problem • loaded node	(c)	(e)	(f)
Structural Analysis	Standard	Standard +	Standard +

Fig. 5 SUMMARY OF SUBSTRUCTURE METHODS

the consideration of any superposition in three dimensions of plane vertical and inclined body waves and surface waves in horizontally layered visco-elastic systems with nonlinearities considered by the equivalent linear method.

The scattering problem has been eliminated, see Fig. 5, since it is identical to the site response problem. And the impedance problem is simpler than any of the problems shown in Figs. 5(c) and 5(e) because of the more regular surface boundary. It can be solved as the inverse of the flexibility problem by state-of-the-art methods for plane systems, Waas (1972), for axisymmetric systems, Kausel (1974), and approximately for general three-dimensional configurations by appropriate use of the axi-symmetric solution, Wolf and von Arx (1978).

The structural analysis step is only slighly complicated by the addition of more interacting nodes and follows the same procedure as for the flexible boundary methods.

A finite element computer code, SASSI, which implements the flexible volume method in three dimensions is currently being developed by the writers at the University of California at Berkeley.

Overview of Substructure Methods

A major argument for using substructure methods in design has been that they are simpler and cheaper to perform. This is obviously so for the case of a single structure with rigid foundation on the surface of a uniform half-space. However, most real structures have flexible basements of complicated shapes embedded in a layered half-space and for such situations substructure interaction analysis is anything but simple. Simplified methods have been devised for single embedded structures on rigid foundations of regular shape, Hall and Kissenpfennig (1975), Kausel et al. (1978). However,

these methods work only for vertically propagating waves and cannot consider structure-structure interaction. This leaves us for more general situations with the three methods shown in Fig. 5, all of which for practical situations require finite element, finite difference or other complicated methods of analysis.

The rigid boundary methods obviously cannot consider the flexibility of the basement and like the flexible boundary methods they involve two complicated problems (scattering and impedance) which for general geometries require two finite element analyses.

It would therefore appear that for embedded structures the rigid and flexible boundary methods are not competitive cost-wise with the equally or more rigorous complete methods described below (unless simplifying assumptions are made), but that the flexible volume method might be.

Actually, the substructure methods apply only to linear problems. If the nonlinear properties of the soil deposit have to be considered in detail, say by the equivalent linear method, it is preferable to use one of the complete methods described in the following section. On the other hand, a major advantage of the substructure methods is that once the scattering and impedance problems have been solved they do not have to be repeated if the properties of the structure are changed in the design process. Similarly, if the seismic environment is changed the impedance analysis does not have to be repeated.

COMPLETE METHODS

Complete methods are here defined as methods in which the motions of the soil mass and the structure are determined simultaneously. Complete methods are most often of the finite element type and only this type will be

discussed in this section. The complete methods fall into two groups (a) Pseudo-interaction methods, which do not allow for energy dissipation through all boundaries of the finite element model and (b) True interaction methods which do. The pseudo methods are by far the most commonly used in current practice.

Pseudo-Interaction Methods

Figure 6 shows a typical computation scheme for pseudo-interaction analysis of a structure for vertically propagating waves, first introduced by Seed and Idriss (1973) and Seed et al. (1975), (see also Part IV of this report). In this procedure the site response problem is solved first, see Fig. 6(a), by deconvolution of the surface control motion to some level below the ground surface where it can be assumed that the presence of the structure will not influence the ground motion. A typical depth is one structure diameter below the base of the structure.

In the second step the computed base motion is used as a specified boundary motion for a finite element analysis of the soil-structure system shown in Fig. 6(b).

The finite element model used must be very wide or must be equipped with transmitting lateral boundaries in order to avoid reflections, Lysmer et al. (1975), Seed et al. (1975). However, numerous computations, e.g., Gomez-Masso (1978), have shown that for typical seismic motions only insignificant reflections will occur from the lower rigid boundary on which the control motion is specified. This is so because the interaction wave field consists mainly of surface waves which do not penetrate deeply into the ground. Thus, this pseudo method is essentially a true method when it is used correctly.

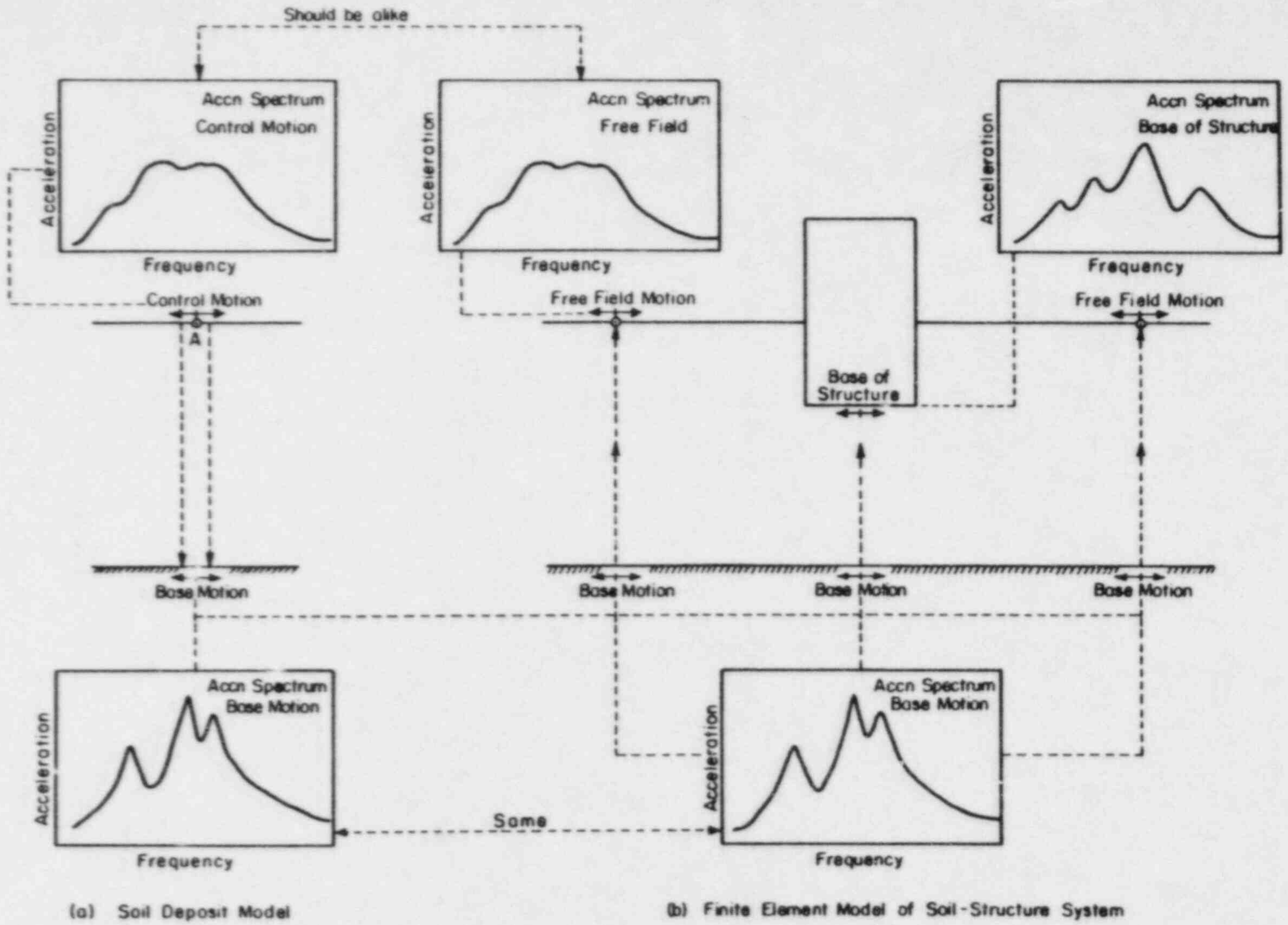


Fig. 6 SCHEMATIC REPRESENTATION OF SOIL-STRUCTURE INTERACTION ANALYSIS USING FINITE ELEMENT MODEL

The computational scheme indicated in Fig. 6 may in principle be used to analyze any three-dimensional soil-structure interaction problem involving vertically propagating waves and a horizontally layered site. However, well-known restrictions on computer cost and capacity limit our current ability in treating the third dimension rigorously except for the case of axial symmetry of the structure, Kausel and Roesset (1974), Berger (1976), Berger et al. (1975a, 1975b), Tajimi and Shimomura (1976), Shimizu et al. (1977). An approximate method for three-dimensional analysis has been presented by Hwang et al. (1975). The model used by Hwang is shown in Fig. 7. The three-dimensional effect is achieved by placing Lysmer-Kuhlemeyer (1969) dashpots at all nodal points of the plane-strain finite element since analyzed. These dashpots are only active on the interaction velocities and simulate the propagation of shear waves in the direction perpendicular to the plane of analysis. Structure-soil-structure interaction can be handled by this method for several structures in the plane of analysis.

Although usually applied to the case of vertically propagating wave fields only, the model shown in Fig. 6(b) may be used to study the effects of travelling waves. However, in such analyses the usual transmitting lateral boundaries cannot be used and the lateral extent of the finite element model must be kept large; even then, special methods must be used to evaluate the free-field motions to be specified on the lateral boundaries, Udaka (1975). Special problems occur when the boundary on which the motion is specified is not horizontal. For such cases the specification of appropriate boundary motions becomes very difficult without a prior scattering analysis to determine the motion of the boundary. Since this would involve an additional effort of the same order of magnitude, it is therefore better

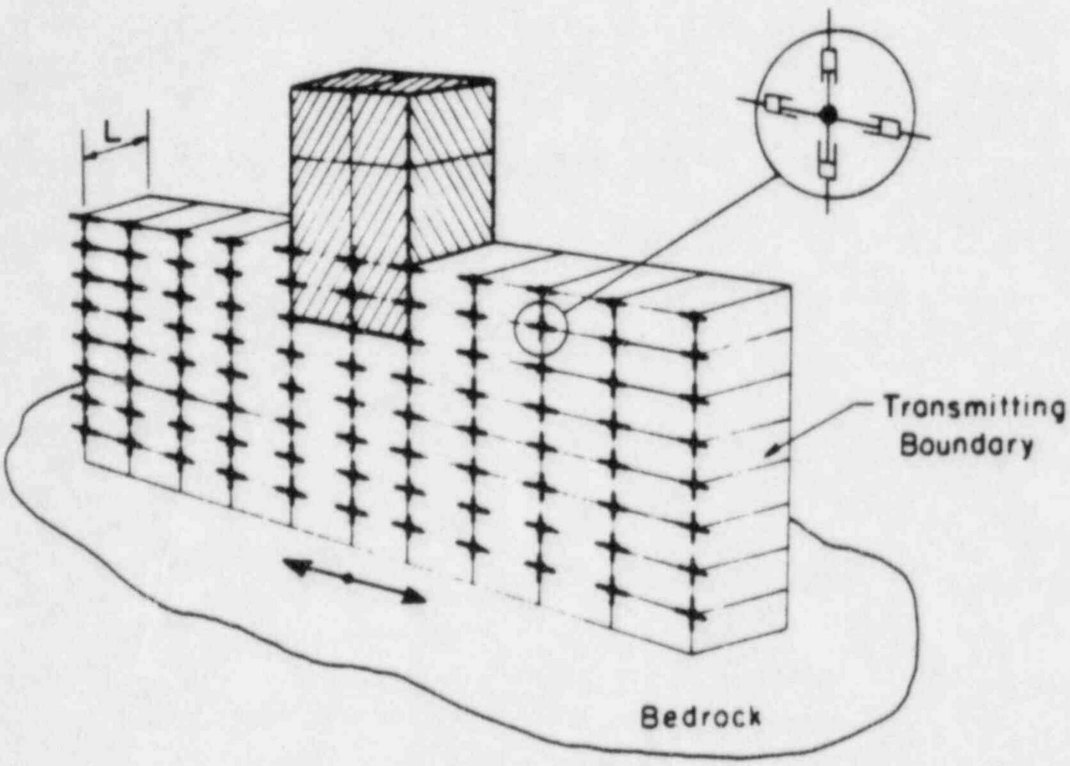


Fig. 7 SCHEMATIC VIEW OF A SIMPLIFIED 3-D MODEL

to keep the control boundary horizontal even if it requires a larger model.

The pseudo-interaction methods can, in principle, handle general nonlinear soil properties for the region inside the lateral transmitting boundaries. However, equivalent linear analysis is more common in practice since the perfect transmitting boundary can only be defined for this case.

True Interaction Methods

When a pseudo-interaction method is conducted in such a way, either through the use of an extensive mesh or by the use of boundaries which represent the radiation of waves into the half-space, it may be considered a true interaction method. In this sense the methods discussed in connection with Figs. 6 and 7 might be classified as true interaction methods when lateral boundaries are used to absorb the surface waves and the control boundary is kept sufficiently deep.

The scheme illustrated by Fig. 2 (Part I) has been used by Gomez-Masso (1978) and Gomez-Masso et al. (1979) to solve a number of soil-structure interaction problems involving transient vertically propagating waves and Rayleigh waves in soil and rock sites. A typical model for the interaction part of the analysis is shown in Fig. 8. The model was equipped with transmitting boundaries as indicated in the figure and also viscous boundaries as indicated in Fig. 7 to account for three-dimensional effects. The site response problem, Fig. 2(b) (Part I), was solved using a much deeper model of varying depth according to the wave length of the Rayleigh waves considered. Typical mode shapes for Rayleigh waves in a rock site are shown in Fig. 9. The results obtained showed that the motions of the structure were independent of whether or not a viscous boundary was used at the base of the model, indicating that most of the interaction displacements consist of surface waves which dissipate through the lateral boundaries rather than

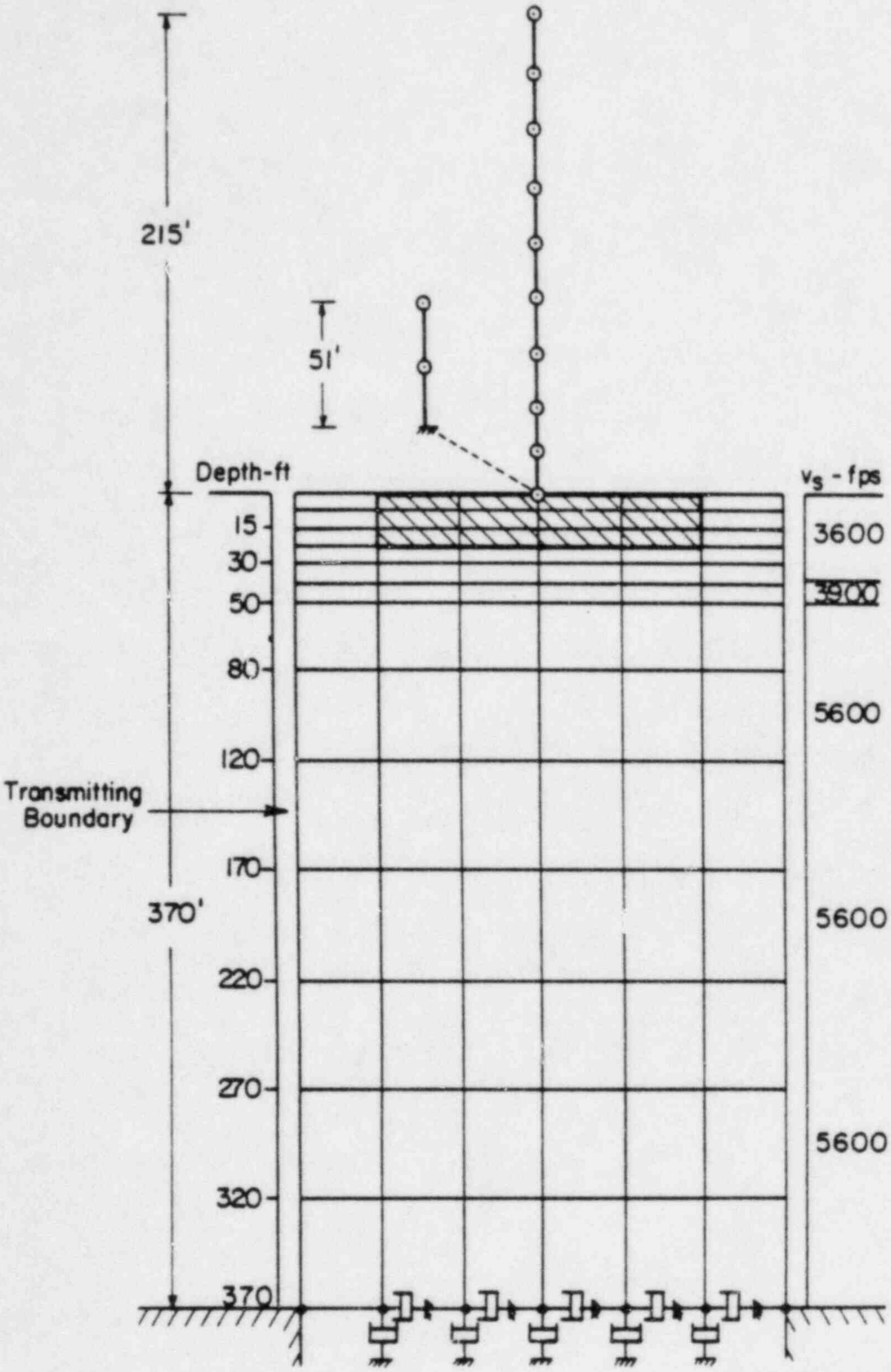


Fig. 8 FINITE ELEMENT MODEL

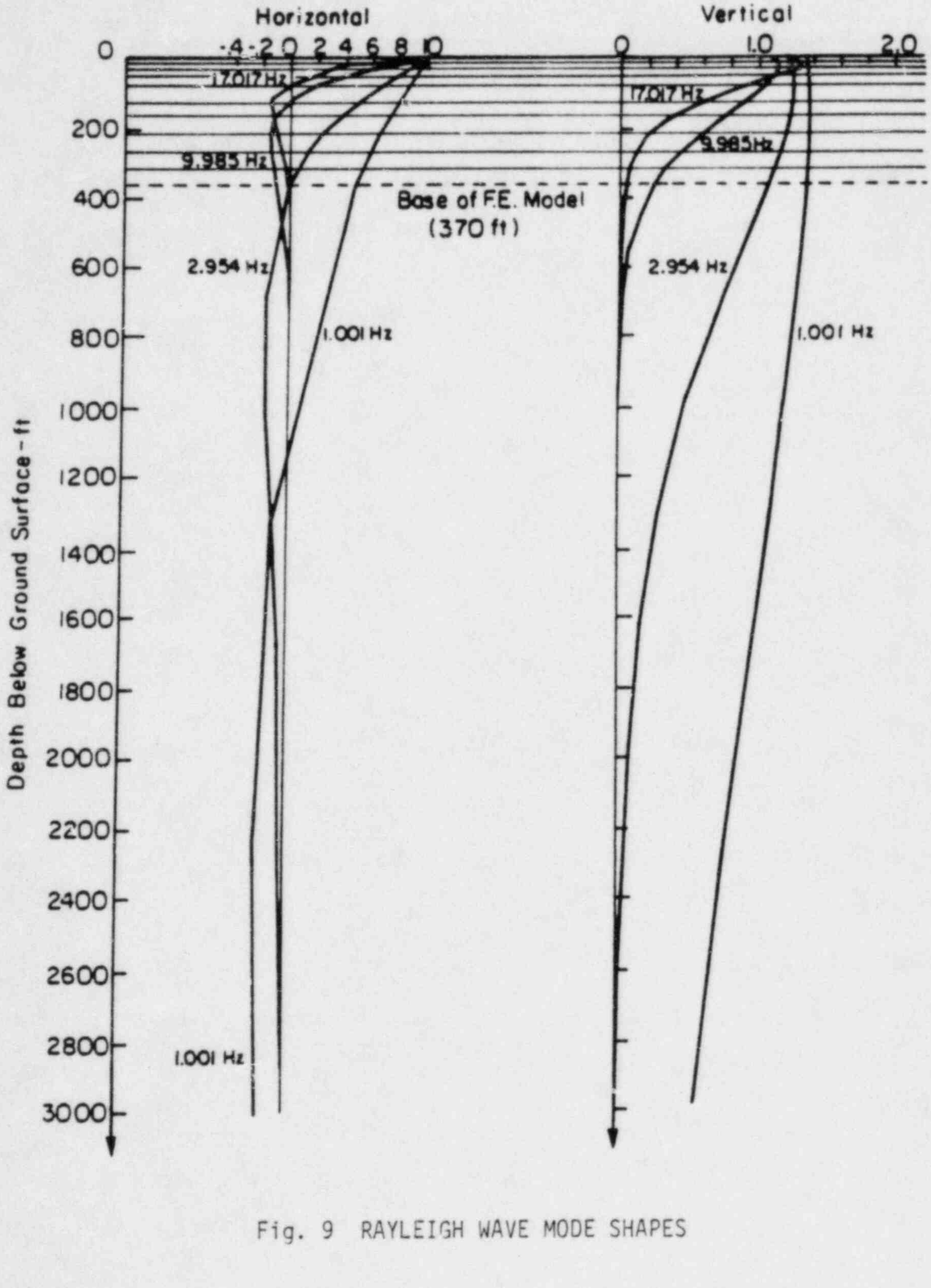


Fig. 9 RAYLEIGH WAVE MODE SHAPES

the base of the model. Control computations using the pseudo-interaction procedure illustrated by Fig. 6 and vertically propagating waves indicated, as expected, identical results for the two methods. A more complete presentation of the method and the results obtained is provided in Part V of this report.

Because of the superposition scheme involved, the above method is applicable only to linear or equivalent linear problems. However, as suggested by Joyner (1975), Herrera and Bielak (1977) and Toki and Sato (1977) nonlinear analysis of the region surrounding the structure might be possible as long as the half-space surrounding this region is linear. Only the writers' interpretation of the Joyner scheme will be given here. The method has its origin in the Joyner-Chen model shown in Fig. 7 (Part II).

Consider first the case of vertically propagating waves and a model of the type shown in Fig. 10(a). In this model the seismic excitation is applied in the form of a shear stress, $\tau = \delta V_s \dot{y}_H(t)$, at the base of the model, where $\dot{y}_H(t)$ is the horizontal outcrop control motion and the radiation damping is accounted for by a Lysmer-Kuhlemeyer viscous boundary. A similar scheme is used for the vertical control motion. According to the theory presented in Part II (Eqs. (7) - (10)), this model will for all practical purposes account for the existence of a uniform elastic half-space below the boundary. Also, since linearity and horizontal layering need not be assumed for the region above the boundary and the calculations can be carried out in terms of total displacements, the method appears to be an excellent candidate for nonlinear analysis of interaction and scattering problems involving vertically propagating waves. However, the method is not exact since it assumes normal incidence of the waves reflected from the structure. As shown by Lysmer and Kuhlemeyer (1969), this may not be too critical since the waves

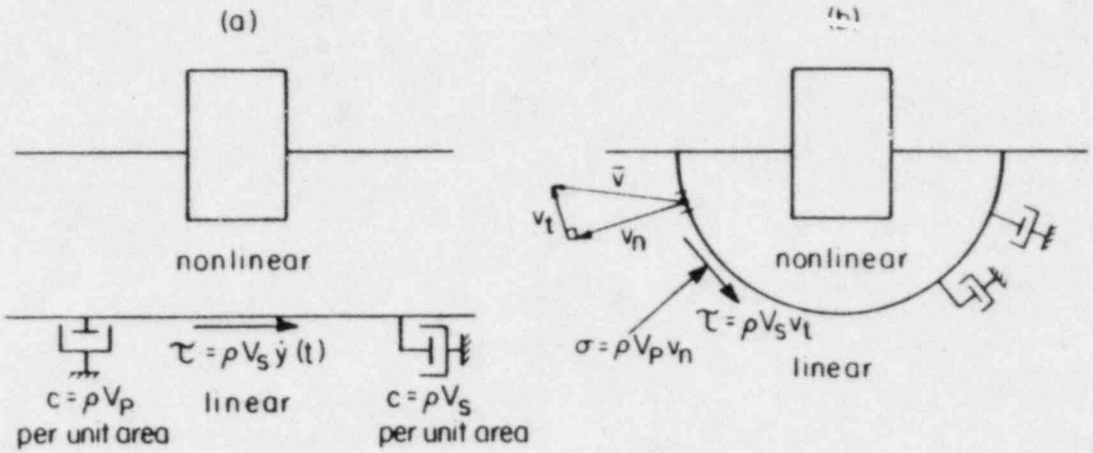


Fig. 10 NONLINEAR INTERACTION MODELS

radiating from the structure will impinge at near normal incidence in the region near the structure.

Joyner (1975) has also suggested an approximate method for analysis of irregular regions surrounded by a uniform elastic half-space. Consider the model shown in Fig. 10(b), and let it be assumed that the scattering problem involving the half-space in the absence of the structure and the nonlinear region has been solved, resulting in the velocities $\overline{v}$ along the free boundary. If it is now assumed for each segment of the boundary that this motion is generated by the normal reflections of an S-wave and a P-wave, Eq. (10) (Part II) would apply and the nonlinear region could be analyzed as indicated in Fig. 10(b). The method is approximate since the assumption of normal incidence of both the seismic waves and the waves reflected from the structure is known to be in error. However, some realistic results have been obtained by the method, Joyner (1975), and similar methods, Toki and Sato (1977), Ayala and Aranda (1977).

COMPLETE VS. SUBSTRUCTURE METHODS

The major advantage of complete methods over the substructure methods is that the analysis is performed at the actual stress level for all soil elements and that it is therefore possible with these methods to consider the spatial effects of nonlinearity, say by the equivalent linear method.

A disadvantage of the complete methods is that more computer time and storage are required to perform the analysis of the larger models involved. However, as pointed out by Wight (1977), experience has shown that while the complete methods are computer-intensive, the substructure methods, even the approximate rigid base methods, are manpower-intensive and the total dollar cost of analysis will be about the same for the two methods.

Wight (1977) has also pointed out that "the judgment required for each approach varied appreciably. The finite element approach [complete analysis similar to Fig. 6] is relatively straightforward in implementation, and very little is required of the analyst. On the other hand, the lumped mass approach requires that the analyst quantify the effects of layers on the springs and dashpots, determine the appropriate input excitation, and establish a radiation damping coefficient."

The final test, of course, is the ability of the different methods to predict what actually happens in the field during an earthquake. This requires case studies, too few of which are performed, although a notable example is the study of the motions of the Humboldt Bay Nuclear Power Plant, see Part VI. This case, which was studied by the pseudo-interaction procedure shown in Fig. 6, showed that realistic motions can indeed be computed by that procedure. It is clearly desirable that other procedures be checked against this benchmark case.

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Part IV

SOIL-STRUCTURE INTERACTION ANALYSES BY FINITE ELEMENTS - STATE OF THE ART

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Abstract

In this paper the authors have attempted to summarize the current capability for evaluating soil-structure interaction effects during earthquakes using finite element procedures. A concise summary of methods available, together with their capabilities and relative costs is presented. It is suggested that finite element procedures provide a powerful tool for use in the design of nuclear plants, especially for embedded structures, and their applicability in this respect is illustrated by comparing computed results with those recorded in a nuclear plant during a strong motion earthquake. It is concluded that when the methods are used in conjunction with good engineering judgment and with full recognition of their limitations, they provide evaluations of response with a level of accuracy entirely adequate for engineering design.

Introduction

Analyses of soil-structure interaction effects during earthquakes for nuclear power plant structures are usually made by one of two methods--either by means of a complete interaction analysis involving consideration of the variation of motions in the structure and the adjacent soil, or by an inertial analysis in which the motions in the adjacent soil are assumed to be the same at all points above foundation depth. For surface structures, the distribution of free field motions with depth in the underlying soils has no influence on the structural response and thus, provided the analyses are made in accordance with good practice good results may be obtained using either method of approach. For embedded structures, however, consideration of the variation of ground motions with depth is essential if adequate evaluations of soil and structural response are to be obtained without undue conservatism.

At the present time, a variety of methods of analysis including these effects have been developed using finite element techniques. Not only do finite element methods provide an excellent tool by means of which the significant aspects of an idealized complete interaction analysis for embedded structures may be considered with a high degree of accuracy on theoretical grounds, but recent observations of the response of the Humboldt Bay Nuclear Power Station to strong shaking induced by the Ferndale, California earthquake of June, 1975 show that this method of approach provides response evaluations which are in good accord with those observed under field conditions.

This does not mean that all finite element analyses of soil-structure interaction provide adequate evaluations of response. Like all analyses, they can be performed with different degrees of approximation or sophistication. The basic requirements for a good analytical procedure may be summarized as follows:

1. The analysis should consider the variation of soil characteristics with depth for the soil profile existing at the proposed site.
2. The analysis should consider the non-linear and energy-absorbing characteristics of the different soil strata comprising the soil deposit.
3. For embedded structures, the analysis should consider the variation of ground motions with depth; this variation should be consistent with established theories of engineering mechanics for evaluating the response of the continuous soil-structure system and be in reasonable accord with available knowledge concerning

the variation of ground motion characteristics with depth during earthquakes.

4. The analysis should be capable of taking into account the three-dimensional nature of the problem.

5. The analysis should be capable of considering the effects of adjacent structures on each other.

It is not always necessary to meet all of these requirements--for example, where a simple structure is involved, accurate evaluations of the motions at the base of a structure can be obtained using a two-dimensional analytical model--but, in general, all of the requirements listed above should be taken into account.

One of the primary arguments against the use of finite element methods of analysis is their high cost. This of course depends on the efficiency of the computer program used but it is true that in the recent past, analyses of this type have been substantially more costly than analyses using the inertial interaction approach in conjunction with half space theories; this limitation now seems to have been overcome through the development of more efficient computer techniques.

It is the purpose of this paper to review the current level of accomplishment which may be achieved using finite element techniques for the performance of complete interaction analyses. It is hoped to show that they provide an efficient procedure for evaluating translational, vertical and rocking modes of excitation in accordance with all the desirable requirements listed above without incurring excessive costs for design and analysis.

Two-dimensional Finite Element Analyses

A complete analysis of the soil-structure interaction problem would involve a determination of the response of a structure when it is subjected to earthquake ground motions which vary from point to point in the soil and rock around and underlying the structure and travel in some unknown way across the base of the structure (see Fig. 1(a)).

