SEISMIC DESIGN TECHNOLOGY FOR BREEDER REACTOR STRUCTURES

Volume 2: Special Topics in Soil/Structure Interaction Analyses

AGBABIAN ASSOCIATES
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Volume 2: Special Topics in Soil/Structure Interaction Analyses

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PREFACE

In 1974 Agbabian Associates embarked on a project to develop and update seismic design criteria and analysis methods as part of the national long-range program on Breeder Reactor Seismic Technology. This work was initiated by the Division of Reactor Development and Demonstration of the Energy Research and Development Administration. The work was continued under contract with the Department of Energy (DOE) and under contract with Union Carbide Corporation. The Technical Monitors for the project were C. Chester Bigelow for DOE and Claude Pugh and Richard Gwaltney for Union Carbide. The Project Managers at Agbabian Associates were Dr. George A. Young and Dr. Damoder P. Reddy.

The purpose of the study was to identify the problem areas associated with the application of seismic hazard assessment procedures and to establish guidelines for design and analysis of earthquake-resistant breeder reactor structures, equipment, and piping. The result of these research efforts was a series of thirty topical reports covering various topics related to the overall objective; these reports were produced over the span of years 1975-82.

The present volume is part of a subsequent shorter series that condenses the information from the original volumes into four volumes. There are three reasons for this condensed version. The first is that the original volumes were published on a limited distribution basis, and it is intended that the information published in the present volumes be available for a wide distribution and greater accessibility. Second, the information in the new, shorter form will be easier to use and therefore will find wider reception and greater utility in this way. Third, since much of the

information contained in the topical reports has general application, the text of these four volumes is prepared to serve as a useful reference on many general topics in the analysis and design of earthquake-resistant nuclear facilities.

The general title for the four volumes is Seismic Design Technology for Breeder Reactor Structures, and the volumes are entitled:

- Vol. 1. Special Topics in Earthquake Ground Motion
- Vol. 2. Special Topics in Soil/Structure Interaction
- Vol. 3. Special Topics in Reactor Structures
- Vol. 4. Special Topics in Piping and Equipment

The present volume, Volume 2, consists of six chapters which cover these respective topics pertaining to nuclear power plants:

<u>Chapter</u>	<u>Topic</u>
1	Definition of Seismic Input Ground Motion
2	Review of State-of-the-Art Procedures
3	Analysis Guidelines
4	Rock/Structure Interaction Analysis
5	Comparison of Two- and Three-Dimensional Analyses
6	Comparison of Analyses Using FLUSH and TRI/SAC Codes

These chapters are based on six topical reports SAN/1011-122, SAN/1011-111R, SAN/1011-130, SAN/1011-102, SAN/1011-128, and SAN/1011-132.

Technical contributions to these six topical reports were made by M.S. Agbabian, L. Gebhardt, Y.C. Lee, S.F. Masri, D.P. Reddy, S.D. Werner, D. Van Dillen, and G.A. Young. The four volumes were produced under the general direction of M.S. Agbabian;

technical editor was J.W. Athey. While preparing these volumes, Agbabian Associates received guidance from C. Chester Bigelow and Richard Gwaltney. It is hoped that these volumes will be a useful reference on topics that are essential for the safe design of nuclear facilities.

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CHAPTER 1

DEFINITION OF SEISMIC INPUT GROUND MOTION

The results of a comparative study in which one- and two-dimensional mathematical models were used to compute the variation in intensity of earthquake vibratory ground motion with depth below the ground surface are presented here. The objective is to provide information that can be used when selecting a mathematical model to predict the vibratory ground motions required as input to soil/structure interaction models. Such models are used in the design analysis of nuclear power plants.

First, regulatory requirements are reviewed, along with important characteristics of one- and two-dimensional analytical models and the importance of the soil profile and different waveforms. Problems in modeling techniques are discussed next. Then two comparative analyses are given, the first using comparative one-dimensional SHAKE and two-dimensional finite element analyses for a site (Humboldt Bay) for which only strong body wave earthquake motion was recorded; the second using such analyses for a site (Orion Avenue) at which both strong body and surface wave motions were recorded. In this way, it can be verified that the two codes give comparable response to one-dimensional wave forms, and also it can be determined how well the two codes represent surface wave motion and the attenuation of surface wave motion with depth.

1.1 BACKGROUND INFORMATION

1.1.1 REGULATORY REQUIREMENTS

The Code of Federal Regulations 10 CFR 100, Appendix A, requires that design earthquake vibratory ground motions, i.e., the Safe Shutdown Earthquake (SSE) and the Operating Basis Earthquake (OBE), be defined by

response spectra. The response spectra for the SSE must correspond to the maximum vibratory accelerations that may occur at the elevations of the foundations of the various nuclear power plant structures, taking into account the characteristics of the underlying soil material in transmitting the earthquake-induced motions. This regulation also requires that the engineering method used to ensure that the required safety functions of the nuclear power plant are maintained during and after the occurrence of the vibratory ground motions associated with the SSE, shall involve the use of either a suitable dynamic analysis or a suitable qualification test. The analysis or test must take into account soil/structure interaction effects and the expected duration of the vibratory ground motion. At the present time, verification of response of large structures is provided only by mathematical analyses.

Acceptable procedures for defining earthquake vibratory ground motion and acceptable soil/structure interaction analysis procedures are described in the Standard Review Plan* (SRP) of the U.S. Nuclear Regulatory Commission (USNRC, 1975). Section 3.7.1 of this document indicates that design response spectra should be defined for the free-field soil or rock condition (i.e., with no structure present) and that it should ordinarily be considered applicable at the finished grade level (i.e., ground surface) of the site. Acceptable soil/structure interaction analysis procedures are set forth in

*The Standard Review Plan was prepared by NRC for the guidance of the regulatory staff responsible for the review of license applications. Therefore, it does not represent a federal regulation, but represents procedures which if followed will generally be found acceptable to NRC.

Section 3.7.2 of the SRP. Either discrete mass/spring or finite element methods may be used for most site and structure embedment conditions, but only the latter is currently accepted for deeply embedded structures in soils.

Acceptable procedures for developing input time histories for soil/structure interaction analyses are described in Section 3.7.1 of the SRP. This section discusses time histories to be input only at the base of the soil/structure interaction system (model) and identifies deconvolution analyses based on the SHAKE code as an acceptable method. Even though this section indicates that other equivalent computer codes and analysis techniques are acceptable, the combined application of the SHAKE/FLUSH* computer codes for computing the base input and the soil/structure response have become an almost standard procedure.

The acceptance criteria set forth in the SRP for the time history computed for the base of the soil/structure interaction system model require that when the time history is applied at the base of the idealized soil profile model (SHAKE) or at the base of the soil/structure interaction system model (FLUSH), the resulting free-field vibratory ground motion computed at the finished grade level (ground surface) should give response spectra that envelop the design (SSE) response spectra. A second requirement is that the response spectrum of the vibratory ground motion calculated at the elevation of the structure foundation using free-field models should be not less than 60% of the (ground surface) design response spectrum at all significant frequencies (0.2 to 15[†] Hz), unless a lower response can be justified. The analysis method is also required to consider the variation in soil properties within the soil profile, and to account for the dependence of the soil modulus and damping on the strain level.

Although not stated in the SRP, the reason for the 60% limitation on the response at the foundation level is that SHAKE code analyses frequently indicate a lower value of response at the foundation level in a frequency band that includes the fundamental shear mode frequency of the site model. This problem was brought to the attention of the Seismic Subcommittee of the Advisory Committee on Reactor Safeguards (ACRS) in February 1977. The Subcommittee concluded that the applicability of one-dimensional models in the future should be reviewed on a case-by-case basis, with the recommendation that the SHAKE code not be used for rock sites covered with a shallow depth of soil. Since then, the engineering profession has proceeded with caution and uncertainty in this area. For this reason, basic studies are needed to provide guidance on conditions for which one-dimensional models can be used, as well as guidance on conditions that require more complex models.

1.1.2 ANALYTICAL MODEL CHARACTERISTICS

For the comparison of analytical models, the one-dimensional models are based on the SHAKE code and the two-dimensional models are based on the TRI/SAC code. A summary of important characteristics of models formulated from these two codes is provided here.

*The SRP refers to the SHAKE and LUSH codes; FLUSH is a later version of the LUSH code. All reference here to this computer code will be to the FLUSH code.

†The SRP indicates 0.2 to 50 Hz. However, earthquake records are not reliable above 15 Hz. It is normally assumed that there is no amplification of the ground acceleration for system frequencies above 33 Hz. Therefore, the response between 15 and 33 Hz can be easily inferred and response above 15 Hz does not usually need to be computed.

1.1.2.1 SHAKE Code Characteristics

The SHAKE code basically computes the horizontal response of a system of homogeneous, viscoelastic soil layers of infinite horizontal extent to a vertically propagating (one-dimensional) shear wave. It utilizes a continuous solution to the one-dimensional wave equation (Kanai, 1950) that has been adopted for use with transient motions through a Fast Fourier Transform Algorithm (Cooley and Tukey, 1965). Each soil layer in the system is defined by its shear modulus, damping ratio, density, and thickness. The shear modulus and damping ratio are assumed to be independent of frequency, but to be dependent upon the level of shear strain developed in the soil layer. The strain dependence of these properties is introduced by means of an equivalent linear soil model which utilizes an iterative procedure to obtain the layer shear modulus and damping compatible with the effective shear strain developed in each layer. The base of the model is assumed to have arbitrary elastic properties (rock) that do not vary with strain. The elevation of the base is arbitrarily established by the investigator and is determined largely by judgment based on the soil and rock conditions being considered.

Input motions for the SHAKE code can be specified at any layer in the system and resulting motions can be computed at any other layer. The most common procedure is to specify the input motion at the ground surface (i.e., the design (SSE) ground motion). The wave equation is then used to compute the motions at different subsurface soil layers and at the base of the model by a deconvolution routine. However, developers of the code (Schnabel et al., 1972) indicate that results from this procedure can be quite sensitive to small differences in surface input motion and subsurface soil properties. There are two additional input procedures. One is to specify

the input at the base of the model. The second is to specify surface rock outcrop motions which are then converted to subsurface motions in an identical rock medium at a given distance below the ground surface (i.e., at the base of the model). This computation introduces the difference in boundary conditions between the free rock surface and the subsurface rock medium constrained by the overlying soil layers.

The SHAKE code has three important characteristics. First, because the analytical technique is based on the use of a Fast Fourier Transform algorithm, it is possible to provide iterative solutions at low cost such that the final solution is based on equivalent elastic soil properties that are supposedly consistent with the strain level of the computed response. Second, relationships between effective strain levels and equivalent elastic moduli and damping for cohesive and cohesionless soils are included in the code although other relationships can be used. This capability is considered important when analyzing soil sites because of the nonlinear characteristics of most soils. Third, SHAKE is the only code that has the capability of computing the input at subsurface levels from a surface input. This is extremely important since there are practically no subsurface records that can be used to devise input for the base of the soil profile model. However, the absence of subsurface records means that there is little experimental verification for SHAKE code deconvolution solutions. This is an important factor that is frequently overlooked.

The major weaknesses of the SHAKE code are associated with its simplicity. First, only one-dimensional vertically propagating shear wave motions are transmitted by the model. Second, the code can consider only sites with horizontal soil layers and topography. Third, since only a vertical propagating shear wave is transmitted normal to horizontal soil layers, the

site model responds primarily in the fundamental shear mode (i.e., primarily as a one-degree-of-freedom system). Thus, the response may be abnormally strong in a frequency band that includes the fundamental shear mode frequency of the model. Fourth, the horizontal and vertical components of motion must be decoupled and treated separately. As a result, even though a Rayleigh wave (which has elliptical particle motion) or a Love wave (which is essentially a horizontally propagating shear wave) may be present in the input, the horizontal component of motion will be modeled as a vertically propagating shear wave by the SHAKE code. Also, since the two components must be treated separately, the program may iterate to different material properties for the vertical component of motion than used in the solution for the horizontal component of motion, unless the analyst locks out the iterative routine for one of the components.

When FLUSH is used to model a free-field condition (i.e., the structure is not present), the resulting model behaves essentially as a one-dimensional model. This code utilizes the SHAKE deconvolution routine for computing the base motion. Since the FLUSH code was not used in this investigation, its characteristics will not be described further. Reference should be made to Schnabel et al. (1972) and to Lysmer et al. (1975) for additional information on the SHAKE and FLUSH codes, respectively.

1.1.2.2 Finite Element Code Characteristics

Large capacity finite element computer programs can be used to model free-field soil profiles and to provide soil/structure interaction models. The capability exists to provide either one-, two-, or three-dimensional models. Since computing costs quickly become excessive with three-dimensional finite element continuum models, most models are one- or two-dimensional. Two-dimensional models are required to

treat inclined soil and rock layers. Two-dimensional models can also transmit body (shear and dilatational) and surface* (Rayleigh) seismic wave motions simultaneously. For such models, the seismic input motion can be applied at either a side or bottom boundary. Two components of motion (horizontal and vertical) should be input simultaneously to two-dimensional models. Experience indicates that the input motion quickly adjusts to become compatible with the model geometry and material properties assumed. As a result, a variety of seismic wave types of different directions of propagation can develop and be transmitted by a two-dimensional finite element model. Also, in soil/structure interaction models, the seismic energy can be radiated away from the structure in a realistic manner (i.e., in vertical, horizontal, and inclined directions).

Disadvantages of two-dimensional finite element models result primarily from computer time (cost) requirements. Soil properties are generally represented by equivalent elastic properties, although computer programs are available that can treat nonlinear soil behavior. Since time step analyses are required, several hours of computer time are usually needed for each elastic analysis and an even greater amount is required for a nonlinear analysis. While equivalent elastic soil properties are used, it is usually not practical to use iterative solutions to obtain material properties that are completely compatible with the response strain level. Instead, approximate agreement is

*The Rayleigh wave appears in a vertical radial plane which passes through the epicenter and the site. The Love wave appears in a plane normal to the radial plane. While the Rayleigh wave can be propagated in a conventional two-dimensional finite element model, special modeling techniques are required to propagate Love waves with a two-dimensional model (see Lysmer and Drake, 1971).

obtained by first performing a SHAKE code analysis to obtain an indication of the approximate strain levels and approximate equivalent elastic moduli and damping levels applicable to the different soil layers. Modal damping rather than element damping (SHAKE code) is used in two-dimensional finite element elastic models. It should be particularly noted that the modal damping in two-dimensional finite element models and the element damping used in the SHAKE and FLUSH codes are not equivalent.

There are several computer programs, such as SAP (Wilson et al., 1970) and TRI/SAC (AA, 1976c), that can be used to provide two-dimensional elastic finite element analyses. Examples of the application of TRI/SAC to a site with inclined rock layers and to a rock/structure interaction analysis are provided in AA (1976a and 1975, respectively). The TRI/SAC code is used to provide the comparative two-dimensional finite element free-field analyses required for this study.

The TRI/SAC code is a large capacity digital computer program for three-dimensional analysis of structural systems using the finite element approach. The TRI/SAC code provides for specific representation of the more common types of structural elements including trusses, beams, membranes, axisymmetric solids, three-dimensional solids, plates, shells, and nonlinear springs and dashpots. A more general type of structure may be modeled by combinations of these elements. The structural elements in TRI/SAC were taken from the SAP code (Wilson et al., 1970).

The analysis types available include static response, dynamic response by normal mode method, dynamic response by direct integration method, response spectrum and determination of mode shapes and frequencies. The program includes a capability for the solution of certain

nonlinear dynamic problems by the direct integration method. The mode shapes and frequencies of a structural system can be extracted using three different procedures: Rayleigh-Ritz method, inverse iteration method, and subspace-iteration method.

The code also provides options to produce three-dimensional plots of the finite element model, to minimize the bandwidths of the stiffness matrix, and to plot response time histories.

All the soil/structure interaction analyses performed under this study utilized only two finite element types from the available element library in TRI/SAC. These are plane-stress and plane-strain elements and three-dimensional solid elements. The plane-stress and plane-strain finite elements are useful in representing an elastic continuum, which is in a known state of lateral constraint.

1.1.3 IMPORTANCE OF THE SOIL PROFILE

The manner in which strong earthquake vibratory ground motion is transmitted is complex. This results primarily from the complexity of the faulting mechanism which releases the vibratory energy, and from the complex geology through which the seismic motion is transmitted. Generally neither has been well defined for past earthquakes and can only be assumed for future earthquakes. Near surface soil properties at the site, such as soft clays, may also have some influence on the motions experienced at the ground surface, as well as at shallow depth. However, there is frequent disagreement on the relative importance of the near surface soil profile on earthquake vibratory ground motion, particularly for firm soil conditions.

Much of the difference in opinion on the relative importance of the near surface soil profile results from the manner in which the

problem is approached. One approach, which is from a broader scientific viewpoint, gives consideration to the fact that the ground motion usually consists of a combination of body and surface waves. The effect of the total geologic profile on the orientation and combination of wave forms is considered important, as well as the ratio of the wave length of the transmitted seismic waves to the thickness of different rock and soil layers. For example, rock and soil layers for which the density increases with depth below the ground surface will refract both dilatational and shear waves upward so that at reasonable distances from the fault, both body waves will usually be propagating in a nearly vertical direction. However, low modulus near surface layers may also trap energy and lead to a buildup in the amplitude of surface waves. Therefore, from an overall point of view the wave combinations are complex and cannot all be represented by one-dimensional mathematical models.

The alternate approach, currently used by the geotechnical engineer, is a simple and directly applicable engineering solution that is neither costly nor time consuming. The geotechnical engineer assumes that since the body waves are usually oriented nearly vertically, with the dilatational wave producing the vertical motion and the shear waves producing the horizontal motion, it is logical to treat these waves separately with one-dimensional mathematical models. These procedures in effect ignore the surface wave contribution. When using earthquake records that include surface wave components as input to one-dimensional mathematical models, the horizontal component of the surface waves will automatically be converted to vertically propagating shear waves, as previously noted. Therefore, the manner in which the surface wave motions are propagated and attenuate with depth will not be correctly represented by one-dimensional models.

Since geotechnical engineers are strongly interested in soil properties, great emphasis in the past has been placed on the effect of the nonlinear behavior of the soil profile on one-dimensional seismic wave motion analyses. While this may be essential, it should not be overlooked that other equally important factors, such as the orientation and the effect of different seismic wave forms, are ignored. The method of treating nonlinear behavior is also greatly simplified and subject to question. For example, the soil is assumed to remain elastic, but the modulus is reduced and the energy loss is represented by equivalent damping. Both values (the reduced modulus and damping) are based largely on laboratory tests of soils in shear. Also, with a one-dimensional shear wave model, the greatest strains generally occur at the base of the model. Therefore, the greatest reduction in modulus and the greatest damping also occur at the base of the model, which is not realistic if the model is extended to a depth of several hundred feet. The combined effect of converting all horizontal components of motion to vertically propagating shear wave motions and considering all energy losses to be due to shear strains may therefore overestimate the effect of the soil profile on wave motion. It would seem that an essential prerequisite to the use of the one-dimensional shear wave model for engineering predictions is assurance that the dominant seismic wave forms that will be experienced at a proposed site be nearly vertically propagating body (shear and dilatational) waves. There is strong evidence that this condition has not prevailed at some sites where strong earthquake ground motions have been recorded.

1.1.4 IMPORTANCE OF SEISMIC WAVE FORMS

Studies by Drake and Mal (1972) and by Hanks (1975) indicate the importance of the surface wave contribution to the ground motions

recorded at many stations during the 1971 San Fernando earthquake. The study by Drake and Mal is based on a two-dimensional finite element model of a north-south profile of the San Fernando Valley from which surface wave (Love and Rayleigh) mode shapes were extracted at different stations along the profile. The study by Hanks provides a comparison of the long period ground motions recorded at 78 sites along four radial profiles from the epicenter of the 1971 San Fernando earthquake. One profile includes the profile modeled by Drake and Mal. These studies supplement each other and are of interest to this investigation.

The study by Drake and Mal is important to the engineer because it demonstrates the effect of layered rock and soil media of increasing density with depth on the shape of the surface wave, and also demonstrates the depth to which surface wave effects frequently extend. The two-dimensional finite element model of the San Fernando Valley profile considered by Drake and Mal extended to a depth of 125,000 ft and considered a radial plane 16,000 ft in horizontal extent from Rinaldi to Nordhoff Streets in the San Fernando Valley. These street locations are approximately 9 and 12 mi, respectively, from the epicenter of the San Fernando earthquake. The Orion and Vanowen Street strong motion recording stations are located approximately 1 and 3 mi south, respectively, of Nordhoff Street (epicentral distances of about 13 and 15 mi, respectively). The alluvial soil layer increases in depth across the valley with distance from the epicenter, although its exact depth is not precisely known.

The study indicated that surface wave motions extend several thousand feet below the ground surface and do not attenuate significantly in the top 50 to 100 ft, which is the depth of greatest interest in soil/structure interaction analyses.

It was noted that, for Rayleigh waves, the surface amplitude of the vertical component of the fundamental mode is nearly doubled, but the horizontal component is reduced in passing through the model. However, the horizontal component of the first higher mode is greatly increased, as is the vertical component. This is significant since the authors note that more than 30% of the incident fundamental Rayleigh wave mode at a period of 1/2 sec is converted into higher mode wave energy by the model. Even though an attenuation in amplitude is normally expected with distance, larger amplitude Love and Rayleigh wave motions are indicated as the seismic waves advance for the subsurface conditions considered.

Because of the great number of records obtained from the 1971 San Fernando earthquake, Hanks (1975) was able to provide a very informative study of the long period ground motion resulting from this earthquake. One profile considered was approximately due north-south, essentially along the profile considered by Drake and Mal. Hanks notes the observed growth in amplitude of the Rayleigh and Love waves along this profile with distance from the epicenter. His data includes records from 8244 Orion Avenue and 15107 Vanowen Street along the north-south profile. Figure 1-1a provides a comparison of the displacements computed from the N-S components of acceleration for the Orion and Vanowen stations, and the E-W components are compared in Figure 1-1b. The Rayleigh wave motions appear in the radial plane on the vertical and horizontal (N-S) components of the record, while the Love wave motions appear on the transverse (E-W) component. Displacement records were used in this study, as the presence of the surface wave components is more clearly depicted in the computed displacement record than in the recorded acceleration time histories.

FIGURE 1-1. COMPARISON OF RADIAL AND TRANSVERSE DISPLACEMENT TIME HISTORIES FOR ORION AND VANOWEN STREET RECORDING STATIONS, 1971 SAN FERNANDO EARTHQUAKE (from Hanks, 1975)

While higher frequency motion is superimposed on both records, strong Rayleigh wave motion is evident in Figure 1-1a. Likewise strong Love waves are evident at both stations in Figure 1-1b. It is of interest to note that the maximum amplitudes of the Rayleigh and Love waves are greater at Vanowen Street than at Orion Avenue, and that the maximum amplitudes of each represent the maximum displacements that occur on the record. It is also of interest that the maximum amplitude of the Love wave component occurs on the same cycle at the two stations (at about 17 sec).^{*} However, the maximum displacement amplitude on the N-S component (Rayleigh wave) appears to have occurred slightly later (at about 10 sec) at the Orion than at the Vanowen stations (at about 8 sec) probably due to the superposition of a higher frequency wave motion on the Orion record at 8 sec.

Although not shown in Figure 1-1, the computed velocity records for the two horizontal components of the Vanowen and Orion stations

indicate peak velocities on the same cycles during which peak displacements occurred. The peak velocities at Vanowen Street were again stronger than occurred at Orion Avenue. The results described above are in agreement with the analytical study by Drake and Mal which indicates that surface wave energy is fed into the alluvial soil layer as the surface waves progress across the valley. Since the peak velocities and peak displacements at these stations are definitely associated with strong surface waves, it is difficult to conclude that these motions could be realistically considered by a one-dimensional model which would automatically convert the large amplitude horizontal component of surface wave motions into vertically propagating shear wave motions.

^{*}The two recorders were not activated simultaneously but were activated by the arrival of the dilatational wave at each site. Hanks adjusted the time scale to include the time of travel between stations.

1.2 PROBLEMS IN MODELING TECHNIQUE

1.2.1 ROCK SITE COVERED WITH SHALLOW ALLUVIUM

Comparative analyses using SHAKE and TRI/SAC codes demonstrate that almost identical results can be obtained with the two codes for rock sites covered with a shallow depth of soil, even though the Seismic Subcommittee of the Advisory Committee on Reactor Safeguards in February 1977 recommended that the SHAKE/FLUSH codes not be used for such sites. The problem results from the manner in which the deconvolution routine is used, and not from the use of a one-dimensional model.

1.2.1.1 SHAKE Analyses

The site considered consists of 45 ft of sand underlain by a deep bed of basaltic rock several thousand feet in thickness. The shear modulus and damping assumed at the start of the SHAKE code analyses, and the properties used in the final (fifth) iteration, are given in Table 1-1. These values resulted from a

TABLE 1-1. MATERIAL PROPERTIES USED IN SHAKE CODE ANALYSIS

Depth, ft	Material	Shear Modulus, lbs/in ²		Damping, percent	
		Initial	Final	Initial	Final
0-5	Sand	25,000	20,000	6	4
5-10	Sand	26,200	13,850	6	8
10-15	Sand	27,000	11,100	6	11
15-25	Sand	28,000	9,600	6	14
25-35	Sand	29,000	6,500	6	17
35-45	Sand	40,000	10,700	6	16
45-70	Basalt	405,000	377,500	1	1
70-95	Basalt	405,000	370,000	1	1
95-120	Basalt	405,000	366,000	1	1
120-145	Basalt	405,000	365,000	1	1

SHAKE model of a profile 145 ft deep (45 ft of soil and 100 ft of basalt). The N69°W component of the Castaic 1971 San Fernando record, scaled to a peak horizontal ground acceleration

of 0.54 g, was used as a base input at the 145 ft depth. The scaled spectrum for this input for 5% damping is compared in Figure 1-2a with a Regulatory Guide 1.60 spectrum scaled to 0.34 g. The latter is a design criteria spectrum for the site. In this case the design criteria spectra were defined for the level of the rock surface which is also the foundation level of the major structures. Ordinarily, the design criteria spectra are defined for the ground surface. It can be noted in Figure 1-2a that by scaling the input motions to 0.54 g, an

(a) Horizontal spectra

(b) Vertical spectra

FIGURE 1-2. COMPARISON OF INPUT SPECTRA WITH REGULATORY GUIDE 1.60 SPECTRA

input response spectrum is obtained that equals or exceeds the Regulatory Guide spectrum between 0.8 and 20 Hz, with only minor exceptions. Ordinarily a record would be used that would envelop the criteria spectrum over a broader frequency range. This time history has been used in this example only because it is short and economical to use, and the resulting spectra envelop the criteria over the frequency range of greatest concern to nuclear power plant design. Two different input cases were considered.

In the first case, the horizontal time history (Fig. 1-2) was input at the 145 ft depth with no correction to account for the fact that a free surface does not exist at this depth. The resulting SHAKE code response at a 45 ft depth experienced only slight modification from the input at a 145 ft depth (see Fig. 2-2, AA, 1979). This should be expected in a high velocity rock with low damping between the 45 ft and 145 ft depths (see Table 1-1). Between 1 and 3.5 Hz, the response was amplified slightly at the rock surface level (45 ft depth) in comparison to the input spectrum, and between 3.5 and 12 Hz there were slight amplifications and attenuations. The variations in response between 3.5 and 12 Hz were probably partly due to the fact that free surface input motions were used at the 145 ft depth.

While the input response spectrum was modified only slightly by the basalt, significant amplification occurred in the soil layer. Peak response is indicated at about 2.5 Hz, but maximum amplification occurred at about 3.5 Hz. The response decayed gradually for frequencies both above and below this central value. Thus, the behavior of the one-dimensional model was much like that of a single-degree-of-freedom system, which experienced its greatest amplification when the input and model frequencies were resonant. This suggested that the

fundamental shear mode of the soil layer is about 3.5 Hz.

In the second analysis for this site in which the Castaic input time history was applied at the ground surface, the response at the rock surface (depth of 45 ft) was computed using the deconvolution routine in the SHAKE code. The resulting response spectrum at the 45 ft depth was compared with the response spectrum at the 45 ft depth for the first case analysis. The latter was based on the same input, but the input was applied at the base of the model (145 ft depth). It was noted that in the 1 to 7 Hz range, the response spectrum computed by deconvolution is much lower than the spectrum computed from the base input, with the greatest deviation occurring in the 2.5 to 3.5 Hz range. In the first case the peak response in the input and at all depths occurs at 2.5 Hz, whereas in this case (Fig. 1-3) the maximum response in the input (at the ground surface) at 2.5 Hz does not appear in the deconvolved ground motion. In fact, there is strongly suppressed response at the rock surface between 2 and 6 Hz.

The peak ground acceleration computed at the surface from the base input model is about 0.8 g, rather than the 0.54 g used in the scaled N69°W Castaic record. Thus, to be comparable, the surface input to the deconvolution model should have been scaled to 0.8 g rather than to 0.54 g. While this will raise the level of the depressed zone of the spectrum computed by deconvolution, the stronger spectrum resulting from the stronger input motion will still not match the rock surface spectrum computed from the base input model. This is demonstrated in Figure 1-4.

In the third analysis (Fig. 1-4), rock surface response spectra were computed by deconvolution from the Castaic record scaled to two different intensity levels and input at the

FIGURE 1-3. COMPARISON OF ROCK SURFACE RESPONSE SPECTRA COMPUTED BY SHAKE DECONVOLUTION AND BASE INPUT PROFILE MODELS — SHALLOW SOIL SITE

FIGURE 1-4. COMPARISON OF ROCK SURFACE RESPONSE SPECTRA FOR TWO DIFFERENT LEVELS OF SURFACE INPUT FROM SHAKE CODE — SHALLOW SOIL SITE

ground surface. The Regulatory Guide 1.60 design criteria spectrum scaled to 0.34 g is also provided. It will be noted that increasing the strength of the input motion to 0.8 g does not fill the depressed central portion of the spectrum computed by deconvolution, but merely translates the spectrum upward with only a slight change in shape. The stronger rock

surface spectrum (Fig. 1-4) is 30% too low in the 3 to 4 Hz range, and 30% too high in the 8 to 10 Hz range when compared to the Regulatory Guide 1.60 criteria spectrum. The 8 to 10 Hz bulge would make the spectrum uneconomical to use in nuclear power plant design.

Reflection on the comparative results from the above three cases quickly leads to the conclusion that the distortion in the shape of the subsurface spectrum computed by deconvolution can be avoided only by using the time history computed for the ground surface from the base input model as input to the deconvolution analysis. In other words, both the surface response spectrum and the surface time history must be completely compatible with the site subsurface conditions assumed for the model. If the deconvolution analysis is based on a time history that matches an arbitrary surface spectrum, the resulting subsurface spectrum will usually exhibit suppressed response in the frequency range that should be exhibiting maximum site response.

