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SOIL BEHAVIOR UNDER EARTHQUAKE LOADING CONDITIONS

INTERIM REPORT NO. 1

Prepared Under

Subcontract No. 3354

UNION CARBIDE CORPORATION

NUCLEAR DIVISION

OAK RIDGE NATIONAL LABORATORY

OAK RIDGE, TENNESSEE

For

U.S. ATOMIC ENERGY COMMISSION

Contract No. W-7405-eng-26

December 1971

SW-AJA

A Joint Venture of

SHANNON & WILSON, INC. AND AGBABIAN-JACOBSEN ASSOCIATES

Seattle • Los Angeles

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FOREWORD

This report presents the results of four separate studies undertaken as part of the continuing first year work effort on the subcontract to evaluate free field soil behavior under earthquake loading conditions. The four subjects covered include 1) soil modeling studies, 2) a study of in situ testing procedures, 3) laboratory shake table feasibility studies, and 4) supplementary laboratory tests for study of liquefaction. These studies are important steps in the overall project for determination of the best methods of evaluation and prediction of soil behavior of potential nuclear power plant sites under seismic loading conditions.

This is the second report prepared by the joint venture of Shannon and Wilson, Inc. (SW) and Agbabian-Jacobsen Associates (AJA) under Subcontract No. 3354 with the Union Carbide Corporation, Nuclear Division, Oak Ridge National Laboratory, Oak Ridge, Tennessee, as a part of Contract No. W-7405-eng 26 between the Union Carbide Corporation and the United States Atomic Energy Commission.

A number of contributors from both members of the joint venture assisted in the preparation of this report. For SW, the primary contributors were Mr. S. D. Wilson, Mr. F. R. Brown, Mr. E. D. Schwantes, Mr. Mohamed S. Aggour, and Mr. Atef Azzam. Dr. R. P. Miller was the Project Engineer for SW and is also the Project Manager for the joint venture. For AJA, the primary contributors were Dr. J. W. Workman and Dr. G. A. Young, Project Engineers, assisted by Dr. Y. C. Eric Lee, Principal Investigator. Dr. M. Agbabian reviewed this report in detail, particularly Chapter 2, and provided helpful guidance for its preparation. Dr. H. Bolton Seed, Professor of Civil Engineering, University of California, who is special consultant to the joint venture on this project, provided valuable guidance in these studies and in the report preparation.

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CHAPTER 1

INTRODUCTION

This interim report presents the technical results of four of the six major work areas accomplished during the first year of subcontract work for evaluating free field soil behavior under earthquake loading conditions. These work areas are summarized in Table 1.1. Also included in this report is a descriptive plan and schedule of work for the second subcontract year. The first two areas of work have been completed and partially reported. The first report, "Soil Behavior Under Earthquake Loading Conditions, Program Definition" was submitted initially in June, 1970, and consisted of the results of literature studies, interviews, and planning (Item 1 in Table 1.1) for the purpose of defining in detail the overall approach to the subcontract objectives. This report was subsequently revised and resubmitted in November, 1970. To reflect the changes made from the original "Program Definition," the title of this report was modified to "Revised Program Definition."

The other major report, "Soil Behavior Under Earthquake Loading Conditions - State of the Art, Evaluation of Soil Characteristics for Seismic Response Analyses," which was presented in draft form in June, 1971, following a review and evaluation of pertinent procedures and data, summarizes the state-of-the-art relative to current practices, capabilities, limitations, and research and development efforts available to the engineering profession for evaluation of soil characteristics for seismic response analyses. In addition, the state-of-the-art report contains a list of symbols, a glossary of standard nomenclature, terminology and definitions, and an annotated bibliography. This report (SW-AJA, 1972) will be printed in final form in January, 1972.

The purpose of the aforementioned reports was to review current field, laboratory, and analytical procedures, and to define technical areas in which data or methods of analyses are

lacking or are insufficient. Where usual test methods were found limited or inadequate, improved techniques were sought. In searching for improved methods to verify information obtainable by existing procedures, emphasis was placed on in situ measurements.

TABLE 1.1

FIRST YEAR WORK SCHEDULE

<u>Work Area</u>	<u>Work Description</u>
1.	Literature Study & Program Definition
2.	State of the Art Review
3.	Soil Modeling & Analysis of Sand & Clay Soils
4.	Study of In Situ Testing Procedures
5.	Laboratory Shake Table Feasibility Studies
6.	Supplementary Small-Scale Laboratory Tests

This interim report presents the results of Work Areas 3 through 6 in Table 1.1. The results obtained from soil model studies and soil response calculations are presented in Chapter 2. Alternate methods of describing the one-dimensional response of soil deposits are compared using identical representations of soil behavior. The development of a nonlinear hysteretic model which directly accounts for strain dependence of the modulus and energy dissipation when direct integration solution techniques are used, is reported. The results of calculations with the nonlinear hysteretic model and an interactive procedure used with a strain dependent equivalent linear model are compared. The influence of independent variations of shear modulus and damping ratio on soil response are also reported. In addition, the response of two-dimensional soil deposits illustrating two extreme conditions are shown. The relative costs of calculations with the various methods are indicated and a preferred approach to soil response calculations outlined. Chapter 3 consists of a report on studies of possible testing devices and/or hardware, procedures, or concepts which may provide an improved method for determining the dynamic shear modulus for direct use in seismic response analyses.

Emphasis is placed on obtaining in situ measurements, utilizing larger energy sources than presently used in dynamic field test procedures, thus inducing higher strains in the soil.

The feasibility of utilizing shake tables as a means for studying soil liquefaction is treated in Chapter 4. Included in this chapter is a discussion of the development of shake tables, and a description of shake table systems. The use of shake tables and the type of information that can be obtained from them is also presented. The primary advantages and existing limitations are described, together with improvements that are necessary and potentially feasible. An alternate method for studying liquefaction with a large scale simple shear test is also described.

The results of supplementary laboratory tests are presented in Chapter 5. This limited laboratory test program was undertaken to investigate the mechanism of pore pressure buildup in a cyclic triaxial test in anticipation that this might lead to a better interpretation of the validity of such tests. These tests were conducted under various types of cyclic stress applications.

The tasks to be performed during the second subcontract year are described in Chapter 6. Included in this last chapter is a proposed schedule for accomplishing this program. The scope of work described there has been reduced from that outlined in the "Revised Program Definition" report of November, 1970, because of limitations in funding.

CHAPTER 2

SOIL MODELS AND ANALYTICAL TECHNIQUES

Analytical studies have involved a comparison of one-dimensional and two-dimensional computational methods and soil models for predicting surface motions from simulated bedrock motions. A San Francisco site and an El Centro site consisting of layers of predominantly sand and clay materials, respectively, were used for this study because they represent two types of soils where nuclear plants may be located and because strong motion accelerograph records of earthquakes were available from these sites. Limitations of the various one-dimensional models used for earthquake motion predictions with respect to modeling the mechanical properties of soils are discussed. A one-dimensional finite element analytical technique using a variable modulus hysteretic soil model is compared with two of the most frequently used one-dimensional techniques with respect to surface motion predictions for the above sites. The sensitivity of surface motion predictions to variations in soil property parameters has been investigated by use of a one-dimensional linear elastic finite element model.