This admittedly complex problem is usually idealized for purpose of analysis so that motions in the vicinity of the structure are considered to be due to vertical propagation of body waves from underlying stiffer formations (see Fig. 1(a)). This is clearly an approximation and is justified on the grounds that (1) it is believed to be sufficiently accurate for engineering purposes; (2) there is a growing body of evidence to show that variations of motions with depth computed in this way are in reasonable accord with field observations of ground response during actual earthquakes; and (3) for computations of translational, vertical and rocking modes of structural response, the assumption that all ground motions are due to vertically propagating shear waves is usually conservative (Luco, 1977).

On the basis of the above approximation, finite element analysis procedures satisfying the first three of the basic requirements for an adequate procedure have been available for a number of years (Seed et al, 1974 and 1975). The basic problem is illustrated in Fig. 1(b). A control motion is defined at some point in the free-field and it is desired to determine the corresponding motions at the base of a structure and within the structure.

The general method of analysis is shown schematically in Fig. 2. The specified control motion is deconvolved in a free-field analysis of the soil deposit only, to determine the

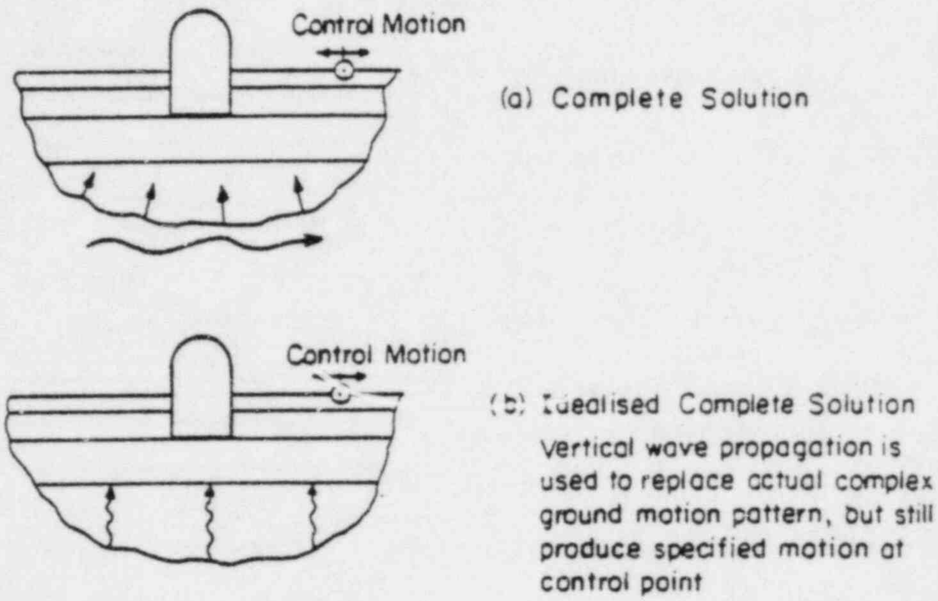


Fig. 1(a) COMPLETE AND IDEALIZED COMPLETE ANALYSES OF SOIL-STRUCTURE INTERACTION

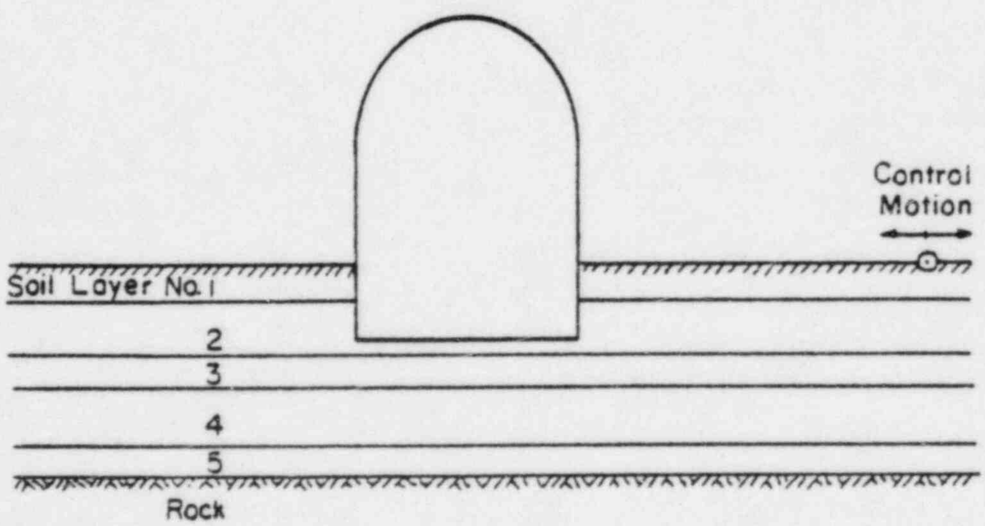


Fig. 1(b) SOIL-STRUCTURE INTERACTION PROBLEM

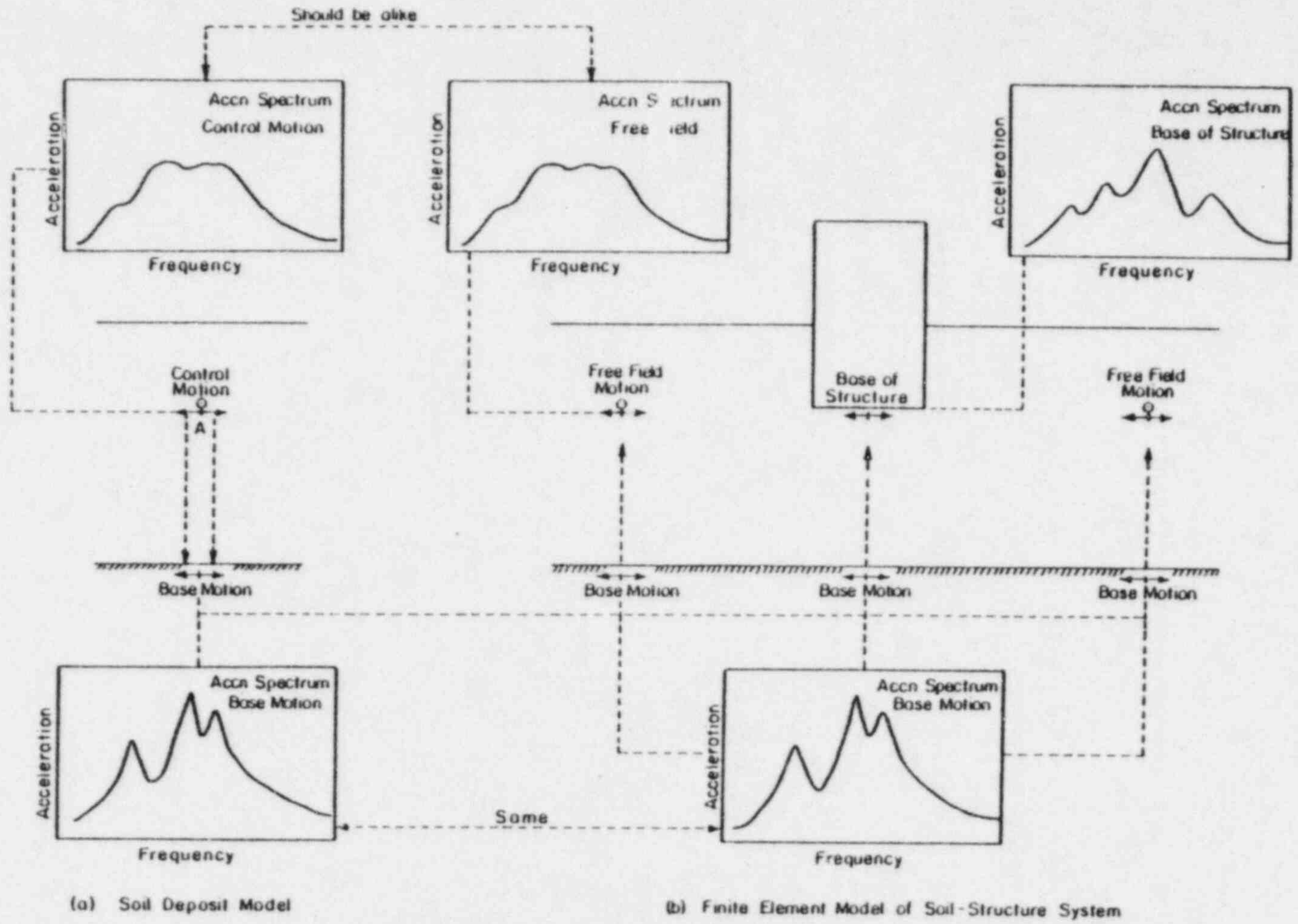


Fig. 2 SCHEMATIC REPRESENTATION OF SOIL-STRUCTURE INTERACTION ANALYSIS USING FINITE ELEMENT

motions at some depth, such as a soil-rock interface, which would be required to produce the required control motions at the ground surface. One-dimensional amplification theory can be used for this purpose (Schnabel et al, 1972a and 1972b). Then the motion computed at this depth is used as input to a finite element model of the soil-structure system, and the response computed at points of special interest. Another method of approach is to compute transfer functions relating the motions and forces at desired points in the soil or structure to the control motion applied at a point on the surface of the soil well away from the structure (Kausel and Roesset, 1974). In either case, the analysis should be performed iteratively to allow for the strain dependent nature of the nonlinear soil characteristics (Seed and Idriss, 1969; Schnabel et al, 1972). In each iteration the analysis is linear but the soil properties are adjusted from iteration to iteration until the computed strains are compatible with the soil properties used in the analysis.

Using this approach, different soil properties may be assigned to every element, if desired, so that there is no difficulty in considering the variation of soil characteristics with depth, while the iteration procedure permits consideration of the non-linear stress-strain and damping characteristics of the soils. In order to control the damping ratios to the desired values it has been found desirable to use the complex response method of analysis and the computer program LUSH (Lysmer et al, 1974) has been widely used for analyses of this type. Other methods of analysis (modal analyses, methods using Rayleigh damping, etc.) do not always provide the necessary freedom to adjust damping ratios to specified values in each element.

In the LUSH-approach the soil-structure system is represented as a two-dimensional finite element model. It has been shown that such two-dimensional representations of the soil-structure system can provide good evaluations of the response at the base of the structure (Berger et al, 1975) but, not necessarily within the structure. Thus in order to obtain adequate evaluations of response using this approach it has been necessary to compute the overall response of structures which cannot be considered plane in two stages: (1) a two-dimensional analysis of the soil-structure system to determine the motions in the portion of the structure below the ground surface and (2) a three-dimensional analysis of the structure to determine its response to the base motions computed in stage (1).

Clearly the main limitations of this approach are that it fails to satisfy fully the last two requirements listed on page 2, i.e. full consideration of the three-dimensional nature of the problem and the ability to satisfactorily evaluate the effects of adjacent structures. In addition, computer costs for extensive systems have been very considerable. Accordingly, the following modifications have been made to remedy these deficiencies, leading to a more versatile and efficient analysis procedure.

Three-dimensional Effects

In recognition of the need to consider the three-dimensional nature of the soil-structure system, finite element analysis procedures for axisymmetric structures have been developed by several investigators (Ghosh and Wilson, 1969; Agrawal et al, 1973; Kausel and Roesset, 1974; Berger et al, 1975). The analytical problem involved is illustrated schematically in Fig. 3(a). In making such analyses it is necessary to ensure that the boundaries of the finite element model are sufficiently far removed from the structure that the

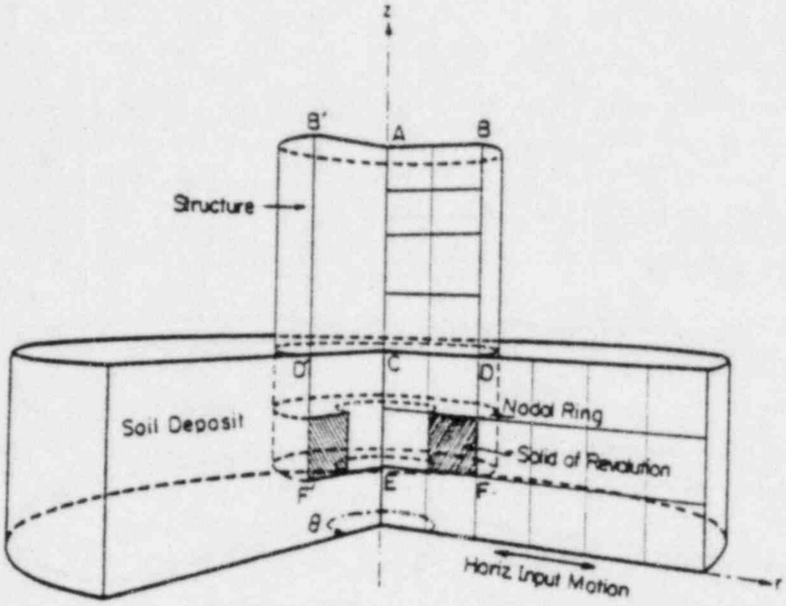


Fig. 3(a) SCHEMATIC VIEW OF AN AXISYMMETRIC MODEL

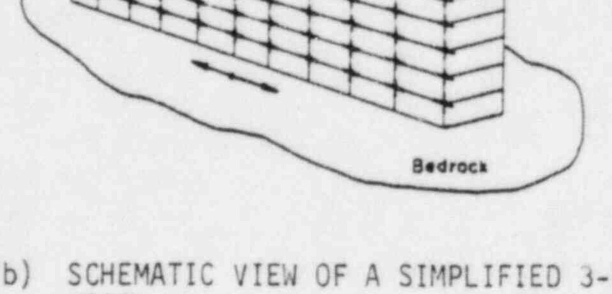


Fig. 3(b) SCHEMATIC VIEW OF A SIMPLIFIED 3-DIMENSIONAL MODEL

full effects of radiation damping are correctly represented (Berger et al, 1975). Alternatively, the analytical model may be provided with transmitting boundaries which absorb any wave effects emanating from the structure and thus simulate the effects of an extensive soil deposit (Kausel and Roesset, 1974).

While some three-dimensional effects may be considered in this way, solutions cannot be obtained once the axi-symmetric nature of the structure ceases to exist; this might occur, for example, in the simple case of two adjacent structures, each axi-symmetric but the combined system no longer having axi-symmetric characteristics. In a nuclear plant involving many different types of structures, axi-symmetric situations are the exception rather than the rule and other methods of approach are required. Never-the-less the axi-symmetric solutions provide an essential standard by means of which the validity of other approaches may be judged and, in this respect, represent a vital aspect of the development of analytical techniques.

An alternative method of approach for including the three-dimensional features of a soil-structure system is that suggested by Hwang et al (1975). The method involves the use of viscous boundaries along the planar surfaces of a slice of soil on which one or more structures are located. This idealization of the soil-structure system is illustrated schematically in Fig. 3b.

If the soil slice is made sufficiently long, then wave energy radiating along the axis of the slice will be absorbed by material damping while energy radiating in a direction normal to the axis of the slice is absorbed by the viscous boundaries. It has been shown that this form of analytical model for a single structure provides essentially the same response values as an analysis for an axi-symmetric system such as that shown in Fig. 3(a) (Hwang et al, 1975). A typical example is provided by the comparison, shown in Fig. 4, of the responses determined by analyses using the two different models. It may be seen that there are only minor differences in the computed spectra for the motions at the base of the structure and at the ground level within the structure. Since analyses using the model shown in Fig. 3(b) have the advantage of being considerably faster than axi-symmetric analyses and can also be used for non-axisymmetric situations, while at the same time they provide results of comparable accuracy for problems which can be analyzed by either method, it would seem logical to adopt the faster and more versatile approach provided by the model shown in Fig. 3(b) for design purposes.

Alternatively, motions below ground can be determined by a two-dimensional model with a reasonable degree of accuracy, although such models do not provide good evaluations of above ground response for three-dimensional structures. This is illustrated by the results in Fig. 5, which compare the computed responses at the ground level within an embedded structure and near the top of a structure for analyses performed using an axi-symmetric model of the system using the computer program ALUSH (Berger, 1975) with those computed by a two-dimensional model of the same structure. At and below ground level, the responses are reasonably similar but above ground level, the two-dimensional analysis considerably underestimates the response.

This suggests the possibility that two-dimensional analyses might be used to compute the horizontal and rocking motions within a structure for points below the ground surface

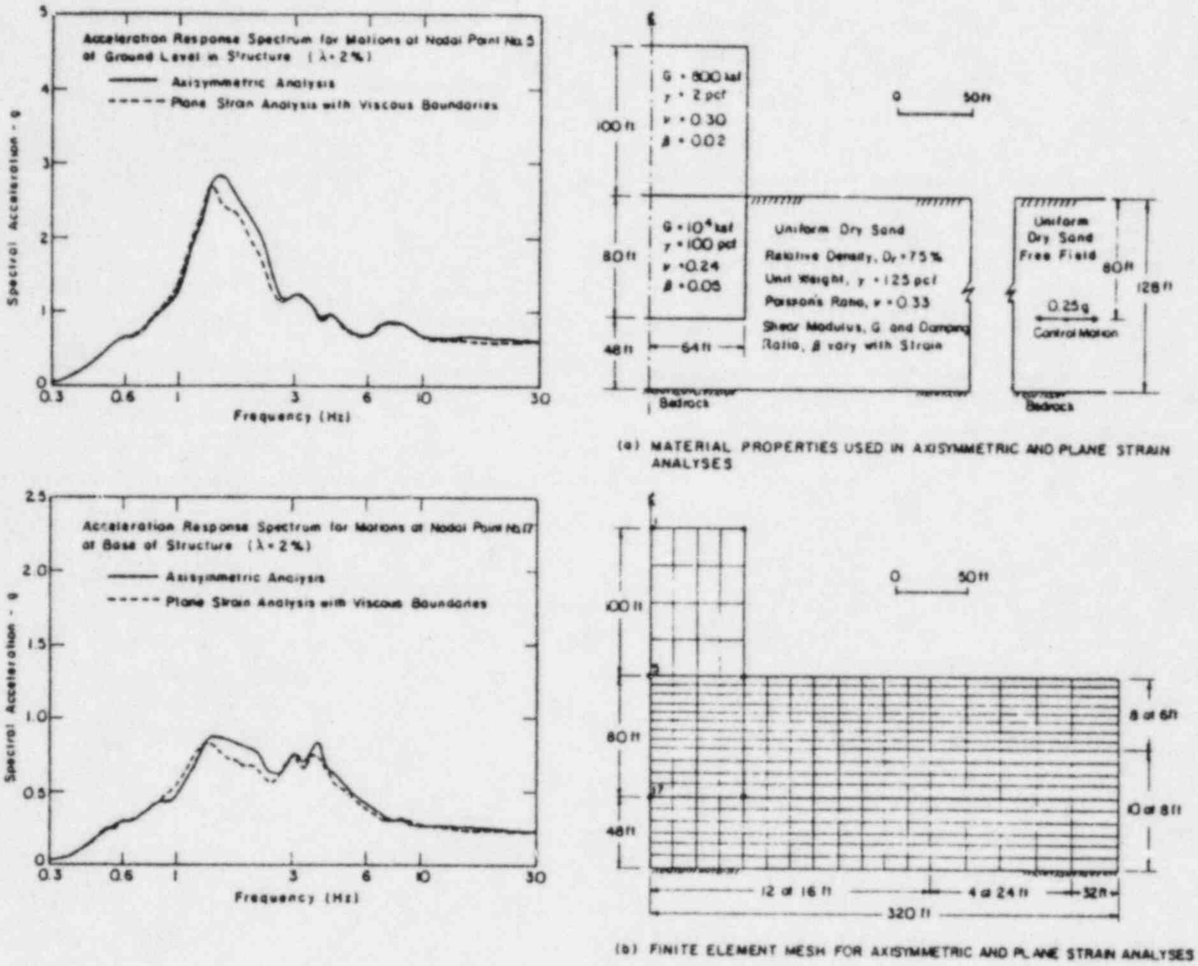


Fig. 4 COMPARISON OF COMPUTED MOTION CHARACTERISTICS FOR AXISYMMETRIC AND PLANE STRAIN ANALYSES USING VISCOUS BOUNDARIES

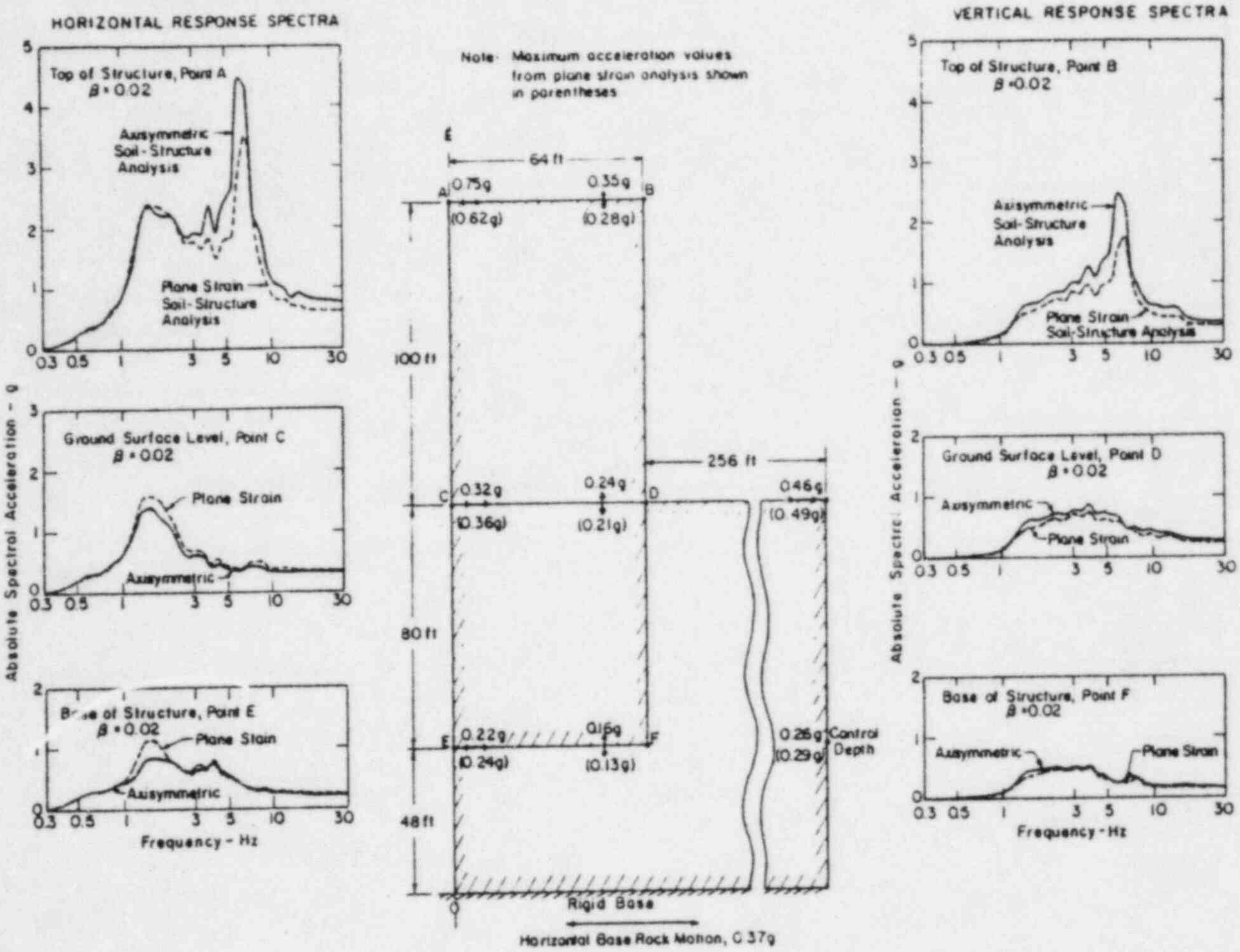


Fig. 5 COMPARISON OF HORIZONTAL AND VERTICAL RESPONSE SPECTRA FROM AXI-SYMMETRIC AND PLANE STRAIN ANALYSES

with a sufficient degree of accuracy. Then the computed motions at the ground surface can be used as base excitation for a three-dimensional model of that part of the structure extending above ground level. A comparison of results obtained using this type of approach with those obtained from a three-dimensional axi-symmetric analysis is shown in Fig. 6. It may be seen that this approach leads to a significant improvement in evaluations of structural response over that obtained using a simple two-dimensional model.

The dimensions of the finite element mesh illustrated in Fig. 3(b) and Fig. 4 can be drastically reduced if the model is provided with transmitting boundaries at the ends, as shown in Fig. 7. The analytical model shown in Fig. 7 has the added advantage that it can be used to compute the combined response of multiple structures having the same width as the soil slice. This might involve, for example, two rectangular structures of approximately equal widths standing side by side. Such a system could not be analyzed using an axisymmetric analysis formulation but the three-dimensional effects for adjacent structures can readily be included using the model formulation shown in Fig. 7(b).

The benefits of using transmitting boundaries in this way are illustrated by the comparative results shown in Fig. 8 for analyses made using an extensive mesh and using a limited mesh with transmitting boundaries close to the structure. Differences in the computed results are minor.

Thus it would appear that where three-dimensional axi-symmetric analyses are essential, computer programs such as ALUSH (Berger, 1975) may be used, but for other analyses, involving models of the type shown in Fig. 7, more efficient and versatile programs can be developed, such as the program FLUSH (Lysmer et al, 1975). This latter program incorporates the following features:

1. Viscous boundaries are provided to represent three-dimensional effects.
2. Transmitting boundaries are provided to minimize the required number of finite elements.
3. An out-of-core equation solver is available, making it possible to solve large problems on a relatively small computer.
4. Linear bending elements are provided for better modelling of basement walls and structural frames.
5. A capability to perform the deconvolution of near surface motions within the program using the same finite element mesh as that used for the soil-structure interaction analysis.

Since this program is considerably faster than the original LUSH program it has been called FLUSH (for <u>F</u>ast <u>LUSH</u>). The increase in speed can be judged from the approximate relative computer times for different analytical approaches now available listed in Table 1. Furthermore for particularly deep soil profiles, a transmitting boundary may be incorporated along the base, if required, to provide radiation damping effects equivalent to a half-space. Experience shows that this is rarely necessary due to the fact that for usual sites the soil stiffness invariably increases with depth causing wave reflections at layer interfaces.