Prior to the action of the Seismic Subcommittee of the ACRS in February 1977, the input time histories for the base of soil/structure interaction models were frequently computed using a SHAKE code deconvolution analysis based on artificial time histories that matched a Regulatory Guide 1.60 ground surface criteria spectrum. This generally resulted in an input time history that gave greatly reduced subsurface response in the frequency range that should be exhibiting the greatest site response. Justification for the use of these time histories cannot be based on the logic that the distorted response results because Regulatory Guide 1.60 spectra stipulate overly conservative response at the surface that is not consistent with the site profile. Since the RG 1.60 spectrum is not consistent with site profile, the Regulatory Guide 1.60 spectrum may be less conservative at frequencies where deconvolution analyses

the Regulatory Guide 1.60 design criteria spectra through the widest frequency range possible. Specifying input to the TRI/SAC model requires care and sometimes a preconditioning analysis. The problem is how to phase the input along the vertical input boundary, when only the input record for the ground surface is known. Since the phasing and variation in intensity of motion with depth are not known, it has been found from experience that a workable procedure is to input the two components of motion simultaneously along the left vertical boundary with no time phasing or variation in intensity of motion. The input wave motions will then be modified when transmitted through the model to be consistent with the geometry (i.e., layer thickness and inclination) and material properties assigned to the model. Response motion can then be recorded at different depths along vertical Reference Lines (i.e., A, B, and C in Fig. 1-5) to establish that the motion does become stable and properly phased. Properly phased motion that is stable and consistent with the model (for example, along Reference Line C in Fig. 1-5) can then be monitored at different depths and used as input to the soil/structure interaction model. The soil/structure interaction model in this case would be Figure 1-5 modified to include the structure.

Although greater detail could have been used in selecting soil properties for the TRI/SAC model given in Figure 1-5, an average equivalent elastic shear modulus was selected for the entire 45 ft depth of soil. Table 1-1 indicates that the equivalent elastic shear modulus obtained during the final iteration with the SHAKE code for this profile varies from 6500 psi at the 25 to 35 ft depth to 20,000 psi for the upper 5 ft of the soil profile. The average final shear modulus given in Table 1-1 for the soil is about 11,000 psi, with the average initial value being about 27,000 psi. An

average equivalent elastic shear modulus for the soil of 15,000 psi was arbitrarily selected and used for the TRI/SAC analyses. A uniform value of 372,000 psi was assumed for the equivalent elastic shear modulus of the basalt. Poisson ratio's of 0.42 and 0.30, respectively, were assumed for the soil and rock. Equivalent Young's moduli were, therefore, 43,400 psi for the soil, and 966,000 psi for the basalt.

It has been noted that the SHAKE code utilizes element damping that is selected by iterative solutions to be compatible with the strain level experienced by each soil element, or layer. The TRI/SAC code in contrast utilizes modal (viscous) damping that is expressed as a function of the mass and stiffness matrices. The modal critical damping ratio, λ_n , is expressed as a function of the modal natural frequency, ω_n , and the mass and stiffness matrix constants α and β , respectively, by the following equation:

$$\lambda_n = \frac{\alpha}{2\omega_n} + \frac{\omega_n}{2} \beta \quad (1-1)$$

Considerable judgment and experience are required to select proper values for the constants α and β .

In Figure 1-5, only the top 45 ft of the model is soil, while the lower 730 ft is basalt. Table 1-1 indicates that layer damping in the basalt from the SHAKE code analysis is about 1%, while layer damping in the soil averages 10 to 15%. It is obviously not possible to select one set of modal damping values that will approximate the behavior of both materials. Therefore, if motion in the top 45 ft is of prime concern, the damping should be determined largely by the soil. If, on the other hand, rock motions are of greater concern, the damping should be determined largely by the rock. In order to demonstrate this, two modal damping relationships have been used in separate

TRI/SAC analyses. The two relationships have been designated as the "A" damping (soil) relationship and the "B" damping (rock) relationship. In "A," damping values for α and β are chosen as 2.09 and 0.0027, respectively. In "B," damping values of $\alpha = 1.05$ and $\beta = 0.0013$ are used.

Figures 1-6 and 1-7 provide a comparison between the TRI/SAC and SHAKE code analyses. Figure 1-6 provides response spectra for the ground surface. For "A" damping (soil), almost the same response results from the TRI/SAC analysis as obtained from the SHAKE analysis except the peak response occurs at a higher frequency for the TRI/SAC analysis. This implies that there is a difference in the one- and two-dimensional fundamental mode frequencies. If "B" damping (rock) is used in the TRI/SAC model, extremely high ground surface response occurs at the shear mode frequency (3.5 Hz) of the soil layer. Obviously, the "A" damping (soil) is more realistic when determining the surface (soil) response. However, the top 5 ft of a sand layer is far from being an elastic material under shear wave motion. Therefore, the peaks computed by both the SHAKE analysis at 2.5 Hz and the TRI/SAC analysis at 3.5 Hz for "A" damping, are no doubt greater than the true response. This could be established only by a nonlinear analysis.

Figure 1-7 provides a comparison of the computed rock surface response at a depth of 45 ft based on the SHAKE and TRI/SAC analyses. The SHAKE code analysis is based on input at the base of the 145-ft model. For "B" damping (rock), excellent agreement is obtained between the SHAKE and TRI/SAC analyses. The higher "A" damping (soil) causes a loss of energy and a lower response in the rock.

The rock surface spectrum computed by the SHAKE deconvolution procedure is not

FIGURE 1-6. COMPARISON OF SHAKE AND TRI/SAC SURFACE SPECTRA FOR SHALLOW SOIL SITE

FIGURE 1-7. COMPARISON OF SHAKE AND TRI/SAC ROCK SURFACE SPECTRA (45-FT DEPTH) FOR SHALLOW SOIL SITE

shown in Figure 1-7. Reference to Figure 1-4 should confirm the fact that the deconvolved spectrum does not compare favorably with any of the spectra in Figure 1-7.

1.2.1.3 Summary

These comparative analyses demonstrate that for a site having a shallow soil layer underlain by high density rock, practically the same response can be obtained from one-dimensional (SHAKE) and two-dimensional (TRI/SAC) analytical models, provided the input to the SHAKE model is made at the base of the model. However, the conditions considered in this example are ideal for obtaining agreement, since the high velocity rock layer generates vertically propagating shear waves in the soil layer in the TRI/SAC model that are comparable to the wave motions assumed in the SHAKE model. The comparison further indicates that the major problem in using a SHAKE model for this site condition results when the deconvolution procedure is used with a surface time history and a surface response spectrum that are not consistent with the site profile.

1.2.2 LAYERED ROCK SITE WITH A LOW-MODULUS SURFACE LAYER

Comparative SHAKE and TRI/SAC analyses for a layered rock site with a low modulus surface layer demonstrated that for sites where wave forms other than nearly vertically propagating shear waves are present, the SHAKE and TRI/SAC models do not give comparable results (AA, 1979). These analyses show that the use of a time history that is not consistent with the model properties for SHAKE code deconvolution analyses, results in greatly distorted response spectra at the foundation level of deeply embedded structures.

The mesh used to compute the free-field response of a site consisted of alternating beds of siltstone and limestone with a low modulus surface layer of weathered siltstone. The mesh adapted to this site is similar to the one shown in Figure 1-5. The total width of the mesh is 1050 ft and the height is 774 ft. The top 30 ft is identified as weathered siltstone over a 184-ft layer of siltstone. Below this, a 130-ft layer of limestone was located above a 150-ft layer of siltstone, below which a 280-ft layer of limestone is situated. Material properties used in the model are given in Table 1-2.

Regulatory Guide 1.60 response spectra scaled to a peak ground acceleration of 0.18 g were defined as the design criteria spectra at the foundation level of the structures for this site, which is assumed to be at a depth of 94 ft. Artificial time histories which provide response spectra that envelop the horizontal and vertical criteria spectra were used as input. The two components of motion were input simultaneously at the left vertical boundary with no time phasing or variation in intensity with depth.

Since this is a rock site, damping is considerably less than occurs for soil sites. An average of about 4% of critical was specified for modes between 2 and 25 Hz. The response spectrum at the foundation level of the structures is amplified somewhat for frequencies below 6 Hz and is attenuated above 7 Hz, when compared to the input spectrum. The input time history would need to be modified slightly to provide closer spectral agreement, but this

TABLE 1-2. MATERIAL PROPERTIES USED IN LAYERED ROCK SITE MODEL

Material	Young's Modulus, psi	Shear Modulus, psi	Poisson's Ratio	Unit Weight, pcf	Shear Wave Velocity, fps
Weathered Surface Layer	250,000	96,000	0.3	155	1700
Siltstone	1,000,000	385,000	0.3	165	3300
Limestone	1,500,000	577,000	0.3	165	4000

is not needed for the comparisons provided here. The ground surface spectrum is amplified significantly when compared to the input spectrum, with the greatest amplification occurring between 4 and 10 Hz. Obviously, in order to maintain the amplifying characteristics of the site between 4 and 10 Hz with a SHAKE surface deconvolution analysis, the input time history would have to agree with the site-dependent surface spectrum. Deconvolution from a time history that matches the Regulatory Guide 1.60 spectrum for this site can only result in a badly distorted subsurface response spectrum.

1.3 COMPARATIVE ANALYSES BASED ON HUMBOLDT BAY SITE

1.3.1 INTRODUCTION

The results of one-dimensional SHAKE and two-dimensional finite element TRI/SAC analyses that have been based on the subsurface conditions assumed to be present at the Humboldt Bay Power Plant site are provided here. This site and earthquake record were selected for the analyses for two reasons. First, a study of the records of this earthquake indicates that the vibratory ground motions are predominantly body wave motions. Since the site is also a deep soil site that can be assumed to have nearly horizontal soil layers, the conditions should be ideal for one-dimensional SHAKE code analysis. Second, this site and earthquake were used by Valera et al. (1977) to provide the SHAKE and FLUSH code soil/structure interaction analyses. The results of these analyses indicate that the SHAKE deconvolution routine may have important limitations when applied to deep soil sites. Therefore, the site was also selected for comparative analyses to determine if such limitations exist.

The free-field ground motions were recorded only at the ground surface at the

Humboldt Bay site. Since the objective of this investigation is to estimate the variation in intensity of earthquake ground motion with depth below the ground surface, it is evident that comparative analyses cannot establish quantitatively that a model based on a particular code gives the correct or the best estimate of subsurface ground motions. However, it can be demonstrated that for the same set of assumptions, similar models from two different codes do, or do not, give almost comparable responses. In the event similar models do not give comparable responses, the code giving the better response can be identified, keeping in mind that the overall objective is to provide recommended procedures for defining input ground motion for soil/structure interaction analyses. Additional details of the analyses are provided in AA (1979).

1.3.2 EARTHQUAKE CHARACTERISTICS

The epicenter of the Ferndale earthquake of June 7, 1975 was located about 24 km (15 mi) south-southwest of the Humboldt Bay site. The focal depth was estimated also to be about 24 km, which provides a hypocentral distance of about 33 km (21 mi). This earthquake was assigned a Richter magnitude of 5.5 (Smith, 1975). Smith notes that the epicenter lies just north of the surface trace of the Russ Fault. The June 7 earthquake appears to fall within a band of previous seismic activity in this area that dips slightly to the north. He further states that whether or not this indicates a subsurface extension of the Russ fault is not known. The focal mechanism determined from the main shock and several aftershocks indicates strike-slip faulting with nearly horizontal left lateral motion on a plane striking N75°E and dipping S72°E.

1.3.3 SITE CHARACTERISTICS

The Humboldt Bay site is located near the

northern end of the California Coastal Ranges section of the Pacific Coast province. The Klamath Mountains section lies to the north. Bedrock consists of deformed and sheared rock of the Jurassic-Cretaceous age Franciscan assemblage which consists of sandstone, shale, chert, basalt, and schist. The late Cretaceous age Yager formation, consisting primarily of sandstone, shale, and conglomerate, overlies these rocks. Compact mudstone, claystone, and minor sandstone of Tertiary age comprise the near surface rock.

Soil conditions at the site consist of about 25 ft of medium to stiff clay, underlain successively by about 30 ft of medium dense to dense sand, 10 ft of very stiff clay, followed by a deep bed of dense sand containing some clay lenses extending to a depth of about 400 ft (Valera et al., 1977). Soil at this site has been estimated to extend to a depth of at least 800 ft. Valera et al. considered three profiles of the site for their analyses. These profiles differ in only the upper 20 ft. Profile A represents lower bound properties for this depth and Profile B represents upper bound values. Profile C represents an average profile.

The dynamic shear moduli and damping characteristic of the soils were determined by Valera from resonant column and cyclic triaxial tests on undisturbed samples reconsolidated under in-situ confining pressures. It is noted that no field determined seismic shear wave velocities are available for the site. Shear wave velocities used in analyses were therefore computed wave velocities based on laboratory test data. In general, field determined values would be significantly higher.

1.3.4 INSTRUMENT LOCATION

Strong motion instrumentation at the Humboldt Bay power plant consisted of an early Teledyne type MTS-100 seismic recording system

that had been in operation since September 1971 (Valera, 1975). Two sensor locations were in the refueling building of Unit 3 of the power plant and one was in a storage building approximately 300 ft to the southeast of the power plant building. Each sensor is comprised of three FB-103 triaxial force balance accelerometers. The storage building is a single-story concrete block building on a reinforced concrete slab. The storage building sensor is located on the slab at plant grade level. The acceleration records recorded at the storage building sensor (Valera et al., 1977) are used as input for the analyses here.

1.3.5 ANALYSIS OF STRONG MOTION RECORDS

The accelerometers at the Humboldt Bay site were calibrated as a matter of routine maintenance 7 days before the June 7, 1975 earthquake and were again checked after the earthquake. The calibration factors were found to be identical to those obtained prior to the earthquake. The three components of motion recorded were east-west (E-W), north-south (N-S), and vertical. The three components were digitized by using an analog to digital converter which sampled the analog FM signal at a rate of 0.005 sec. Additional details of the digitization can be found in the report by Valera (1975). The E-W and N-S records were converted into transverse and longitudinal components in which the longitudinal component represents a radial component extending from the epicenter or focus to the sensor site. The transverse component is normal to the longitudinal or radial component.

Figure 1-8 provides 20 sec of the recorded motion for each component. It will be noted that strong motion persisted for less than 4 sec of the record (i.e., between 3 and 7 sec). The peak acceleration on the three components of the records given in Figure 1-8 were 0.35 g

FIGURE 1-8. ACCELERATION RECORDS FOR STORAGE BUILDING RECORDER (Valera, 1975)

on the transverse component, 0.26 g on the longitudinal component, and 0.07 g on the vertical component. Comparison of the initial strip charts of the records for the storage building with those recorded in the power plant indicates only one abnormality. The half-amplitude of the maximum vertical motion in the two power

plant sensors varied from 2.6 to 3.2 mm, but the half-amplitude of the vertical component of the storage building record was only 0.85 mm. The peak acceleration on the vertical component of the storage building record is comparable to the noise level of the recorder, and is only one-tenth of the peak acceleration recorded on the transverse component. Since the low amplitude of the vertical component of motion at the storage building site cannot be explained, great confidence has not been placed in the vertical component of motion for this record.

The first arrival of the shear wave can be identified on the records by its amplitude and travel time. In Figure 1-8, the first arrival of the shear wave on the transverse and longitudinal component is about 3.3 sec. The accelerometer does not record at the very beginning of the fault movement, but is started by the arrival of the P-wave. Therefore, the time of travel of the shear wave should be slightly greater than 3.3 sec. From the travel time table suitable for California (Gutenberg, 1951), the time interval between the dilatational and shear wave phases should be about 4 sec for a hypocentral distance of about 35 km. Hence the appearance of the first shear wave arrival at 3.3 sec after the start of recording is reasonable.

The magnitude of the June 7, 1975 Fern-dale earthquake is estimated to have been 5.5 and the fault rupture producing this magnitude could be up to about 8 km. If the fault rupture velocity is assumed to be 3 km/sec, the duration of the emitted S-wave should be about 2.5 sec. This is consistent with the duration of S-wave motion observed on the record.

Surface waves consisting of Rayleigh and Love waves are also frequently produced by the energy released from fault movement (see Sec. 1.1.4). The Rayleigh wave has longitudinal and vertical components of motion and

should appear on these two components of the record. Since the vertical accelerations are small on all site records, the Rayleigh wave contribution must be negligible.

1.3.6 SHAKE CODE ANALYSIS

Previous experience has indicated that more realistic subsurface response motions have been obtained with the one-dimensional SHAKE code when base input analyses are provided than when surface deconvolution analyses are used. Therefore, only the base input SHAKE analysis was performed. Comments on the depth of model, soil properties, and the input motions selected for the analyses are provided here.

The source of the body wave motion for the June 7, 1975 Ferndale earthquake has been estimated to be about 80,000 ft (24 km) below the ground surface, and roughly the same horizontal distance from the Humboldt Bay site. The ray path for the body wave motions recorded at the Humboldt Bay site was therefore more than 100,000 ft in length. While body wave motion may experience high near-surface damping as a result of nonlinear soil response, this type of behavior cannot persist to great depth; otherwise unrealistically high intensities of motion would have to exist at a depth of only a few hundred feet. A model was selected that is nearly 800 ft in depth so that the attenuation characteristics of the motion could be observed to considerable depth. There is no reason why a wave propagation model should not perform to this depth. However, the results indicate that some adjustment must be made to the SHAKE code iterative procedure that relates soil modulus, damping, and strain level if realistic response is to be maintained.

For this SHAKE code analysis, Profile C was selected. This profile was divided into

13 layers. The computed initial shear moduli ranged from 1500 psi at the top to 62,500 psi at the bottom of the profile. The final damping values did not decrease uniformly from top to bottom, but ranged from 13% to 4%.

The 4-sec strong motion portion of the acceleration record for the transverse component (from 3 sec to 7 sec in Fig. 1-8) was used as surface input to the SHAKE deconvolution analyses. This component was recorded at the site and should be an ideal input record for reasons discussed previously. SHAKE code solutions were not provided for the vertical component of motion. Since the real transverse time history is not available at any depth below the ground surface, there were three alternatives for defining the input time history for the base input model. One alternative would have been to use a time history computed by deconvolution from the surface time history. However, this assumes the deconvolution analysis gives the correct time history at depth, which is not believed to be the case. The second alternative would have been to select a time history from a firm soil or rock site, or devise an artificial time history. However, the exact frequency content of the motion for this earthquake at the 775 ft depth is not known and would have to be assumed. It was concluded that a better indication of deconvolution errors would be obtained if the same record (i.e., the surface record) were used for the base input analysis as was used in the deconvolution analysis. This third alternative was the procedure followed.

1.3.7 TWO-DIMENSIONAL FINITE ELEMENT ANALYSES

A two-dimensional finite element analysis for the Humboldt Bay site was performed using the TRI/SAC code. Both the horizontal and vertical components of motion for the 4 sec of strong motion indicated in Figure 1-8 were

input simultaneously to the TRI/SAC models in this analysis.

The geometry and boundary conditions used in the TRI/SAC model for base input analysis are provided in Figure 1-9. The locations of reference lines and nodal points at which response motions were monitored are also indicated. The base input model has two vertical energy-absorbing side boundaries.

The base input model should be able to provide a better simulation of body wave motion since body waves propagate in a nearly vertical direction. By introducing the horizontal and vertical components simultaneously, the true direction of motion should be simulated in a two-dimensional model. This is also an economical procedure since two components of response can be determined from one analysis. The side input model in which the input is

applied along the left vertical boundary of the model (Fig. 1-9) is more appropriate for transmission of surface wave motions since Rayleigh waves have two-dimensional elliptical retrograde motions at and near the ground surface in the longitudinal plane, and Love waves are essentially horizontally propagating shear waves in the transverse plane. Neither of these waves is present to detectable levels in the Humboldt Bay site record for the June 7, 1975 Ferndale earthquake. Therefore, the base input model is the preferred model for this analysis. If both surface and body waves are present, two separate models may be required (side input and base input models) unless it can be demonstrated that one model can satisfactorily handle both wave forms.

The soil properties used in the TRI/SAC analyses are the final iteration values obtained

FIGURE 1-9. TRI/SAC FREE-FIELD FINITE ELEMENT BASE INPUT MODEL FOR HUMBOLDT BAY SITE

in the SHAKE analysis. However, as discussed earlier, TRI/SAC uses Rayleigh damping, and efforts were made to select damping values for TRI/SAC analyses compatible with those used for SHAKE analyses. The damping used in TRI/SAC analyses resulted in a minimum damping of 6% at 3.5 Hz and 10% damping at frequencies of 1 Hz and 10 Hz.

The horizontal and vertical components of the 4 sec of strong motion given in Figure 1-8 were input simultaneously at the input boundary of the TRI/SAC model with no time phasing of motion along the input boundary. An integration time step of 0.02 sec was used for each analysis. It should be noted that definition of subsurface input motions is a modeling problem for both the TRI/SAC and SHAKE codes and probably led to the development of the SHAKE deconvolution technique. There is, however, no comparable procedure for the TRI/SAC code. Time phasing of the input motion across the input boundary for the base input TRI/SAC model is probably not required because the body waves should be advancing nearly vertically across a relatively broad wave front.

1.3.8 COMPARISON OF SHAKE AND TRI/SAC BASE INPUT ANALYSES

A comparison of horizontal response spectra at the ground surface and at the 45 ft and 100 ft depths computed from the SHAKE and TRI/SAC base input analyses is provided here. The major differences in the codes are the manner in which damping is treated, and the degrees of freedom, or modes, in which the two models can transmit energy. A short discussion of the possible effects of these factors precedes the discussion of the results.

It has been previously noted that the SHAKE code has the capability to assign different material damping (equivalent to hysteretic stress/strain behavior) to each soil layer. In

contrast, the TRI/SAC code utilizes modal damping which is expressed as a function of the mass and stiffness matrices. Assuming the damping is known, the SHAKE code handles damping in a much more effective and reliable manner. The example in Section 1.2 has demonstrated the difficulty of obtaining satisfactory responses for a site profile that has both lower damped rock and higher damped soil layers with a model based on modal damping.

SHAKE code models are one dimensional, which in effect restrains the soil profile to shear mode response that is comparable to that which would occur in a vertical column. As a result, most of the energy is transmitted in the fundamental shear mode of the column. The frequency of response of the column is affected by the shear modulus of the soil and the depth of soil column. In contrast, the two-dimensional TRI/SAC model has an infinite number of two-dimensional modes of response. Therefore, the wave energy will be transmitted over a wider range of frequencies. Because of the nature of the input, and the layering assumed in this model, much of the energy in the TRI/SAC model will be transmitted in the fundamental mode of the model, but the fundamental mode of the TRI/SAC model is not necessarily a one-dimensional shear mode response. As a result, the SHAKE code should be more sensitive to a frequency shift in shear mode response resulting from a change in shear modulus.

One additional difference in the two analyses should be noted. A vertical component of motion is included in the TRI/SAC model input so that both P- and S-waves are being transmitted simultaneously similar to true earthquake phenomena. More energy is therefore being introduced into the TRI/SAC model. This could lead to higher near surface response of the TRI/SAC model over a broader band of frequencies than occurs with the SHAKE model.

Discussion of the results of the comparative analyses follows.

Figure 1-10 provides horizontal response spectra for the input, and computed spectra at three different elevations using base input SHAKE and TRI/SAC analyses. A more logical discussion can be provided by comparing the response in progressive steps from the lowest elevation to the ground surface. Starting with the response spectra given in Figure 1-10c for a depth of 100 ft below the ground surface, it will be noted that almost identical response results from the two models for frequencies above 1.5 Hz. This is remarkable and somewhat fortuitous. Both models indicate the same attenuation in response between the spectrum for the input motions at the 775 ft depth and the spectrum at the 100 ft depth in this frequency range.

For frequencies below about 1.5 Hz, the TRI/SAC spectrum at the 100 ft depth indicates greater attenuation in response than the SHAKE spectrum when compared to the spectrum for the base input motion. The perturbation at 1 Hz in the SHAKE solution is a layer reflection effect that should be disregarded. (This is based on a comparison of SHAKE spectra for depths of 45, 72.5, 100, 166.7, and 200 ft, all of which are not included here.) Nevertheless, the SHAKE response below 0.9 Hz at the 100 ft depth is not affected by the perturbation and remains stronger than the TRI/SAC response.

The exact reason why the SHAKE response is stronger at frequencies below 1.5 Hz cannot be stated. One factor could be that damping is very high for the TRI/SAC analysis for modal responses below 1 Hz, considering that at depths below 200 ft the SHAKE code solution has only 4% material damping, but the TRI/SAC solution has damping greater than 10%. However, TRI/SAC solutions ordinarily show little effects of damping on response at frequencies

(a) Ground surface spectra - Reference line B

(b) Spectra at 45-ft depth - Reference line B

(c) Spectra at 100-ft depth - Reference line B

NOTES:

SPECTRAL DAMPING = 5% OF CRITICAL

SEE FIGURE 1-9 FOR LOCATION OF REFERENCE LINES

FIGURE 1-10. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE AND TRI/SAC BASE INPUT MODELS - HUMBOLDT BAY SITE

below about 1 or 2 Hz. A more likely pair of factors is the difference in modal response of the one-dimensional and two-dimensional models in this frequency range, and the narrow band frequency of the input motion. Because of the peaked input spectrum, the input motion is concentrated in a narrow band which includes the 2-Hz frequency. This has resulted from the fact that the soil profile has already filtered out most of the other frequencies from the Storage Building record which was used as input. Also, the fault movement did not produce long period motion, such as would result if surface waves had been present. As a result, the one-dimensional model, which is more sensitive to frequency shifts resulting from changes in shear moduli than the TRI/SAC model, is responding with maximum amplification at the 100 ft depth at a frequency below 1 Hz. However, this is not too evident since the peak response is still maintained at 2 Hz because of the narrow band of the input motion which is in the 2-Hz range.

Figure 1-10b provides a comparison of horizontal response spectra at the 45 ft depth. For frequencies above 5 Hz, the TRI/SAC code demonstrates greater attenuation in response than the SHAKE code when compared to the input spectrum. This is probably due to the difference in the damping provisions in the two codes. The general shapes of the response spectra computed from the two models are quite comparable, but the frequency of maximum response has shifted from 2 to 2.5 Hz for the TRI/SAC spectrum, permitting the TRI/SAC spectrum to closely match a bulge in the input spectrum in the 3 to 4 Hz frequency range. The SHAKE spectrum maintains the input spectrum peak at 2 Hz, but attenuates below the response indicated at the 100 ft depth. Amplification of the SHAKE spectrum above the input motion is indicated only at frequencies below 0.7 Hz. The TRI/SAC spectrum indicates

strong attenuation between 0.9 and 2 Hz at the 45 ft depth compared to the spectrum at the 100 ft depth, but it changes little in this distance for frequencies below 0.9 Hz.

Comparison of the horizontal response spectrum given in Figure 1-10b for the ground surface level for the two codes shows important differences, some of which are not too conspicuous. First, both codes show amplification in response between the 45 ft depth and the ground surface at all frequencies. Above about 1 Hz, the TRI/SAC code shows stronger amplification than the SHAKE code. This difference is attributed almost entirely to differences in damping, although additional energy is input to the TRI/SAC code by virtue of the presence of a vertical component of motion. Below a frequency of about 1 Hz, both codes have provided about the same amplification between the 45 ft depth and the surface, but this can be detected only by closely comparing spectra in both Figures 1-10a and 1-10b.

The peak response for the surface TRI/SAC spectrum occurs at 2 Hz, similar to the peak in the input spectrum. The peak in the SHAKE response spectrum at the surface also occurs at 2 Hz; however, a secondary peak occurs at 1 Hz of almost equal (velocity) response, which results in a broad rounded spectral peak. Comparison of the SHAKE and TRI/SAC spectra at both the surface and at the 45 ft depth reveals that maximum amplification of the SHAKE spectrum in this depth interval occurs at about 1 Hz, while the maximum amplification of the TRI/SAC spectrum occurs at about 1.5 Hz. However, the shape of the TRI/SAC surface spectrum is a much better simulation of the input spectrum which is the spectrum for the site surface record. It has, however, been amplified by a factor of slightly less than two.

The comparison of SHAKE and TRI/SAC analyses leads to the conclusion that while

neither code provides a perfect solution to body wave propagation problems for deep soil sites, models from either code can be used to provide good estimates of the response. The major problems in such analyses are the uncertainties in the input motions and uncertainties in the soil properties, assuming the analyst is knowledgeable of the limitations of each code. The advantages and disadvantages of the two codes are the following:

- a. The SHAKE code has the capability to assign material damping (equivalent to hysteretic stress/strain behavior) to each soil layer. In contrast, the TRI/SAC code utilizes modal damping which is expressed as a function of the mass and stiffness matrices. Assuming that the damping is known, the SHAKE code handles damping in a much more effective and reliable manner.
- b. SHAKE code models are one dimensional, which restrains the soil profile to shear mode response that is comparable to that which would occur in a vertical column. As a result, most of the energy is transmitted in the fundamental shear mode of the column. In contrast, the two-dimensional TRI/SAC model has an infinite number of two-dimensional modes of response. The TRI/SAC code can, therefore, treat the true directional characteristics of body (or surface) wave motion by inputting the horizontal and vertical components of motion simultaneously, and can treat inclined soil and rock geometries.
- c. The SHAKE code has an iterative routine which provides an adjustment in damping and shear modulus so that these values are consistent with the strain level experienced in each soil

layer. However, it is doubtful that this routine can be reliably applied to soil profiles more than about 50 to 75 ft in depth. Therefore, properties for shear moduli and damping should be specified for models of deep soil sites and solutions obtained with no change in soil properties.

- d. Although the TRI/SAC solution appears to provide slightly better results, the SHAKE code is more economical to use. Therefore, base input models based on either code can be recommended for deep soil sites experiencing nearly vertically incident body wave motion.

The following additional conclusions are based on the studies reported here.

- a. The SHAKE code deconvolution routine has serious limitations when used for deep soil sites in that spurious response may be obtained in frequencies of importance to nuclear power plant design. While the spurious response can be reduced by filtering the input record, the response is strongly affected by the filter frequency which has to be selected arbitrarily by the analyst. SHAKE base input rather than deconvolution solutions is recommended for defining subsurface ground motions for deep soil sites.
- b. The best procedure for defining input motion for either a SHAKE or TRI/SAC base input model is by trial solution, assuming a ground surface design response spectrum must be matched by the computed response. The analyst should not expect to match a Regulatory Guide 1.60 spectrum for all soil sites.

- c. Assuming that the input ground motions have been correctly defined, damping is the next most important analysis parameter since it has significant effect on the strength of the response. The shear modulus of the soil is a less important parameter since errors in estimating the shear moduli will in general not change the maximum level of response but will merely shift its frequency. This can be compensated for by broadening the frequency band of amplified response in accordance with NRC practice.

1.4 COMPARATIVE ANALYSES BASED ON ORION SITE

1.4.1 INTRODUCTION

The results of one-dimensional SHAKE and two-dimensional finite element TRI/SAC analyses based on the soil profile estimated to be present at the strong motion recording station at 8244 Orion Avenue, Los Angeles, California, are provided here. This recording station is located near the center of the San Fernando Valley, and recorded a peak horizontal ground acceleration of 0.26 g at the ground surface during the February 9, 1971 San Fernando earthquake. The vibratory ground motions recorded at this station during this earthquake have been used as input motions for the comparative analyses. The Orion site and records were selected for the comparative studies because strong body and surface waves were both recorded.