A two-dimensional finite element model has been used to illustrate the effect of sloping rock-soil interfaces on surface motion for the San Francisco site, and the effect of combined vertical and horizontal shaking of the El Centro site. Limitations of the various analytical techniques and their relative cost are discussed.

2.1 BACKGROUND

Studies of the influence of soil deposits on earthquake motion have been conducted by a number of investigators. The subject is not a new one, having been of continuing interest for several decades. A certain urgency in enlarging research activities in this area has followed every major earthquake. In recent years increased levels of activity have been spurred by concern for public safety with particular regard for nuclear reactor

siting and by the occurrence of earthquakes affecting areas with modern structures.

The studies conducted to date share several common features including 1) descriptions of the soil deposit and the mechanical properties of its constituent soils, 2) analysis of records of earthquake motion and 3) simple models for soil behavior, soil deposits, and the transmission of earthquake motions through soil layers.

In the ideal situation, complete knowledge of the local and regional geology would be required, clearly defining the shape and extent of the soil deposit and its layers and the rock or rock-like materials surrounding it. In situ mechanical properties of the soils should also be available. A number of accelerograph records of strong motion collected from an array of stations covering the ground surface, rock outcrops, the soil-rock contact and some of the more interesting points within the soil deposit should complete the ideal data set.

The ideal data set would contain considerably more information than is required for a static foundation stability survey, and exceeds even that currently obtained for predicting site response under earthquake loading. Obtaining the ideal data set is, however, hampered by several difficulties, including economic limitations and nature's failure to cooperate in man's undertakings. The cost of subsurface soil investigations increases rapidly as the number and depth of borings increases. An extensive subsurface investigation program can, therefore, only be undertaken at the most promising potential sites. The cost of acquiring, installing, operating and maintaining accelerograph stations also prevents their general use. In addition, certain problems exist relative to establishing such an installation on privately held land. Finally, even in seismically active regions, the time of occurrence and the location of the epicentral site of major earthquakes is not yet predictable or controllable. The locations of existing accelerograph stations have been selected to meet other requirements so that the strong motion records

available may not provide a direct means of establishing the local influence of soils on earthquake motion. For example, accelerographs required in new high-rise construction in Los Angeles are attached to the building at basement, intermediate, and upper story levels. Their primary purpose is to monitor structural performance. Basement level records may not accurately represent soil motion, since the dynamic interaction of the structure with the adjacent soil may have some effect on both the amplitude and frequency content of the motion. Other instruments are located to take advantage of rock outcrops, remoteness from disturbing influences (city traffic, heavy industry, etc.) or to monitor conditions at large civil structures such as dams or reservoirs, whose failure could represent a major catastrophe. The resulting coverage is therefore spotty; better coverage is found in seismically active regions--Japan, Western South America, California, and Mexico. With few exceptions when new instruments become available coverage of new areas has taken precedence over multilevel (bedrock and soil surface) pairing of stations (as at Union Bay, Tokyo, San Francisco, etc.) which would allow direct measurement of the local influence of the soil deposit. For the present, an ideal data collection is not available and reliance must be placed on reasonable extrapolations from the incomplete data now in hand.

2.1.1 Mathematical Models Of Soil Behavior

The mechanical behavior of soil media is admittedly complex, exhibiting nonlinear inelastic characteristics even at strain levels which elicit nearly perfect linear elastic response in other engineering materials. As economic pressures have reduced the levels of conservatism that can be tolerated, more and more accurate means of predicting soil response have been required. Empirical rules have been supplemented by analytical methods borrowed from other fields. Linear continuum models have been supplanted by elastic-plastic, nonlinear elastic, viscoelastic, hysteretic, and other complex models of the behavior of soils. Support of theoretical investigations of this type has

come from studies of the response of protective structures to ground shock environments considerably more hostile than those of even a major earthquake. Because of the complexity of the soil models which have resulted, they have not received widespread application in the engineering problems of site selection or facility design. Other models which provide the conceptual simplicity of the linear models while retaining the major characteristics of observed nonlinear behavior are receiving much wider attention.

A brief description of the soil models more commonly used to represent the behavior of soils in one-dimensional and two-dimensional analyses for estimating soil response under earthquake loadings is presented below. A soil model has been developed which considers hysteretic losses at various strain amplitudes and confining pressures and is described in more detail in the following section of this report.

a. Linear Elastic Model

This model has constant moduli and constant external viscous damping. The damping coefficients are proportional to the undamped vibration frequencies of the soil layer. It may be adopted in one-, two-, or three-dimensional analyses. Though the constant moduli and frequency-dependent damping of this model prevent it from representing mechanical behavior of soil realistically, reasonably good results may be obtained from this model in the case of microtremors under which the strain range induced in the soils is small.

b. Linear Viscoelastic Models

This material model has constant moduli and internal viscous damping characteristics. The response is effectively that of the linear elastic model with external damping. However, the relationship between damping properties and frequency need not be linear. Like elastic models, it may be used in multi-dimensional analyses. Though it gives a more realistic description

of soil properties under vibratory loading than the elastic model, its meaningful usage is still restricted to the area where the earthquake intensity is low.

c. Bilinear Hysteretic Model

This model is used primarily in one-dimensional analyses, and is considered to be an improvement over linear models. The value of its shear modulus G is equal to either G_1 or G_2 , depending on the strain relative to the yield strain e_y (Fig. 2.1), and the sign of the load increment. This model introduces a limited amount of hysteresis if a load reversal of sufficient magnitude occurs; e.g., symmetric cyclic shear. The area within a hysteresis loop represents the energy dissipated in a stress cycle, and is, in effect, a direct measure of soil damping characteristics. Additional viscous damping may also be applied to account for energy dissipation under conditions where hysteresis loops are not formed by this model. Because a constant value is assigned to e_y , it can only be used in one-dimensional analyses. It is considered as a better model in representing soil properties under earthquakes of strong intensity. Some drawbacks, such as limitation of its damping mechanism and constant moduli, are still retained in it.

d. Elastic-Plastic Model

This model was developed from the theory of plasticity. The bilinear hysteretic model is a simplified one-dimensional model derived from this class. A detailed review of this class of material models is given by DiMaggio and Sandler, 1970, including a description of a strain hardening model. For a material like soil, a particular model was proposed by DiMaggio and Sandler. It is basically an elastic-plastic model which has a special yield surface. Used with an associated flow rule and linear elastic behavior, this model gives hysteretic stress-strain relationships under cyclicly reversing loading conditions. Because the model follows the same linear path during unloading

FIG. 2.1. STRESS-STRAIN CURVE FOR A BILINEAR MODEL

FIG. 2.2. TRIAXIAL STRESS TESTS FOR MC CORMICK RANCH SAND
(FROM DIMAGGIO AND SANDLER, 1970)

and reloading processes, no hysteretic loop will be generated under the condition where no complete unloading occurs. For example, the model unloading/reloading path A-B-A-C in Fig. 2.2 provides no hysteresis loss, although the scatter in the experimental data points appears to indicate that there was probably such a loss occurring during unloading and reloading events. Since most soils exhibit nonlinear behavior and failure develops gradually, linear elastic-perfectly plastic models are not significantly better than bilinear models. Nonlinear elastic strain hardening models more accurately describe soil behavior, but represent considerable increases in complexity and computing time requirements.

e. Equivalent Linear Model.

