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

A REVIEW OF SOIL-STRUCTURE INTERACTION

J. M. Roesset
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Austin, Texas

June 1980

Lawrence Livermore Laboratory

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CONTENTS

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

The effect of the flexibility of the soil on the dynamic response of structures, particularly when subjected to seismic excitation, has been a subject of considerable interest and research in the last ten years. This effect is important when dealing with very stiff and massive structures, such as nuclear power plants, and relatively soft soils. A number of sophisticated mathematical techniques, elaborate computer codes, and simpler engineering approximations have been developed to account for soil flexibility in a single step, considering the combined soil-structure system, or in three separate steps, using substructuring techniques.

When applied to a linear problem, both approaches can and should give the same results if properly implemented. In practice, differences in the results arise from the details of the soil and structural models, from variations in the simplifying assumptions, and from the numerical procedures used for the solution. While the advantages and disadvantages of these two approaches and the conditions under which they will give equivalent and sensible results have been discussed by various authors, some controversy persists regarding the validity or adequacy of each method. Comparative studies, intended often to show the superiority of one of the approaches, have in the past indicated some confusion as to the correct way to apply the other. The situation may have been further aggravated by the larger emphasis recently placed on the development of computer codes, rather than the derivation of simplified procedures, comprehensive studies which would shed more light on the importance of various approximations, or adequate justification of the numerical procedures used. Moreover, when studies of this type have been conducted, they have often been more concerned with the evaluation of specific computer programs (not always properly used) than with the investigations of the methodologies.

This report discusses further the available mathematical techniques, their advantages and limitations, and the common aspects and important

features of the solution. It is important to keep in mind that numerous uncertainties are present in the various phases of the analysis and that no single procedure or computer program will be able to generate deterministically exact answers to the true physical problem. A considerable amount of engineering judgment will always be necessary to estimate key parameters and to select an appropriate model. The most one can reasonably expect is results which are physically logical and within the range of acceptable engineering accuracy. Explicit recognition of the fact that no solution can actually be labeled as exact would probably help in the controversy over appropriate methods. It is clear, on the other hand, that several aspects of the solution are now well understood and that any method should be able to reproduce them at least approximately. The existence of uncertainties should not be allowed to negate this knowledge, to obscure the need for further research, or to inhibit the use of improved methodologies.

Some of the basic features of the soil-structure interaction problem are discussed in Chapter 2, using a very simple model. Definitions are also presented for some of the terms used extensively in the following chapters.

In Chapter 3, the methods of analysis and models most commonly used in practice today are discussed. These models can be classified in many different ways depending on the criteria used: continuum versus discrete formulations, solutions in the time or frequency domain, etc. In this work, they have been classified in two general categories: a direct approach in which the complete soil-structure system is analyzed and a substructure approach in which the solution is carried out in several separate steps. While this classification is somewhat arbitrary, it represents a very clear distinction in the philosophy of the solution.

While it is always desirable to obtain a solution which is as accurate as possible, there are a number of possible simplifications which can be introduced with little loss in accuracy. Approximate methods can provide reasonable solutions at great savings in time and computational effort, but even more importantly, they allow one to obtain preliminary estimates, to assess the effects of variations in different parameters, to identify the key variables and to check the results of more complex analyses. Some approximations are discussed in Chapter 4.

In Chapter 5, the sensitivity of the results to various models and assumptions is discussed in the context of the uncertainties involved. The number of serious parametric and comparative studies which have been conducted to date is still limited. As a result, while use has been made of published results to arrive at the figures quoted in this chapter, the figures must be considered as engineering estimates, based to a large degree on personal judgment.

Research needs are briefly discussed in Chapter 6. There is a need on the one hand for additional basic research to improve the definition of the design earthquake, the determination of soil properties in situ, and the mathematical modeling of the soil behavior; there is a need on the other hand for improved computational techniques and further evaluation of some of the models. There is also a need for reliable and properly validated approximate methods which can be used to verify results of other analyses.

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CHAPTER 2

DEFINITION OF THE PROBLEM

For many years, the seismic analysis of structures was performed assuming that the design earthquake could be applied directly at the base of the building, and that it was the same motion that would occur at the free surface of the soil before the structure was built. However, it is now recognized that this assumption would be correct, from a theoretical point of view, only if the building were founded on rigid rock, and for practical purposes, if it were founded on a very stiff, rocklike material. The properties of the underlying soil affects the motions to which the building is subjected in three different ways:

- Even before the structure is built, the motions at the free surface of the soil deposit are influenced to some degree by the characteristics of the soil profile.
- Once a stiff foundation is placed on the surface, waves traveling horizontally will be filtered, giving rise to rotational components of motion. If the foundation is embedded, these effects occur even for vertically propagating waves, because of the geometry of the excavation.
- Once the structure is built, its own vibrations give rise to a base shear and overturning moment which produce additional deformation of the soil, and therefore modify, the foundation motions.

The first point is often referred to as the soil amplification problem, a name which is somewhat misleading, since both amplifications and deamplifications of the motions may occur, depending on the frequency range. A considerable amount of work has been done on this problem, although it remains a controversial subject. From the point of view of this report, this problem is of interest when site-specific spectra are sought (to account for local soil conditions very different from what would be termed an average firm

ground), or when it is desired to obtain compatible motions at various levels in the free field. In addition, amplification studies are conducted to estimate soil properties consistent with the level of earthquake motions in order to account for nonlinear soil behavior.

The second point is sometimes referred to as the wave scattering or the kinematic interaction problem. The first name would suggest that it is not a part of the soil-structure interaction process, while the second makes it an integral part of it. A discussion of the merits of different names serves no useful purpose. The important fact is that this effect can be ignored in soil-structure analyses with an earthquake excitation, only when dealing with a surface foundation and vertically propagating waves.

The third point corresponds to what normally would be called the soil-structure interaction problem, as defined for cases where the dynamic excitation is directly applied to the structure (design of machine foundations, offshore structures, etc.). It is sometimes referred to as the inertial interaction problem to distinguish it from the second type of effect.

To illustrate the basic features of the soil-structure interaction phenomenon, it is convenient to look at a very simple, single-degree-of-freedom system, representing a structure founded on the surface of a soil deposit. It will be assumed that the seismic excitation is caused by vertically propagating shear or dilatational waves. In this way, only the third type of effect must be considered. By looking next at the same single-degree-of-freedom system, representing a structure with an embedded foundation, the characteristics of the second type of effect can be discussed.

2.1 STRUCTURE ON A SURFACE FOUNDATION

The model considered here consists of a single mass M, lumped at a height h above the base; a spring with a constant k, representing the stiffness of the structure; and a set of springs and dashpots to simulate the flexibility of the soil. This simple model (Fig. 2-1) has been used by several researchers (Parmelee, Sarrazin, Bielak, Veletsos) to study the effect of soil-structure interaction.

FIG. 2-1. Simple single-degree-of-freedom system.

For the case of a horizontal excitation,

$$u = u_G + u_o + y + h\phi_o \quad ,$$

$$ky = k_x u_o \quad ,$$

$$khy = k_\phi \phi_o \quad ,$$

where k_x is the horizontal spring representing the foundation translational stiffness, k_ϕ is the corresponding rocking spring, u is the absolute displacement of the mass, y is the distortion of the structural spring, u_o and ϕ_o are the deformations of the foundation springs, and u_G is the ground displacement in the free field (before the structure is built).

The equation of motion for the mass is

$$M\ddot{u} + ky = 0 \quad ,$$

which can be written as

$$M\left(1 + \frac{k}{k_x} + \frac{kh^2}{k_\phi}\right)\ddot{y} + \kappa y = -M\ddot{u}_G \quad .$$

The natural circular frequency of the structure on a rigid base (without soil-structure interaction) would be

$$\omega_o = \sqrt{\frac{k}{M}} \quad .$$

Accounting for the flexibility of the foundation, the frequency becomes

$$\omega = \frac{\omega_o}{\sqrt{1 + \frac{k}{k_x} + \frac{kh^2}{k_\phi}}} \quad .$$

As could be expected, the deformability of the soil results in a decrease in the effective natural frequency, indicating that the system is more flexible. The magnitude of this change is a function of the relative stiffness of the structure with respect to the soil, as indicated by the terms k/k_x and kh^2/k_ϕ. If the term k/k_x is the important one, then the interaction effect is associated mainly with the horizontal translations of the base (caused by the base shear); on the other hand, if the main contribution comes from the term kh^2/k_ϕ, it will be the base rotation (resulting from the base overturning moment) that will represent the main interaction effect.

Assuming (1) that the structure has an internal damping D_{st} which is frequency independent, i.e, of a hysteretic type, (2) that the soil has an internal material damping (also hysteretic) D_s, and (3) that viscous dashpots c_x and c_ϕ are associated with the foundation springs k_x and k_ϕ (to reproduce the loss of energy by radiation of waves away from the structure), then the effective damping D of the simple system at its natural frequency ω is given approximately by

$$D = D_{st}\left(\frac{\omega}{\omega_o}\right)^2 + D_s\left[1 - \left(\frac{\omega}{\omega_o}\right)^2\right] + \left(\frac{\omega}{\omega_o}\right)^2\left[\frac{k}{k_x}\frac{\omega c_x}{2k_x} + \frac{kh^2}{k_\phi}\frac{\omega c_\phi}{2k_\phi}\right] .$$

When the internal soil damping D_s is smaller than the structural damping D_{st}, the effective damping of the soil-structure system may be smaller than that of the structure on a rigid base (D_{st}). This is, however, a rather unusual situation. In most cases, D_s is at least equal to D_{st}, and the interaction effect produces an increase in the effective damping. The amount of the increase depends on the magnitude of the last term, representing the contribution of the radiation damping.

For the case of a vertical excitation,

$$v = v_G + v_o + w \quad ,$$

$$kw = k_z v_o \quad ,$$

$$M\ddot{v} + kw = 0 \quad ,$$

$$M\left(1 + \frac{k}{k_z}\right)\ddot{w} + kw = -M\ddot{v}_G \quad ,$$

where k_z is the vertical spring representing the foundation stiffness, v is the absolute displacement of the mass, w is the distortion of the structural

spring, v_o is the deformation of the foundation, and v_G is the vertical ground displacement in the free field. Notice that in this case k represents the vertical (axial) stiffness of the structure rather than the lateral one.

The ratio between the natural vertical frequencies of the structure on a flexible foundation and on a rigid base is then given by

$$\frac{\omega}{\omega_o} = \frac{1}{\sqrt{1 + \frac{k}{k_z}}} ,$$

and the effective damping in vertical vibration at the frequency ω is approximately

$$D = D_{st}\left(\frac{\omega}{\omega_o}\right)^2 + D_s\left[1 - \left(\frac{\omega}{\omega_o}\right)^2 + \left(\frac{\omega}{\omega_o}\right)^2 \frac{k}{k_z} \frac{\omega c_z}{2k_z}\right],$$

which, in this case, can be rewritten as

$$D = D_{st}\left(\frac{\omega}{\omega_o}\right)^2 + \left(D_s + \frac{\omega c_z}{2k_z}\right)\left[1 - \left(\frac{\omega}{\omega_o}\right)^2\right] .$$

From the analysis of this simple, one degree-of-freedom system, it can be seen that the main effects of soil-structure interaction are

- o A decrease in the natural frequency of the system, depending on the relative stiffness of the structure with respect to the soil. For typical nuclear power plants, a two- or threefold decrease can take place (depending on the properties of the soil). The importance of this change in frequency depends on the frequency content of the design earthquake (i.e., the shape of the design response spectrum). In many practical cases, the change in acceleration and forces will be relatively small.
- o A change (in most cases an increase) in the effective damping of the system. When the internal damping in the structure and the soil are similar, the main factor contributing to the increase in damping is

the loss of energy by radiation of waves away from the foundation. The important point is then to determine whether this form of energy dissipation can indeed take place. The absolute acceleration of the mass and the forces in the structure can be estimated to be proportional to $(D_{st}/D)^{0.4}$.

- For the case of a horizontal earthquake excitation, the appearance of rotational components of motion at the base. If the Fourier transform of the input earthquake in the free field is given by

$$X(\Omega) = \int_{-\infty}^{+\infty} \ddot{u}_G(t) e^{-i\Omega t}\, dt \quad ,$$

then the time history of accelerations of the base is given by the terms

$$u_o + u_G = \frac{1}{2\pi}\int_{-\infty}^{+\infty} \left[1 + \frac{k^*}{k_x^*} \frac{M\Omega^2}{k^* - M\Omega^2\left(1 + \frac{k^*}{k_x^*} + \frac{k^*h^2}{k_\phi^*}\right)} \right] X(\Omega) e^{i\Omega t}\, d\Omega \quad ,$$

and

$$\phi_o = \frac{1}{2\pi} \int_{-\infty}^{+\infty} \frac{k^*h}{k_\phi^*} \frac{M\Omega^2}{k^* - M\Omega^2\left(1 + \frac{k^*}{k_x^*} + \frac{k^*h^2}{k_\phi^*}\right)} X(\Omega) e^{i\Omega t}\, d\Omega \quad ,$$

where

$$k^* = k(1 + 2iD_{st}) \quad ,$$

$$k_x^* = (k_x + i\Omega c_x)(1 + 2iD_s) \quad ,$$

$$k_\phi^* = (k_\phi + i\Omega c_\phi)(1 + 2iD_s) \quad .$$

These expressions are only approximate because of the way the internal soil damping is handled.

The simple model used above, or a more complicated one, including several masses and a better representation of the structure, could be utilized to directly determine accelerations and forces in the structure (as well as response spectra at different levels for the design of the equipment); to determine modified natural frequencies, mode shapes, and damping values due to interaction effects (which could then be used for a modal solution of the structure's response); or to obtain the horizontal and rotational components of the base motion (the vertical component in the case of a vertical earthquake excitation). In this last case, the structure's response would be computed through standard methods of dynamic analysis with the modified motions as input.

In order to use the above formulas to estimate the magnitude of interaction effects, it is necessary to know the values of the terms k_x, c_x, k_ϕ, c_ϕ, k_z, and c_z, which represent the dynamic stiffnesses of the foundation (modeled here as equivalent springs and dashpots).

2.2 FOUNDATION STIFFNESSES

For a circular foundation on the surface of an elastic half space, Veletsos and Wei have tabulated the values of the horizontal and rocking stiffnesses, expressing them in the form

$$k_x = k_{xo}\left(k_{11} + \frac{i\Omega R}{c_s} c_{11}\right) ,$$

and

$$k_\phi = k_{\phi o}\left(k_{22} + \frac{i\Omega R}{c_s} c_{22}\right) ,$$

where

$$k_{xo} = \frac{8GR}{2 - \nu} ,$$

$$k_{\phi o} = \frac{8GR^3}{3(1 - \nu)} .$$

These terms are the static stiffnesses for a smooth footing (relaxed boundary conditions at the contact between the footing and the soil), Ω is the excitation frequency, R is the radius of the foundation, G is the shear modulus, ν is Poisson's ratio, and c_s is the shear-wave velocity of the soil. The term $\Omega R/c_s = a_o$ is a dimensionless frequency. The dynamic stiffness coefficients k_{11}, c_{11}, k_{22}, and c_{22} are as shown in Fig. 2-2.

An inspection of this figure indicates that the horizontal stiffness terms are nearly independent of frequency. Therefore, their representation as a spring and dashpot introduces little error for surface foundations or a half space. The coefficients for the rocking stiffness, on the other hand, show a significant variation with frequency for values of a_o smaller than 3 (and over a larger range for Poisson's ratio close to 0.5).

Consider a nuclear reactor building with a mass of 10^8 kg and a fundamental frequency on a rigid base of 6 Hz. The equivalent stiffness is then $k \simeq 1.4 \times 10^{11}$ N/m. Taking $R \simeq 30$ m, $h \simeq 15$ m, a soil with a shear-wave velocity (reduced to account for the level of strains) of 200 m/s, and a shear modulus $G \simeq 8 \times 10^7$ N/m^2, the foundation stiffnesses are for $\nu \simeq 0.4$,

$$k_{xo} \simeq 5GR \simeq 1.2 \times 10^{10} \quad ,$$

and

$$k_{\phi o} \simeq 4.5\, GR^3 \simeq 9.7 \times 10^{12} \quad .$$

Then

$$\frac{k}{k_{xo}} \simeq 11.67 \quad ,$$

$$\frac{kh^2}{k_\phi} \simeq 3.24 \quad ,$$

$$\omega \simeq \frac{\omega_o}{\sqrt{16}} = 0.25\omega_o \quad ,$$

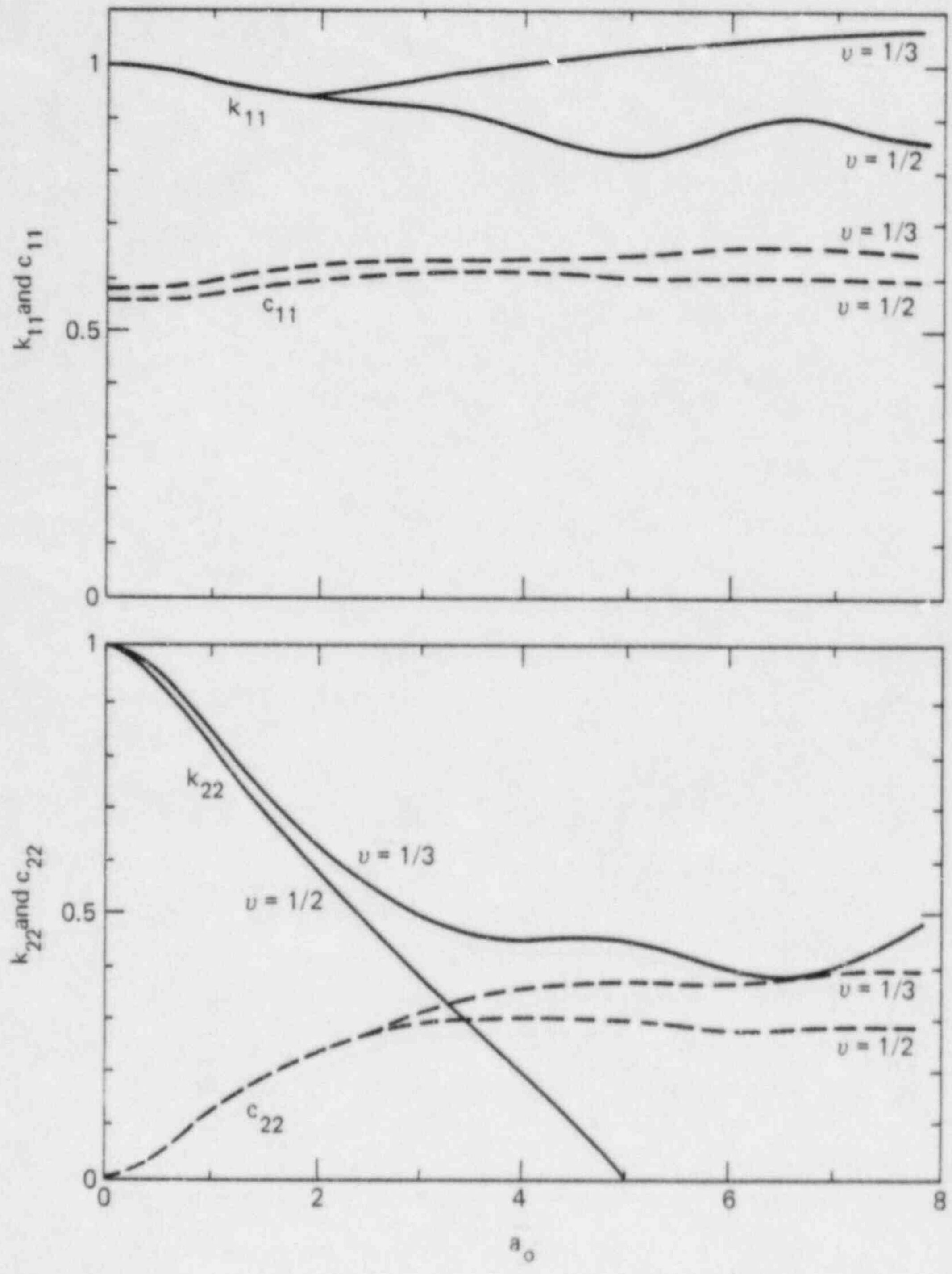


FIG. 2-2. Stiffnesses for circular foundation on an elastic half space (after Veletsos).

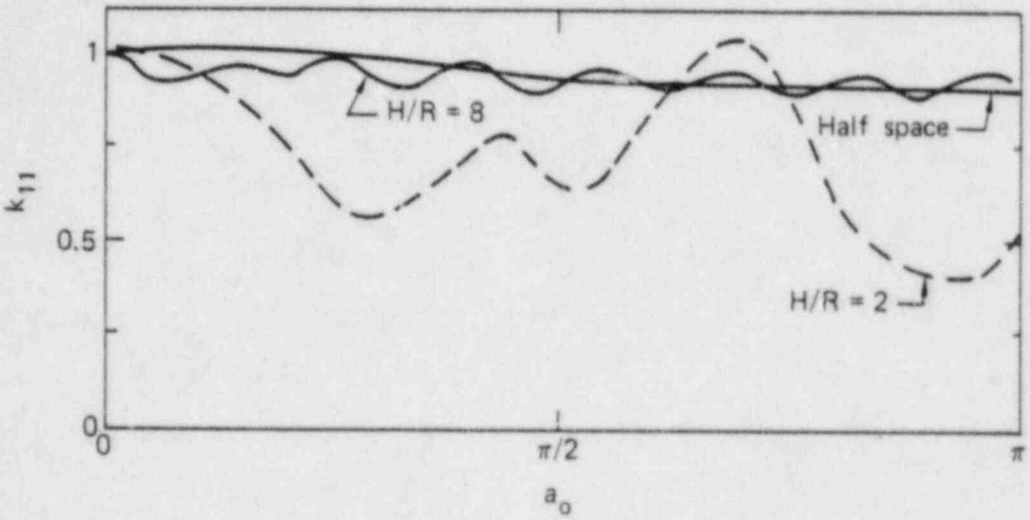
or $f \approx 1.5$ Hz. For a broad-band design response spectrum such as the Newmark-Blume-Kapur (NBK) spectrum, this considerable change in the natural frequency of the system would push it outside the range of maximum spectral accelerations. A decrease of the maximum response acceleration of about 25% could be expected.

Assume that both the structural and the internal soil dampings are 0.07. The dimensionless frequency corresponding to the natural frequency of the system is $a_o \approx 1.4$, leading to $k_{11} \approx 1$, $c_{11} \approx 0.6$, $k_{22} \approx 0.8$, and $c_{22} \approx 0.2$. This would provide an effective damping of approximately 0.40, the contribution of the radiation damping being 0.33. Because of this increase in damping, the response acceleration might decrease by a factor of two.

In this extreme hypothetical case, the effect of the foundation flexibility would be a reduction of the natural frequency from 6 to 1.5 Hz, an increase in the effective damping from 0.07 to 0.40, and a reduction in the response acceleration (and the base shear) of about 60%. The radiation damping is the main factor causing this decrease in accelerations and forces.

An elastic half space with constant properties is rarely encountered in practice. In most cases, and particularly when considering the effect of the seismic waves, the soil properties vary with depth. More importantly, a stiffer rocklike material may be encountered at a certain depth. Figure 2-3 shows the stiffness coefficients for a layer of soil of finite depth on a rigid base. The static stiffnesses will increase somewhat, depending on the R/H ratio, where H is the thickness of the stratum (approximate formulas are presented in Chapter 4). If the soil has some internal damping, the coefficients k_{11} and k_{22} are not very different from those of a half space, but the damping terms c_{11} and c_{22} are essentially zero below the fundamental frequency of the stratum.

Now assume a layer of soil with the same properties of the previous example and a depth of 50 m. The fundamental frequency of the stratum is 1 Hz. The value of k_{xo} may increase by 30% and the value of $k_{\phi o}$ by 10%, leading to a frequency for the soil-structure system of 1.66 Hz, which is larger than that of the soil deposit. As a result, the soil-structure interaction effects would be very similar to those computed before for a half space.



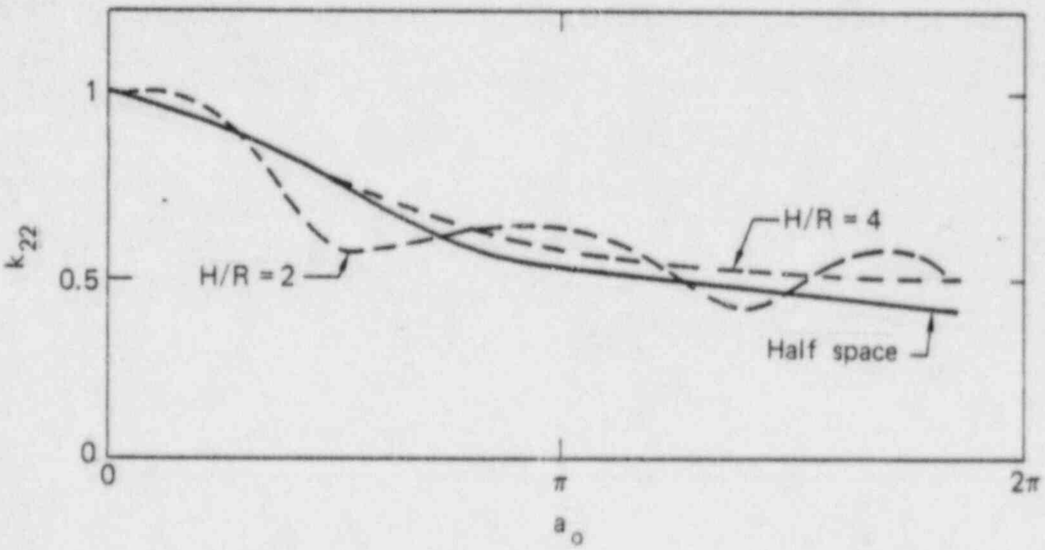


FIG. 2-3. Effect of layer depth H on real (a) and imaginary (b) stiffness coefficients.