The analyses provided for the Humboldt Bay site have demonstrated that body wave motions propagating in a nearly vertical direction can be represented by SHAKE and TRI/SAC base input models. In contrast, logic indicates that surface wave motions can be

physically represented only by a two-dimensional model, and that the surface wave input motions should be made at a vertical side boundary. If both body and surface wave motions are to be considered in an analysis, and if a two-dimensional side input finite element model is required to adequately represent the surface wave motion, it would be convenient and economical to use the two-dimensional side input model to treat the entire record. This assumes that body wave motions can be properly represented by this model. Furthermore, it would be desirable to know whether comparable response to surface wave motion can be obtained from a SHAKE one-dimensional analysis that assumes all motions to be vertically propagating body waves.

Analysis of the Orion site strong motion records of February 9, 1971 reveals that strong body wave motion dominated the early part of the records, and strong surface wave motion dominated the late part. To provide information on the modeling problems described in the above paragraph, it was considered advisable to divide the Orion records timewise into two segments: a body wave dominant segment, and a surface wave dominant segment. Separate analyses were then made with each segment of the records using both SHAKE and TRI/SAC models.

1.4.2 EARTHQUAKE CHARACTERISTICS

The San Fernando earthquake of February 9, 1971 has been assigned a local magnitude of 6.4 by Allen et al. (1973). Hanks (1974) has indicated that the fault was initiated beneath the San Gabriel mountains at a hypocentral depth of about 13 km. The thrust fault is estimated to have had a strike of N67°W and a dip of 52° to the northeast at this depth. Faulting proceeded southward and upward along the main thrust plane. Near the earth's surface, the dip decreased to about 20° to 25°

1.4.3 SITE CHARACTERISTICS

The Orion site is located approximately midway from north to south across the San Fernando Valley, where the alluvial soils have been estimated to be about 800 to 1000 ft in depth. The alluvium overlays the Modelo siltstone (upper Miocene) which extends to a depth of about 8000 ft. The Topanga formation (Middle Miocene) then extends an additional 6000 ft to basement rock.

There is little reliable information available on the soil properties at the Orion site. Estimated properties were used in SHAKE code analyses reported in NUREG-75/072 (SW/AA, 1975); however, these values were not based on exploratory borings or laboratory tests. In seismic refraction studies, Duke et al. (1971) estimated shear moduli that are significantly higher than given in NUREG-75/072. All SHAKE and TRI/SAC comparative analyses reported herein were arbitrarily based on the properties given by Duke et al. (1971).

1.4.4 INSTRUMENT LOCATION

The instruments recording the earthquake ground motion at 8244 Orion Avenue were Earth Science AR-240 strong motion accelerographs. The records used in this investigation were taken from the recorder mounted on the ground floor slab of the Holiday Inn, near the southeast corner of the building at the Orion Avenue address. This site is approximately 22 km (13 mi) due south of the estimated epicenter of the earthquake (16 mi from the hypocenter). The Holiday Inn is a seven-story reinforced concrete structure, approximately 62 x 160 ft in plan dimensions (Blume, 1973). The ground floor slab is placed on grade over 2 ft of compacted fill. The building foundation system consists of 38 in. deep pile caps, supported by two to four 24-in.-dia. reinforced concrete CIP piles 40 ft in length. The pile groups are

centered under the main building columns. All pile caps are tied laterally by a grid of tie and foundation beams.

1.4.5 ANALYSIS OF STRONG MOTION RECORDS

The Orion site and records were selected for this investigation to provide an example site at which strong surface wave motions have been experienced. The studies by Hanks (1975) and Drake and Mal (1972) provided an excellent basis for this selection. However, an independent analysis of the strong motion records for the Castaic, Griffith Park, and Orion Avenue recording stations was given in detail in AA (1979).

The Orion Street and Griffith Park stations lie within the region indicated by Hanks (1975) to have experienced strong Rayleigh wave motion. Castaic, in contrast, lies to the northwest of the fault in a region that did not experience significant surface waves. The acceleration, velocity, and displacement time histories for the Orion site resulting from the 1971 San Fernando earthquake are given in Figure 1-11. The arrival time of the shear (body) wave at the Orion site is indicated in the figure, and has been estimated to be at about 2.8 sec after the start of recording.

Particle motion trajectories in the vertical radial plane were constructed from the radial and vertical components of motion for the three sites. Particle motion trajectories for the Orion site indicate strong body wave motions between 0 and 10 sec on the record, with strong surface waves dominating the record after 10 sec.

Since the first 2.8 sec of the Orion record do not indicate strong motion (see Fig. 1-11), it was eliminated from the record to reduce the computer time required for each analysis. Also, after 25 sec, the wave intensity has decayed significantly; therefore, the portion of

FIGURE 1-11. THREE COMPONENTS OF GROUND MOTION RECORDED AT ORION SITE FROM 9 FEBRUARY 1971 EARTHQUAKE (from Hanks, 1975)

the record after 25 sec was eliminated for the same reason. The remaining 22 sec of the horizontal (radial) and vertical components of the Orion records are shown in Figure 1-12. These records were then divided into two segments and used to provide separate SHAKE and TRI/SAC analyses. The first 7 sec of the records (from 2.8 to 10.0 sec portion of original

records) constitute the first segment and represent the time during which the body wave motion is dominant. The second segment is represented by the 10 to 25 sec portion of the original records, and represents the time during which the surface wave motion is dominant. It should be again noted that surface waves may occur during the first segment of the records, and body waves also occur during the second segment, but each is much less dominant in these respective segments of the records. If a frequency bound could be established above

FIGURE 1-12. PORTIONS OF ORION SITE ACCELERATION RECORDS USED IN ANALYSES

which surface wave motions do not occur, and below which body waves do not occur, the acceleration records given in Figure 1-12 could have been frequency filtered to provide independent body and surface wave records. However, at the present time, no frequency boundary of this type has been established. Therefore, the above expedient of dividing the records at a specified time interval was used to provide separate records that in one case essentially represent body wave motions, and in the second case essentially represent (Rayleigh) surface wave motions.

1.4.6 SHAKE CODE ANALYSES

It was noted that dynamic soil properties at the Orion site remain uncertain. Therefore, comparative SHAKE deconvolution analyses based on the Orion records were made using the soil properties estimated by NUREG-75/072 and Duke et al. (1971). The strain/modulus and strain/damping relationships incorporated into the SHAKE code were used for these analyses. These analyses indicated that better results were obtained with the soil properties estimated by Duke et al. (1971). Therefore, all the analyses reported here were performed using these properties. The layer damping and moduli for the first and final iterations for this case are given in Table 1-3.

1.4.7 TWO-DIMENSIONAL FINITE ELEMENT ANALYSES

Four separate two-dimensional finite element TRI/SAC analyses of the Orion site were performed using the models shown in Figure 1-13. Two of the analyses used the body wave dominant portion of the Orion site records (first 7 sec) given in Figure 1-12 as input. The third analysis was based on the surface wave dominant portion (last 15 sec) of these records, and the fourth considered the total record. One body wave analysis utilized a

TABLE 1-3. DYNAMIC SOIL PROPERTIES FOR ORION SITE DERIVED FROM SHAKE CODE ANALYSIS USING DUKE ET AL. (1971) PROPERTIES AS INPUT

Depth, ft	Material	Shear Modulus, psi		Damping, Percent	
		Initial	Final Iteration	Initial	Final Iteration
0-10	Sand	9,200	7,000	10	5
10-20	Sand	9,200	10,000	10	7
20-30	Sand	9,200	11,300	10	8
30-40	Sand	9,200	12,200	10	9
40-70	Sand	52,700	12,300	9	10
70-100	Sand	52,700	12,700	9	11
100-130	Sand	108,300	18,700	8	10
130-160	Sand	108,300	20,300	8	10
160-200	Sand	108,300	22,100	8	10
200-250	Sand	144,000	25,600	7	9

model in which the two components of input motion (horizontal and vertical) were input at the base of the model since body waves characteristically propagate in a nearly vertical direction (see Fig. 1-13a). The second body wave analysis utilized a model in which the same two body wave components were input at the left vertical boundary (see Fig. 1-13b). The objective in this case was to determine if comparable response to body wave motion could be obtained from the two models. The results of these two analyses are presented and compared first. The third and fourth TRI/SAC analyses, which used the surface wave dominant portion of the records and the total record, respectively, as input, used the side input model given in Figure 1-13b.

The geometry and boundary conditions used for the two models are also given in Figure 1-13. The locations of reference lines and nodal points at which response motions were monitored are indicated. The base input model has two vertical energy absorbing side boundaries, and the side input model is the same except for the change in boundary conditions. Input to this model is through the left vertical boundary. Since the side input analysis is

(a) Base input model

(b) Side input model

FIGURE 1-13. TRI/SAC FREE-FIELD FINITE ELEMENT MODELS FOR ORION SITE

known to associate with a large energy loss with an energy absorbing boundary at the base, a free surface was provided at the base and an energy absorbing boundary was provided at the right vertical boundary only.

The initial shear moduli given in Table 1-3 were used in all the TRI/SAC analyses. Modal damping assumed for all TRI/SAC analyses increases from about 6% at 3 Hz to about 10% at 2 and at 10 Hz. This damping level was selected to be consistent with the damping assumed for the upper 100 ft of the soil profile

in Table 1-3. The damping is too high, however, for response predictions at lower levels.

1.4.8 COMPARISONS OF SHAKE AND TRI/SAC ANALYSES

Four comparisons between the SHAKE and TRI/SAC analyses are given here. Although the same input was used for all analyses, the strength of response is not comparable in most cases. The comparisons must, therefore, be based on general shape characteristics and not on strength of response. The following comparisons are provided:

- Body Wave Input

SHAKE vs. TRI/SAC base input models

SHAKE base input vs. TRI/SAC side input models

- Surface Wave Input

SHAKE deconvolution vs. TRI/SAC side input models

- Total Input Record

SHAKE base input and deconvolution vs. TRI/SAC side input models

1.4.8.1 Base Input Analyses - Body Wave Input

The best results for body wave motions should result when base input models are used for both the SHAKE and TRI/SAC analyses. These two models gave good results for the Humboldt Bay site comparison. Figure 1-14 provides the comparison for the Orion site. At the 100 ft depth, maximum amplification occurs at about 1 Hz for the SHAKE model but occurs at about 1.5 to 2 Hz for the TRI/SAC model. The SHAKE model maintains a peak at about 2.5 Hz at all depths, but the TRI/SAC model indicates that this is not present at the 100 ft depth but develops in the upper 40 ft of the model. Some detail is lost in the TRI/SAC response spectra above 5 Hz at the 100 ft depth but it reappears at upper elevations.

At the 40 ft depth and at the surface, the TRI/SAC spectra experience the strongest amplification at about 2 to 3 Hz, while the SHAKE spectra have the strongest amplification at about 1 Hz. The input spectrum is also amplified at these elevations down to a frequency of 0.5 Hz by the SHAKE code model. This includes frequencies that are not characteristic of body wave motion for this site. Both codes give acceptable results. However, the general shape characteristic of the input spectrum is matched at the ground surface better

NOTES:

SPECTRAL DAMPING = 5% OF CRITICAL

BASED ON INITIAL SOIL PROPERTIES GIVEN IN TABLE 1-3

TRI/SAC SOLUTIONS ARE FOR REFERENCE LINE B

(a) Surface spectra

(b) Spectra at 40-ft depth

(c) Spectra at 100-ft depth

FIGURE 1-14. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE AND TRI/SAC BODY WAVE BASE INPUT MODELS - ORION SITE

by the TRI/SAC solution although the difference is slight.

1.4.8.2 SHAKE Base Input vs. TRI/SAC Side Input Analyses - Body Wave Input

The horizontal response spectra computed for the surface and at depths of 40 and 100 ft using the SHAKE base input model and the TRI/SAC side input model are compared in Figure 1-15. Since the TRI/SAC side input model gives comparable response to the TRI/SAC base input model, the comments given above in Section 1.4.8.1 are applicable to the SHAKE base input and TRI/SAC side input comparison and will not be repeated.

1.4.8.3 SHAKE Surface Deconvolution vs. TRI/SAC Side Input Analyses - Surface Wave Input

A comparison between the SHAKE deconvolution model and the TRI/SAC side input model, both based on surface wave input, is given in Figure 1-16. The SHAKE deconvolution solution was used in this comparison since the strength of surface response for the two solutions is more nearly equal than results with the base input analysis. The SHAKE code gives perfect agreement at the surface since the surface spectrum is used as input at this elevation. There is no reason, however, why the SHAKE code should give agreement in the 1 to 3 Hz region at lower elevations since the model converts the motion to a vertically propagating shear wave which should attenuate more rapidly than the surface wave. The 1 to 3 Hz frequency range represents the frequencies experiencing greatest site amplification in the SHAKE body wave analyses. Examination of Figure 1-16c shows that strong attenuation is experienced in this frequency range for the SHAKE model but not for the TRI/SAC model. It is concluded that the SHAKE code should not be used to simulate surface wave motion.

1.4.8.4 SHAKE vs. TRI/SAC Analyses - Total Record Input

It has been pointed out above that the

results of the SHAKE surface deconvolution analysis were compared with the results from the TRI/SAC side input analysis for surface wave input because this comparison provides spectra at the surface that are more nearly of equal strength than would occur if the results of the SHAKE base input analysis had been used. Primarily for the same reason, the results of the SHAKE base input analysis were compared with the results of the TRI/SAC side input analysis for body wave motion in Section 1.4.8.2.* A comparison between analyses based on the two codes is, therefore, difficult when the total record (i.e., the body wave and surface wave segments of the record) is used as input. For this reason, comparisons are provided here between results from the TRI/SAC side input analysis and results from both SHAKE surface deconvolution and base input analyses. The former provides spectra at the ground surface that are more nearly of equal strength in the surface wave frequency domain, while the latter provides spectra that are more nearly of equal strength at the ground surface in the body wave frequency domain. A brief discussion of the results follows.

A comparison of spectra computed from the SHAKE surface deconvolution and TRI/SAC side input analyses is given in Figure 1-17. Since the acceleration record is assumed to be input at the surface in the SHAKE deconvolution analysis, no further site amplification effects can occur at this level (see Fig. 1-17a). The same input is used for the TRI/SAC analysis, but it is applied at the side of the model and some amplification of response will be observed to occur at the surface for this analysis in Figure 1-17a at frequencies that are characteristic

*An additional reason for using the results of the SHAKE base input analysis in this case was the fact that the SHAKE base input analysis is believed to be more reliable than the deconvolution analysis for body wave motion.

NOTES:

SPECTRAL DAMPING = 5% OF CRITICAL

BASED ON INITIAL SOIL PROPERTIES GIVEN IN TABLE 1-3

TRI/SAC SOLUTIONS ARE FOR REFERENCE LINE B

(a) Surface spectra

(b) Spectra at 40-ft depth

(c) Spectra at 100-ft depth

FIGURE 1-15. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE BASE INPUT AND TRI/SAC SIDE INPUT MODELS FOR BODY WAVE MOTION — ORION SITE

NOTES:

SPECTRAL DAMPING = 5% OF CRITICAL

BASED ON INITIAL SOIL PROPERTIES GIVEN IN TABLE 1-3

TRI/SAC SOLUTIONS ARE FOR REFERENCE LINE B

(a) Surface spectra

(b) Spectra at 40-ft depth

(c) Spectra at 100-ft depth

FIGURE 1-16. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE DECONVOLUTION AND TRI/SAC SURFACE WAVE SIDE INPUT ANALYSIS — ORION SITE

NOTES:

SPECTRAL DAMPING = 5% OF CRITICAL
TOTAL RECORD USED AS INPUT

(a) Surface spectra

(b) Spectra at 40-ft depth

(c) Spectra at 100-ft depth

FIGURE 1-17. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE DECONVOLUTION AND TRI/SAC SIDE INPUT MODELS BASED ON TOTAL RECORD — ORION SITE

of body wave motion (i.e., between 2 and 6 Hz). Very little surface amplification occurs for the TRI/SAC analysis below 2 Hz which is the frequency range characteristic of surface wave motion. (Slight attenuation occurs between 0.25 and 0.45 Hz and slight amplification occurs between 0.7 and 2 Hz, but these differences are slight.) The major difference in response in the two analyses at the surface between frequencies of 2 and 6 Hz must be considered when comparing the response at lower elevations. In other words, the comparisons with response at lower elevations must be made on a relative basis.

In Figure 1-17b, the results from the two analyses are quite comparable for frequencies below 2 Hz. However, between 3 to 6 Hz a large depression in the spectrum is indicated from the SHAKE code analysis, but a relatively uniform response acceleration is indicated for the TRI/SAC spectrum in this frequency range. Both spectra attenuate between the surface and the 40 ft depth for frequencies between 2 and 3 Hz but the greatest attenuation occurs for the SHAKE analysis. At the 100 ft depth, the SHAKE solution has completely attenuated the spectral peak that was present in the input between 1 and 2 Hz, and has developed a valley in the spectrum at about 2 Hz (Fig. 1-17c). It is evident that shear wave representation of the input motion by the SHAKE code has forced stronger attenuation in response with depth between 1 and 3 Hz than should occur, assuming that the spectral peak at 1.5 Hz is due to surface wave motion.

A comparison of response spectra at different elevations computed from SHAKE base input and TRI/SAC side input analyses is given in Figure 1-18. It is of interest to note in Figure 1-18a that the spectrum of the input motion has been amplified at the ground surface in the SHAKE analysis throughout the entire

FIGURE 1-18. COMPARISON OF HORIZONTAL RESPONSE SPECTRA FROM SHAKE BASE INPUT AND TRI/SAC SIDE INPUT MODELS BASED ON TOTAL RECORD — ORION SITE

frequency range between 0.3 and 7 Hz. Maximum amplification occurs between frequencies of 1 and 3 Hz. The TRI/SAC surface spectrum has been described above in the discussion of Figure 1-17a. The surface spectrum is amplified primarily in the 2 to 4 Hz range (body wave frequency range) with little amplification occurring in the lower frequencies (surface wave frequency range).

In Figures 1-18b and 1-18c, the spectra from the SHAKE base input solution attenuate with depth at all frequencies above about 0.6 Hz with the greatest attenuation again occurring at about 2 Hz. The valley frequency in the spectrum at 2 Hz corresponds to the same valley frequency given in Figure 1-17c. The peak in the input spectrum has not been completely attenuated at 1.5 Hz at a depth of 100 ft in this analysis (as occurred in Fig. 1-17), but high amplification occurred at this frequency at the surface. Therefore, attenuation of response with depth is rapid between 1 and 3 Hz in the SHAKE base input analysis. In contrast, the TRI/SAC solution indicates response amplification at the surface and at a depth of 40 ft, but the response of the input motion at the 100 ft depth is maintained in this frequency range. At lower frequencies the TRI/SAC analysis indicates only slight attenuation down to a depth of 100 ft which should be characteristic of surface wave motions for this site.

Based on the above comparisons, it is concluded that the best overall response is obtained with the two-dimensional TRI/SAC finite element model when both body and surface wave motions are present in the input and are to be simulated by the model response.

CHAPTER 2

REVIEW OF STATE-OF-THE-ART PROCEDURES

State-of-the-art procedures for seismic soil/structure interaction analysis of LMFBR facilities include: (1) continuum techniques for analyzing a rigid footing located on or in an elastic continuum; (2) the extension of these techniques to analyzing the dynamic response of a structure supported on a rigid-footing/elastic-continuum foundation system; (3) finite element techniques for dynamic soil/structure interaction analyses; and (4) alternative procedures for analyzing such facilities. These state-of-the-art procedures are reviewed here; additional details are provided by AA (1980).

2.1 TECHNIQUES FOR ANALYSIS OF RIGID FOOTINGS ON OR IN AN ELASTIC CONTINUUM

2.1.1 GENERAL BACKGROUND

The first analyses of dynamic soil/structure interaction effects considered the response of a rigid circular disk that was bonded to the surface of an elastic half-space continuum and subjected to harmonic internal forces (e.g., from vibrating machinery). Spring-dashpot analogs to these theoretical results were subsequently developed and provided a simple and widely used means for applying the theory in the analysis of soil/structure interaction effects for actual structures such as nuclear plants. However, the theory and its spring-dashpot analog had the disadvantage of being directly applicable only to the idealized rigid disk and elastic half-space configuration – a configuration not representative of most actual foundation-soil systems. Therefore, in recent years, considerable research has been directed toward extending and generalizing the basic theory. As a result, the class of solutions for analyzing rigid foundations and continuum soil models can now

consider horizontally layered sites, various base pressure distributions, viscoelastic soil materials, arbitrary foundation shapes, embedded foundations, excitations from arbitrarily incident seismic waves, and through-soil coupling of the response of adjacent foundations.

2.1.2 THEORY OF A RIGID DISK ON AN ELASTIC HALF-SPACE

Two classes of solution techniques have been developed for the response of a rigid disk on the surface of an elastic half-space. One is based on an assumed contact stress distribution at the disk/half-space interface, and the other is based on solutions to the mixed boundary value problem, which involves displacements prescribed along the disk/half-space interface and zero contact stresses specified over the remainder of the ground surface. This latter technique involves certain simplifying assumptions regarding the contact condition at the disk/half-space interface.

2.1.3 ANALOG SOLUTIONS TO THE ELASTIC HALF-SPACE THEORY

Practical application of the above elastic half-space theory has often involved replacement of the half-space continuum by discrete springs and viscous dashpots. In this, the springs represent the stiffness characteristics of the continuum while the dashpots represent radiation damping effects. Although the spring and dashpot constants are actually frequency dependent, they are often assumed to be independent of frequency in many practical applications.

2.1.4 EXTENSIONS OF BASIC THEORY

Although the elastic half-space theory is

mathematically convenient, it is not fully representative of actual siting conditions for nuclear plants or other major structures. For example, the soil profile at a nuclear plant site will not be an elastic, homogeneous, isotropic half-space; rather, it may consist of layers of soil materials that extend for a finite distance to bedrock and exhibit nonlinear stress-strain behavior and hysteretic energy losses. Also, the structure foundation will not be sited on the ground surface; instead it will invariably be embedded within the soil materials, and the degree of embedment may be significant for nuclear plants. Finally, although nuclear plant foundations are often quite stiff, the contact stress distributions at the foundation-soil interface can differ from those corresponding to a rigid foundation; this also occurs on rock or soil when inelastic action limits the extremely large stresses near the edge of the foundation, thereby increasing the stresses near the center of the foundation.

In recognition of these factors, the basic elastic half-space theory has been extended to more nearly represent actual siting conditions. These extensions involve consideration of layered soil profiles, variations in the contact stress distribution at the soil/structure interface, stiffness and material damping characteristics of the soil medium, embedment of the structure foundation beneath the ground surface, arbitrary foundation shapes, spatially varying seismic waves, and through-soil coupling effects.

2.2 RESPONSE OF DEFORMABLE STRUCTURES AND CONTINUUM SOIL MODEL

2.2.1 GENERAL DISCUSSION

For deformable structures and a continuum soil model, the soil/structure interaction analysis techniques use the rigid-footing/elastic-continuum results summarized above, in

conjunction with a model of a deformable elastic structure. In this, substructuring techniques are used in which dynamic equilibrium and displacement compatibility conditions at the interface of the structure and footing/continuum systems are accounted for, in order to merge these two systems for the subsequent dynamic analysis.

2.2.2 IMPEDANCE MODELS

The earliest results of this type utilized a very simple single-mass structure model with a footing/continuum system comprised of a rigid circular disk on an elastic half-space. These early results consisted only of computing frequency-dependent impedance characteristics of this simple soil/structure system model as a function of the various model parameters; i.e., no actual seismic analyses were carried out using these models.

These simple models, called "impedance models," were subsequently extended to accommodate linear hysteretic behavior of the soil medium and the structure. Some models of this type have also included effects of foundation embedment, soil layering, and through-soil coupling of adjacent structures.

Application of this class of impedance models to seismic analysis of nuclear plants during the mid-to-late 1970's typically used multimass cantilever beam type models of the nuclear plant structure. These simple structure models were used together with spring-dashpot analogs to elastic half-spaces, in order to represent the soil medium underlying the structure foundation mat (which typically was assumed rigid and circular or rectangular in shape). Prescribed input motions were applied to the springs and dashpots as response spectra or time histories, and a normal mode technique was typically used to carry out the subsequent seismic analysis.

The formulation of the stiffness and mass matrices for the above dynamic system is straightforward; however, the estimation of equivalent modal damping ratios for the combined soil/structure system involved use of approximate procedures such as the weighted energy method, weighting according to modal response characteristics, the ratioing of work done by damping forces to the maximum potential energy, and the weighted sum approach.

2.2.3 SEISMIC ANALYSIS MODELS

The most recent models of this type compute not only the impedance characteristics of the structure/soil-continuum model, but also the three-dimensional dynamic response of the model to a specified seismic excitation. This class of models has the following significant advantages over the simpler impedance models: (1) a finite element representation of the structure can be used, permitting treatment of any number of arbitrarily configured elastic structures; (2) the footings can be rigid or deformable, and can have any arbitrary shape; (3) the soil medium can be comprised of horizontal, linear viscoelastic layers; and (4) the seismic excitations can be induced by any combination of arbitrarily incident body waves or surface waves.

2.3 RESPONSE OF SOIL/STRUCTURE SYSTEMS USING FINITE ELEMENT METHODS

2.3.1 GENERAL DISCUSSION

Finite element techniques have become widely used for evaluating soil/structure interaction effects. This approach represents the soil/structure system as a network of discrete structural and continuum elements interconnected at their node points. The assumed strain distributions within each element are such that compatibility of strains is usually, but not always, maintained along interfaces

common to adjacent elements. Also, this analysis technique satisfies time-dependent force equilibrium requirements at each node point, as well as the stress/strain characteristics specified for each element. The theoretical basis for the finite element method is described in numerous textbooks (see AA, 1976b).

Finite element techniques offer several attractive features for performing soil/structure interaction analyses. Perhaps their more important features are their ability to represent nearly arbitrary soil layering characteristics and material properties in the analysis. Furthermore, structure embedment effects can also be easily included, as well as through-soil coupling of the response of adjacent structures. However, to reduce the substantial computer costs that might otherwise be involved, application of finite element methods to nuclear plants typically involves certain simplifications in the modeling of the actual three-dimensional structure and nonlinear soil medium. These simplifications often involve use of equivalent two-dimensional, rather than three-dimensional, models of soil/structure systems; approximate but relatively inexpensive equivalent linear soil material models to simulate strain-dependent soil properties; and approximate energy-absorbing boundary conditions to represent radiation damping effects for certain idealized site conditions.

2.3.2 GENERAL APPROACHES

There are three basic approaches commonly used to apply finite element techniques to soil/structure interaction problems: the control motion method, the soil island approach, and substructuring techniques.

2.3.2.1 Control Motion Method

Currently the most commonly used approach for applying finite element methods to

analyze soil/structure interaction analysis techniques is through the use of control motions. In this, the control motion is typically specified at some point in the free field, and corresponds to the seismic input criteria developed for the site. Once the control motion is established, it is necessary to determine the motions that would have to develop in an underlying rock-like formation in order to produce the specified motions at the control point. Ordinarily this is carried out by deconvolution techniques based on one-dimensional wave theory (e.g., SHAKE code). Then, the same base-rock motion is used as excitation for a mathematical model of the soil/structure system, leading to an evaluation of the motions and/or stresses at any selected points within the system. In this, the soil/structure system analysis is checked by comparing the computed motions at the control point location, with the control motions that were originally specified.

2.3.2.2 Soil Island Approach

The soil island approach for analyzing soil/structure interaction effects has been widely used in blast-induced ground shock as well as earthquake problems. It involves a three step procedure. The first step designates a fictitious boundary to the soil medium that surrounds the structure. The second step involves computation of free-field earthquake motions at this boundary; this can be based on vertically propagating shear waves, or more complete descriptions incorporating surface waves and/or nonvertically incident body waves. Once these motions are determined, the volume of soil and the structure enclosed by the fictitious boundaries (referred to as the soil island) are analyzed in detail using the computed free-field ground motions as the seismic excitations.

In this process, the boundaries of the soil island are chosen far enough from the location of the structure that, when it is included, it

causes only a slight perturbation at the boundaries. The efficiency and accuracy of the soil island approach depends on choosing an island which is small enough to be manageable and large enough so that boundary effects are negligible. Studies have shown that soil islands extending 2 to 3 characteristic dimensions beyond the structure are adequate.

2.3.2.3 Substructuring Techniques

The substructuring techniques used with a finite element model of an elastic soil/structure system are conceptually identical to those employed in conjunction with continuum soil models. The key element of these techniques is the development of the dynamic impedance matrix of the soil medium, in accordance with various existing procedures summarized in AA (1980).

2.3.3 LINEAR ELASTIC FINITE ELEMENT ANALYSIS TECHNIQUES

Because of their simplicity, the first finite element codes to be developed were linear elastic in nature; in fact, many linear elastic finite element codes are still widely used in today's practice. These codes developed from the simplest two-dimensional planar codes to the more general three-dimensional codes that are widely used now.

There are three types of linear elastic finite element analysis techniques. These include procedures that use two-dimensional planar soil/structure system models, methods that treat axisymmetric soil/structure systems subjected to nonaxisymmetric loads, and techniques that incorporate separate substructures for the soil and structural systems.

2.3.4 EQUIVALENT LINEAR FINITE ELEMENT ANALYSIS TECHNIQUES

As of the late 1970's, the most common technique used to analyze seismic soil/structure

interaction effects for nuclear plants was the FLUSH code (Lysmer et al., 1975). This code utilizes the control motion method, whereby established control motions at some point in the soil grid are deconvolved to define corresponding motions in the underlying rock medium; these rock motions, in turn, serve as input to a two-dimensional soil/structure interaction analysis. The deconvolution process, as well as the soil/structure interaction analysis, presumes that the motions are caused only by vertically propagating shear waves or P-waves.

In addition to its incorporation of the control motion method, another feature of the FLUSH code that sets it off from other finite element codes is the manner in which the soil properties are defined. The FLUSH code utilizes an equivalent-linear approach whereby an iterative procedure is used to define a secant shear modulus and damping ratio for each soil element. These are used as equivalent linear soil properties throughout each response history computation, and are adjusted in an iterative manner from one computation to the next until they become consistent with the level of strain developed in the element during the ground shaking. Therefore, the FLUSH code is not a linear code, since the soil properties are adjusted to be strain compatible in an average sense. Also, it is not a truly nonlinear code, since the soil properties are not modified to correspond to the variations with time of strain or stress invariants that occur during the ground shaking.

2.3.5 NONLINEAR FINITE ELEMENT ANALYSIS TECHNIQUES

An important class of finite element techniques for soil/structure interaction analyses utilizes inelastic variable modulus and/or theory of plasticity representations of the soil materials. These models differ from the equivalent

linear model used in FLUSH code in that they represent the nonlinear stress/strain characteristics of the soil medium at a particular instant of time, based on incremental stress/strain laws, yield criteria, and flow rules that have been incorporated into the methodology. However, as a result of this increased refinement, the computer time and costs required to perform a dynamic analysis of a given soil/structure system are greatly increased. Therefore, it is not practical to use these models in parametric studies for assessing effects of uncertainties in the free-field input motions, soil properties, and structure properties on the response of the soil/structure system. Furthermore, current soil testing techniques are, in some cases, not capable of providing a sufficiently complete set of data with which to fit some of the plasticity-based models. In this regard, the requirements of such models have had an influence on the types of soil and rock tests being conducted; i.e., there have been a number of recent reports in which the complete states of stress necessary to provide sufficient data for fitting these models have been measured. However, as of the late 1970's, such experimental procedures were not yet incorporated into nuclear plant site investigations.