This model has strain-dependent shear modulus and damping coefficient which under "average" excitation provide stiffness and dissipative properties equivalent to those observed. The relationship between the modulus and damping coefficients for this model and the bilinear hysteretic model is illustrated in Fig. 2.3. Characteristic modulus and viscous damping parameter values as functions of equivalent strain amplitude are shown in Figs. 2.4 and 2.5 for typical sand and clay soils, respectively. This model is essentially an improved linear model, where the modulus and damping coefficient are constants assigned according to the peak (or average) local strain amplitude; it is more versatile than bilinear or elastic plastic models for analysis of problems where no residual displacements are involved. Success in applying it to many site earthquake response analyses has been shown (Idriss and Seed, 1968b); however, it cannot be used to evaluate residual displacements in soil deposits subjected to earthquake excitation.

2.1.2 Analysis Methods

One-dimensional analytical techniques have been used with the above soil models to predict soil surface motions from estimated bedrock seismic inputs. These methods require the

FIG. 2.3. RELATIONSHIP BETWEEN BILINEAR AND EQUIVALENT LINEAR MODULI AND DAMPING CHARACTERISTICS

FIG. 2.4. MODULI AND DAMPING RATIOS FOR MEDIUM SANDS--RELATIVE DENSITY ≈ 75 PERCENT (AFTER IDRISSE AND SEED, 1968a)

FIG. 2.5. MODULI AND DAMPING RATIOS FOR SATURATED CLAYS
(AFTER IDRISSE AND SEED, 1968a)

assumptions of semi-infinite horizontal soil layers overlaying a horizontal bedrock half-space and a single component horizontal motion time-history input at the bedrock-soil interface which is transmitted upward through the soil column to the surface in the form of shear waves. Three frequently applied one-dimensional analytical techniques are described below.

a. Lumped-Mass Shear Beam Method

This is a one-dimensional numerical method operating in the time domain for the response analysis of horizontally layered systems subjected to horizontal excitation at the base. For this method, a horizontal site consisting of soil layers having different material properties is idealized by a lumped-mass/lumped-stiffness system. The resulting matrix representation, when used with a time-marching solution technique, permits incorporation of nearly arbitrary descriptions of soil behavior. With a modal superposition solution technique, only linear or equivalent linear (a linear system representing average response characteristics of a nonlinear system) soil models are permitted; however, computing time is considerably reduced with modal superposition. In an analysis of soil response to the March 1957 San Francisco earthquake (Idriss and Seed, 1968a), a strain-dependent equivalent linear model was used with damping at each level in the lumped-mass representation provided by viscous dashpots. A rigid boundary is used at the base, but in most instances where large impedance contrasts exist, this limitation has little effect on the results (Lysmer, Seed, and Schnabel, 1970).

b. Modified Shear Beam Methods

Two modified shear beam methods which take into account the deformability of the baserock have been proposed by Tsai, 1969, and another by Roesset and Whitman, 1969.

The first model consists of a shear beam connected at its base to the input excitation by a suitable viscous dashpot, which in effect is a substitute for the half-space foundation.

This model is an exact analog of a given layered system that is nonviscous. A technique of continuous modal analysis is used to analyze the model directly. For practical purposes, usually it is sufficient to consider only a finite number of modes of the shear beam. The resultant system of equations to be solved is a set of second-order differential equations coupled with a first-order differential equation representing the motion of the dashpot. Since viscous damping will have to be included in the model if the given layered system is viscous, an iterative process is proposed for determining the modal damping directly from the transfer function of the layered system.

The second model is a further approximation of the original layered system, and it is exact if the half-space foundation is rigid. It consists of a viscous shear beam resting on a rigid foundation. The technique of continuous modal analysis is again used to analyze this model. To account approximately for the energy lost into the foundation, a certain amount of viscous damping is introduced in each model by the iterative scheme used for analyzing the first model. Therefore, there is always some viscous damping in the model. This model gives good results only for computing the response at or near the surface of a layered system which exhibits a considerable stiffness contrast at the soil foundation interface; this condition is satisfied in most cases of practical interest. The Roesset and Whitman model uses the same added modal damping to account for energy transmitted to the foundation.

c. Continuum Shear Beam Method

This method involves the solution of the one-dimensional wave equation for a viscoelastic medium as developed by Kanai, 1950. The Kanai solution provides the wave transmission coefficients (amplification spectrum) for a given one-dimensional viscoelastic layered system subjected to excitation by steady-state sinusoidal waves applied at its base. Sinusoidal steady-state analysis is capable of indicating precisely the frequency-

selective properties of a given layered system. However, it is insufficient for studying the effect of a layered system on transient earthquake-like excitation. To obtain a transient analysis of viscoelastic layered media, the wave equation is solved in the frequency domain, as above, to obtain transmission coefficients. The Fourier spectrum of the input motion is then calculated and multiplied by the corresponding wave transmission coefficients of the layered system to produce the Fourier spectrum of the output motion. The transformation of the output spectrum is then inverted numerically to obtain the time history of the output (Whitman, 1968; Roesset and Whitman, 1969; Lysmer, Seed and Schnabel, 1970). Any linear viscoelastic model may be adopted for this method. Three linear viscoelastic models frequently used in structural and continuum mechanics are shown in Fig. 2.6. Other models such as the linear hysteretic model discussed by Dobry, Whitman, and Roesset, 1971, may also be accommodated. In addition, this method readily allows the incorporation of the influence of base rock deformability into the computations.

2.1.3 Finite Element Methods

A finite element method, used for two-dimensional analysis, has been modified for use as a one-dimensional method, and is discussed in Section 2.2, which presents results obtained from this method. Of particular interest to this study are finite element codes which treat linear elastic and nonlinear and elastoplastic material behavior. By way of background, the following paragraphs present a brief description of the basis for this method of analysis.

The finite element method may be described as a numerical discretization procedure by means of which a continuum can be idealized as an assemblage of discrete elements interconnected at their nodal points, i.e., this technique permits an infinite number of degrees of freedom to be replaced by a finite-degree-of-freedom system. This finite-degree-of-freedom system will behave in essentially the same manner as the continuum.

(a) MAXWELL MODEL

(b) VOIGT-KELVIN MODEL

(c) STANDARD LINEAR MODEL

FIG. 2.6. THREE COMMONLY USED LINEAR VISCOELASTIC MODELS

The equations of motion of nodal points within the finite element system when subjected to earthquake motion can be expressed at any instant of time, t , in matrix form as follows:

$$[M]_t \{\ddot{r}\}_t + [C]_t \{\dot{r}\}_t + [K]_t \{r\}_t = \{R\}_t$$

where $[M]_t$, $[C]_t$ and $[K]_t$ are, respectively, the mass, damping and stiffness matrices at time, t , and $\{\ddot{r}\}_t$, $\{\dot{r}\}_t$ and $\{r\}_t$ are the acceleration, velocity and displacement vectors of the nodal point relative to the base, respectively.