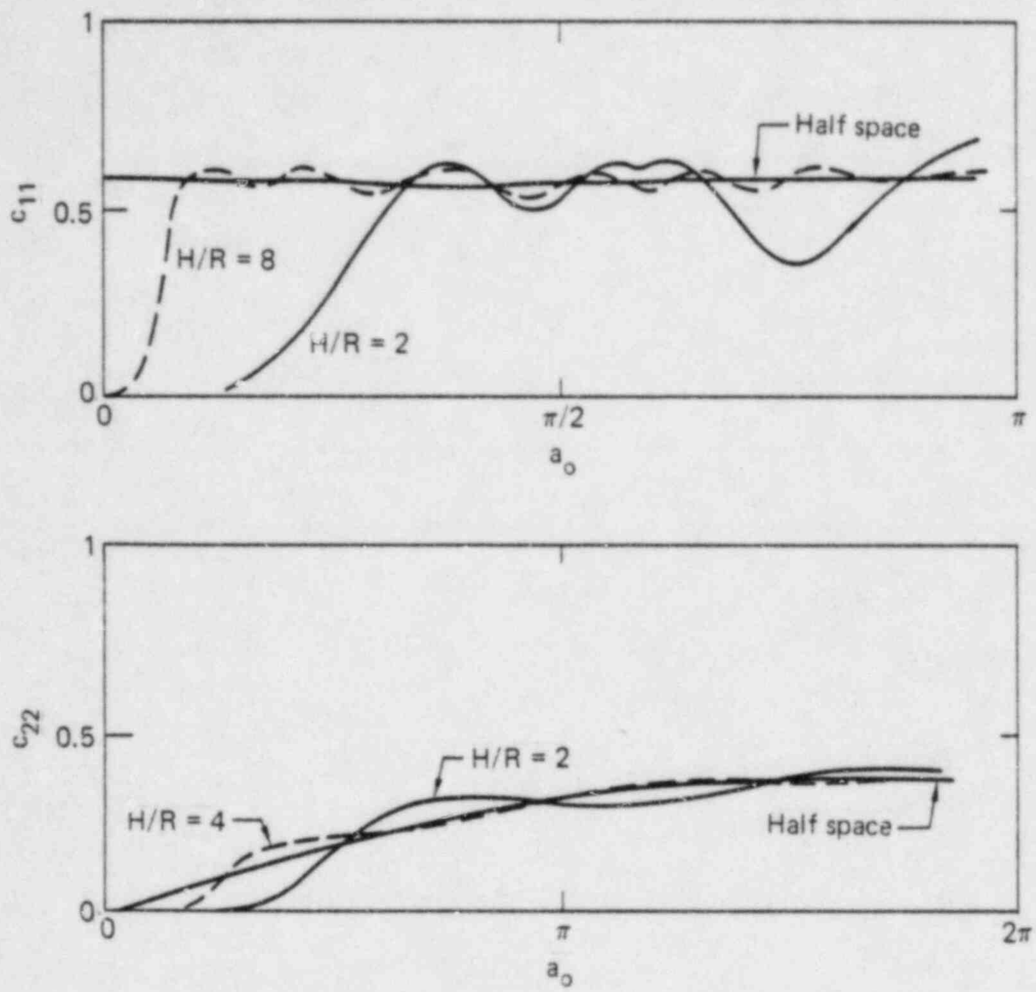


FIG. 2-3 (continued).

Consider on the other hand a stratum with a depth of only 20 m. The frequency of the layer is then 2.5 Hz; the value of k_{xo} increases by 75% and that of $k_{\phi o}$ by 25%. The natural frequency of the soil-structure system is 1.9 Hz smaller than that of the soil layer. In this case, the radiation damping would be very small (zero if the soil were perfectly elastic). The effective damping in the first mode could be of the order of 7% to 10%, and the reduction in the response acceleration and the base shear might be 15% or less.

This simple example illustrates the importance of assessing in practical situations whether the soil profile can be assimilated to a half space or whether a layer of finite depth must be considered. This decision will be an important one in setting up the mathematical model for soil-structure interaction analysis, whatever the formulation.

2.3 EMBEDDED FOUNDATIONS

Most nuclear reactor buildings have their foundations embedded to a certain degree. Two effects must be considered in this case:

- The static values of the foundation stiffnesses increase. For an embedment depth of 5 m, the horizontal stiffness could increase by 10% in the case of a half space and by 35% for a finite soil layer 30 m thick. The rocking stiffnesses would increase by 33% to 50% for the same cases. Figure 2-4 shows typical variations of the stiffness coefficients for an embedded foundation. The main difference with respect to a surface foundation is the increase in the value of the imaginary terms representing the radiation damping. It should be noticed, on the other hand, that when dealing with a finite soil layer these terms are still essentially zero below the fundamental frequency of the stratum.
- The input motion to be used for the soil-structure interaction analysis should no longer be the specified earthquake at the free surface of the soil deposit in the free field. Even under the assumption of vertically propagating shear waves, the horizontal (translational) motion experiences some reduction (particularly in the

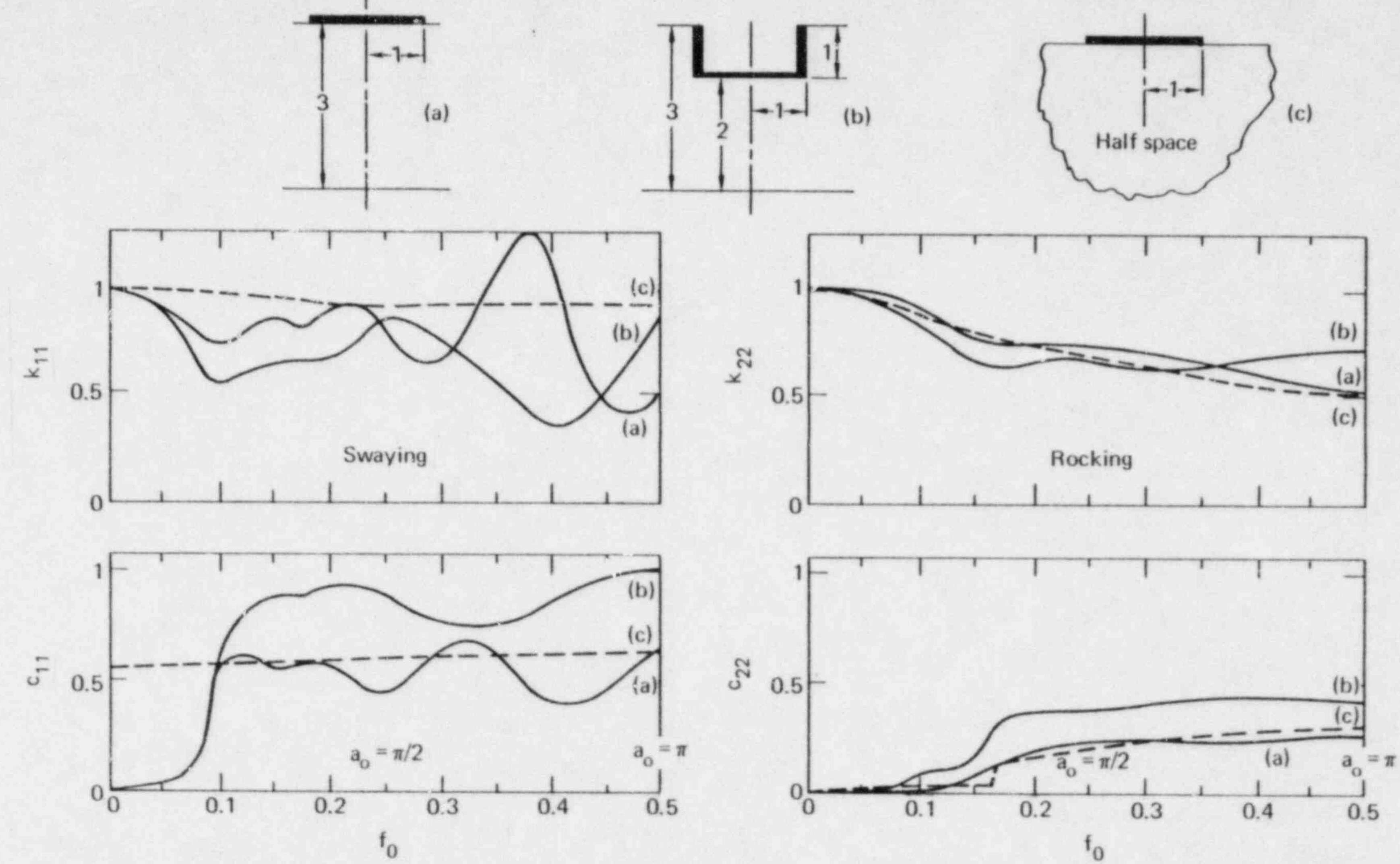


FIG. 2-4. Dynamic stiffness coefficients for an embedded foundation, compared with surface foundations. Poisson's ratio was taken as 0.33, the soil material damping as 0.05.

high-frequency range), and there is in addition a rotational component. Figure 2-5 shows typical amplitudes of the transfer functions for the translational and rotational motions to be applied at the base of an embedded structure with respect to the motion specified at the surface.

In this case, let u be the absolute displacement of the mass, y be the distortion of the structural spring, u_o and ϕ_o be the deformations of the foundation springs, and u_G and ϕ_G be the translational and rotational input motions (at the foundation level, accounting for the excavation). The absolute displacement and rotation of the foundation are

$$u_f = u_o + u_G \quad ,$$

and

$$\phi_f = \phi_o + \phi_G \quad .$$

The absolute displacement of the mass is

$$u = u_f + y + \phi_f h = u_o + y + \phi_o h + u_G + h\phi_G \quad .$$

The equations in this case become

$$ky = k_{xx}u_o + k_{x\phi}\phi_o \quad ,$$

and

$$khy = k_{\phi x}u_o + k_{\phi\phi}\phi_o \quad .$$

The coupling terms $k_{x\phi} = k_{\phi x}$ would also exist in reality for the case of a surface foundation, but can typically be neglected for that situation. They must, however, be included for an embedded foundation. They represent the fact that the equivalent springs should be lumped at a certain height d from the base, where $d = -k_{x\phi}/k_{xx}$.

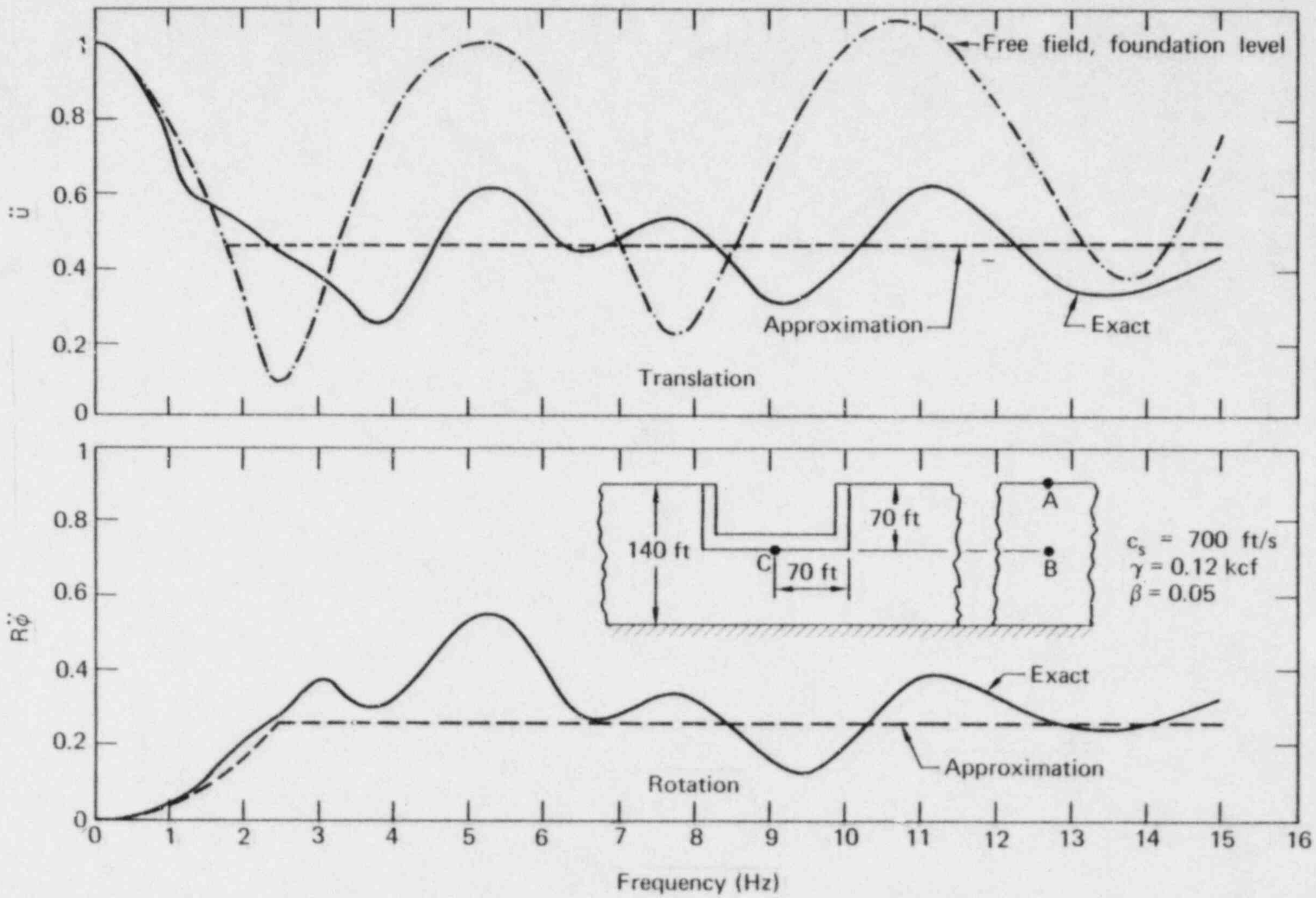


FIG. 2-5. Absolute values of transfer functions for the motion of a massless foundation; β is the soil material damping, and γ is the unit weight.

Now let

$$k_x' = \frac{k_{xx}k_{\phi\phi} - k_{x\phi}k_{\phi x}}{k_{\phi\phi} - hk_{x\phi}} \quad ,$$

$$k_\phi' = \frac{\left(k_{xx}k_{\phi\phi} - k_{x\phi}k_{\phi x}\right)h}{hk_{xx} - k_{\phi x}} \quad ,$$

$$u_o = \frac{k}{k_x'}y \quad ,$$

$$\phi_o = \frac{kh}{k_\phi'}y \quad .$$

The equation of motion for the mass,

$$M\ddot{u} + ky = 0 \quad ,$$

can then be rewritten as

$$M\left(1 + \frac{k}{k_x'} + \frac{kh^2}{k_\phi'}\right)\ddot{y} + ky = -M\left(\ddot{u}_G + h\ddot{\phi}_G\right) \quad .$$

A simple model can still be used if one computes the effective stiffnesses k_x' and k_ϕ' and the effective input motion $\ddot{u}_G + h\ddot{\phi}_G$. It is important to notice that the contribution of the rotational base motion to the response may be of the same order of magnitude as that of the base translation (from 10% to 50% for the acceleration of the mass, even larger when computing amplified response spectra at the top levels of a structure with a more sophisticated model). Ignoring the rotational component, but reducing the horizontal motion to account for embedment, could therefore lead to unconservative results. However, the net effect in most cases is a reduction in the effective motion and the base shear.

When dealing with a foundation embedded in a deep soil deposit, which can be considered as a half space, the increase in the static values of the foundation stiffnesses reduces the interaction effects (i.e., it decreases the natural frequency and increases the damping). However, the increase in the imaginary terms may compensate for the reduction in the terms k/k_x' and kh^2/k_ϕ', and the net effect may be similar to that for a surface foundation. In addition, the decrease in the excitation with depth generally produces a further reduction in the base shear. In this case, both effects (the so-called kinematic and inertial interactions) may be of the same order of importance.

When dealing with a foundation embedded in a relatively shallow finite soil layer, where the frequency of the soil-structure system is smaller than that of the layer, the main effect is the reduction in the earthquake excitation, because of the embedment (kinematic interaction).

2.4 CONCLUDING REMARKS

The study of the simple one-degree-of-freedom system described above helps to illustrate some of the key features of the soil-structure interaction phenomenon. To reproduce these basic features, it will be necessary in each particular case to estimate the appropriate values of the dynamic stiffnesses. This requires consideration of the shape of the foundation, whether it is embedded, and the effective soil properties. In addition, for an embedded foundation, it is necessary to estimate the effective excitation. Different methods to model these effects are discussed in the next chapters.

CHAPTER 3

METHODS OF ANALYSIS

Two general methods are used at present for the seismic analysis of nuclear power plants, including the effect of foundation interaction:

- A direct approach in which the combined soil-structure system is solved in a single step. The structure is modeled through a combination of finite elements and linear members, and the soil is discretized using finite elements or finite differences. Because a discrete model is used to reproduce a semi-infinite domain, special attention must be paid to imposing appropriate boundary conditions at the edges. The main advantage of this approach is that it permits a true nonlinear analysis with the complete interaction effects. However, rigorous solution requires a fully three-dimensional model, which is expensive, and an appropriate set of nonlinear constitutive equations for the soil, which may not be available. In practice, these two requirements are rarely met. Most direct solutions use a two-dimensional or a pseudo-three-dimensional soil model (with dashpots added to the lateral faces of the soil slice), and they approximate nonlinear soil behavior through equivalent linearization procedures. Application of this method requires, in addition, the determination of a consistent seismic motion at the boundaries of the discrete domain (a preliminary step closely related to the first step of the substructure approach), and in many cases, an a posteriori analysis of a more refined structural model.
- A three-step or substructure approach. In this case, the problem is broken into three separate parts:
  a. Determination of a compatible seismic input for the foundation. If the foundation can be assumed to be rigid, which is normally the case, this motion consists at most of six components (three translations and three rotations). For a very flexible foundation

(as a large mat supporting several buildings), it is necessary to determine three components of motion at a sufficient number of contact points between the foundation and the soil (or between the structure and the foundation).

b. Determination of the foundation stiffnesses. For a rigid foundation, this implies obtaining the terms of a six-by-six matrix by applying unit harmonic displacements and rotations on the foundation and by computing the resulting forces and moments. The stiffness coefficients are complex functions of frequency. For a flexible foundation, unit harmonic displacements must be applied at a sufficient number of contact points between the foundation and the soil (or between the structure and the foundation), and corresponding forces must be evaluated at each one of these points.

c. Dynamic analysis of the structure supported on an elastic medium represented by the stiffness matrix of the foundation (sometimes referred to as equivalent, frequency-dependent springs) and subjected to the motions computed in the first step.

This approach is limited to a linear elastic solution, since it is based on the principle of superposition. To simulate nonlinear soil behavior, it is necessary to estimate equivalent values of soil moduli and damping based on the seismic excitation alone. Additional nonlinearities created by the vibrations of the structure must therefore be neglected or estimated in an approximate way. On the other hand, it provides considerably more flexibility in the way each one of the steps is solved, and it allows one to study better the effect of various parameters and uncertainties on each one of the phases.

3.1 THE DIRECT APPROACH

Figures 3-1 through 3-4 show typical models of the soil-structure system used for a direct solution. In Fig. 3-1, both the structure and the soil are modeled with toroidal finite elements in cylindrical coordinates. For linear

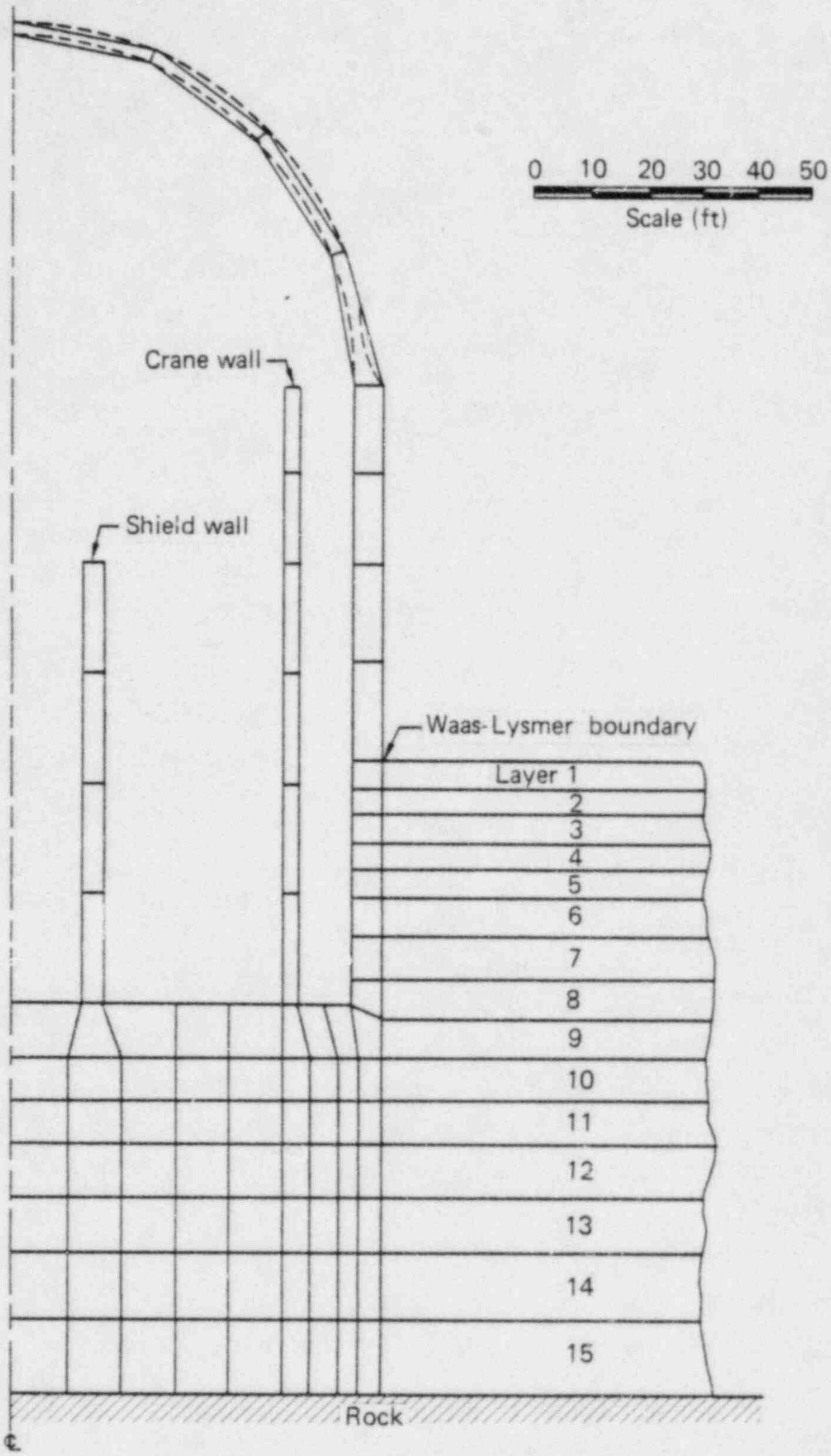


FIG. 3-1. Typical cylindrical finite element model.

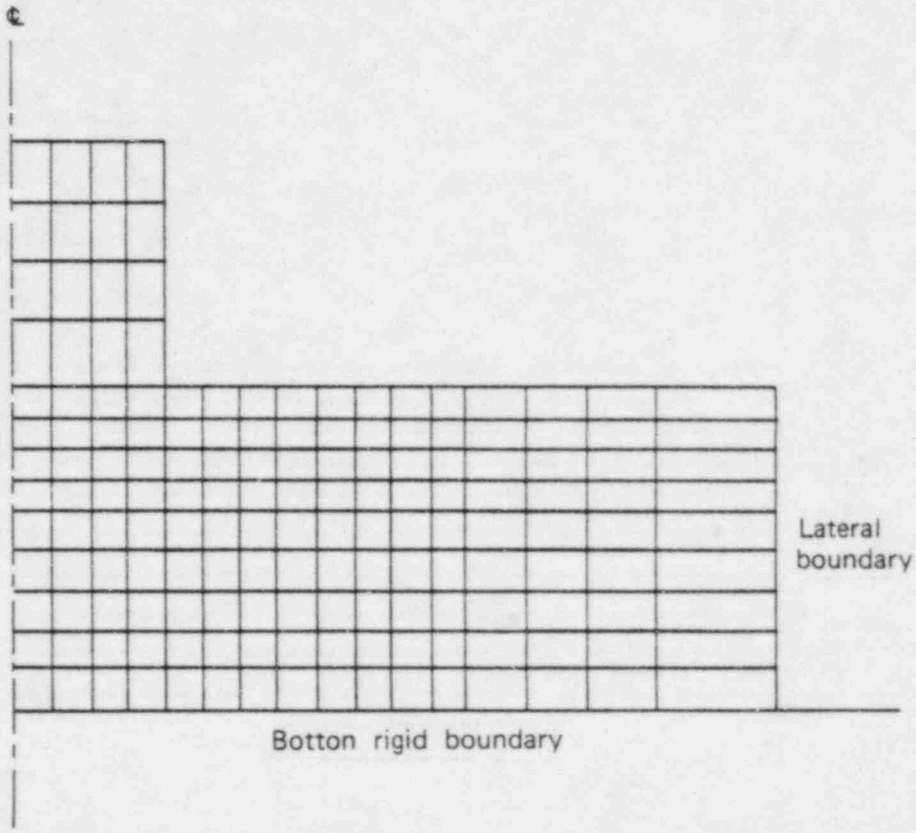


FIG. 3-2. Typical two-dimensional finite element model.

systems where the effects of shear and dilatational waves can be uncoupled, studied separately, and then superimposed, this model would be a truly three-dimensional one. The solution is expressed in terms of a Fourier series in the circumferential direction. When the structure and the soil have axisymmetric geometry and properties, and the excitation is assumed to consist of shear or dilatational waves vertically polarized, the procedure is particularly effective and economical: only the $n = 0$ term of the Fourier series is needed for a vertical (symmetric mode) or a torsional (antisymmetric mode) excitation, and the $n = 1$ term for horizontal excitation. For more complicated geometries or excitations, the model is still applicable using a sufficient number of terms of the Fourier series, but it loses a great deal of its computational advantage. This model is implemented in computer programs such as TRIAX (Stone and Webster).