Prior applications of nonlinear finite element analysis techniques range from early two-dimensional analysis techniques for analysis of earth dams and nuclear plants to more recent two-dimensional, axisymmetric, and three-dimensional techniques for analysis of reinforced concrete dams, nuclear plant containment structures, and soil test samples. These techniques all utilized nonlinear variable modulus or cap models with various incorporated yield criteria and flow rules. These general model types are found in several current nonlinear finite element programs, such as ANSYS, TRANAL, MARC, NASTRAN, and ADINA.

2.4 ASSESSMENT OF SOIL/STRUCTURE INTERACTION ANALYSIS TECHNIQUES

An assessment of the applicability of the various soil/structure interaction analysis techniques to nuclear plants is summarized here, based on the main features of the various techniques as shown in Table 2-1.

- a. Techniques for analyzing a finite element model of the soil and structure, (i.e., with three-dimensional and nonlinear programs that are available) represent the most complete approach, from a purely technical consideration, that now exists for analyzing soil/structure interaction effects. However, the excessive costs associated with the application of these finite element techniques have prevented them from being used to their fullest capability in nuclear plant design applications, and have necessitated the use of certain approximations in the modeling. The most important of these approximations is the current practice of using two-dimensional techniques to analyze soil/structure systems that are essentially three-dimensional problems. Other approximations necessitated by cost considerations involve the use of simplified soil material models that shorten the calculations for each time step, and the implementation of approximate energy-absorbing boundary conditions that reduce the grid size. However, the finite element method, even in simplified form, has several attractive features for carrying out soil/structure interaction analyses. For example, this method can consider the effects of structure embedment, through-soil coupling of the response of adjacent structures,

and irregular topography and soil layering.

- b. The input motions and properties of the soil and structure should be varied when analyzing nuclear power plant structures, in order to account for uncertainties in defining these parameters. Cost considerations preclude the use of finite element methods for parametric studies that investigate effects of these variables on the structure response; such parametric studies are more feasible using the simpler continuum or spring-dashpot methods.
- c. The FLUSH code is now (late 1970's) the most widely used of the finite-element techniques for nuclear plant applications. However, the code is limited to considering only vertically propagating body waves. Also, the FLUSH code, which is not a nonlinear code, utilizes a simplified "equivalent linear" model of the soil properties and therefore can represent strain-dependent effects at less cost than the more sophisticated nonlinear finite element techniques. The important uncertainties associated with the use of the equivalent linear model require careful evaluation.
- d. Continuum methods, which utilize a three-dimensional finite element model of the structure and a continuum representation of the soil medium, show considerable promise for future applications to nuclear plants. The methods can carry out three-dimensional analyses much less expensively than can finite element methods, and they can consider arbitrarily incident seismic waves that propagate through

TABLE 2-1. SUMMARY OF FEATURES OF SOIL/STRUCTURE INTERACTION ANALYSIS TECHNIQUES

Item	Finite Element Method	Continuum Method	Spring-Dashpot Method
Structure Representation	Can represent three-dimensional linear or nonlinear structural behavior using a model comprised of beam, plate, shell, membrane or solid elements. However, because of the high costs associated with three-dimensional nonlinear analyses, two-dimensional models are nearly always used in nuclear plant design applications.	Can represent three-dimensional linear structural behavior using a model comprised of beam, plate, shell, membrane, or solid elements. Versatility in representing structure is equivalent to that of finite element method.	Can compute three-dimensional linear structural response using a model consisting of discrete masses interconnected by beam-type elements. Properties of elements in some cases are developed from static condensation of finite element models, however in most cases a simple model comprised of linear beam elements is used.
Costs	Costs of three-dimensional nonlinear finite element analyses are very high although progress is being made in this area. Because of this, two-dimensional planar models are typically used in nuclear plant design applications. FLUSH code model using equivalent linear soil representation is much less expensive than two-dimensional nonlinear analyses but contains uncertainties and limitations.	Costs associated with three-dimensional analyses are much less than for finite element method. Parametric analyses of effects of variations in input motions can be carried out quite economically (for a given soil/structure system). If properties of structure or soil medium are varied, costs of parametric analyses are higher.	Costs are relatively low. Because of this, method is widely used for carrying out parametric analyses of effects of uncertainties in input motions, soil properties, and structure properties.
Site Soil Profile and Material Property Representation	In general, can represent any irregular topography or soil layering contained in soil profile, and can use nonlinear material models to represent strain-dependent soil properties. FLUSH code considers horizontal soil layers and uses equivalent linear soil model to simulate strain-dependent behavior.	Considers site profile comprised of viscoelastic, horizontally layered soil media.	Considers site profile to be an elastic half-space.
Foundation Representation	Can represent any number of arbitrarily configured deformable or rigid foundations that can be embedded to any depth below ground surface. Separation of foundation and soil medium can be considered.	Can represent any number of arbitrarily configured rigid foundations bonded to surface of soil medium, it is relatively simple to extend this capability to accommodate deformable foundations as well.	Represents single foundation as an equivalent rigid circular or rectangular footing. Embedment effects can be approximated using results of Novak and his associates (see AA, 1980).
Through-Soil Coupling of Response of Adjacent Structures	Can be represented.	Can be represented.	Cannot be represented.
Wave Propagation Effects	Can represent effects of waves scattered and diffracted at the foundations at layer interfaces, and at the ground surface. Radiation damping can be approximated by means of energy-absorbing boundary conditions along base and sides of soil grid. In general traveling wave effects from arbitrarily incident seismic waves can be represented, although FLUSH code considers only vertically incident body waves.	Can represent effects of waves scattered and diffracted at the foundations at layer interfaces, and at the ground surface. Radiation damping is represented exactly. Traveling wave effects from arbitrarily incident seismic waves can be represented.	Radiation damping is represented only at low frequencies (consistent with spring-dashpot assumptions). Wave scattering and diffraction effects cannot be represented.

viscoelastic horizontally layered soil media to interact with any number of foundations of arbitrary shape. The main drawbacks of continuum methods are that they cannot yet consider embedded foundations of arbitrary shape (although research is now being carried out to improve the

capabilities of the methods in this area) and they do not represent strain-dependent soil properties (as of the late 1970's).

- e Spring-dashpot methods, which model the structure using an assemblage of one-dimensional elements and the soil

as a spring and dashpot based on elastic half-space theory, are the simplest of the existing methods for analyzing soil/structure interaction effects. As a result, computational costs associated with the implementation of these methods are quite low and the methods are therefore often used to carry out parametric studies of effects of uncertainties in input motions and model parameters. However, the simplified structural and soil model used in spring-dashpot methods in general will not represent

all of the important soil/structure system response characteristics. For example, effects of through-soil coupling of adjacent structures; irregular topography, foundation shape, and soil layering; and wave-scattering effects at the foundation/soil interface cannot be represented using the spring-dashpot method. The method can, however, approximately represent three-dimensional response characteristics and foundation embedment effects.

CHAPTER 3

ANALYSIS GUIDELINES

3.1 INTRODUCTION

The main objective of soil/structure interaction analyses is to predict the dynamic response of the structure, such as the in-structure response spectra and the state of stress and deformation in the structure, including the restraint of the structure by the soil. This chapter summarizes the recommended procedures that can be used to perform soil/structure interaction analyses for nuclear power plant structures. Additional details on the soil/structure interaction analysis guidelines are presented in AA (1981b).

The main elements of a typical soil/structure interaction analysis are (1) specification of the free-field ground motion, (2) modeling of the soil/structure system, and (3) calculation of the system response.

3.2 CHARACTERISTICS OF ANALYSIS TECHNIQUES

Available analysis techniques for handling soil/structure interaction problems can be broadly classified on the basis of the type of information desired: (1) deterministic methods, where specific response values are calculated for a particular seismic excitation, or (2) probabilistic methods, where statistical estimates of the response under a (theoretically) infinite ensemble of seismic excitations are determined.

3.2.1 DETERMINISTIC METHODS

Within the class of deterministic methods, various techniques can be classified according to two methods of approach: direct methods and substructure methods.

In the direct methods, a set of free-field ground motion is applied to the subsurface boundaries of a discrete model, and then the

response of the combined soil/structure system is computed. Discretization techniques can involve finite element or finite difference approaches of any desired degree of sophistication, although finite element methods are much more widely used in current practice. The responses of the soil as well as that of the structure are determined simultaneously. Due to practical cost and time considerations, only the overall dynamic behavior of the structure is typically determined in the direct method of analysis. A subsequent structural analysis utilizing a refined model subjected to soil interface loadings computed from the soil/structure interaction analysis must be performed if detailed structural response information is needed.

Unlike the direct method, where the dynamic response of the soil and structure is handled in one step, the substructure methods reduce the soil/structure interaction problem to the solution of a sequence of three simpler problems: (1) solution of the kinematic interaction problem, (2) calculation of the foundation impedance functions, and (3) solution of the coupled soil/structure system; the resulting solution is then superposed to obtain the total solution. This method offers considerable cost and time savings over finite element methods when fully three-dimensional analyses are required; however, due to a reliance on the principle of superposition, only systems that can be adequately modeled as equivalent linear systems can be handled by this approach. This method has not been widely used for carrying out actual soil/structure interaction analyses for nuclear plant structures except as a check on other solutions in some special cases. It may be more widely used in the future, however, when current research efforts directed toward

incorporating embedment effects for arbitrarily shaped foundations are completed.

3.2.2 PROBABILISTIC METHODS

Of the various methods that can be used to calculate the seismic response of structural systems, those based on random vibration techniques offer the most rational approach for handling the response of soil/structure systems under stochastic excitations. However, due to the mathematical difficulties that arise in the solution of complex soil/structure interaction problems under nonstationary random excitation, the state of the art of random vibration analysis methods is such that numerous simplifying assumptions (including many assumptions of dubious validity) have to be made before an approximate probabilistic solution can be obtained.

Due to the different limitations on the range of validity of such analysis methods, it is recommended that the direct approach discussed above should be used at this time to analyze soil/structure interaction problems for the design of nuclear power plant systems. Perhaps there will exist compelling reasons that justify the use of another method and in that case other approaches should be considered, but the basis for the conclusion recommended here can be substantially validated (AA, 1981b).

3.3 MODELING OF SOIL/STRUCTURE SYSTEMS

The deformational theories used in the modeling of the soil/structure system can vary from relatively simple linear two-dimensional models to general nonlinear three-dimensional models of the soil and the structure. The following deformational formulations are commonly used in soil/structure interaction analyses:

- Plane-strain formulation, which is

suitable for modeling structures whose dimension normal to the plane of the model is very long

- Axisymmetric formulation, which can be used if the structure as well as the soil has axisymmetric geometry and material properties, and if the loads are separable into Fourier harmonics expanded in the circumferential direction
- Pseudo-three-dimensional formulation which uses conventional two-dimensional plane-strain formulation that is augmented by out-of-plane viscous dampers acting in the third dimension to simulate the three-dimensional effects of radiation damping
- General three-dimensional formulation which can handle arbitrary geometries for the soil and structure, including their nonlinear properties

It is important that, regardless of which formulation is used, care must be exercised in using a refined second-stage analysis for the detailed three-dimensional response of the structure to two uncoupled horizontal excitation components and one vertical component.

On the basis of cost/benefits ratio, it is recommended that for cases of soil/structure interaction analysis that are not research oriented, the soil/structure interaction investigation should use a two-dimensional or axisymmetric formulation.

3.3.1 SOIL CONFIGURATION AND BEHAVIOR

Soil configurations have significant effects on the response of soil/structure systems. Among the commonly used configurations are:

- The uniform elastic or viscoelastic half-space, which is an appropriate model for a foundation resting on a

relatively thick and uniform soil layer.

- A single layer of soil of uniform thickness and infinite horizontal extent overlaying a stiff uniform half-space.
- A sequence of uniform elastic or viscoelastic layers of uniform thickness and infinite horizontal extent.
- A discrete model (usually finite element) of a soil profile of arbitrary topography and elastic, viscoelastic, or inelastic soil layers of arbitrary orientation and geometry.

When using the direct method for soil/structure interaction analyses, a common but not necessary assumption, particularly for plane-strain or pseudo-three-dimensional representation, is that the soil configuration consists of horizontally stratified soil layers.

In dealing with the direct method of analysis, two approaches are available to analyze a soil/structure interaction system with nonlinear soil properties:

- Equivalent linear analysis involving the iteration of the properties of a linear viscoelastic material model characterized by two elastic parameters and a damping factor.
- Nonlinear analyses using suitable nonlinear constitutive soil models. Such procedures are still in a research and development stage, and are not appropriate for structural design purposes.

Since the actual state of stress in the soil under earthquake excitation is three-dimensional, the generation of three-dimensional nonlinear constitutive models is an active area of research. The nonlinear models that are currently most widely used include:

- The cap model, which is a continuum model based on the classical incremental theory of plasticity.
- The multisurface plasticity model that simulates hysteretic effects.
- The endochronic model, an alternative to the plasticity models, that uses intrinsic time as a basis for measuring the memory of past deformation history.

3.3.2 FOUNDATION PROPERTIES

For soil/structure interaction, the main characteristics of concern in modeling foundations are the configuration, mass, and stiffness properties, and their proximity to one another.

Actual nuclear power plant structures possess extremely complicated geometries whose authentic detailed three-dimensional representation while theoretically possible, involves prohibitive computer costs when finite element direct methods of analyses are used. Therefore, simplified two-dimensional representations of these structures are typically used in current practice, although neither systematic analytical nor experimental studies have been conducted to the degree necessary to evaluate the effects of soil/structure interaction analysis. The nature of these simplifications requires careful evaluation on a case-by-case basis.

When representing three-dimensional foundations by two-dimensional models, care should be exercised in selecting an appropriate cross section. No systematic procedure is currently available to select an equivalent cross section for foundations with complicated geometrical shapes.

3.3.3 STRUCTURAL MODELS

Structural models used in soil/structure interaction analysis can be relatively crude and

designed to represent only the dominant features of the structural dynamic response. This phase of the analysis should be followed by another dynamic analysis in which a detailed three-dimensional model of the structure is subjected to one vertical component and two uncoupled horizontal components of foundation motion resulting from the earlier soil/structure interaction analysis.

Modeling of complicated multidegree of freedom structural systems, particularly if they are nonlinear, requires careful attention in order to obtain valid results. Where appropriate test data are available, modern methods of system identification should be used in determining the significance of various modes and phenomena that should be included in an appropriate model.

3.4 CONSIDERATIONS ON CALCULATING THE RESPONSE

3.4.1 SPECIFICATION OF FREE-FIELD GROUND MOTION

Factors that define earthquake motions at a site are the size and location of the earthquake, its transmission path, and local site properties. The usual procedure for characterizing earthquake ground motion is to specify the earthquake intensity, spectral characteristics, and duration.

Because of computational convenience, it is common to assume that the wave propagating mechanism at a site consists of vertically incident P- and SH-waves. Such an assumption leads to: (a) vertical motion is entirely transmitted by P-waves, (b) horizontal motion depends only on SH-waves, (c) all points on the soil surface undergo identical free-field deformations, and (d) no torsional component of the input motion is generated. This assumption may not be conservative, particularly for very stiff sites and significant surface wave components.

When using the direct method for soil/structure interaction analysis, a reference (control) motion and a reference (control) point must be specified, in addition to the free-field ground motion which must be specified at the boundaries of the model. The location of a control point should be a free surface and should be an assumed outcropping either at depth or at the surface of the soil.

3.4.2 SOLUTION PROCEDURE

Analytical solution techniques can be separated into three basic methods: modal analysis, direct integration, and frequency-domain. Table 3-1 provides a comparison of these three analysis methods. A comparison of the four most commonly used methods for modal extraction is given in Table 3-2. For the time domain methods, a choice must be made between response spectrum analysis and time-history analysis. Table 3-3 provides a comparison of these two choices, and Figure 3-1 compares the results obtained from response spectrum and time-history methods for a nuclear reactor structure having fourteen different floor slabs.

FIGURE 3-1. COMPARISON OF TIME HISTORY AND RESPONSE SPECTRUM METHODS

TABLE 3-1 COMPARISON OF DIFFERENT METHODS OF ANALYSIS FOR NUCLEAR PLANT STRUCTURES

No	Item to be Compared	Modal Analysis Methods*	Direct Integration Methods†	Fourier Transform Method
1	Major Computational Effort Involved	Extract mode shapes and frequencies	Triangularization and back substitution for IIM and formulation of the load vector for EIM	Frequency response functions and forward and inverse Fourier transforms
2	Total Cost of Computations	Most economical Cost is essentially independent of the duration of the input record	Most expensive Cost is directly proportional to the duration of the input record	Intermediate Cost is directly proportional to the duration of the input record
3	Input for Analysis	Design spectrum for RSM A time history to envelop design spectrum for THM	A time history to envelop design spectrum	A time history to envelop design spectrum
4	Multidirectional Input	In RSM, responses from the individual analyses must be combined in some manner Can be accounted for in one operation for THM	Can be accounted for in one operation by simultaneously applying time histories for horizontal and vertical input motions	Responses due to each input motion must be combined in the frequency or time domain to obtain the total responses
5	Type of Analyses	Only linear analyses are possible	Linear and nonlinear analyses are possible	Nonlinear analyses through equivalent linear analyses are possible
6	Energy Absorbing Boundary Using Dashpots	Not possible	Can be used effectively to simulate infinite extent of soil media	Theoretically possible However, the existing computer programs do not seem to include this capability
7	Form of Damping	Modal damping ratios can directly be used The damping matrix must be such that the equations are decoupled under appropriate transformation	Damping matrix must be constructed from the given modal damping ratios using an available method Also the form of the damping matrix is not restricted Rayleigh type damping, when used, conserves computer storage	Damping matrix must be constructed from the given modal damping ratios using an available method Also the form of the damping matrix is not restricted Rayleigh type damping, when used, conserves computer storage
8	Different Damping in Different Parts of the System	Cannot be handled directly An approximate procedure must be used to estimate effective damping ratios	Can be handled directly if a damping matrix can be provided for each part	Can be handled directly if a damping matrix can be provided for each part
9	Accuracy of Solution	Depends on the number of modes considered For RSM, the maximum modal responses are combined in some approximate way	Desired accuracy can be obtained, in general, by adjusting the integration time step	Desired accuracy can, in general, be obtained by adjusting the number and range of frequencies considered

*RSM = Response Spectrum Method
THM = Time History Method

†IIM = Implicit Integration Method
EIM = Explicit Integration Method

TABLE 3-2. COMPARISON OF METHODS FOR COMPUTING MODE SHAPES AND FREQUENCIES

No	Item for Comparison	Householder-Inverse Iteration	Generalized Jacobi Iteration	Determinant Search	Subspace Iteration
1	Is the preliminary transformation to the standard form* $([A]\{Y_i\} = \lambda_i\{Y_i\})$ required?	Yes	No	No	No
2	Major computational effort	Reduction of [A] to tri-diagonal form and iterations for computing eigenvalues and eigenvectors	Jacobi rotation matrices are used to transform [K] and [M] to diagonal form	Triangular factorization of $[K] - \omega^2[M]$ and inverse iterations	Solution for trial vectors and computations of reduced [K] and [M] and solutions of the reduced eigenvalue problems
3	Whether all or a few eigenvalues are computed in one operation	All eigenvalues and eigenvectors are computed	All eigenvalues and eigenvectors are computed	One eigenvalue and one eigenvector are computed	A desired number of eigenvalues and eigenvectors are computed in one operation
4	Whether the method takes advantage of the banded form of the stiffness matrix	No	Yes	Yes	Yes
5	Is the method suitable when the matrices have small bandwidth?	No	Yes	Yes	Yes
6	Is the method suitable when the matrices have large bandwidth?	Yes	No	No	Yes

*This transformation can be accomplished in several ways as described by Clough (1969). For example, if the mass matrix is diagonal with nonzero positive elements the transformation is given by

$$[A] = [M^{-\frac{1}{2}}][K][M^{-\frac{1}{2}}], \{Y_i\} = [M^{-\frac{1}{2}}]\{\phi_i\}, \lambda_i = \omega_i^2$$

TABLE 3-3. COMPARISON OF NORMAL MODE METHODS

Item to be Compared	Response Spectrum Analysis	Time-History Analysis
1 Computational Economy	Most economical Cost is independent of the duration of the input record, since no earthquake record is actually used for the analysis	Computational effort increases with the duration of the input record
2 Accuracy of Solution	Approximations involved in considering a limited number of modes, and in the way the maximum modal responses are combined	Depends on the number of modes considered in the analysis Desired level of accuracy can generally be obtained
3 Major Computational Effort	Computation of desired number of frequencies and mode shapes	Computation of desired number of frequencies and mode shapes
4 Input for Analysis	Design spectra can be directly used	A time history (real or artificial) to envelop the design spectra must be developed
5 Multidirectional Input	Responses from the individual analyses must be combined in some manner	Can be accounted for by simultaneously applying time histories of horizontal and vertical input motions
6 Linear and Nonlinear Analyses	Only linear analyses are possible	Only linear analyses are possible
7 Energy Absorbing Boundary	Not possible	Not possible
8 Form of Damping	Modal damping ratios can directly be used The damping matrix must be such that the equations are decoupled under appropriate transformation	Modal damping ratios can directly be used The damping matrix must be such that the equations are decoupled under appropriate transformation
9 Different Damping in Different Parts of the System	Cannot be handled directly An approximate procedure must be used to estimate effective damping ratios	Cannot be handled directly An approximate procedure must be used to estimate effective damping ratios
10 Nearly Equal Frequencies	Special procedure is required to combine the maximum modal responses	Can be handled routinely
11 Relative Displacement between Structures	Assumption must be made regarding phasing of adjacent building responses Out-of-phase assumption typically leads to overly conservative values	Can be estimated properly (if two buildings included in same model)
12 Floor Response Spectra for the Analysis of the Equipment	Special empirical methods must be used for estimation	Can be obtained routinely from computed time histories of motion at appropriate locations within the structure

3.4.3 MESH SIZE

The mesh size used in discrete mathematical models (whether finite element or finite difference) affects the solution in a way similar to the effects of time or frequency increments. For accurate reproduction of a frequency of interest in the solution domain, the mesh size must be sufficiently small; however if the mesh size is excessively small, prohibitive computer costs will result. Decisions regarding the optimum mesh size should adhere to the following recommendations:

- The mesh size should not exceed about one-seventh of the shortest wave length of interest.
- The mesh size near the edge of the foundation should be smaller than those in the middle.
- Higher derivatives of the response, whether spatial or temporal, require a finer mesh if they are to be calculated with the same accuracy as the basic response quantity.
- A mesh size of at least 10 elements over the width of the foundation mat is needed for accurate calculation of stress distribution under the structure.
- If the primary response is due to the vertical propagation through the soil of P-waves or shear waves, it is acceptable to keep the height of the soil elements constant but to change their horizontal size.
- When, for the sake of economy, a fine mesh around the foundation is enlarged as the distance from the foundation increases, the change in mesh size should be gradual.
- When dealing with general wave fronts, or if both the horizontal as

well as the vertical dimensions of the mesh size are changed, calibration checks should be conducted on the accuracy of the results over the frequency domain of interest.

3.4.4 SIZE OF SOIL REGION

The location of soil boundaries in a soil/structure interaction analysis using a finite element direct method of analysis is influenced by static (e.g., pressure under foundation) and dynamic (e.g., wave reflection at artificial boundary) effects. It is noted that for substructure methods, radiation damping is represented exactly; i.e., only for finite element methods does the possibility of unwanted wave reflections from artificial boundaries of the soil grid require careful assessment of the locations of these soil grid boundaries. For accurate results the following rules should be followed:

- The dimension of the soil core region should be such that viscous or elementary boundaries are placed at a distance of 5 to 10 radii (or a half-foundation width) from the foundation edge.
- The location at the bottom boundary should coincide with a sharp transition in the soil properties.
- Unless rock is present at a shallower depth, the lower boundary should be at least 2 to 4 radii below the foundation.

3.4.5 INTEGRATION TIME STEP

Timewise integration schemes used in direct methods of analysis can be conveniently separated into two basic groups: implicit methods which, due to their unconditional stability, can tolerate a relatively larger time step; explicit methods which require a relatively small time step for stable solution, but which do

not involve the inversion of large matrices at every time step.

As in the case of mesh size, care should be exercised in selecting the time step size so that it is small compared to the shortest period (highest frequency) of interest. Normally, when the time step is less than one-fourth the shortest period of interest, satisfactory results are obtained. Careful attention should be directed to the selection of time step size and mesh size in nonlinear systems to assure that true components of motion are not filtered out.

3.4.6 FREQUENCY SPECIFICATIONS

When using soil/structure interaction solution procedures that employ the frequency domain approach, the frequency range used should be sufficiently wide to encompass the significant frequency components of both the excitation and response. Transfer function calculations should use at least 150 equally spaced frequency points, and Fourier transform calculations should use at least 2048 frequency points. Typically it is satisfactory to limit the upper bound of this frequency range to 20 Hz.

3.4.7 DAMPING VALUES

Energy losses that must be considered include material damping and hysteretic losses within the structure or in the surrounding soil materials; frictional losses at structure connections, equipment mounts or at soil/structure interfaces; separation of the soil and structure along their interface; and the radiation of energy away from the structure foundation and into the surrounding soil (radiation damping). The magnitudes of these energy losses depend on such factors as the amplitude and frequency content of the applied excitations, operating temperatures, material properties, site conditions, soil/foundation interface condition, and

the configuration and design details of the nuclear plant.

For purposes of computational expediency, the various energy losses in nuclear plants are typically represented in dynamic analyses as equivalent viscous (i.e., relative velocity dependent) damping factors. However, the implications of this type of approximation, and the basis for selection of equivalent viscous damping factors for purposes of design, must always be carefully evaluated.

3.4.8 EXCITATION CHARACTERISTICS

When random vibration techniques are used, the format of the input should include a probabilistic characterization of the loads expressed in terms of appropriate statistical quantities that represent the earthquake loads as nonstationary random processes.

If a deterministic analysis is to be conducted, then for the cases that need a time history of the excitation, available computer codes should be used to generate an artificial earthquake record compatible with any specific design spectra.

It is usually assumed that the wave propagation mechanism at a site consists of vertically incident P- and SH-waves. The significance of nonvertically incident propagating waves on the structural response depends on the nature of the earthquake motion, characteristics of the site, and the design of the structure. At present, no general guidelines are available to assess a priori the importance or unimportance of this phenomenon. Most soil/structure interaction analyses are based on the assumption that the input consists of vertically propagating waves.

For solution procedures that use the design response spectra directly, the excitation is completely specified by the design spectra that are applicable to the site.

3.4.9 SOIL PROPERTIES

Accurate representation of realistic soil configurations encountered in soil/structure interaction analysis should account for the non-linearity of soil properties as well as the three dimensionality of the stress/strain relationship in a manner consistent with the type of data generated by the field and laboratory soil tests.

3.4.10 STRUCTURE-STRUCTURE INTERACTION

If the separation of some structures is relatively small, then reliable estimates of the extent of coupling through the soil in the dynamic response between the closely spaced structures requires three-dimensional analysis. Simplified two-dimensional analyses may overemphasize this effect.

3.4.11 GEOMETRICAL ASPECTS

Commonly used deformational formulations in soil/structure interaction analyses are: plane-strain, axisymmetric, two-dimensional, pseudo-three-dimensional, and general three-dimensional. The appropriateness and limitations of each of these formulations have been detailed in other reports (AA, 1980; 1981b).

As noted above, the state of the art of computational techniques is such that for conventional soil/structure interaction analysis (i.e., not of a research nature), a two-dimensional or axisymmetric formulation will yield the optimum cost/benefit ratio.

3.4.12 BOUNDARY CONDITIONS

Due to the inherent limitations of finite element models to generally represent a semi-infinite medium, there is a need to eliminate the physically incorrect effects of artificial boundaries along the sides and bottom of the discretized model of finite dimensions. This is

usually accomplished by introducing boundary elements that tend to eliminate the reflection caused by regular boundaries, with the aim of making the response of the soil at distances that are far from the structure essentially the same as the free-field conditions in the physical system.

It is common practice to assume that the vertical radiation of energy below the bottom fictitious boundary is negligible. In dealing with boundary conditions involving the lateral boundaries, several types of mathematical boundaries are available: elementary, viscous, and consistent and transmitting. The subject of boundary conditions has been presented in detail (AA, 1976b).

3.4.13 INCLINED ROCK STRATA

Only limited information is currently available regarding the effects of inclined rock strata on the site response analyses of nuclear power plant sites. In one study (AA, 1976a) two rock site examples were considered: in one example, the sequence of layers of sedimentary rocks were horizontal, while in the other example layers were inclined (i.e., dipping). On the basis of this study it was concluded that: (1) ground surface records can adequately represent vertical interior motions at the foundation level and (2) horizontal ground surface records far exceed the corresponding response at the foundation for these sites.

3.4.14 NONLINEARITIES

Sources of nonlinearities encountered in soil/structure interaction analysis can be conveniently divided into two groups: material and geometric. Material nonlinearities include those associated with the soil and structure, while geometric nonlinearities involve phenomena such as debonding between soil and structure,

TABLE 3-4. NONLINEAR FINITE ELEMENT ANALYSIS PROCEDURES

Items for Comparison	ANSYS (DeSalvo and Swanson, 1979)	TRANAL (Baylor, Bieniek, and Wright, 1974)	MARC (Marc, 1979)	NASTRAN (McCormick, 1979)	ADINA (Bathe, 1978)
Static Analyses	Linear, thermal, plastic, buckling, creep Nonlinear material properties and geometry	None	Linear, thermal Nonlinear material properties and geometry	Linear, buckling, thermal Nonlinear material properties and geometry	Linear, thermal Nonlinear material properties and geometry
Eigenvalues and Eigenvectors	Yes	No	Yes	Yes (3 methods, restartable, complex roots)	Yes
Dynamic Analysis	Modal (linear) Nonlinear Transient Implicit Harmonic response	Nonlinear Step-by-step Explicit	Linear and nonlinear Step-by-step Implicit Explicit Modal	Linear and nonlinear Step-by-step Implicit	Linear and nonlinear Step-by-step Implicit Explicit
Element Types					
Truss	Yes	No	Yes	Yes	Yes
Beam	Yes	No	Yes	Yes	Yes
Plate and Flat Shell	Yes	No	Yes	Yes	Yes
Curved Shell	Yes	No	Yes	Yes	Yes
Solid	Yes	Yes	Yes	Yes	Yes
Two-Dimensional	Yes	No	Yes	Yes	Yes
Special	Pipe and fluid elements	No	Concrete pipe and fluid elements	Substructures Pipe and fluid elements	Fluid elements
Loading					
Nodal Point	Yes	Yes	Yes	Yes	Yes
Member	Yes	Yes	Yes	Yes	Yes
Gravity	Yes	Yes	Yes	Yes	Yes
Initial Stress/Strain	Yes	No	Yes	Yes	Yes
Kinematic Boundary Conditions for Dynamic Analysis	Displacement, velocity, and acceleration	Displacement	Displacement	Displacement, velocity, and acceleration Energy-absorbing boundary conditions	Displacement
Maximum Number of Node Points	Dynamically allocated (D A)	D A	D A	D A	D A
Maximum Number of Elements	D A	D A	D A	D A	D A
Maximum Half-Bandwidth	Wavefront technique	D A	D A	D A (active column technique)	D A
Maximum Number of Load Cases	D A	D A	D A	D A	D A
Maximum Number of Materials	D A	D A	D A	D A	D A
Maximum Number of Cross Sections	D A	D A	D A	D A	D A
Graphic Output					
Grid or Mesh Plot	Yes	Yes	Yes	Yes	No
Mode Shapes	Yes	No	Yes	Yes	No
Time History	Yes	Yes	Yes	Yes	No
Response Spectra	Yes	No	Yes	Yes	No
Contour Plot	Yes	No	Yes	Yes	No
Automatic Mesh Generation	Yes	Yes	Yes	Yes	Yes
Bandwidth Minimization	Wavefront technique	Not applicable	Yes	Yes	No
Constrained DOF (slaving)	Yes	No	Yes	Yes	Yes
Special Features	Early conversion of Cartesian to cylindrical or polar coordinates Large deflections Interactive mode of computation Extensive output graphics	Bonding and debonding capability Subcycle integration capability	Extensive output graphics	Sparse matrix methods Extensive output graphics Executive control	

impacts between adjacent structures or elements, sliding of structures, etc. Nonlinearities may be distributed throughout the solution domain (e.g., soil properties) or be localized (closing of gaps at a certain point).