The stiffness matrix of the system at any time, t , is obtained from an assumed displacement field within the finite elements (Clough, 1965). The mass matrix may be determined from a direct physical lumped-mass approximation (Felippa, 1966). The damping matrix takes into account the energy dissipation in the material when subjected to dynamic loading. While the finite element approach is capable of treating damping which is completely general, i.e. damping properties separately assigned to each element, uniform proportional damping of the form $[C] = \alpha[M] + \beta[K]$ is usually assumed.

The techniques, the normal mode method and the step-by-step procedure, may be applied to solve the equations of motion. The step-by-step approach can be used for problems with any material model, linear as well as nonlinear. The normal mode method can only be used for problems with linear elastic material properties, with which the superposition of modes is allowed. When modal damping, i.e., damping factors appropriate to each mode shape, is considered in the elastic analysis, the normal mode method is the better approach.

The finite element technique was first used to study the dynamic response of earth dams subjected to earthquakes (Clough and Chopra, 1966) in which linear elastic models with modal damping were considered. The seismic response of soil deposits underlain by a sloping rock boundary was evaluated using this technique (Dezfulian and Seed, 1970), where equivalent linear strain-dependent material properties were applied. The finite element technique used in earthquake analyses assuming uniform

base motion, was extended to account for spatial variations in ground motion along the base (Dibaj and Penzien, 1969b). A finite element formulation of nonlinear (elastic-plastic) dynamic response of earth structures of a general type subjected to either uniform or nonuniform base motion was also given (Dibaj and Penzien, 1969a).

To avoid difficulties with waves reflected from boundaries of the finite element mesh which are not physical boundaries of the soil deposit being modeled, "transparent" boundary techniques have been developed. When such a technique is incorporated, the transmission of signals into material outside the mesh is simulated and the so called "radiation" damping is accounted for.

2.2 . DEVELOPMENT OF A VARIABLE MODULUS HYSTERETIC SOIL MODEL

The commonly used soil models described briefly in the previous section do not accurately account for the hysteretic losses incurred when a soil is cyclically loaded at various strain amplitudes during an earthquake loading. An analytical procedure using a new variable modulus hysteretic soil model which does account for these hysteretic losses has been developed. A comparison between hypothetical laboratory test results and the computed results from the above soil model confirms its accuracy. The following paragraphs describe the theoretical approach used for development of the variable modulus hysteretic soil model and present comparisons between the hypothetical laboratory test data and computer-generated hysteresis loops for sand and clay soils. In addition typical time history and stress-strain curves obtained with this soil model are presented.

2.2.1 Theoretical Basis for a Variable Modulus Hysteretic Soil Model

Because of the freedom in specifying modulus values reflecting the current and previous stress-strain states of the material, a variable modulus hysteretic model was chosen to account for the variations related to changes in peak strain amplitude and confining pressure. Three loading conditions are

recognized--virgin loading, unloading, and reloading--so that somewhat simpler mathematical descriptions of the stress-strain paths may be developed. Stress-strain relations and the loading conditions are defined as follows:

a. Virgin Loading

$$\dot{s}_{ij} = 2G_{VL} \dot{e}_{ij} \quad \text{if} \quad |I_2'| > |I_2'|_{\max}$$

$$\dot{I}_2' \text{ is positive}$$

$$\dot{p} = 3K_{VL} \dot{e} \quad \text{if} \quad |I_1| > |I_1|_{\max}$$

$$\dot{I}_1 \text{ is positive}$$

b. Unloading

$$\dot{s}_{ij} = 2G_{UL} \dot{e}_{ij} \quad \text{if} \quad |I_2'| < |I_2'|_{\max}$$

$$\dot{I}_2' \text{ is negative}$$

(2.1)

$$\dot{p} = 3K_{UL} \dot{e} \quad \text{if} \quad |I_1| < |I_1|_{\max}$$

$$\dot{I}_1 \text{ is negative}$$

c. Reloading

$$\dot{s}_{ij} = 2G_{RL} \dot{e}_{ij} \quad \text{if} \quad |I_2'| < |I_2'|_{\max}$$

$$\dot{I}_2' \text{ is positive}$$

$$\dot{p} = 3K_{RL} \dot{e} \quad \text{if} \quad |I_1| < |I_1|_{\max}$$

$$\dot{I}_1 \text{ is positive}$$

in which I_1 , I_2 , $\dot{I}_1$, and $\dot{I}_2$ are the first strain invariant, the second invariant of deviatoric strain, and their increments, respectively; $|I_1|_{\max}$ and $|I_2|_{\max}$ are the absolute maximum values of the invariants registered in the material in previous loading steps; G_{VL} , K_{VL} , G_{UL} , K_{UL} , G_{RL} , and K_{RL} , functions of the strain invariants, are shear and bulk moduli during virgin loading, unloading, and reloading, respectively; and p , s_{ij} , and e_{ij} are the mean stress ($\frac{1}{3} \sigma_{kk}$), the stress deviator, and the strain deviator.

The total stress in the solid σ_{ij} , the mean stress p , and the stress deviator s_{ij} are related as follows:

$$\sigma_{ij} = \delta_{ij} p + s_{ij} \quad (2.2)$$

where δ_{ij} is the Kronecker delta ($\delta_{ij} = 1$, $i = j$; $\delta_{ij} = 0$, $i \neq j$). The mathematical forms of expressions for the moduli G , K in Equation 2.1 are chosen to fit the hypothetical experimental data.

At the present time, stress-strain curves representing some symmetric cyclic shear tests for both sands and clays have been constructed. These curves represent a direct shear test, where a constant uniform confining pressure is applied to the soil sample, while cyclic shearing forces are imposed on two parallel surfaces of the sample. In the case considered, the shear modulus is clearly a function of the shear strain and the confining pressure. For the conditions of the test and for application in shear beam analyses, only the shear behavior need be considered, and Equation 2.1 can be simplified to yield:

$$\dot{\tau} = \begin{cases} G_{VL} \dot{\gamma} & \left\{ \begin{array}{ll} \text{if } |\gamma| > |\gamma|_{\max} & \text{Virgin Loading} \\ \text{and } \dot{\gamma} > 0 \end{array} \right. \\ G_{UL} \dot{\gamma} & \left\{ \begin{array}{ll} \text{if } |\gamma| < |\gamma|_{\max} & \text{Unloading} \\ \text{and } \dot{\gamma} < 0 \end{array} \right. \\ G_{RL} \dot{\gamma} & \left\{ \begin{array}{ll} \text{if } |\gamma| < |\gamma|_{\max} & \text{Reloading} \\ \text{and } \dot{\gamma} > 0 \end{array} \right. \end{cases} \quad (2.3)$$

where $\dot{\tau}$ and $\dot{\gamma}$ are increments of shear stress and engineering shear strain, respectively, and $|\gamma|$ and $|\gamma|_{\max}$ are the absolute values of the engineer-shear strain and its maximum value registered in the previous steps.