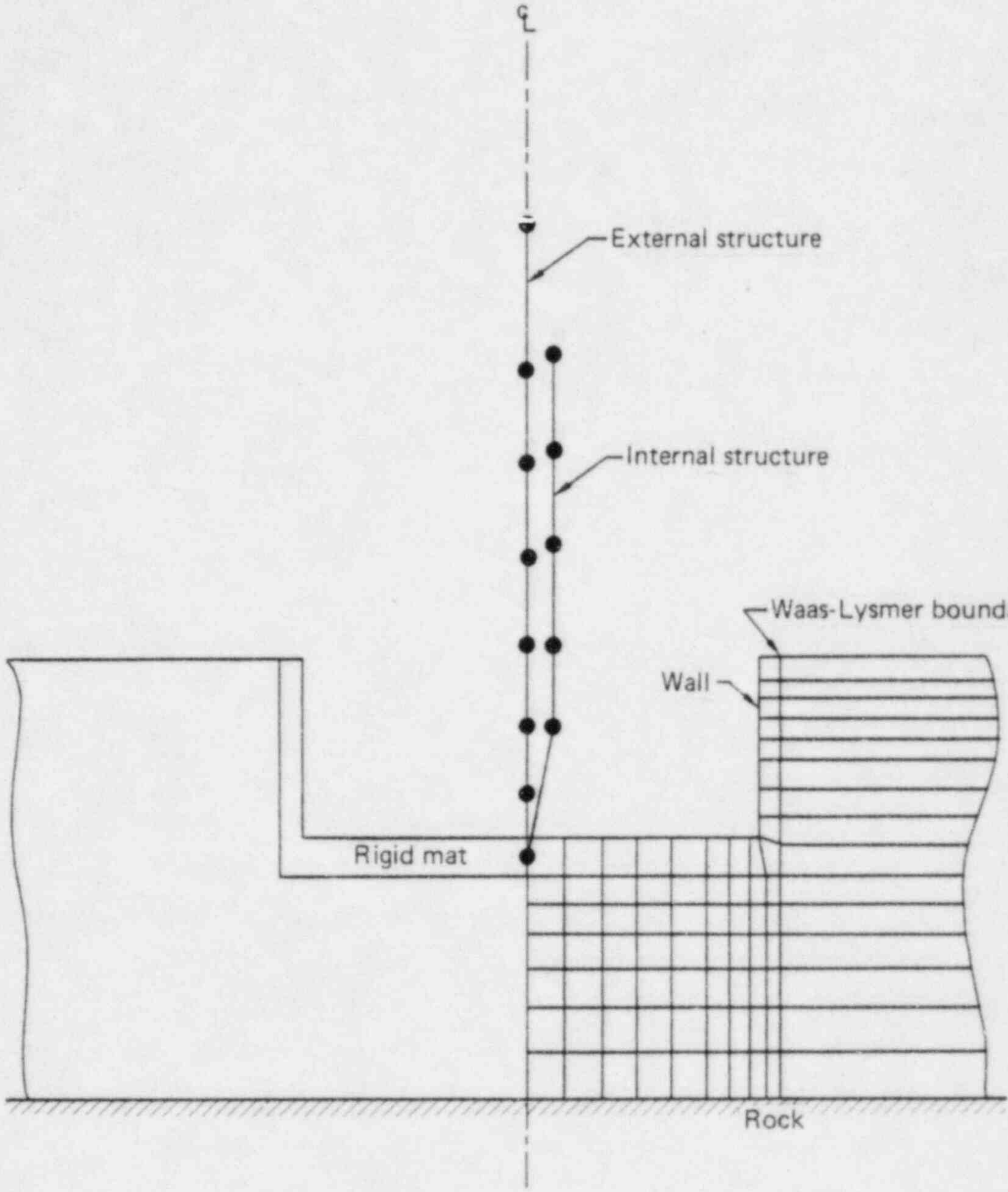


FIG. 3-3. Typical two-dimensional stick model.

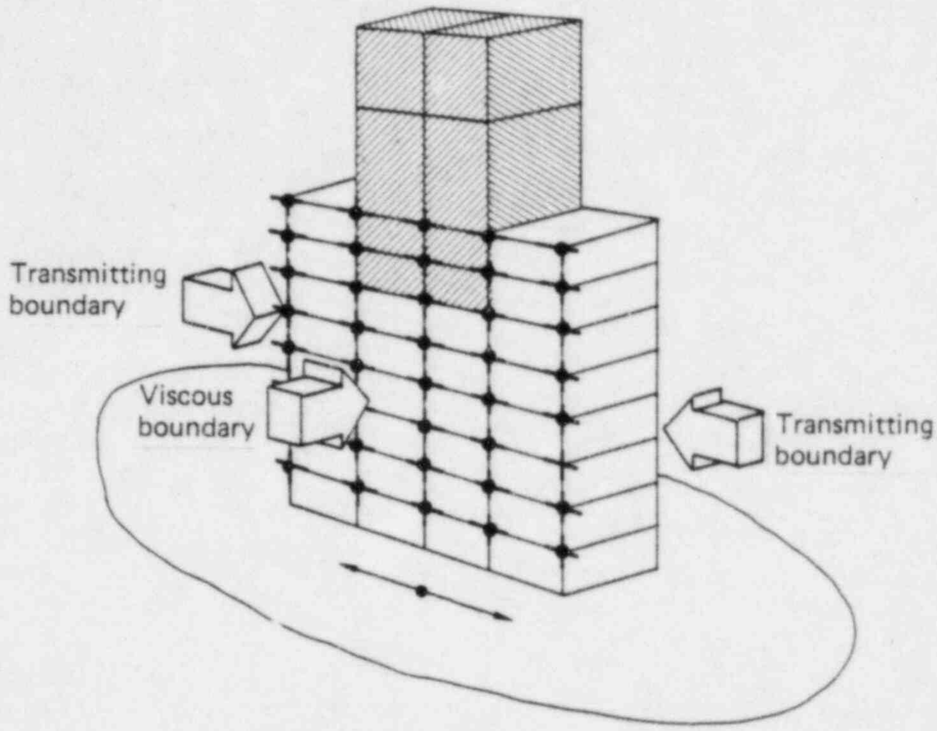


FIG. 3-4. Typical pseudo-three-dimensional finite element model.

In Fig. 3-2, both the structure and the soil are modeled with two-dimensional finite elements, corresponding normally to a plane-strain condition. This model is appropriate to study the vibrations in the short direction of elongated structures such as walls, dams, etc. When applied to a structure with a nearly square or circular base, such as a reactor building, replacing the three-dimensional problem by a two-dimensional model introduces an approximation which must be carefully evaluated. Moreover, if the structure is reproduced by a continuous two-dimensional block, as shown in the figure, it is clear that member forces or stresses cannot be obtained from the analysis. Even floor accelerations may not be reliable, and the most one can expect is to obtain reasonable estimates of the translational and rotational motions at the base. An analysis of a more refined structural model subjected to these components of motion must then be performed a posteriori.

An improvement is obtained by modeling the structure through a combination of linear members, as shown in Fig. 3-3. The simple representation of these stick models in typical figures has given the impression that they are independent, close-coupled, shear-beam models. This is not necessarily true. Stick models can be rather accurate representations of the various structural components, including axial, shear, and bending deformations; eccentricities between the centers of twist and the centers of mass of different floors; and even coupling between the various components.

Two-dimensional plane-strain models are the ones most commonly used in practice today. They are implemented in computer programs especially intended for soil-structure interaction analyses, such as LUSH (University of California, Berkeley) or PLAXLY (MIT). General-purpose finite element codes such as SAP, STRUDL, or ADYNA are also sometimes used to analyze these models. An equivalent formulation, modeling the soil with finite differences instead of finite elements, forms the basis for the program DAMSEL (Dames and Moore).

A variation from the basic two-dimensional model described above is provided in the program FLUSH (University of California, Berkeley). While the stiffness characteristics of the structure and the soil are still two-dimensional, additional radiation damping is introduced by placing viscous dashpots on the sides of the soil slice (Fig. 3-4). The adequacy of this modification will be discussed later.

Three-dimensional models in Cartesian coordinates are generally too expensive for practical design purposes. Analyses with these models have been conducted by Wolf and by Weidlinger Associates (using the computer program TRANAL, a finite element code).

The main steps involved in a direct solution of the soil-structure interaction problems are:

- Selection of a specific model (two-dimensional plane-strain, pseudo-three-dimensional, cylindrical, or general three-dimensional; finite elements or finite differences; etc.). It is clear that a general three-dimensional model would be theoretically the best, but a cylindrical model might be quite satisfactory, and even plane-strain

discretizations can furnish sensible results if properly implemented. This decision may be dictated by economic considerations and the availability of specific computer codes. Some time should be spent, however, considering the alternatives, and preliminary studies with simplified models would be advisable to justify the decision.

- Selection of a finite domain (core region) and of boundary conditions adequate for the dimensions of this domain. Some preliminary computations may again be desirable.
- Selection of an appropriate mesh size for the finite element or finite difference discretization.
- Selection of an appropriate structural model.
- Selection of appropriate soil properties and a model to simulate nonlinear soil behavior.
- Computation of compatible motions to be input as excitation at the boundaries of the finite domain.
- Selection of a solution scheme (direct integration of the equations of motion in the time domain, direct solution in the frequency domain, modal analysis in the time or frequency domains). This decision may again be influenced by the characteristics of the specific computer code selected. Some thought must be given, however, to the selection of parameters such as the time step of integration or the frequency increment.

3.1.1 Two- versus Three-Dimensional Models

As pointed out before, a solution in cylindrical coordinates can be considered fully three-dimensional for the cases where it is applicable (axisymmetric geometry, simple seismic excitations, and a linear system; or the use of several terms of the Fourier series expansion and a linear system). A point of more concern and the subject of some discussion is the degree of approximation provided by two-dimensional or pseudo-three-dimensional solutions when dealing with a system which is basically three-dimensional. Clearly in this case, torsional effects cannot be studied and must be handled separately. This may not be too serious, considering the

uncertainties involved in describing a torsional excitation, which would make more appropriate an independent evaluation of these effects. It shows, however, the inability of the direct approach, using a two-dimensional model, to solve the complete problem in a single step as intended. This capability is further jeopardized by the need to analyze independently the effects of motions in two orthogonal planes, which limits the accuracy of two-dimensional nonlinear analyses (even if one has a good set of nonlinear constitutive equations).

Comparisons between two-dimensional and three-dimensional solutions for a linear system excited in the plane of the model have been performed by a number of authors. An original study by Luco and Hadjian considered a number of alternatives and gave rules to select the optimum dimensions of the equivalent strip footing. It concluded, however, that the agreement would always be poor, and it showed differences of up to 50% in the amplified in-structure response spectra, with the plane-strain solution being unconservative (the 2D solution overestimated the radiation damping for the dimensions chosen). It should be noticed, though, that the comparison was performed for an elastic half space, a situation which is not always realistic. In this case, the strip footing has zero static horizontal stiffness, and the agreement over the low-frequency range is always poor.

A second study by Berger, Lysmer, and Seed also showed important differences between the amplified spectra from two-dimensional and three-dimensional analyses. It concluded, however, that these differences were due to the model of the structure, and that the horizontal components of motion at the base of the structure for the two cases were very similar (surprisingly, no mention was made of the rotational component of motion). The study recommended the use of a complete two-dimensional soil-structure model just to obtain the base motion, and then the use of a structural analysis program, with a three-dimensional model of the structure and the computed motion applied at the base, to determine structural response parameters. Ignoring the rotation at the base of the structure could be a serious mistake. More importantly, this approach would then be closer to the substructure method than to the direct solution.

A comparison by Kausel for the case of an actual reactor building on a finite soil layer resting on much stiffer rock (assumed to be rigid) indicated differences between 2D and 3D solutions, but only of the order of 15%. It was felt, however, that this case was not sufficient to produce general conclusions.

Jakub performed more comprehensive comparisons between 2D and 3D solutions, but only in relation to the foundation stiffnesses. He found that, as pointed out by Luco and Hadjian, it is not possible with a plane-strain model to match all the stiffness terms of a square or circular foundation, but he concluded that the agreement might be reasonable when dealing with a finite soil layer, not too deep, resting on rigid rock. Furthermore, he recommended that preliminary checks be conducted, using simplified models such as those presented in the ATC-3 study, in order to assess the effect of the approximation on the structural response and to guide in the selection of the equivalent dimensions. In order to select these dimensions, it may be appropriate to match the damping terms rather than the stiffnesses, or to attempt to minimize the error on those combinations of these terms which more significantly affect the response (depending on the characteristics of the structure).

Some recent papers and reports have suggested that a plane-strain model will always underestimate the radiation damping and thus provide results which are on the conservative side. This is the basis for the additional dashpots placed on the sides of the soil slice in the computer program FLUSH. It is important to notice that this is not always the case, but that whether the damping is under- or overestimated depends on the selection of the equivalent width of the footing and the thickness of the slice. The results presented by Luco and Hadjian illustrate clearly a case where damping is overestimated by a 2D solution.

More extensive comparative studies may be necessary to define better the range of applicability of 2D solutions and the magnitude of the potential errors for typical, realistic situations. It is not likely that errors will be as large as those reported by Luco and Hadjian if the parameters are carefully chosen. The fact that they exist and that they may be of the same order of magnitude as those introduced by simplified models should, however, be recognized.

3.1.2 Boundary Conditions

When using a discrete model to reproduce a soil deposit with very large dimensions (extending for practical purposes to infinity), it is necessary to create a finite domain of manageable dimensions by introducing fictitious boundaries at the bottom and on the sides. To ensure that the model still simulates properly the behavior of the true physical problem, appropriate boundary conditions must be imposed at the edges of the finite domain.

Most finite element and finite difference programs assume the bottom boundary to be rigid, thus neglecting the effect of vertical radiation. When there is a soil deposit resting on much stiffer, rocklike material, with an abrupt change in elastic properties, this assumption is nearly correct if the boundary is set at the interface between the soil and the rock. On the other hand, when dealing with a deep soil deposit with nearly constant properties, or with properties that vary smoothly with depth, the bottom boundary introduces an error, which is a function of the depth at which it is placed and the amount of internal damping in the soil. To guide in the selection of the location of the boundary in this case, some preliminary computations, using simplified models, should be performed to ensure that the fundamental frequency of the soil-structure system is above the fictitious natural frequency of the soil deposit. An improvement is obtained by placing viscous dashpots at the bottom of the profile, as suggested by Tsai, when the underlying soil has relatively constant properties.

Various types of boundary conditions are used at the lateral edges as discussed below.

Elementary Boundaries. For elementary boundaries, conditions are specified on forces (free boundaries), displacements (fixed boundaries), or combinations of both (roller boundaries). Along free boundaries, one must specify forces equal to the resultants of the stresses in the free field due to the seismic motions. Along fixed boundaries (equivalent to the soil-island approach of Weidlinger Associates), one must specify the displacements (or accelerations) which would occur in the free field. When the motions are caused by waves

traveling vertically, roller boundaries (horizontal rollers for shear waves, vertical rollers for compressional waves) reproduce exactly the conditions in the free field.

Elementary boundaries are intended to reproduce correctly the vibrations of a horizontally layered soil deposit excited by seismic waves. For the soil-structure interaction problem, the vibrations of the structure will cause additional waves which radiate away from the foundation When these waves hit the boundaries, they reflect back into the core region, creating a box effect which distorts the solution. The importance of this effect depends on the distance at which the boundaries are placed, the amount of internal damping in the soil, and the range of frequencies of interest.

Viscous Boundaries. Viscous dashpots similar to those suggested by Tsai for the bottom boundary can be placed along the lateral edges. Solutions for the finite element method have been presented by Kuhlemeyer and for a finite difference discretization by Ang and Newmark. Small refinements on these basic solutions have been suggested by a large number of authors. These boundaries absorb (or transmit) a train of plane body waves hitting the boundary at a specific angle. Unfortunately, as was shown by Kuhlemeyer, the waves created by the vibrations of the structure are far from being a plane wave front and include not only combinations of body waves at various angles, but also a large component of surface waves. An alternative suggested by Kuhlemeyer is to use viscous dashpots that absorb exactly the first Rayleigh-wave mode (computation of the constants of these dashpots, varying with depth, is considerably more laborious than that of the standard solution).

Because of the complex nature of the waves generated by the structural vibrations, viscous boundaries are only approximate, and while they represent a small improvement over the elementary boundaries, they still must be placed at a sufficient distance from the edge of the foundations (which is a function of the amount of internal damping and the frequency range of interest) to guarantee an acceptable solution.

The fact that Kuhlemeyer's work was initially intended to study the vibrations of soil deposits or soil-structure systems caused by an excitation located within the core region (machine vibrations, for instance) has led an alarming number of authors to state that these boundaries are not applicable

for a seismic excitation. This is incorrect. Viscous boundaries can be applied to the case of interest here with the same order of accuracy inherent in the case of an excitation within the core region, by imposing them on the relative motion between the edge nodes and the free field (i.e., the disturbance from the free-field solution caused by the presence of the structure). This requires a priori computation of the free-field motions. In addition, forces corresponding to the resultants of the free-field stresses must be concentrated at the edge nodes to guarantee that the core region, without structure or excavation, behaves exactly as the free field. It must be noted, however, that as pointed out above these boundaries are not very good.

Consistent Boundaries. Consistent boundaries were originally developed by Waas for the two-dimensional, plane-strain model, with the excitation contained within the core region. Chang Liang extended them, showing how they can be applied to the case of a seismic excitation (by imposing the boundary matrix on the relative motion between the edge nodes and the free field, and by applying additional forces as indicated above for the case of viscous boundaries). Kausel derived the consistent boundary for the case of cylindrical geometry and the different terms of the Fourier series expansion in the circumferential direction. Consistent boundaries are implemented in FLUSH, PLAXLY, and TRIAX.

Consistent boundaries are obtained by using the exact analytical solution in the horizontal direction and a displacement expansion consistent with that used in the core region in the vertical direction. They can be shown to correspond to a solution where equal columns of finite elements (of differential width) extend all the way to infinity, and they provide an "exact" solution within the accuracy of the finite element discretization for a horizontally stratified soil deposit of finite depth (resting on rigid rock). When the geometry and the soil properties do not change in the horizontal (or radial) direction, consistent boundaries can be placed directly at the edge of the foundation with excellent results. When trying to simulate nonlinear soil behavior, accounting for variation in the levels of strain in the horizontal direction, even these boundaries should be placed at some distance from the foundation.

Consistent boundaries are available at present only for 2D or cylindrical formulations and a solution in the frequency domain (the terms of the boundary matrix are functions of frequencies). An excellent approximation to these boundaries can be obtained using substructuring techniques, and within some restrictions on the variation of soil properties, this solution can be extended to general three-dimensional geometries in Cartesian or spherical coordinates (including also the bottom boundary). An additional alternative, which has not yet been tried, but which should give good results, is to derive consistent boundary matrices for both the lateral and the bottom boundaries. In this case, waves hitting exactly the corner or perimeter of the model (the intersection of the lateral and the bottom surfaces) would not be absorbed, but all other waves would be properly handled.

Superposition. A solution which can be applied both in the frequency and in the time domain is provided by the superposition at each boundary node of two solutions, corresponding to two different sets of boundary conditions. This procedure, known as the Smith boundary, has not received much attention, because it would normally require 2^n solutions if there are n boundary points. It has been suggested, however, by the Advanced Technology Group of Dames and Moore that when implemented in the time domain it is only necessary to compute at each step the two solutions at each boundary node and to average them. While only approximate, it appears that this solution provides very good results (additional work may be necessary to evaluate more fully the degree of accuracy).

Boundary Location. In selecting the location of the boundaries and thus the dimensions of the core region (which will influence directly the cost of the analysis), it is necessary to take into account both static and dynamic effects. One must, on the one hand, be able to reproduce the static pressure bulb under the foundation, and it must be ensured, on the other hand, that the waves reflected at the boundary have a small amplitude when reaching the structure (the amplitude decaying with distance in terms of wavelengths due to the internal material damping). Unless simplified preliminary analyses are conducted to justify the position of the boundaries, it is recommended that

elementary or viscous boundaries be placed at a minimum of five radii (or half widths of the foundation) from the edge of the foundation (a distance of five to ten radii is typically appropriate). The bottom boundary should be placed at the level where a sharp transition in soil properties takes place, or if this is too deep, at a minimum of two to four radii below the foundation if a rigid bottom is assumed. There are not enough data on the appropriate location when a viscous or consistent bottom boundary is used (it must be remembered that soil properties will vary with depth), but two radii would seem to be a minimum unless clear rock is present at smaller depths.

3.1.3 Mesh Size

Once the dimensions of the core region have been selected, it is necessary to define the finite element or finite difference mesh. This will directly affect the number of degrees of freedom of the problem, and therefore the cost of the analysis, as well as the accuracy of the results.

In the selection of a mesh, static and dynamic effects must again be considered. It is necessary first to reproduce adequately the stress distribution under the foundation, particularly in relation to the rotation of the structure. If this rotation plays a significant role in the response, one should have about 10 to 12 elements (or mesh points) over the width of the mat, and it is convenient to have the elements near the edge smaller (narrower) than those in the middle. The mesh size just outside the foundation should also be kept small. It should be noticed, however, that the mesh size necessary to reproduce the dynamic response of the structure accurately is not as small as that needed to determine stresses or strains in the soil accurately (that is, to assess the stability of the foundation, liquefaction potential, etc.). In this report, only the structural response is being considered.

To reproduce dynamic effects accurately, the size of all elements (or the mesh size in a finite difference solution) should be kept smaller than one-sixth to one-eighth of the smallest wavelength (highest frequency) which must be accurately reproduced. This rule of thumb, which is based on simple physical considerations, but has been substantiated by a number of parametric

studies, has often been misinterpreted. The rule as stated applies to the determination of transfer functions representing the response to a steady-state excitation at a single frequency. When interested in obtaining the response to a broad spectrum excitation, one must first assess the contribution of the high-frequency components to the desired effect. This contribution is clearly small for displacements, somewhat larger for accelerations, and most important for amplified response spectra, particularly with low values of damping. It is always less than unity.

When elementary or viscous boundaries are used, it has become customary, in order to place the boundaries at a sufficient distance without increasing excessively the number of degrees of freedom, to use a relatively fine mesh in the neighborhood of the foundation and to enlarge it some distance away. When dealing with a situation which is fundamentally one-dimensional (shear waves or P-waves propagating vertically through the soil), keeping the height of the elements constant and increasing their horizontal dimensions is a reasonable approach. The increase in size should, however, be gradual and should start at some distance from the edge of the foundation (otherwise, the static solution may be misrepresented). For more general wave fronts, or when all dimensions of the mesh are increased, some preliminary studies with simplified models and checks on the selected mesh should be conducted to assess the accuracy of the results over various frequency ranges. Some of the meshes which are shown in papers are suspect.

3.1.4 Structural Model

Constraints on the structural model are imposed by the original decision on the overall model. For a three-dimensional formulation in Cartesian coordinates, there is complete flexibility in the representation of the structural components; for a cylindrical formulation, the structure must be axisymmetric or nearly so; a plane-strain model also requires a two-dimensional structure. In each case, the structural model can consist of linear members, finite elements, or both. It should be noticed, however, that even in the first case a structure as complex as a nuclear reactor building is rarely modeled in three dimensions with the refinement necessary to reproduce

each linear member (beam or column), wall, slab, etc. This degree of sophistication would not even be required of a static analysis. More often, various structural assemblies are analyzed separately with a structural-analysis program (such as SAP or STRUDL) and are reduced to simple stick models which can have two, three, or six degrees of freedom per node and which normally incorporate the actual variation with height of the center of mass and the center of stiffness at each level. Where the structural components are connected through slabs, equivalent connecting members are placed between the stick models. In many cases, the properties of the linear members forming the sticks can be derived from relatively simple computations without the need for computer analyses.

Guidelines for the construction of the structural model are provided in the Regulatory Guides. It is required in particular that all modes with frequencies smaller than 33 Hz be included in the analysis, or that the number of modes be such that the first neglected mode contribute less than 10% to the response. For some of the structural models often used, it is doubtful that modes with frequencies close to 33 Hz could be computed with accuracy (for instance, the two-dimensional blocks of plane-strain elements).