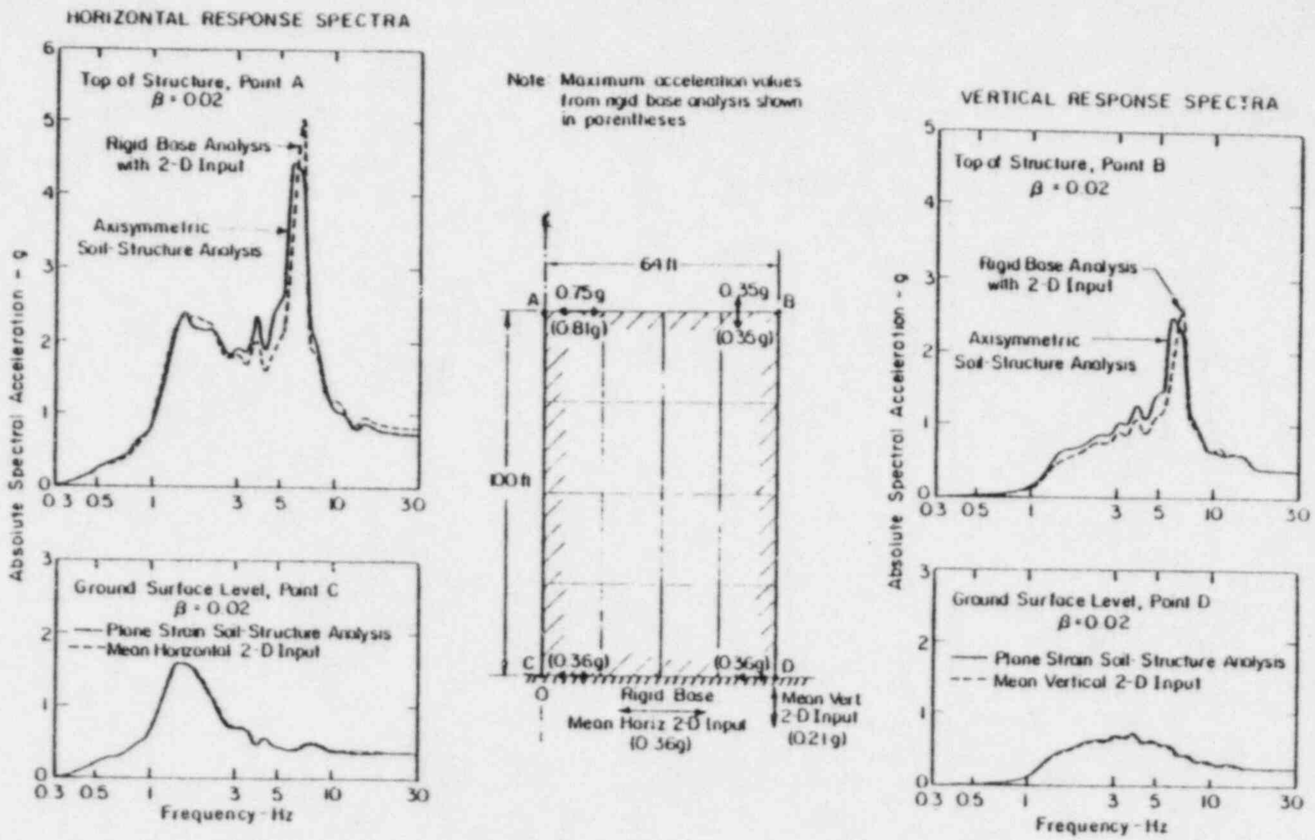


Fig. 6 COMPARISON OF HORIZONTAL AND VERTICAL RESPONSE SPECTRA FOR AXISYMMETRIC SUPERSTRUCTURE CUT AT GROUND SURFACE

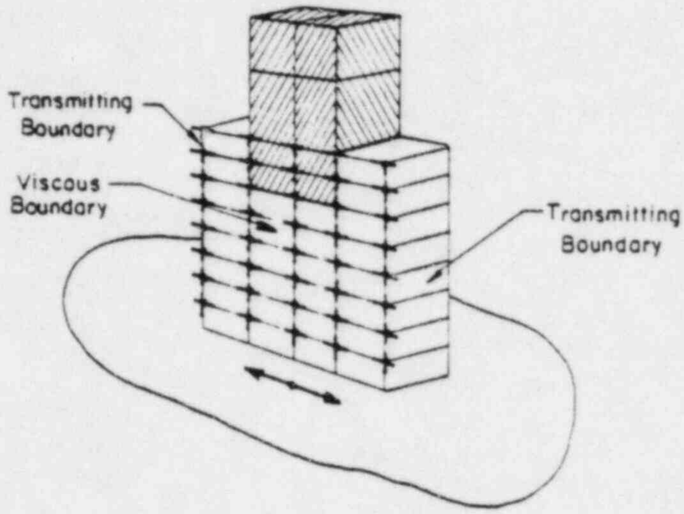


Fig. 7(a) SCHEMATIC VIEW OF A SIMPLIFIED 3-DIMENSIONAL MODEL WITH TRANSMITTING BOUNDARIES

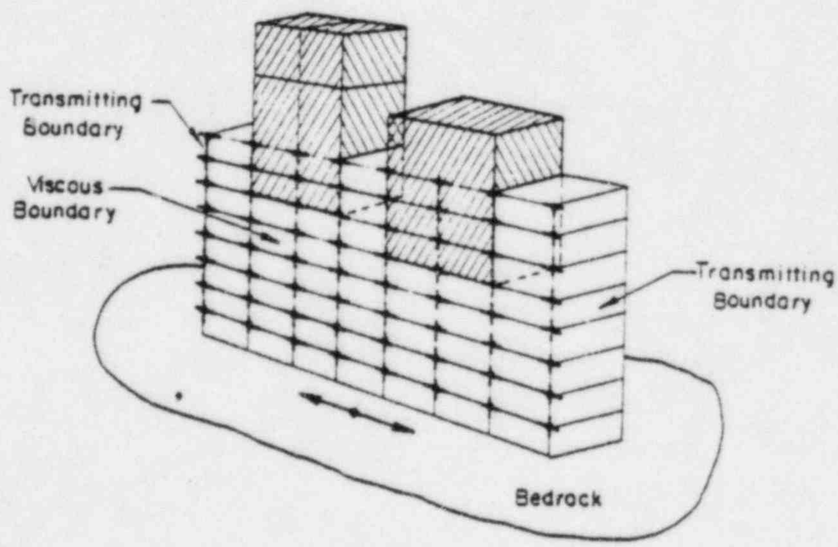


Fig. 7(b) SCHEMATIC VIEW OF A SIMPLIFIED 3-DIMENSIONAL MODEL WITH MULTIPLE STRUCTURES

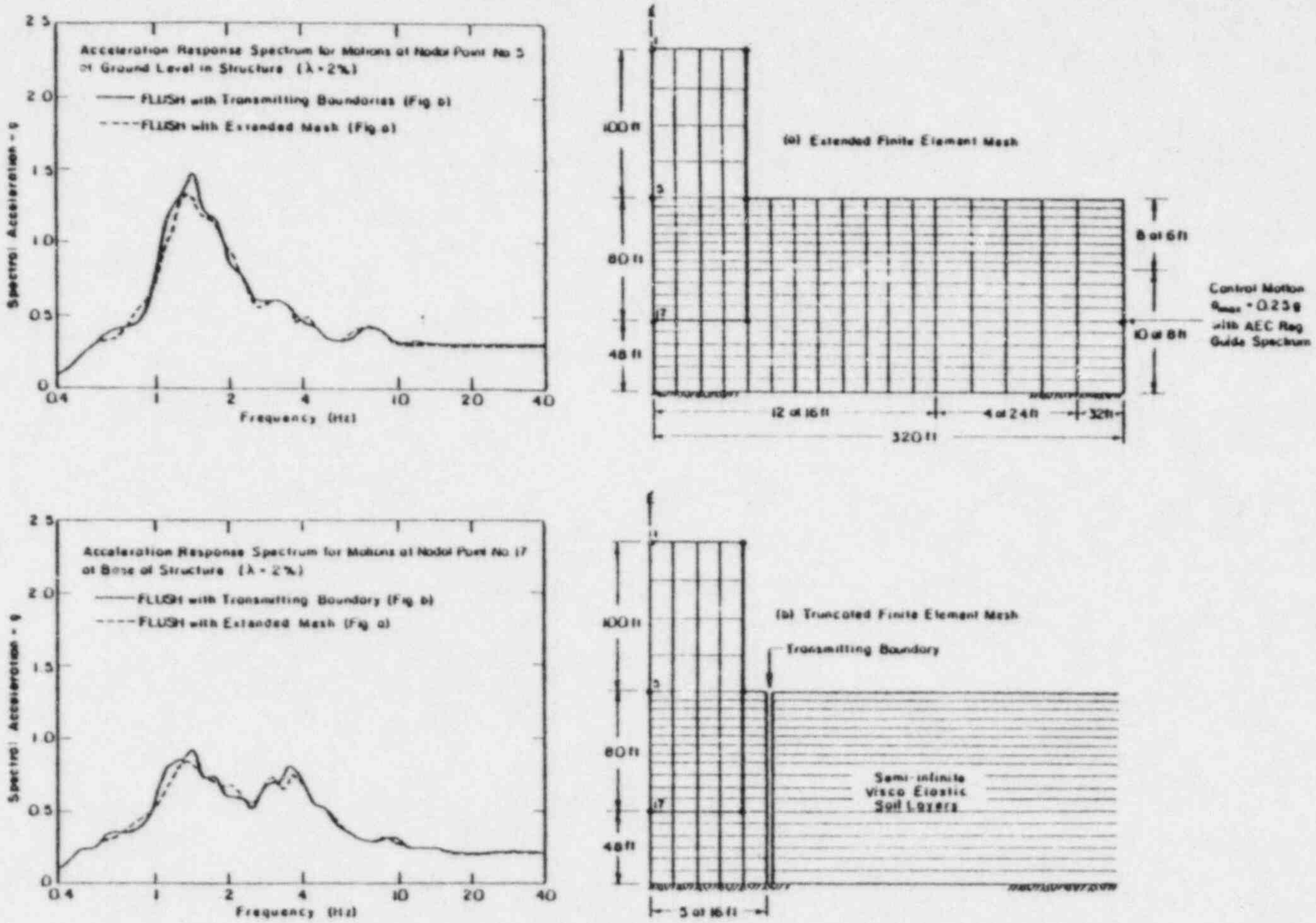


Fig. 8 COMPARISON OF COMPUTED MOTION CHARACTERISTICS FOR ANALYSES WITH EXTENSIVE MESH AND TRANSMITTING BOUNDARIES

Table 1

Type of Analysis	Computer Program	Relative Computer Time
Axi-symmetric analysis with extensive mesh (deterministic)	ALUSH	1.00
Axi-symmetric analysis with transmitting boundaries (deterministic)	(after Kausel)	0.50
Plane strain analysis with extensive mesh (deterministic)	LUSH	0.35
Plane strain analysis with transmitting boundaries (deterministic)	FLUSH	0.19
Three-dimensional analysis with viscous and transmitting boundaries (deterministic)	FLUSH	0.20
Three-dimensional analysis with viscous and transmitting boundaries and deconvolution through finite element mesh (deterministic)	FLUSH	0.10
Probabilistic three-dimensional analysis with viscous and transmitting boundaries	PLUSH	0.05

Effects of Building-Building Interaction

The necessity of considering the interaction between adjacent structures in evaluations of structural response has been recognized for some years. In one case examined, using the simplified three-dimensional model shown in Fig. 7(b), it was found that the presence of adjacent structures increased the maximum response of a containment building by about 60 percent, as shown in Fig. 9; this is a substantial effect. However it is considerably less than that which would be indicated by a plane-strain two-dimensional analysis with no viscous boundaries of the same soil-structure system (see Fig. 9), thereby confirming the need for consideration of the effects of adjacent structures but only by procedures which can give adequate consideration to the three-dimensional nature of the problem.

Probabilistic Analysis for Single Control Motion Specification

In the design of nuclear power plants, the control motion for the seismic soil-structure interaction analysis, often termed the Design Basis Earthquake, is defined in the form of a response spectrum of specified shape. The designer can then develop, for analysis purposes, any reasonable time-history of motions which has a spectrum falling above that specified for the control motion. Analyses performed using such a time-history are deterministic in nature, [illegible]gh parametric studies, particularly involving variations in soil properties, are customarily required for the determination of design motions in the plant itself.

In fact, there are theoretically an infinite number of time-histories of motion whose spectra would have the form prescribed by the control motion and the use of different time histories will lead to some degree of variation in the computed response. To determine the magnitude of these possible effects, a new probabilistic analysis procedure has recently been developed, based on the finite element analysis approach used in the program FLUSH, but incorporating in a probabilistic way, all possible time-histories of motion having a power spectrum corresponding to the response spectrum of the design basis earthquake (Romo et al.

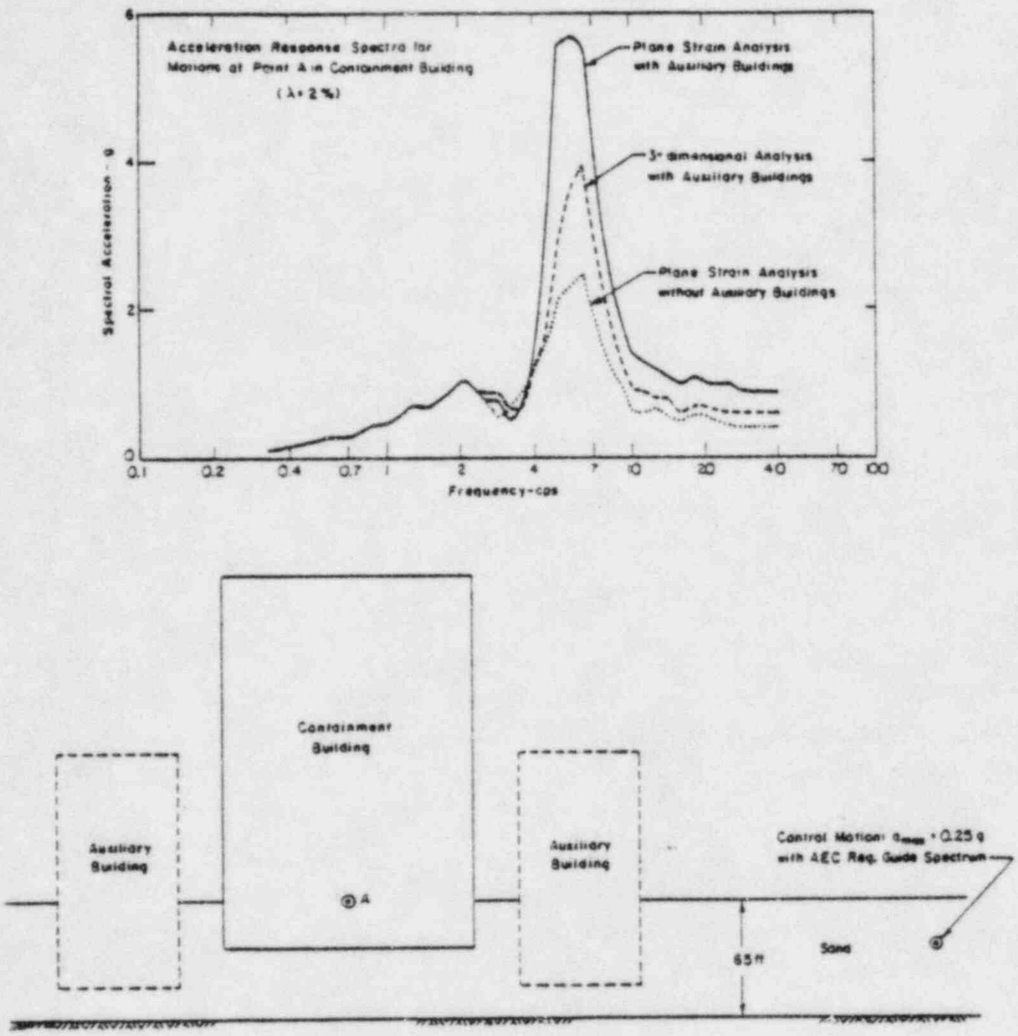


Fig. 9 ANALYSES OF SOIL-STRUCTURE SYSTEM WITH MULTIPLE STRUCTURES

1977). The computer program for accomplishing this has been named PLUSH (Probabilistic LUSH).

The procedure eliminates the need for generation of a time history of motion, it presents response data in probabilistic form in terms of confidence limits, and it permits the analysis to be made still more efficiently than following a deterministic procedure (see Table 1). Typical results of an analysis using this procedure are shown in Fig. 10. It is believed that the capability to present potential variations in structural response in this way aids considerably in selecting criteria for structural design.

Non-vertical Wave Effects

In recent years there has been much discussion concerning the possible need to consider motions other than vertically-incident shear and compression waves in the evaluation of soil-structure interaction effects. Methods of considering non-vertically incident waves for evaluating ground response (Joyner et al, 1976), horizontally propagating motions in soil deposits (Isenberg, 1970; Chen, 1976), horizontally propagating motions at the base of soil deposits (Udaka, 1975) and non-vertically incident SH waves (Luco, 1976) on soil-structure interaction have all been developed and are available to the designer.

It is the general conclusion of these studies that for design purposes, vertically propagating waves provide an adequate evaluation of response for engineering design purposes. For example as shown in Fig. 11, Joyner et al (1976) found that the angle of incidence of wave motions to the rock boundary at the base of the soil deposit shown in the figure had virtually no effect on the computed horizontal spectral ratios between the rock boundary and the ground surface or on the computed response of the deposit; Udaka (1975), see Fig. 12, found only slight differences in horizontal motions in a reactor containment structure whether the analysis was made for travelling base motions or for rigid base motions; and Luco (1976) found that vertically propagating SH waves (for translation and rocking motions) usually provide a somewhat more conservative evaluation of structural response than non-vertical SH-waves.

Thus although finite element analysis techniques are available for investigating these types of motions, the problem seems to be of little practical importance except in so far as it might influence the torsional motions developed on a structure. There does not appear to be any totally rational procedure for evaluating torsional effects at the present time since they are determined by the spatial variations of motions in a soil deposit, and virtually no factual information in the form of recorded data is available on this subject. Thus it is necessary to use simplistic models of torsional interaction effects supplemented by considerable judgment to determine appropriate design criteria for such motions at the present time.

A similar state of affairs exists with respect to evaluations of rocking motions. The most recent development in finite element procedures is the ability to characterize a specified free-field ground motion by a system of Rayleigh waves and, by means of finite element techniques, evaluate soil-structure interaction effects for this type of excitation. Sufficient studies have not been conducted to draw general conclusions but it appears that analyses for this type of motion will show rocking motions to be somewhat greater than those determined by presently available techniques. However even if this result is confirmed,

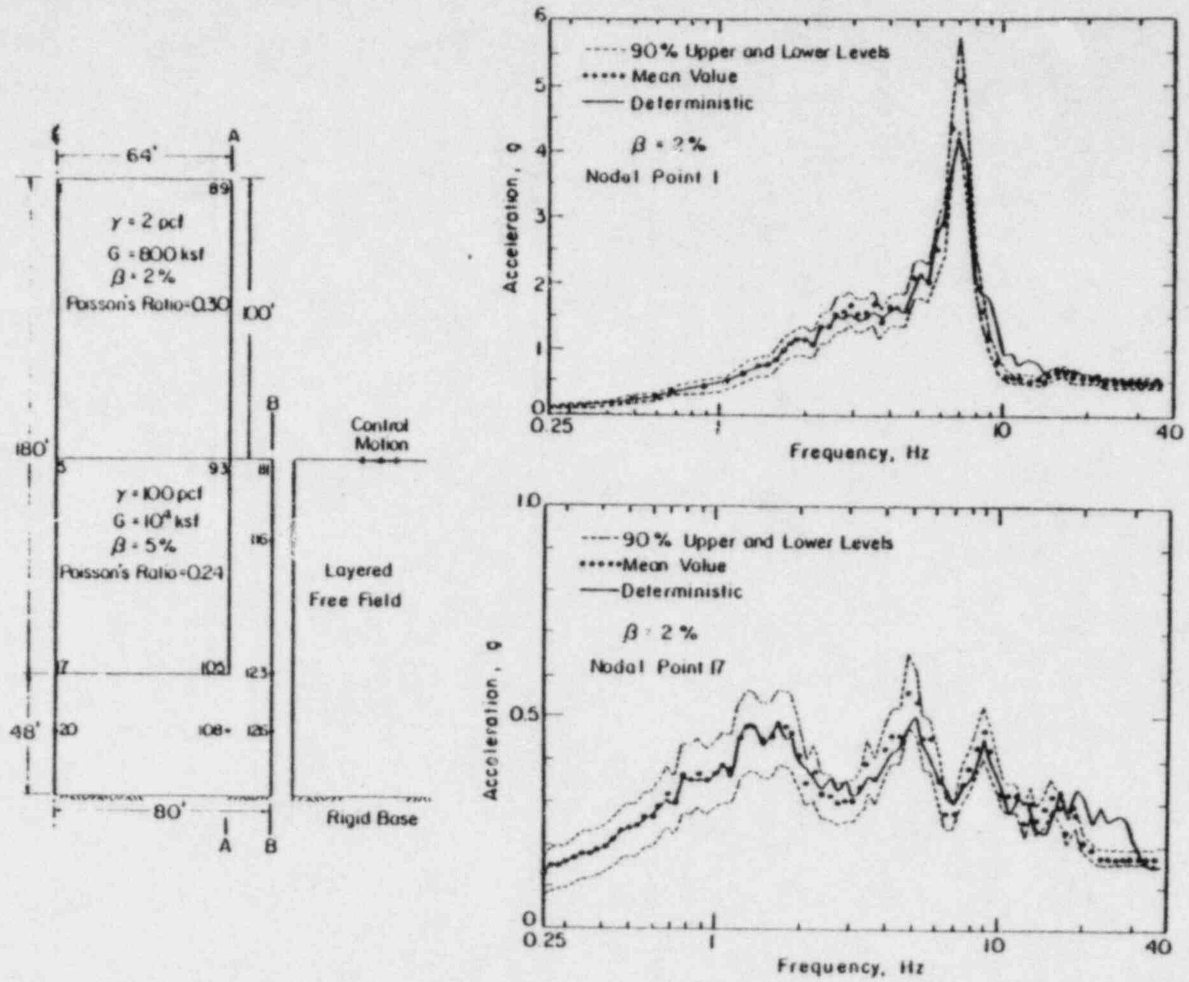
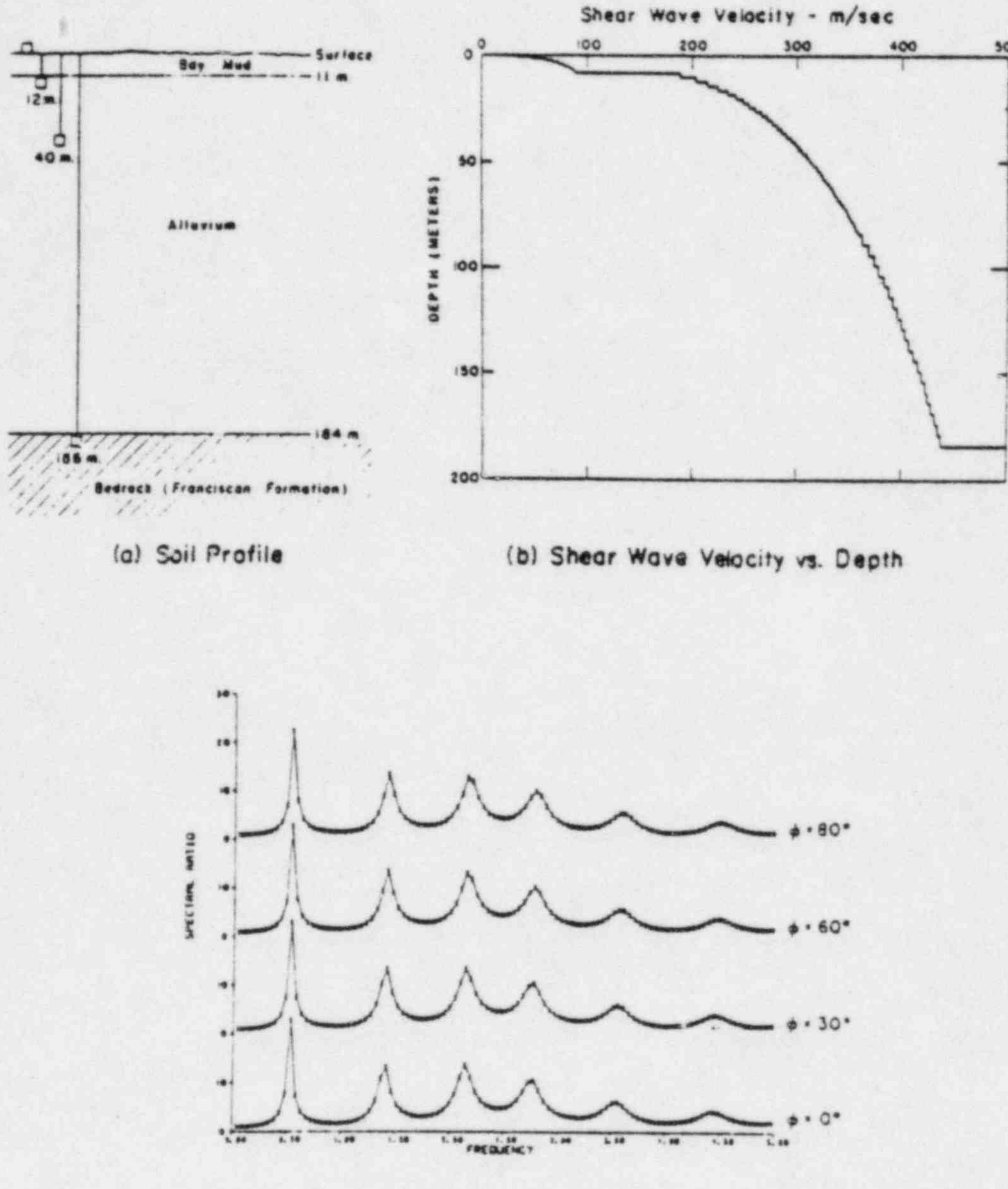


Fig. 10 PROBABILISTIC ANALYSIS OF SOIL-STRUCTURE INTERACTION



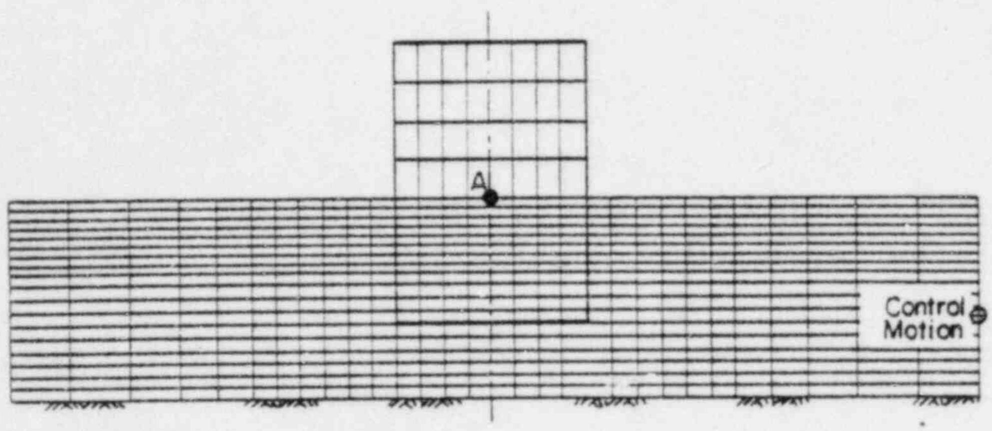
(a) Soil Profile

(b) Shear Wave Velocity vs. Depth

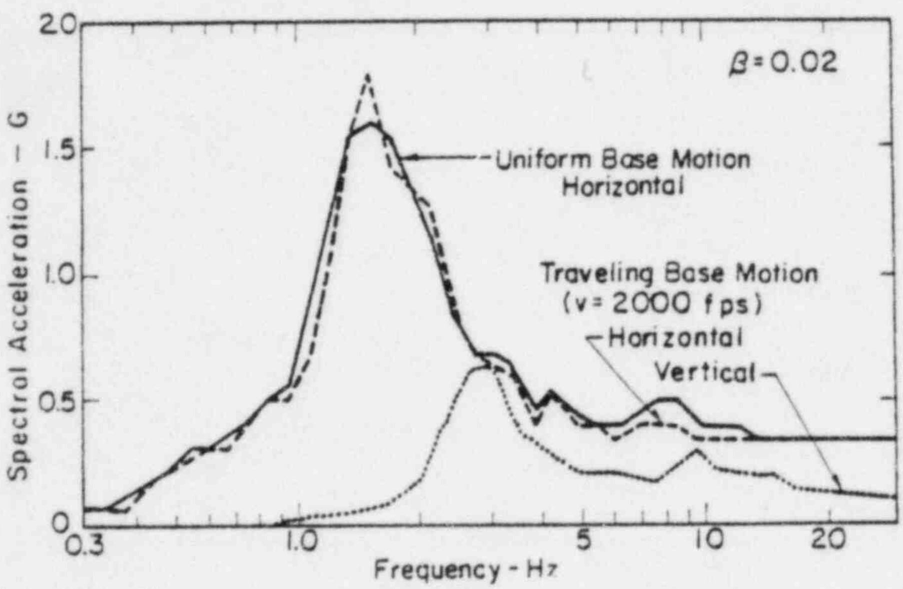
(c) Computed Horizontal Spectral Ratios Between Surface and 186 m

Fig. 11 INFLUENCE OF ANGLE OF SHEAR WAVE INCIDENCE ON COMPUTED SURFACE RESPONSE

(after Joyner et al., 1976)



Finite Element Mesh for Nuclear Power Plant



Acceleration Response Spectra for Motions at Point A

Fig. 12 COMPARISON OF SOIL-STRUCTURE INTERACTION EFFECTS FOR UNIFORM AND TRAVELLING BASE MOTIONS

it will still be necessary to determine the degree to which field motions can be represented by any such systematic pattern of waves and it seems likely that the non-uniformity of ground motions likely to develop in the field will never be fully characterized by systematic wave systems of any type. Thus the results obtained by such wave systems are likely to over-estimate the response of the soil-structure system in some respect; for example a system of horizontally propagating SH waves is likely to over-estimate torsional response effects in structures, while a system of horizontally propagating Rayleigh waves is likely to over-estimate rocking response. Until more detailed information is available on the actual characteristics of ground motions induced by earthquakes, all analytical results must therefore be used with judgment in the final selection of design response values.

Applicability of Finite Element Methods for Evaluation of Soil-Structure Interaction

An excellent opportunity to evaluate the applicability of any theory of soil structure interaction was provided by the motions recorded in the Humboldt Bay Power Station during the Ferndale earthquake of June 7, 1975. The base of the Refuelling Building for this plant is located at a depth of about 80 ft below the ground surface and motions were recorded in the free-field at the ground surface about 330 ft away from the Refuelling Building, at the base of the Refuelling Building, and at ground level within the Refuelling Building. A generalized cross-section through the plant and the accelerograms for motions recorded in horizontal directions are shown in Fig. 13. It may be noted that the peak recorded ground surface accelerations had values of 0.35g and 0.25g in the longitudinal and transverse directions.

A fortuitous aspect of this event was the fact that the soil conditions at the plant site had been determined by a comprehensive investigation only about 12 months before the earthquake occurred. Thus the data is available, in terms of known surface motions in the free-field and soil characteristics to check the adequacy of seismic design procedures against the known performance of a prototype structure under known field conditions of considerable intensity.

The results of such a study obtained using finite element procedures to compute the motions within the Refuelling Building from the known records of free-field ground motion are described by Valera et al (1977). The results of the studies (using the program FLUSH) are shown in Fig. 14. It should be noted that separate analyses were made for the longitudinal and transverse records of free-field motion and for various soil profiles considered to be representative of the field conditions. The ranges of analytical results are presented in Fig. 14 in the form of response spectra, where they are also compared with the spectra for the recorded motions.

It may be seen that for both longitudinal and transverse motions, the recorded motions at the base of the structure are in reasonably good agreement with those computed using the finite element procedure for implementation of an 'idealized' complete interaction analysis. For both components of motion the analysis procedure indicates a higher peak in the response spectrum at a frequency of about 3 Hz than actually developed, but considered overall, the agreement between computed and recorded base motion spectra is both gratifying and encouraging.