The area of nonlinear dynamic analysis and characterization of nonlinear processes is an active area of research in which numerous practical problems still await adequate treatment.

3.5 AVAILABLE COMPUTER CODES

Computer codes that perform linear or equivalent linear analyses include (1) SAP, (2) FLUSH, (3) CLASSI, (4) STARDYNE, (5) TRI/SAC, (6) STRUDL, and (7) CAST1. Codes capable of performing nonlinear analyses include: (1) ANSYS, (2) TRANAL, (3) MARC,

(4) NASTRAN, (5) STEALTH, (6) NONSAP, and (7) ADINA.

Major features of nonlinear and equivalent linear computer codes are summarized in Tables 3-4 and 3-5. Further details regarding this subject may be found in Werner and Hung (1982).

Code PLUSH can perform a probabilistic soil/structure interaction analysis on linear or nonlinear systems that can be treated as piecewise linear.

Codes that are acceptable providing their limitations are observed are (1) CLASSI and (2) CAST1. Other codes are also acceptable if their suitability is verified by comparison with those on the acceptable list.

TABLE 3-5. MAJOR FEATURES ON NONLINEAR AND EQUIVALENT LINEAR COMPUTER CODES

Code	Soil Properties	Type	Element Types	Large Strain Capability	Energy-Absorbing Boundary	Seismic Input	Strongly Nonlinear Interface	3-D Effects	Structure-Structure Interaction
TRANAL	Nonlinear	Explicit Time Domain, Finite Element	Continuum	Yes	No	General	Yes	Yes	Yes
STEALTH	Nonlinear	Explicit Time Domain, Finite Difference	Continuum	Yes	Yes	General	Yes	Yes	Yes
SAP 7	Nonlinear	Implicit Time Domain, Finite Element	Continuum, Structural	Yes	No	General	No	Yes	Yes
NONSAP	Nonlinear	Implicit, Time Domain, Finite Element	Continuum, Structural	Yes	No	General	No	Yes	Yes
DYNA3D	Nonlinear	Explicit Time Domain, Finite Element	Continuum	Yes	No	General	Yes	Yes	Yes
FLUSH	Equivalent Linear	Implicit, Frequency Domain, Finite Element	Continuum, Structural	No	Yes	Rigid Bedrock Shaking, Vertically Propagating Body Waves	No	No	Yes
SASSI	Equivalent Linear	Implicit Frequency Domain, Finite Element	Continuum Structural	No	Yes	General	No	Yes	Yes

CHAPTER 4

ROCK/STRUCTURE INTERACTION ANALYSIS EXAMPLE

4.1 INTRODUCTION

The description and results of a two-dimensional soil/structure interaction analysis of an example Liquid Metal Fast Breeder Reactor (LMFBR) plant using the TRI/SAC computer code are presented here. The objectives of the study were: (1) to evaluate the TRI/SAC code for solving problems of this magnitude with a rock site, (2) to evaluate the method of applying the input motion at a vertical side boundary, and (3) to obtain data for comparison with a future LUSH code analysis of the same problem.

4.2 DESCRIPTION OF LMFBR AND SITE

The LMFBR site was assumed to be underlain at shallow depth by rock which is basically composed of siltstone and limestone units. A geologic cross section through the plant along the section analyzed is shown in Figure 4-1. Overburden at the site consists of clay-type soils and varies in depth from 1 to 56 ft near the Category I structures. Beneath the overburden is a zone of rock that has been weakened by the weathering process. Beneath the

weathered rock is the sound and continuous rock.

The LMFBR was assumed to consist of eight main buildings, as shown in Figure 4-2, of which the Reactor Service, Reactor Containment, Steam Generator, Control, and Diesel Generator buildings are classified as Category I structures. These five Category I buildings share a common foundation mat that rests on the sound rock approximately 80 ft below the ground surface. Figure 4-3 shows building cross sections and the general arrangement of the LMFBR plant. The Reactor Service and Reactor Containment buildings are designed to have independent lateral force resisting structural elements, while the Steam Generator, Control, and Diesel Generator buildings share some common shear walls. Sufficient clearance is provided between the Reactor Service and Reactor Containment buildings and between the Reactor Containment and Steam Generator buildings to permit independent vibratory motion of these structures. Buildings such as the Plant Service and Turbine buildings, with their foundations on grade, are not classified as Category I structures.

FIGURE 4-1. GEOLOGIC CROSS SECTION THROUGH LMFBR SITE

FIGURE 4-2. LMFBR BUILDING PLAN

FIGURE 4-3. LMFBR BUILDING CROSS SECTIONS

The LMFBR site seismic design response spectra, which define the vibratory ground motions of Safe Shutdown Earthquake (SSE), are assumed to be the scaled Regulatory Guide 1.60 spectra (USAEC, 1973), so that the time motions generated from them will have peak accelerations of 0.18 g.

4.3 MATHEMATICAL MODEL

The mathematical model used for the analysis (shown in Fig. 4-4) is a two-dimensional finite element model consisting of plane strain, plane stress, and beam elements. It has 930 node points, 902 plane strain/stress elements, and 22 beam elements. The soil island and foundation mats were represented by plane strain elements and the buildings by plane stress elements. Beam elements were used only to model the dome of the Reactor Containment

Building and the gantry crane on the Steam Generator Building.

The input motion was applied at each node along the left boundary of the soil island. At each node along the base and right boundary of the soil island a pair of dashpots were placed to absorb the incident waves. This is the absorbent boundary technique which is used to prevent unwanted reflections from the boundaries of the model that do not correspond to the physical boundaries of the problem (Lysmer and Kuhlemeyer, 1969).

The rock and soil which constitute the soil island were assumed to be horizontally layered with the properties given in Table 4-1. It was also assumed that the overburden had been removed and replaced with backfill material. The dimensions selected for the large soil elements were based upon the shear wave speed

FIGURE 4-4, FINITE ELEMENT TRI/SAC MODEL OF LMFBR

TABLE 4-1. ROCK AND SOIL PROPERTIES USED FOR THE ANALYSIS

Soil Type	Density, lb/ft ³	Shear Modulus, lb/in. ²	Poisson's Ratio	Elevation, ft	
				Left Side of Model	Right Side of Model
Rock	150	388,000	0.3	336 - 760	336 - 774
Sand	120	12,500	0.35	760 - 776	
Sand	120	11,500	0.35	776 - 786	774 - 786
Sand	120	10,600	0.35	786 - 796	786 - 798
Sand	120	9,440	0.35	796 - 806	798 - 810
Sand	120	6,600	0.35	806 - 816	810 - 816

in rock, as calculated from the properties contained in Table 4-1.

The following technique was used to represent the three-dimensional structures by a two-dimensional model:

- a. The structures, as they appear in Section AA of the plant, were divided into plane sectors. (A sector, for example, might be defined by adjacent floors and partitions of a building and represented by one or more plane elements in the model.)
- b. The stiffness and mass properties of all structures and equipment lying in this sector were combined and divided by the combined lengths of the Steam Generator and Diesel Generator Buildings in the direction normal to Section AA to give average mass and stiffness properties. (The model used in this analysis was formulated in inch units, so the plane strain elements used to model the soil island and foundation mats had mass and stiffness properties corresponding to a slice 1 in. thick.)
- c. The average mass and stiffness values were used to define the properties of the finite elements for that sector.

The length used to average the mass and stiffness data is obviously a very important

parameter since it essentially determines the thickness of the soil island which supports the buildings. Clearly, one would expect less interaction with a thicker soil island. Since there are presently no definitive guidelines for selecting this parameter, the problem warrants additional study.

4.4 INPUT MOTION

The N69°W and vertical components of the Castaic records of the 1971 San Fernando earthquake (CIT, 1973) were used for input motion in the analysis. These records are graphed in Figures 4-5 and 4-6.

Only the first 3 sec of these records were used for the response analysis. The vertical component was scaled by the factor 1.33, which changed the peak acceleration from 0.156 g to 0.208 g. The magnitude of the horizontal component was unaltered. The response spectra of the scaled motions generally enveloped the criteria spectra based on Regulatory Guide 1.60 spectra. These records were applied uniformly along the left boundary of the model. Assuming the direction of wave travel is nearly horizontal, it is not necessary to phase input motion which is applied at a vertical boundary. This is considered a reasonable assumption for hard rock sites.

The Castaic records were selected because the strong motion occurs in the first few seconds, which makes it possible to truncate them

FIGURE 4-5. CASTAIC TIME-MOTION RECORDS, N69°W COMPONENT — SAN FERNANDO EARTHQUAKE, FEBRUARY 9, 1971

FIGURE 4-6. CASTAIC TIME-MOTION RECORDS, VERTICAL COMPONENT — SAN FERNANDO EARTHQUAKE, FEBRUARY 9, 1971

and still obtain the significant excitation, and because their response spectra match the criteria spectra reasonably well in the frequency range which contains the fundamental building frequencies.

4.5 RESPONSE ANALYSIS

The dynamic response of the model was obtained by the method of direct integration using the TRI/SAC code (AA, 1976c) and the 1108 UNIVAC computer. The equations were solved for 3.02 sec using a time step of 0.01 sec. Two hours and 35 min of computer time were required, or 30 sec per time step. Approximately one additional hour was required to check out the model and process the response data.

Use of a smaller time step would have given somewhat better results. This was demonstrated by integrating the equations for

three steps using a time step of 0.005 sec and comparing the peak free-field accelerations with those obtained by using a time step of 0.01 sec. The comparison revealed higher response values for the shorter time step, and these values were very nearly equal to the input values. Thus, the larger integration step of 0.01 sec effectively clipped the peaks of the input motion. This was especially true for the vertical component (Fig. 4-6) which has less low frequency content than the horizontal component (Fig. 4-5). The comparisons between input and free-field response spectra, shown in Figures 4-7 and 4-8, are quite good, however, and it was decided the improved accuracy gained by using the 0.005 sec time step would not justify the additional computing cost.

The TRI/SAC code utilizes Rayleigh damping, which is frequency dependent and completely defined by two parameters. For this

FIGURE 4-7. COMPARISON OF HORIZONTAL FREE-FIELD RESPONSE SPECTRUM WITH HORIZONTAL INPUT MOTION SPECTRUM AND HORIZONTAL SSE SPECTRUM

FIGURE 4-8. COMPARISON OF VERTICAL FREE-FIELD RESPONSE SPECTRUM WITH VERTICAL INPUT MOTION SPECTRUM AND VERTICAL SSE SPECTRUM

example the parameters were selected to give 5% damping at 3 and 10 Hz. The minimum damping of 4.3% occurs at 5 Hz. The damping at other frequencies is given by the damping vs. frequency curve in Figure 4-9.

Response data was processed for 30 selected nodal points in the model. These points are indicated in Figure 4-4 by large dots. For each of these points the horizontal and vertical acceleration histories and response spectra were computed. Representative examples are provided in Appendix A of AA (1975).

FIGURE 4-9. FREQUENCY-DEPENDENT DAMPING USED IN THE RESPONSE ANALYSIS

4.6 CONCLUSIONS

- The relatively good agreement between the input and free-field spectra (Figs. 4-7 and 4-8) demonstrates that the TRI/SAC code can be used to solve soil/structure interaction problems of this magnitude for a hard rock site.
- Application of the input motion at the side boundary of the soil island presented no analytical problems. For rock sites, this method of input is more realistic than applying the input at the base of the model.
- Thirty seconds of computing time was required for each integration step.
- More accurate results could have been obtained by using a time step smaller than 0.01 sec, but the improved

accuracy would not have justified the additional computing costs.

- e. In constructing the plane model it was assumed the buildings are supported by a foundation slice 336 ft thick. This is an important assumption which merits further study, since it affects the response of the structures.

- f. Study of the results indicates that for future analyses better agreement could be obtained between the input and free-field response spectra (Fig. 4-8) by providing a greater depth of rock below the structure, or by modifying the energy absorbing characteristics of the lower horizontal boundary.

CHAPTER 5

COMPARISON OF TWO- AND THREE-DIMENSIONAL ANALYSES

5.1 INTRODUCTION

It is important to determine how well two-dimensional analytical soil/structure interaction models can represent the three-dimensional response of symmetrical and unsymmetrical structures to earthquake ground motion. The results can be used to define criteria for soil/structure interaction analyses of future nuclear plant structures. To accomplish these objectives, the following analyses were made:

- Two-dimensional free-field analyses of a site
- Three-dimensional free-field analysis of a site
- Two-dimensional soil/structure interaction analyses with symmetrical structure
- Three-dimensional soil/structure interaction analysis with symmetrical structure
- Two-dimensional soil/structure interaction analyses with unsymmetrical structure
- Three-dimensional soil/structure interaction analysis with unsymmetrical structure

The soil/structure interaction analyses of embedded nuclear reactor structures are currently being performed using two-dimensional finite element models. Since these two-dimensional models cannot include torsional or out-of-plane response, discrete mass-spring models are used to estimate torsional response. However, the mass-spring models cannot treat the embedment effects and the interaction of closely clustered structures. Also, the overall response is computed using a square root of the sum of squares (SRSS) summation of the responses obtained from the individual two-dimensional analyses. The SRSS summation of the peak responses may result in an overly

conservative analysis. Therefore, an objective of the present study is to determine whether the savings in design costs resulting from a three-dimensional finite element analysis that considers embedment and utilizes three simultaneous components of input would more than offset the additional cost of this more sophisticated analysis.

5.2 BACKGROUND

The requirement for soil/structure interaction analyses is set forth in Code of Federal Regulations 10 CFR 100, Appendix A. This regulation requires that the engineering method used to ensure that required safety functions of nuclear plants are maintained during and after the occurrence of the design earthquake ground motions must take into account soil/structure interaction effects (CFR, 1974). Acceptable soil/structure interaction analysis procedures are set forth in Section 3.7.2 of the Standard Review Plan (SRP) of the U.S. Nuclear Regulatory Commission (USNRC, 1975). Either discrete mass-spring or finite element methods may be used for most site and shallow embedment conditions, but only the latter is currently accepted for deeply embedded structures in soils.

Section 3.7.2 of the SRP indicates that torsional, rocking, and translational response of nuclear power plant structures about all principal axes must be considered at the foundation, or support level, when soil/structure interaction effects are considered. However, only two-dimensional finite element analyses, which cannot consider torsional response about the vertical axis, are currently provided for deeply embedded structures on soil sites. The torsional, or out-of-plane, response is generally

estimated in two stages. Two-dimensional finite element soil/structure interaction models are first used to estimate the structure response in two orthogonal vertical planes, neglecting all torsional responses. The response of the structure at the ground surface obtained from these models is then used as input to a three-dimensional model of the aboveground structure, which is unconservative. The two-dimensional first-stage soil/structure interaction models also neglect torsional base moments that result from three-dimensional input motions, since the two horizontal components of motion are applied to separate models. The true directional characteristics of the input motion are, therefore, not used in the analyses.

Soil/structure interaction, even for a symmetrical structure, is a three-dimensional wave radiation problem. There is no verification that two-dimensional finite element soil/structure interaction models can adequately consider three-dimensional wave radiation relationships, although one computer code is assumed to have this capability. There is, therefore, the need to provide comparative two- and three-dimensional finite element soil/structure interaction analyses to verify that two-dimensional finite element models can adequately predict the overall seismic response of embedded unsymmetrical structures to the true three-dimensional characteristics of the input motion. In the event this cannot be done for all cases, the conditions for which three-dimensional soil/structure interaction models are required should be defined.

Soil/structure interaction analyses are currently based on elastic analyses. Finite element computer codes are capable of modeling the nonlinear behavior of soils in soil/structure interaction analyses. At the present time, the relative importance of nonlinear soil behavior on soil/structure interaction response is not clearly understood. Since important, massive nuclear plant structures are not ordinarily founded on

poor soil, the results of hypothetical analyses based on poor soil conditions can be misleading. Also, since the degree of nonlinearity in dynamic soil behavior is a function of the strain level (i.e., the intensity of ground motion), elastic analyses may be satisfactory in some regions and not in others, depending on the intensity of ground motion that must be considered. A series of comparative analyses using elastic and nonlinear codes is needed to establish if and when more complex nonlinear analyses are required.

It has been noted above that dynamic finite element soil/structure interaction analyses are required for deeply embedded nuclear plant structures on soil sites. Soil/structure interaction is a three-dimensional relationship even for symmetrical structures. While the capability exists to perform dynamic three-dimensional finite element soil/structure interaction analyses, the cost of three-dimensional analyses is many times greater than the cost of simpler two-dimensional analyses. Therefore, for economic reasons, it has become nearly standard practice to use several less costly two-dimensional soil/structure interaction models, rather than a three-dimensional model, to estimate the response of nuclear plant structures to earthquake ground motion. Several procedures are in common practice at this time.

One procedure is to provide separate, two-dimensional finite element soil/structure interaction analyses that represent idealizations of the structure along two orthogonal vertical planes. These models obviously cannot consider torsional response about a vertical axis, but do consider horizontal and vertical translation within the plane of each model as well as rotation about horizontal axes normal to each respective plane. Also missing from these models are interaction effects (i.e., energy radiation) normal to the plane of each two-dimensional model. One code (FLUSH) attempts

to represent these effects with dashpots (Lysmer et al., 1975), but this representation has significant limitations. A three-dimensional model of the aboveground structure is then used in this procedure to obtain the structure response. The input to this model is the response of the structure computed at the ground level from the two separate two-dimensional soil/structure interaction analyses. This approach neglects the torsional responses of the belowground structures that are difficult (if not impossible) to define.

A second procedure provides the two separate two-dimensional soil/structure interaction analyses described above, with torsional response about the vertical axis as estimated from separate discrete mass/spring structure models for each horizontal component of motion. These models usually neglect soil/structure interaction on the vertical exterior faces of the structure, but consider the torsional resistance of the soil on the assumed rigid base of the model. The overall floor response is then computed in each of three orthogonal directions by considering the contribution from each mode.

Other variations to the two procedures follow. For example, even though the Standard Review Plan indicates that finite element procedures should be used for deeply embedded structures on soil sites, it is not unusual for discrete mass/spring models to be used for all analyses, including the soil/structure interaction analyses. The procedures, based on simple models, are frequently justified on the basis that the capability does not exist to perform three-dimensional finite element soil/structure interaction analyses for complex structures. While this is true, the need still remains to determine how well current procedures represent the true three-dimensional response of the structure. This can be determined from comparative two- and three-dimensional finite element models of relatively

simple structural configurations that can be modeled by present codes. The results of these comparative analyses are needed to verify present procedures and to provide improved procedures, if needed.

5.3 FINITE ELEMENT MODELS

In order to investigate the nature of the errors introduced by analyzing soil/structure interaction problems using two-dimensional techniques, three cases are studied using the finite element program TRI/SAC. All of these cases involve the same site conditions and the same input base motion. The first case is that of free-field motion, the second involves the response of a symmetrical structure to ground shaking, and the third models the response of an unsymmetrical structure. In all of these cases, the problem is modeled in both two dimensions and three dimensions, and the results of the analyses are compared. These comparisons are expected to identify the deficiencies that would result from analyzing the problem in two dimensions.

All of the finite element models, both two dimensional and three dimensional, are described here, but first the soil properties, structure properties for each case, and input base motions are given.

5.3.1 SOIL PROPERTIES

The selected soil properties are modeled after the site properties at the strong motion recording station at 8244 Orion Avenue, Los Angeles, California. The strong motion station produced several useful records during the San Fernando Valley earthquake of February 9, 1971. This site was chosen for two principal reasons: first, the soil conditions at the site are such that the site could support a nuclear reactor plant, making it a representative site for nuclear plants; second, the site

has been extensively analyzed previously (see Chapt. 1), and consequently the required soil properties are readily available.

The soil properties used in the previous analysis of the Orion site originate from a subsurface model constructed by Duke et al. (1971). The portion of this model used in the analysis was obtained partially from a shallow seismic survey and partially from the application of a modified Faust formula (see Duke et al., 1971). The shallow seismic survey consists of generating P-waves and S-waves at the ground surface by striking a wooden plank with a 25-lb hammer, and recording the arrival of the wave fronts along a line of geophones, also on the ground surface. The tests carried out for the Orion site consisted only of shear waves, and the records were very noisy with wave arrivals being extremely difficult to measure or even nonexistent. The resulting shear wave velocities, therefore, are considered to be of low reliability.

The shear wave velocities determined from the shallow seismic survey, as well as those determined from the modified Faust formula, must be regarded as applying to extremely small strain levels. It is widely accepted, however, that soil shear moduli and damping characteristics vary with effective strain amplitude, and that strain amplitudes experienced during strong ground shaking are large enough to cause significant changes from the small-strain soil properties. The strain-amplitude-dependent shear moduli and damping properties assumed for the Orion site in the previous study are those of Schnabel et al. (1972) and are similar to Figures 5-6 and 5-7 of SW/AJA (1972) with the small strain shear moduli being those of the Duke subsurface model (Duke et al., 1971).

The TRI/SAC finite element computer code is a linear code and is not intended to consider strain-amplitude-dependent properties as input. It is necessary, therefore, to determine a priori

what average strain levels in the soil medium are induced during the response analysis and to use secant shear moduli and damping ratios compatible with those levels of shear strain. This was accomplished by using the SHAKE code (Schnabel et al., 1972) in an iterative mode. The SHAKE code is a one-dimensional wave propagation code in which the input base motion can be applied to the subsurface model, thereby simulating free-field motion. An initial estimate of strain level is made to determine initial values of shear modulus and damping ratio for each layer, and the response is calculated. The strain levels actually obtained in the model response are then used to re-estimate the strain-dependent soil properties and a subsequent iteration is performed. This can also be done in the deconvolution mode in which the input ground motion may be specified at any level (for example, at ground surface) and the base motion is calculated as output. Since the strong motion records obtained at the Orion site correspond to the ground surface, the SHAKE deconvolution analyses were necessary to obtain motions at different levels. The actual strong motion records obtained at the Orion site during the February 9, 1971 earthquake were used as the input for the ground surface motion for the SHAKE model. The final iteration provided the average soil properties and the input base motion to be used in the TRI/SAC two-dimensional and three-dimensional models.

The soil properties resulting from this iterative deconvolution process are shown in Table 5-1. The shear moduli and damping ratios shown in this table are substantially different from those initially estimated. Furthermore, the strain-amplitude-dependent soil properties were determined based on the Orion strong motion records. However, a different set of input motion records (Castaic) was selected for the TRI/SAC analyses (see Sec. 5.3.3 below). For these reasons, the

TABLE 5-1. SOIL PROPERTIES USED IN THE TRI/SAC FINITE ELEMENT CALCULATIONS

Depth, ft	Shear Modulus, psi	Unit Weight, pcf	Shear Wave Velocity, ft/sec	Poisson's Ratio	Damping, %
0-10	6944	105	554	0.30	5
10-20	9931	105	662	0.30	7
20-40	11632	105	716	0.30	8
40-70	12222	110	717	0.30	10
70-100	12639	110	730	0.30	11
100-135	18542	115	864	0.30	10

properties shown in Table 5-1 are not to be regarded as an accurate subsurface model of the Orion site. This does not defeat the purpose of the study, however. The objectives of the study are satisfied as long as the soil model used is typical of a site selected for a nuclear power plant. This model must then be consistently used for all of the three-dimensional and two-dimensional finite element analyses in order to identify differences due to the geometry of the model. The properties shown in Table 5-1 are therefore used in all the analyses in this study.

The vertical layering of all of the TRI/SAC finite element models employed in this study, both two dimensional and three dimensional, adheres to the divisions shown in the left column of Table 5-1. The shear modulus, shear wave velocity, and damping parameter are all values taken to be applicable to the average strain levels expected to be encountered in the analyses. In this respect, the TRI/SAC solutions can be regarded as linear solutions to materially nonlinear problems. The values of unit weight and Poisson's ratio are also provided for each layer to complete the material characterization.

The TRI/SAC finite element code provides for only Rayleigh damping (Bathe and Wilson, 1976) for the linear step-by-step analysis option

used in this study. This damping model allows specification of overall system damping for a specified range of frequencies, but the specification of distinct damping ratios for distinct layers is not possible. The global damping matrix $[C]$ for Rayleigh damping is expressed in terms of the global mass matrix $[M]$ and the global stiffness matrix $[K]$ according to the relationship

$$[C] = \alpha[M] + \beta[K] \quad (5-1)$$

where α and β are coefficients to be selected by the user. In terms of these coefficients, the damping ratio for a particular vibration mode of the overall finite element model can be expressed as

$$\lambda_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \quad (5-2)$$

where ω_n is the natural circular frequency of the n th vibration mode and λ_n is the damping ratio for that mode. The values of α and β used in all of the TRI/SAC finite element models in this study are $\alpha = 1.1321$ and $\beta = 3.1540 \times 10^{-3}$ and the resulting frequency-dependent damping ratio is plotted in Figure 5-1. This

FIGURE 5-1. FREQUENCY-DEPENDENT DAMPING USED IN ALL TRI/SAC ANALYSES FOR ORION SITE

relationship was chosen to have a minimum damping ratio of about 6% for a frequency of 3 Hz in that the peak values of the response spectra of the input motions selected for these analyses occur at about 3 Hz, and 6% appears to be a representative value of the damping ratio for the upper portion of the soil profile

(see Table 5-1). While the damping values of Figure 5-1 do not model the assumed soil profile perfectly, they appear to be the best damping values available considering the limitations of the TRI/SAC code in specifying damping values for the analysis option used in this study.

5.3.2 REPRESENTATION OF SURFACE STRUCTURES

Three distinct cases are considered in order to define the conditions for which a two-dimensional analysis is acceptable: (1) free-field soil response, (2) soil/structure interaction of a symmetrical structure, and (3) soil/structure interaction of an unsymmetrical structure. In the first, no structure is present. For the other two cases, in which the structures are included, the difference between the two- and three-dimensional analyses will involve merely the effects of applying the N/S horizontal motion and the E/W horizontal motion separately to two-dimensional models and simultaneously to the three-dimensional model.

The soil/structure interaction analysis is not intended to compute the detailed motions within the structure, which are ordinarily determined in a subsequent run in which only the structure is modeled in detail and the motion determined from the soil/structure interaction analysis is used as input. The only properties of the structure required for the present analysis, therefore, are those that control its effect on the soil motion, such as its overall mass and stiffness properties. The effective stiffness of the structure greatly exceeds that of the soil, and hence the precise modeling of its stiffness properties is relatively unimportant. In view of these considerations, the structures were modeled using brick or quadrilateral continuum elements having density and stiffness properties that have been chosen to match the corresponding effective properties of the structure.

5.3.2.1 Symmetrical Structure

The response of any nuclear power plant structure to vibratory ground motion is three dimensional. Even if the structure were to have a plane of symmetry, and even if all of the motion of the incident wave train were to be in that plane of symmetry, portions of the reflected wave train (outgoing vibratory energy) would be oblique to the plane of symmetry. Meanwhile, reflected vibratory energy radiating away from adjacent locations of free-surface response would be continuously passing through the plane of symmetry. Neither of these latter two phenomena is taken into account in a two-dimensional analysis.

In the case of a symmetrical structure for which the incident wave motion is in the plane of symmetry, the only response of the structure is in the plane of symmetry. The response of the structure will consist only of vertical translations, horizontal translations within the plane of symmetry, and rotations within the plane of symmetry. The incident wave train actually consists of components within the plane of symmetry and components normal to it. If the structure has a second vertical plane of symmetry at right angles to the first, then the response to the components of the input motion normal to the first plane of symmetry will lie entirely in the second. After superposing these two responses it can be seen that the response to any three-dimensional combination of input motions will contain no twisting about the vertical axis if the structure has two orthogonal vertical planes of symmetry. By contrast, the response of an unsymmetrical structure will consist of six degrees of freedom including the twisting moment about the vertical axis. If the response of the structure is to be constructed by superposition of two orthogonal two-dimensional analyses, it is seen that all of the degrees of freedom essential to the analysis are present for the symmetrical case while the

twisting about the vertical axis is not present for the two-dimensional analysis of the unsymmetrical case. With this distinction in mind, the term "symmetrical structure" is understood to refer to a structure having two orthogonal vertical planes of symmetry.

Finite element models of the symmetrical structure used in this soil/structure interaction study are shown in Figure 5-2. In the three-dimensional model, the vertical planes containing

(a) Three-dimensional model

(b) Two-dimensional model

FIGURE 5-2. FINITE ELEMENT MODELS OF SYMMETRICAL STRUCTURE

the centers of opposite faces are planes of symmetry forming an orthogonal pair. Thus, this structure satisfies the above stated definition of symmetrical structure. The structure has a

square cross section 40 ft on a side, and is 60 ft high with 40 ft above the ground surface and 20 ft below. The material properties are designated by circled numbers in the figure, which correspond to material parameters in Table 5-2. Note that the continuum properties with which this structure is modeled become denser and stiffer toward the bottom of the structure. This simulates thicker concrete walls and slabs toward the bottom of the structure.

TABLE 5-2. MATERIAL PROPERTIES USED FOR THE SYMMETRICAL AND UNSYMMETRICAL FINITE ELEMENT MODELS

Material Number	Young's Modulus, psi	Poisson's Ratio	Shear Modulus, psi	Mass Density lb-sec ² /in ⁴
1	1 325 x 10 ⁶	0.2	0 552 x 10 ⁶	0 730 x 10 ⁻⁴
2	2 650 x 10 ⁶	0.2	1 104 x 10 ⁶	1 460 x 10 ⁻⁴
3	3.650 x 10 ⁶	0.2	1 521 x 10 ⁶	1 505 x 10 ⁻⁴
4	4 300 x 10 ⁶	0.2	1 792 x 10 ⁶	1 932 x 10 ⁻⁴
5	1 834 x 10 ⁶	0.2	0 764 x 10 ⁶	1 539 x 10 ⁻⁴
6	2 163 x 10 ⁶	0.2	0 901 x 10 ⁶	1 753 x 10 ⁻⁴

The two-dimensional model shown in Figure 5-2b is used to simulate the structure in both the N/S direction and the E/W direction. Different ground motion input is used for these two cases. The materials used in this two-dimensional model are the same as those used in the three-dimensional model. Since the model is a plane strain model, the structure is assumed to have an infinite dimension in the out-of-plane direction. In general, the procedure used in the preparation of the two-dimensional model is that in existence in the industry for this type of analysis.