3.1.5 Determination of Compatible Motions

The design earthquake for a nuclear power plant is typically specified at the free surface of the soil deposit in the free field (without any structure or excavation). Before proceeding to the analysis of the complete soil-structure system by the direct approach, it is normally necessary to compute compatible motions at some or all of the boundaries of the discrete soil model.

If the bottom boundary is assumed to be rigid (the most common assumption in practice), a compatible motion at the interface between the soil layer and the underlying soil (below the boundary) must be determined. This motion is a function of the soil properties and cannot be inferred without some analyses. If the bottom boundary has viscous dashpots simulating the existence of a half space below it (Tsai's solution), then the motion that must be computed is the one that would occur at a hypothetical free surface of the underlying half space. (The input consists then of equivalent forces, computed as the product

of the dashpot constants and the velocity of the motion.) In this case, it would be possible to specify directly this motion as input without any computer analyses (the motion could have the characteristics of the NBK spectra if the underlying half space were "firm" ground, or the characteristics of Mohraz's spectra if it were competent rock). While this would be a reasonable approach, it must be kept in mind that in this case the motions at the free surface of the soil deposit would be site dependent and would generally not have the characteristics of the NBK spectra over the complete frequency range. The maximum ground acceleration might also have to be adjusted. If having the characteristics of the NBK spectra at the soil surface is required, computer analysis would again be needed to compute the compatible motion at the hypothetical outcropping of the half space.

When elementary boundaries with roller supports are used at the lateral edges, the amplitudes of the motions or stresses over the depth of the profile are not needed (as long as the motion consists of waves propagating vertically). For all other lateral boundaries, motions, stresses, or both must be computed at the location of all mesh points in the free field.

To compute these compatible motions, an assumption must be made regarding the types of waves propagated during the earthquake. The most common assumption is that the soil is horizontally stratified and that the excitation consists of vertically polarized waves (waves traveling vertically, perpendicular to the layers). This is a particularly simple case, because shear waves then cause only horizontal motions and dilatational waves (P-waves) produce only vertical accelerations. The problem then becomes one-dimensional. It should be noticed, however, that solutions can be obtained for any plane train of body waves (at arbitrary angles of incidence), for surface waves, or for a combination of various wave fronts. The main reason vertically polarized waves are used in practice is not the simplicity of the model (although this is an important consideration), but the fact that currently there is not enough information on the wave content of a potential earthquake at the design site.

Two basic procedures can be used to compute compatible motions:

- o A direct analysis in which the motions (and stresses) at various heights and at the surface are computed from the motion at the bottom (normally referred to as bedrock) or the motion at a hypothetical free surface of an underlying half space (normally referred to as an outcropping). This procedure is known as convolution.
- o An inverse analysis in which the motion at the bottom (bedrock) or at a hypothetical free surface of an underlying half space (outcropping) is computed from the motion at the surface. This procedure is known as deconvolution.

For a linear system, it is particularly convenient to perform these analyses in the frequency domain, obtaining then the transfer function from the bottom to any point or from the surface to any level. The transfer function from the surface to the bottom of the profile is just the inverse of the transfer function from the bottom to the surface.

3.1.6 Nonlinear Soil Behavior

It has long been recognized that, in order to study the propagation of seismic waves through a soil deposit for moderate or large earthquakes, it is necessary to account, at least approximately, for nonlinear soil behavior. Two procedures can be used for this purpose:

- o Iterative linear analyses making use of equivalent linearization techniques. After each analysis, maximum strains are computed at representative points for each soil sublayer, finite element, or finite difference mesh. (If the solution is carried out in the frequency domain, this implies converting to the time domain to obtain time histories of strains and scanning for the maximum value of each record.) An alternative, which is more economical, is to obtain root-mean-square strains. From experimental curves relating shear modulus and damping to shear strain, values of these two parameters, corresponding to a characteristic strain (typically two-thirds of the maximum), are obtained. A new analysis is then performed using the

soil properties so determined. The process continues until values of the strains or soil properties computed in two consecutive cycles differ by less than a specified tolerance (typically 5% or 10%). This procedure is implemented in computer programs such as SHAKE and FLUSH (University of California, Berkeley), going from the surface of the soil down (deconvolution). SHAKE uses a continuum solution for each linear analysis, FLUSH a finite element model. These two programs have been extensively used.

o Nonlinear analyses in the time domain, using an appropriate set of constitutive equations for the soil. This alternative is implemented in the program CHARSOIL (University of Michigan), which has enjoyed some popularity in recent years. Constitutive equations for the soil are based on a Ramberg-Osgood model, and the solution is carried out by the method of characteristics. A simpler approach is to model the soil deposit with finite elements, or lumped masses and nonlinear springs (a simple shear-beam model). These solutions proceed normally from the bottom up (convolution), although Jakub has shown that they could also be applied for the inverse problem. In this latter case, however, it is necessary to obtain derivatives of the results, which leads to a problem which is not well conditioned.

A question exists as to the validity of the iterative procedure and the accuracy of its results. Initial studies by Constantopoulos, using a Ramberg-Osgood model for the soil and the curves of modulus and damping versus strain corresponding to the same model, indicates that for the convolution process the "true" nonlinear solution yields accelerations at the surface 10% to 20% smaller than the iterative procedure, but displacements (and strains) up to 50% larger. The characteristics of the motion at the free surface obtained by the two methods are relatively similar over the range of frequencies of practical interest. However, Constantopoulos studied only two relatively shallow soil profiles (100 ft), and maximum accelerations at the bottom of 0.35 g or less. More recent studies by Dobry indicate that the iterative procedure underestimates the response in the low-frequency range, because the longer-period surface waves arriving at the end of the record are treated with the same values of damping used for the portion of the record

corresponding to the higher intensity of excitation and r ponse. These results are consistent with the conclusions of Constantopc.los.

On the other hand, Richart compared results obtained using SHAKE and CHARSOIL and found that iterative solutions as implemented in SHAKE yield amplified response spectra at the surface (for the convolution process) which are much lower than those obtained with CHARSOIL in the high-frequency range. Maximum surface accelerations would then be badly underestimated. A recent report by D'Appolonia shows similar results and concludes that the iterative procedure (or SHAKE) is applicable for the case of relatively shallow soil deposits and small earthquake excitations, but that its results are unreliable for deep profiles and high-intensity motions. A similar conclusion was reached by Dames and Moore, comparing SHAKE and STEALTH, but on the basis of entirely opposite findings. Their results indicate that for deep profiles and high levels of excitation the iterative solution may overestimate by a factor of two or more the maximum surface acceleration and the values of the response spectra in the high-frequency range.

It is apparent that the accuracy of the iterative procedure is open to question and that assuming the same damping for all frequencies, independently of the variation of the response amplitude, may not be correct. On the other hand, the contradiction in the results obtained by nonlinear time solutions with CHARSOIL and STEALTH is also disturbing. Several factors must be considered in interpreting these results. Joyner and Chen, in a study of the nonlinear response of soil deposits, found that their solution had a very large high-frequency content due to numerical inaccuracies (noise). They concluded that in order to avoid these spurious components it was necessary to include some viscous damping. It is clear that overshooting and backtracking effects, due to inaccurate definition of the times at which changes in stiffness occur, can introduce erroneous high-frequency components. In his studies, Constantopoulos used a fourth-order Runge-Kutta method and corrected for overshooting and backtracking at each of the four intermediate steps for each time increment. It is not known how this problem is handled in the other computer programs. If nothing is done about it, high-frequency components may be exaggerated. If viscous damping is introduced to filter them out, the filtering may be excessive.

In the Dames and Moore study, it is stated that the Ramberg-Osgood model is not adequate to simulate nonlinear soil behavior, because it does not have complete memory. Constantopoulos implemented a Ramberg-Osgood model with complete memory, keeping track of all reversal points and ensuring that all loops (external and internal) were stable. It is not known how this aspect is handled in CHARSOIL.

Because of lack of knowledge on the details of implementation of the various computer programs, it is hard to reach a conclusion on the magnitude of the errors caused by the iterative procedure or to judge its range of applicability.

3.1.7 The Deconvolution Process

A second point of concern is related to the application of the deconvolution process. Figure 3-5 shows typical transfer functions from the bottom to the surface for a homogeneous soil layer. The transfer functions from the surface to any depth would be the inverse, with shapes as shown in Fig. 3-6. The amplitude of these transfer functions increases without bound for increasing values of the parameter $\omega z/c_s$, where ω is the frequency in rad/s, z is the depth, and c_s is the shear-wave velocity of the soil, if the system has some internal damping of a hysteretic nature (the increase would be even faster for viscous-type damping). As the deconvolution process proceeds down the soil profile, layer by layer, the amplitudes of the high frequencies continuously increase and will eventually cause numerical problems (overflows). This problem is aggravated by the fact that with increasing amplitudes of the high-frequency components the effective shear-wave velocity of the soil decreases and the effective damping increases in a nonlinear or iterative analysis. If the motion specified at the free surface has a finite amplitude content in the range of frequencies up to 33 Hz, the solution may only proceed (with believable results) to depths of 50 or 100 ft. To avoid this limitation, it has become common practice to truncate the Fourier spectrum of the design motion at the free surface above a certain threshold frequency. For shallow layers and low- to moderate-intensity motions, the

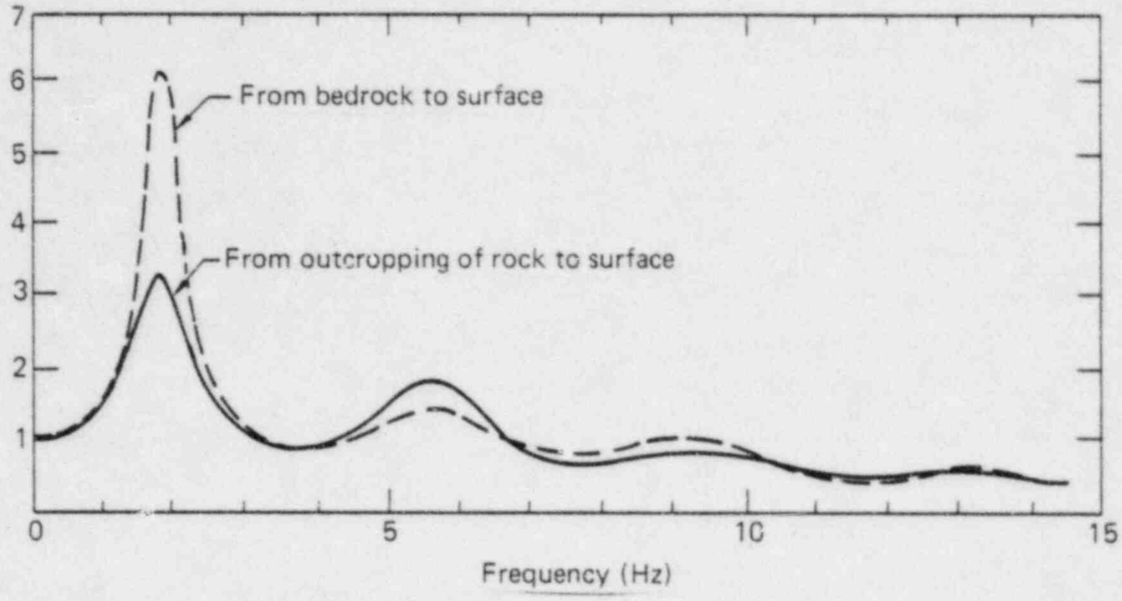


FIG. 3-5. Typical convolution amplification functions.

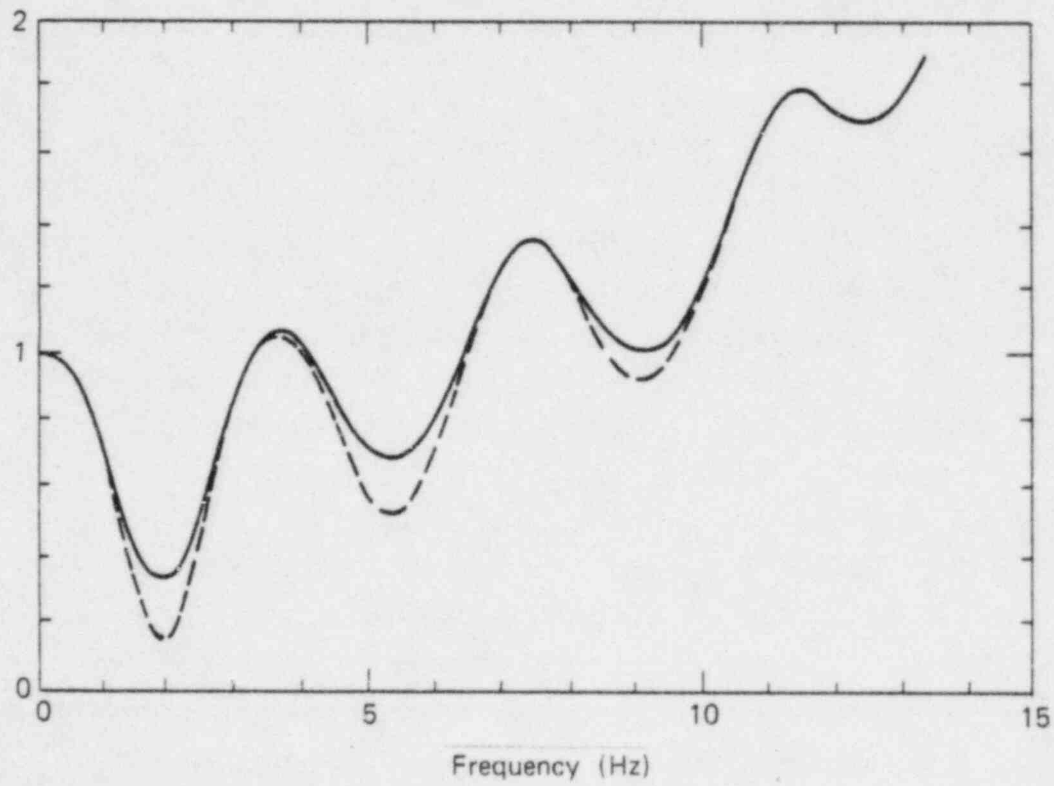


FIG. 3-6. Typical inverse (deconvolution) amplification functions.

threshold frequency may be about 20 Hz. For deep, soft soil profiles and moderate to high seismic excitations, it may have to be as low as 15 or even 8 Hz.

The most important consideration in the determination of the compatible motions is that, when these motions are applied to the discrete soil model to be used for the soil-structure interaction analyses, without structure or excavation, the specified motion should be reproduced at all nodes along the surface (if the design motion was specified there, as is normally the case). Truncation of the high-frequency components violates this requirement. If the threshold frequency is about 20 to 25 Hz, the consequences will not be serious (one must keep in mind that our knowledge of the actual frequency content of real earthquakes above this range is very limited). If the threshold is as low as 10 Hz, there is cause for serious concern.

When dealing with nonlinear problems, the solution may not be unique and may depend on the initial conditions. This situation is also encountered in the typical deconvolution process. Depending on the soil properties assumed in the first analysis (modulus and damping), the iterative procedure may converge to different results. An alternative to the deconvolution which may be worth considering is to start with an assumed motion at the bottom (or outcropping), perform a direct analysis, compare results at the surface with the desired motion, and introduce corrections in the input. In the cases where this procedure has been used, performing each analysis in the time domain with a nonlinear soil model, it has produced satisfactory results after four or five cycles. More studies are, however, necessary to evaluate this alternative (notice also that if this step is performed in the time domain, so should the complete analysis for consistency). Additional work is also needed on the causes of the deconvolution problem. The fact that a deconvolution, even for a linear system, produces unrealistic results below a certain depth may indicate basic limitations of the one-dimensional solution (the assumption of a single train of vertically polarized waves) or the inappropriateness of considering a constant value of damping for all frequencies (the point discussed above in relation to the iterative analysis).

A final point which must be considered is that in order to conduct consistent analyses the determination of the compatible motions must be

performed with the same mathematical model used for the study of the complete soil-structure system. Performing the deconvolution with SHAKE (continuum solution) and the main analyses with LUSH or FLUSH (finite element formulation) introduces errors whose magnitude depends on the degree of refinement of the mesh. Both analyses should also be conducted with the same procedure (time or frequency domain).

3.1.8 Solution Schemes

Once appropriate models have been developed for the soil and the structure and compatible motions (and stresses if needed) have been computed at the boundaries, the solution of the complete soil-structure system can be obtained in one of three ways:

- o Direct integration of the equations of motion in the time domain.
- o Direct solution in the frequency domain.
- o Modal analysis in the time or frequency domain.

Time Integration. Time integration is implemented in computer codes TRANAL (finite elements), DAMSEL, and STEALTH (finite differences), and in a number of programs developed for research purposes at academic institutions. This method is particularly appropriate, and in fact is the only rigorous one if it is desired to conduct a nonlinear analysis using an adequate set of constitutive equations for the soil. Material damping for the soil then derives naturally from the nonlinear behavior, although sometimes additional damping is put into the system to prevent the appearance of high frequencies (this was mentioned in the previous section). If viscous dashpots are placed on the boundaries, they can be directly assembled into the damping matrix. A problem arises, however, if the structure is linear and some structural damping is to be included. A solution often adopted for forming a damping matrix is to assume that it is a linear combination of the mass and stiffness matrices (the so-called Rayleigh damping). This solution is acceptable if care is taken to ensure that the damping remains within acceptable bounds over the complete frequency range of interest. Frequencies above and below this

range will be overdamped. Alternatively, one can obtain the natural frequencies and mode shapes of the structure on a rigid base and compute a damping matrix that produces any specified amount of damping in each mode.

The problem of selecting an appropriate damping matrix is more serious when performing a linear analysis in the time domain, since both structural and soil (material) damping must then be reproduced: the soil damping is generally different for the various layers and is of a hysteretic nature (i.e., it is frequency independent). Forming a damping matrix which maintains these properties requires some careful thinking (assembling it from individual damping matrices for each finite element of the Rayleigh type may not produce the desired effects for the complete system). It should be remembered in any case that this matrix is intended to reproduce the internal material damping. Any viscous dashpots at the boundaries to simulate radiation effects would then be assembled directly into the damping matrix.

Some thought should also be given to the numerical integration procedure to be used. Implicit methods which are unconditionally stable have the advantage that the time step of integration is not limited by stability considerations. When their stability is achieved by introducing fictitious damping into the system, as in Wilson's theta method (which has been used in the past for the dynamic nonlinear analysis of soil deposits), the results may be dangerously unconservative. Explicit procedures, such as the central difference formula, are particularly convenient for this type of problem if lumped masses are used (it has been shown that, for finite elements with linear displacement expansion, lumped masses are at least as accurate as consistent mass matrices). In this case, it is not necessary to assemble any matrices, and the computations in each step are extremely simple. While these methods require a very small time step of integration (typically about 0.002 s) for stability, this normally assures adequate accuracy of the results.

The main drawback of analyses in the time domain is that, currently, transmitting boundaries with the accuracy of the consistent boundary are not available (the Smith boundary used as suggested by Dames and Moore can provide a solution, but it needs further evaluations). With elementary or viscous boundaries, the edges of the domain must be placed at a sufficient distance from the structure, and the number of degrees of freedom of the model is

large. In spite of this, analyses in time with two-dimensional models can be competitive with frequency analyses, particularly for deep profiles (with large bandwidths). For general three-dimensional models in Cartesian coordinates, time analyses with an explicit integration scheme are the most sensible solution and perhaps the only possible one. Analyses in the time domain are better conducted with a nonlinear soil model, even if an approximate one. (This does not increase the cost of computation, and it avoids the problems of forming a damping matrix.)

For research purposes, three-dimensional analyses with nonlinear constitutive equations are needed. Currently, however, the lack of a proven, reliable nonlinear soil model and the number of uncertainties involved in seismic analyses makes this approach too expensive and too hard to justify for design purposes.

Solutions in the Frequency Domain. Analyses in the frequency domain are implemented in the computer programs LUSH, FLUSH, and TRIAX. Since this type of analysis is based on the principle of superposition, it is only applicable to linear systems. Nonlinear soil behavior can then be estimated in two different ways:

- By determining from the computation of the compatible motions values of the shear modulus and damping for the different soil layers, consistent with the levels of strain induced by the seismic motions in the free field (without the additional vibrations caused by the structure). These are sometimes referred to as primary nonlinearities. A single linear analysis is then performed for the complete soil-structure system. Damping can have any variation with frequency, but must be independent of amplitude in this model. The normal approach is to assume that the damping is independent of frequency (linear hysteretic damping), an assumption consistent with experimental results. On the other hand, it is clear that material damping is, in reality, a function of amplitude, and the amplitudes of the different frequency components are not the same. This is the same

point mentioned in the discussion of the iterative linear analyses for convolution or deconvolution processes. Linear hysteretic damping is modeled by using a complex modulus.

- By performing iterative linear analyses (such as those described earlier) on the complete model. If there are questions about the validity of the iterative procedure for one-dimensional situations, these questions become more serious for two- or three-dimensional states of stresses. In these cases, it is not clear what shear strain is to be used to determine consistent values of modulus and damping. Normally, the maximum shear strain is used, but this strain is changing direction as well as magnitude with time. It is necessary, in addition, to define the variation of a second parameter (Young's modulus, the bulk modulus, the constrained modulus, or Poisson's ratio) and the damping associated with volumetric deformations, as well as the coupling between both types of deformations. Normally, coupling effects are neglected. In LUSH and FLUSH, it is assumed that Poisson's ratio remains constant, which implies that Young's modulus and the bulk modulus decrease with increasing shear strain proportionally to the shear modulus. It also implies large values of damping in volumetric deformation due to shear strain. These effects do not seem logical. A different approach is to assume that the bulk modulus or the constrained modulus remains constant. This solution is more sensible for saturated soils. In this second case, Poisson's ratio increases with shear strain, causing some changes in the behavior if its original value is about 0.3.

For two-dimensional analyses, the fact that motions in two orthogonal directions must be considered separately introduces further limitations on the validity of the iterative procedure (or for that matter a true nonlinear analysis).

Analyses with cylindrical models (such as TRIAX) are intended for linear systems and normally use the first approach, therefore neglecting the additional nonlinearities caused by the vibrations of the structure (sometimes referred to as secondary nonlinearities). It is possible to simulate the

effect of the additional nonlinearities by considering an average of the maximum shear strains over the circumference and by assuming that the computed soil properties (moduli and damping values) apply in an average sense over the complete ring. The improvement in accuracy introduced by this procedure is, however, questionable.

Analyses with two-dimensional (LUSH) or pseudo-three-dimensional (FLUSH) models perform iterations on the complete model. Due to the limitations of this procedure, whether the increased cost of computation is justified is highly debatable (three to five cycles, representing three to five complete analyses, are typically carried out). Comparative studies performed by Ettouney for a two-dimensional model indicate that including or neglecting secondary nonlinearities for moderate- to high-intensity motions makes little difference in the results (not only gross effects but even amplified response spectra). For earthquakes of very small intensity, where the nonlinearities caused by the seismic waves are minor and can no longer be considered as primary, the differences would be larger. For most practical cases, one can expect results within 20% of each other using either approach (the analyses with only primary nonlinearities will normally be on the conservative side).

Analyses in the frequency domain are performed by:

- Computing the Fourier transform of the input motion. The consistent motions at the boundaries (their Fourier transforms) are expressed as the product of a function of frequency and the transform of the input.
- Determining the steady-state response of the soil-structure system, assuming a harmonic excitation for a number of frequencies. This requires for each frequency the solution of a system of linear equations with complex coefficients. For each desired effect (acceleration, displacement, or force), the values obtained for the various frequencies form a transfer function.
- Multiplying the transfer function of each desired effect by the Fourier transform of the input motion and obtaining the inverse Fourier transform. This provides the time history of the desired response parameter.

The computation of the Fourier transform is done with fast Fourier transform techniques. A number of points which is a power of two, corresponding to equal time and frequency increments, must be used. In practice, this number is often taken equal to 2048. If the motion is digitized at increments of 0.02 s and has a total duration of 30 s, this allows for about 10 s of zeroes at the end of the record (these trailing zeroes are needed to allow the free vibration components of the solution to be damped out). If the motion is digitized at increments of 0.01 s, this only allows for some 20 s of earthquake without any trailing zeroes. Use of a minimum of 4096 points and a Δt of 0.01 s is recommended (even 8192 points may be considered). It should be noticed that for 2048 points and $\Delta t = 0.02$, the frequency increment is of the order of 0.025 Hz, and the maximum frequency is about 25 Hz (there are 1024 complex points); however, frequency components of the motion larger than 10 or 12 Hz cannot be accurately reproduced. With $t = 0.01$ s, frequencies of up to 20 or 25 Hz are well reproduced, but the frequency increment is 0.05 Hz and the lack of trailing zeroes implies that the free vibration terms will distort the solution.

The computation of the transfer functions, i.e., solving the complete system of equations for each frequency, is never performed for all points of the Fourier transform (whether 1024, 2048, or 4096). Normally, the solution is performed at a frequency increment much larger than that of the Fourier transform and intermediate results are obtained by interpolation. Ideally, the frequency increment should be variable (smaller in the immediate neighborhood of the first few resonant frequencies or peaks in the transfer functions and larger in the high-frequency range). It is even possible to detect automatically during the solution the location of peaks and to adjust the frequency increment in their neighborhood, computing intermediate values. A solution of this kind was implemented by Scaletti. In most cases, however, the frequency increment is constant. Some checks should then be performed, estimating with simplified methods the first fundamental frequencies and verifying that the peaks are well reproduced. Otherwise, the interpolation procedure (whether applied on the transfer function or the inverse) may lead to important errors.