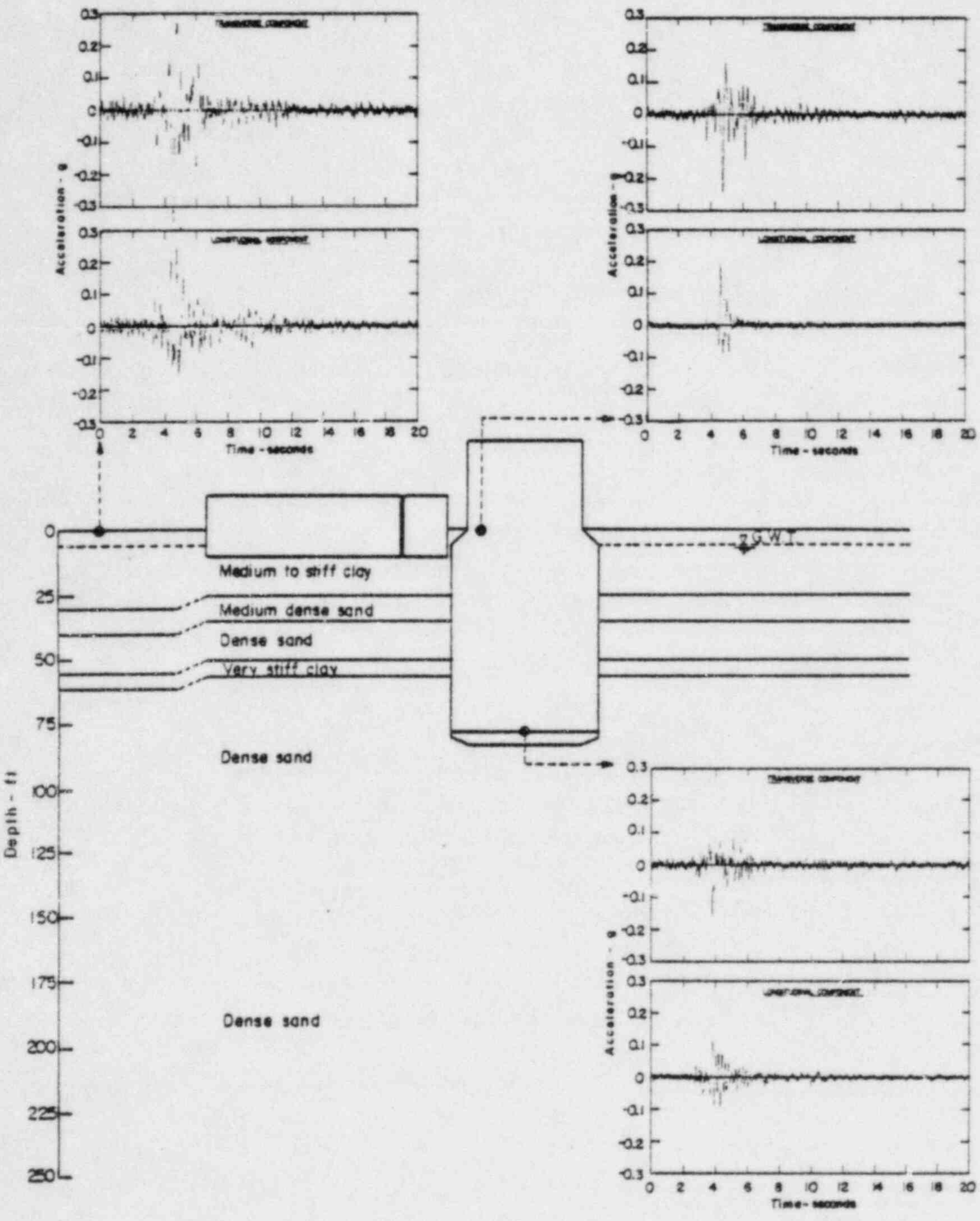


Fig. 13 GROUND MOTION RECORDS AT HUMBOLDT BAY POWE? PLANT

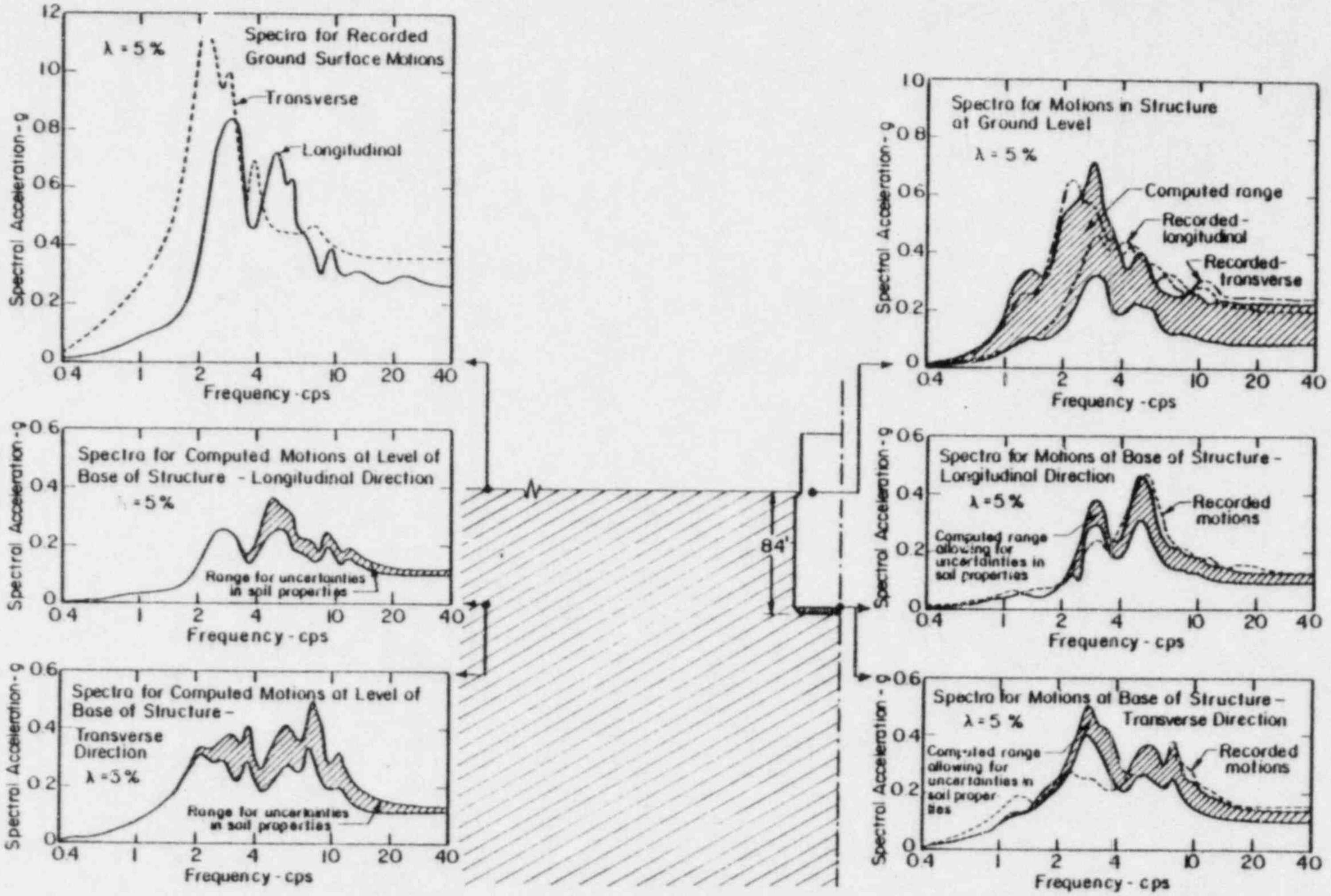


Fig. 14 COMPARISON OF RECORDED AND COMPUTED SPECTRA IN REFUELING BUILDING - HUMBOLDT BAY POWER PLANT

Similarly the recorded motions in the structure at ground level fall essentially within the range computed by the interaction analysis procedure, providing further confirmation of the ability of a complete interaction analysis to compute the structural response with an adequate degree of accuracy in this case.

It is recognized, of course, that one such test of the applicability of any analytical procedure does not necessarily provide proof that it will always lead to good evaluations of field performance. Nevertheless in the current absence of any other opportunity to check analytical methods for computing response under strong shaking of prototype structures, the results obtained in even this single case can give designers increased confidence in the usefulness of the analytical tools at their disposal.

Conclusion

In the preceding pages the authors have attempted to summarize the current capability for evaluating soil-structure interaction effects during earthquakes using finite element procedures. A concise summary of methods available, together with their capabilities and relative costs is presented in Table 1. It is believed that these procedures provide a powerful tool for use in the design of nuclear plants. However like all such procedures they must be used with an intimate knowledge of the technical details which are built into the various computer programs and which are necessary for adequate modelling of soil-structure systems. Used in this way, in conjunction with good engineering judgment, and with full recognition of their limitations (Christian, 1975), they provide evaluations of response with a level of accuracy entirely adequate for engineering design as evidenced by the recently completed study of the motions developed in the Humboldt Bay Power Plant (Valera et al, 1977). This is not meant to imply that other procedures, not involving finite element techniques, cannot provide equally good evaluations of response in many cases. However any method used for evaluating the response of embedded structures should provide the same level of capability; without this, computed responses may need more careful modification on the basis of the judgment of the engineer in order to provide a valid basis for the design of critical structures.

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Part V

SOIL-STRUCTURE INTERACTION WITH RAYLEIGH WAVES*

By Alberto Gomez-Masso[1], A. M. ASCE, John Lysmer[2], M. ASCE
Jian-Chu Chen[3], and H. Bolton Seed[2], F. ASCE

INTRODUCTION

The current interest in nuclear power and concerns regarding the seismic safety of the facilities involved has led to the development of many methods of seismic soil-structure interaction analysis, Idriss et al (9) and Lysmer (17). A complete analysis of the interaction problem consists of several parts. First, the seismic environment must be defined. Second, an analytical model must be designed for the soil-structure system, and, third, the response of the model must be evaluated by some effective and accurate numerical technique.

The models which have been proposed fall into two main groups: Complete models and substructure models, each of which may be formulated in terms of continua or finite elements. Only complete finite element methods will be discussed herein. Such methods have been developed for a wide range of geometries and simple seismic environments consisting of vertically propagating body waves, e.g. Kausel and Roesset (10), Seed et al (22) and Lysmer et al (16).

The seismic environment is usually defined in terms of a broad-band acceleration spectrum for the motion of a single control point, usually at the

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ground surface, and a synthetic time-history of acceleration which approximately fits the design response spectrum is used as the control motion. No requirement is made by the regulatory agencies concerning the nature of the wave field which produces the motion at the control point, but the usual assumption is, as mentioned above, that the soil motions are primarily due to vertically propagating shear waves and compression waves. On the basis of this assumption, analytical techniques based on one-dimensional wave propagation theory for a viscoelastic layered system and equivalent linear soil modeling techniques have been developed to calculate the motions everywhere in the free field. Results obtained by this approach have been found to be in good agreement with field observations of ground response (Schnabel et al (18)), and soil-structure interaction (Valera et al (25)), during actual earthquakes.

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, and some authors, e.g., Wong and Trifunac (28), Luco (11), 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, surface waves have been observed in several earthquakes such as El Centro, 1940 (Trifunac (23)), Parkfield, 1966 (Anderson (1)), and San Fernando, 1971 (Hanks (8)).

Methods for approximating the effects of horizontally propagating waves on structures and earth dams have been proposed by Scavuzzo (20), Dezfulian and Seed (5), Dibaj and Penzien (6), Udaka (24), Scanlan (19), Werner et al

(27), Wong and Trifunac (28), Luco (11) 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 deposits and for methods of soil-structure interaction analysis capable of handling a wider range of possible seismic environments. As a step in this direction a method is presented herein which can accept as input any type of plane-strain wave pattern in a horizontally layered free-field deposit.

The method makes use of viscoelastic finite elements, solves the equilibrium equations in the frequency domain by the complex response method, and uses the equivalent linear method to approximate nonlinear soil behavior. All of these methods are described in a recent state-of-the-art report (Idriss et al (9)).

The method also employs several types of transmitting boundaries to simulate three-dimensional effects and the semi-infinite geometry of the soil-structure interaction problem. These boundaries have been previously described by Lysmer and Kuhlemeyer (13), Waas (26), Lysmer and Waas (15) and Lysmer and Drake (14). The manner in which the boundaries are used is similar to the application in the FLUSH computer code, Lysmer et al (16).

The proposed method can, in principle, accept any plane strain seismic environment with the limitation that the free field must be horizontally layered. The environment may consist of any type of P- or SV-waves and different modes of Rayleigh waves. However, due to space limitations and our current limited ability to specify what types of waves will actually occur only vertically propagating body waves and fundamental-mode Rayleigh waves will be considered herein.

While the computation of the free-field motions caused by vertically propagating body waves is now a well-established part of the state-of-the-art of earthquake engineering, this may not be true for the case of Rayleigh waves. Hence, a method is outlined for the computation of Rayleigh wave fields in horizontally layered viscoelastic systems. Then, as an illustration of the application of the complete method, comparative results are presented from analysis of a typical nuclear structure subjected to a body wave field and to a Rayleigh field with the same control motion (1) on a rock site and (2) on a sand site. Finally, conclusions are drawn as to the relative importance of the nature of the seismic environment on hard and soft sites.

OUTLINE OF METHOD

The proposed method of analysis is based on a superposition theorem described by Clough and Penzien (4) and Lysmer (17). A simplified version of this theorem has previously been used by Aydinoglu and Cakiroglu (2) for surface structures. The full theorem has been used by Gomez-Masso (7) for embedded structures with flexible basements. The theorem is illustrated in Fig. 1. It states that the total displacements of a soil-structure system can be computed from the superposition

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

where u are the displacements of the total, SSS, system, u_f are the free-field motions of the FFS system and u_i are interaction displacements computed from the NET system. The NET system is similar to the SSS system but all forces act on the lower part of the structure. Assuming that all three models in Fig. 1 are idealized by finite elements these forces can be computed from

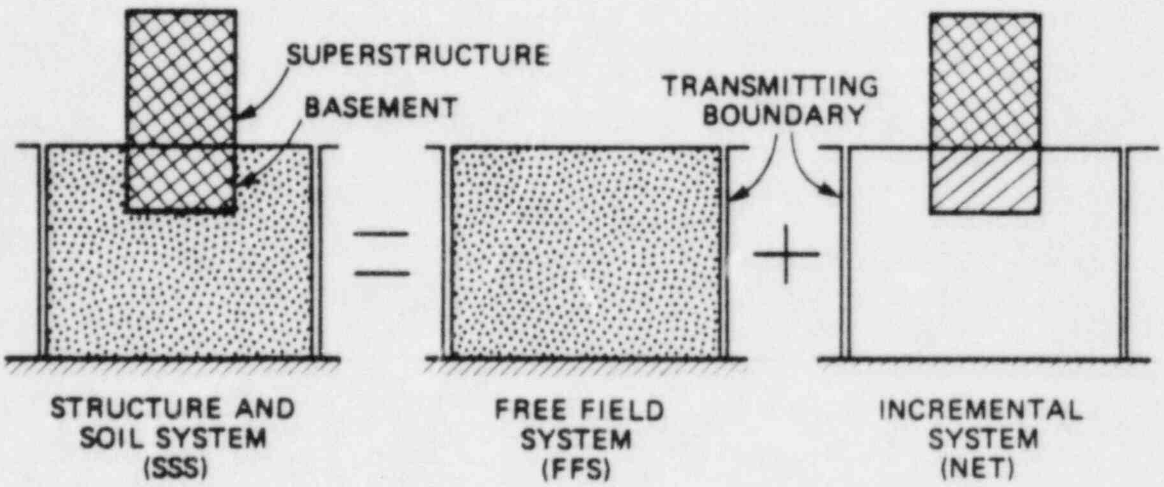

FIG. 1 SUPERPOSITION STAGES FOR COMPLETE ANALYSIS

(Lysmer (17), p. 1270)

$$\{q_i\} = -([M] - [M_f])\{\ddot{u}_f\} - ([C] - [C_f])\{\dot{u}_f\} - ([K] - [K_f])\{u_f\} \quad (2)$$

where [M], [C] and [K] are the mass, damping and stiffness matrices, respectively, for the SSS system and $[M_f]$, $[C_f]$ and $[K_f]$ are the corresponding matrices for the FFS system. The stiffness and damping matrices include any effects of transmitting boundaries.

The superposition theorem as expressed by Eqs. (1) and (2) is valid in three dimensions. However, in this paper we will only consider two-dimensional plane-strain situations with an approximation for three-dimensional effects. The proposed method consists of two major steps,

Step A - Determination of the free-field motions, $\{u_f\}$, by finding some solution to the FFS model (or some other model representing the free-field).

Step B - Determination of the interaction motions, by applying the forces from Eq. (2) to the NET model.

followed by the superposition stated by Eq. (1). The need to use superposition implies, as discussed by Lysmer (17), that, general nonlinear soil-structure interaction analysis is not possible. Nonlinearities can, however, be approximated by the equivalent linear method (Seed and Idriss (21)).

COMPUTATION OF THE FREE-FIELD MOTIONS

The types of waves considered are vertically propagating S- and P-waves and horizontally propagating Rayleigh waves (R-waves) in a site consisting of horizontal viscoelastic layers overlying a viscoelastic half space.

Let the control motion, $\ddot{y}(t)$, be specified at the top of the nth layer at x = 0 and let it be assumed that this motion has been digitized at N

(power of 2) points. Then the control motion can be expanded into the Fourier series

$$\ddot{y}(t) = \mathrm{Re}\sum_{s=0}^{N/2} \ddot{Y}_s \cdot \exp(i\omega_s t) \tag{3}$$

where $\omega_s = 2\pi s/(N\cdot\Delta t)$ is the circular frequency of the sth harmonic. The complex amplitudes, Y_s, are most conveniently computed by the Fast Fourier Transform technique.

Similarly, the free-field motions can be written

$$\{u_f\} = \mathrm{Re}\sum_{s=0}^{N/2} \{U_f\}_s \cdot \exp(i\omega_s t) \tag{4}$$

where $\{u_f\}$ is a vector containing the displacements at the top of each layer, and $\{U_f\}_s$ are the corresponding complex amplitudes.

According to the complex response method each of the harmonics in the above expressions can be treated independently and we can define a vector of transfer functions, $\{A_f\}_s$, from control motion acceleration amplitudes to free field displacement amplitudes through

$$\{U_f\}_s = \{A_f\}_s \ddot{Y}_s \tag{5}$$

If these transfer functions can be determined we have immediately, from Eq. (4)

$$\{u_f\} = \mathrm{Re}\sum_{s=0}^{N/2} \{A_f\}_s \ddot{Y}_s \exp(i\omega_s t) \tag{6}$$

which can be evaluated by the Inverse Fast Fourier Transform technique, and the free field motions have been determined.

Transfer Functions for Body Waves

Transfer functions for vertically propagating body waves can be determined by continuum techniques, Schnabel et al (18) or finite element techniques, Lysmer et al (16). The latter approach was used for the work presented herein with the slight complication that simultaneous propagation of P- and S-waves was considered in order to create fields which were compatible with those produced by R-waves. This involved the use of two control motions, a horizontal control motion, $\ddot{y}^H(t)$ for S-waves and a vertical control motion, $\ddot{y}^V(t)$ for P-waves. The only change introduced by this modification is that Eq. (6) should be replaced by

$$\{u_f\} = \mathrm{Re}\sum_{s=0}^{N/2}(\{A_f^H\}_s \ \ddot{Y}_s^H + \{A_f^V\}_s \ \ddot{Y}_s^V)\ \exp(i\omega_s t) \tag{7}$$

The notation being the obvious extension of what was used above.

Transfer Functions for Rayleigh Waves

Transfer functions for Rayleigh wave fields can be computed by the technique described by Lysmer (17). According to this technique it is assumed that displacements vary linearly between layer interfaces and that the underlying half space is rigid. With these assumptions the general motion of an M-layer visco elastic system at the frequency ω_s is fully described by the following vector which contains the displacements at the layer interfaces:

$$\{u_f(t,x)\}_s = \mathrm{Re}\sum_{j=1}^{2M} R_j\{v\}_j \cdot e^{i(\omega_s t - k_j x)} \tag{8}$$

Here x is the coordinate in the direction of wave propagation, $k_j = k_j(\omega_s)$ and $\{v\}_j = \{v(\omega_s)\}_j$ are corresponding eigenvalues and eigenvectors of an eigenvalue problem of the form

$$([A]k^2 + [B]k + [C] - \omega_s^2[M])\{v\} = 0 \qquad (9)$$

and $R_j = R_j(\omega_s)$ are unknown mode participation factors.

The constant 2M x 2M matrices, [A], [B], [C] and [M] in Eq. (9) are simple functions of the properties of the layered system. Methods for their formation from layer submatrices and solution of the eigenvalue problem have been presented by Lysmer and Drake (14) and Waas (26). In a viscoelastic system all of the k_j-values will be complex with negative imaginary parts corresponding to wave motions which decay in the direction of wave propagation. The term with the smallest value of $Re(k_j)$ is here called the fundamental mode and it will, as shown by Lysmer (13) be identical to the fundamental Rayleigh wave mode defined by seismologists for a layered system overlying a deformable half space provided the depth to the rigid half space assumed herein is deep enough. The depth required to simulate a half space solution is essentially inversely proportional to frequency. Hence very deep models are required at low frequencies. This problem has been solved by Chen (3), who observed that for the purpose of computing fundamental modes a deformable half space can be simulated accurately by ten sublayers over a rigid base provided the thicknesses of these sublayers are made inversely proportional to frequency. Thus low frequency solutions can be obtained without increasing the total number of layers and with this the computational effort; provided the depth to the rigid base is made frequency-dependent. Accordingly, all free-field Rayleigh motions used herein were computed from a model in which the total soil system was represented by a layered soil system overlying ten uniform layers with half space properties and changing thickness depending on frequency. This led to models which were several thousands of feet deep in the low frequency range and accurate

fundamental Rayleigh modes in the entire frequency range considered.

In general, it is not possible to determine all of the mode participation factors, R_j, appearing in Eq. (8). However, if it assumed that the fundamental mode, j = 1, predominates at all frequencies; i.e., that $R_j = 0$ for $j \neq 1$ the motion corresponding to the frequency ω_s reduces to

$$\{u_f\}_s = \mathrm{Re}(R_s\{v\}_s\, e^{i(\omega_s t - k_s x)}) \tag{10}$$

where the subscript, s, on R, k and {v} now refer to the frequency ω_s ($R_s = R_1(\omega_s)$ etc).

The corresponding free-field acceleration amplitudes at and below the control point (x = 0) are

$$\{\ddot{U}_f\}_s = -\omega_s^2\{U_f\}_s = -\omega_s^2 \cdot R_s \cdot \{v\}_s \tag{11}$$

One of the components, say the rth, of $\{\ddot{U}_f\}_s$ is the control motion amplitude $\ddot{Y}_s$ defined above, and since $\{v\}_s$ is known from the solution of the eigenvalue problem in Eq. (9) we can directly compute the mode participation factor from

$$R_s = \frac{-\ddot{Y}_s}{\omega_s^2 \cdot v_{rs}} \tag{12}$$

It immediately follows, by backsubstitution into Eq. (10), that the free-field transfer functions are

$$\{A_f\}_s = \frac{-1}{\omega_s^2 \cdot v_{rs}} \cdot e^{-k_s x} \tag{13}$$

and the complete transient free-field displacement field, which now decays in the x-direction follows from Eq. (6). The transfer functions are not defined for s = 0. However, this difficulty is easily removed by setting $\ddot{Y}_o = 0$.

Nonlinearities

Nonlinearities in the free field were as mentioned above approximated by the equivalent linear method according to which strain-compatible soil moduli and damping values are obtained by iteration. However, since the above theory for Rayleigh waves is valid only for perfect horizontally layered systems the material properties could not be allowed to vary in the x- irection. Thus all iterations on layer soil properties were performed using strains computed on a vertical line through the control point, i.e., at $x = 0$. No iterations were performed for the material properties of the underlying (rock) half-space.

Further, although the computer program CREAM, Gomez-Masso (7), has this ability, no further iterations on soil properties were performed during the determination of the interaction displacements described below. This is justified by the observation, by the writers and several other authors, that such secondary iterations have virtually no effect on the final displacements as long as iterated free-field soil properties are used in the interaction part of the analysis.

COMPUTATION OF THE INTERACTION MOTIONS

The interaction motions were computed using program CREAM, Gomez-Masso (7) from the NET model shown in Fig. 2. In this model all motions are assumed to occur in the XY-plane. Within the central block, which includes the structure, the basement and the immediately adjacent soil, irregular elements and iteration on soil properties can be used. If the soil properties within this region are different from those in the free-field, the entire region is considered to be part of the structure, i.e., the forces $\{q_i\}$ in Eq. (2) are nonzero at all nodes of the central region. Each of the blocks surrounding

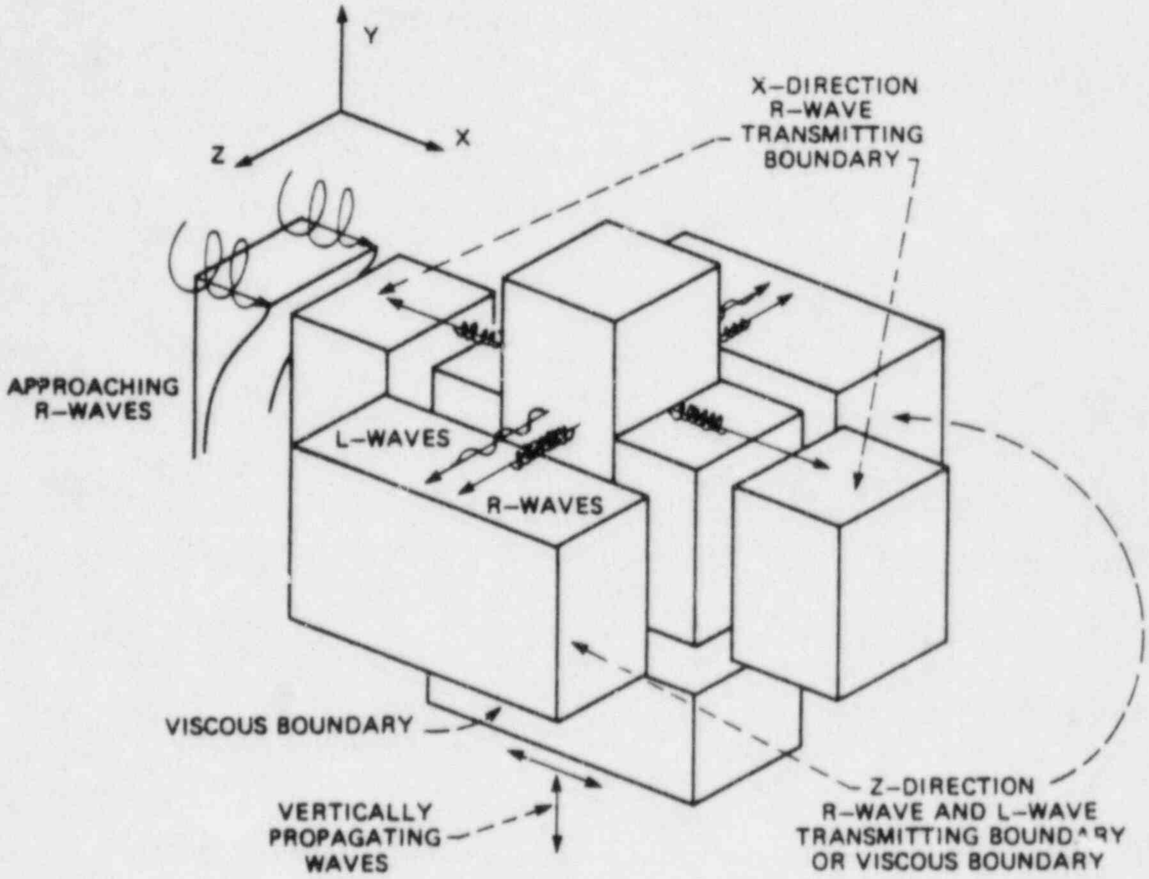

FIG. 2 MODEL FOR INCIDENT AND REFLECTED SEISMIC WAVES

the central block in Fig. 2 represent different types of transmitting boundaries which, as will be discussed below, are designed to account for waves radiating from the excited central region.

All materials are assumed to be viscoelastic with some fraction of critical damping, β, which may vary from element to element. Complex response analysis is used throughout. Hence, material damping can be accounted for by using complex moduli, $G^* = G \cdot \exp(2i\beta)$ etc., in the formation of all stiffness matrices (Idriss (9), Appendix A). The notation $[K^*]$ etc. will be used below to indicate where complex moduli are used.

The equation of motion for the NET system is

$$[M]\{\ddot{u}_i\} + [C]\{\dot{u}_i\} + [K]\{u_i\} = \{q_i\} - \{f\} \tag{14}$$

where $-\{f\}$ are the forces on the transmitting boundaries.

Introduction of the Fourier expansions

$$\{u_i\} = \mathrm{Re}\sum_{s=0}^{N/2}\{U_i\}_s \exp(i\omega_s t) \tag{15}$$

$$\{f\} = \mathrm{Re}\sum_{s=0}^{N/2}\{F\}_s \exp(i\omega_s t) \tag{16}$$

$$\{q_i\} = \mathrm{Re}\sum_{s=0}^{N/2}\{Q_i\}_s \exp(i\omega_s t) \tag{17}$$

and elimination of $[C]$ by the above mentioned use of complex moduli yields the following equation of motion for each frequency ω_s, $s = 0,\ldots,N/2$.

$$([K^*] - \omega_s^2 [M])\{U_i\}_s = \{Q_i\}_s - \{F\}_s \tag{18}$$

where

$$\{Q_i\}_s = (([K^*] - [K_f^*]) - \omega_s^2 ([M] - [M_f]))\{U_f\}_s \tag{19}$$

The latter follows from Eqs. (2) and (17).

Transmitting Boundaries

Three types of transmitting boundaries are used in program CREAM.

Rayleigh wave boundaries for which

$$\{F\}_s = [R^*]_s\{U_i\}_s \tag{20}$$

Love wave boundaries for which

$$\{F\}_s = [L^*]_s\{U_i\}_s \tag{21}$$

and Viscous boundaries for which

$$\{F\}_s = i\omega_s[V^*]\{U_i\}_s \tag{22}$$

The Rayleigh and Love wave boundaries have been completely described by Waas (26), Lysmer and Waas (15) and Lysmer and Drake (14). These boundaries are "exact" in that they correctly model the existence of a semi-infinite layered system attached to the central region. They will, no matter how close they are placed to the structure, transmit any incident plane waves; be they surface waves or body waves. The matrices which appear in Eqs. (20) and (21) are frequency dependent and can be determined from the solution to the eigenvalue problem stated by Eq. (9).

The viscous boundary has been previously described by Lysmer and Kuhlemeyer (12). The matrix $[V^*]$ is diagonal and frequency independent. While simpler to use this boundary condition is not perfect except for the special case of normal incidence of body waves.

In obtaining all of the results presented at the end of this paper Rayleigh wave boundaries were used on the YZ-planes of the central block shown in Fig. 2 and viscous boundaries were used on the XY-planes to simulate three-dimensional effects. A rigid boundary was assumed as the base of the central block. This approach is similar to that used in the FLUSH program, Lysmer et al (16).

Several computations were performed using the theoretically more attractive Love and Rayleigh wave boundaries on the XY-planes. However, as can be seen from the results presented in Fig. 3 the potential improvement was trivial, less than 7 percent on spectral values, and the simpler viscous boundaries were adopted for further computations. In Fig. 3 the curves marked FLUSH-2D correspond to a plane-strain analysis with no attempt to model 3-D effects, i.e., no transmitting boundaries on the XY-planes. The curves marked FLUSH dashpots correspond to viscous boundaries on the XY-planes, and the curves marked CREAM correspond to results obtained by using the above mentioned Love and Rayleigh wave boundaries.