5.3.2.2 Unsymmetrical Structure

Finite element models of the unsymmetrical structure used in this soil/structure interaction study are shown in Figure 5-3. It is easily seen that the three-dimensional finite element

(a) Three-dimensional model

(b) N-S two-dimensional model

(c) E-W two-dimensional model

FIGURE 5-3. FINITE ELEMENT MODELS OF UNSYMMETRICAL STRUCTURE

model of the structure contains no planes of symmetry. Note the compass directions included in the figure to define the N-S direction and the E-W direction. The material properties used for the continuum elements making up the model are the same as those used for the symmetrical model (see Table 5-2).

Figure 5-3b shows the two-dimensional finite element model of the unsymmetrical structure for the N-S direction viewed from the west. The quadrilateral elements making up the two-dimensional model have the same dimensions as the blocks making up the three-dimensional model. This was also true in the case of the symmetrical model, in which each quadrilateral element corresponded to a two-block-thick

region of the three-dimensional model. In the symmetrical case, each quadrilateral element was assigned the same material properties as the corresponding region of the three-dimensional model. In the unsymmetrical case, some of the quadrilateral elements correspond to two-block-thick regions of the three-dimensional model. In such cases, these quadrilaterals are assigned the same properties as the corresponding regions of the three-dimensional model. The quadrilaterals in Figure 5-3b for which the material designations are 2, 3, or 4 are elements of this type. The remaining elements above the ground surface correspond to one-block-thick regions of the three-dimensional model. Consequently they have been assigned

material parameters for which the stiffness and density values are one-half the corresponding values of the three-dimensional model. Material Numbers 5 and 6 are assigned to elements that correspond to one-block-thick regions of structure in addition to one-block-thick regions of soil. Accordingly these materials are taken as averages of the corresponding structural properties from Table 5-2 and the corresponding soil properties from Table 5-1.

The two-dimensional model of the unsymmetrical structure in the E-W direction as viewed from the south is shown in Figure 5-3c. Note that when viewed from this direction, the outline is no longer rectangular. As in the case of the N-S model, those quadrilateral elements that correspond to two-block-thick regions of the three-dimensional model have been assigned the same properties as the three-dimensional model. Those quadrilaterals (in this case there is only one) corresponding to a one-block-thick region are given properties of half the corresponding stiffness and density. Those quadrilaterals that correspond to a combination of structure and soil are assigned material Properties 5 and 6 as described above.

5.3.3 INPUT GROUND MOTIONS

The selected earthquake ground motion records used as input were the three components of ground motion recorded at Castaic, California, during the February 9, 1971 San Fernando earthquake. These records were chosen primarily because they develop strong ground shaking of relatively short duration, thereby allowing more economical computer analysis than a longer-duration record would. The three components of motion along with their response spectra for 5% damping are shown in Figure 5-4. The acceleration record of the N21°E instrument is the N-S record; the acceleration record of the N69°W instrument is the

E-W record. Also shown on the response spectra plots of Figure 5-4 are the Regulatory Guide 1.60 spectra (USAEC, 1973). In the 4 to 30 Hz range, the Castaic spectra generally approximate these Regulatory Guide 1.60 spectra.

5.3.4 FINITE ELEMENT MODELS

The three-dimensional finite element model used for the free-field analysis is shown in Figure 5-5, while the corresponding two-dimensional model is shown in Figure 5-6. In both models the vertical zoning corresponds to that of the left column of Table 5-1; and hence the six materials specified in this table correspond to the six layers spanning the vertical dimension of both models. The horizontal zoning in both directions of the three-dimensional model is the same, and corresponds exactly to the horizontal zoning of the two-dimensional model. Thus, geometrically, the two-dimensional model serves to represent either the N-S direction or the E-W direction.

Figures 5-7 and 5-8 show, respectively, the three-dimensional and two-dimensional finite element models for the case of the symmetrical structure. The zoning in the soil portions of these two models is the same as that for the free-field models, and the soil materials are the same. The difference is the presence of the structure. Due to symmetry of the structure, the same two-dimensional model is suitable for both the N-S analysis and the E-W analysis.

Figure 5-9 shows the three-dimensional finite element model for the case with the unsymmetrical structure, Figure 5-10 shows the corresponding two-dimensional model for the N-S direction, and Figure 5-11 shows the two-dimensional model for the E-W direction. Except for the structure, all details of these models are the same as for the free-field analysis and for the case with the symmetrical structure.

(a) N-S acceleration

(b) N-S response spectra

(c) E-W acceleration

(d) E-W response spectra

(e) Vertical acceleration

(f) Vertical response spectra

FIGURE 5-4. CASTAIC, CALIFORNIA, GROUND MOTION RECORDS AND RESPONSE SPECTRA FOR 5% DAMPING (February 9, 1971)

FIGURE 5-5. FINITE ELEMENT MODEL FOR THREE-DIMENSIONAL FREE-FIELD ANALYSIS

FIGURE 5-6. TRI/SAC FREE-FIELD FINITE ELEMENT MODEL OF ORION SITE

FIGURE 5-7. FINITE ELEMENT MODEL FOR THREE-DIMENSIONAL ANALYSIS WITH SYMMETRICAL STRUCTURE

FIGURE 5-8. TWO-DIMENSIONAL FINITE ELEMENT MODEL WITH SYMMETRICAL STRUCTURE

FIGURE 5-9. FINITE ELEMENT MODEL FOR THREE-DIMENSIONAL ANALYSIS WITH UNSYMMETRICAL STRUCTURE

FIGURE 5-10. TWO-DIMENSIONAL ELEMENT MODEL WITH UNSYMMETRICAL STRUCTURE FOR N-S DIRECTION

FIGURE 5-11. TWO-DIMENSIONAL FINITE ELEMENT MODEL WITH UNSYMMETRICAL STRUCTURE FOR E-W DIRECTION

All of these models, whether two dimensional or three dimensional, are excited by the Castaic ground motion records applied uniformly across the base of the models. For the three-dimensional models, the base is driven in the E-W direction by the E-W component of the Castaic ground motion record, in the N-S direction by the N-S Castaic component, and in the vertical direction by the vertical component of the Castaic acceleration record. For the N-S two-dimensional models, the base is driven in the horizontal direction by the N-S component of the Castaic ground motion record and in the vertical direction by the vertical component. Similarly for the E-W two-dimensional models, the base is driven in the horizontal direction by the E-W component of the Castaic ground motion record and in the vertical direction by the vertical component. Some consideration was given to using half the vertical component as input for the two-dimensional models. If this were to be done, the results from a N-S two-dimensional model and an E-W two-dimensional model could be simply superposed to create the correct combination of input motions. However, the possibility of a direct comparison between a

two-dimensional model and a three-dimensional model would have been sacrificed by so doing. In view of this, it was decided to use the full vertical component in the two-dimensional models.

All vertical boundaries of all finite element models, both two-dimensional and three-dimensional, are energy absorbing boundaries (AA, 1976c). These boundaries consist of dampers applied in all coordinate directions to each node of the boundary. The damping coefficients of these dampers are chosen to minimize the reflection back into the model of outward bound waves of vibratory energy. The purpose of this boundary type is to make the model dimension appear infinite even though it is finite.

The responses of all finite element models were computed using the TRI/SAC code with a time increment of 0.02 sec. The three-dimensional models were run for 200 steps for a total duration of 4.0 sec, while the two-dimensional models were run for 255 steps for a total duration of 5.1 sec. Inspection of Figure 5-4 reveals that for both two-dimensional and three-dimensional cases, the duration of the

calculation exceeds the duration of strong ground motion by a long enough time interval to allow shear waves to travel from the base to the surface of the model. By dividing the height of each layer by its shear wave velocity given in Table 5-1, it is seen that this travel time for shear waves is about 0.185 sec.

All the three-dimensional calculations were performed on a CDC Cyber 176 computing system, while all the two-dimensional calculations were performed on a Univac 1108 computer. The three-dimensional calculations required on the average 20 sec of central processor time per step of the solution. On the basis of experience with computational capabilities of Cyber 176 and Univac 1108 computers, the two-dimensional calculations are estimated to require an equivalent of 0.2 sec central processor time of Cyber 176 per solution step. Thus, the three-dimensional calculation is about 100 times more expensive than the corresponding two-dimensional calculation as far as computing cost is concerned.

5.4 COMPARISON OF RESULTS

The results of the three-dimensional and two-dimensional finite element models are compared in order to determine the effects of the various modeling techniques on the results of soil/structure interaction analyses. It was previously stated that the purpose of a soil/structure interaction analysis is to determine the motions to which a structure is subjected so that these motions can be applied to a more refined structural model. In view of this, the response spectra of motions plotted against frequency in a log/log format were chosen as the most effective means of comparing the two-dimensional and three-dimensional results. The response spectrum is the response of a damped, one-degree-of-freedom oscillator to a particular motion record (Hudson, 1979). It is obtained as a function of frequency by holding the

damping ratio of the one-degree-of-freedom oscillator constant and varying its natural frequency. The response spectrum of pseudoveLOCITY is found by determining the peak displacement for a given frequency and multiplying it by the frequency expressed in radians per second. Similarly, the pseudoacceleration can be determined by multiplying the pseudoveLOCITY by the frequency. When the pseudoveLOCITY is plotted against frequency on a log/log scale, the displacement and pseudoacceleration spectra are available along diagonal axes of the same plot. Thus, information pertaining to peak structural response, in terms of displacement, velocity, and acceleration, for a range of frequencies is available from a single curve. This curve is of great significance in the design of structures, and therefore, the pseudoveLOCITY response spectrum has been chosen as the means of comparison of the soil/structure interaction analyses.

5.4.1 FREE-FIELD COMPARISONS

An extensive set of comparisons of two-dimensional and three-dimensional free-field analyses is provided by AA (App. A, AA, 1981a). These comparisons are in the form of log/log response spectra, all obtained using a damping ratio of 5.0%. Figure 5-12 shows a sample of the response spectra comparisons contained in AA (1981a). The point for which the comparisons were obtained is located by means of the three views of the three-dimensional finite element model (see large dot in figure). Note that the two-dimensional finite element models correspond to either the E-W or N-S vertical sections, depending on whether the E-W or N-S component of ground motion is used as input. In this case, the point is located on the ground surface in the center of the three-dimensional model. The point has a counterpart in both the E-W and N-S two-dimensional models as shown in the vertical sections.

FIGURE 5-12. COMPARISON OF RESPONSE SPECTRA AT GROUND SURFACE, ON CENTERLINE, FOR FREE-FIELD ANALYSES

The comparisons of response spectra of the three components are shown in Figure 5-12. For frequencies below about 2 Hz, the agreement between the three-dimensional and two-dimensional spectra is excellent except for very low frequencies. At very low frequencies, in the range of 0.25 Hz and lower, there are often large differences between two-dimensional and three-dimensional spectra (see Fig. 5-12b). At first this phenomenon was attributed to the differences in the durations of the calculations. The three-dimensional analysis covered a time span of 4.0 sec (using as input the first 4.0 sec of each record in Fig. 5-4), while the two-dimensional calculation had a duration of 5.1 sec. However, a comparison of response

spectra of input motions (Fig. 5-4) of durations 4.0 sec and 5.1 sec indicated no appreciable differences in the response spectra.

For frequencies above about 2.0 Hz, Figure 5-12 indicates that for all three components of motion the response of the three-dimensional calculation exceeds that of the two-dimensional calculation. This observation applies to virtually all of the horizontal components studied, and to those vertical components located along the grid centerline. For other locations, the two-dimensional response sometimes exceeds the three-dimensional response for these frequencies. In general, the agreement is usually very good for frequencies below about 2.0 Hz, and poorer for higher frequencies. In the higher

frequency range, the three-dimensional response often exceeds the two-dimensional response, but sometimes this trend is reversed and in some cases the agreement remains good throughout the frequency spectrum.

In order to appreciate what is implied by the response spectra comparisons of Figure 5-12, Figure 5-13 shows similar comparisons of the acceleration time histories of the same three components of the same node. These comparisons show excellent agreement in terms of phase, but there are considerable differences between peak amplitudes for the two analyses. In general, the zero-crossings of the acceleration histories compare very well, but

the peak values of the three-dimensional response vary from 0 to 50% greater than the peak values of the two-dimensional response. In that the positive and negative differences between the acceleration histories tend to balance out, differences in integrals of the acceleration over finite time intervals will be appreciable only for time intervals short enough to contain a poor balance of positive and negative peaks. Since differences in acceleration "impulse" are significant only for short duration integrals, it follows that there should be differences in pseudovelocity response only for the high frequency portion of the response spectrum. This correlates well

FIGURE 5-13. COMPARISON OF ACCELERATION HISTORIES AT GROUND SURFACE, ON CENTERLINE, FOR FREE-FIELD ANALYSIS

with the observations made regarding Figure 5-12. While the differences in pseudoveLOCITY response shown in Figure 5-12 do not appear very large, Figure 5-13 shows that the corresponding differences in peak acceleration are substantial, amounting to as much as a 50% difference in favor of the three-dimensional response.

Since the same element sizes, boundary locations, and time increments were used for both the two-dimensional and three-dimensional calculations, it was expected that the results would be about the same for the two-dimensional and three-dimensional models. There are only two features of these finite element models whereby significant differences between the two-dimensional and the three-dimensional models exist. It is assumed that the differences in results must be attributable to at least one of these two features.

The first of these differences between the two-dimensional and three-dimensional analyses is that the three-dimensional analyses contain the out-of-plane component of horizontal motion while the two-dimensional analyses do not. For example, the N-S two-dimensional free-field model is excited by only the vertical and the N-S components of the Castaic ground motion records, whereas the three-dimensional model contains also the E-W component. It was expected that the omission of the out-of-plane component for the two-dimensional analyses would have no effect on the results since for free-field conditions, the equations of motion are uncoupled with respect to the three displacement components. The fact that the displacement components are, indeed, uncoupled is evident from the fact that the N-S and the E-W two-dimensional analyses produced exactly the same vertical response along the mesh centerline even though the horizontal input motions for the two cases were greatly different. This

mutual independence for the horizontal and vertical response components is not preserved as one moves away from the mesh centerline. In other words, a particular node of the two-dimensional models has the same vertical response for the N-S and E-W horizontal input motions if the node is on the mesh centerline, and different vertical responses if it is not. But this indicates that the response varies from point to point in the model even while remaining at the same elevation. This should not happen for a free-field analysis, and it indicates that boundary effects influence the response throughout the model. Since there is no coupling on the mesh centerline, the displacement component coupling is not an important factor in the comparisons. If displacement component coupling were present, it would have been virtually impossible to obtain the same vertical motions for different horizontal motions anywhere in the model.

The second of the two differences between the two-dimensional and three-dimensional finite element models involves the boundary conditions. As described in Section 5.3.4, the vertical boundaries consist of dampers intended to minimize the reflection of outward bound waves back into the model. This type of boundary is known as an energy-absorbing boundary, and it attempts to make the model behave as if there were no boundary. The fact that the responses for both the two-dimensional and three-dimensional analyses differ among the points at the same elevation indicates that the energy-absorbing boundaries were not effective for these types of analyses. Rather than responding as if no boundaries were present, the models responded as if a reflecting boundary were present within a finite distance of the model centerline. In the two-dimensional models, these boundaries exist in the in-plane horizontal direction while the model is of infinite extent in the out-of-plane direction. In the

three-dimensional model, these boundaries are present in both horizontal directions.

It is of interest to investigate how well the two-dimensional and three-dimensional responses compare with each other and with the actual free-field response of the soil medium. In order to determine the actual free-field response, the one-dimensional finite element models shown in Figure 5-14 were used. Since in the free field every column of soil is undergoing exactly the same motion as every other column of soil, induced lateral stresses are resisted by equal lateral stresses in adjacent soil columns. This results in no horizontal motion of any point in the soil column in response to the vertical base motion. Furthermore, since all adjacent soil columns are undergoing the same motion as any particular column, there is no resistance to vertical response due to the influence of neighboring soil columns. Thus, for vertical base motion, this model simulates a soil deposit of infinite lateral extent and therefore predicts free-field response at different elevations in the medium.

FIGURE 5-14. ONE-DIMENSIONAL FINITE ELEMENT MODELS FOR FREE-FIELD ANALYSIS

The model of Figure 5-14b is used for horizontal base motion. In the free field, every column of soil is undergoing exactly the same motion as any other column, and hence there is no resistance to horizontal motion induced by neighboring columns. Any vertical motion experienced by a point in the soil medium would have to be experienced by every other point at the same level in the soil medium. This would result in net dilatational strains in the soil medium. Since there are no normal stresses associated with horizontal base motion, these dilatational strains would have to come about through the mechanism of shear-induced dilatation. Therefore, the free-field model for horizontal motions should be such that no difference in vertical position is permitted between nodes at the same elevation. Since the constitutive models of all soil materials used in these analyses are elastic (see Table 5-1), no shear-induced dilatation can occur for these materials. Therefore, the vertical displacement of the soil column must be zero. Thus, this finite element model simulates a soil deposit of infinite lateral extent and is therefore suitable for free-field analysis.

Comparisons of the response spectra of the one-, two-, and three-dimensional finite element models are shown in Figures 5-15, 5-16, and 5-17. The one-dimensional results are taken as the actual free-field response. These figures show comparisons of all three components of motion at three different depths: ground surface, 20 ft depth, and 70 ft depth. These comparisons show that the response spectra of the two-dimensional and three-dimensional finite element models differ from the actual free-field response spectra by as much as they differ from each other.

The reason these comparisons show such differences among the three free-field analyses is that the two-dimensional and three-dimensional finite element models were unable to

FIGURE 5-15. COMPARISON OF FINITE ELEMENT RESULTS WITH ONE-DIMENSIONAL FREE-FIELD RESPONSE SPECTRUM AT GROUND SURFACE

FIGURE 5-16. COMPARISON OF FINITE ELEMENT RESULTS WITH ONE-DIMENSIONAL FREE-FIELD RESPONSE SPECTRUM AT 20-FOOT DEPTH

FIGURE 5-17. COMPARISON OF FINITE ELEMENT RESULTS WITH ONE-DIMENSIONAL FREE-FIELD RESPONSE SPECTRUM AT 70-FOOT DEPTH

accurately simulate soil deposits of infinite lateral extent. Such a simulation was attempted using the viscous boundary technique, the energy-absorbing boundary option available in the TRI/SAC code. The viscous boundary was shown to be effective in reducing or eliminating reflected wave amplitudes when the source of the propagating waves is contained entirely within the finite element model (Lysmer and Kuhlemeyer, 1969). For such a situation, the concern is only with waves propagating from the interior of the region to the exterior, and their continued propagation through the model boundary is effectively simulated (as far as the model is concerned) by absorbing the vibratory energy through the use of dampers.

The situation under investigation here, however, is different from that described above. The source of vibration is a layer of bedrock of infinite lateral extent, and it is not entirely contained within the model. Under

these conditions there are waves present in the soil deposit propagating from the exterior of the model to the interior as well as from the interior to the exterior. The viscous boundary is able to prevent outward-bound waves from reflecting back into the model, but inward-bound waves are omitted from the finite element models. One method of implementing the viscous boundary for this type of problem may be the approach incorporated into program FLUSH (Lysmer et al., 1975). In this approach the forces applied to the nodes of the viscous boundary are proportional to $\{\dot{u}\} - \{\dot{u}\}_f$, where $\{\dot{u}\}$ is a vector of velocity components for nodes on the boundary and $\{\dot{u}\}_f$ is the corresponding vector of free-field velocity components, which can be determined from finite element models such as those of Figure 5-14. Since the exteriors of the finite element models conform to free-field conditions, any difference in boundary motion from that of the free field must be the result of an outward-propagating wave. By

damping out only that portion of the boundary motion corresponding to the difference from free-field motion, the reflection of the remaining portion back into the model simulates the effects of inward-propagating waves.

The ability to subtract out the free-field velocities from the boundary velocities in applying the viscous boundary technique is not currently available in TRI/SAC and other general finite element codes. However, the analyses performed under this investigation are accurate examples of typical engineering applications of the general finite element codes. Because of strong boundary effects, it is difficult to tell whether differences in the calculations involving structures are due to differences in two-dimensional and three-dimensional methods of modeling the structures themselves, or merely to differences in the two-dimensional and three-dimensional boundary conditions. These effects can be separated by comparing the responses of two-dimensional and three-dimensional models with structures to those of the free-field calculations. That portion of the difference that remains constant between models with structures and free-field models is due to boundary effects, while that portion that appears only in the calculations with structures is due to the modeling of the structure.

5.4.2 ANALYSIS OF SYMMETRICAL STRUCTURE

An extensive set of comparisons of response spectra from two-dimensional and three-dimensional soil/structure interaction analyses involving a symmetrical structure are presented by AA (App. B, AA, 1981a). A sample figure from this set of comparisons is shown in Figure 5-18. This figure shows comparisons of response spectra for two-dimensional and three-dimensional SSI analyses corresponding to the location shown in Figure 5-18a. The

response spectra comparisons of parts b, c, and d of this figure show many of the same characteristics of the free-field comparisons. In particular there is very good agreement between the three-dimensional and two-dimensional response spectra for frequencies below about 2.0 Hz. This is not true for very low frequencies for which the period of vibration is of the same order as the duration of the finite element analyses. For frequencies above 2.0 Hz, the response spectra corresponding to two- and three-dimensional analyses are significantly different. These differences are also seen in the response spectra comparisons at other locations. For the frequency range above 2.0 Hz, for which the agreement is not good, the response of the three-dimensional model exceeds that of the two-dimensional models over the entire frequency range for virtually all of the horizontal components and many of the vertical components. For some of the vertical components the two-dimensional response is greater and in some cases the agreement is good. These phenomena are exactly the same as those observed for the free-field response spectra comparisons.

It is expected that some of the difference between a three-dimensional response spectrum and the corresponding two-dimensional response spectrum is attributable to differences in three-dimensional and two-dimensional modeling of the structure. However, there are also differences in the modeling of boundary conditions. To determine what portion of the differences shown in Figure 5-18 is attributable to differences in modeling the structure, this figure must be compared to Figure 5-19, which contains the free-field comparisons corresponding to the same location as Figure 5-18. For the two horizontal components, the comparison of the models with the structure shows a greater difference in response in the frequency range from 2 to 5 Hz. For the vertical component,

FIGURE 5-18. COMPARISON OF RESPONSE SPECTRA AT BASE OF SYMMETRICAL STRUCTURE, ON CENTERLINE, 20 FEET BELOW GROUND LEVEL

FIGURE 5-19. COMPARISON OF RESPONSE SPECTRA AT 20-FOOT DEPTH, ON CENTERLINE, FOR FREE-FIELD ANALYSIS

the free-field comparison shows a larger difference over a portion of the same frequency range. For other frequencies the two figures are essentially the same.

By comparing response spectra with the structure to those without for analyses of the same dimension, boundary effects seen in the comparisons may be minimized. For example, the two-dimensional analyses with the symmetrical structure and the two-dimensional free-field analyses have the same boundary conditions. Figure 5-20 shows such a comparison for the three-dimensional analyses, while Figure 5-21 shows the corresponding comparisons for the two-dimensional analyses. It can be seen that for the three-dimensional models there is very little change in response due to the presence of the structure. Where such differences are present, the free-field response is typically larger than the response with the structure present. Figure 5-21 looks very much like Figure 5-20 except that it contains some larger differences in the 2 to 5 Hz frequency range. It can be inferred from these comparisons that the presence of the structure exerts a greater effect on the resulting motions in two-dimensional modeling than in three-dimensional modeling. This may be because the structure is modeled using its full mass density, which is the same way a structure of infinite out-of-plane extent would be modeled. Thus, in general, the two-dimensional model represents a structure that is stiffer and heavier than it actually is.

Figures 5-20 and 5-21 show that the presence of a symmetrical structure has little effect on the response of the soil mass, whether modeled in three dimensions or in two. The differences between these effects in two dimensions and in three reveal the errors introduced by two-dimensional modeling. These differences are generally small. For the example problem, the results from the two- and three-dimensional

analyses with a symmetrical structure indicate that two-dimensional models are acceptable for soil/structure interaction analyses.

5.4.3 ANALYSIS OF UNSYMMETRICAL STRUCTURE

An extensive set of comparisons of response spectra from two-dimensional and three-dimensional soil/structure interaction analyses involving an unsymmetrical structure is presented by AA (App. C, AA, 1981). Comparison of these figures with those pertaining to the symmetrical structure, indicate that the asymmetry of the structure does very little to alter the differences between three-dimensional and two-dimensional soil/structure interaction models. The differences that exist are primarily the same as those already discussed. Figure 5-22 shows comparisons of response spectra obtained for three-dimensional models involving the symmetrical and unsymmetrical structures. Figure 5-23 shows similar comparisons for two-dimensional soil/structure interaction analyses. These figures indicate that two-dimensional models are acceptable for soil/structure interaction analyses with unsymmetrical structures. However, this conclusion is based on the analyses performed on only one type of structure and soil medium.

5.5 CONCLUSIONS

In performing the soil/structure interaction analyses, it was observed that the two-dimensional and three-dimensional finite element models were unable to accurately simulate free-field conditions. This was because the energy-absorbing boundary was not completely effective. Since the source of excitation consists of a plane of bedrock of infinite lateral extent, free-field response consists of waves propagating both into and out of the region represented by the finite element model. The application of the energy absorbing boundary

FIGURE 5-20. COMPARISON OF RESPONSE SPECTRA FOR THREE-DIMENSIONAL ANALYSES WITH AND WITHOUT SYMMETRICAL STRUCTURE

FIGURE 5-21. COMPARISON OF RESPONSE SPECTRA FOR TWO-DIMENSIONAL ANALYSES WITH AND WITHOUT SYMMETRICAL STRUCTURE

FIGURE 5-22. COMPARISON OF RESPONSE SPECTRA FOR THREE-DIMENSIONAL ANALYSES WITH SYMMETRICAL AND UNSYMMETRICAL STRUCTURES

FIGURE 5-23. COMPARISON OF RESPONSE SPECTRA FOR TWO-DIMENSIONAL ANALYSES WITH SYMMETRICAL AND UNSYMMETRICAL STRUCTURES

employed herein was successful in eliminating reflections from waves propagating outward from the interior of the model, but failed to take into consideration waves propagating inward from the exterior of the model. An improved energy absorbing boundary might result if the damping forces are applied only to that portion of the boundary motion that differs from free-field response.

The inability of the boundary conditions to properly cause the model to behave as if it were part of a medium of infinite lateral extent introduced some uncertainties into the comparisons of three-dimensional and two-dimensional modeling procedures. The two-dimensional finite element models assumed plane strain conditions, and so the resulting response was subjected to the influence of boundaries only in the planes of the analyses. There were no boundaries present in the out-of-plane direction. In the three-dimensional models, boundaries were present in all directions. Therefore, because of the inability of the energy-absorbing boundary to simulate the absence of a boundary, corresponding two-dimensional and three-dimensional models were not actually modeling the same set of field conditions. Due to this, a certain amount of discrepancy was inherently built into any comparison of two-dimensional and three-dimensional results. Any effects caused by approximations in modeling the structure in two dimensions were also present, but the fundamental difference in modeling the soil medium always had to be taken into account when comparing two-dimensional and three-dimensional results. This difficulty was overcome by finding some meaningful ways of

comparing two-dimensional results with other two-dimensional results and similarly for three-dimensional results. By doing this, boundary effects were minimized in the comparisons.

The results of the various finite element calculations undertaken during this study indicate that the presence of a structure did not greatly affect the response of the soil medium. This finding applies more to three-dimensional modeling (see Fig. 5-20) than to two-dimensional modeling (see Fig. 5-21). These figures show, in three dimensions and two dimensions, respectively, that the response spectra change very little from those of the free-field analyses when a symmetrical structure is included in the model. Furthermore, it can be seen from Figures 5-22 and 5-23 that altering the structure into an unsymmetrical one does almost nothing to change the response of the soil medium. These findings reflect the fact that the site conditions chosen for this investigation are suitable to support a nuclear power plant. On softer soils the presence of a structure would be expected to exert greater influence on the response of the soil medium. But for the relatively stiff soil conditions modeled herein, the effects of the structure on the soil are not very significant.

On the basis of the soil/structure interaction analyses performed under this study, it appears that two-dimensional models are generally acceptable. This conclusion applies to both symmetrical and unsymmetrical structures. Additional analyses with different sites and different types of structures will help confirm these conclusions.

CHAPTER 6

COMPARISON OF ANALYSES USING FLUSH AND TRI/SAC CODES

6.1 INTRODUCTION

The study reported in Chapter 5 involved two- and three-dimensional, linearly elastic finite element soil/structure interaction analyses involving a symmetrical structure, an unsymmetrical structure, and the free field. These analyses investigated the effects of two-dimensional approximations to the three-dimensional soil/structure system. The study is extended here to employ the FLUSH finite element program to investigate the effects of strain-dependent soil stiffness, strain-dependent damping properties that can be individually assigned to each element of the model, and a transmitting boundary that is based on closed-form solutions of a layered half-space (Waas and Lysmer, 1972).

The finite element models for FLUSH use the same symmetrical and unsymmetrical soil/structure systems as those used for TRI/SAC. The zoning of these models is designed to meet the requirements of the FLUSH code, but wherever possible, nodes have been located at the same coordinates as in the TRI/SAC models in order to facilitate comparisons between the two sets of analyses. The FLUSH analyses consisted of both N-S and E-W soil/structure interaction analyses of the symmetrical structure and N-S and E-W analyses of the unsymmetrical structure. All of these analyses involved free-field solutions as part of the analysis. Comparisons were made primarily through the response spectra of nodal points of the various finite element models. In addition, contour plots of shear strain and shear wave velocity were helpful in assessing the effects of the strain-dependent material properties.

6.2 FINITE ELEMENT MODELING CONSIDERATIONS

The formulation of the finite element models used in the FLUSH soil/structure interaction analyses depends on features of the FLUSH code, soil property parameters, surface structure representation, input ground motions, and the finite element meshes used for the various analyses.

6.2.1 DESCRIPTION OF FLUSH COMPUTER CODE

The FLUSH computer code is a two-dimensional, finite-element code designed specifically for applications to soil/structure interaction analyses (Lysmer et al., 1975). The FLUSH system (Norton, 1979b) used here is a version maintained by Cybernet Services of Control Data Corporation. The FLUSH system consists of the main program, along with several companion programs (Norton, 1979a; 1979b; CDC, 1981) that perform various data-preparation or postprocessing services. The capabilities of FLUSH discussed below pertain to the entire family of programs available through Cybernet.

FLUSH is capable of modeling a plane section consisting of one or several buildings embedded in a soil medium (see Fig. 6-1a). The buildings may be of varying sizes and shapes, but they must all be centered on the plane of analysis. The soil motion is assumed to be driven entirely through the mechanism of base motion of the model, where the model base is assumed to correspond to the interface between bedrock and the soil deposit. Only those components of the base motion that lie within the plane of analysis can be considered by the FLUSH code.

(a) Physical system to be analyzed

(b) FLUSH model

FIGURE 6-1. SOIL/STRUCTURE INTERACTION ANALYSIS USING FLUSH COMPUTER CODE

Figure 6-1b shows a FLUSH model of the physical system portrayed in Figure 6-1a. The intersections of the two structures with the plane of analysis are shown as networks of beam elements while the soil medium is represented by solid (plane-strain) elements. The FLUSH model is terminated on the bottom at the location of the soil/bedrock interface. This boundary of the model is taken to be fixed, and is assigned a motion corresponding to the vertical or horizontal component of the free-field bedrock motion, which is determined in an

auxiliary calculation. The ends of the FLUSH model consist of transmitting boundaries which simulate the effects of the soil deposit extending to infinite dimensions from the interaction zone. This simulation is accomplished through the use of closed-form solutions of a layered half-space with the same material properties as the soil deposit in the analysis. While the FLUSH model is two dimensional, energy transmission normal to the plane of analysis, as shown in Figure 6-1a, is simulated through the use of dampers. The dampers used for this

purpose are distributed among all the nodes below grade level, and they damp only the relative velocity of each node with respect to the free-field velocity corresponding to the same elevation.