When the structure is nearly axisymmetric and the primary nonlinearities are significant, an analysis in the frequency domain using the cylindrical formulation, with consistent boundaries placed very close to the edge of the foundation and with soil properties obtained during the computation of the boundary motions, is particularly efficient and accurate. Two-dimensional models can place the consistent boundaries at the edge of the footing for linear analyses. If secondary nonlinearities are taken into account, which in most cases will not be necessary, the boundaries must be placed at a distance where these additional effects become negligible. In addition, as pointed out in a previous section, the dimensions of the equivalent footing (the thickness of the soil slice) should be selected so as to maximize the agreement with the key parameters of the actual three-dimensional foundation.

Modal Analysis. Modal analyses in the time or frequency domain are rarely performed with the direct approach. The soil-structure system, with different values of damping for the various layers, does not generally have normal modes in the classical sense. While it is possible, in the frequency domain particularly, to work with complex modes, their physical interpretation is difficult. If the damping is ignored and the free vibrations of the system are studied, normal modes are obtained, but they are still hard to interpret. There are typically groups of modes with very similar frequencies, and a large number of modes is needed to obtain an accurate solution. Damping in this case has to be estimated for each mode by weighing the contribution of the different components, a procedure which is only approximate.

3.2 THE SUBSTRUCTURE APPROACH

Figure 3-7 shows in a very schematic way the three steps involved in a solution with the substructure approach. While only translational motions in the plane and rotations around an axis perpendicular to the plane are shown in the sketch, it should be noted that the situation can be much more general, considering six degrees of freedom for a rigid foundation or any number of components for a flexible one. In the same way, while in the last step the

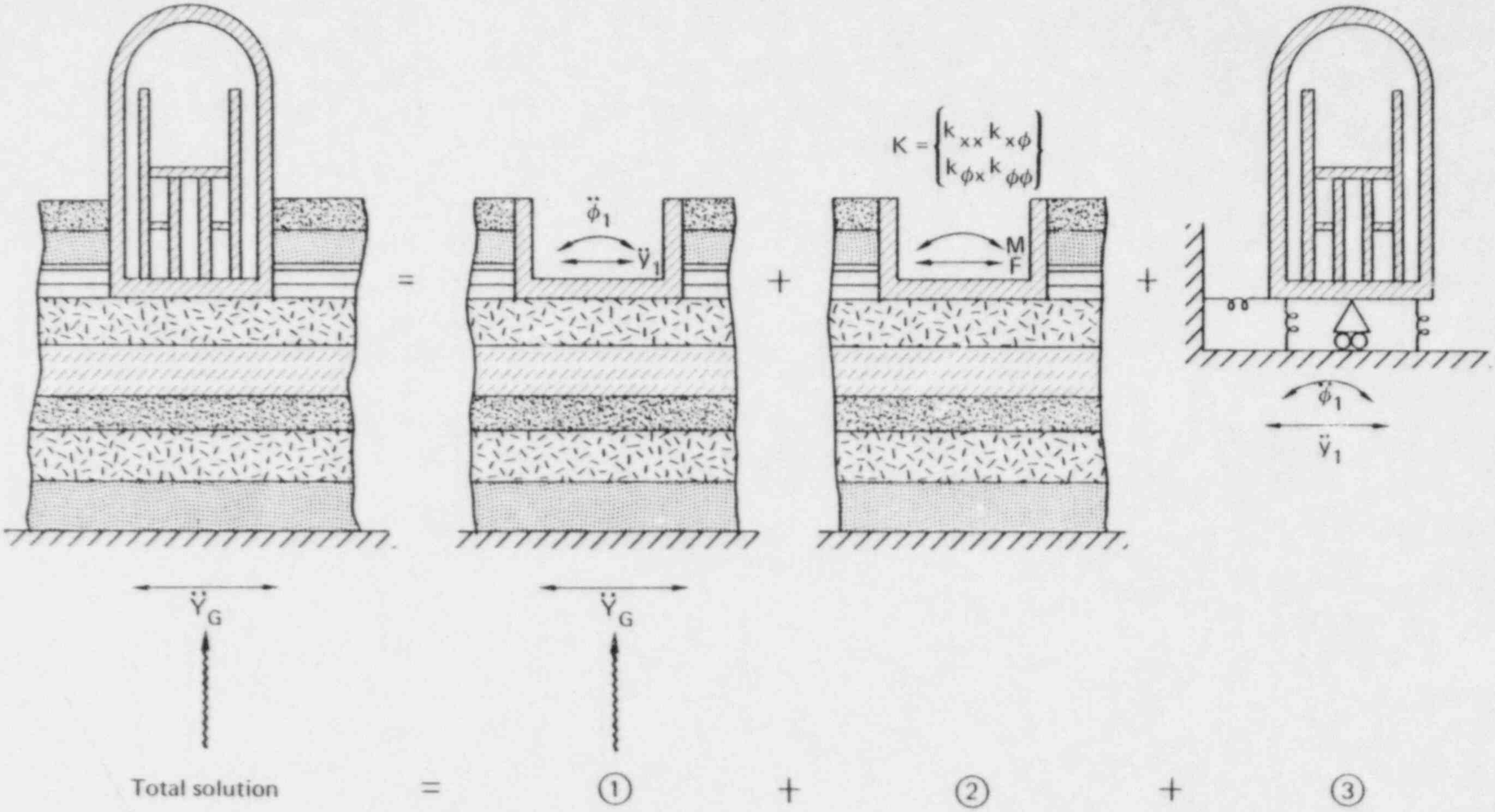


FIG. 3-7. Schematic of the three steps that constitute the substructure approach to analyzing soil-structure interaction.

foundation is represented by springs and dashpots for the sake of simplicity, these springs are intended to represent full stiffness matrices whose terms are complex functions of frequencies.

Since the basis for a solution by the substructure approach is the superposition of the results of individual steps, it is limited to a linear system (particularly with a cylindrical formulation), in much the same way as a direct solution in the frequency domain. Nonlinear soil behavior must be accounted for by including only primary nonlinearities, although iterative schemes to include effects of secondary nonlinearities can be applied in some cases.

The decisions which have to be made in the selection of appropriate models and solution procedures for the substructure approach are very similar to those discussed in some detail for the direct approach. There is, however, more flexibility in the use of different models for each step: for instance, selecting a two-dimensional model for the first step, where the agreement between 3D and 2D solutions may be acceptable, still allows the use of three-dimensional idealizations for the other two steps; discrete models may be used for one phase and continuum analytical solutions for another; and solutions in the time domain, frequency domain, and with modal analysis can be combined.

A second important characteristic of the substructure approach is that intermediate results can be evaluated, checked for numerical accuracy or gross mistakes, and smoothed or modified to account for uncertainties (though some care must be exercised to avoid introducing excessive conservatism). As a result, it is easier to identify the key features of the response and the key parameters contributing to the observed behavior.

If secondary nonlinearities are neglected in both methods, the substructure approach can be made to provide exactly the same results as the direct procedure. If secondary nonlinearities are considered in the direct approach, there will be some differences in the results (even if they are also incorporated in an approximate way in the substructure method). These differences will be minor for earthquake motions of moderate or high intensity; they will be larger for very low intensity earthquakes, but in this case it would be hard to say which solution is more accurate (the significance

of the seismic forces in the design would also be smaller, and introducing extra conservatism may not be serious).

3.2.1 Selection of Soil Properties

If only primary nonlinearities are to be considered, a preliminary analysis is needed to determine appropriate soil properties (values of modulus and damping) to be used in the first two steps. This analysis is similar to that performed for the determination of the compatible motions and is therefore subject to the same comments and limitations (types of waves, convolution versus deconvolution, validity of the iterative approach, etc.). It should be noticed, on the other hand, that the soil properties are less sensitive to these factors than are the details of the compatible motions. For simple, typical soil profiles, it is even possible to derive simplified procedures to obtain the variation of properties with depth as a function of the peak surface acceleration (along the lines of work done by Jakub).

This preliminary analysis is normally conducted on a one-dimensional model (assuming waves propagating vertically), in the frequency domain, and using the iterative approach. If the analysis were carried out in the time domain, using a nonlinear soil model, it would be necessary to select characteristic average values from the variation of modulus and damping with time, or to conduct the first step in the time domain with the same model. In this last case, compatible motions would again be needed.

3.2.2 Determination of the Foundation Motions

For a rigid surface foundation (or for a structure with very shallow embedment), the input earthquake specified at the free surface of the soil deposit can be assumed to apply directly at the foundation level (for a massless foundation and no structure) as long as the motion is assumed to be caused by vertically propagating waves (uniform motion in phase at all points in a horizontal plane). The first step of the substructure approach can then be bypassed.

When other types of waves are considered, even for surface foundations, there is a filtering of the translational motions and the appearance of rotational components (both rocking and torsional motions are possible, depending on the trains of waves). These effects would decrease as the flexibility of the foundation increases, but then different points of support of the structure would have motions out of phase.

For embedded foundations, the filtering of the translational motions and the appearance of rocking components take place even under the normal assumption of vertically propagating waves. The rotations are caused by the lack of shear strains compatible with those of the free field along the sides of the excavation.

It has sometimes been argued that the determination of the foundation motions as normally done involves an approximation because of the assumption of a massless foundation. It is important to clarify this misconception. If the model of the structure used for the soil-structure interaction analysis in the third step includes the foundation and its mass, with the stiffness matrix representing the flexibility of the soil attached to the bottom of the foundation (or to the points of contact between the foundation and the soil), a massless foundation must be considered in the first step. Alternatively, if the structural model does not include the foundation, then the slab and sidewalls must be included with their mass in the first two steps. The stiffness matrix computed in the second step should then apply to the top of the mat or to the contact points between the various structural components and the foundation. Both approaches are clearly possible. For a rigid foundation (an assumption which is normally valid when dealing with a single structure), only six components of motion must be determined in the most general case (three translations and three rotations). In this case, it is usually more convenient to include in the first two steps the kinematic constraints corresponding to the condition of a rigid foundation, but to ignore the mass (to be included in the structural model). When dealing with a flexible foundation (a large mat supporting several structures), it would be necessary to select a sufficiently large number of contact points between the foundation and the soil and to determine up to three components of motion at each one of these points. In this case, it may be more convenient to include the model of

the foundation with its stiffness and mass and to determine the appropriate components of motion at the points of contact between the structures (or structural subassemblies) and the foundation. The same model must then be used for the second step.

A second point worth mentioning is that the motions of the foundation without structure, often referred to as motions at the foundation level, should not be confused with the motion that occurs in the free field at the depth of the foundation. For a one-dimensional soil model, the latter has only a translational component. The transfer function from the surface to this depth is similar to those shown earlier for the bottom of the soil profile, the depth being the only difference. At the natural frequencies of the embedment layer, it exhibits deep valleys (zero values if the soil had no internal damping), and in between it has peaks whose amplitude increases with frequency if there is some material damping. The maximum acceleration is smaller than that specified at the free surface up to a certain depth, but could be larger for substantial depths if the high frequencies are not truncated. The large oscillations of this transfer function has been a cause of concern and has led to the imposition of limitations on the foundation motion (typically, it is required that the response spectra of this motion be no less than 60% of the corresponding spectra of the surface motion at any frequency). It should be noticed, however, that the geometry of the excitation gives rise to scattering of waves, creating a more complex two- or three-dimensional situation. The transfer function for the translational motion at the base of a rigid, massless foundation would still have some oscillations with frequency, but it would be much smoother than that of the free field. In addition, there is the rotational component, which is rarely mentioned, but which would contribute importantly to the structural response. A limitation on the deamplification that can take place at different frequencies may still make sense to account for the variation in the results due to the type of waves. However, the restrictions should not impose excessive conservatism. These limitations would also cause variations in the results between solutions with the direct and the substructure approach. (It is hard to impose them on the former method, since the motion without the structure is not computed; for the direct approach, the conditions would have

to be enforced on the free-field motion at the foundation level or on the foundation motion when the structure is included, alternatives which are not equivalent.)

A variety of methods can be used to determine the foundation motions. For a linear, horizontally layered soil deposit, knowing the motion at the surface, it is possible to determine the amplitude (as a function of frequency) of any specified train of waves which would produce this motion (free-field conditions). The foundation motions are then obtained by imposing a disturbance over this solution which, when superimposed on the free-field conditions, satisfies a situation of zero stresses along the excavation (or on the sides of the foundation). This problem is essentially identical to that which must be solved in step 2 to determine the foundation stiffness (except for the existence of the initial state of stresses and displacements due to the free-field motions). If the problem is to be solved in the frequency domain, which is normally the case, the foundation is typically discretized by a sufficiently fine mesh of points. If U_o and P_o are the displacements and equivalent forces (stress resultants) at the mesh points due to the free-field condition, and F is the flexibility matrix of the foundation (displacements at all mesh points due to unit forces), the condition at each frequency is

$$U = U_o - FP_o \quad ,$$

where the set of U's are the desired amplitudes of motion of the foundation nodes. In this general case, F would have three degrees of freedom per node for a 3D model and two degrees of freedom for a plane condition. If the foundation is assumed to be rigid and the corresponding constraints are obtained by expressing the displacements U of the various nodes in terms of the displacements U* of a fixed point (the center of the slab, for instance) in the form

$$U = LU^* \quad ,$$

then the results of the nodal forces at the same point are given by

$$P^* = L^T P \quad .$$

Then

$$U^* = (L^T K L)^{-1} L^T K U_o - (L^T K L)^{-1} L^T P_o \quad ,$$

where $K = F^{-1}$ is the stiffness matrix of the foundation before the rigid body conditions are imposed, $L^T KL$ is the stiffness matrix of the rigid foundation, and $(L^T KL)^{-1}$ is the corresponding flexibility matrix. This formulation is particularly convenient if the stiffness (or flexibility) matrix of the foundation is to be obtained by a continuum, semianalytical approach. If a dicrete model (finite elements or finite differences) is to be used for step 2, it is more convenient to express the foundation motions in terms of the dynamic stiffness matrix of the domain and the compatible motions at the boundaries (determined in the preliminary analysis), as done by Elsabee and Morray. The same type of model is used if the solution is to be carried out in the time domain.

3.2.3 Determination of the Foundation Stiffnesses

The second step in a solution using the substructure approach is the determination of the foundation stiffnesses. For a rigid foundation, the dynamic stiffnesses are typically obtained by applying unit harmonic displacements and rotations to the foundation and computing the resulting forces and moments (solution in the frequency domain). In the general case of a rectangular foundation or a slab of arbitrary shape, six degrees of freedom must be considered. For a circular or square foundation, on the other hand, only five terms have to be computed (horizontal force and moment due to a unit horizontal displacement, moment due to a unit rotation, vertical force due to a unit vertical displacement, and torsional moment due to a unit torsional rotation). For a flexible foundation, it would be necessary to apply unit harmonic displacements in three orthogonal directions at each of a number of

contact points between the foundation and the soil and to obtain the corresponding forces at all these points. Alternatively, in both cases, one could apply unit harmonic forces and moments and compute the corresponding displacements and rotations. These would then be the coefficients of the flexibility matrix, and the stiffness matrix would be obtained by inversion.

Available computational procedures include the following:

- For a circular foundation on the surface of a deep soil profile with relatively constant properties, the foundation stiffnesses have been computed and tabulated (Veletsos and Wei, Veletsos and Verbic). These solutions can also be used with sufficient accuracy for square or nearly square foundations (defining an equivalent radius) on the surface of a basically homogeneous deep soil deposit, or with a small embedment. Approximate solutions which have in many cases more than sufficient accuracy for practical purposes can be obtained for other cases from these basic solutions (simplified procedures are discussed in the following chapter).
- Continuous, semianalytical-type solutions and discrete models can be used to determine the foundation stiffnesses for arbitrary cases. The basis for most analytical-type solutions is the determination of the displacements and stresses at any point within the soil domain due to a unit force applied at the point of the foundation-soil contact. The analytical form of this influence (or Green's) function is not available for the general case. Approximate forms are available, however, for some cases, and the function can be evaluated numerically for others. For surface foundations on an elastic half space, Luco and Wong have presented expressions for this function in terms of double integrals from minus infinity to plus infinity. For a layered half space or a layered soil deposit of finite depth, solutions can also be obtained by expanding the point force (or a pulse) into a Fourier series, as done by Chopra for the two-dimensional case and by Gazetas for the two- and three-dimensional cases. The analytical solutions for each term of the series can be readily computed and superimposed. More recently, Luco, using Harkreider's formulation,

derived a procedure to obtain Green's function numerically for a point force applied at any point within a layered hysteretic medium. This allows one to compute the stiffness matrix for embedded foundations of arbitrary shape (appropriate matrix transformations are necessary to take into account the existence of the excavation). This type of approach has been used by a number of authors to derive the stiffness coefficients for various kinds of foundations: Lysmer used it for a circular surface foundation, dividing it into concentric rings; Elorduy et al. applied it to the solution of rectangular foundations; Chopra and Gazetas, to strip footings; Luco and Wong, to foundations of arbitrary shapes.

- Alternatively, the influence function can be obtained numerically using a discrete model. Gonzalez and Vardanega used the cylindrical finite element formulation with the consistent boundary placed directly around a single cylinder with very small radius to obtain displacements due to a unit force, then computed with this formulation the stiffnesses of rectangular surface foundations. A variation is provided by using the boundary integral equation (or boundary element) method. In this case, a simpler Green's function, as that corresponding to a complete three-dimensional space, can be used. Dominguez computed the stiffnesses of surface and embedded rectangular foundations with this procedure.
- Discrete models can be used, on the other hand, to obtain the foundation stiffnesses directly. This approach is particularly convenient when dealing with rigid foundations, whether surface or embedded, since one can then avoid computing the stiffness or flexibility matrix corresponding to all mesh points, and obtain directly the reduced matrix with two, three, or six degrees of freedom. A plane-strain model, such as LUSH, can be used for strip footings or elongated rectangular foundations, and a cylindrical model, such as TRIAX, for circular footings. Surprisingly, the pseudo-three-dimensional model of FLUSH has not been used to compute the stiffness of rectangular foundations (a test case that would provide a better insight into the adequacy of this formulation by allowing comparison of results to those obtained with other

procedures). When a 2D model is used to compute the stiffnesses for circular or rectangular (nearly square) foundations, equivalent dimensions (width of footing and thicknesses of soil slice) should be carefully selected so as to match the key parameters controlling the response, as discussed for the direct approach. With these models, the solution is normally obtained in the frequency domain. The same comments made earlier for the direct approach in regard to bottom and lateral boundaries, mesh side, etc., apply here.

- An interesting alternative has been suggested by Frazier and Day. In their approach, foundation stiffnesses are computed with a discrete model (a finite element model was suggested, but the formulation would be equally valid for finite differences) by determining in the time domain the appropriate impulse response functions. In this case, absorbing boundaries do not have to be used as long as the boundaries are placed at a sufficient distance to ensure that the waves generated by the foundation and reflected at the edge do not have time to reach the structure again. By obtaining the Fourier transform of these impulse response functions, one would have the stiffness coefficients as functions of frequency.

The above list of procedures is not intended to be exhaustive. It is included here to illustrate the fact that a large number of alternatives are available, and that for a linear elastic soil deposit (with equivalent soil properties to account for primary nonlinearities) it is possible today to obtain the solution with any desired degree of accuracy for foundations of arbitrary shape, surface or embedded, rigid or flexible. It should be noticed, on the other hand, that these solutions may be expensive if one insists on a degree of accuracy which may not be justified, considering all other uncertainties involved. When dealing with a half space or with a soil deposit which can be well reproduced by a small number of layers with different properties, continuum-type solutions are particularly appropriate. When a large number of layers are needed to model the variation of soil properties with depth with sufficient accuracy, discrete models become more

advantageous. The cost of the latter is not affected by the variation in properties, while the cost of the former is highly dependent on the number of layers.

Economic considerations and availability of computer programs are clearly important factors in the selection of a model for this step and the previous one. One must keep in mind in addition the basic features of the physical problem which must be reproduced. Preliminary analyses with simplified models are always advisable to assess the adequacy of the selected procedure before embarking upon expensive computations.

3.2.4 Soil-Structure Interaction Analysis

The last step of the substructure approach corresponds properly to the soil-structure interaction analysis. In the most general case, for a flexible foundation, a three-dimensional model of the structure rests on a foundation represented by a frequency-dependent, complex stiffness matrix with three degrees of freedom for each contact point. This matrix is applied to the difference between the motions of the contact points for the complete model and the motions at these points computed in the first step. This model is rarely used in practice. Generally, the foundation is assumed to be rigid, and the corresponding matrix has six degrees of freedom or less (if horizontal excitations in two orthogonal directions and vertical motions are studied separately). The same three solution schemes discussed for the direct approach can be applied in this case.

Direct integration of the equations of motion is not as attractive in this case as for the direct solution, since nonlinearities cannot be completely considered. Furthermore, unless the foundation stiffnesses are represented by the impulse response functions and the problem is formulated in terms of integral equations, it is necessary to assume values of the real and imaginary stiffnesses independent of frequency. While the approximation introduced by the use of constant stiffness and damping terms may be quite acceptable, particularly if their values are selected at the frequency of interest (the fundamental frequency of the soil-structure system), this causes minor discrepancies in the results compared with those of a direct solution.

Solution in the frequency domain is particularly appropriate for the last step of the substructure approach, since the method itself implies linearity (or is based on the assumption of linear behavior). It is particularly easy in this case to account for the frequency dependence of the stiffnesses and their complex nature, and to incorporate different forms of damping (hysteretic or frequency-independent damping for structural and soil material nonlinearities; damping which varies with frequency for radiation effects). Furthermore, results are obtained first in terms of transfer functions which can be analyzed to detect fundamental frequencies, amount of effective damping, etc.

Modal analysis is often used in this last step. When performed in the frequency domain, it represents only a variation of the computational scheme from the direct frequency solution (one can then obtain complex modes and account for all effects discussed above). When performed in the time domain using classical modes, the procedure has the advantage that the results can be easily interpreted along the lines of normal dynamic analyses of structures. Inspection of the natural frequencies and corresponding mode shapes provides valuable insight into the nature and magnitude of the interaction effects, the relative influence of swaying (translational motion) and rocking, etc. It requires, on the other hand, not only the use of constant (frequency-independent) stiffness and damping terms for the foundation (as for the direct time integration of the equations of motion), but also the estimation of approximate values of damping for each mode. Several rules have been proposed to compute equivalent modal dampings. The weighted-average rule is based on equating the energy dissipated by the real system with that dissipated by the equivalent system when vibrating in a steady-state condition at a natural frequency. If this procedure is used, it is important to account for the nature of the various damping terms (frequency dependent or independent). Otherwise, if it is assumed that all damping terms are of a viscous nature (proportional to frequency), the results will be unconservative in the high-frequency range, whereas assuming all forms of energy dissipation (including radiation damping) to be frequency independent would yield unconservative results for low frequencies. An alternative, suggested by Tsai, is to select the damping in each mode so as to match the peak of the transfer function at a particular point.

CHAPTER 4

APPROXIMATE PROCEDURES

No method of analysis can be considered to reproduce exactly the true physical response of a structure subjected to uncertain loads. This is even more so in the area of seismic soil-structure interaction because of the large uncertainties involved in estimating the characteristics of the potential earthquake and the soil properties. However, analyses of nuclear power plants are conducted with as much sophistication and rigor as possible, within the limitations imposed by the availability of specific computer programs and the requirements imposed by codes (which introduce inaccuracies in order to guarantee a conservative design). The importance of nuclear power plants has led naturally to a desire to incorporate in practice results of academic research as soon as they are available, sometimes before they can be properly tested and evaluated and on some occasions generalizing them to situations for which they were not intended. There has also been, understandably, an excessive emphasis on the development of general computer programs, which can be qualified and used as a standard (even if they are drastically modified every year or two), at the expense of parametric studies which would shed more light on the significance of various variables or the derivation of simplified, engineering-type procedures which would allow one to estimate the importance of various effects. Approximate methods are nevertheless of great value, not only to obtain solutions which often have the desired degree of accuracy, but also to check the results of more complex analyses.

There are only a small number of simplifications which can be introduced in direct analyses of the complete soil-structure system, and they have already been discussed in the previous chapter. Examples include the use of a two-dimensional or pseudo-three-dimensional model instead of a true three-dimensional representation of the soil; the use of the equivalent linearization, with iterations carried out on the two-dimensional model or only on the one-dimensional wave-propagation problem, in place of a true nonlinear analysis; and the use of a crude model of the structure rather than a more detailed one. These simplifications are justified mainly on the basis

of lack of knowledge of the true nonlinear behavior of the soil and the lack of proven transmitting boundaries in the time domain. Three-dimensional finite element analyses, with an appropriate mesh size, are clearly more expensive than present two-dimensional solutions. However, the increase in cost would not be large enough to prohibit the use of these models if a real increase in accuracy (taking into account all uncertainties) could be guaranteed.

Approximate procedures are particularly suited for solutions using the substructure approach, where each one of the steps can be investigated, and the results interpreted, by more than one method. The flexibility of this approach allows one to combine solutions obtained by different procedures, as long as they are based on consistent assumptions.