Even smaller differences were found in a study of the effect of using a viscous transmitting boundary at the bottom of the central block instead of a rigid base, confirming the often observed fact that reflections from an assumed rigid base are insignificant since most of the interaction displacements consist of shallow surface waves which do not penetrate deeply into the soil.

Summary of Procedure

Substitution of Eqs. (20)-(22) into Eq. (18) yields for each frequency

$$([K^*] + [R^*]_s + [L^*]_s + i\omega_s[V^*] - \omega_s^2[M])\{U_i\}_s = \{Q\}_s \tag{23}$$

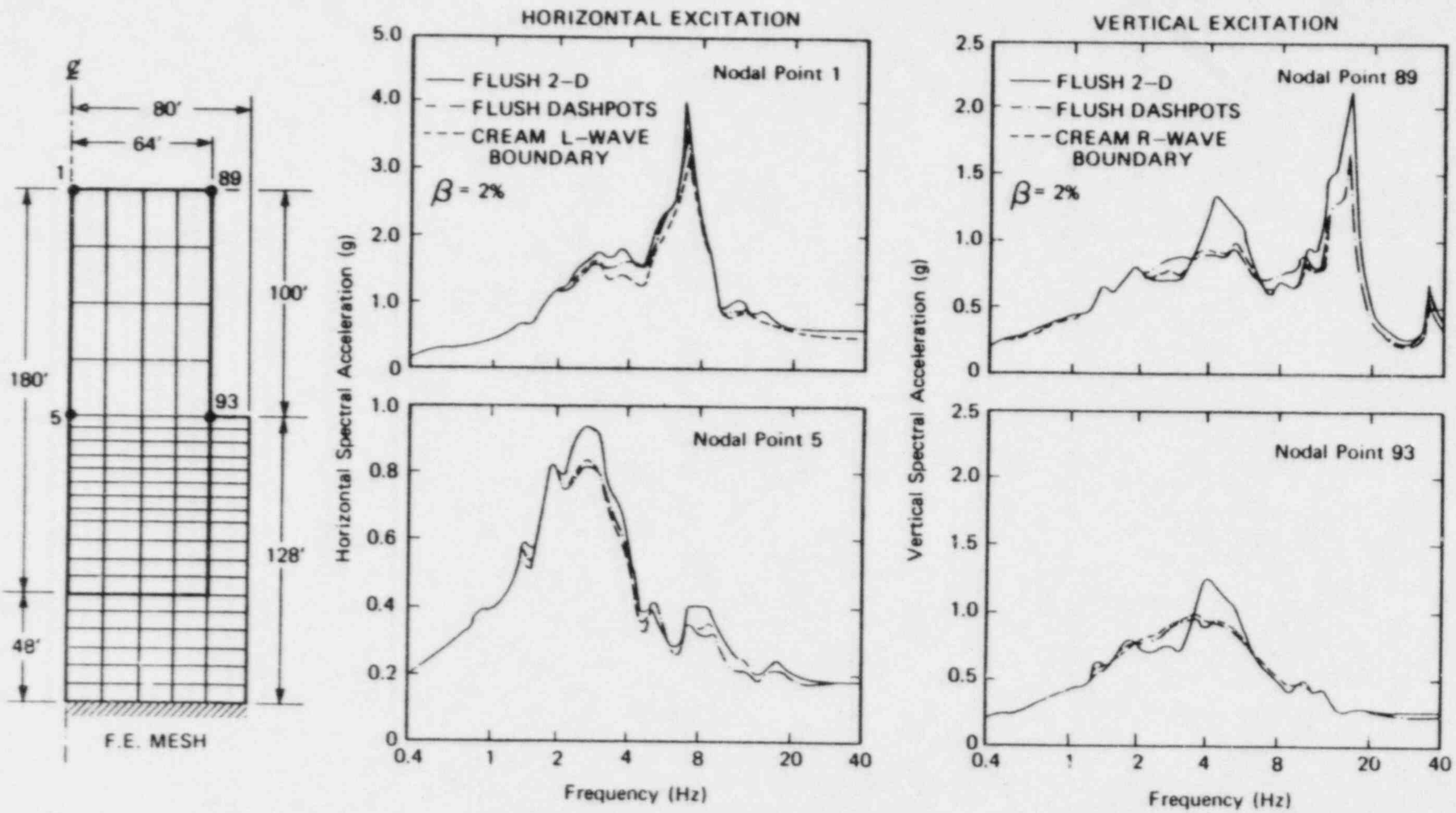

FIG. 3 FINITE ELEMENT MODEL AND COMPARISON OF RESPONSE SPECTRA FOR DIFFERENT METHODS OF 3-D SIMULATION

which can be solved by Gaussian elimination for the interaction displacement amplitudes $\{U_i\}_s$.

The main steps of the total procedure are now clear:

1. Form $[K^*]$, $[K_f^*]$, $[V^*]$, $[M]$, $[M_f]$ and $\ddot{Y}_s$, $s = 0,\ldots,N$

For each frequency, ω_s, $s = 0,\ldots,N/2$

2. Determine free-field amplitudes $\{U_f\}_s$, Eq. (11)

3. Compute $\{Q_i\}_s$ from Eq. (19)

4. Form $[R^*]_s$ and $[L^*]_s$

5. Find $\{U_i\}_s$ from Eq. (23)

and finally

6. Superimpose according to Eq. (1)

$$\{u\} = \sum_{s=0}^{N/2} (\{U_f\}_s + \{U_i\}_s) \exp(i\omega_s t)$$

CASE STUDIES

The above procedure has been implemented in the computer program CREAM and we present below two case studies in which the motions of a typical nuclear structure subjected to a pure Rayleigh wave field are compared with the response of the same structure subjected to a combined field of vertically propagating P- and S-waves which produce the same motions at the control point as the Rayleigh wave field.

In both cases the control motion (the horizontal component of the control point) had a maximum acceleration of 0.25g and a 2% acceleration response spectrum similar to that currently specified by the NRC Regulatory Commission. The actual spectrum is shown in Fig. 4 (Point G). In all analyses

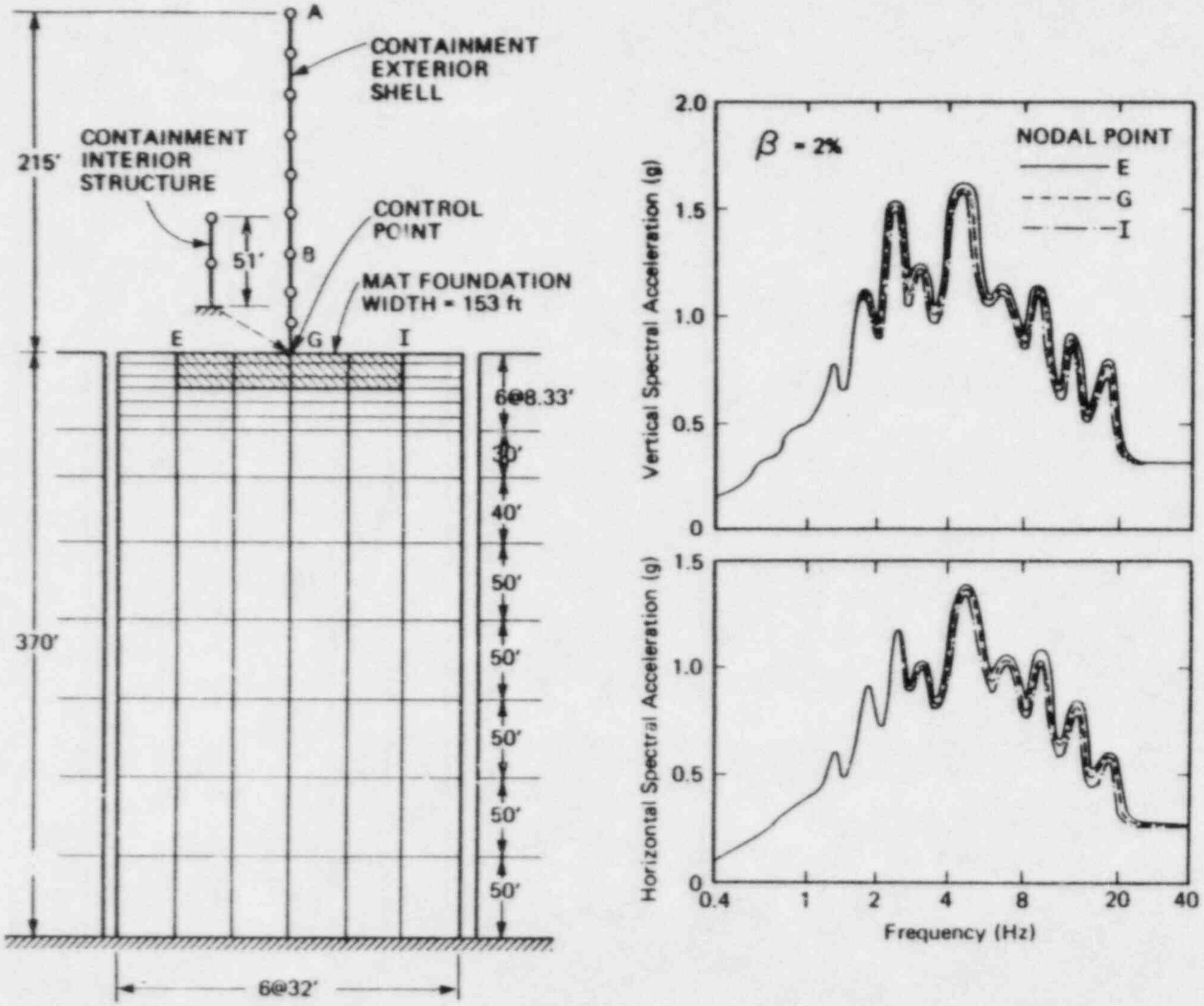

FIG. 4 FINITE ELEMENT MESH AND FREE FIELD R-WAVE MOTIONS FOR A ROCK SITE

the control point was located at the free-field ground surface. However, as will be discussed later, its location in relation to the structure was different for the two cases studied.

ROCK SITE

The first model studied involved a containment structure on a rock site, consisting of a 50 ft layer overlying a half space. The shear wave velocities of the two layers were 3600 fps and 5600 fps, respectively, and a damping ratio of 2% was used for both layers. The NET model for the system is shown in Fig. 4. The model was truncated at a depth of 370 ft and a rigid boundary was assumed at this depth.

Free-Field Motions

The free-field motions were, as discussed previously, computed using a much deeper model. The control point was located at point G in Fig. 4. Only the Rayleigh wave field will be discussed herein. Assuming waves propagating from the left the field will as indicated by Eq. (8) decay towards the right. However, due to the low damping ratio of the rock the decay within the width of the structure is relatively insignificant as indicated by the spectra shown for the free-field motions of points E, G, and I in Fig. 4. The same spectra show that the vertical motions are only about 15% higher than the horizontal motions as opposed to the 50% which would have been expected from half space theory. Thus this example clearly indicates the need to consider layering, even in rock profiles.

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 7. The maximum horizontal

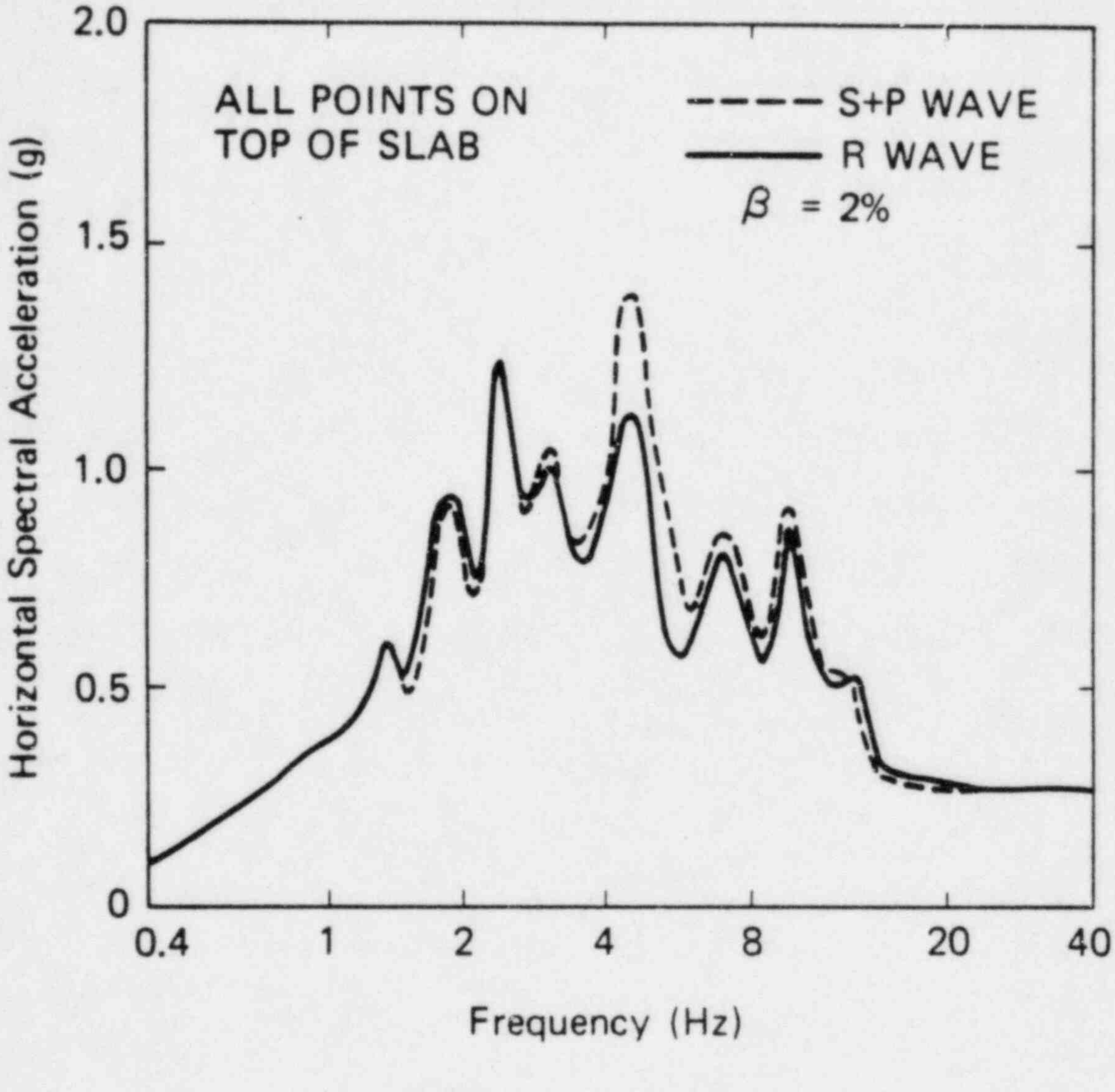

FIG. 5 HORIZONTAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (ROCK SITE)

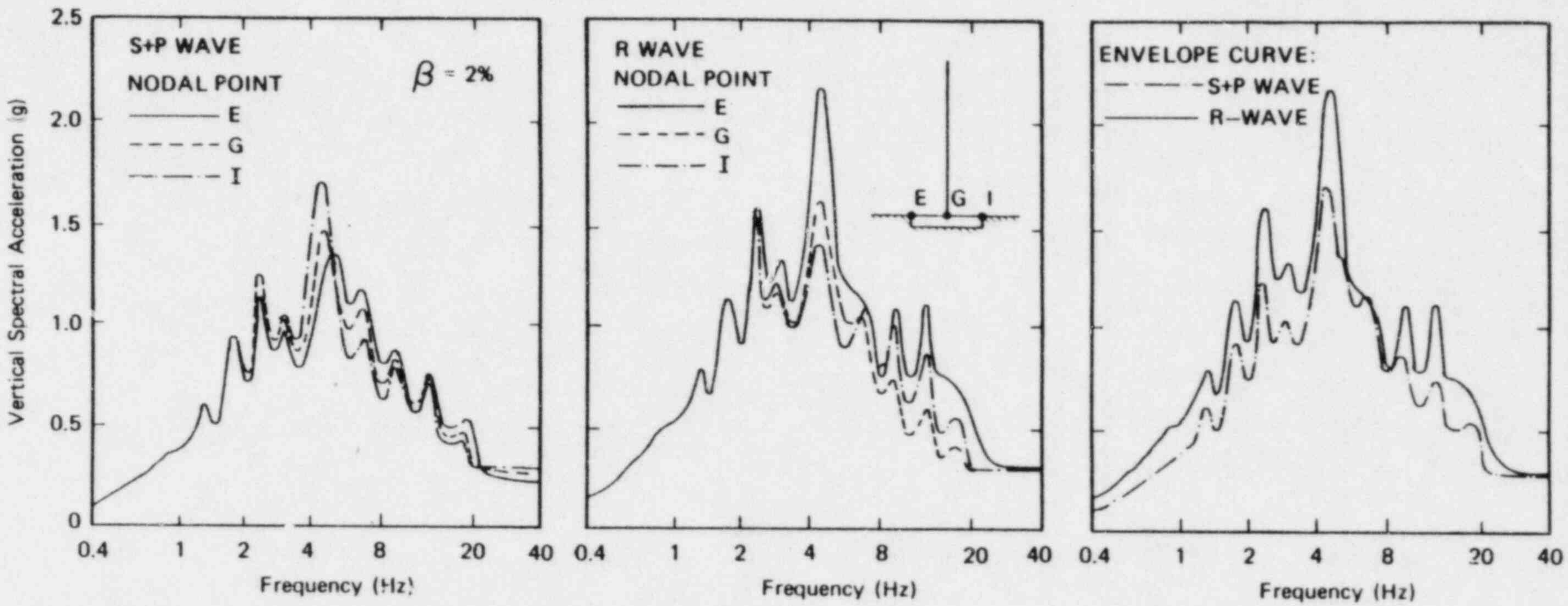

FIG. 6 VERTICAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (ROCK SITE)

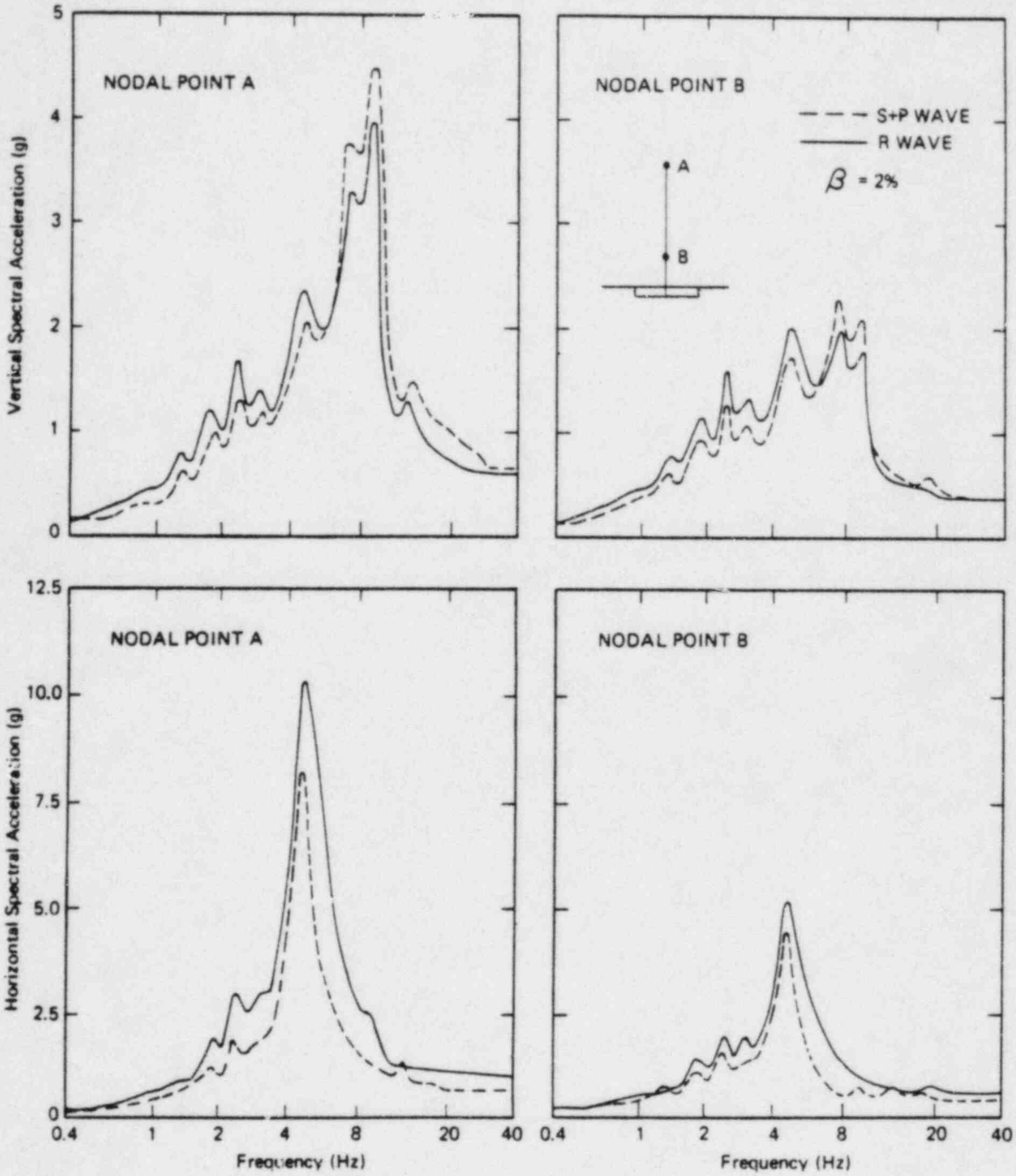

FIG. 7 COMPARISON OF RESPONSE SPECTRA AT NODAL POINTS A AND B (ROCK SITE)

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 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 25% and 15%, respectively, 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 along the vertical axis of the structure for the R-wave case were found 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 structure, 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 NET 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.

SAND SITE

The second study involved an identical structure founded on a site consisting of a 128 ft thick layer of sand overlying bedrock. The NET model used is shown in Fig. 8. The soil properties were typical of dense sand to a depth of 48 ft and of very dense sand for the remaining part of the profile.

Free-Field Motions

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. As described by Chen (3) high frequency Rayleigh waves decay rapidly in soil sites and it is unrealistic to assume that Rayleigh waves of frequencies higher than 2 Hz will survive in a sand profile. 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 to the left of 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 at that point (Point G in Fig. 8).

The characteristics of the horizontal and vertical free field motions of the R-wave field which propagates from the left on the right are shown in Fig. 8 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.

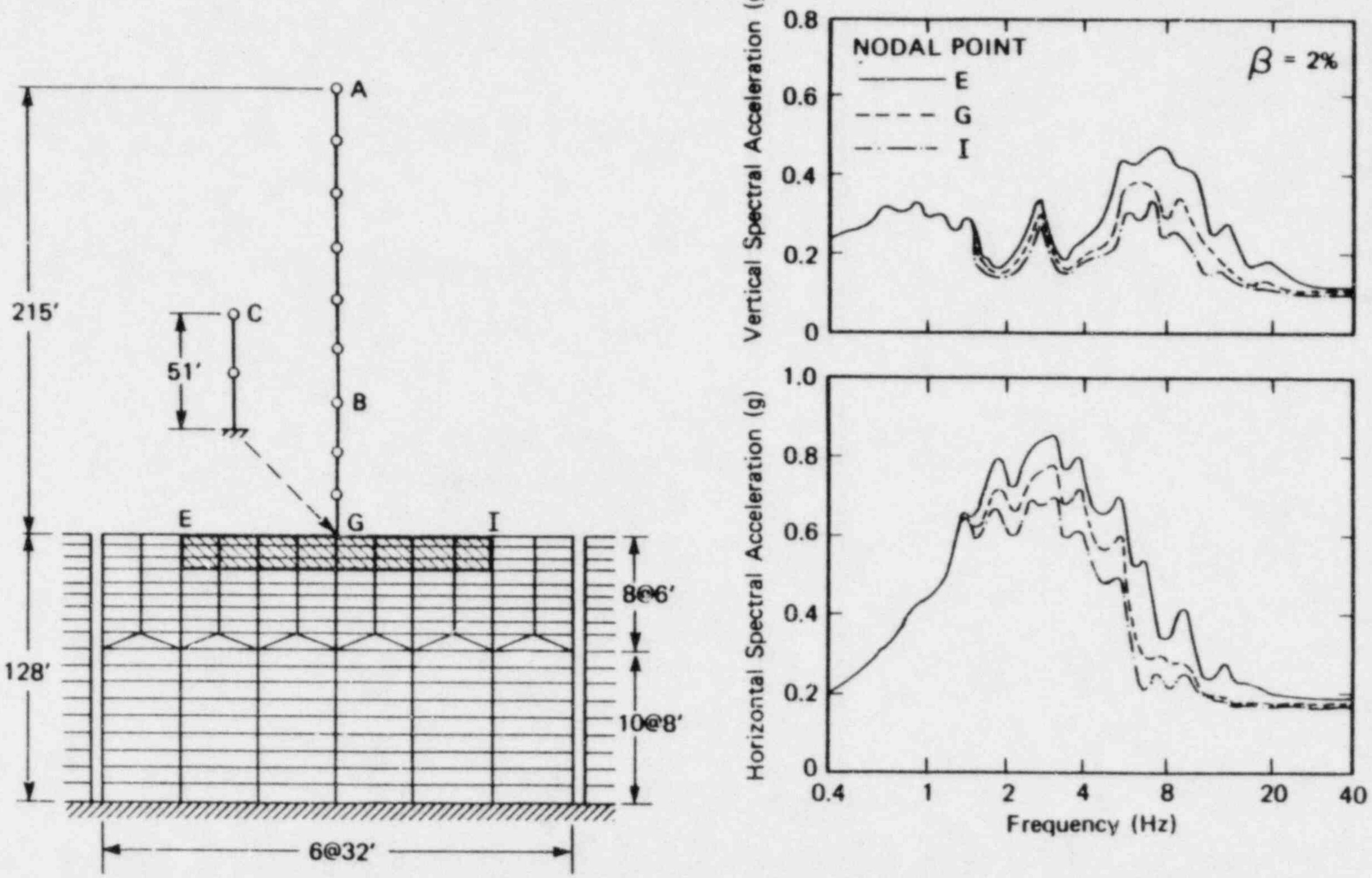

FIG. 8 FINITE ELEMENT MESH AND R-WAVE FREE FIELD MOTIONS FOR A SAND SITE

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. 9 through 11. 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. 9.

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. 10. The first and second plots in Fig. 10 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 vertical beam element are plotted in Fig. 11. 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 high response peaks which were nonexistent in the R-wave results.