The element library available in FLUSH consists of four-node and three-node plane solid (plane-strain) elements, two-node beam elements, and void elements. The latter are used only to occupy the interiors of structures below grade level to provide tributary areas in computing the damping coefficients for simulating out-of-plane energy dissipation. The beam elements, used for constructing the structural models, are linear elastic elements having the capability of transmitting axial, shear, and bending stresses in the plane of analysis. By incorporating appropriate areas and densities of the beam elements into the model, the dead weight of the structure is automatically taken into consideration. However, it is also possible to prescribe additional concentrated inertia values, both linear and rotational, at specified nodal points to account for the presence of heavy machinery. The solid elements can be assigned invariant stiffness and damping properties, but are more usually assigned properties that vary according to the existing level of shear strain in the element. The properties must remain constant throughout a given calculation, so an iterative approach must be used to converge to a set of stiffness and damping properties that are compatible with the resulting level of shear strain in the element. Relationships for both stiffness and damping as functions of shear strain may be prescribed by the user, and different layers (and even different elements within a layer) may be assigned different relationships. Then, throughout the iterative solution process, each element converges to its own combination of strain-compatible stiffness and damping properties.

Damping within the soil portion of the FLUSH model is considered on an element-by-element basis using a linear viscoelastic constitutive model formulated in terms of the complex modulus. The real part of the complex modulus corresponds to the stiffness of the element while the material damping within the element is related to the imaginary portion of the complex modulus. The value of the complex modulus for a given element is assumed to be time- and frequency-independent throughout the calculation, thereby simplifying the applicability of Fourier transform techniques to the time-response aspects of the problem. In particular, the equations of motion are formulated in matrix form incorporating the complex stiffness matrix. This complex stiffness matrix includes not only the complex moduli, but also terms associated with the various transmitting boundary dampers located throughout the model. Furthermore, the force vector contains terms relating to the transmitting boundary dampers operating on the free-field soil motions, and is therefore also complex valued. Application of the Fast Fourier Transform (FFT) algorithm reduces these equations to a system of frequency-dependent algebraic equations (e.g., Otnes and Enochson, 1972). These equations can be solved or interpolated at each frequency value of the FFT analysis to determine a set of amplitude functions. The technique known as the complex response method incorporates these amplitude functions into an inverse FFT operation and takes the real part to provide the time-dependent displacement field solving the problem (Lysmer et al., 1975).

This method of complex response is greatly simplified by permitting the complex moduli to remain constant and frequency-independent throughout the calculation. However, differing levels of shear strain magnitude are known to influence the stiffness and damping properties of the soil. To account for this effect, FLUSH

employs the equivalent linear method in which the stiffness and damping values of each element are held constant throughout the calculation, but at values that are compatible with the level of shear strain experienced by each element. This level of shear strain is referred to as the effective shear strain, and is taken to be 65% of the peak shear strain in the element. While the true relationship between a shear strain time history and its effective shear strain should take into account the entire strain record, 65% of the peak is a good rule of thumb (Lysmer et al., 1975); and in view of the relatively weak dependence of stiffness and damping on effective shear strain, even sizable errors in effective shear strain should not greatly affect the material parameters. Using the equivalent linear method, therefore, requires an iterative solution strategy in which each cycle begins with an estimate of the material properties. The dynamic response of the system to the applied base motion is computed by the complex response method, and the effective shear strain for each element is determined. New values of the material parameters for each element are determined on the basis of the effective shear strain, and if any element shows a greater-than-allowable difference between the new and old material parameters, an additional cycle is taken using the new parameters as the initial estimates. Acceptable convergence is usually obtained in about 3 to 5 iterations (Lysmer et al., 1975).

Ground shaking is applied to the FLUSH finite element model at the soil/bedrock interface, as shown in Figure 6-1. However, the FLUSH code is able to accept only one component of the input motion at a time. In that the FLUSH model is two-dimensional (even though it contains a few features that attempt to simulate three-dimensional effects), there are two ground motion components to be considered: one horizontal and one vertical. These two

components must be applied in two separate calculations with the resulting solutions superposed to obtain the overall system response to the combined horizontal and vertical ground motion. In order for the superposition to be valid, both the horizontal and vertical calculations must employ the same material properties. This implies that the equivalent-linear iterative solution strategy can be applied to only one of the two ground motion components. Usually the horizontal component contains stronger levels of excitation, and should be used to determine the strain-compatible material parameters. It is assumed, then, that when the results of the vertical calculation are summed in, the effective shear strain levels will not have been greatly altered. This assumption must be verified for the particular ground motion employed in any given FLUSH model. There is no recommended course of action in the FLUSH literature for a case in which the above assumption proves not to be valid.

In addition to FLUSH, which performs the entire complex response solution procedure within the equivalent-linear iterative strategy, and provides nodal acceleration histories as output, several additional programs that provide supplementary services are available in the FLUSH family. These include:

- NOODLE – to simplify mesh generation
- FORCE – to compute resultant forces at selected nodal points
- STRAIN – to compute strain and stress time histories for selected soil elements
- AFIT – to compute time histories of mean linear and rotational acceleration for groups of nodal points
- DISCOM – to compute relative displacements between specified pairs of nodal points

CYBERNET maintains additional programs for obtaining computer generated plots and for assembling super-elements from structural

models intended for use in other programs. Programs NOODLE, STRAIN, and AFIT were utilized here and will be referred to.

6.2.2 SOIL PROPERTIES

The soil material model available in the FLUSH code consists of a constant Poisson's ratio ν along with a strain-dependent shear modulus G and a strain-dependent damping ratio β . These latter two parameters combine to form the complex shear modulus G^* according to the formula (Lysmer et al., 1975):

$$G^* = G(1 - 2\beta^2 + 2i\beta\sqrt{1 - \beta^2}) \quad (6-1)$$

The parameters ν and G^* are the two elastic constants required to define the constitutive properties of the equivalent-linear soil model. The strain-dependent parameters, G and β , are defined in terms of functional relationships dependent on the effective shear strain, γ_{eff} . These relationships are

$$G = G_0 \cdot R(\gamma_{eff}) \quad (6-2)$$

$$\beta = \beta(\gamma_{eff}) \quad (6-3)$$

where G_0 is the limit approached by the secant shear modulus for cyclic shear loading as the amplitude of the strain cycles approaches zero, and R is a strain-dependent reduction factor that quantifies the ratio of G to G_0 . The two strain-dependent functions, R and β , are specified in FLUSH as tables of values that correspond to specific values of γ_{eff} . Between these values, R and β are determined by interpolation. The specification of material parameters, therefore, involves the specification of tables for R and β , as well as the specification of ν , G_0 , and the weight density of the material. In addition, initial estimates of the strain-compatible shear modulus and damping ratio are required.

The soil properties selected for the soil/structure interaction analyses performed here are those described in Chapter 5, from the

strong motion recording station at 8244 Orion Avenue in Los Angeles. However, the site investigation included only a determination of the shear wave velocities at small strain amplitudes, and the quality of these measurements was not very good. As a result, a body of data suitable for constructing relationships between shear modulus and effective shear strain and between damping ratio and effective shear strain does not exist. For the purposes of this study, however, it is not essential to have such a body of data. The primary consideration is that the properties used in the FLUSH calculations here should be similar to the properties used in the comparative calculations in Chapter 5. The soil properties are repeated in Table 6-1 (compare to Table 5-1 in Chapt. 5) except that the modulus units have been converted from psi to ksf, and the damping ratio is expressed as a range of values. This is due to the use of Rayleigh damping for which the damping ratio varies with frequency, and cannot vary from element to element within the model.

The material parameters of Table 6-1 were selected to be compatible with the strain levels expected in the various calculations for which they were used. The moduli corresponding to small strain values must, therefore, be determined. To do this it is first necessary to have

TABLE 6-1. SOIL PROPERTIES USED IN THE TRI/SAC FINITE ELEMENT CALCULATIONS

Depth, ft	Unit Weight, pcf	Shear Modulus, ksf	Shear Wave Velocity, ft/sec	Poisson's Ratio	Damping Ratio, %
0-10	105	1000	554	0.30	~6-10
10-20	105	1430	662	0.30	
20-40	105	1675	716	0.30	
40-70	110	1760	717	0.30	
70-100	110	1820	730	0.30	
100-135	115	2670	864	0.30	

relationships for shear modulus reduction factor (Eq. 6-2) and damping ratio (Eq. 6-3) as functions of effective shear strain. Typical relationships for sand and clay have been provided in the FLUSH manual and are reproduced in Table 6-2. In the absence of any other measurements pertaining to the Orion site, the relationships from Table 6-2 corresponding to sand were used as a first estimate for the strain-compatible soil properties for the FLUSH analyses.

TABLE 6-2. TABLE OF TYPICAL STRAIN-COMPATIBLE SOIL PROPERTIES*

Effective Shear Strain, γ_{eff} (%)	$\log (\gamma_{eff})$	Shear Modulus Reduction Factor†		Fraction of Critical Damping, %	
		Clay	Sand	Clay	Sand
$\leq 1. \times 10^{-4}$	-4.0	1.000	1.000	2.50	0.50
3.16×10^{-4}	-3.5	0.913	0.984	2.50	0.80
1.00×10^{-3}	-3.0	0.761	0.934	2.50	1.70
3.16×10^{-3}	-2.5	0.565	0.826	3.50	3.20
1.00×10^{-2}	-2.0	0.400	0.656	4.75	5.60
3.16×10^{-2}	-1.5	0.261	0.443	6.50	10.0
1.00×10^{-1}	-1.0	0.152	0.246	9.25	15.5
0.316	-0.5	0.076	0.115	13.8	21.0
1.00	0.	0.037	0.049	20.0	24.6
3.16	0.5	0.013	0.049	26.0	24.6
≥ 10.00	1.0	0.004	0.049	29.0	24.6

*From Lysmer et al. (1975).

†This is the factor which has to be applied to the shear modulus at low shear strain amplitudes (here defined as 0.0001%) to obtain the modulus at higher strain levels.

In order to back-calculate the small-strain shear modulus, it is necessary to compute the shear strain levels compatible with the shear moduli of Table 6-1. The one-dimensional free-field calculations using TRI/SAC (see Chapt. 5) were used for this purpose. These calculations consisted of one-dimensional representations of the TRI/SAC soil model subjected to the two horizontal motions and the vertical

motion from the 1971 Castaic ground motion record, each component applied to a separate calculation. The only strain component present in the calculation subjected to vertical motion is the normal component ϵ_{yy} . Similarly the only component present in the calculation subjected to N-S excitation is γ_{xy} , and for the E-W calculation the only component is γ_{yz} . Superposing the solutions of these three one-dimensional calculations, it can be shown that at any given time the maximum shear strain in the soil is

$$\gamma_{max} = \sqrt{\epsilon_{yy}^2 + \gamma_{xy}^2 + \gamma_{yz}^2} \quad (6-4)$$

in which the shear strains, γ_{ij} , are understood to be the engineering components of shear strain defined as

$$\gamma_{ij} = \frac{\partial u_i}{\partial x_j} + \frac{\partial u_j}{\partial x_i} \quad (6-5)$$

where x_i and x_j are the i and j coordinate axes and u_i and u_j are the displacement components in the i and j directions, respectively. The maximum shear strain, γ_{max} , is then scanned over the time interval to determine its maximum value. Taking 65% of this maximum value determines the effective shear strain for the element under consideration. Using this value of effective strain for interpolation in Table 6-2, one determines the shear modulus reduction factor $R(\gamma_{eff})$ and the damping ratio $\beta(\gamma_{eff})$ for the element. Then dividing the shear modulus from Table 6-1 by $R(\gamma_{eff})$, one determines the small-strain shear modulus for the element. These operations have been carried out in Table 6-3.

Comparing the values of damping ratio in Table 6-3 with the range of values in Table 6-1 shows that the FLUSH properties obtained using Tables 6-2 and 6-3 contain excessively high damping values. In order to bring them down into the range of values used in the TRI/SAC calculations, the relationship between damping ratio and effective shear strain was modified by multiplying each value by 0.6. The final

TABLE 6-3. COMPUTATION OF SMALL-STRAIN SHEAR MODULI FOR FLUSH SOIL MODEL

Depth, ft	Effective Shear Strain, %	Effective Shear Modulus, ksf	Damping Ratio, %	Small-Strain Shear Modulus, ksf
0-10	2.15×10^{-2}	1000	8.5	1,943
10-20	4.45×10^{-2}	1430	11.6	3,720
20-40	7.38×10^{-2}	1675	14.1	5,622
40-70	1.22×10^{-1}	1760	16.4	7,877
70-100	1.59×10^{-1}	1820	17.7	9,431
100-135	1.64×10^{-1}	2670	17.9	14,074

strain-compatible soil properties are shown in Table 6-4. These relationships were used for every soil element in all of the FLUSH models. Only the small-strain shear modulus and the weight density were varied from layer to layer to represent different materials.

TABLE 6-4. STRAIN-COMPATIBLE SOIL PROPERTIES USED FOR FLUSH ANALYSES

γ_{eff} Effective Shear Strain, %	$R(\gamma_{eff})$ Shear Modulus Reduction Factor	$\beta(\gamma_{eff})$ Damping Ratio, %
$\leq 1.00 \times 10^{-4}$	1.000	0.30
3.16×10^{-4}	0.984	0.48
1.00×10^{-3}	0.934	1.02
3.16×10^{-3}	0.826	1.92
0.0100	0.656	3.36
0.0316	0.443	6.00
0.100	0.246	9.30
0.316	0.115	12.6
1.00	0.049	14.8
3.16	0.049	14.8
≥ 10.00	0.049	14.8

At first it might appear that the small-strain moduli of Table 6-3 and the strain-compatible functions of Table 6-4 complete the material specification process. This is not quite true, however, in that the small-strain moduli of Table 6-3 are associated with the grid layering of Column 1 of that table. It is necessary,

therefore, to verify the frequency-transmission capability of the layering configuration implied in the table. For this purpose, the FLUSH manual provides the formula

$$h_{max} = \frac{1}{5} \cdot \frac{V_s}{f_{max}} \quad (6-6)$$

where V_s is the lowest shear wave velocity the element will possess throughout the equivalent-linear iteration process, f_{max} is the highest frequency (in Hz) of interest in the analysis, and h_{max} is the largest element height compatible with the above criteria. Using Equation 6-6 along with the zoning configuration and expected shear moduli of Table 6-3, one can evaluate the frequency-transmission capabilities of the TRI/SAC models. This results in Table 6-5, which shows the maximum layer height for a maximum frequency of 10 Hz. It is seen that for all of the layers below 20 ft, Equation 6-6 is violated for a value of 10 Hz for f_{max} . This is also evident from the last column of the table in which Equation 6-6 was used in conjunction with the existing zoning to compute the maximum frequency that can be transmitted by the layer. In all cases below 10 ft, the limiting frequency is below 10 Hz. This means that if 10 Hz is to be retained as the maximum frequency of interest, the layering scheme will have to be redefined.

The free-field model of Figure 6-2 was proposed as a means of approximately satisfying Equation 6-6 for a maximum frequency of 10 Hz while retaining all of the node elevations used in the TRI/SAC analyses. This latter condition facilitates comparisons between the FLUSH results and the TRI/SAC results. The primary difference between the zoning of Figure 6-2 and the zoning indicated in Table 6-5 is that many of the layers in the latter model have been subdivided into two layers. The layer midheights of the newly divided sublayers are at different elevations than the midheight of the original, undivided layer, and so the small-strain moduli

TABLE 6-5. ESTIMATED FREQUENCY-TRANSMISSION CAPABILITY OF MODEL USED FOR TRI/SAC CALCULATIONS

Layer Number	Depth, ft	Estimated Effective Shear Strain, %	Estimated Shear Modulus, ksf	Estimated Shear Wave Velocity, fps	Maximum Layer Height for Max Freq = 10 Hz, ft	Maximum Frequency Compatible with Existing Layer Height, Hz
1	0-10	0.0215	1000	554	11.1	11.1
2	10-20	0.0445	1430	662	13.2	13.2
3	20-40	0.0738	1675	716	14.3	7.2
4	40-70	0.122	1760	717	14.3	4.8
5	70-100	0.159	1820	730	14.6	4.9
6	100-135	0.164	2670	864	17.3	4.9

FIGURE 6-2. ONE-DIMENSIONAL SOIL MODEL FOR TESTING PROPOSED LAYERING

of the new sublayers must be modified to account for the resulting changes in confining pressure. Hardin and Drnevich (1972) show that the relationship between mean principal effective stress $\bar{\sigma}_o$ and small-strain shear modulus G_o is

$$G_o = 1230 \frac{(2.973 - e)^2}{1 + e} (\text{OCR})^K \bar{\sigma}_o^{1/2} \quad (6-7)$$

where $\bar{\sigma}_o$ and G_o are in psi, e is the void ratio, OCR is the overconsolidation ratio, and K is a constant that depends on the plasticity index. Assuming that OCR and the plasticity index are constant throughout a layer, Equation 6-7 can be simplified to

$$G_o = C \sqrt{\bar{\sigma}_o} \quad (6-8)$$

where G_o and $\bar{\sigma}_o$ can now be expressed in any set of stress units as long as the constant C takes the new units into account. The mean principal effective stress $\bar{\sigma}_o$ is computed according to the relation

$$\bar{\sigma}_o = \frac{1}{3} (\sigma_{xx} + \sigma_{yy} + \sigma_{zz}) \quad (6-9)$$

where the terms on the right-hand side of the equation are the three components of normal stress for a given coordinate system (x , y , z). In the absence of any data pertaining to horizontal stresses, it is assumed that they are proportional, within the layer, to the vertical

component, σ_{yy} . Then Equation 6-8 can be rewritten as

$$G_o = C' \sqrt{\sigma_{yy}} \quad (6-10)$$

where C' is a new constant. Using the strain levels from Table 6-3, the small-strain moduli of Table 6-5, and Equation 6-10, the constant C' for each layer is computed, followed by the small-strain shear modulus for each sublayer. The resulting values are shown in Table 6-6. Assuming that the strain level in each sublayer is the same as in the original layer from which it was obtained, the estimated strain-compatible shear moduli are listed in the last column of the table. Comparing both these values with their respective values in Table 6-3, it is seen that the values in Table 6-3 fall in the approximate middle of the range of values for the corresponding sublayers in Table 6-6.

Table 6-7 summarizes the soil layer properties for the FLUSH models. These values of Poisson's ratio, weight density, and small-strain shear modulus for each layer of the model, together with the strain-dependent functions of Table 6-4, define the material model used for the FLUSH analyses. Strain compatible values of the shear modulus and damping ratio must still be found for each element through iterative

techniques. Estimated values of each are provided in Table 6-7 for each layer of the model. These estimates are used as starting values for the equivalent-linear iterative process. The closer they are to the final values, the fewer iterations are required. Comparing the estimated shear moduli and damping ratios with the corresponding values of Table 6-1, a great deal of similarity can be observed. In view of these similarities it is relatively easy to imagine that both sets of properties could have been derived from the same body of test results if such a data set were to exist.

6.2.3 REPRESENTATION OF SURFACE STRUCTURES

The structures to be analyzed using the FLUSH finite element program include the symmetrical and the unsymmetrical structures described in Chapter 5. These structures, which were modeled using continuum elements in the TRI/SAC analyses, are modeled using beam elements in the FLUSH analyses. As in the case of the soil properties, it is highly desirable for the structural models used in the FLUSH analyses to behave similarly to those used in the TRI/SAC analyses.

TABLE 6-6. COMPUTATION OF SMALL-STRAIN SHEAR MODULUS FOR FLUSH SOIL MODEL

Layer Number	Depth, ft	Density, pcf	σ_y at Mid-Depth of Layer, ksf	Small-Strain Shear Modulus, ksf	Estimated Effective Shear Strain, %	Estimated Effective Shear Modulus, ksf
1	0-10	105	0 525	1,943	0 0215	1000
2	10-20	105	1 575	3,720	0 0445	1430
3	20-30	105	2 625	5,132	0 0738	1529
4	30-40	105	3 675	6,073	0 0738	1809
5	40-55	110	5 025	7,300	0 122	1631
6	55-70	110	6.675	8,414	0 122	1880
7	70-85	110	8 325	8,996	0.159	1736
8	85-100	110	9.975	9,847	0 159	1900
9	100-117 5	115	11 806	13,510	0 164	2563
10	117.5-135	115	13 819	14,616	0 164	2773

TABLE 6-7. SOIL PROPERTIES FOR FLUSH SOIL/STRUCTURE INTERACTION ANALYSES

Layer Number	Depth, ft	Poisson's Ratio	Density, pcf	Small-Strain Shear Modulus, ksf	Estimated Shear Modulus, ksf	Estimated Damping Ratio, %
1	0-10	0.3	105	1,943	1000	5.1
2	10-20	0.3	105	3,720	1430	7.0
3	20-30	0.3	105	5,132	1529	8.4
4	30-40	0.3	105	6,073	1809	8.4
5	40-55	0.3	110	7,300	1631	9.9
6	55-70	0.3	110	8,414	1880	9.9
7	70-85	0.3	110	8,996	1736	10.6
8	85-100	0.3	110	9,847	1900	10.6
9	100-117.5	0.3	115	13,510	2563	10.7
10	117.5-135	0.3	115	14,616	2773	10.7

The continuum properties used in the TRI/SAC analyses were obtained by assuming the structure to consist of reinforced concrete slab floors and walls. The concrete has a Young's modulus of 5.0×10^6 psi, a Poisson's ratio of 0.2, and a unit weight of 150 pcf. The concrete slabs comprising the roof and the floors were assumed to be 3 ft thick. Interior wall slabs were assumed to be 3 ft thick while exterior walls were taken as 4 ft thick above grade level and 6 ft thick below. The foundation slab is 8 ft thick. These same assumptions were also used in constructing the FLUSH models.

The FLUSH model for the symmetrical structure is shown in Figure 6-3. The square plan view means that the N-S and E-W models are identical; hence, the figure doesn't show compass directions. The corresponding model used in FLUSH for both the N-S and E-W analyses is shown in Figure 6-3b. The lines in this figure correspond to beam elements, and the large dots correspond to nodal points. These nodal points are actually double nodes, the first of each pair corresponding to translational degrees of freedom and the second to a rotational degree of freedom. The beam elements have the capability of transmitting axial

(a) Three-dimensional representation of symmetrical structure

(b) Two-dimensional model

FIGURE 6-3. MODEL OF SYMMETRICAL STRUCTURE USED IN FLUSH ANALYSES

forces, shear forces, and bending moments. Each circled number refers to a set of beam properties found in Table 6-8 that corresponds to the span of beam elements located nearest the number. Thus, the two vertical spans labeled ② correspond to an exterior wall above grade level. Such a wall has a thickness of 4 ft as shown in Table 6-8 for Beam Set 2. Similarly, the vertical spans at the right of the model correspond to an interior wall, which is taken as 3 ft thick. However, since this line is a line of symmetry, only half the wall corresponds to the half-model illustrated. The area and flexural stiffness of the wall are therefore divided by 2, as shown in Table 6-8 under Beam Set 5.

TABLE 6-8. BEAM ELEMENT SECTION PROPERTIES FOR STRUCTURAL MODELS OF FLUSH ANALYSES

Beam Set	Thickness, ft	Moment of Inertia Per Unit of Thickness, ft^4/ft
1	3.0	2.250
2	4.0	5.333
3	6.0	18.000
4	8.0	42.667
5	1.5	1.125
6	2.0	2.667
7	3.0	9.000
8	3.5	3.792
9	4.0	21.333
10	4.5	10.125

Figure 6-4 shows the FLUSH models for the unsymmetrical structure. Again, the circled number identifies the corresponding span of beam elements. It can be noted in the N-S structural model that while the outline of the structure is symmetrical, some portions of the structure have only half the thickness of other portions, and the distribution of such portions is unsymmetrical. Thus the asymmetry of the model of Figure 6-4b arises from the beam properties, and not from the beam config-

uration. The thickness of the structural model is considered to be 40 ft, as in the case of the symmetrical structure. The regions of the model that are only 20 ft thick are then represented by dividing the beam thicknesses and moments of inertia by 2. This has the additional effect of reducing the mass of the beam by a factor of 2. Thus, while the vertical members below grade level on the left and the right of the model both represent exterior walls of 6 ft thickness, those on the right span only 20 ft in the out-of-plane direction. The corresponding section properties (beam set 7 in Table 6-8) are therefore only half those of the left side (beam set 3). Similar considerations were applied to the formulation of the E-W structural model.

Although the models used in the FLUSH analyses are different from those in the TRI/SAC analyses in their element composition, their mechanical behavior is quite similar. In both the FLUSH and TRI/SAC analyses, the stiffnesses of the structural representations greatly exceed those of the soil representations. The most important characteristic of the structures for the purposes of soil/structure interaction analysis is their weight. Table 6-9 compares the weights of the FLUSH models with the weights of the corresponding three-dimensional TRI/SAC analyses. It is seen that in all cases these values are well within 10% of each other.

TABLE 6-9. COMPARISON OF MASS CHARACTERISTICS BETWEEN FLUSH MODELS AND 3-D TRI/SAC MODELS

Model	Weight of Structural Model Per Foot of Thickness, lbs	
	TRI/SAC	FLUSH
Symmetrical Structure	2.476×10^5	2.310×10^5
Unsymmetrical Structure, N-S	1.662×10^5	1.710×10^5
Unsymmetrical Structure, E-W	1.662×10^5	1.785×10^5

FIGURE 6-4. MODELS OF UNSYMMETRICAL STRUCTURE USED IN FLUSH ANALYSES

6.2.4 INPUT GROUND MOTIONS

The input ground motions selected for these soil/structure interaction analyses using the FLUSH program are the same motions described in Chapter 5, that is, the three components recorded at Castaic, California, during the 9 February 1971, San Fernando earthquake. These records were chosen primarily because they develop strong ground shaking of relatively short duration, thereby allowing more economical computer analysis, using step-by-step integration procedures, than would a longer duration record. It should be noted

that by using FLUSH, which employs the complex response method, the duration of the ground motion record does not play an important role in the economy of the analysis.

The three components of motion along with their pseudovelocity response spectra are shown in Figure 5-4 of Chapter 5.

6.2.5 FINITE ELEMENT MODELS

The finite element model for the soil/structure interaction analysis of the symmetrical structure is shown in Figure 6-5. It was

FIGURE 6-5. FINITE ELEMENT MESH USED FOR FLUSH SSI ANALYSES OF SYMMETRICAL STRUCTURE

necessary to model only half of the soil/structure system, because advantage could be taken of a line of shear symmetry located along the centerline of the symmetrical structure; so, only the portion to the left of the centerline was modeled. The term "shear symmetry" has been coined for usage here to describe a situation which is not a true symmetry condition. Because of the linearity of the solution technique, superposition is permissible, and the solution can be computed as the sum of the responses to the horizontal and vertical components of the input base motion taken separately. For the case of a purely horizontal input motion, the horizontal component of the response is symmetric about the centerline while the vertical component is antisymmetric. This situation can be modeled by preventing vertical displacements along the centerline while allowing freedom for horizontal displacements and rotations. For the case of a purely vertical input motion, the horizontal component of the response is antisymmetric about the centerline

while the vertical component is symmetric. This situation can be modeled by preventing horizontal displacements and rotations along the centerline and allowing displacements in the vertical direction. Then, employing the superscript H to signify a response component corresponding to a horizontal base motion and V corresponding to a vertical base motion, the combined response is

$$\left. \begin{aligned} u_x(x,y,t) &= u_x^H(x,y,t) + u_x^V(x,y,t) \\ u_y(x,y,t) &= u_y^H(x,y,t) + u_y^V(x,y,t) \end{aligned} \right\} x \leq x_c$$

$$\left. \begin{aligned} u_x(x,y,t) &= u_x^H(2x_c-x,y,t) - u_x^V(2x_c-x,y,t) \\ u_y(x,y,t) &= -u_y^H(2x_c-x,y,t) + u_y^V(2x_c-x,y,t) \end{aligned} \right\} x > x_c$$

(6-11)

where x is the horizontal coordinate, y is the vertical coordinate, t is time, and x_c is the x -coordinate of the line of shear symmetry. Thus, while the combined response is not at all symmetric, it is obtainable by the superposition of two solutions that do possess certain symmetrical and antisymmetrical attributes. The complete solution is obtainable, therefore, by modeling only half of the soil/structure system. The appropriate modeling considerations are presented in the FLUSH manual (Lysmer et al., 1975).

The left boundary of the finite element model of Figure 6-5 is a transmitting boundary (see Sec. 6.2.1 above). This type of boundary simulates the continuation of the soil medium into a region of infinite extent by applying energy absorbing dampers to the difference in motion between a given node and the free-field response corresponding to the elevation of the node. The structure is represented by beam elements that model the shear, flexural, and mass characteristics of concrete slabs of thicknesses varying between 3 and 8 ft (described

earlier). The 40-ft width of the structure is included in the model so that out-of-plane energy radiation effects may be taken account of by dampers distributed throughout the model. These dampers, as do the dampers of the transmitting boundary, act on the difference between the nodal motion and the free-field response corresponding to the same elevation.

As in the case of the TRI/SAC analyses, the input ground motions are applied to the base of the model. The motions themselves are actually ground surface motions recorded at Castaic, California, and FLUSH has the capability of allowing the motions to be applied at the ground surface and deconvolving a corresponding base motion. However, in order to facilitate comparisons with the TRI/SAC results, the motions are applied to the base of the model. As mentioned above, the horizontal and vertical components of the input base motion are applied separately, and different boundary conditions are applied along the line of shear symmetry for each component. Two soil/structure interaction analyses are performed, one corresponding to the N-S direction and one to the E-W direction. In each case the horizontal motion is applied first and the iterative equivalent-linear procedure is applied until convergence of the material properties is obtained. The vertical component is then applied with the final properties from the horizontal analysis, and no iterations are taken. The combined strain state is then checked to determine if it is compatible with the material properties used. If it is not compatible to within an acceptable tolerance, a new set of material properties must be used in both the horizontal and vertical calculations.

Figure 6-6 shows the finite element model used to represent the N-S plane for the analysis of the unsymmetrical structure. While the structure appears to be symmetrical in this

FIGURE 6-6. FINITE ELEMENT MESH USED FOR N-S SSI ANALYSIS OF UNSYMMETRICAL STRUCTURE USING FLUSH

model, differing beam properties destroy the apparent symmetry (see Sec. 6.2.3). Note that the full soil/structure system must now be included due to the lack of symmetry. Both ends of the model are transmitting boundaries to simulate a soil deposit of infinite lateral extent. Since this model corresponds to the N-S direction, only the N-S and vertical components of motion are applied. These are applied in separate calculations, however, since FLUSH can handle only a single component of base input motion. The horizontal component is applied first and iterations are taken until a set of strain-compatible soil properties is obtained. Then the vertical component is applied using the final properties from the horizontal analysis. In a similar manner the E-W and vertical components of base motion are applied to the E-W model of the unsymmetrical structure (Fig. 6-7).