The simplest model for a soil-structure interaction analysis is the one used in Chapter 2 to illustrate the basic features of the problem. The structure is reproduced as a single-degree-of-freedom system, with a mass lumped at an appropriate height and a shear or axial spring representing the structural stiffness. The parameters of this system (mass, stiffness, and height) may be chosen from simple physical considerations or so as to represent the first mode of the structure (this implies that a dynamic analysis of the structure on a rigid base has been performed previously, a practice which has some merit). In the most elementary form, the foundation stiffnesses are represented as springs (horizontal and rocking springs for a horizontal earthquake excitation, a vertical spring for vertical accelerations), their constants being determined from the available expressions for a rigid circular slab on the surface of an elastic half space (static values). The first natural period of the combined soil-structure system can then be estimated, as can the effective damping, including the contributions of the structural damping, the internal soil damping, and the loss of energy by radiation. If the structural model is intended to reproduce the first mode, it is assumed that higher modes are not affected by interaction effects.

This very simple model has sometimes been considered as synonymous with the substructure approach. Thus the distinction in regulatory guides between finite element solutions (implying a direct solution) and the lumped

spring-mass method (implying a substructure analysis). As a result, the substructure approach was only accepted for surface foundations, deep soil deposits which would be considered as a uniform half space, and very simple situations. It is clear that the above idealization is too crude for the dynamic analysis of most nuclear power plants: the structure cannot be reproduced well by a single-degree-of-freedom system because of the various components present, and because the soil is rarely uniform and the foundations are in most cases embedded. Even so, it is important to make three points:

- The substructure approach, as explained in the previous chapter, is much more general than a spring-mass model and can cover any practical situation within the limitations of a linear analysis.
- Between the most sophisticated forms of the substructure approach and the single-degree-of-freedom system with constant springs at the base, there are numerous possible variations which increase the accuracy while increasing only slightly the complexity. There are, for instance, various means to estimate the motions at the foundation level, accounting for the excavation; to compute stiffnesses for embedded foundations; and to account for the frequency variation of the different terms.
- While the simplest model is too crude for many practical situations, there are cases in which it may provide all the necessary accuracy. More importantly, it provides in all cases a means to estimate the importance of interaction and the effect of various parameters.

In the following discussion, some of the possible approximations in each of the steps of the substructure approach are discussed. The presentation is not intended to be exhaustive, but it is indicative of the many options available.

4.1 DETERMINATION OF THE MOTIONS AT THE FOUNDATION LEVEL

For foundations with small embedment ratios (i.e., where the ratio of the embedment depth to the base dimension is 15% or less), present regulations

allow one to consider directly the design motion at the free surface of the soil deposit as the one occurring at the foundation level. This is reasonable for average soils where the motion has been directly specified at the surface, since the effect of embedment is then small. It may not be as appropriate when dealing with very soft soils, particularly in the embedment region, when the surface motion is computed through amplification studies to include the effect of site characteristics.

In most practical cases, it is advisable to compute compatible motions at the foundation level, accounting for the geometry of the excavation. Elsabee and Morray conducted a series of parametric studies for circular foundations embedded in a finite layer of soil with uniform properties and obtained the transfer functions from the surface to the foundation level, assuming a train of vertically propagating shear waves. From these studies, they suggested a simplified procedure. The transfer function for the horizontal motion is given by

$$F_u(\Omega) = \begin{cases} \cos \dfrac{\pi}{2} \dfrac{f}{f_1} & \text{for } f \le 0.7f_1 \ , \\ 0.453 & \text{for } f > 0.7f_1 \ , \end{cases}$$

where $f_1 = c_s/4E$, c_s is the shear-wave velocity of the soil (over the embedment layer), f is the frequency of interest $(\Omega/2\pi)$, and E is the embedment depth. For the rotational motion, the transfer function is

$$F_\phi(\Omega) = \begin{cases} \dfrac{0.257}{R}\left(1 - \cos \dfrac{\pi}{2} \dfrac{f}{f_1}\right) & \text{for } f \le f_1 \ , \\ \dfrac{0.257}{R} & \text{for } f > f_1 \ , \end{cases}$$

where R is the radius of the foundation. These simple formulas are intended to provide smoothed average effects. The true transfer functions oscillate around these curves.

Similar studies have been conducted by El Khoraibie for strip footings and by Dominguez for rectangular foundations, considering not only vertically

propagating shear waves, but also trains of body waves at other angles. The results are very similar, on the average, with the lower bound (0.453 for Morray's case) increasing slightly with increasing internal soil damping and for other wave trains.

Present regulations impose a limit on the amount of reduction of the translational motion which can occur with embedment. Imposing such a limitation on the transfer function makes sense in order to account for uncertainties in the type of waves, soil conditions, etc. The simplified procedure suggested by Morray is particularly suited for this kind of consideration. One could for instance, in order to introduce some conservatism, take the transfer function for the translational motion as

$$F_u(\Omega) = \begin{cases} \cos \frac{\pi}{2} \frac{f}{f_1} & \text{for } f \leq 0.59 f_1 \ , \\ 0.6 & \text{for } f > 0.59 f_1 \ . \end{cases}$$

Currently, the limitations are being imposed on the response spectra ratio rather than on the transfer function, which does not make much sense. More importantly, the limitations are supposed to apply to the motion that would occur at the foundation level in the free field, which is not very logical, since this motion has no direct relation to that which accounts for the excavation, and it does not include rotational components. In fact, if the base rotation is neglected, it may not be appropriate to consider any reduction in the horizontal motion.

A procedure like that suggested by Elsabee and Morray, with further verification and possible improvements through more extensive parametric studies, would provide a simple means to obtain solutions which are reasonably approximate and which can account for some of the uncertainties. In its original form, the method was intended for uniform soil profiles. When the properties vary with depth, c_s should be a representative or average value of the shear-wave velocity over the embedment height. Alternatively, parametric studies could be conducted with typical soil profiles to determine the value of c_s or f to be used (to verify that f_1 should still be the

fundamental frequency of the embedment layer). For unusual soil profiles where thin layers of very soft soil may be interspersed with harder strata, individual studies should be conducted.

A second purpose of the motion studies is to estimate soil properties consistent with the expected levels of strain. When considering only primary nonlinearities, one-dimensional amplification studies in the free field are very simple and economical. Even so, the uncertainties involved in the soil characteristics and the accuracy of the iterative linearization scheme are such that even simpler procedures may be justified. Jakub conducted a series of parametric studies for uniform soil profiles and a soil deposit whose properties increased with depth, assuming in all cases a Ramberg-Osgood model for the soil. The constitutive equation was given by

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{\tau_y}\left(1 + \alpha\left|\frac{\tau}{\tau_y}\right|^{r - 1}\right) \quad ,$$

Where γ is shear strain, τ is shear stress, and r and α are empirical constants. The initial shear modulus, for very low levels of strain (of the order of 10^{-5} to 10^{-6}), is $G_o = \tau_y/\gamma_y$, and the modulus for a shear stress τ is

$$G = G_o \frac{1}{1 + \alpha\left|\frac{\tau}{\tau_y}\right|^{r - 1}} \quad .$$

The internal soil damping for the same stress level is

$$D = \frac{2}{\pi}\,\alpha\left(\frac{r - 1}{r + 1}\right)\left(\frac{G}{G_o}\right)\left|\frac{\tau}{\tau_y}\right|^{r - 1} \quad .$$

Jakub assumed in his studies a value of $r = 2$ for simplicity (the appropriate value for most soils would seem to be between 2 and 3), a value of $\gamma_y = 10^{-5}$, and a value of $\alpha = 0.05$. For these parameters, he obtained the variation of G/G_o and D with depth for the various soil profiles and for motions with the characteristics of the NBK spectra, as commonly used for the

seismic analysis of nuclear power plants. He then generalized his results, finding curves of $\tau_m/\rho az$ versus a/H, where τ_m is the maximum shear stress at depth z, ρ is the mass density of the soil, a is the peak ground acceleration at the free surface, and H is a characteristic depth. Knowing the value of τ_m and defining the characteristic stress as $\frac{2}{3}\tau_m$, it is then possible from the formulas to compute the corresponding values of G and D at various depths.

While the actual curves derived by Jakub need further verification through additional parametric studies (considering a larger number of earthquake motions and soil profiles), the procedure seems interesting for typical soil deposits (for unusual conditions, individual studies would always be required). Its validity is of course dependent on the consideration of only primary nonlinearities. Neglecting secondary nonlinearities (caused by the vibrations of the structure) is usually reasonable for moderate- or large-intensity earthquakes, but would be conservative for very light motions.

4.2 DETERMINATION OF THE FOUNDATION STIFFNESSES

The main improvements to be introduced in the simple model described earlier have four aims:

- To account for the frequency variation of the stiffness coefficients.
- To account for the nonuniformity of the soil profile (variation of properties with depth).
- To account for the shape of the foundation.
- To account for effects of embedment.

4.2.1 Frequency Variation of Stiffness Coefficients

For a surface foundation on a deep and relatively uniform soil deposit, which could be considered as a half space, the variation of the stiffness coefficients with frequency has been tabulated by Veletsos et al. A solution of the soil-structure interaction problem by a computer performing the computations in the frequency domain is straightforward and inexpensive.

Accounting in this case for the frequency variation of the stiffness functions offers no particular difficulty. However, if it is desired to obtain a solution (or an estimate of the solution) by hand, it is possible to perform a few cycles of iteration: starting with the static values, or the values corresponding to an estimated frequency, the natural period (and frequency) of the combined system is computed; the values of the stiffnesses at this frequency are obtained from the tables; and the process is repeated until the variation in results is less than the desired tolerance. It should be remembered in this context that the horizontal stiffness coefficients are almost independent of frequency, a significant variation occurring only in the rocking and vertical modes. Instead of the tables, one can use simplified, but rather accurate, expressions for the stiffness coefficients, as suggested by Veletsos and by Kausel. Similar tables or expressions are not available for foundations with other shapes, for embedded foundations, or for foundations on layered soils. Approximations for these cases are discussed next.

4.2.2 Nonuniformity of Soil Profile

It is clear that soil deposits rarely have uniform properties, particularly when considering the variation of modulus and damping with level of strain, that is, after the initial properties are modified to account for primary nonlinearities. Jakub conducted a limited number of parametric studies on strip footings (two-dimensional model) using the same soil profiles as in his amplification studies and the soil properties computed for different earthquake levels. From these studies, he concluded that a reasonable approximation to the foundation stiffnesses could be obtained by assuming a uniform soil deposit with the properties of the soil at a depth of 0.6B, if the soil deposit were originally uniform, or at a depth of 0.4B, if the original modulus increased with depth, where B is the half-width of the footing. Considering the uncertainties in the soil properties, it may be sufficient in all cases to take the adjusted properties at a depth of 0.5B, or 0.5R for a circular foundation.

While this approach needs further verification, it is consistent with many approximate methods in static geotechnical engineering problems. Combined with the procedure suggested also by Jakub to estimate strain-compatible soil properties, it provides an extremely easy solution which accounts at least qualitatively for the key features of the problem.

A more important consideration related to the layering characteristics of the soil deposit is the possible existence of a finite soil stratum resting on much harder rocklike material, in contrast to a half space. The key effects in this case are:

- o An increase in the values of the static stiffnesses.
- o A change in the frequency variation of the stiffness coefficients.

From parametric studies with circular foundations resting on the surface of a homogeneous soil stratum of variable depth, Kausel suggested formulas to account for the first of these effects:

$$K_h = K_{ho}\left(1 + \frac{1}{2}\frac{R}{H}\right) = \frac{8GR}{2 - \nu}\left(1 + \frac{1}{2}\frac{R}{H}\right) \text{ for the horizontal static stiffness,}$$

$$K_r = K_{ro}\left(1 + \frac{1}{6}\frac{R}{H}\right) = \frac{8GR^3}{3(1 - \nu)}\left(1 + \frac{1}{6}\frac{R}{H}\right) \text{ for the rocking static stiffness,}$$

where K_{ho} and K_{ro} are the half-space static stiffnesses, G is the shear modulus of the soil and ν its Poisson ratio, R is the radius of the foundation, and H is the layer depth. For vertical stiffness, Kausel and Ushijima have suggested

$$K_v = K_{vo}\left(1 + 1.28\,\frac{R}{H}\right) = \frac{4GR}{1 - \nu}\left(1 + 1.28\,\frac{R}{H}\right) \quad ,$$

whereas the torsional stiffness seems for practical purposes to be almost independent of R/H. More studies of this kind are needed to verify or refine these expressions for variable soil profiles. In the meantime, the above formulas can be used as reasonable approximations.

The second effect mentioned above can be discussed in terms of separate effects on the real and imaginary stiffness coefficients. For the real stiffness coefficients, the existence of a rigid or much stiffer boundary at a finite depth causes oscillations associated with the natural frequencies of the stratum. These oscillations are large when the soil has a very small amount of internal damping and are hard to reproduce through approximate formulas. However, when dealing with moderate to large seismic motions and internal soil dampings of about 7% to 10% or larger, the oscillations are much smaller. In this case, neglecting them, assuming the same frequency variation as for a half space, provides an acceptable approximation.

For the imaginary coefficients the key factor is the lack of radiation damping below the fundamental frequency of the stratum. This situation can be approximated by taking the imaginary terms as zero below the layer's frequency and as equal to the half-space values for larger frequencies (a solution which should be on the conservative side). This produces reasonable results unless the natural frequency of the combined system is very close to that of the layer. In this case, a more refined analysis would be warranted.

4.2.3 Foundation Shape

The foundations of nuclear power plants are not always perfectly circular slabs. In many cases, they may be polygonal or even rectangular mats. It should be noticed that even the more sophisticated analyses using the direct approach ignore this effect and that in fact the approximation provided by a circular mat is generally better than that of a strip footing (two-dimensional or pseudo-three-dimensional solution). For foundations whose dimensions are nearly equal in two orthogonal directions, using the solution for an equivalent circular mat with the same area will be quite acceptable. For rectangular foundations, it is common practice to derive equivalent radii for horizontal or vertical excitation (based on the same area) and for rotational motions (based on equating the appropriate moment of inertia). It is generally accepted that these solutions are acceptable for aspect ratios up to 4:1 (the ratio of the larger to the smaller side). This belief is confirmed

for the static case by comparing the results with the curves derived by Barkan and Gorbunov-Possadov.

Studies by Vardanega and by Dominguez seem to indicate that the approximation is also valid for the dynamic stiffnesses, that is, for the frequency variation of the stiffness coefficients, although their results need some further interpretation.

4.2.4 Embedment Effects

A relatively large number of studies have been conducted to determine the static and dynamic stiffnesses of embedded foundations, and the results are available in the literature. Using Baranov's equations, Novak obtained spring constants for the soil surrounding the foundation. In this approach, the embedment is treated as a Winkler foundation with distributed (or lumped), uncoupled horizontal and vertical springs applied to the lateral walls. The springs are complex functions of frequency, therefore including inertial and radiation effects. At a specific frequency, one can interpret these complex terms as a combination of springs and viscous dashpots. For very low frequencies, results must be extrapolated, since the solution is not valid for the static case.

While it involves the approximation inherent in a Winkler foundation, Novak's solution is particularly attractive because of its flexibility. For each individual case, the effect of embedment can be evaluated by adding to the dynamic stiffnesses of a surface foundation the contribution of the lateral springs and dashpots. It is possible in this form to consider foundations of arbitrary shape, as well as foundations which are embedded over only a fraction of their perimeter. It appears that more parametric studies should be conducted using this approach to compare results with those of other formulations and to investigate the effect of partial embedment. In addition to the expressions for the lateral springs, Novak obtained solutions for a number of cases and presented curves which have been extensively used in practice, as well as simple expressions for the effect of embedment on the static stiffnesses.

Elsabee conducted a series of parametric studies with circular foundations embedded in a homogeneous soil stratum. From the results of these studies, he suggested, as the static horizontal and rocking stiffnesses, the expressions

$$K_h = K_{ho}\left(1 + \frac{2}{3}\frac{E}{R}\right)\left(1 + \frac{5}{4}\frac{E}{H}\right)\left(1 + \frac{1}{2}\frac{R}{H}\right) \quad ,$$

$$K_r = K_{ro}\left(1 + 2\,\frac{E}{R}\right)\left(1 + 0.7\,\frac{E}{H}\right)\left(1 + \frac{1}{6}\frac{R}{H}\right) \quad ,$$

where K_{ho} and K_{ro} are the static stiffnesses for a surface foundation on a half space, as defined before; E is the embedment depth; R is the radius of the foundation; and H is the layer depth if dealing with a finite stratum. For the case of a half space, the last two terms become equal to unity. The coupling horizontal-rocking term is given by

$$K_{hr} = K_{rh} = \left(0.4\,\frac{E}{R} - 0.03\right) K_h R \quad .$$

These expressions provide a good approximation for values of $R/H \leq 0.5$ and $E/R \leq 1$. For foundations with deeper embedment, the increase in the stiffness is larger than that predicted by the formulas.

Elsabee recommended using the same frequency variation of the real stiffness coefficients as for a surface foundation, an approximation which is reasonable. It appears, however, that as the frequency increases the effect of embedment is at least partially lost. If the stiffness coefficients are normalized, this would imply that the values for an embedded foundation are smaller than those for a surface foundation in the high-frequency range.

In the case of a finite soil stratum, the imaginary stiffness coefficients representing the radiation damping are still essentially zero below the fundamental frequency of the soil layer (without embedment). Above this frequency, they are substantially larger than those of a surface foundation. Elsabee noted, however, that the values of these coefficients are affected by

the conditions of the backfill and suggested, as a conservative measure, the use of the results for a surface foundation. He did not attempt, therefore, to obtain expressions for the radiation damping as a function of the embedment ratio. Studies by Scaletti, on the other hand, indicate that when separation occurs between the sidewalls of the foundation and the backfill a frictional loss of energy takes place that more than compensates for the decrease in radiation damping. It would thus be safe and reasonable to account for the increase in the imaginary stiffness coefficient due to embedment.

Following the work of Elsabee, Kausel and Ushijima derived expressions for the vertical and torsional stiffness of embedded foundations:

$$K_v = \frac{4GR}{1-\nu}\left(1 + 1.28\,\frac{R}{H}\right)\left(1 + 0.47\,\frac{E}{R}\right)\left[1 + \left(0.85 - 0.28\,\frac{E}{R}\right)\frac{E/H}{1 - E/H}\right] ,$$

$$K_t = \frac{16GR^3}{3}\left(1 + 2.67\,\frac{E}{R}\right) .$$

For the stiffness coefficients, Kausel and Ushijima again recommended using the same frequency functions as for surface foundations, except for the imaginary terms and for frequencies smaller than the appropriate natural frequency of the layer (for the case of a finite stratum), where they suggested a transition formula from zero (static case) to the half-space value. This function is a function of the internal soil damping.

Similar studies were conducted by Jakub for strip footings and by Dominguez for rectangular foundations. Dominguez pointed out that for embedded rectangular foundations the definition of an equivalent radius, in order to use the results for circular foundations, becomes more difficult, since both the area (or moment of inertia) of the slab and its perimeter (the area of the sidewalls in contact with the backfill) are involved. He also indicated that in the case of a half space the increase in radiation damping takes place over the complete frequency range and is particularly important in the low frequencies for the rotational stiffnesses. While the imaginary term of these stiffnesses starts with a zero value for zero frequency and increases

rather slowly in the case of a surface foundation, it starts with finite values in the case of embedment. These results need further interpretation.

While additional work seems necessary to compare these different solutions and to validate their range of applicability, it appears that any of these methods provides a simple and reasonable solution.

4.3 INTERACTION ANALYSIS

Nuclear power plants are typically very stiff, massive, and complicated structures. To properly reproduce forces or motions at different levels, it is normally necessary to model various components such as the containment building, the shield, and the reactor with seve l masses. On the other hand, to estimate interaction effects, the base motions (including interaction), or levels of acceleration in the structure, relatively simple models may be sufficient. Because of its large stiffness, it is often possible, as a first approximation, to simulate the nuclear reactor building as a rigid mass. The resulting model then has a single degree of freedom for vertical excitation, and two degrees of freedom (translation and rotation) for the horizontal case. It is then extremely easy to estimate maximum response accelerations from the response spectrum or to obtain the analytical expression for the transfer functions of interest (the transfer functions of the base motions for instance). Once the base motions, including interaction effects, have been computed, the dynamic analysis of the structure can be carried out by conventional methods.

An improvement over the rigid mass model is the single-degree-of-freedom system used in Chapter 2. When this model is used to estimate total response instead of just the response of one mode, it should probably be extended by including the remaining mass of the structure and the mass of the foundation at the base, by considering mass moments of inertia, and by accounting for the thickness of the foundation. This converts it into a two-degree-of-freedom system for vertical excitation and a three or four-degree-of-freedom system for horizontal earthquakes. The rotational degrees of freedom may be ignored, however, when the rocking stiffness of the foundation is very large (case of a pile foundation). When the parameters of the single-degree-of-freedom system

are selected to reproduce the first mode (or any other mode) of the building on a rigid base, it may still be appropriate to refine the model by considering the foundation's mass and mass moment of inertia.

While these models may seem too crude, it should be noticed that they are not any worse than some of the idealizations used in more complex analyses with the direct approach (when the structure is reproduced as a shear block). They have the advantage of providing a fast and economical means of estimating global response parameters and of checking the results of computer analyses.

Once the input motions and the foundation stiffnesses have been obtained, by any of the approximate methods described above or through more rigorous analyses, the solution in the frequency domain of a more accurate model of the structure, with all its components, is straightforward and rather economical. It does, however, normally require the use of a computer. If an appropriate program for this type of analysis is not available, several alternatives are possible. Modal solutions, in terms of the modes of the structure on a rigid base or in terms of the modes of the combined system, involve some approximations, such as the estimation of the effective values of modal damping. Nonetheless, modal solutions are attractive, because most engineers engaged in dynamic analyses are familiar with the procedure. Several methods to perform a modal solution in an efficient way have been suggested in the literature.

CHAPTER 5

SENSITIVITY OF THE RESULTS

Soil-structure interaction analyses are subject to a large number of uncertainties. While there are a number of key features of the problem which are now clearly understood and which should be reproduced, at least qualitatively, by any analytical model, it must be remembered that an exact solution is never available for the purposes of comparison. In estimating the effect of various assumptions on the results, one should also keep in mind that the importance of the effects may be very different depending on the quantity of interest: strains in the soil, maximum acceleration at a given point of the structure, amplified response spectra, etc. Simplified models are particularly convenient for assessing the sensitivity of the structural response.

The main sources of uncertainty in interaction analyses are, in order of importance:

- The specification of the design motion. This involves the assessment of the seismic risk for the region; the translation of this risk into a single parameter, such as peak ground acceleration, and a family of smooth response spectra; the specification of the location or level where the motion applies; assumptions as to the types of waves involved; etc.
- The determination of the soil properties in situ and their variation with level of strain, and the selection of a mathematical model to reproduce the soil behavior.
- The analytical model used for the interaction analysis. Of particular importance in this respect are the use of a finite soil layer rather than a half space, the use and location of lateral boundaries in discrete models, and the use of two-dimensional or pseudo-three-dimensional rather than true three-dimensional models.

- The details of the analytical solution: time increment and integration scheme for solutions in the time domain, frequency range and increment for solutions in the frequency domain, evaluation of modal damping values for modal analyses.
- Features which are normally neglected, such as the presence of adjoining buildings, separation or debonding effects between the structure and the foundation, etc. Little is known about the realistic treatment of these effects.

None of the models commonly used at present can reliably reproduce soil stresses or strains in the immediate neighborhood of the foundation. An accurate nonlinear constitutive model for the soil, a three-dimensional analysis in the time domain, an idealization which includes adjacent buildings, and a precise estimate of the soil conditions in the field (including the initial state of stresses) would be needed to solve the problem. While considerable progress is being made in this field, our knowledge is still insufficient. As a result, the discussion in this chapter centers mainly on the structural response (forces and accelerations in the structure) and the amplified response spectra for design of equipment. In this context, it should be noticed that errors in the estimation of stiffnesses (the real part of the foundation stiffnesses or the stiffness of the structure) affect primarily the natural frequency of the combined soil-structure system. The result is a shift in the peaks of the amplified response spectra and a change in the values of the accelerations or forces--a change which depends mainly on the frequency range of interest and its position relative to the frequency content of the earthquake. The fundamental frequencies of most nuclear reactors are in the range of constant (maximum) response acceleration, and small shifts in frequency do not cause significant changes in the response. The shift in the peaks of the amplified response spectra, which are normally obtained for very low values of damping, may be more significant for design purposes, but the importance of this effect is diminished considerably if broadening of the spectra is used to account for uncertainties. As a result, in most cases, small errors in the estimation of the stiffness terms are not as significant as errors in the evaluation of the

effective damping (which affects the magnitude of the peaks). Since the effective damping is a combination of the structural damping (between 4% and 7% and mostly guesswork), internal soil damping (a function of the level of excitation), and radiation damping (coming from the imaginary terms of the foundation stiffnesses), the importance of the last term depends on the magnitude of the other two. For small earthquake intensities, when the material damping in the structure and the soil is small, an accurate estimation of the radiation damping becomes a key feature of the analysis. For large-intensity motions, when the internal soil damping can be substantial, this point becomes less critical.

5.1 SPECIFICATION OF THE DESIGN MOTION

The first step in any seismic analysis is the specification of the design motion. It is in this step that the largest uncertainties exist; therefore, a large degree of conservatism is normally introduced. This is particularly so in areas with low seismic activity, where a considerable amount of guessing is needed and extrapolations are rather arbitrary. The specification of the design motion involves several decisions: selection of a peak ground acceleration and a set of response spectra, specification of where the motion is to be applied, and assumptions as to the wave content of the earthquake.