Careful examination of the stress-strain curves suggests that a piecewise linear virgin loading curve and parabolic unloading-reloading curves would give reasonably accurate representations of stress-strain behavior of the soil during the three loading stages. The mathematical equations of this stress-strain law are given as follows:

$$\tau = k_i \gamma; |\gamma_{i-1}| < |\gamma| < |\gamma_i| \quad \text{for virgin loading conditions} \quad (2.4)$$

$$\tau = c + a\gamma + b\gamma^2 \quad \text{for reloading and unloading conditions}$$

in which k_i has a constant value over a prescribed range of strain (piecewise linear), and a , b , and c are constants for a continuous unload or reload path. The values of a , b , and c are functions of loading type, overburden pressure, and stress and strain values at the time when the load increment changes sign. By differentiating the shear stress expressions in Equation 2.4 with respect to strain, the following equations are obtained:

$$\frac{d\tau}{d\gamma} = k_i \quad \text{for virgin loading} \quad (2.5)$$

$$\frac{d\tau}{d\gamma} = a + 2b\gamma \quad \text{for unloading and reloading}$$

Comparing Equations 2.3 and 2.5 allows the obvious identification of the shear moduli as follows:

$$\begin{aligned}
G_{VL} &= k_i \\
G_{UL} &= a_U + 2b_U\gamma \\
G_{RL} &= a_R + 2b_R\gamma
\end{aligned} \tag{2.6}$$

The dependence of a and b on peak strain may be evaluated from the steady cyclic shear test data for different strain levels. The strain levels selected for defining a and b should cover the ranges of stress and strain which the soil may experience during earthquake shaking at amplitudes of interest. In the present application, a and b are expressed as polynomial functions of the peak strain in the form:

$$\begin{aligned}
a &= \sum_{i=1}^n \alpha_i \gamma^i = \alpha_1 \gamma + \alpha_2 \gamma^2 + \dots + \alpha_n \gamma^n \\
b &= \sum_{i=1}^n \beta_i \gamma^i = \beta_1 \gamma + \beta_2 \gamma^2 + \dots + \beta_n \gamma^n
\end{aligned} \tag{2.7}$$

where γ is the maximum strain in a steady-stress cycle and n is the number of strain amplitudes for which data were obtained. The coefficients α_i and β_i are determined from experimental data from tests at n -values of peak strain, $\gamma_1, \dots, \gamma_n$, which provide $2n$ equations for the $2n$ unknowns. Thus, in matrix form

$$\begin{Bmatrix} a_1 \\ \vdots \\ a_n \end{Bmatrix} = \begin{Bmatrix} \gamma_1^1 & \gamma_1^i & \gamma_1^n \\ \vdots & \vdots & \vdots \\ \gamma_n^1 & \gamma_n^i & \gamma_n^n \end{Bmatrix} \begin{Bmatrix} \alpha_1 \\ \vdots \\ \alpha_n \end{Bmatrix} \tag{2.8}$$

$$\begin{Bmatrix} b_1 \\ . \\ . \\ . \\ . \\ b_n \end{Bmatrix} = \begin{Bmatrix} \gamma_1^1 & \gamma_1 & \gamma_1^n \\ . & . & . \\ . & . & . \\ . & . & . \\ \gamma_n^1 & \gamma_n^i & \gamma_n^n \end{Bmatrix} \begin{Bmatrix} \beta_1 \\ . \\ . \\ . \\ . \\ \beta_n \end{Bmatrix} \quad (2.8)$$

Upon inverting the above matrix, the coefficients α_i and β_i are obtained in abbreviated matrix form:

$$\{\alpha\} = [\gamma]^{-1}\{a\} \quad (2.9)$$

$$\{\beta\} = [\gamma]^{-1}\{b\}$$

At actual sites, mechanical properties of a soil of uniform composition may vary due to the increase of overburden pressure with depth. To accommodate such variations of material properties, data should be obtained for a number of representative depths (or confining pressures). Coefficients α_i and β_i would then be determined as functions of both peak strain and mean stress or depth.

2.2.2. Computer-Generated Cyclic Stress-Strain Curves Using The Variable Modulus Hysteretic Soil Model

In order to demonstrate the curve fitting procedures used in the variable modulus hysteretic soil model, cyclic stress-strain curves were generated numerically for the sites used in this study, i.e., the San Francisco State Building site and the El Centro site, and compared with the hypothetical data for the soils at these sites (see Section 2.3.1 for a description of the sites from which the model was derived). Hypothetical stress-strain paths for steady state dynamic cyclic shear tests are shown in Fig. 2.7 for sands at three depths at the State Building site; similar data for clay soils at two depths at the El Centro site

FIG. 2.7. HYPOTHETICAL STRESS STRAIN CURVES FOR DYNAMIC CYCLIC SHEAR TESTS OF SANDY SOILS FROM THE SAN FRANCISCO STATE BUILDING SITE

are shown in Fig. 2.8. Computed stress-strain curves for the sands and clays from the same sites at corresponding depths, shown in Figs. 2.9 and 2.10, respectively, illustrate the accuracy of the model curve fitting and the ability of this model to generate hysteresis loops under the partial removal and reapplication of the load. The maximum deviations between the stress-strain paths corresponding to the model and data, in most cases, are less than 5 percent; one hysteresis loop for sand at a depth of 150 ft has a maximum difference of 9 percent.

The method used to obtain the input data for the computed cyclic shear stress-strain curves shown in Figs. 2.9 and 2.10 is described in detail in the following paragraphs.

Soil data, i.e., hypothetical stress-strain curves for three strain amplitudes at three different depths for the State Building site, were derived from the data presented in Fig. 2.4. The energy losses represented by the area enclosed by each hysteresis loop are related to the damping curve indicated as Curve B in the figure. The coefficients describing the shear modulus of soil at the three depths were then obtained by applying Equations 2.8 and 2.9. For depths other than these for which soil property data were given, values of a and b at each strain level were obtained by employing an interpolation (curve fitting) technique. A parabolic dependence on depth was assumed in the interpolation process, i.e.,

$$\begin{aligned} a &= f(y, \gamma^2) \\ b &= g(y, \gamma^2) \\ k &= h(y, \gamma^2) \end{aligned} \quad (2.10)$$

where a , b , and k are material property constants of soil layer at depth y for a specific strain amplitude, and f , g , and h are functions of depth y .

Equation 2.10 may be solved either by hand or by computer. To achieve better accuracy and eliminate human calculation error, a simple computer program was written to compute the constants

FIG. 2.8. HYPOTHETICAL SHEAR STRESS-STRAIN CURVES FOR DYNAMIC CYCLIC SHEAR TESTS ON CLAY SOILS FROM THE EL CENTRO SITE SOIL

(a) SOIL AT 15-FT DEPTH

(b) SOIL AT 65-FT DEPTH

FIG. 2.9. COMPUTER SIMULATION OF CYCLIC SHEAR TESTS OF SAN FRANCISCO STATE BUILDING SITE SOILS AT THREE DEPTHS

(c) SOIL AT 150-FT DEPTH

FIG. 2.9. (CONTINUED)

(a) SOIL AT 25-FT DEPTH

(b) SOIL AT 75-FT DEPTH

FIG. 2.10. COMPUTER SIMULATION OF CYCLIC SHEAR TESTS AT EL CENTRO SITE SOILS AT TWO DEPTHS

describing the soil properties at all desired levels from Equation 2.10. For the soil profile under the State Building Site, a total of ten soil layers with different material properties and overburden pressures were used; material property coefficients defined by Equation 2.8 for each layer are tabulated in Table 2.1. It is noted that a and b values are listed for only two strain amplitudes in Table 2.1, as the values for the lowest strain level have no significant influence on the model representation.