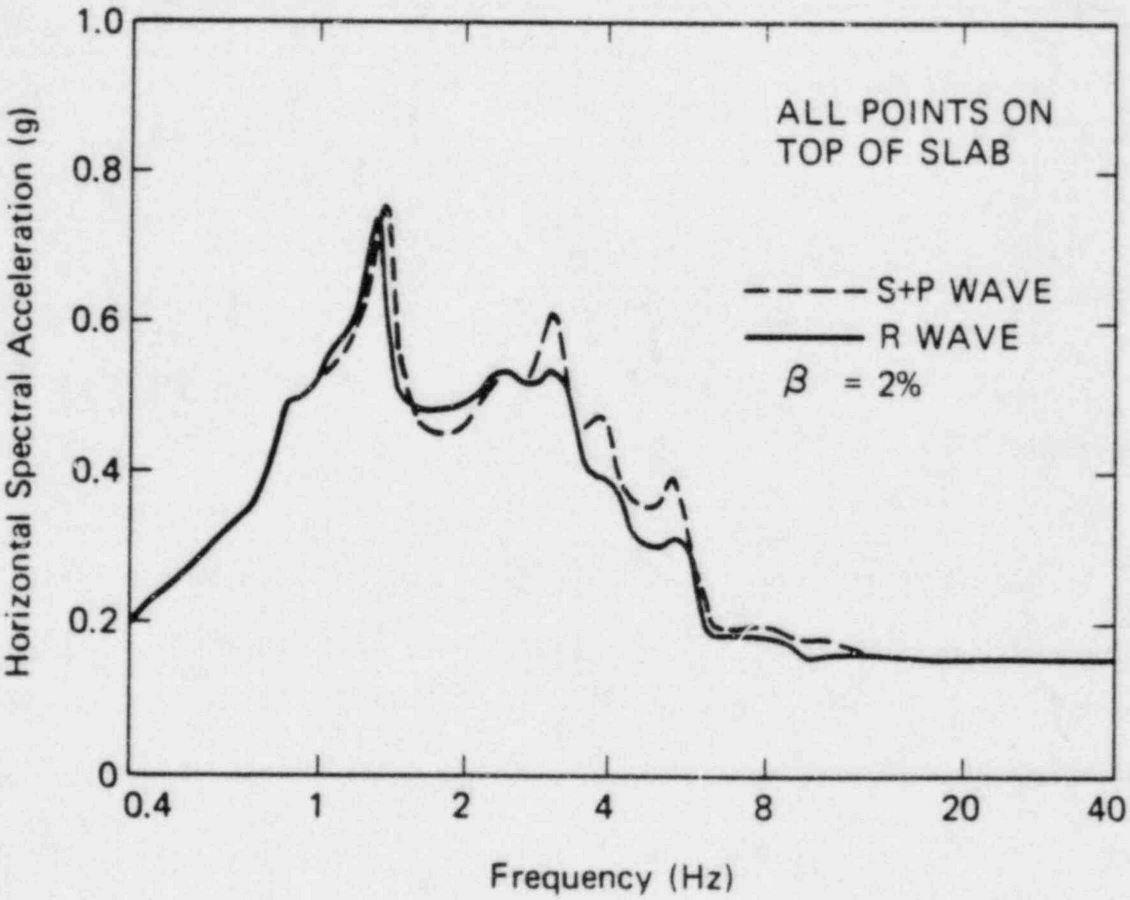

FIG. 9 HORIZONTAL RESPONSE SPECTRA ALONG THE TOP OF THE FOUNDATION SLAB (SAND SITE)

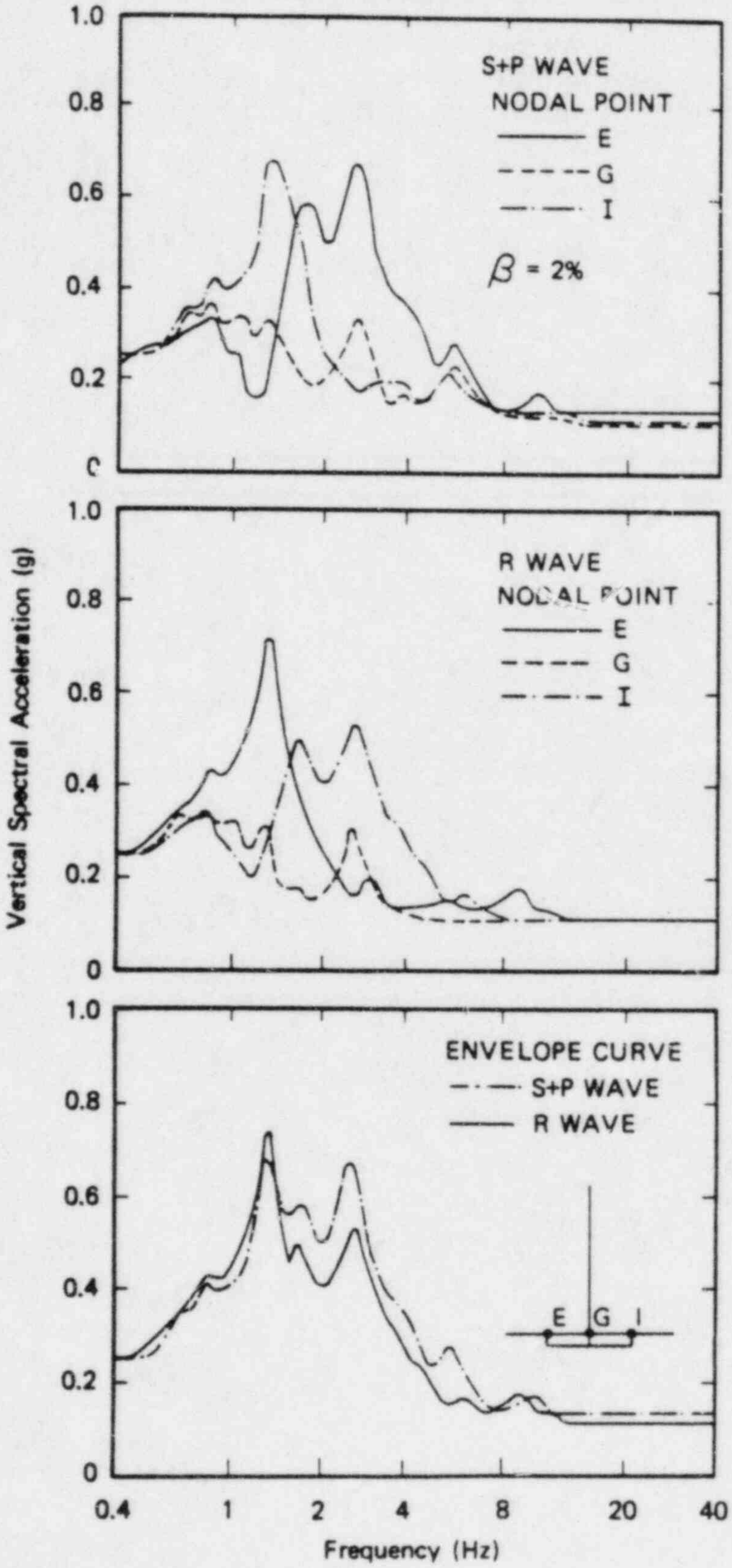

FIG. 10 VERTICAL RESPONSE SPECTRA ALONG THE TOP OF THE SLAB (SAND SITE)

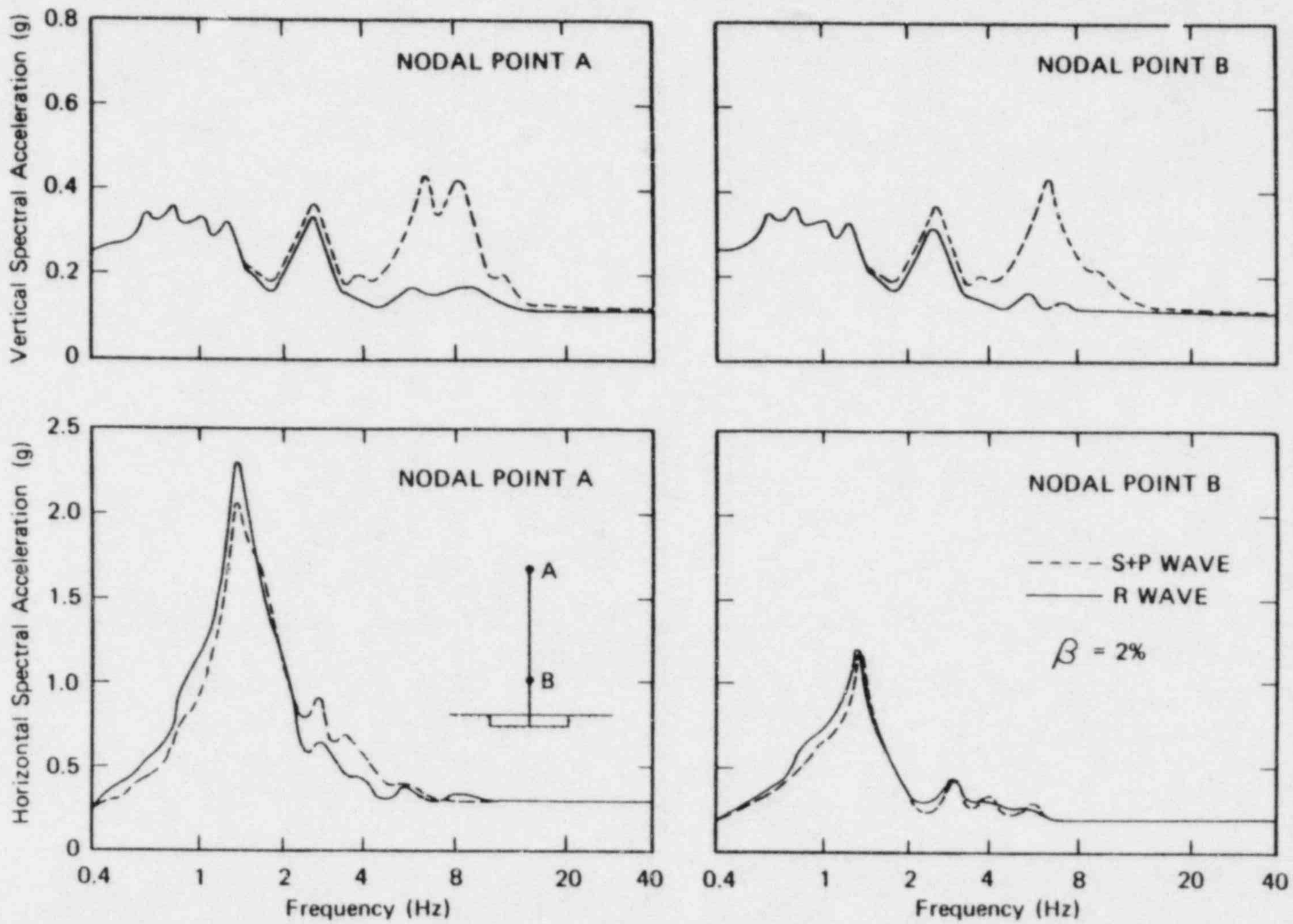

FIG. 11 COMPARISON OF RESPONSE SPECTRA AT NODAL POINTS A AND B (SAND SITE)

The maximum beam bending moments and shear and axial forces calculated for the R-wave and S + P wave analyses were found to differ by amounts ranging from 5% to 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 analysis is carried out in two steps. In the first step, the free field motions are calculated. In 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.

The main conclusions of this study are the following:

1. Soil structure interaction problems with an arbitrary seismic environment can be solved by finite element methods.
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 environment formed only by 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 the 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 fundamental mode Rayleigh waves. Neither of these cases is likely to occur in nature. In reality, Rayleigh waves have not been observed in the frequency range above

1 or 2 Hz, and thus need not be considered in the range of frequencies (typically 2 to 25 Hz) of interest in the design of nuclear power plants. Moreover, calculations have shown that the seismic environments produced by slightly inclined body waves and higher mode surface 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 provide an appropriate design procedure for most practical purposes.

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Part VI*

SOIL-STRUCTURE INTERACTION EFFECTS AT THE HUMBOLDT BAY POWER PLANT IN THE FERNDALE EARTHQUAKE OF JUNE 7, 1975

by

Julio E. Valera,[1] H. Bolton Seed,[2] C. F. Tsai[3] and J. Lysmer[4]

Introduction

One of the many controversial aspects of nuclear power plant design in the past several years has been that of evaluating the seismic soil-structure interaction effects during design levels of earthquake shaking. Basically two methods of approach are available for determining these effects: (1) complete interaction analyses which attempt to make some evaluation of the variations in earthquake motions both in the structure and the soil in which it is embedded; and (2) inertial interaction analyses in which the motions in the soil surrounding the structure are considered to be some representative average motion having the same characteristics at all points (Seed et al, 1975b). The former approach has usually been applied through the use of finite element methods of analysis while the latter, although it can be performed using finite element techniques, has usually been associated with half-space analyses of elastic or visco-elastic layered systems. It appears to be the

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prevailing opinion "that for near surface structures, good results can be obtained by a well-performed analysis of either type. However for embedded structures, the complete interaction analysis approach comes closest to representing in a rational way all the important aspects of the problem" (ASCE Ad-hoc Committee on Soil Structure Interaction for Design of Nuclear Power Plants, 1976). The principal limitation of this approach at the present time is usually considered to be the cost of the analysis and, in some cases, the less expensive inertial interaction approach may often provide results with sufficient accuracy for practical purposes. However as increasingly efficient and versatile computer programs are developed for finite element analyses and progressively more sophisticated forms of half space analysis are developed, which introduce the essential concepts of a complete interaction approach, it seems that both methods of analysis may ultimately develop to the point where they give similar results for embedded structures.

A major contributing factor to the continuing debate concerning the merits of any form of analytical approach has been the total absence of recorded field performance by which the adequacy of such an approach might be judged--making it necessary for engineers to adopt one approach or the other on the basis of their personal appraisals of such factors as the degree of sophistication of the analysis, the potential savings in design costs, the potential losses in overall project costs, their degree of understanding of the nature of the phenomena and principles involved, etc. In the absence of known field performance, all reasonable suggestions for design approaches must be considered potentially applicable and considerable

insight, wisdom and intellectual honesty is required to select a design method which offers the greatest potential for combining adequate safety for critical structures with reasonable overall economy in the cost of the completed facility. It is for these reasons that the motions recorded at the Humboldt Bay Power Plant in the Ferndale earthquake of June 7, 1975 are of major significance.

A general view of the plant is shown in Fig. 1. Units 1 and 2 are fossil fuel units whereas Unit 3 is nuclear. The buried reactor structure within the Refueling Building of Unit 3 consists of a massive concrete caisson embedded at a depth of about 85 feet below the ground surface. The various surrounding structures are lightweight structures and are founded at or close to the ground surface. The facility was constructed in 1963 and has been operating satisfactorily since that time.

Strong motion instruments at the plant have been in operation since September 1971. These are located at elevation +12 (plant grade level) and elevation -66 in the Refueling Building, and in a Storage Building (elevation +12) some 330 feet south of the Refueling Building.

The June 7, 1975 earthquake (magnitude about 5.5) had its epicenter some 15 miles south of the plant site and triggered strong motion instruments in the surrounding area including those located at the Humboldt Bay Plant (Valera and Brady, 1976). The earthquake records obtained at the Humboldt Plant are shown in Fig. 2. Although the duration of strong shaking was only about 3 to 5 records, the baseline-corrected peak accelerations developed in the free field (Storage Building) were 0.35g and 0.26g in the transverse and

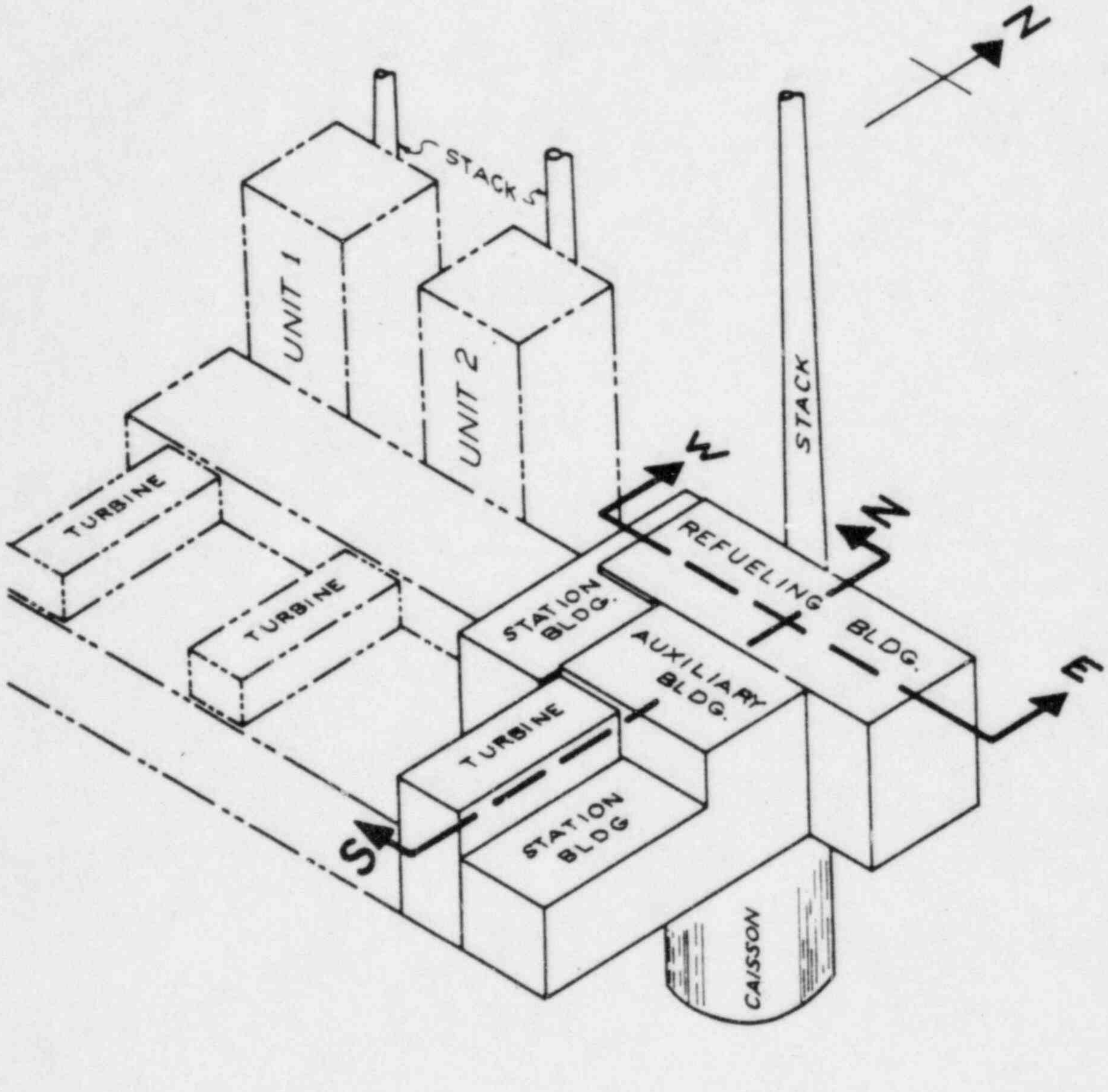

Fig. 1 GENERAL VIEW OF HUMBOLDT POWER PLANT
(After Bechtel Corp.)

longitudinal directions, respectively, making these the strongest earthquake motions to which a nuclear power plant has so far been subjected. However thre was no observable damage to the facility resulting from these motions.

A fortuitous aspect of the records obtained from the Humboldt Bay Plant was the fact that the soil conditions at the plant site had been determined by a comprehensive field investigation only about 12 months before the earthquake occurred. In fact, extensive soil structure interaction analyses using finite element procedures with accompanying determinations of soil characteristics at the site, had been completed several months prior to the earthquake of June 7, 1975. These studies were carried out by Dames & Moore using analytical techniques developed at the University of California at Berkeley (Seed et al, 1975a). In this respect it is interesting to note that these analyses had predicted a peak acceleration at the base of the Refueling Building of 0.13g for a free-field ground surface acceleration of 0.25g while the subsequent earthquake produced an average peak acceleration at the base of the Refueling Building of 0.14g for an average free-field ground surface acceleration of 0.30g. This result alone, predicted in advance of the event and published in design reports, is of considerable interest.

While these facts are of major importance, perhaps the most significant feature of the June 7 event is the opportunity it provides to check the adequacy of seismic design procedures against the known performance of a prototype structure under known field conditions of considerable intensity. The results of such an evaluation are presented in the following pages.

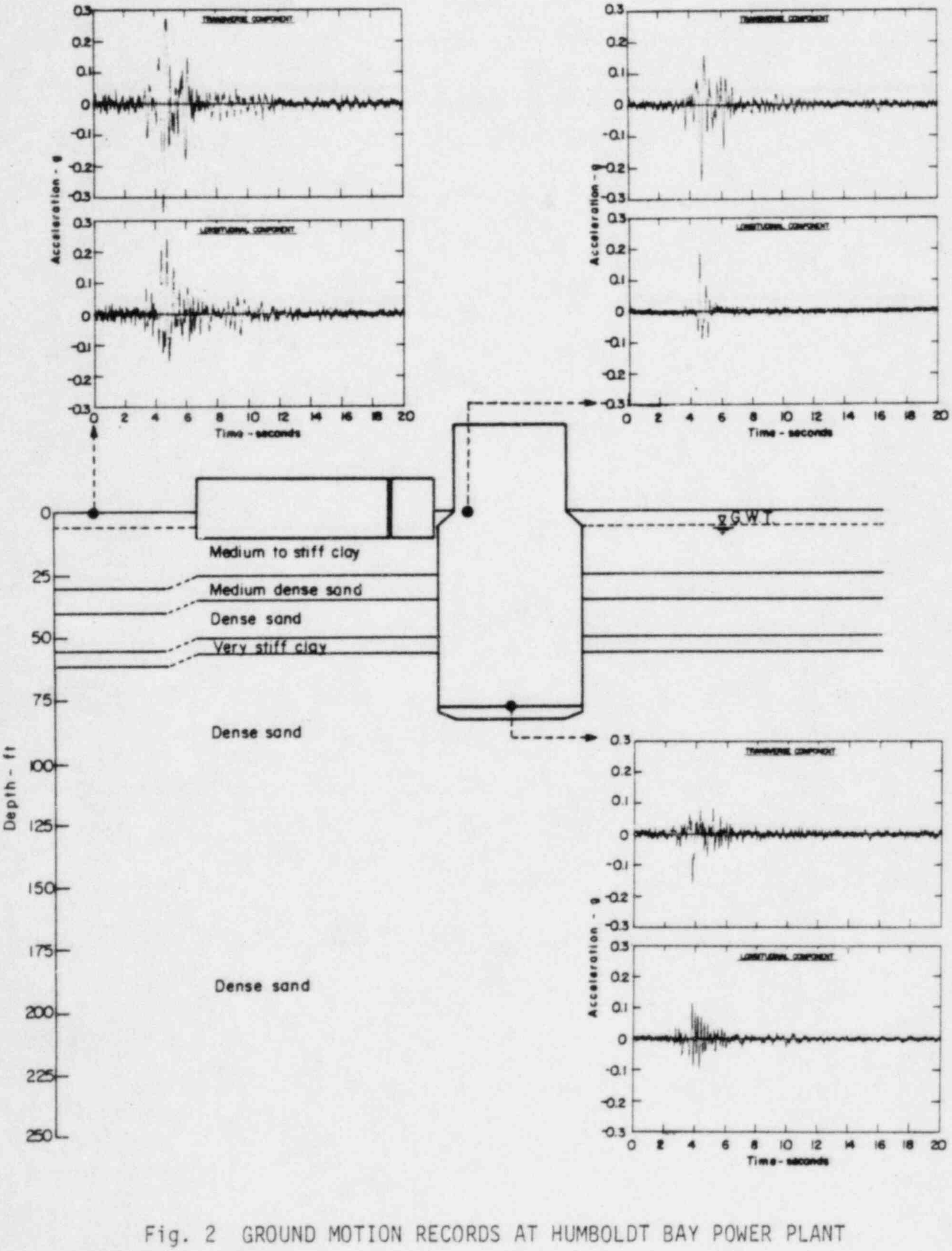

Fig. 2 GROUND MOTION RECORDS AT HUMBOLDT BAY POWER PLANT

Site Conditions and Soil Properties

A general description of the subsurface soil conditions at the plant site has been presented by Valera and Brady (1976). A cross-section through Unit 3 in the N-S direction is shown in Fig. 2. Basically the soils around the Refueling Building consist of about 25 ft of medium to stiff clay (increasing to about 30 ft at the Storage Building), underlain successively by about 30 ft of medium dense to dense sand, 10 ft of very stiff clay and then a deep bed of dense sand containing some clay lenses extending to a depth of about 400 ft. All of the soils surrounding the Refueling Building are overconsolidated with an average overconsolidation ratio of at least 6 to 8, indicating that the coefficient of earth pressure at rest in the sands would be on the order of one or more. The soil profile and soil properties used in the pre-earthquake soil-structure interaction studies are presented in Figs. 3(a) and 4, respectively. The soil profiles and soil properties used in the present study are presented in Figs. 3, 4, and 5. The profile for the conditions adjacent to the Refueling Building was identical to that used in the pre-earthquake analyses.

At the site of the Storage Building itself where the free-field records were obtained, there is some uncertainty about the actual strength of the top 30 ft of clay as the closest boring is at least 100 feet away and there is considerable scatter in the measured values of shear strength for undisturbed samples of clay taken from three borings surrounding the building. This uncertainty is reflected by the ranges of strength values for these soils indicated in Fig. 3(b). To allow for this uncertainty, analyses were made for a number of soil profiles involving clay strengths varying considerably in the upper 20 ft, as illustrated by soil profiles A, B and C in Fig. 5. Results for all profiles investigated fell within the range represented by profiles A and B.

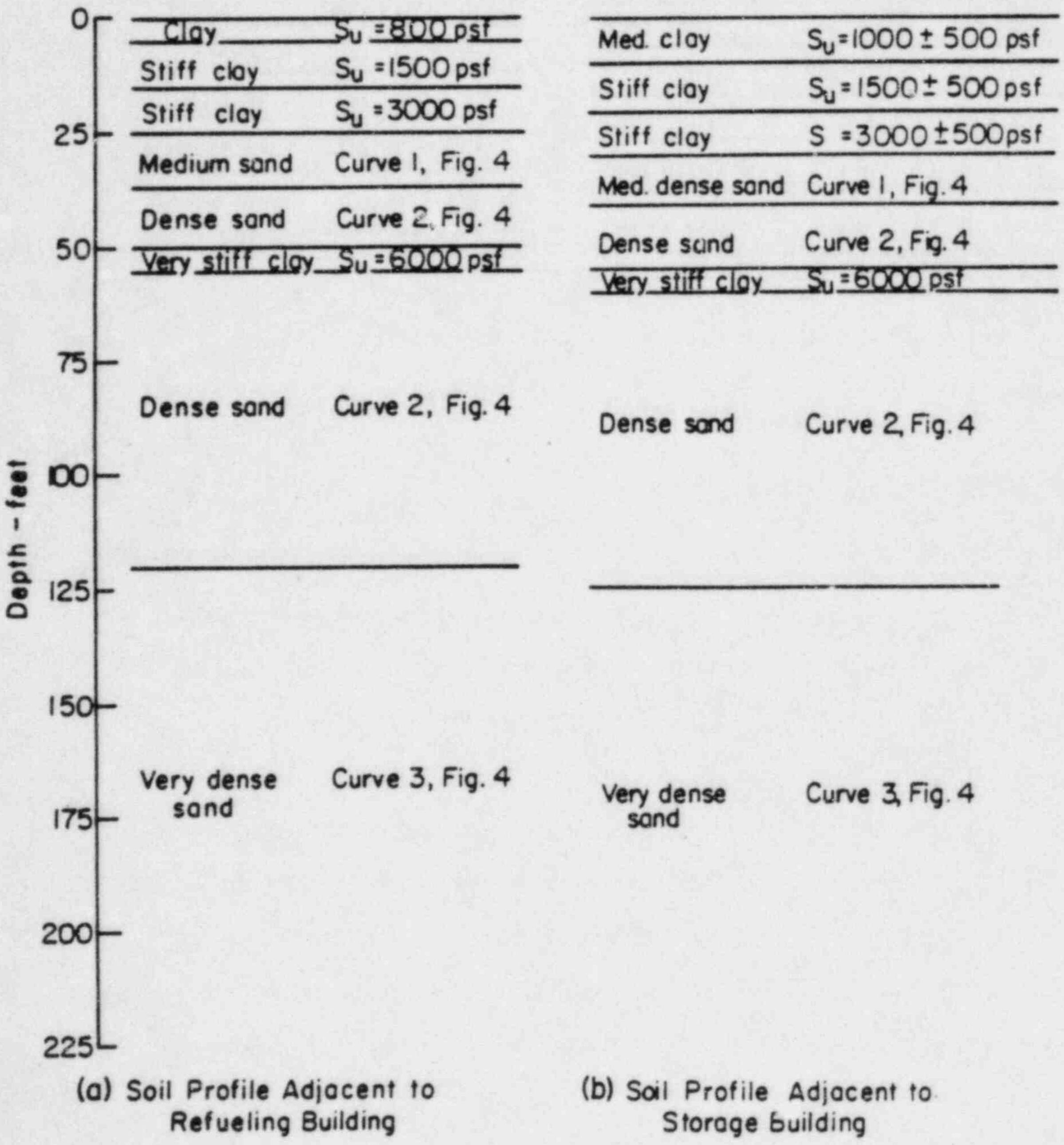

Fig. 3 SOIL PROFILES AT HUMBOLDT BAY POWER PLANT SITE

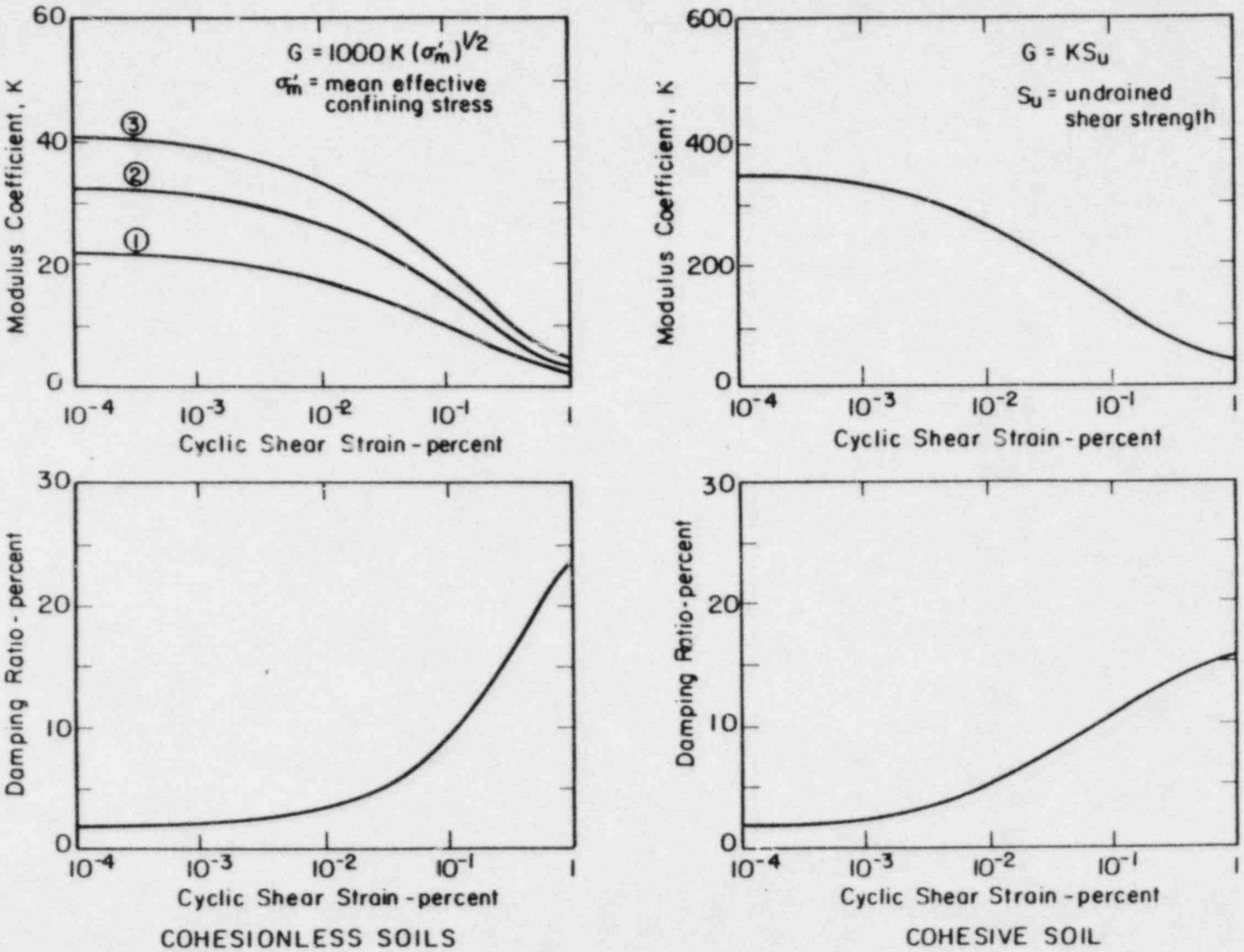

Fig. 4 AVERAGE DYNAMIC SOIL PROPERTIES

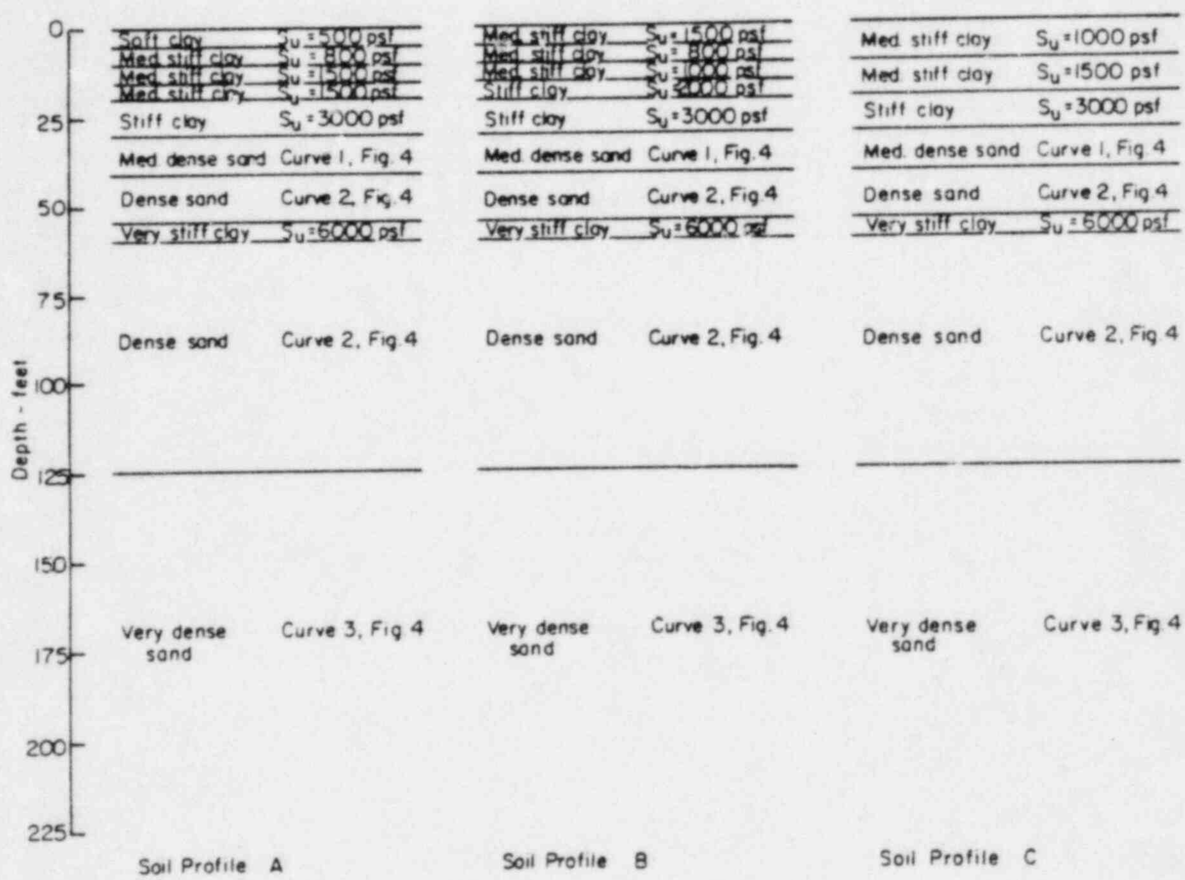

Fig. 5 TYPICAL SOIL PROFILES AT STORAGE BUILDING USED FOR DECONVOLUTION STUDIES

The dynamic shear moduli and damping characteristics of the soils were determined by standard soil testing procedures using resonant column tests and cyclic triaxial tests on undisturbed samples reconsolidated under the in-situ confining pressures. These are shown in Fig. 4. It is pertinent to note that these results were determined and filed with the Nuclear Regulatory Commision before the earthquake of June 7, 1975. At the time the studies were initiated (early 1973) it was not considered necessary to make determinations of field shear wave velocities since it was clear from preliminary studies that shear moduli at moderate to large strains, such as can be determined by strain-controlled cyclic loading triaxial tests, were required for the analysis.