5.1.1 Seismic Risk Analysis

Seismic risk analysis involves the study of the seismic history of the region and the location of all active faults. For a deterministic analysis, it is assumed that the largest earthquake within the same tectonic zone can occur at the site. For probabilistic analyses, assumptions have to be made as to the rate of occurrence of earthquakes along faults or in certain regions; the separation between main shocks, foreshocks, and aftershocks (whether they are correlated or uncorrelated must also be considered); the attenuation laws; and the relation between magnitude, Mercalli intensity, and peak ground acceleration. All these steps involve large uncertainties and the need for somewhat arbitrary extrapolations. Even so, and perhaps surprisingly, when

both deterministic and probabilistic assessments of the seismic risk are properly conducted, results for areas of high seismic activity often come within 20% or 30% of each other, which must be considered excellent agreement. The situation is much more difficult for areas with low seismicity, where a great deal of conservatism is normally introduced.

One of the main limitations of seismic risk analyses as conducted at present is the characterization of the design motion by a single parameter, typically the peak ground acceleration. Cornell has shown that if a single quantity is to be used the acceleration is the best measure when dealing with nuclear power plants. This does not detract from the fact that the use of more parameters, such as peak ground acceleration, velocity, and displacement (or at least the first two for stiff structures), would be much more satisfactory. Unfortunately, the correlations between magnitude and distance, on the one hand, and peak ground acceleration, velocity, and displacement, on the other, have tremendous coefficients of variation.

5.1.2 Characterization of the Design Motion

Once the design ground acceleration has been established, it is customary to scale a standard set of smooth, broad-band response spectra such as the NBK spectra (or preferably the Newmark-Hall spectra). This implies that all potential earthquakes, with different source mechanisms, magnitudes, and epicentral distances, would have the same frequency characteristics at the site, an assumption which is not realistic from a physical point of view and which is only justified by our lack of knowledge. Alternatively, one could say that the standard design spectra represent an envelope of spectra with different frequency characteristics, corresponding to the various potential earthquakes. It is therefore not clear what degree of conservatism (or unconservatism for certain effects) may be introduced by enveloping the effects of various earthquakes, instead of considering them separately. The use of the standard spectra may be especially questionable in the epicentral region, that is, when dealing with earthquakes which have a very large peak acceleration but a small duration. The introduction of the concept of an effective peak ground acceleration is an attempt to recognize this situation, but it is not a satisfactory solution.

The standard response spectra represent the mean plus one standard deviation of the spectra corresponding to a number of real earthquakes with different characteristics, recorded on a variety of soils. The use of these spectra implies that the results will also be the mean plus one standard deviation of those which would have been obtained from the collection of real earthquakes, scaled to the same ground acceleration, an assumption which is also questionable (particularly when dealing with nonlinear multi-degree-of freedom systems).

Once the design spectra are established, it is customary to generate artificial earthquakes whose spectra generally envelop the target spectra over the frequency range of interest and for all pertinent damping values. To generate these synthetic accelerograms, a power spectrum consistent with the design spectra is first obtained. A stationary process is then generated by superposition of sinusoidals with amplitudes corresponding to those of the power spectrum and random phases, then the motion is multiplied by an intensity function which represents the time variation of the earthquake intensity and the departure from stationarity. Finally, corrections are applied in a series of cycles to improve the match to the target spectra. The actual implications of these artificial motions, which have a larger intensity and energy than real earthquakes, have never been thoroughly investigated, and there seems to have been some reluctance to recognize their arbitrary features and implicit assumptions.

If it is desired to account better for the variability of the results due to the uncertainties in the potential earthquake, it would be more logical to work with a complete probabilistic formulation. A step in this direction was taken by Romo Organista. Several procedures have also been suggested recently for directly deriving statistics of the structural response or amplified response spectra. In all these cases, however, one starts by deriving a power spectrum compatible with the standard response spectra, using the same approach followed to generate artificial earthquakes. It would make more sense to specify the design earthquake directly, in terms of an average power spectral density, obtained in turn from the analysis of real earthquakes in much the same way as the NBK spectra were determined, and in terms of higher-order statistics (e.g., coefficients of variation as a function of

frequency). Several additional questions remain open in relation to this approach: the effect of stationarity of the random process versus either nonstationarity (due to the variation of the intensity with time alone) or complete nonstationarity (which accounts for variation of amplitude and frequency content with time); the general validity of fudge factors introduced to estimate peak response from root-mean-square values; etc.

5.1.3 Location Specified for the Motion

The two steps briefly discussed above precede the soil-structure interaction analysis. It is important to recognize, however, that the largest uncertainties and the most arbitrary decisions are associated with these preliminary steps.

A third step which affects the interaction studies more directly is the specification of the location where the design motion applies. It is common practice now to assume that the motion is the one that would occur at the free surface of an average firm ground (a soil deposit with a shear-wave velocity for low levels of strain of about 600 to 1500 ft/s). This is quite reasonable if one takes into account that the earthquakes used to derive the standard response spectra were recorded on a variety of average type soils. It would be convenient to have, in addition, design spectra applicable to the surface of very soft soil profiles (with shear-wave velocities for low levels of strain smaller than 600 ft/s) or very hard, rocklike materials (with shear-wave velocities larger than 1500 or 2000 ft/s).

Problems arise in practice when dealing with an unusual soil profile which has one or more layers of very soft soil. It is then often necessary to derive site-specific spectra. The most sensible alternative is to start from motions specified at a hypothetical outcropping of rock or rocklike material (if there is such a material underlying the soil) and to determine compatible motions in the soil layer by assuming a specific train of seismic waves (typically, vertically polarized shear waves). In some cases, however, it is assumed that the design motion (with a standard spectrum corresponding to

average firm ground) applies at bedrock, that is, the interface between the soil and the underlying rock. Depending on the characteristics of the particular soil profile, the results obtained from these two approaches may differ by 20% to 100%, the latter being much more conservative. Differences could be even larger in some cases (two- or threefold differences in the in-structure response spectra have been reported).

Similar variations in results occur when dealing with embedded foundations (the most common case in practice). The most sensible alternative would again be to start from the design motion specified at the free surface, or a rock-compatible motion at a hypothetical outcropping of rock, and to compute, by assuming a specific train of waves, compatible motions at the base of the soil profile and at the foundation level, accounting for the geometry of the excavation. This produces both translational and rotational components of motion, the first showing a decrease in amplitude (particularly above the fundamental frequency of the embedment layer) with respect to the surface motion. In some instances, however, it is assumed that the design motion takes place at the foundation level in the free field (without any excavation). This makes little sense, since the motion at this level is strongly affected by the characteristics of the soil above. The resulting motion of the foundation (without structure) then exhibits unnatural peaks at the resonant frequencies of the embedment layer. A variation on this approach is to specify the motion at the free surface, but to impose limitations on the resulting motion at the foundation level in the free field. While some limitations on the deamplification of the translational motion with depth are appropriate to account for the effects of different types of waves, imposing them in the free field again produces unreasonable peaks in the foundation motion. A last alternative, sometimes proposed, is to assume that the design motion occurs directly at the foundation level, accounting for the excavation. No rotational components of motion are then considered. This approach has the practical advantage of reducing the computational effort by making unnecessary the determination of compatible motions. On the other hand, it ignores existing knowledge on the physical behavior of the solution, and it may lead to excessive conservatism for deeply embedded foundations. Variations in the results obtained from these approaches may again range from 20% to 100%, or even larger in some cases.

5.1.4 Wave Content of the Design Motion

It is not common practice at present to specify the wave content of the design motion, mainly because little is known about the subject. In many cases, however, it is desired or required to study the possible effect of various types of waves, in contrast to a single train of vertically polarized waves. Love waves or horizontally propagating shear waves give rise to torsional excitation of the foundation, accompanied by a filtering of the translational motion. Rayleigh waves produce similar effects, inducing a rotational motion around a horizontal axis instead of torsion. In order to account for these additional effects, it is sometimes assumed that the seismic motion is composed entirely of one of these wave trains, traveling horizontally at a specified apparent velocity. This assumption does not seem realistic.

When considering the translational motion of the foundation, the normal assumption of vertically propagating waves generally produces conservative results. On the other hand, this assumption may exaggerate the nonlinear effects in the soil (reducing the shear modulus and increasing the material damping).

If other trains of waves are to be considered, the most important point is to maintain consistency in the analysis: one should not account for the reduction of the translational motions while neglecting the rotational components, or vice versa. If consistency is maintained, results are not likely to change by more than 20% in most cases. On the other hand, for a particular soil profile, it is always possible to find a specific train of waves that will produce very large peaks at some frequencies. Assuming that the whole earthquake consists of these waves (with a particular ratio of amplitudes) would be unreasonable.

It appears that in many cases the rotational effects caused by traveling waves can be accounted for realistically by relatively simple considerations: torsional excitations are induced by assuming an accidental eccentricity if none exists in the structure, and rotations around a horizontal axis are induced in embedded foundations, even by vertically polarized waves. Currently this type of approach may be advisable.

To improve on this situation, reproducing better the true physical behavior, more knowledge is needed on the wave content of real earthquakes. The best way to obtain this knowledge is through arrays of instruments displayed not only at various depths, but more importantly in two orthogonal horizontal directions (so as to measure the apparent horizontal velocity of propagation). Such arrays are now being installed. A second approach is to try to simulate earthquake motions analytically from a physical basis (in contrast to the artificial earthquakes derived only on the basis of mathematical, probabilistic models without a physical justification). A considerable amount of research is being done in this area, and several models are now available. These models seem able to reproduce seismic displacements reasonably well at some distance from the epicentral region. Reproducing accelerations is, however, much more difficult, since some important and arbitrary assumptions have to be made about the source mechanism (e.g., the velocity of propagation of the fracture). Thus, while this research is extremely promising and valuable, some care must be exercised to avoid the temptation of applying it in practice before it has been thoroughly validated (as happened, for example, with the artificial earthquakes).

5.2 SOIL PROPERTIES

A second large source of uncertainties is related to the determination of the soil properties to be used in the analysis. This involves measuring soil properties in the laboratory and relating them to the properties in situ, determining the variation of these properties with level of strain, and choosing the mathematical model to reproduce the nonlinear soil behavior.

5.2.1 Determination of Soil Properties

Soil properties are typically determined in the laboratory by triaxial, resonant column, or cyclic shear tests. Each one of these procedures yields somewhat different results and requires a different interpretation. More importantly, the properties determined in the laboratory do not coincide with those in situ, and some extrapolation is needed to predict the soil characteristics to be used in the analysis.

Shear moduli for low levels of strain, as measured in the laboratory, are typically smaller than those obtained in the field, often by two- or threefold. Several reasons are offered for this discrepancy; for instance, the unavoidable disturbance of the soil samples. For clays it appears that the main cause for the differences is the increase of the shear modulus with time during secondary consolidation. Several procedures are available to extrapolate laboratory values of shear modulus for low levels of strain to the field, and they seem to provide reasonable estimates. However, there will always be some measure of uncertainty and variability (soil layers will not be as uniform, homogeneous, and isotropic as those considered in the analysis).

5.2.2 Variation with Level of Strain

In triaxial or cyclic shear tests, the variation of the soil properties (shear modulus and damping) with level of shear strain is determined from hysteresis loops. For each amplitude of shear strain, the shear modulus is determined as the secant modulus; the value of damping is obtained from the area of the hysteresis loop. In resonant column tests, on the other hand, the shear modulus is backfigured from the resonant frequency, and the damping is computed from the exponential decay (logarithmic decrement). This implies an assumption of viscous damping. Each test requires its own careful interpretation, since the states of stress and strain are different. Other factors whose effects are not completely known, such as strain rate (apparently not important for sands, but significant for clays) sometimes come into the picture.

A more important question is the extrapolation to the field of the experimental curves showing variation of shear modulus and damping with shear strain. Three procedures which yield very different results have been suggested: a proportional or percentage method in which the laboratory curves for shear modulus are scaled at all strain levels by the ratio of the G_{max} in the field to the G_{max} in the laboratory; an arithmetic method in which the difference between these two values of the low-strain shear modulus is added to the experimental curves at all values of the shear strain; and an intermediate solution. Recent tests seem to indicate that for shear strains

smaller than 10^{-3} to 10^{-2} the arithmetic increment is more correct. For larger values of strain, however, it does not seem physically logical. When dealing with moderate to large levels of excitation, the differences in the predicted shear modulus (at strain levels of 10^{-2}) may be about an order of magnitude. If, for instance, G_{max} in the field is twice G_{max} in the laboratory and G/G_{max} at the desired level of strain is 0.1, then the percentage method predicts the same ratio in the field, but the arithmetic increment estimates $G = 0.55G_{max}$. As the ratio of the G_{max} in the field to the G_{max} in the laboratory increases and as larger levels of strain are considered, the discrepancy increases. These differences in the predicted value of the shear modulus affect equally the estimation of the internal soil damping if one considers

$$D = D_{max}\left(1 - \frac{G}{G_{max}}\right) \quad .$$

Twofold difference or more are possible between the damping values predicted by these procedures.

It appears that the proportional method is the one most commonly used in current practice. From the above discussion, and from the fact that vertically propagating shear waves are normally assumed to estimate the levels of strain in the soil deposit, one would conclude that the reduction in shear modulus and the values of internal soil damping are likely to be overestimated. Some limit on the maximum reduction and damping would be in order.

5.2.3 Mathematical Model

Three alternatives are currently available to account for nonlinear soil behavior:

- To perform a single linear analysis, with soil properties (shear modulus and damping) obtained from appropriate curves on the basis of estimated levels of shear strain. These estimations can be obtained by iteratively solving the one-dimensional soil amplification problem (equivalent linearization) or from simplified procedures.

- o To perform a series of linear analyses on the complete soil-structure model, obtaining from each one measures of characteristic strains and adjusting correspondingly the modulus and damping for the new analysis. This is an application of the equivalent linearization technique to the two- or three-dimensional problem.
- o To perform nonlinear analyses in the time domain for the complete soil-structure model, using an appropriate set of nonlinear constitutive equations for the soil. From a mathematical point of view, this procedure is clearly superior to the other two, but from a practical point of view, the main question is the availability of a model which will accurately reproduce the soil behavior in the field and which depends on a number of parameters easy to measure.

It is sometimes said that the first alternative, used with the substructure approach, accounts only for primary nonlinearities (those caused by the seismic waves alone), neglecting the additional nonlinear effects (secondary nonlinearities) caused by the structural vibrations. The terms primary and secondary nonlinearities are used to indicate that the earthquake excitation constitutes the main or primary source of nonlinear effects, while the structural vibrations cause only secondary changes. This is normally true when dealing with moderate- to large-intensity earthquakes and when considering only the structural response. As for the distribution of stresses and strains in the soil in the immediate neighborhood of the foundation, both effects are equally important.

The first alternative has the advantage of simplicity and economy. The second would appear to be more accurate, but one must keep in mind the uncertainties involved in applying to a 2D or 3D state of stresses experimental curves intended for one-dimensional conditions. One of the clear problems is the definition of a second elastic constant (Young's modulus, the bulk modulus, the constrained modulus, or Poisson's ratio) and its variation with shear strain, as well as the effect of volumetric strains.

In most cases, results obtained with either of the first two procedures are within 20% of each other, the results from the first generally being on the conservative side. When the differences in the results are large, one

must question the validity of either approach, since such discrepancies imply that two- or three-dimensional effects become important. Neither of these two alternatives reliably estimates stresses or strains in the soil.

A considerable amount of research is being conducted at present on the third alternative. Three soil models are of particular significance:

- The cap model of DiMaggio and Sandler. This model, in its original form, had serious problems in reproducing cyclic behavior with large reversals of strain, as occurs during seismic excitation. More recently it has been adapted to include both isotropic and kinematic hardening, and in this form it is much more promising.
- The multiple yield surfaces model of Iwan and Mroz, as applied to soils and generalized by Prevost. This model is particularly attractive because of its physical and mathematical consistency, its generality, and the possibility of obtaining the appropriate parameters from simple laboratory tests.
- The endochronic model. While this model has a strong theoretical basis and is being applied to reproduce the behavior of other materials, such as concrete, the applications to soils have been limited. The number of parameters needed to define the model, and the laboratory tests to be used, are not clear beyond the simplest one-dimensional cases. The model may have promise, but its application would seem to be at a much earlier stage of development than the other two.

Continued research with these models, in particular the modified cap model and Prevost's model, will provide invaluable insight into the effects of nonlinear soil behavior on soil-structure interaction and into the validity of the simplified approaches used at present (the first two alternatives above). More work and expertise with these models is needed, however, before a procedure of this kind is justified. It must be remembered that uncertainties in the soil parameters and in the extrapolation of laboratory results always exist and that these uncertainties may offset any increase in accuracy in the mathematical model.

5.3 MODELING ISSUES

There are several modeling issues that can have significant effects on the results of soil-structure interaction analyses. Among these, the most important are the location of the bottom boundary defining a finite soil deposit, in contrast to a half space; the location and type of lateral boundaries; and the use of two-dimensional versus three-dimensional models.

5.3.1 Finite Soil Layer versus Half Space

Discrete models, as used in the direct approach, must always operate with a finite soil layer. In the substructure approach, on the other hand, it is possible to use either a finite layer or a layered half space. Clearly, the first point that must be considered is the true physical situation; when there is a clear layer (or layers) of soft soil, underlain by much stiffer rocklike material, establishing a bottom boundary at the interface is realistic and more reasonable than assuming a homogeneous half space (a layered half space would be the most accurate model). In fact, if there is a layer of soft soil over a second layer of much stiffer soil and finally rock, with the relative change in properties being larger between the first two layers than between the second and the rock, it is preferable to establish the bottom at the first interface. On the other hand, when dealing with a deep soil deposit whose properties are uniform or increase very slowly with depth, some thinking must be done to decide the position of the bottom boundary or the transition to a half space.

Considering a finite soil layer when it does not physically exist affects both the real and the imaginary parts of the foundation stiffnesses. The real part, the equivalent of a spring, increases slightly, the change being more pronounced in swaying (horizontal motion) than in rocking. For a value of H (depth of soil deposit) larger than four foundation radii (R), the change with respect to a half space is less than 10%, which produces less than a 5% change in the fundamental frequency of the system. Considering the uncertainties in the soil modulus, it would appear that this is not a significant effect and that from the point of view of the real stiffnesses results will be adequate as long as H is larger than 2R to 3R.

A more important consideration relates to the imaginary part of the stiffnesses (representing the radiation damping). When a finite stratum is considered, either because this is the physical situation or because an arbitrary bottom boundary is introduced, there is no radiation damping below the fundamental frequency of the layer. For a half space, on the other hand, there is radiation damping at all frequencies. Above the fundamental frequency of the layer, the estimation of the radiation damping is quite appropriate for H larger than about 2R or 3R. It is thus important, when establishing the bottom boundary or deciding on the use of formulas for a homogeneous half space, to estimate the natural frequency of the soil-structure system (using a simple approximate method) and the fundamental frequency of the layer. If the first frequency is clearly above the second, this modeling issue is not very significant. On the other hand, when the first is below the second, large differences in results may occur. Using the finite layer for a deep soil deposit or a uniform half space solution for a stratum of finite depth would be incorrect, the former providing a conservative solution, the latter being on the unconservative side. In the extreme case, when the earthquake excitation is very light and, correspondingly, the internal soil damping is small, improper placement of the bottom boundary, negating radiation damping when it exists or accounting fully for it when it is not there, can produce variations in the results of 50% to 100%. This type of variation can be reduced considerably with proper planning of the model (unlike some of the other uncertainties discussed previously).

A final effect of using a finite soil layer is that the variation of the foundation stiffnesses with frequency exhibits some marked oscillations, corresponding to the natural frequencies of the layer, which do not appear for a half space. These oscillations are very large and a source of concern for an elastic medium. When there is some internal soil damping (about 5% to 10%), as is normally the case for seismic analyses, these oscillations decrease considerably in amplitude. More important in this case are oscillations in the high-frequency range caused by the mesh size of the discrete model.

5.3.2 Lateral Boundaries

The use of lateral boundaries, in order to create a finite domain, is associated with discrete models, when there is also a bottom boundary. To some extent, the lateral boundaries affect the real stiffness (although in general to a much smaller extent than the bottom boundary), and more importantly, the radiation damping above the fundamental frequency of the layer and the variation of the stiffness functions with frequency. When elementary or viscous boundaries are used and the fundamental frequency of the soil-structure system is above that of the layer, these boundaries should be placed at a distance of five to ten foundation radii from the edge of the foundation, the actual distance depending on the amount of internal soil damping (with less distance needed as the damping increases). Furthermore, the mesh size of the discrete model must be kept small; if it is increased in a horizontal direction, this must be done gradually from the foundation outward.

For solutions in the frequency domain, the use of consistent boundaries eliminates this problem, since the boundaries can be placed directly at the edge of the foundation with excellent results. For solutions in the time domain, more research work is needed to obtain reliable lateral boundaries (the Smith boundary as implemented by Dames and Moore seems promising but needs more testing). In any case, the question of lateral boundaries is at worst one of cost; by placing any boundary at a sufficient distance (with the corresponding increase in the number of degrees of freedom of the problem), the error in the solution can be controlled and kept within a desired tolerance.

5.3.3 Two-Dimensional versus Three-Dimensional Solutions

Much has been written about the validity of using two-dimensional or pseudo-three-dimensional models to reproduce a truly three-dimensional situation. Two parameters (width of footing and thickness of soil slice) can be chosen to define the two-dimensional model. The degree of approximation depends, therefore, on whether these parameters are chosen so as to match

(over a certain range of frequencies) the real stiffnesses, the radiation damping in rocking and swaying, or any other combination of variables.

For the extreme case of an elastic half space, if the fundamental frequency of the soil-structure system with the two-dimensional model is small, the error can be large. Selecting the width and thickness so as to match the real stiffnesses for dimensionless frequencies of 0.3 or larger, Luco and Hadjian found differences of 50% between the 2D model and an exact 3D solution, the former being unconservative. In practical cases, when there is physically a finite soil layer or when soil properties increase with depth, if the parameters are chosen appropriately (to match the radiation damping in swaying and rocking, or to match the real and imaginary parts of the horizontal stiffness over the frequency range of interest for deeply embedded foundations), discrepancies are likely to be much smaller, about 10% to 20%.

Some preliminary computations using approximate formulas seem advisable to justify a two-dimensional model. However, with some precautions to guarantee the adequacy of the model, it would seem that in many, if not the majority of the cases, a 2D model can provide an adequate solution, the variations in the results being smaller than those caused by other uncertainties.

5.3.4 Other Modeling Issues

In the preceding discussion, it has been assumed that the model used for the analysis accounted properly for layering (variation of soil properties with depth to the bottom boundary) and for embedment. If more simplified procedures are used, further variations in results may be expected.

Using available solutions for the foundation stiffnesses corresponding to a uniform elastic layer or half space, and adopting the soil properties at a specific depth (typically between 0.5 and 0.75 radii) provide a reasonable solution, within 10% or at most 20% of that given by a more accurate model, if the variation of soil properties is smooth. It may not be appropriate when there are relatively thin layers of soil with very different characteristics.

The use of approximate expressions to account for embedment (such as Elsabee's or Novak's) or simple models with springs which are frequency

dependent (as suggested by Novak), in contrast to a more accurate analysis, may again produce variations of about 20% in the values of the stiffnesses, but much smaller variations in the structural response. It must be remembered that as embedment increases, and with it the stiffnesses, the interaction effects decrease.

Similar considerations apply with respect to the modeling of a rectangular foundation or a foundation of arbitrary shape (as a circular slab or a strip footing). It is commmonly accepted that formulas for a circular foundation can be extended to rectangular foundations (defining an appropriate equivalent radius for each case), as long as the aspect ratio is less than four. Comparisons performed by Vardanega seem to confirm this belief within 10% or 20%. Again, this type of error in the stiffnesses produces variations in the structural response which are at least twofold smaller.

While additional work is needed to compare and evaluate simplified models in order to define better their range of applicability, results provided by these approximate procedures, when properly implemented, usually have smaller errors than those caused by other simplifying assumptions.

Some consideration must be given finally to the structural model. Simple one-degree-of-freedom systems or shear blocks (as sometimes used with the direct approach) may be sufficient to obtain the base motion, including interaction effects, but are not appropriate to estimate either forces or, in some cases, floor motions and in-structure amplified response spectra. In these cases, a more detailed and traditional dynamic analysis of the structure is normally performed later, with the base motion as input. It is important in this case not to neglect the rotational component of the base motion, unless it is shown to be negligible. Some of the computer programs for dynamic analyses of structures do not allow for rotational excitations. The steps necessary to construct an appropriate structural model for dynamic analysis are well known, and the accuracy is easy to control.

5.4 SOLUTION DETAILS

An aspect of the analysis which is rarely mentioned in regulations relates to details such as the time step of integration for solutions in the

time domain, and the frequency range and increment when working with Fourier transforms. Another source of possible errors is the evaluation of appropriate values of modal damping for modal analyses.

5.4.1 Time Step of Integration

When the solution of the soil-structure system is performed in the time domain (the approach that would be followed to account in a more accurate way for nonlinear soil behavior), some consideration must be given to the integration scheme. Implicit methods which are unconditionally stable have the advantage, from an economic point of view, that a time step which is not extremely small can be used without impairing the stability of the solution. On the other hand, it is sometimes hard to assess the accuracy of the results.