In the case of the El Centro site, soil data for four strain levels at two different depths were prepared. The hypothetical stress-strain curves obtained are based on the data shown in Fig. 2.5. By using a straight-line interpolation technique, the soil properties for other desired depths were computed as listed in Table 2.2. As in the case of the soil model for the San Francisco State Building site, the constants corresponding to the lowest strain levels for the El Centro site are not listed.

The soil material model expressed by Equations 2.3 through 2.9 was coded into a computer program which was used as the material properties package in the INDEPS Code. INDEPS is an inelastic, finite element code developed at AJA for dynamic response analyses. The material constants listed in Tables 2.1 and 2.2 were the soil property input data used with the INDEPS Code to calculate soil response at the San Francisco State Building site and El Centro site, respectively.

At the beginning of earthquake motion, the stress in an element is based on the modulus given by the virgin loading path. If the incremental shear strain and total shear strain obtained from the finite element analysis in each subsequent time step satisfy the first relation of Equation 2.3, then the virgin loading relation (see Equation 2.6) is used to evaluate the shear stress in the element. The virgin loading curve traveled by the material in this period is represented by the path O-A in Fig. 2.9(a). If at point A the computed shear strain and incremental shear strain in the element satisfy the second relation of Equation 2.3, then the material of the element is considered to be in the unloading stage.

TABLE 2.1

INPUT DATA FOR HYSTERETIC SOIL MODEL, SAN FRANCISCO SITE

Soil Layer No.	Depth, Ft	a_1^*	b_1	γ_1	a_2	b_2	γ_2	k_1 , psi	k_2 , psi	k_3 , psi	k_4 , psi	k_5 , psi
								$\gamma \leq 10^{-5}$	$10^{-5} < \gamma \leq 0.3 \times 10^{-4}$	$0.3 \times 10^{-4} < \gamma \leq 10^{-4}$	$10^{-4} < \gamma \leq 0.3 \times 10^{-3}$	$0.3 \times 10^{-3} < \gamma$
1	10	0.12×10^{-3}	0.85×10^{-5}	10^{-4}	0.33×10^{-3}	0.31×10^{-4}	0.3×10^{-3}	0.21×10^5	0.2×10^5	0.14×10^5	0.69×10^4	0.19×10^4
2	30	0.12×10^{-3}	0.82×10^{-5}	10^{-4}	0.33×10^{-3}	0.31×10^{-4}	0.3×10^{-3}	0.25×10^5	0.22×10^5	0.18×10^5	0.84×10^4	0.23×10^4
3	50	0.12×10^{-3}	0.79×10^{-5}	10^{-4}	0.33×10^{-3}	0.31×10^{-4}	0.3×10^{-3}	0.29×10^5	0.24×10^5	0.21×10^5	0.97×10^4	0.27×10^4
4	70	0.12×10^{-3}	0.78×10^{-5}	10^{-4}	0.33×10^{-3}	0.32×10^{-4}	0.3×10^{-3}	0.32×10^5	0.26×10^5	0.24×10^5	0.11×10^5	0.31×10^4
5	90	0.12×10^{-3}	0.78×10^{-5}	10^{-4}	0.33×10^{-3}	0.33×10^{-4}	0.3×10^{-3}	0.35×10^5	0.29×10^5	0.25×10^5	0.12×10^5	0.34×10^4
6	110	0.12×10^{-3}	0.79×10^{-5}	10^{-4}	0.33×10^{-3}	0.34×10^{-4}	0.3×10^{-3}	0.37×10^5	0.31×10^5	0.26×10^5	0.12×10^5	0.36×10^4
7	130	0.12×10^{-3}	0.81×10^{-5}	10^{-4}	0.33×10^{-3}	0.35×10^{-4}	0.3×10^{-3}	0.39×10^5	0.34×10^5	0.26×10^5	0.13×10^5	0.38×10^4
8	150	0.12×10^{-3}	0.84×10^{-5}	10^{-4}	0.33×10^{-3}	0.37×10^{-4}	0.3×10^{-3}	0.4×10^5	0.37×10^5	0.25×10^5	0.13×10^5	0.41×10^4
9	170	0.12×10^{-3}	0.87×10^{-5}	10^{-4}	0.33×10^{-3}	0.39×10^{-4}	0.3×10^{-3}	0.41×10^5	0.41×10^5	0.23×10^5	0.13×10^5	0.41×10^4
10	190	0.12×10^{-3}	0.92×10^{-5}	10^{-4}	0.33×10^{-3}	0.41×10^{-4}	0.3×10^{-3}	0.44×10^5	0.44×10^5	0.21×10^5	0.13×10^5	0.42×10^4

*Subscripts 1 and 2 on a, b, and γ denote two different strain levels.

TABLE 2.2

INPUT DATA FOR HYSTERETIC SOIL MODEL, EL CENTRO SITE

Soil Layer No.	Depth, Ft	a_1^*	b_1	γ_1	a_2	b_2	γ_2	k_1 , psi	k_2 , psi	k_3 , psi	k_4 , psi	k_5 , psi
								$\gamma \leq 0.25 \times 10^{-4}$	$0.25 \times 10^{-4} < \gamma \leq 0.1 \times 10^{-3}$	$0.1 \times 10^{-3} < \gamma \leq 0.3 \times 10^{-3}$	$0.3 \times 10^{-3} < \gamma \leq 0.8 \times 10^{-3}$	$\gamma \leq 0.8 \times 10^{-3}$
1	10	0.86×10^{-3}	0.7×10^{-4}	0.8×10^{-3}	0.21×10^{-2}	0.16×10^{-3}	0.2×10^{-2}	0.11×10^5	0.82×10^4	0.52×10^4	0.31×10^4	0.15×10^4
2	30	0.86×10^{-3}	0.68×10^{-4}	0.8×10^{-3}	0.21×10^{-2}	0.16×10^{-3}	0.2×10^{-2}	0.17×10^5	0.13×10^5	0.82×10^4	0.49×10^4	0.25×10^4
3	50	0.86×10^{-3}	0.64×10^{-4}	0.8×10^{-3}	0.21×10^{-2}	0.15×10^{-3}	0.2×10^{-2}	0.24×10^5	0.17×10^5	0.11×10^5	0.67×10^4	0.34×10^4
4	70	0.86×10^{-3}	0.6×10^{-4}	0.8×10^{-3}	0.21×10^{-2}	0.15×10^{-3}	0.2×10^{-2}	0.3×10^5	0.22×10^5	0.14×10^5	0.86×10^4	0.45×10^4
5	90	0.86×10^{-3}	0.58×10^{-4}	0.8×10^{-3}	0.21×10^{-2}	0.14×10^{-3}	0.2×10^{-2}	0.36×10^5	0.27×10^5	0.17×10^5	0.1×10^5	0.5×10^4

*Subscripts 1 and 2 on a, b, and γ denote two different strain levels.