Complete Interaction Analysis Procedure

The general procedure for making a complete interaction analysis (Seed et al, 1974) is illustrated schematically in Fig. 6. The known ground surface motions developed in the free-field are first analyzed by a deconvolution procedure for the soil deposit alone to determine the motions which would have to be developed at a considerable depth below the ground surface (say 150 to 200 ft) in order to produce the actual ground surface motions by transmission of body waves (vertical shear waves) through the soil deposit. This can be accomplished through the use of a computer program such as SHAKE (Schnabel et al, 1972).

These same base motions are then used to analyze the response of a finite element model of the soil-structure system and the results of this latter analysis are checked by ensuring that the required free-field motions are indeed developed in the free field. The basic

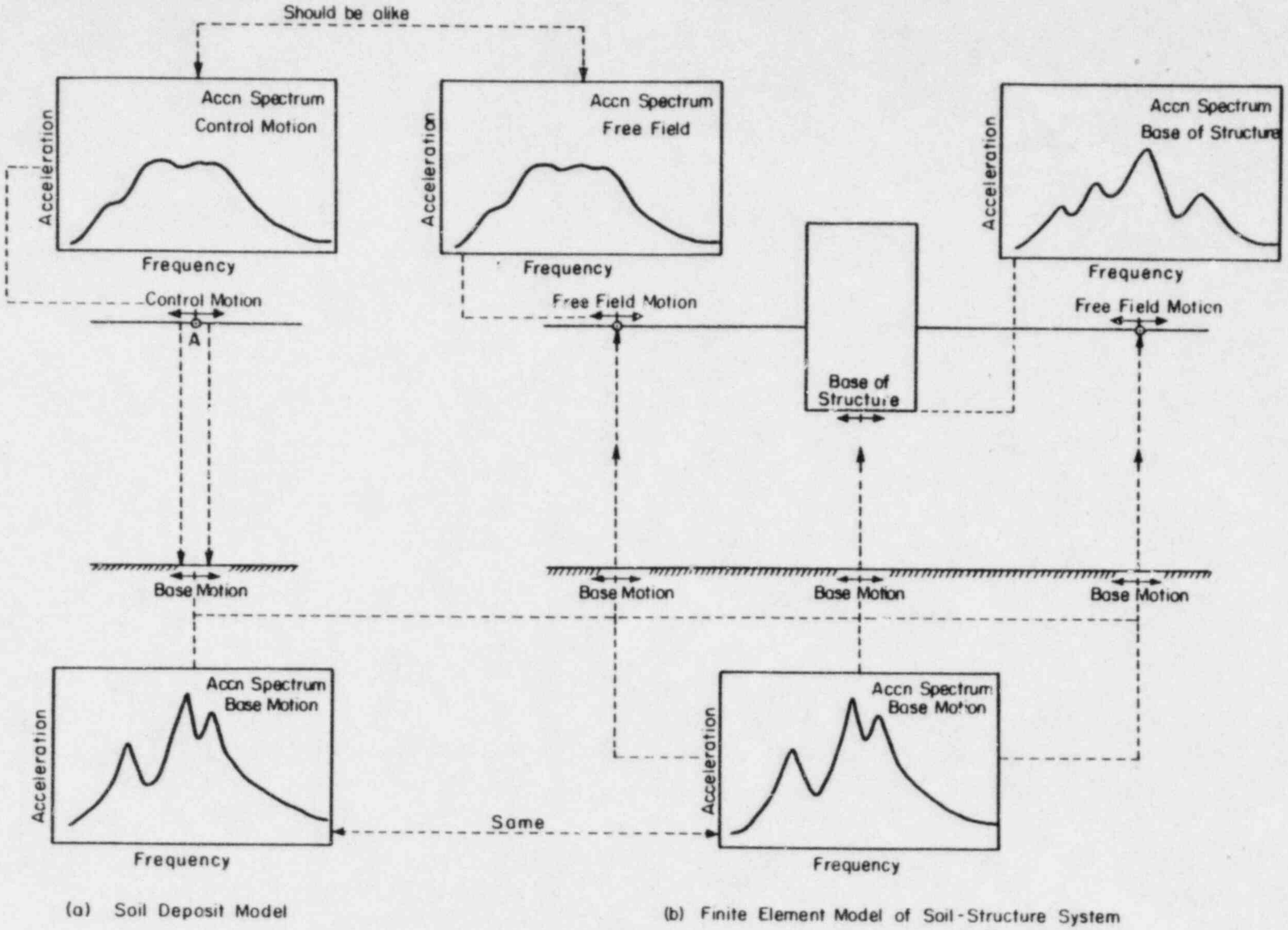

Fig. 6 SCHEMATIC REPRESENTATION OF SOIL-STRUCTURE INTERACTION ANALYSIS USING FINITE ELEMENT MODEL

requirements of a suitable analysis and computer program (Seed et al, 1975b) are that it should be capable of considering

(1) The variation of ground motions with depth,

(2) The three-dimensional nature of the problem,

(3) The effects of adjacent structures on each other where this is appropriate,

(4) The variation of soil characteristics with depth,

and (5) The non-linear stress-strain and energy-absorbing characteristics of the soil.

Results of Pre-Earthquake Analysis

The pre-earthquake studies performed by Dames and Moore were made using the computer programs SHAKE and LUSH (Lysmer et al, 1974). Analyses were carried out for cross-sections in the N-S and E-W directions (Fig. 1) and for various levels of peak ground surface acceleration.

The soil properties shown in Figs. 3(a) and 4 together with the structure characteristics shown in Tables 1, 2 and 3 were assigned to the finite element model. Damping values of 4% and 7% were used for the structures for analyses conducted using peak ground surface accelerations of 0.25g and 0.4g, respectively.

Table 1. Structural Properties of Reactor Caisson

Depth Below Ground Surface, ft	Shear Modulus $\times 10^6$ psf	Density pcf	Poisson's Ratio
0-15	289	0	0.2
15-31	86	0	"
31-44	80	0	"
44-71	76	0	"
71-78	83	0	"
78-87	4160	158	"

Table 2. Masses Lumped at Center Line of Reactor Caisson

Depth Below Ground Surface, ft - - -	0	15	25	37	51	57	71
Weight of Mass (kips) - -	82	82	76	44	43	47	54

Table 3. Structural Properties of Refueling Building

Depth Above Ground Surface, ft	Shear Modulus x 10^6 psf	Density pcf	Poisson's Ratio
0-17.5	5	25	0.2
17.5-35	5	10	0.2

From the results of the initial studies it was found that the effects of the adjacent structures on the response of the buried reactor caisson were relatively minor. Thus the adjacent structures were not included in the finite element model used for the later studies. Since transmitting boundaries are not included in the computer program LUSH it was necessary to use an extensive mesh in the horizontal direction to ensure that the computed response of the Reactor Caisson and Refueling Building was not influenced by the boundary conditions of the analytical model. However previous studies (Hwang, 1973) have shown that it is only necessary to consider the response of the soil deposit to a depth of about one half the structure width below the base of the structure; consequently the base of the analytical model was taken at a depth of 150 ft below the ground surface.

Deconvolution Studies

In performing a deconvolution analysis of a ground surface motion to determine a corresponding base motion for use in a soil-structure interaction analysis, it is often necessary to filter out the high frequency components of the ground surface motion in order to obtain meaningful results. There are two reasons for this requirement:

(1) The specified ground surface motion may contain high frequency components which would not, in reality, be developed

for the site conditions under consideration. This is particularly true for sites consisting of deep (over 250 ft) bodies of soil or including layers of soft to medium stiff clay and sand (Seed et al, 1974).

(2) Deconvolution by a wave propagation analysis using equivalent-linear properties to represent the non-linear stress-strain characteristics of the soil inevitable leads to an excessive amplification with depth of high frequency motions.

In the pre-earthquake deconvolution analyses the acceleration time history shown at the top of Fig. 7 was used as the free-field ground surface motion. The spectra for this time history closely match the NRC design spectra stipulated in Regulatory Guide 1.60. In these studies it was necessary to use a cutoff frequency of 15 to 20 Hz in order to ensure that the accelerations at depth did not become excessive.

Acceleration time histories computed at various depths within the free-field soil profile are also presented in Fig. 7. It may be seen that there is both a decrease in the amplitude of the motion and an increase in the frequency content with an increase in depth within the profile.

Soil-Structure Interaction Analyses

Using the base motions computed at a depth of 150 ft in the deconvolution studies, analyses were then made using the program LUSH and a suitably fine but extensive mesh to compute the response of the soil-structure system. Computations were made for a variety of soil

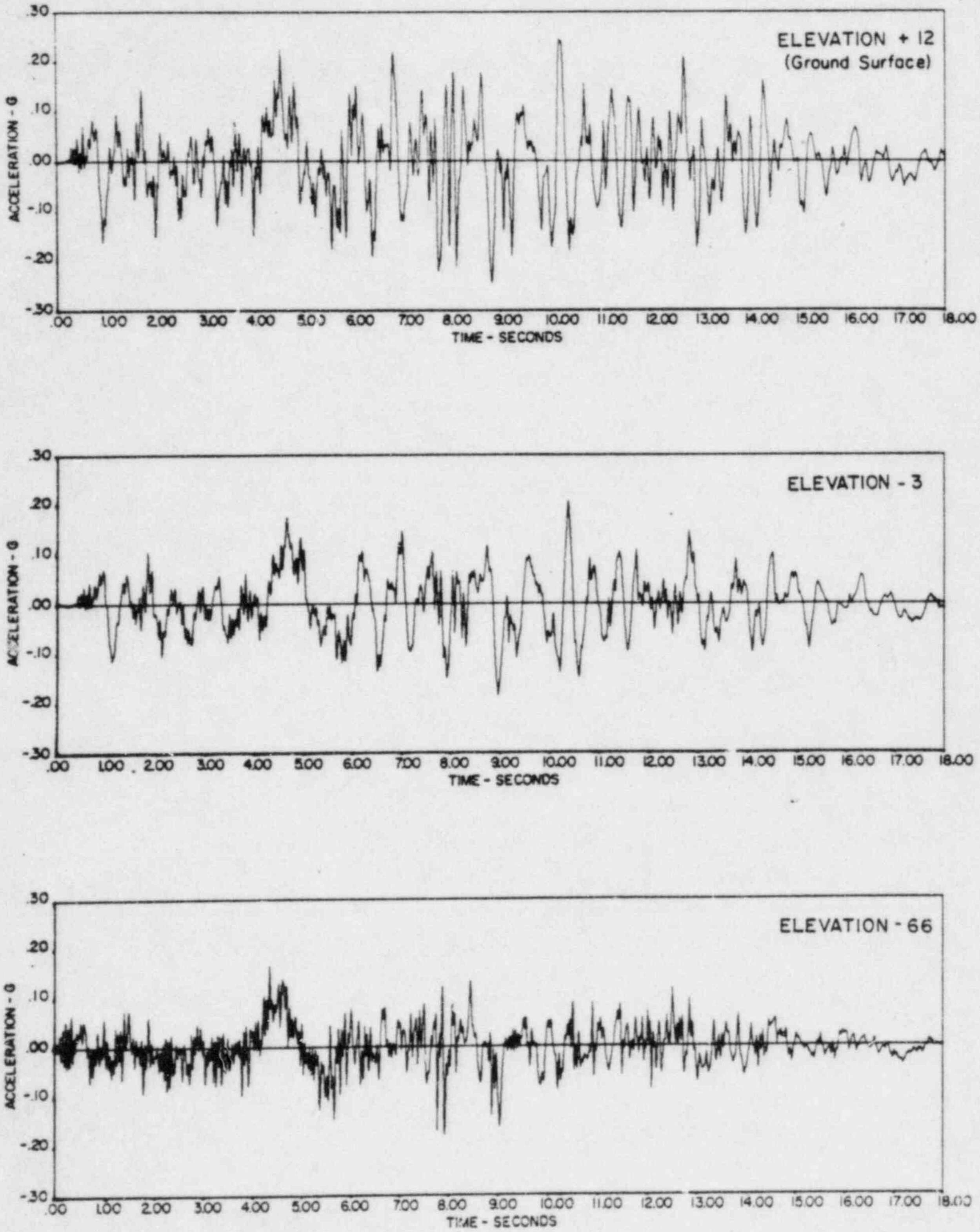

Fig. 7 HORIZONTAL ACCELERATION HISTORIES COMPUTED BY DECONVOLUTION OF SURFACE MOTIONS FOR FREE FIELD CONDITIONS (a_{max} = 0.25g) IN PRE-EARTHQUAKE ANALYSES

properties and envelope spectra for motions at various levels within the structures were finally selected for design, based on the range of computed results supplemented by engineering judgment.

In the course of these studies, analyses were made for ground surface motions having peak accelerations of 0.4g and 0.25g. Since these are in the range of peak accelerations developed in the transverse and longitudinal directions during the June 7 earthquake, it is of interest to compare the values of computed and recorded peak accelerations at instrument locations in the structure. Such a comparison is shown in Table 4. It may be seen that the values show a remarkably high degree of agreement although there is some indication that the actual stiffness of the structure was somewhat less than that used in the analysis. Nevertheless the good agreement in these values is an encouraging aspect of the analytical procedure used in the studies.

Table 4. Comparison of Recorded and Computed Accelerations

Location	Elevation	Max. Accelerations for Recorded Motions		Max. Accelerations for Computed Motions	
		Transverse	Long.		
Free-field (Storage Building)	+12	0.35g	0.26g	0.40g	0.25g
Refueling Building	+12	0.25g	0.20g	0.23g	0.15g
Reactor Caisson	-66	0.16g	0.12g	0.22g	0.13g

Results of Post Earthquake Analyses Using Recorded Motions

Post earthquake studies of soil-structure interaction effects were performed following the same basic procedure as that described

above but using the computer programs SHAKE and FLUSH (Lysmer et al, 1975) since the latter provides a more versatile capability than LUSH and is also more economical. Advantage was taken of the results obtained in the earlier studies and the effects of the adjacent structures were therefore neglected in the analyses. Because the program FLUSH uses transmitting boundaries, it was only necessary to use the finite element mesh shown in Fig. 8 for the soil-structure interaction analyses.

Deconvolution Studies

As stated previously there are valid reasons why some filtering of a given ground surface motion is required in performing a deconvolution analysis to determine motions at various depths. To determine the significance of such effects for the recorded motions at the Humboldt Bay site, deconvolution analyses were made for Soil Profile A in Fig. 5 at the Storage Building site and the recorded surface motions, using filtering or cut-off frequencies of 20, 15 and 12.5 Hz. The results of these studies, in terms of the computed variation of maximum acceleration with depth in the soil profile, are shown in Fig. 9. It may be seen that the cut-off frequency, within the range investigated, had little influence on the results of the analysis, all of the studies for both the longitudinal and transverse recorded motions showing a marked decrease in magnitude of the peak acceleration from the ground surface to a depth of about 30 ft and below. In fact the peak accelerations computed to develop in the free-field at the level of the base of the Refueling Building (about 85 ft) is in the range of 0.10g to 0.14g or less than 60 percent of the maximum acceleration at the ground surface.

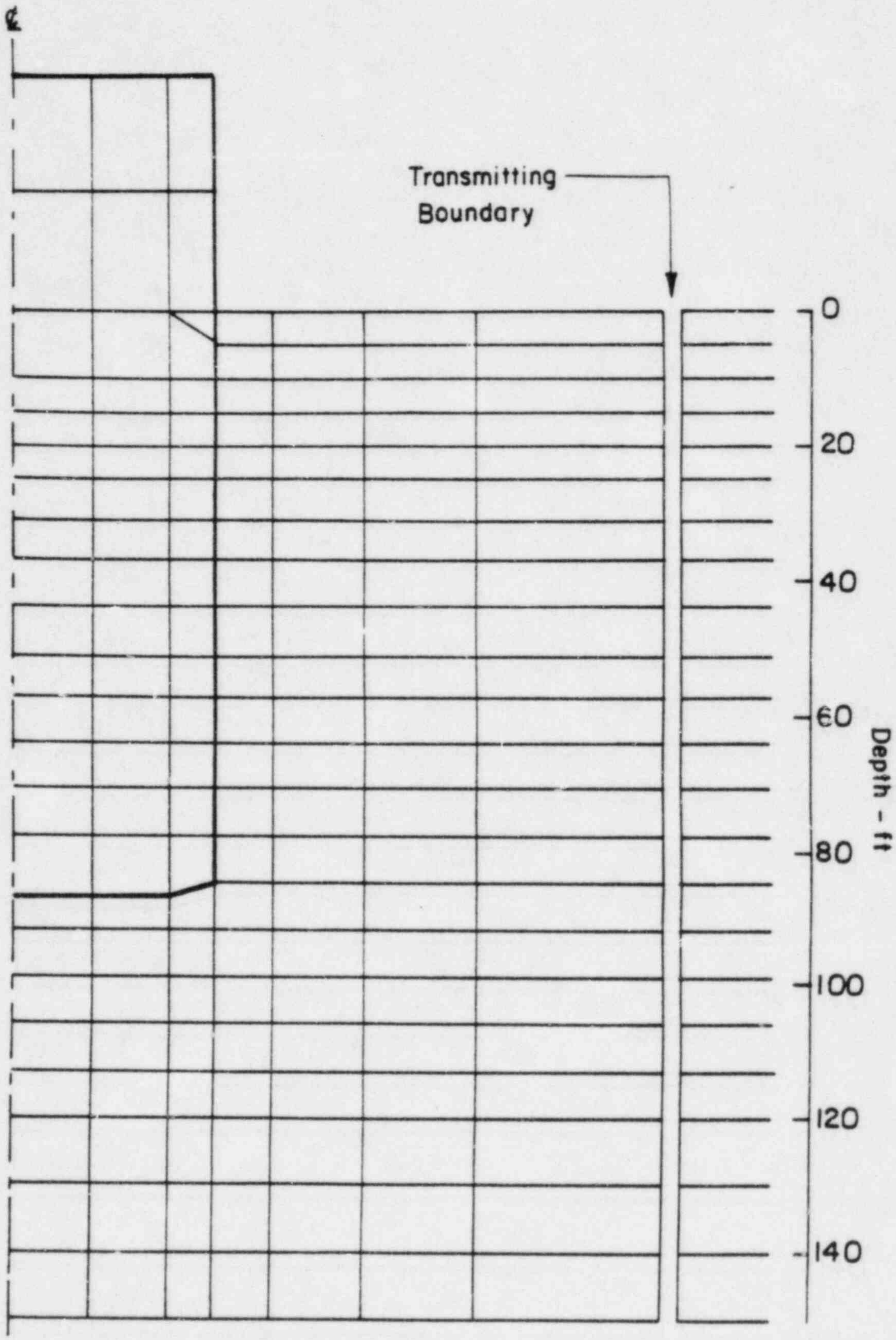

Fig. 8 FINITE ELEMENT MESH USED FOR ANALYSIS

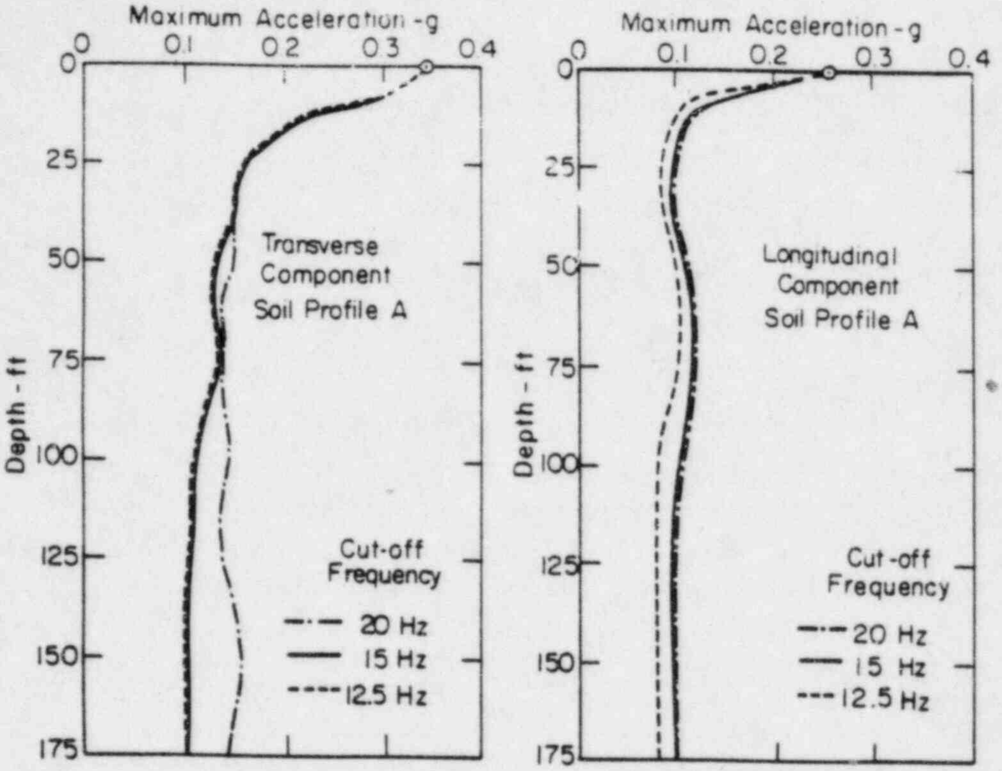

Fig. 9 ACCELERATION DISTRIBUTIONS COMPUTED BY DECONVOLUTION OF RECORDED SURFACE MOTIONS - HUMBOLDT BAY POWER PLANT, JUNE 7, 1975

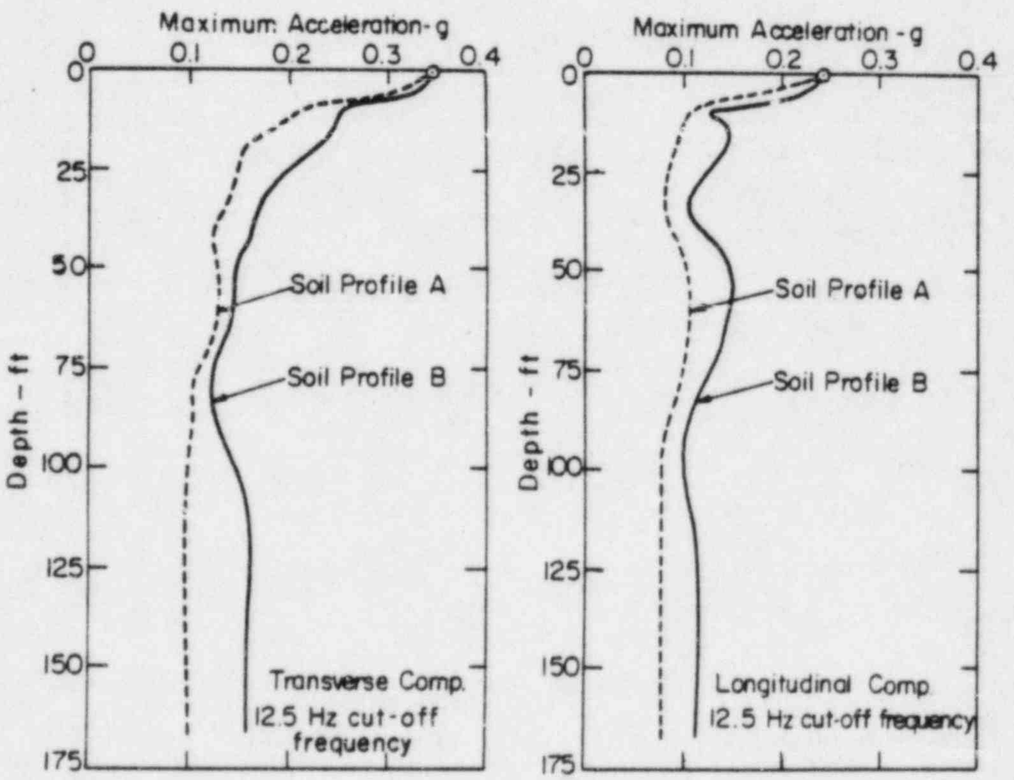

Fig. 10 EFFECT OF SOIL PROFILE ON VARIATION OF MAXIMUM ACCELERATION WITH DEPTH COMPUTED BY DECONVOLUTION OF GROUND SURFACE MOTION RECORDS

It may also be seen from Fig. 10 that generally similar results are obtained whether Soil Profile A or B is used for the analysis. Although they are not shown, results for Soil Profile C fell within the range shown for Soil Profiles A and B. Thus it would seem reasonable to conclude from these results that:

(1) The recorded ground surface motions have no significant content of very high frequencies as might be expected for a deep soil condition such as that at the Humboldt Bay Plant site.

(2) The results of soil-structure interaction analyses made with a cut-off frequency of 12.5 Hz will be comparable to those made using higher cut-off frequencies. Since there is a marked reduction in computer costs associated with the use of a lower cut-off frequency, the soil-structure interaction studies described in the following section were made for these conditions.

Soil-Structure Interaction Studies

Having determined the base motions required in the soil profile at a depth of 150 ft to produce the recorded motions at the ground surface under free field conditions, the same motions were used as excitation at the base of the soil-structure model shown in Fig. 8 to compute the motions developed (1) at the base of the structure and (2) in the structure at the level of the ground surface, where motions were recorded during the earthquake of June 7. Separate analyses were made for the longitudinal and transverse records of free-field motion and for the various soil profiles. The ranges of analytical results

are presented in Fig. 11 in the form of response spectra, where they are also compared with the spectra for the recorded motions.

It may be seen that for both longitudinal and transverse motions, the recorded motions at the base of the structure are in reasonably good agreement with those computed using the finite element procedure for implementation of an 'idealized' complete interaction analysis. For both components of motion the analysis procedure indicates a higher peak in the response spectrum at a frequency of about 3 Hz than actually developed, but considered overall, the agreement between computed and recorded base motion spectra is both gratifying and encouraging.

Similarly the recorded motions in the structure at ground level fall essentially within the range computed by the interaction analysis procedure, providing further confirmation of the ability of a complete interaction analysis to compute the structural response with an adequate degree of accuracy in this case.

It is recognized, of course, that one such test of the applicability of any analytical procedure does not necessarily provide proof that it will always lead to good evaluations of field performance. Nevertheless in the current absence of any other opportunity to check analytical methods for computing response under strong shaking of prototype structures, the results obtained in even this single case can give designers increased confidence in the usefulness of the analytical tools at their disposal.