6.3 ITERATION CYCLES

Strain-dependent material properties are handled in the FLUSH finite element program using the equivalent linear method. As

FIGURE 6-7. FINITE ELEMENT MESH USED FOR E-W SSI ANALYSIS OF UNSYMMETRICAL STRUCTURE USING FLUSH

described earlier, this method involves specifying the strain-dependent behavior, and then performing successive iterations until the properties used are compatible with the strain levels resulting from the analysis. Additional considerations other than modeling parameters enter into the execution of the iteration cycles. These are discussed here as those that pertain to free-field analyses and those that pertain to soil/structure interaction analyses.

6.3.1 FREE-FIELD ANALYSES

The free-field response of the soil medium to the prescribed base motion plays a prominent role in the FLUSH finite element program. It is used directly in the transmitting boundaries. It plays a role in connection with the dampers used to simulate out-of-plane energy dissipation. Free-field analysis also serves as an economical means of arriving at a reasonable first approximation to the strain-compatible soil properties of the soil/structure interaction (SSI) analysis. In connection with this it also provides an early opportunity to verify whether or not the grid zoning is sufficiently fine according to the criterion of Equation 6-6. For the purposes of this study, it also provides an

opportunity to compare the FLUSH soil model with that of the TRI/SAC analyses in order to evaluate whether or not it is reasonable to presume that the two models could have arisen from the same body of raw data if such data were to exist.

SSI analyses using FLUSH are performed two dimensionally. The availability of ground motion records in three mutually perpendicular directions serves to dictate that there will be two two-dimensional SSI analyses, each corresponding to one of two mutually perpendicular vertical planes. Each of these SSI analyses will involve its own iteration process for strain-compatible properties, thereby virtually assuring that the N-S SSI analysis will have different soil properties than the E-W SSI analysis. Virtually any SSI analysis using a two-dimensional, materially nonlinear program will result in these same circumstances. Since the free-field solution is to be used in conjunction with a SSI analysis, it is essential that the free-field soil properties be consistent with those of the corresponding SSI analysis. Therefore, the vertical and horizontal free-field analyses corresponding to the N-S SSI analysis must use strain-compatible properties relevant to that analysis, and the E-W free-field analyses must correspond to the E-W SSI analysis. Since the strain-compatible soil properties for these SSI analyses are determined using the horizontal base motion component, the N-S and E-W free-field analyses are computed with different properties from each other, and consequently there are two different vertical analyses, one for the properties of each of the two SSI analyses.

The final strain-compatible soil properties resulting from these free-field analyses are shown in Table 6-10. Two sets of shear moduli and damping ratios are shown. These correspond to the N-S and the E-W free-field analyses. Note that the expectation that these two

TABLE 6-10. STRAIN-COMPATIBLE SOIL PROPERTIES RESULTING FROM THE FREE-FIELD ANALYSES

Layer Number	Depth, ft	Small-Strain Shear Modulus, ksf	Strain-Compatible Properties for N-S		Strain-Compatible Properties for E-W	
			Shear Modulus, ksf	Damping Ratio, %	Shear Modulus, ksf	Damping Ratio, %
1	0-10	1,943	843	6.2	1,062	4.7
2	10-20	3,720	1,186	8.1	1,548	6.4
3	20-30	5,132	1,483	8.6	1,842	7.4
4	30-40	6,073	1,650	8.9	1,813	8.4
5	40-55	7,300	2,069	8.7	1,832	9.2
6	55-70	8,414	2,280	8.9	1,925	9.7
7	70-85	8,996	2,311	9.1	1,866	10.3
8	85-100	9,847	2,891	8.5	2,044	10.3
9	100-117.5	13,510	5,314	6.8	3,927	8.6
10	117.5-135	14,616	5,808	6.8	4,529	8.2

sets of properties would be different from each other has indeed been realized. However, the magnitude of this difference is surprisingly large. These two sets of properties were converged upon using the same strain-dependent material model, a convergence criterion of 2%, and the two horizontal ground motion components shown in Figure 5-4 (see Chapt. 5); the corresponding response spectra are also shown. These input motions are the only point of difference between the two analyses, and intuitively there does not appear to be much difference between them. The result, however, in terms of strain-compatible, free-field soil properties involves differences as high as 30%. Comparing the soil properties used in the TRI/SAC analyses (see Table 6-1) with the two sets of properties of Table 6-10, and realizing that these latter two sets were arrived at from exactly the same material model, it is not out of line to believe that all three sets could have been derived from the same body of raw data. Therefore, it is postulated that the SSI analyses using FLUSH and the soil model described above are eligible for comparison against the TRI/SAC SSI analyses of Chapter 5.

Comparisons of response spectra for the free-field analyses as performed by FLUSH and TRI/SAC were presented in detail in AA (1982). For the E-W component, these comparisons typically appear quite favorable, while for the N-S component, the FLUSH response spectra exhibit much higher pseudoaccelerations than do the TRI/SAC response spectra. This result may be partially explained by observing that, at least for the upper layers of the soil column, the shear moduli of the TRI/SAC analyses (Table 6-1) agree more closely with the E-W properties than the N-S properties of Table 6-10. An additional effect of this phenomenon is that the vertical spectrum from the E-W analysis is closer to the TRI/SAC spectrum than is the vertical spectrum from the N-S analysis.

It has been pointed out that the strain-compatible soil properties used in these free-field analyses are the result of equivalent-linear iteration cycles for the soil column excited by the horizontal component of base motion alone. A natural question to ask is whether these strain-compatible properties would have been

much different if it had been possible to consider the combined strain state resulting from both horizontal and vertical base excitation. To actually implement an iteration scheme based on the combined strain state is beyond the standard capabilities of the FLUSH system, and would therefore require a great deal of hand calculation between cycles. While such an approach is not practical, the question posed above is important enough that at least one small analysis ought to be dealt with in this way. The N-S free-field analysis was selected for this investigation, primarily due to the relatively small size of free-field analyses.

A free-field analysis subjected to horizontal base excitation produces a strain state of pure shear, while one excited by vertical base motion produces only vertical normal strains. The superposition of these two strain states, corresponding to the application of combined horizontal and vertical base motion, consists of two components: (1) the shear strain γ_{xy} in the vertical plane of the horizontal component of base motion, and (2) the normal strain component ϵ_{yy} in the vertical direction. For any given instant of time t the maximum shear strain γ_{max} for this state of strain is

$$\gamma_{max}(t) = \sqrt{[\epsilon_{yy}(t)]^2 + [\gamma_{xy}(t)]^2} \quad (6-12)$$

The effective shear strain γ_{eff} in the FLUSH program is taken to be 65% of the peak value of γ_{max} or

$$\gamma_{eff} = 0.65 \times M_t (\gamma_{max}(t)) \quad (6-13)$$

where the function M_t indicates the maximum absolute value of its argument over the time interval. Since for the vertical case the maximum shear strain equals the normal strain in the y -direction, it can be shown that

$$\gamma_{eff} \leq \sqrt{(\gamma_{eff}^H)^2 + (\gamma_{eff}^V)^2} \quad (6-14)$$

where γ_{eff}^H is the effective shear strain for the free-field analysis driven by the horizontal component of base motion and γ_{eff}^V the effective normal strain for the vertical component. The case in which equality holds in Equation 6-14 takes place only when the peak values of both strain histories occur simultaneously.

Table 6-11 shows the effective strains for the horizontal and vertical calculations. These values are combined using Equation 6-14 to determine an upper bound for the resultant effective shear strain corresponding to the combined base motion. It is noteworthy that the largest difference between a resultant

TABLE 6-11. PROPOSED SOIL PROPERTIES FOR AN ADDITIONAL CYCLE OF THE N-S FREE-FIELD ANALYSIS BASED ON COMBINED STRAIN LEVELS FOR HORIZONTAL AND VERTICAL ANALYSES

Layer Number	Depth, ft	γ_{eff}^H Effective Shear Strain, %	γ_{eff}^V Effective Normal Strain, %	Resultant Upper Bound Effective Shear Strain, %	Resultant Shear Modulus, ksf	Resultant Damping Ratio, %
1	0-10	3.416×10^{-2}	6.007×10^{-3}	3.468×10^{-2}	830	6.3
2	10-20	6.732×10^{-2}	1.241×10^{-2}	6.845×10^{-2}	1156	8.2
3	20-30	7.998×10^{-2}	1.578×10^{-2}	8.152×10^{-2}	1442	8.7
4	30-40	8.790×10^{-2}	1.874×10^{-2}	8.988×10^{-2}	1605	9.0
5	40-55	8.102×10^{-2}	1.877×10^{-2}	8.317×10^{-2}	2026	8.8
6	55-70	8.590×10^{-2}	2.013×10^{-2}	8.823×10^{-2}	2250	8.9
7	70-85	9.239×10^{-2}	2.189×10^{-2}	9.495×10^{-2}	2293	9.2
8	85-100	7.358×10^{-2}	1.834×10^{-2}	7.583×10^{-2}	2889	8.5
9	100-117.5	4.243×10^{-2}	1.014×10^{-2}	4.362×10^{-2}	5241	6.9
10	117.5-135	4.134×10^{-2}	9.252×10^{-3}	4.236×10^{-2}	5744	6.8

upper-bound value and the corresponding horizontal effective shear strain is only 3.1%. This indicates that using the horizontal calculation only for computing strain-compatible moduli results in only small errors in strain magnitude. The errors in shear modulus and damping ratio are even smaller.

The strain-compatible soil properties are the result of an iteration procedure that makes use of a convergence criterion of 2%. Thus, if either the shear modulus or the damping ratio compatible with the computed level of effective strain for the current cycle, for any of the ten soil layers, differed by more than 2% from the corresponding value used during the cycle, another iteration cycle was undertaken. The strain-compatible properties shown for the N-S free-field analysis in Table 6-10 were accepted as final because an updated set of properties computed from the new effective strains for the horizontal analysis were not different by more than 2% from the values shown. Table 6-11 shows the new properties that would have been computed from the effective strains of the combined analysis if FLUSH were capable of doing this. The largest difference between these values and the values of Table 6-10 is 2.8%. This is a very small growth over the amount of

error that exists solely due to imperfect convergence based on the horizontal effective shear strain. It is concluded from this investigation that the determination of strain-compatible soil properties using only horizontal base motions does not contribute any significant errors for the base motions and soil models used here.

Table 6-12 investigates the acceptability of the finite element zoning for the free-field analyses in regard to Equation 6-6. For this purpose the maximum frequency of interest was taken to be 10 Hz. The last column of this table shows the allowable layer thickness for each layer using the lower of the shear wave velocities for the two free-field analyses. Compared to the actual layer heights, shown in the next to last column of the table, it is seen that the allowable layer heights approximate the existing values throughout the model. Although there are a few situations for which the existing layer thickness is a slight amount larger than allowable, the amount of excess height is not considered to be serious. The existing zoning is extremely convenient for comparing responses with those of the TRI/SAC analyses, and this was the key element in adopting the zoning of Figure 6-2.

TABLE 6-12. EXAMINATION OF ELEMENT LAYER THICKNESS

Layer Number	Depth, ft	Shear Wave Velocity, fps		Layer Thickness, ft	
		N-S Analysis	E-W Analysis	Existing	Allowable for $f_{\max} = 10$ Hz
1	0-10	508	571	10.0	10.2
2	10-20	603	689	10.0	12.1
3	20-30	674	752	10.0	13.5
4	30-40	711	746	10.0	14.2
5	40-55	778	732	15.0	14.6
6	55-70	817	751	15.0	15.0
7	70-85	823	739	15.0	14.8
8	85-100	920	774	15.0	15.5
9	100-117.5	1220	1049	17.5	21.0
10	117.5-135	1275	1126	17.5	22.5

One final task accomplished through the study of free-field analyses was the choosing of an interpolation schedule for the Fourier amplification functions from which the solution is constructed. These amplification functions are continuous functions of frequency. In frequency ranges for which these functions are not varying very rapidly with respect to frequency, sufficient accuracy is obtained by solving directly for only every Nth frequency value and interpolating for the intermediate frequency values. The interpolation interval, N, must be an integral power of 2. For the time step size and number of time steps used in these analyses (0.02 sec and 512, respectively), the optimum interpolation schedule proved to be that shown in Table 6-13. This schedule was used for all the FLUSH analyses performed here.

TABLE 6-13. OPTIMUM INTERPOLATION SCHEDULE FOR FLUSH ANALYSES

Frequency Range, Hz	Interpolation Interval
0 - 2	2
2 - 5	4
5 - 10	8

6.3.2 SOIL/STRUCTURE INTERACTION ANALYSES

The soil/structure interaction (SSI) analyses performed in this study employed the finite element meshes shown in Figures 6-5, 6-6, and 6-7. The strain-compatible properties used in these analyses were determined using equivalent linear iteration cycles applied to the models excited by the horizontal component of base motion only. Since the properties determined for the free-field analyses showed only negligible errors as a result of neglecting the vertical component of base motion, it is assumed that the same procedure is acceptable for the SSI

analyses as well. A convergence criterion of 5% was used for these analyses based on the values of shear moduli and damping ratios for successive iterations. This convergence criterion was applied by manually inspecting the percent change in these parameters for every element for each iteration, and if any difference was greater than 5%, an additional iteration was taken.

The effects of the soil/structure interaction on the strain-compatible soil properties turned out to be quite significant. To illustrate the nature of these effects, a single case is studied in detail. The case chosen is the N-S SSI analysis of the symmetrical structure subjected to horizontal base motion. Figure 6-8 shows contours of effective shear strain for this case. The column of elements located along the left edge of the model indicates the distribution of effective shear strain in the free field. This

FIGURE 6-8. EFFECTIVE SHEAR STRAIN (PERCENT) FOR SOIL/STRUCTURE INTERACTION ANALYSIS OF SYMMETRICAL STRUCTURE SUBJECTED TO HORIZONTAL BASE MOTION IN THE N-S DIRECTION

figure shows that increased effective shear strain has developed in the vicinity of the structure. The dimensions of this region appear to be comparable in size to the dimensions of the structure itself. A zone of effective shear strain that is greatly magnified over that of the free field has developed immediately adjacent to the structure.

These results indicate that soil/structure interaction effects are very influential in terms of effective strain levels. The extent to which these perturbations in the effective shear strain modify the properties of the soil medium is demonstrated in Figure 6-9, which shows contours of shear-wave velocity for the N-S SSI analysis of the symmetrical structure subjected to the horizontal component of base motion only. As in the case shown in Figure 6-8, the column of elements at the left edge of the model of Figure 6-9 indicates the distribution of shear wave velocity in the free field as a result of

the N-S component of ground motion. This free-field distribution is altered near the structure due to the zone of increased effective shear strain shown in Figure 6-8. However, the magnitude of the alteration in shear wave velocity appears to be substantially smaller than the corresponding alteration in effective shear strain. The latter exhibits a concentration on the order of six-fold, while the shear-wave velocity shows a maximum reduction of approximately two-fold.

While both the effective shear strain distribution and the distribution of shear wave velocity show strong soil/structure interaction effects in the vicinity of the structure, these effects are confined to relatively small regions. It is difficult to forecast, therefore, how much change in response will result from these changes in properties. Figure 6-10 shows comparisons in response spectra for four locations in the N-S SSI analysis of the symmetrical structure subjected to horizontal base motion only. The location of each comparison is marked by the dot in the inset of each of the spectral plots. The plots compare the response spectrum computed using the final strain-compatible soil properties of the SSI analysis with that computed from the initial properties, which are strain compatible for the free-field analyses. These comparisons show very minor differences for the top of the symmetrical structure and virtually no difference for the other locations. In fact, the comparisons are so close in the cases of Figures 6-10c and 6-10d that it appears in each figure that only one curve is present. These results support the conclusion that, although SSI effects do induce significant modifications in the free-field soil properties, they result in only insignificant changes in the response of the soil/structure system. This same finding has been reported by Kausel et al. (1976).

A potential consequence of the reduction in

FIGURE 6-9. SHEAR WAVE VELOCITY (FEET PER SECOND) FOR SSI ANALYSIS OF SYMMETRICAL STRUCTURE SUBJECTED TO HORIZONTAL BASE MOTION IN THE N-S DIRECTION

(a) Top of structure

(b) Grade level of structure

(c) Foundation level of structure

(d) Depth of 70 feet

FIGURE 6-10. COMPARISONS OF RESPONSE SPECTRA FOR VARIOUS LOCATIONS OF N-S SSI ANALYSIS OF SYMMETRICAL STRUCTURE SUBJECTED TO HORIZONTAL BASE MOTION ONLY

shear-wave velocity in the vicinity of the structure, as shown in Figure 6-9, is the diminished capability of the soil model to transmit higher frequency excitation. The layer thicknesses for the finite element model have been determined by applying Equation 6-6, with a maximum frequency of 10 Hz, to the free-field analyses. The results of this exercise are shown in Table 6-12; however, the large reduction in

shear-wave velocity in the vicinity of the structure greatly reduces the maximum frequencies that can be transmitted by elements near the structure. Figure 6-11 shows maximum frequencies permissible for selected elements in the free field and in the interaction zone. Note that the free-field values are slightly different from those of Table 6-12 due to additional iterations during the SSI phase of the analysis.

FIGURE 6-11. MAXIMUM FREQUENCIES, IN Hz, CAPABLE OF BEING TRANSMITTED BY SELECTED ELEMENTS FOR N-S SSI ANALYSIS OF SYMMETRICAL STRUCTURE SUBJECTED TO HORIZONTAL BASE MOTION ONLY

Some of the elements in the interaction zone, however, show substantial reductions in maximum frequency, and raise the question as to whether or not the problem must be remeshed. The only reason the mesh might not need to be refined would be that the frequency content of the soil/structure response may be within the limitations of the mesh. This latter question was investigated, but since the existing mesh has elements incapable of transmitting frequencies greater than 5.2 Hz, a finer mesh was required for this investigation.

Figure 6-12 shows a fine-zoned model of the symmetrical SSI problem. This mesh concentrates into the interaction zone elements of about half the layer thickness of those of Figure 6-5. The N-S component of the Castaic ground motion record was applied to the base of this mesh. The material properties for this mesh are the same as for Figure 6-5 except that the layer midheights are at different elevations, thereby inducing different small-strain moduli

FIGURE 6-12. FINE-ZONED MODEL OF SYMMETRICAL STRUCTURE USED TO ASSESS FREQUENCY TRANSMISSION CAPABILITIES

due to Equation 6-10. After enough iterations to converge to within 5%, the maximum allowable frequency in the highest soil element next to the structure had settled at about 8.0 Hz. While this falls short of the intended 10 Hz, it is enough greater than the 5.2 Hz of Figure 6-11 that comparisons between the two sets of results should point out any deficiencies in the latter model. Power spectral density comparisons between the two meshes for several points of both the free-field and SSI analyses show that there is virtually no excitation of frequencies greater than 5.0 Hz arriving at the structure for either model, and point for point the power spectral densities compare very well with each other (App. B, AA, 1982). It is therefore concluded that the mesh of Figure 6-5 contains adequate frequency-transmission capabilities for the SSI analyses under consideration. This conclusion is reinforced by Figure 6-13, which compares the response spectra at the top of the structure for the two meshes.

As is the case with the power spectral density comparisons, there are no significant differences between the two spectra.

FIGURE 6-13. COMPARISON OF RESPONSE SPECTRA FOR N-S SSI ANALYSES INVOLVING FINITE ELEMENT MESHES OF TWO DIFFERENT LEVELS OF FINENESS

6.4 RESULTS

The results of the various soil/structure interaction (SSI) analyses performed using program FLUSH can be compared with similar results obtained using program TRI/SAC. The TRI/SAC results were reported in Chapter 5. The format chosen for presenting these results is the pseudovelocity response spectrum plotted on a log/log grid. This format provides response spectra of displacement and pseudo-acceleration using diagonal axes within the same plot. These comparisons between FLUSH and TRI/SAC results identify the importance of various modeling considerations on the SSI analyses under investigation. The use of program FLUSH allows the analyst to incorporate strain-dependent material properties into a two-dimensional model, while TRI/SAC permits three-dimensional analysis of a linearly elastic medium. There are also some differences in the boundary conditions available. Comparing the

results of these various analyses enables one to judge the importance of these modeling considerations. No attempt is made to determine whether or not one code is superior to the other, and therefore it has been judged that information such as comparative run times is not relevant to the issues at hand.

6.4.1 RESPONSE SPECTRA COMPARISONS

Comparisons of nodal response spectra for FLUSH and TRI/SAC have been made for four SSI analyses: for the N-S analysis and the E-W analysis of the symmetrical structure, and for the N-S analysis and the E-W analysis of the unsymmetrical structure. The comparisons between the results of the two codes show many similar trends for the four different SSI analyses. In each of the four cases the horizontal response is greater for the FLUSH analysis than for the TRI/SAC analysis over virtually the entire frequency range. This difference is especially large in the frequency range of from 1 to 2 Hz. In the case of the vertical components for the tops of the structures, the responses for the two programs are nearly equal for frequencies below 3 Hz. For frequencies above this value, the FLUSH response exceeds the TRI/SAC response by a considerable amount. As one approaches from the surface to the bedrock, the differences between the FLUSH and TRI/SAC results diminish until the differences appear inconsequential at close to the bedrock.

In general, response spectra comparisons for locations away from the structure centerline show a similar trend in which large differences at the top of the model become gradually smaller with increasing depth. In each of the four cases the vertical response spectrum at the edge of the structure at grade level computed from the FLUSH model exceeds that of the TRI/SAC model over the entire frequency range

(with the exception of extremely low frequencies). This is different from the comparison at any of the other locations investigated, and suggests that larger rocking modes develop in the structures of the FLUSH models than in those of the TRI/SAC models. This may be due to the extreme softening of the soil in the region adjacent to the structure in the FLUSH analyses (see Fig. 6-9). Since the TRI/SAC models are elastic, this phenomenon cannot take place in these models.

Besides differences in material properties between the FLUSH and TRI/SAC SSI models, other possible sources of discrepancy include energy-absorbing boundary conditions, and differences in the two-dimensional analysis in FLUSH and the three-dimensional analysis in TRI/SAC. The effects of two or three dimensions will be explored in the following paragraphs. In regard to the energy-absorbing boundary modeling, FLUSH uses a better formulation for SSI analysis than does TRI/SAC. As noted earlier, FLUSH uses closed-form solutions as basis for the transmitting boundaries. In contrast, TRI/SAC applies dampers to the absolute nodal motion at the boundaries, thereby eliminating incoming waves along with the outgoing waves. This modeling concept will work if the boundaries of the model are remote enough from the region of interest that incoming waves are attenuated by material damping before reaching that region. However, the models used in the TRI/SAC analyses here did not have their boundaries sufficiently far away. Comparisons of one-dimensional free-field spectra with two- and three-dimensional models with no structure showed some discrepancies using TRI/SAC (AA, 1981a). These discrepancies can be due only to the effects of the boundaries.

6.4.2 RESULTANT RIGID BODY MOTION

Soil/structure interaction analyses are customarily used to determine structural motions

so that these may then be applied to a more detailed three-dimensional model of the structure. There is concern in determining these resultant structural motions that to rely on two-dimensional SSI analyses may necessitate the omission of certain components. In fact, it is impossible to determine the resultant torsional component of structural motion about the vertical axis using two-dimensional models alone. This raises some important questions regarding how significant the torsional component is and whether or not other contributions to other components of motion might also have been overlooked by the two-dimensional analyses.

The first step in examining these issues is to determine the resultant motions, of which there are six components:

1. N-S horizontal translation
2. E-W horizontal translation
3. Vertical translation
4. N-S rocking, or rotation about the E-W axis
5. E-W rocking, or rotation about the N-S axis
6. Torsion, or rotation about the vertical axis

These resultant motions may be resolved from a collection of nodal motions using program AFIT (see Sec. 6.2.1). This program uses the horizontal and vertical components of motion for every node of a horizontal baseline. The horizontal motions of these nodes are averaged to determine the horizontal resultant, the vertical motions are averaged to determine the vertical resultant, and the rotation of each node about a specified center is averaged to obtain the resultant rocking. This program was applied to the two (N-S and E-W) FLUSH SSI analyses for a given structure to obtain five components of resultant motion (the torsion was not obtained). AFIT was also applied to the three-dimensional TRI/SAC results to obtain these five components. In addition, by applying AFIT to a

horizontally oriented plane of the three-dimensional model, the torsion was obtained for the TRI/SAC analyses.

Figures 6-14 through 6-19 show appropriate response spectra for both the symmetrical and the unsymmetrical structure. Rocking motion is shown in the latter two figures and, while perhaps not as meaningful for structural design purposes, the rocking mode of the

structure can be portrayed as an angular acceleration history instead of a contribution to a response spectrum. The advantage of this means of display is that the rocking component appears by itself, and not as an addition to some more dominant component of motion. Using this format it is hoped that any omissions due to shortcomings of the model, such as considering only two dimensions rather than three, might be readily apparent.

FIGURE 6-14. RESPONSE SPECTRA OF RESULTANT HORIZONTAL MOTION AT GRADE LEVEL OF SYMMETRICAL STRUCTURE

FIGURE 6-15. RESPONSE SPECTRA OF RESULTANT HORIZONTAL MOTION AT GRADE LEVEL OF UNSYMMETRICAL STRUCTURE

(a) N-S SSI analysis

(b) E-W SSI analysis

FIGURE 6-16. RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL OF SYMMETRICAL STRUCTURE

(a) N-S SSI analysis

(b) E-W SSI analysis

FIGURE 6-17. RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL OF UNSYMMETRICAL STRUCTURE

(a) North wall of structure

(b) East wall of structure

FIGURE 6-18. RESPONSE SPECTRA OF RESULTANT VERTICAL AND ROCKING MOTION AT OUTSIDE WALL AT GRADE LEVEL OF SYMMETRICAL STRUCTURE

(a) North wall of structure

(b) East wall of structure

FIGURE 6-19. RESPONSE SPECTRA OF RESULTANT VERTICAL AND ROCKING MOTION AT OUTSIDE WALL AT GRADE LEVEL OF UNSYMMETRICAL STRUCTURE

Figures 6-20 through 6-23 show resultant angular acceleration histories for both structures. While all of these comparisons show some differences, there are a great many similarities. In particular, peak positive and negative angular accelerations have approximately equal values in each pair of time histories. While in all cases the FLUSH histories appear to contain higher frequency response than the TRI/SAC histories, the peak pulses in each pair

of histories appear to be of approximately equal frequencies, and occur at the same times. Since the FLUSH histories result from two-dimensional analyses while the TRI/SAC histories come from three-dimensional analyses, the great similarity seen in these comparisons suggests that the rocking modes do not suffer any sizable omissions by using two-dimensional analysis.

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-20. RESULTANT ANGULAR ACCELERATION HISTORIES FOR N-S COMPONENT OF ROCKING MOTION OF SYMMETRICAL STRUCTURE

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-21. RESULTANT ANGULAR ACCELERATION HISTORIES FOR E-W COMPONENT OF ROCKING MOTION OF SYMMETRICAL STRUCTURE

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-22. RESULTANT ANGULAR ACCELERATION HISTORIES FOR N-S COMPONENT OF ROCKING MOTION OF UNSYMMETRICAL STRUCTURE

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-23. RESULTANT ANGULAR ACCELERATION HISTORIES FOR E-W COMPONENT OF ROCKING MOTION OF UNSYMMETRICAL STRUCTURE

Figure 6-24 shows two plots of the torsional acceleration history of the unsymmetrical structure as computed by TRI/SAC, plotted to the same scale as the previous plots of rocking acceleration. Note that the two-dimensional FLUSH analyses are unable to calculate this mode of resultant motion. Furthermore, although the three-dimensional TRI/SAC analysis of the symmetrical structure is capable of determining torsional motion, none was found to

be present. The two curves shown in Figure 6-24 correspond to a measurement of torsion by considering the E-W motions of points on a N-S baseline and a second measurement considering the N-S motions of points on an E-W baseline. The fact that these curves are virtually identical arises from the relative rigidity of the structure with respect to the soil. The key observation is that the torsional accelerations are very small compared to the

FIGURE 6-24. RESULTANT TORSIONAL ACCELERATION HISTORIES ABOUT VERTICAL AXIS OF UNSYMMETRICAL STRUCTURE

rocking accelerations. It is highly unlikely that such a small component of motion could be responsible for any major portion of the differences in response spectra between the FLUSH and TRI/SAC analyses. These results indicate that for the soil and structures considered herein, little is lost by considering only two-dimensional analyses. This is especially true in the case of the symmetrical structure, but also in the case of the unsymmetrical structure it appears to be a well-justified conclusion.

6.4.3 EFFECTS OF STRUCTURE CONFIGURATION

Figures 6-25 through 6-30 show comparisons of overlain response spectra of resultant motions for the FLUSH models and the TRI/SAC

models. Spectra arising from the FLUSH models are shown in part (a), while corresponding spectra from the TRI/SAC models are shown in part (b). Each plot contains four spectra, for the symmetrical structure, unsymmetrical structure, free field, and Castaic (input base motion). The spectra pertain to resultant motions determined for the nodes located along the heavy line in the inset diagram in each plot. If this heavy line contains an emphasized point along it, the resultant motion is for that point of the baseline.

A trend evident in all of these figures is that, for both FLUSH and TRI/SAC analyses, there is very little difference between the responses of the symmetrical and the unsymmetrical structures. This indicates that for the soil properties used in this study, the structural response is governed primarily by the soil response, with soil/structure interaction effects playing a less important role. The comparisons of horizontal and vertical resultant spectra computed from the FLUSH models further support this finding in that the free-field spectra show virtually no difference from the resultant spectra for the structures for frequencies below about 3 Hz. This indicates that perhaps the effect of the structure is that of a low pass filter. Power spectral density plots (see App. B, AA, 1982) at grade level in both the structure and the free field confirm that the structure does not transmit any significant frequency response above 3 Hz, and that signals in the neighborhood of 2 Hz are amplified by the structure. Boundary conditions interfering with the response of the structure prevent the same phenomena from being observed from the TRI/SAC results. However, comparisons against free-field analyses having the same boundary conditions (AA, 1981) using TRI/SAC showed very good comparisons with the response of the structures over the entire frequency range. The fact that the low-pass

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-25. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT N-S HORIZONTAL MOTION AT GRADE LEVEL

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-26. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT E-W HORIZONTAL MOTION AT GRADE LEVEL

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-27. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL FROM N-S ANALYSES

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-28. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL FROM E-W ANALYSES

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-29. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL OF NORTH WALL

(a) FLUSH analysis

(b) TRI/SAC analysis

FIGURE 6-30. COMPARISONS OF RESPONSE SPECTRA OF RESULTANT VERTICAL MOTION AT GRADE LEVEL OF EAST WALL

filtering effect is not evident in these TRI/SAC analyses may be because the finite element mesh itself serves as a low-pass filter, as can be inferred from Table 6-5. Finally, Figures 6-29 and 6-30 show that at the edge of the structure the vertical motion exceeds that of the free field. Both the FLUSH and TRI/SAC results show this, although the FLUSH results show this more strongly. This amends the previous

finding such that in addition to acting as a low-pass filter, the structure alters the free-field response primarily through the rocking mode of the structure. It is probably the presence of strain-dependent properties in the FLUSH models that causes this latter phenomenon to be noticed more strongly in the FLUSH analyses.

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