The shear stress of the element is then computed from the shear strain and G_{UL} . It is noted that G_{UL} in Equation 2.6 is a function of a_U , b_U , and shear strain γ . Since a_U and b_U are given as functions of peak strain registered in a previous time step, the constants a_U and b_U remain unchanged in a continuous unloading path. This unloading path is represented by curve A-B in Fig. 2.9(a). If the computed incremental shear strain and shear strain at the point B fulfill the third relation in Equation 2.3, the shear modulus in this element is given by G_{RL} of Equation 2.6. The reloading modulus G_{RL} is calculated in the same manner as that for the unloading phase. The reloading curve for the material is shown by B-A of Fig. 2.9(a). It should be pointed out here that the A and A' should be the same point on the stress-strain curve, but due to numerical integration errors these two points are not coincident in the plot. The discrepancy is considered small, and should not affect the results of the analysis. The loading history described above gives a case where complete reversal of loading (OAB) and reloading (BA') of the material is presented; the area enclosed by these paths gives the energy dissipated in the process. If a positive strain increment is obtained at the point A', then the stress-strain relationship of the material will again follow the virgin loading path, A'-C, until a change of loading condition occurs. The curves C-D and D-C' in Fig. 2.9(a) represent, respectively, the unloading and reloading paths of the material within a small strain range where stress reversal does not occur. It is noted that the unloading and reloading curves, unlike those shown by an elastic-plastic model, do not share the same path; the area enclosed between them represents an energy loss in the material.

2.2.3 Computed Soil Responses Using The Variable Modulus Hysteretic Soil Model

This section describes the application of a one-dimensional finite element computational method with the variable modulus hysteretic soil model to calculate soil response time histories and stress-strain paths for the two sites mentioned

previously. The earthquake bedrock motions used for inputs for these calculations are the same as those used for the studies reported in Sections 2.3 and 2.4 of this report (see Section 2.3.2 for a discussion of the earthquake records).

a. San Francisco State Building Site

The soil profile for the State Building site was divided into 40 equal finite elements, grouped in ten uniform 20-ft layers. Properties were assigned according to the confining pressure at the layer depth by interpolating or extrapolating from the data for depths of 15, 65, and 150 ft. Horizontal roller supports were provided at each nodal point to limit the response of the two-dimensional elements to that representing simple SH-wave excitation. Although it is common practice in linear finite element analyses to select element dimensions so that wave transit times, $t_t = \frac{\Delta Z}{c_s}$, are constant, the variation of shear wave velocity $c_s (= \sqrt{G/\rho})$ with strain amplitude prevents satisfaction of such a requirement in the present problem. To satisfy the requirements for numerical stability and resolution of the shortest vibration period of interest, the integration time step Δt must meet the conditions

$$\Delta t \leq t_t \quad (2.11a)$$

$$\Delta t \leq T/4 \quad (2.11b)$$

where t_t is the transit time $\Delta Z/c_s$, ΔZ is the smallest linear dimension of the element, c_s is the maximum shear wave velocity, and T is the shortest vibration period of interest. The maximum shear wave velocity is encountered at small strain amplitude during the initial phase of unloading or reloading. A pilot calculation indicated a peak modulus value of 146,000 psi ($c_s \approx 2300$ fps) and a maximum allowable time step of 0.0022 sec. An examination of ground motion records by Hudson and Housner, 1958, suggests that the shortest vibration period which must be considered is of the order of 0.2 sec. The condition given by Equation 2.11b is thus clearly satisfied by any time step less than 0.05 sec.

An integration time step of 0.002 sec was chosen, although pilot calculations indicated signs of instability during the early period of low-amplitude shaking. A clearly stable result was obtained with a time step of 0.001 sec, but the need to calculate soil response for several seconds of real time made such a choice for the nonlinear soil model uneconomical. Selected results obtained from 4 sec of real time motion computed with a time step of 0.002 sec are shown in Figs. 2.11 through 2.14. Evidence of a transient numerical instability can be seen in the shear stress records (Figs. 2.13 and 2.14) for times t between 0.25 and 0.50 sec.

A stress-strain path for Element 3 covering about 1 sec of real time is shown in Fig. 2.15. The initial point on this curve corresponds to a time 1.5 sec after the beginning of the earthquake motion. The area enclosed in the complex loop represents energy dissipated in the soil during the time period viewed, $1.5 < t < 2.48$ sec. Note that energy is dissipated in nonsymmetric load-unload-reload cycles, as well as in the symmetric cycles usually treated.

b. El Centro Site

Calculations for the El Centro site paralleled those for the State Building site. To avoid the overresolution (and resulting increase in computing cost) relative to the minimum period of interest, the finite element dimensions were increased from the 5 ft used for the State Building site to 10 ft and the integration time step from 0.002 to 0.005 sec. The 100-ft-thick deposit of clays was considered as five uniform layers, each 20 ft thick. Material properties were assigned according to the depth of the layer by extrapolating from the data for depths of 25 and 75 ft. The calculation was carried to a real time of 5 sec using the time and amplitude scaled N-S component of the 1940 El Centro earthquake. Selected data are shown in Figs. 2.16 through 2.19 and a stress-strain path (hysteresis loop) is shown in Fig. 2.20.

2.3 COMPARISON OF ONE-DIMENSIONAL RESPONSE PREDICTIONS

As was mentioned earlier, one of the chief difficulties in verifying the adequacy of analytical techniques for predicting

FIG. 2.11. COMPUTED GROUND SURFACE VELOCITY TIME HISTORY
AT STATE BUILDING SITE

FIG. 2.12. COMPUTED GROUND SURFACE ACCELERATION TIME HISTORY
AT STATE BUILDING SITE

FIG. 2.13. COMPUTED SHEAR STRESS NEAR THE BASE OF THE SOIL COLUMN (ELEMENT 3) AT STATE BUILDING SITE

FIG. 2.14. COMPUTED SHEAR STRESS AT THE MIDPOINT OF THE SOIL COLUMN (ELEMENT 21) AT STATE BUILDING SITE

FIG. 2.15. CALCULATED STRESS-STRAIN PATH (HYSTERESIS LOOP) FOR ELEMENT 3,
AT STATE BUILDING SITE

FIG. 2.16. COMPUTED GROUND SURFACE VELOCITY TIME HISTORY

FIG. 2.17. COMPUTED GROUND SURFACE ACCELERATION TIME HISTORY

FIG. 2.18. COMPUTED SHEAR STRESS NEAR THE BASE OF THE SOIL COLUMN (ELEMENT 2)

FIG. 2.19. COMPUTED SHEAR STRESS AT THE MIDPOINT OF THE SOIL COLUMN (ELEMENT 5)

FIG. 2.20. CALCULATED STRESS-STRAIN PATH (HYSTERESIS LOOP) FOR ELEMENT 2

soil behavior under seismic loading is the lack of strong-motion accelerograph records for corresponding bedrock/soil interface and soil surface locations at sites representative of those used for power plant siting. For this study, a compromise was made by modifying strong-motion surface accelerograph records to obtain base inputs for two sites where strong-motion surface records were obtained, and where the soils were of interest for this program. The sites were selected because the soil deposits were composed predominantly of either sands or clays, the soils of interest during this phase of the study. The soil data from the State Building site and the surface accelerograph records obtained at the State Building site and the Golden Gate Park site during the San Francisco earthquake of 22 March, 1957, were used as the basis for examining the response of a predominantly sandy soil profile. The soil data and surface accelerograph records obtained at an Imperial Valley Irrigation District Substation site during the El Centro earthquake of 18 May, 1940, were used as the basis for examining the response of a predominantly clay soil profile.