Applicability of NRC Design Procedure

In addition to their use for checking the adequacy of procedures

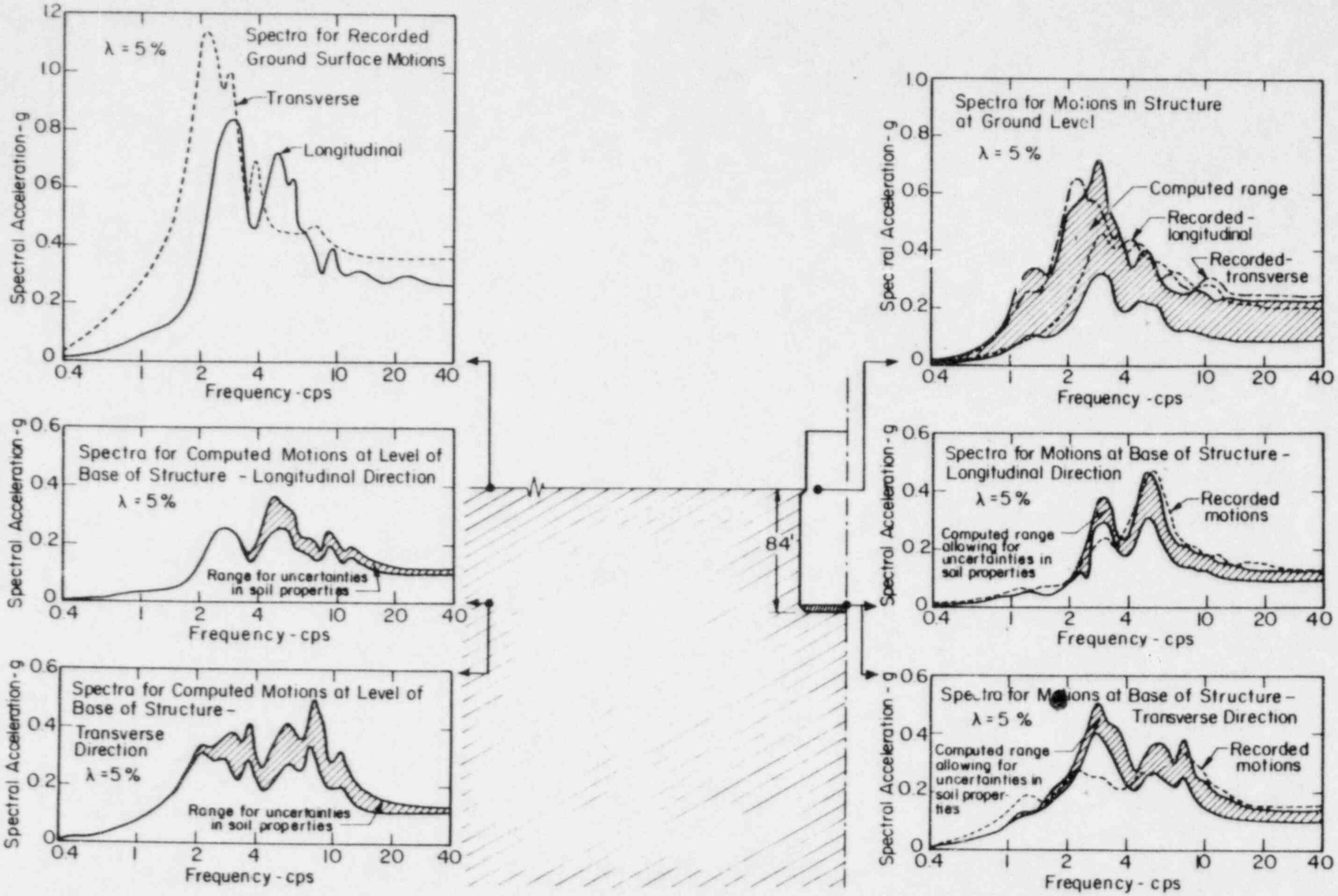

Fig. 11 COMPARISON OF RECORDED AND COMPUTED SPECTRA IN REFUELING BUILDING - HUMBOLDT BAY POWER PLANT

for analyzing soil-structure interaction, the records obtained at the Humboldt Bay Power Plant can also be used to investigate the adequacy of required design practice. At the present time, regulatory requirements for determining soil-structure interaction effects for embedded structures such as the Refueling Building require the specification of a design or control motion at the ground surface having a designated maximum acceleration and a time-history whose spectrum closely matches a standard design spectrum shape specified by the Nuclear Regulatory Commission. Since the average peak acceleration recorded in the free-field at the Humboldt Bay plant was 0.3g, it would seem reasonable to compare the motions recorded at the base of the Refueling Building with those computed following an approved design procedure consistent with a peak free-field ground surface acceleration of 0.3g and the standard design spectrum shape. This is, in fact, the motion whose spectral shape is shown in the upper left corner of Fig. 12. An acceleration time history having this spectrum and having a duration of about 16 seconds was used in the following analyses.

Regulatory practice permits the deconvolution of this motion and the analysis of soil-structure interaction effects using finite element methods as previously described but it also requires:

1. that analyses be made for the most likely values of soil moduli and for values of soil moduli which are increased and reduced by a factor of 1.5 to allow for possible uncertainties in soil property determinations;
2. that the envelope of the resulting spectra for motions computed for a point in the free-field at the level of the base of the structure should be not less than 60 percent

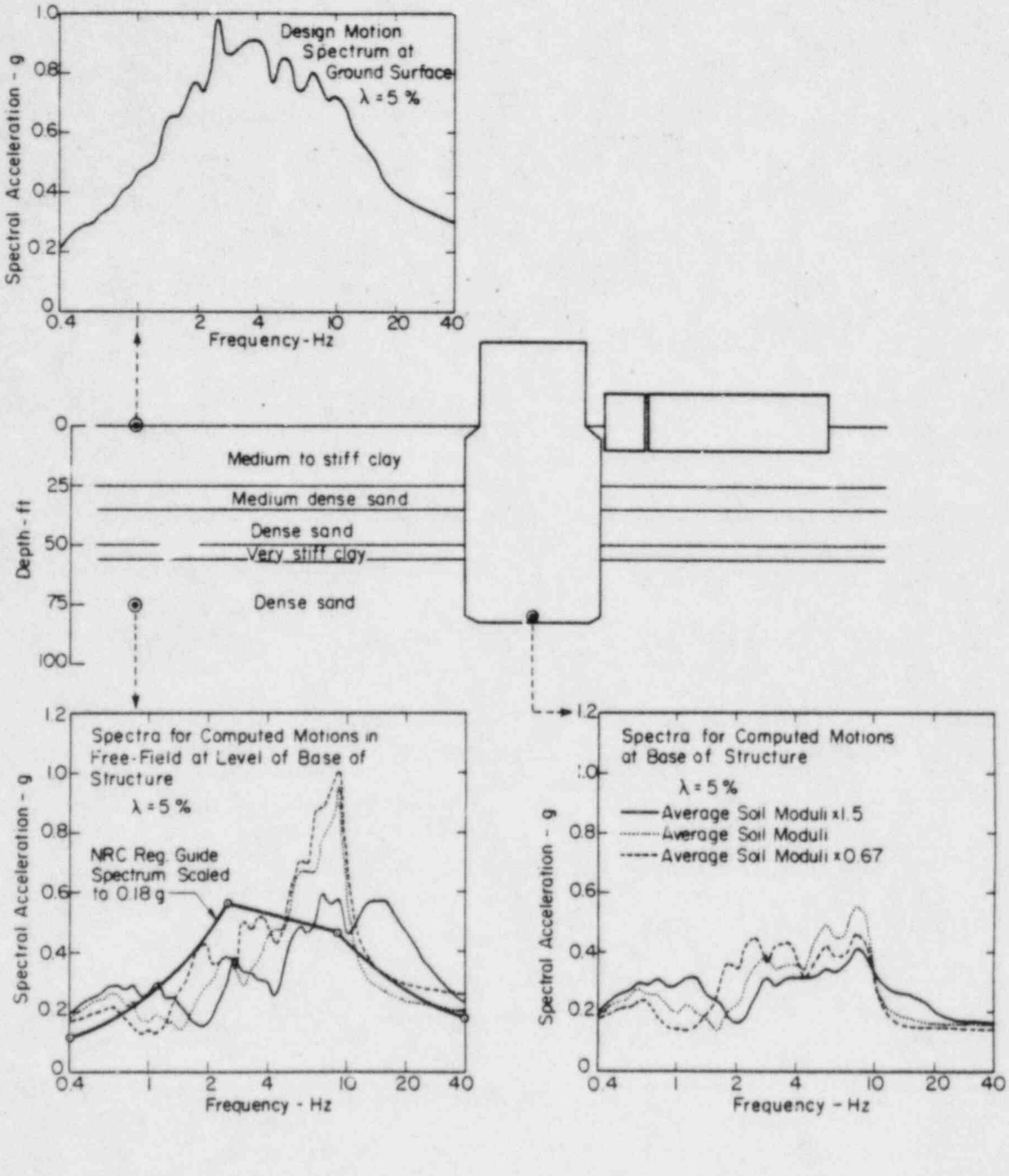

Fig. 12 SUMMARY OF SPECTRA FOR COMPUTED MOTIONS IN FREE-FIELD AND AT BASE OF STRUCTURE--MAXIMUM GROUND SURFACE ACCELERATION = 0.3g

of the spectral accelerations for the ground surface control motion;

and 3. that the structural response be evaluated for motions having a spectral shape enveloping those computed at the base of the structure for free field motions meeting the requirements of (1) and (2) above.

A typical set of calculations for the same ground surface control motion but for the three different values of soil moduli are shown in Fig. 12. In this figure the control motion is shown in the upper left hand corner, the spectra for the computed motions in the free field at the level of the base of the structure are shown in the lower left hand corner and the spectra for the computed motions at the base of the structure are shown on the lower right hand corner. For the analysis conducted with the most likely values of soil moduli and the reduced soil moduli, the control motion was filtered at 10 Hz while for the analysis with increased soil moduli, the control motion was filtered at 20 Hz. The envelope of the computed spectra for the motions at the base of the structure is compared with the motions recorded at the base of the structure in Fig. 13.

It may be seen that although the free-field motions fail to meet the NRC design spectral acceleration requirements in the frequency range from about 2 to 5 Hz, the envelope spectrum for the computed motions at the base of the structure is nevertheless higher than the spectra for the recorded base motions at all frequencies. In fact only at frequencies of about 4.5 to 5.5 Hz does the spectrum for the recorded motions come close to that for the computed base motion envelope spectrum.

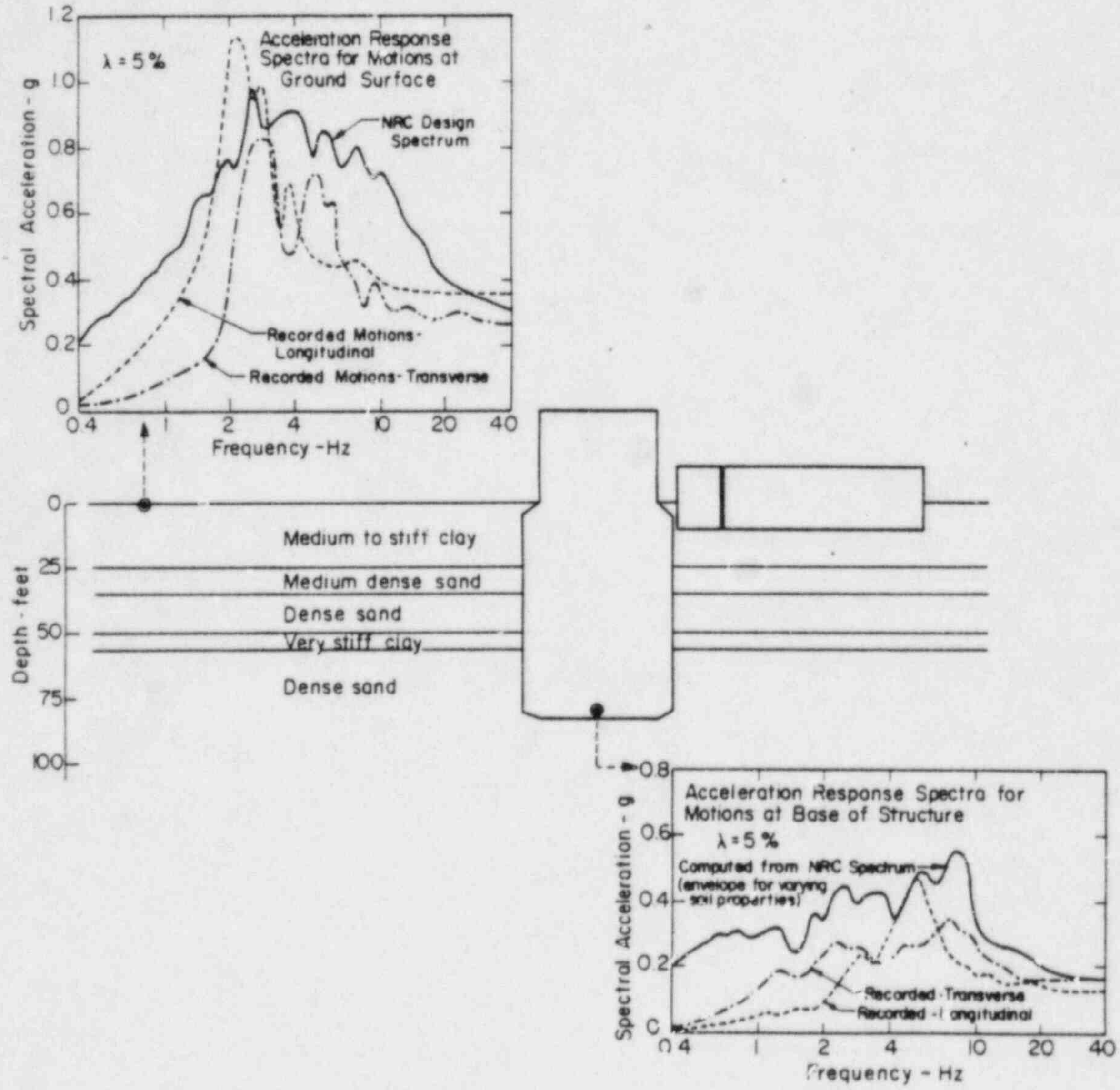

Fig. 13 COMPARISON OF SPECTRA FOR DESIGN AND RECORDED MOTIONS

One means of increasing the free field spectra to meet the 60% of surface control motion requirement, is to increase the ground surface acceleration for the control motion for one or more of the analyses so that after deconvolution it meets the free field requirements. In the present case, this could be achieved by increasing the control motion for the analysis performed using the reduced values of soil moduli by 30 percent. With a satisfactory degree of accuracy, this leads to corresponding increases of 30 percent in both the free field spectrum at a depth of 85 ft and the spectrum for motions at the base of the structure.

The superimposed spectra for the three analyses with this modification are shown in Fig. 14 and the envelope of the spectra for computed motions at the base of the structure is compared with the spectra for the motions recorded at the base of the structure in Fig. 15. It may be seen from Fig. 14 that the envelope of free-field spectra now comes very close to meeting the design spectral requirements at this location; thus the envelope of spectra for motions developed at the base of the structure as shown in Fig. 15 would be essentially acceptable for design purposes. This envelope provides a comfortable margin of safety above the spectra for the recorded base motions and would seem to indicate that, at least for these strong motion records, the current design requirements provide an adequate but not excessively conservative margin of safety for analyses conducted in the manner described above.

Similar studies for other methods of evaluating soil-structure interaction effects would presumably throw some light on the degree of conservatism or unconservatism which they introduce into the design procedure.

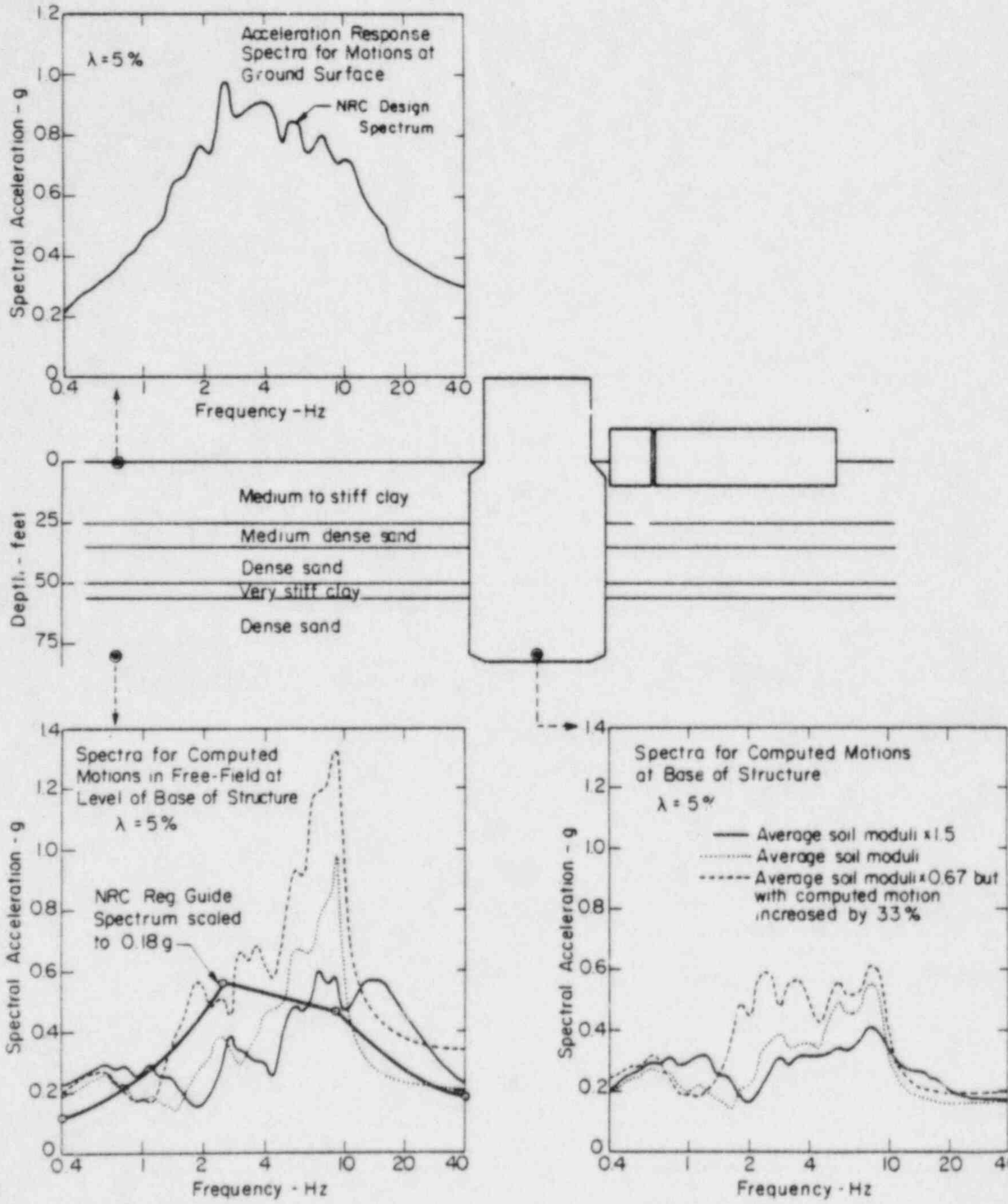

Fig. 14 SUMMARY OF SPECTRA FOR COMPUTED MOTIONS IN FREE-FIELD AND AT BASE OF STRUCTURE USING NRC DESIGN PROCEDURES

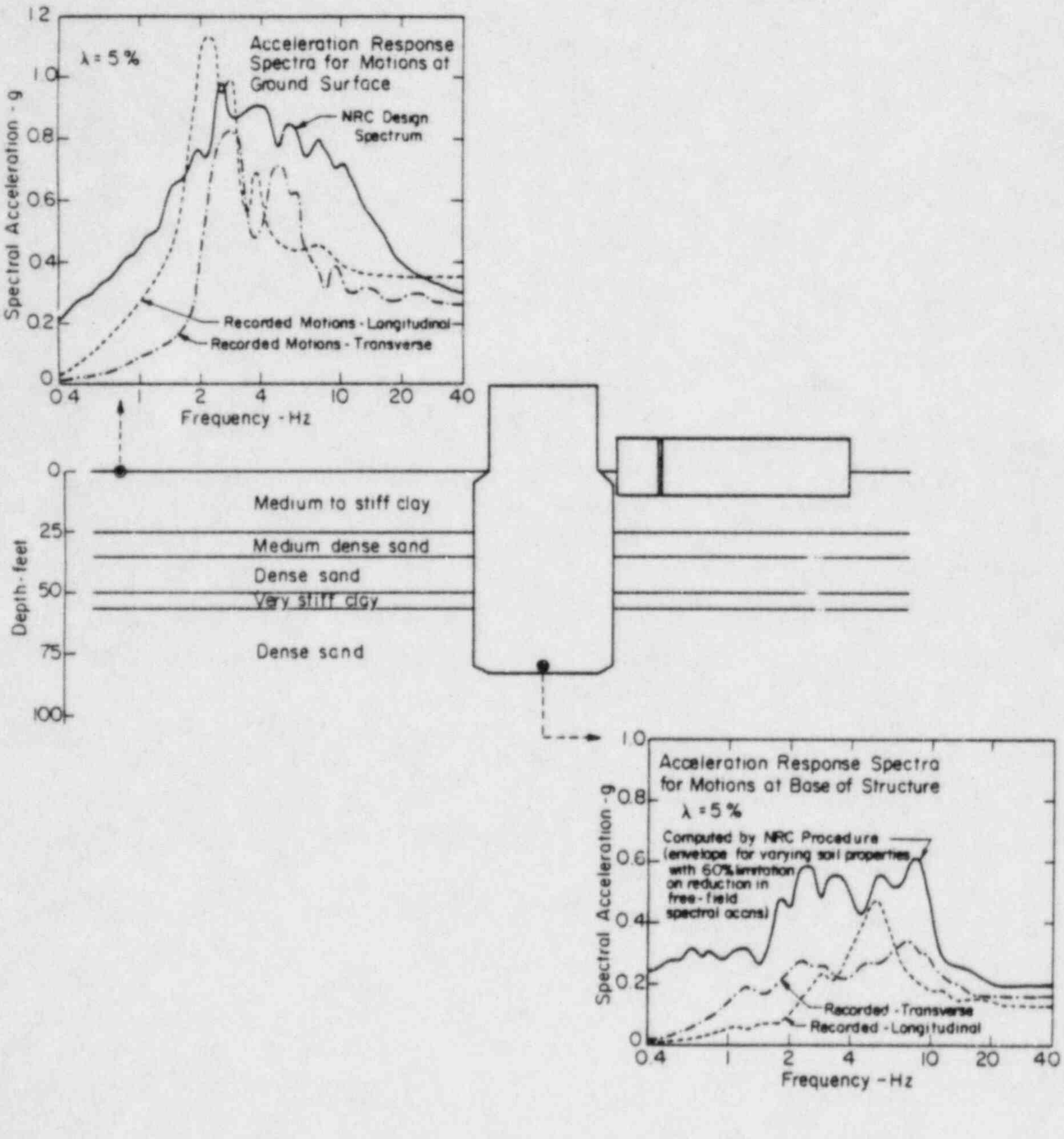

Fig. 15 COMPARISON OF SPECTRA FOR DESIGN AND RECORDED MOTIONS AT BASE OF STRUCTURE

Conclusions

The preceding pages present the results of a study of the distribution of ground motions and structural response in the Humboldt Bay Nuclear Power Station during the Ferndale earthquake of June 7, 1975. Based on a knowledge of the motions developed at the ground surface in the free-field, computations are made using an idealized complete interaction procedure based on finite element analysis, to determine the characteristics of the motions likely to develop at the base of the Refueling Building at a depth of 85 ft below the ground surface and within the Refueling Building at the ground surface level. The computed motions are shown to be in reasonably good agreement with those recorded at these locations in the same earthquake. In addition, the recorded motions are compared with those computed by an analysis procedure which generally meets existing regulatory requirements and it is shown that the regulatory requirements lead to an entirely adequate but not excessively conservative margin of safety based on the motions recorded in this event.

It is of interest to note that Lambe (1973) has recently made a study of the accuracy of engineering predictions of soil behavior under static loading conditions. For this purpose he classified predictions into five groups as follows:

Type A	Prediction made before the event
Type B	Prediction made during the event but before the results are known
Type B1	Prediction made during the event but with results known at the time
Type C	Prediction made after the event but before the results are known

Type C1 Prediction made after the event but with results known at the time.

He concluded that "Type C predictions are autopsies....Our professional literature contains the results of more Type C1 predictions than any other type. Autopsies can of course be very helpful in contributing to our knowledge. However one must be suspicious when an author uses a Type C1 prediction to "prove" that any prediction technique is correct". Lambe also concluded that predicted results within a factor of two of observed field performance constitute very good predictions. It would seem optimistic to expect any better success in predicting dynamic behavior of soil or soil-structure systems.

However the prediction of the base motion peak accelerations shown in Table 4, based on the assumption that the ground surface motions with peak accelerations of 0.25g and 0.40g in the free field, was clearly a class A prediction using Lambe's terminology, in that the report describing this study by means of an idealized complete interaction analysis using finite element techniques was submitted to the Nuclear Regulatory Commission before the event of June 7, 1975 occurred; nevertheless the degree of similarity between peak acceleration values assumed and developed in the free field and those predicted and developed at the base of the Reactor Caisson would seem to show that the prediction was highly satisfactory.

Similarly although the more detailed analyses described in the preceding pages using the same general procedure were made after the event, it might reasonably be claimed that they represent a class A prediction since they permitted virtually no latitude for manipulation of the results in that they were based on:

1. A method of analysis developed prior to the event.
2. Soil properties established and filed with the Nuclear Regulatory Commission prior to the event.

and 3. Fixed surface motions established by the event.

Nevertheless the authors would be the first to agree that the good agreement in this one case between predicted and developed motions at the base of the structure does not necessarily prove the adequacy of the method of analysis used for all cases. Clearly compensating errors might be involved whose effects have not been fully appreciated. On the other hand, it is an encouraging start and the results obtained clearly give some degree of support to the method used. They might also in due course give an equal degree of support to other methods which might be used for analyzing soil-structure interaction effects. These are significant facts in a field where no other data exists by which the adequacy of analytical procedures can be checked. At the same time it is clear that any method of analysis which provides a poor prediction of the results obtained, based on the known values of soil and structural properties and the motions recorded at the ground surface must be considered of dubious validity for future predictions of probable building response.

Acknowledgements

The study described in the preceding pages was conducted as part of an investigation on "Analysis of Soil-Structure Interaction Effects for Massive Embedded Structures During Earthquakes" sponsored by the National Science Foundation. The support of the Foundation and of the Pacific Gas and Electric Company in providing the basic data required for the study and encouraging the investigation is gratefully acknowledged.

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Part VII

CONCLUSIONS

1. A good estimate of the likely site response is the most important part of any soil-structure interaction analysis.
2. Site properties should be considered in deciding on the design control motion to be used in the analysis.
3. If a control point is specified, it should be at the ground surface or at an imaginary outcrop.
4. Since ground motions vary spatially in soil and rock deposits, for embedded structures these variations must be taken into account in soil structure interaction studies.
5. In soil deposits:
   (a) Fundamental mode Rayleigh and Love waves with frequencies higher than 1 Hz damp out rapidly and make no significant contribution to the near surface motions.
   (b) Higher mode surface waves can be represented by inclined body waves.
   (c) Near surface motions are essentially the same whether they are due to inclined or vertically propagating waves. This being the case we might just as well analyze structural response for vertically propagating waves.
   (d) Vertically propagating waves (or even Rayleigh waves) produce vertical variations in horizontal ground motion with a definite pattern determined by the presence of the ground surface discontinuity. Except for unusual soil conditions, the amplitude of the motions will decrease with depth in the upper 70 ft or so and at any depth there will be a definite range of frequencies which will be suppressed.

(e) The general nature of the variations in ground motions with depth can be determined by wave propagation analyses and the variations incorporated into soil-structure interaction analyses performed by finite element methods or, in the case of rigid cylindrical basements, by continuum methods.

(f) For embedded structures analyses which do not take the variation of motions with depth into account will inevitably give incorrect results.

(g) Analyses in which the motions are represented by vertically propagating S and P waves are more conservative than analyses in which the motions are represented by Rayleigh waves.

(h) In view of all the above, analyses using vertical wave propagation provide a perfectly reasonable means of evaluating soil-structure interaction effects.

6. For rock sites

(a) The ground motions may consist of any combination of wave types and at present the appropriate combination is unknown.

(b) The extreme possibilities are that

(1) The motions are all due to vertically propagating waves.

(2) The motions are all due to Rayleigh waves.

Analyses for these extreme possibilities give results which are quite similar.

(c) In any given situation it is unlikely that Rayleigh waves dominate the seismic environment.

(d) In view of conclusions (b) and (c), methods of analysis which assume vertically propagating waves will produce results which are sufficiently accurate for practical purposes. Furthermore, such analyses are simpler to perform.

(e) In any case, the effects of soil-structure interaction for structures on rock are very small and for practical purposes can be neglected in design studies.

7. The only wave type we have not discussed in detail is Love waves. For reasons similar to those used to explain Rayleigh wave behavior the effects of Love waves will be negligible in soil deposits. However in rock formations they may cause increased torsional response and this is the one remaining aspect of soil-structure interaction which has not yet been totally clarified. The effects of Love waves can currently be analyzed only for rigid surface foundations of arbitrary shape. However, since the actual composition of the incoming waves is not currently known, present design procedures of allowing a given degree of eccentricity to the system to allow for torsional input to the base seem to be appropriate for handling this problem.

8. Many current methods of analysis make the assumption that the embedded part of the structure is rigid. The appropriateness of the assumption has not yet been verified.

9. Methods are currently being developed which can handle the general linear three-dimensional interaction problem for embedded flexible basements on horizontally layered sites. These methods can be combined with an equivalent linear analysis of the corresponding site response problem.

10. In view of the many approximations which must necessarily be made in order to perform a soil-structure interaction analysis, it is imperative that the results obtained by any proposed method be checked against full-scale field observations. At the present time, this means that analytical results must be checked against the motions observed in the Humbolt Bay Power Plant during the Fernadale Earthquake of June 7, 1975.

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