Explicit methods, such as the central difference formula, are particularly convenient for this type of problems, especially if lumped masses are used with the discrete model, because the solution can be carried out step by step without ever assembling or inverting large matrices (or solving large systems of equations). On the other hand, explicit methods normally require a very small time step to guarantee stability, the step decreasing with decreasing mesh size. When dealing with nonlinear systems and an incremental, step-by-step solution, errors are committed by overshooting the time at which a transition in properties occurs. Equilibrium corrections are sometimes applied when the system goes from linear-elastic to plastic, but it must be remembered that the correction is not exact. When a reversal occurs and the system which was in a plastic state again becomes elastic, no correction is normally applied. These sudden changes in properties can introduce fictitious high-frequency components into the system. Some authors have recommended the addition of some viscous damping to filter out these high frequencies, considered as noise, but this solution is not satisfactory, since it may produce excessive filtering and thus destroy true components of the motion. Some further investigation of these questions seems necessary. While the main features of the solution (maximum accelerations and peaks of the response spectra) are not largely affected by these errors, the in-structure spectra for the design of equipment may have inaccuracies in the high-frequency range.

5.4.2 <u>Frequency Range and Increment</u>

For solutions in the frequency domain, similar considerations must be made regarding the frequency increment. A minimum of 2048 points, and preferably 4096, should be taken to obtain the Fourier transform of the earthquake. When computing the transfer functions, however, it is not usual to take as many points. In some cases, as few as 50 values are obtained, and intermediate points are determined by interpolation. While several interpolation procedures have been devised to improve the accuracy of the solution, the size of the frequency increment at which the transfer functions are computed should be checked in relation to the fundamental frequency of the system. Under some circumstances, the procedures now used in practice may yield important errors in the estimation of the peak of the transfer functions. A minimum of 150 points would normally seem advisable. An improvement can also be obtained by changing the frequency increment for different frequency ranges or by automatically computing intermediate values when peaks are detected.

The total frequency range considered also affects the results, particularly for high frequencies. This is an important consideration in the preliminary phases if the deconvolution process is used to obtain compatible motions at depth. In some cases, frequencies above 10 to 12 Hz are filtered out in order to maintain stability, without adequately checking the implications.

5.4.3 <u>Mesh Size</u>

Closely related to the time step and the frequency increment is the selection of an appropriate mesh size for discrete models. As pointed out earlier, the mesh size must be sufficiently small in the immediate neighborhood of the foundation to reproduce accurately the static effects, and over the complete domain to transmit the frequencies of interest. A mesh size which is too large will create large oscillations in the stiffnesses for high frequencies.

5.4.4 Modal Damping Values

When the last step of the substructure approach is to be performed using a modal analysis, it is necessary to select appropriate values of the foundation stiffnesses and of the effective damping in each mode. The effect of introducing errors in the values of the stiffnesses depends on the frequency of the soil-structure system with respect to the response spectrum. Using constant stiffnesses can introduce some discrepancies, but these will be small if some iterations are performed to select the values at the frequency of interest.

When the effective modal dampings are computed in a reasonable way, taking into account the different natures of the various damping terms (hysteretic or viscous, i.e., frequency dependent or independent), the errors introduced should be about 10% or less. This is true whether the damping values are computed by matching the peak of the transfer function at a specific point or from energy considerations (weighted modal damping).

On the other hand, large errors can be introduced when the values of modal damping are arbitrarily set or limited by regulations, and interaction effects are important (with substantial radiation). For instance, if modal damping is limited to 10%, errors of up to 50% can occur in some cases.

5.5 ADDITIONAL EFFECTS

There are a number of effects which are typically ignored or only crudely modeled in soil-structure interaction analyses. Examples include the presence of adjoining buildings with different positions in space, the fact that the building may only be embedded over a certain fraction of its perimeter, the conditions of the backfill, and debonding or separation effects between the foundation and the soil.

Discussion of these effects was left to the end, because little is known of their importance. It appears that the presence of other structures primarily changes the natural frequencies of the combined soil-structure system, introducing additional peaks in the transfer functions. The response amplitudes may not change significantly (10% to 20%), unless the adjoining

structures are much heavier or are very deeply embedded, thus forming a resonant box around the building of interest. When the structures are extremely close to each other, as is often the case in nuclear power plants, structure-soil-structure interaction effects depend strongly on the nonlinear characteristics of the soil between the buildings. Without a reliable soil model, estimation of these effects is difficult, and results of linear elastic analyses should be carefully interpreted.

Debonding and separation effects also have been shown to depend strongly on the nonlinear soil behavior. When the soil is assumed to be linearly elastic, the uplifting of a surface foundation tends to increase the response in the high-frequency range. These effects disappear, however, or are at least minimized, when accounting for soil nonlinearity. For the case of embedded foundations, separation along the lateral walls also produces changes in the response. These changes are generally on the favorable side, but the results depend strongly on the initial state of stresses in the backfill.

All these features, which are normally ignored, could produce variations in the results of 20% to 50% over certain frequency ranges. In general, one would expect them to be nearer 20% and not cumulative. As more research is carried out, with more realistic three-dimensional models and nonlinear constitutive equations for the soil, a better understanding of their importance will be achieved.

5.6 SUMMARY

A large number of uncertainties are present in soil-structure interaction analyses. The definition of the design earthquake (including its frequency characteristics, types of waves, and the location where motion is specified) is without doubt the main source of variations. Almost as important are the uncertainties involved in the estimation of soil properties in situ and their dependence on the state and levels of strain.

Variations in the model used for the analysis of the soil-structure system can clearly produce important differences in the results if inconsistent assumptions are made in some steps, or if serious mistakes are committed. It is believed, however, that most of these errors can be

controlled and that the variations which can be expected between the results of different, but properly implemented, models (even introducing engineering simplifications) should not be larger than 20% or 30%.

CHAPTER 6

RESEARCH NEEDS

As pointed out in the previous chapter, there are a large number of uncertainties associated with soil-structure interaction analyses. A list of needed research topics, with corresponding priorities, was prepared at a workshop on Research Needs and Priorities for Geotechnical Earthquake Engineering Applications, sponsored by the National Science Foundation and held at the University of Texas, Austin, in 1977. Work has been done since then on several of the topics mentioned, but the suggested list remains basically valid today. For the purposes of the present discussion it is convenient to distinguish among the topics as follows:

- Basic research to improve the definition of the physical parameters of the problem, such as the seismic environment or the nonlinear dynamic soil properties. While these topics extend beyond the range of soil-structure interaction, addressing the much more general areas of earthquake engineering and soil dynamics, their knowledge is key to an accurate determination or prediction of interaction effects.
- Analytical research to develop new formulations incorporating the improved knowledge on the physical problem.
- Development of more efficient computational techniques for the solution of the three-dimensional problem with nonlinear soil behavior, adjoining structures, partial embedment, separation effects, and an arbitrary seismic environment.
- Engineering research designed to assess through parametric studies the sensitivity of the results to various simplifying assumptions, the relative importance of different effects, and the validity of various formulations.
- Development of simplified, engineering-type procedures, which can be used for preliminary analyses, to estimate the key features of the solution and to check the results of more complex analyses without the

need to repeat them with an alternate, but similarly complex, formulation.

- o Development of a series of test cases which can be used to validate proposed analysis procedures or to define better their range of applicability.

It is clear that each individual researcher or practicing engineer will find some of these areas more attractive and meritorious. All of them, however, must be considered equally important in order to gain more confidence in interaction analyses and to improve the present state of the art. It appears, in particular, that in recent years a substantial effort has been devoted to the first three needs (especially the second and third), to the detriment of the last three. A more balanced and broader outlook on the problem seems necessary.

6.1 BASIC RESEARCH

6.1.1 Definition of the Seismic Environment

Before the interaction analyses can begin, it is necessary to define the motions that would occur at any point within the soil mass before the structure is built. At present the seismic input is defined only in terms of a maximum ground acceleration and a set of smoothed, broad-band response spectra. This definition is clearly insufficient and one of the main sources of uncertainties. For a more complete and realistic analysis, it would be necessary to know the wave content of the design earthquake as a function of its magnitude, source mechanism, distance, and local geology.

It is important to realize that, while most interaction analyses conducted at present assume that the earthquake consists of vertically propagating shear or dilatational waves, the formulation for any arbitrary train of waves or for any combination of plane wave fronts has been known for several years. The fact that other waves are rarely considered and may not be implemented in the most commonly used computer programs is based on the lack of knowledge about the physical problem.

To improve the present state of the art, field observations are needed on the spatial distribution of seismic motions, considering not only their variation with depth (which is rather insensitive to the types of waves), but also their variation along the surface, which would allow one to determine the apparent wave velocity. Arrays of instruments are now being deployed at selected locations, but information will not be available until important earthquakes are felt at those sites.

Seismologists are also conducting analytical wave propagation studies to simulate earthquakes starting from an assumed fracture. Much can be learned from these models, but one cannot logically expect a complete solution for some time. Prediction of the acceleration records in the epicentral region for large earthquakes is difficult because of the sensitivity of the results to the assumptions for the fault breakage.

6.1.2 Nonlinear Soil Behavior

The second key factor controlling the accuracy of seismic soil-structure interaction analyses is the nonlinear behavior of the soil. Present procedures rely heavily on the assumption that the behavior is basically one-dimensional. For this situation, linear analyses using equivalent, estimated soil properties or iterating on the properties on the basis of the computed strains are believed to provide answers which are qualitatively reasonable, although there seem to be unresolved questions as to their accuracy. For more complex states of stresses, present procedures are far from satisfactory.

A number of nonlinear constitutive equations for soils have been proposed in the last years; for instance, the endochronic model, the modified cap model of Weidlinger Associates, and the multiple yield surfaces model of Prevost. The first is interesting from a theoretical standpoint, but it seems far from being applicable to real problems. The last two are at a more advanced stage of development, and they seem very promising. They are still being refined, however, and they need more extensive validation.

To improve present conditions, more testing of soils under different states of stress and, in particular, for stress paths similar to those which

may be encountered in the interaction problem (when dealing with embedded foundations, neighboring building, etc.) is needed. It is important, in particular, to fit the parameters of the analytical models from a series of tests, then evaluate their predictions for different loading patterns and stress paths.

Equally important is establishing more reliable procedures to extrapolate soil properties obtained from laboratory tests to actual field conditions. This includes not only the properties at low levels of strain, but also the complete curves giving the variation of elastic moduli with strain level (including coupling between shear and volumetric strains). Various procedures accepted today in practice can give vastly different results.

6.2 ANALYTIC RESEARCH

It is probably in the area of analytic research that most work has been done in recent years, to the point that our present capabilities far exceed our knowledge of the physical parameters. Dissemination of some of the research results may, however, be necessary, since it appears that some of the new techniques are not well known in practice.

Analytical formulations are now available to solve the general three-dimensional wave-propagation problem with one or more foundations of arbitrary shapes, with various degrees of embedment, and for horizontally stratified soil deposits. Extension to more general soil profiles is not difficult, the main obstacle being the cost of computation. The main limitation of these solutions is the need to consider a linear-elastic (or viscoelastic) material and to assume perfect bonding between the foundations and the surrounding medium.

Nonlinear solutions using discrete models (finite elements or finite differences) and integrating the equations of motion step by step in the time domain are also beginning to appear. Cost is again a main drawback, and some additional work is needed to investigate boundary conditions (absorbing boundaries) in the time domain and the mesh sizes (over the complete range) needed to obtain a given accuracy. Our ability to reproduce accurately the response in the high-frequency range, in light of overshooting and

backtracking effects (while following a nonlinear force-deformation relationship), must also be investigated further.

Thus, while more work is needed on the analytic and numerical formulations, it is probably in this area that the state of the art is most advanced and that better control can be exercised on the accuracy of the solution.

If the accumulation of strain energy along a fault could be constantly monitored and if the physical process giving rise to an earthquake was known, as well as the exact nature of waves generated and their propagation mechanism, it would be possible to predict deterministically the occurrence of an earthquake and its expected characteristics. Strong-motion shaking does not occur accidentally or at random. However, our knowledge of the physics of the problem makes this type of prediction a long-range objective. In the meantime, valuable information can be gained, and has been gained for years, from statistical analyses of recorded earthquakes, from observation of damage to buildings and other structures, and from measurements of response. The theories of stochastic processes and random vibrations can be used to advantage to predict the seismic response of structures, including interaction effects, accounting in a rational way for uncertainties. It must be noticed, on the other hand, that these theories are just additional tools, like computers or finite element methods, which can be of great value in achieving good engineering solutions, but whose results cannot be better than the original data or the assumptions made.

Probabilistic formulations are beginning to find, and rightly so, a place in seismic analysis and design. However, much work remains to be done in this field. Present procedures are often oversimplified, based on dubious assumptions (which tend to be ignored when assessing the results) and relying more on mathematical manipulations than on the physical characteristics of the problem. Three examples might be cited: the implications of the artificial earthquakes generated to match smoothed response spectra are improperly known, particularly when dealing with nonlinear systems; the assumptions of stationarity or independence of the frequency content with time have not been fully explored; and the consideration of a maximum value as the root-mean-square multiplied by a fudge factor, independent of frequency, is also an

approximation in need of further study. It is probably in this area that analytical research, coupled with a better understanding of the physical process, would be more valuable.

6.3 IMPLEMENTATION OF MORE EFFICIENT COMPUTATIONAL TECHNIQUES

Present practice in soil-structure interaction analysis is often dictated by the availability of specific computer programs. While more refined formulations than those commonly used may have been developed, their existence is sometimes not well known, or more importantly, computer codes based on these formulations are not available. Furthermore, the use of complete three-dimensional formulations, including nonlinear effects, is mostly limited by its cost.

Implementation of well-tested and validated techniques into efficient computer programs which can be used in practice is a must. On the other hand, care must be exercised not to impose any one program or formulation as the only acceptable one and not to rush into practice procedures which are still in the research stages. In addition to the computer implementation, research work has been done to develop more efficient and more reliable computational procedures that would decrease the cost of fully three-dimensional analyses without impairing their accuracy.

6.4 SENSITIVITY STUDIES

The solution of practical civil engineering problems always requires constructing an approximate mathematical model to reproduce the physical reality. While computers now allow the construction of models that are far more complex and that account for more factors than in the past, the role of an engineer is to select the simplest and most economical idealization which realistically accounts for all important effects. The engineering judgment needed to decide which significant factors must be accurately modeled is gained through experience with the solution of similar problems and a sound understanding of the behavior. No computer program, no matter how

sophisticated, should attempt to do away with engineering judgment or to replace it. This is particularly important when dealing with the type of problems considered here, where accurate modeling of all important factors is not likely to be possible for many years.

In the previous chapter, an attempt was made to quantify the kind of variations in results which can be expected from various simplifying assumptions. The figures quoted were based on a very limited number of studies carried out to date. To get a better feeling for the relative importance of different factors and the range of applicability of various approximations, many more sensitivity studies should be conducted.

Some of the points which should be further evaluated through parametric studies are discussed below.

6.4.1 Other Wave Types

Types of waves other than vertically propagating shear or dilatational waves can affect the response. It is not sufficient, however to consider just a specific train of waves and to compare one specific effect. To understand better the possible effect of more realistic wave patterns, one should consider the total problem, namely, any combination of wave trains which may occur in a real earthquake. Effects of embedment should be taken into account, as well as nonlinear soil behavior. The flexibility of the mat should also be considered, particularly when dealing with very large slabs supporting several buildings. Finally, the effect of more complicated geometries (rather than a horizontally stratified soil deposit) should be investigated.

All these effects will have to be considered in a certain sequence. It might be convenient to assume first an elastic, horizontally layered stratum and a surface foundation (some results have already been obtained by Luco for this case). Next, one could proceed to the study of embedded foundations for the same general situation. The flexibility of the base slab and the side walls could be incorporated in the third step, followed finally by the treatment of nonlinear soil behavior. The advantage of this approach is that each problem is well understood before proceeding to the next and more complex

one. The disadvantages are that it takes longer to obtain solutions of real practical applicability, and that there is a danger of reaching conclusions which will be invalidated after including further effects.

6.4.2 Equivalent Linearization

The validity of the equivalent linearization to simulate nonlinear soil behavior needs further investigation, both for one-dimensional and more complex states of stresses. Several sensitivity studies have already been conducted for one-dimensional cases. However, the results seem to be contradictory and need further scrutiny (looking in detail at the nonlinear models used, the integration schemes, etc.). Even less information is available on the applicability of the linearization to two- or three-dimensional states of stress. These studies should be conducted using a realistic set of nonlinear constitutive equations for the soil (i.e., some of the models discussed in Sec. 6.2).

Interaction analysis results are also sensitive to the way the modulus- and damping-variation curves are extrapolated from the laboratory to the field. This sensitivity needs further consideration. The limited number of studies conducted on this subject indicate that significant differences can be expected between the various rules and that this factor may be more important than several others which have received considerable more attention.

6.4.3 Other Nonlinear Effects

Other nonlinear effects, such as the possible sliding and separation between the foundation and the soil, must be further explored. Only a few studies have been conducted on this topic. To obtain realistic conclusions, one must also include nonlinear soil behavior and preferably a good constitutive model. Of particular importance in this respect is to try to predict the initial state of stresses existing in the soil due to the excavation and construction, and to account properly for these initial conditions.

6.4.4 Structure-Soil-Structure Interaction

Most interaction analyses are conducted considering an isolated structure and therefore neglecting the fact that several other buildings are located in the immediate vicinity. A few parametric studies have been published on the structure-soil-structure interaction effect, but they have been limited to a linear-elastic soil, a limited number of cases, and distances between footings which are not too small. Few general conclusions have been derived from these studies because of their limited scope. More work, even for these idealized conditions, is needed. When the structures are very close to each other, as is often the case in nuclear power plants, nonlinear soil effects must be taken into account.

6.4.5 Partial Embedment

The effect of embedment on the dynamic stiffnesses of a foundation is now well understood when separation effects are neglected. Almost no work has been done, however, for structures which have their foundations embedded over part of their perimeter and exposed, or with smaller embedment, over the remainder--a situation which occurs often. Some studies for this situation are warranted to assess the amount of effective radiation damping which can be developed with only partial embedment and to study the effect on the foundation motions (the degree of filtering of the translational motion which occurs in this case is not clear).

6.4.6 Other Foundation Shapes

Most sensitivity studies have been conducted with circular foundations or strip footings (plane-strain problem). Procedures to obtain the dynamic stiffnesses of foundations of arbitrary shape are now available, and some comparative studies with circular foundations have been performed (particularly for rectangular mats). More studies are, however, necessary to assess the importance of the foundation shape for surface and embedded structures. A large fraction of the interaction analyses conducted in

practice today are carried out with two-dimensional or pseudo-three-dimensional models. While knowledge is already available to select the most appropriate dimensions of a strip footing to simulate a circular foundation and to assess the effect of this approximation, more extensive parametric studies (including foundations of other shapes) would help to disseminate this knowledge.

6.4.7 Simplifying Assumptions

Additional parametric studies are needed to determine the sensitivity of the results of probabilistic analyses to the underlying simplifying assumptions: the effect of generating artificial earthquakes to match smooth response spectra corresponding to an average plus one standard deviation, rather than matching the average and then taking the mean plus one standard deviation of the results; the implication of these synthetic motions for nonlinear systems; the differences in response caused by the assumption of a purely stationary process, in contrast to an evolutionary process which accounts only for amplitude variation with time or a more general model which also incorporates the time dependency of the frequency variation.

6.4.8 Computational Details

Several computational details need further assessment through parametric studies: the number of points at which transfer functions must be evaluated and the accuracy of various interpolation procedures for solutions in the frequency domain, the degree of approximation with which the high-frequency components of the response can be predicted, the effect of using a coarse mesh at some distance from the foundation for discrete models, etc.

6.5 SIMPLIFIED PROCEDURES

The sensitivity studies described above should not only lead to a better understanding of the problem and the solution behavior, but should also serve as a basis for the development of simplified, engineering-type procedures.

Even if computer programs were to be available soon to perform accurate analyses, including all possible effects, these procedures would be of great value for preliminary design purposes and to check the validity of the computer results. Their importance is even larger at the present time, considering the large number of uncertainties that exist and the advantages of performing several analyses in order to bound the results or to interpret them statistically.

It would be particularly interesting to develop simple procedures to:

- Estimate the motions of a rigid massless foundation (especially an embedded one) from the specified motion at the free surface of the soil in the free field. Procedures as the one suggested by Morray are very attractive. To derive them, similar studies should be conducted considering typical soil profiles (with variation of soil properties with depth as could result in the free field when accounting for levels of strain) and different trains of waves. It should be noticed that when dealing with waves which are not propagating vertically the soil properties at any time would no longer be homogeneous in the horizontal direction, a factor that cannot be considered with the equivalent linearization and whose possible significance is not clearly known.
- Simulate the effects of nonlinear soil behavior. It should at least be possible to obtain rules, following the lines of Jakub's work, to determine strain-compatible values of shear modulus and damping at various depths in the free field, as a function of the level of excitation at the surface, the general characteristics of the seismic motion, and the initial soil properties (for very low levels of strain). This would allow one to avoid conducting deconvolution studies in each case.
- Determine the stiffnesses of surface foundations of arbitrary shape in terms of the published results for a circular foundation on an elastic half space. This implies deriving rules to modify these results (both the static values and the frequency variation of real and imaginary parts) to account for the shape of the foundation and the layering of

the profile (variation of properties with depth). Accounting for the possible existence of much stiffer, rocklike material at a finite depth and for its effect on the imaginary terms (radiation damping) are of particular importance.

- o Modify the stiffnesses computed above to account for effects of embedment. Rules such as those suggested by Novak or Elsabee seem particularly attractive, but additional work is necessary to evaluate the degree of approximation they provide and their range of applicability. To reproduce a more realistic situation, it would also be interesting to account for conditions of the backfill and possible separation effects.
- o Estimate the dynamic response of the structure, including interaction effects, with models slightly more complicated and realistic than the single-degree-of-freedom system used in previous chapters. This would imply including the foundation mass (which is not negligible), mass moments of inertia, and more than one stick to account for the various components. It may be simpler from this point of view to determine the foundation motions (including now the effect of the structure), then use these results for a normal dynamic analysis of the structure. One should notice, however, that in this case classical programs should be modified to account for rotational as well as translational base motions.
- o Derive amplified response spectra at various floor levels, in terms of the characteristics of the design motions, for the design of equipment without the need to use actual or artificial time histories. This may require further studies on some of the procedures suggested in the literature for working directly with response spectra or specifying the motion in terms of the Fourier spectrum (or power spectrum). Since present procedures derive power spectral densities from response spectra in order to generate artificial earthquakes, one has to wonder if it would not be simpler to specify the input directly in terms of its power spectrum.
- o Estimate the effects of variations in the basic parameters (earthquake characteristics, soil properties, etc.) on the various measures of

response, or account for uncertainties with simplified probabilistic formulations.

Some of the procedures discussed above can be obtained and evaluated, at least on a preliminary basis, from present knowledge. Others are dependent on the basic research and developments mentioned earlier and must therefore wait until the state of the art is improved. Many of them are clearly interrelated.

6.6 TEST PROBLEMS

While most of the methods used at present for soil-structure interaction analysis can be assigned within the classification established here (direct solution versus substructure approach), there are many variations in the way they are implemented. Each time a new procedure or computer program is implemented, it is necessary to have it evaluated for consistency and accuracy. Moreover, in each case, even if accepted programs are used, it is necessary to check that the selected model is appropriate and that no errors have been committed (i.e., that the results are reasonable).

To facilitate this job, it is convenient to have not only simplified procedures, such as those described above, but also a series of test problems for comparison of solutions. In evaluating the accuracy of a given procedure, however, one must consider the effect that an error of a given magnitude, in a particular step, will have on the final results (acceleration, forces, or amplified response spectra at various levels) when dealing with the whole problem and taking into account the provision of the Regulatory Guides (broadening of spectra, etc.).

Selection of the appropriate test cases requires some careful thinking. In addition, some work must be done to issue acceptance criteria on the basis of the results. It would appear, however, that any procedure or computer program used in practice, and particularly discrete models, should be checked as to:

- The ability to reproduce the input design motion at the free surface of the soil deposit when the model is used without excavation or

structure. This would help to validate the motion computed at bedrock, the mesh size, and the boundary conditions.

- The ability to reproduce the dynamic stiffnesses of a circular or rectangular foundation on the surface of the soil deposit, both for the case of a deep soil profile and a finite layer, over the complete frequency range of interest. This would again provide a check on the mesh size, boundary conditions, and generality of the method, or would establish the range of applicability.
- The ability to reproduce, at least in a reasonable, qualitative way, the main effects of nonlinear soil behavior as a function of the intensity of shaking. It may be found that certain procedures are only applicable for low-intensity motions (with small nonlinearities), or only for strong shaking (when nonlinearities are large).
- The ability to reproduce the basic effects of embedment, both in relation to the motions which would occur at the base of a rigid massless foundation, and in relation to the dynamic stiffnesses.
- The validity of the structural model and its ability to reproduce accelerations at different levels, internal forces, amplified response spectra, or only base motions. Requirements on the accuracy of different models should be consistent: at the present time, one gets the impression that the requirements on modal analyses are far more restrictive than those imposed on solutions in the time or the frequency domain. When the structural analysis is performed as a separate step by standard programs, using as input the foundation motions which already include interaction effects (obtained with a crude model of the structure), it must be checked that the rotational components are taken into account or that their effect is negligible.
- The ability to treat general-type excitations or the limited case of vertically propagating waves. While the latter may be sufficient for many practical purposes, it is important to define the conditions for which the program or method is applicable.

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