2.3.1 Soil Properties For The Two Sites

The San Francisco State Building site consists mostly of sandy soil layers. The soil depth is approximately 200 ft with the water table located 20 ft below the ground surface level. Fig. 2.21 shows soil layer thicknesses and soil descriptions, as well as the variation of soil modulus with depth at strain amplitudes compatible with those estimated for the 1957 earthquake, obtained from Idriss and Seed, 1968a.

At the El Centro site the soil profile consists of 100 ft of stiff clays underlain by several thousand feet of sediments having a shear wave velocity of 2800 fps. Soil materials with seismic velocities of this magnitude have been considered as rock-like materials. The water table is located about 7 ft below the ground surface. Studies of signal transmission have, therefore, been limited to the clays in the upper 100 ft. Fig. 2.22 shows the corresponding soil classification, Poisson's ratio and

FIG. 2.21. SOIL PROPERTIES AT SAN FRANCISCO STATE BUILDING SITE (AFTER IDRIS AND SEED, 1968a)

L	V _s WAVE SPEED FPS	DENSITY LB/FT ³	T* SEC	H FEET	G 10 ⁶ PSF	DEPTH FEET
1	716	125	0.007	10	2.0	0
2	555	125	0.009	10	1.2	10
3	468	125	0.011	30	0.85	20
4	820	125	0.006	10	2.6	30
5	804	125	0.006	10	2.5	40
6	774	125	0.007	30	2.32	50
7	6500	125	0	--	--	60
						70
						80
						90
						100

* RETARDATION TIME FOR KELVIN-VOIGT MODEL
 $T = \eta/\mu_0$ (SEE FIG. 2.6)

(a) PROPERTY VALUES USED FOR KANAI'S METHOD

(b) SOIL CLASSIFICATION & PROFILE

(c) PROPERTIES USED IN CALCULATIONS

FIG. 2.22. SOIL PROPERTIES AT EL CENTRO SITE
 (FROM SEED et al, 1969)

strength for this site. Modulus values compatible with the calculated strain amplitudes were obtained from the data shown in Fig. 2.5.

2.3.2 Earthquake Records for the Two Sites

For this study, it was necessary to develop suitable estimates of the bedrock earthquake motion corresponding to the surface motions recorded at each site. Unfortunately, bedrock motion records have not been obtained at points below soil surface accelerograph stations.*

a. Records of the 1957 San Francisco Earthquake

Idriss and Seed, 1968a, suggested that the accelerograph records obtained at a rock outcrop in Golden Gate Park during the 1957 earthquake be scaled and used for the bedrock motions under the State Building site; scaling would account for attenuation due to the difference in distance from the earthquake epicenter. It was suggested that the S80E horizontal component of the Golden Gate State Park record be reduced to 80 percent of its recorded amplitude to allow for the additional attenuation expected at the range from the epicenter corresponding to the State Building site (Fig. 2.23). This procedure was adopted and the scaled S80E record with a peak acceleration of 0.1 g was used as the excitation in all numerical studies of the State Building site reported here.

It should, however, be noted that alignment of the accelerographs at the Golden Gate Park site (N10E, S80E) and the State Building site (S09E, S81W) differ by 19 deg. The soil profiles and surface elevations are quite varied between these two sites, and the angle between lines connecting the two sites and the earthquake epicenter is approximately 25 deg. Thus,

*Records of low amplitude shaking have been obtained from the three-level installation at Union Bay (Seattle, Washington) (Tsai and Housner, 1970; Seed and Idriss, 1970a) from a pair of two-level stations near Tokyo (Shima, 1962) and at another site in Japan (Kanai, et al., 1966). A multilevel station has also been installed in the San Francisco Bay region in connection with the new rapid transit system (Aisiks and Tarshansky, 1968).

FIG. 2.23. HYPOTHETICAL SUBSURFACE BEDROCK MOTION USED AS INPUT
MOTION IN CALCULATIONS

existence of other differences between the outcrop record and the unknown bedrock motion beneath the soil deposit, such as differences in frequency content and the direction of dominant motion, is quite likely.

The acceleration and velocity response spectra based on 2.5 percent structural damping and the two horizontal components of motion recorded by the accelerograph at the State Building site are shown in Figs. 2.24 and 2.25, respectively. The value of the damping ratio (0.025) was used for preparing acceleration and pseudovelocity spectra, because this value has been most generally used by other workers in this field (e.g., Idriss and Seed, 1968a). Differences between spectral amplitudes for the two components at shorter wave periods is quite noticeable.

b. El Centro 1940 Earthquake Records

For the 1940 El Centro earthquake, only surface records were obtained at El Centro. Seed, Idriss and Kiefer, 1969, suggested that a scaled surface record be used for the bedrock motion. The scaling technique considers the distance of the site from the epicenter of the earthquake, the intensity and predominant period of the motion, and the soil profile. Following the detailed procedure described by Seed, Idriss and Kiefer, 1969, the estimated bedrock motions for the El Centro site were obtained by multiplying the amplitudes of the surface motion by the factor $20/32$ and by multiplying the time scale of the surface motion by the factor $30/55$. The modified acceleration time history thus obtained and used for bedrock motions for the El Centro earthquake in this study is shown in Fig. 2.26. The corresponding surface motion record for the El Centro site is shown in Fig. 2.27.

2.3.3 Results From Three One-Dimensional Methods

Surface motion at the San Francisco State Building Site and the El Centro Site were computed by three one-dimensional methods using the assumed bedrock motions developed by Idriss and Seed, 1968a, and Seed, Idriss and Kiefer, 1969, as discussed above.

FIG. 2.24. ACCELERATION SPECTRA FOR TWO HORIZONTAL COMPONENTS RECORDED AT STATE BUILDING

FIG. 2.25. PSEUDO VELOCITY SPECTRA FOR TWO HORIZONTAL COMPONENTS RECORDED AT STATE BUILDING

FIG. 2.26. HYPOTHETICAL BEDROCK MOTION OBTAINED FROM SCALING THE GROUND SURFACE RECORD

FIG. 2.27. MEASURED GROUND SURFACE ACCELERATION TIME HISTORY

The models of soil behavior used with the three methods have some basic differences. The lumped mass shear beam model (Idriss and Seed, 1968a) uses an iterative procedure to obtain compatibility between the peak strain dependent equivalent linear modulus profile and the computed peak strains; uniform viscous damping related to the peak strains is applied (a later version of the shear beam code (Idriss and Seed, 1970) permits separate specification of the damping for each spring-lumped mass element). Kanai's method, as applied, represents the soil as a Kelvin-Voigt solid with viscous damping specified for each layer. The finite element code used employs a linear elastic model with uniform proportional damping equivalent to that of the Kelvin-Voigt solid. Differences in the specification of damping can affect the response of the soil deposits as has been previously noted (Idriss and Seed, 1970; Dobry, Whitman and Roesset, 1971). If the soil representations adopted for the methods are identical, then as the resolution of the lumped parameter and finite element models is improved (by increasing the number of masses or elements) the computed soil response will approach that obtained from a continuum model, e.g., Kanai's method. In the following comparisons differences are due to numerical approximations and differences in converting reported soil properties to model parameters, primarily with respect to the number of layers, masses or elements used to represent a soil profile. In an attempt to reduce the bias resulting from these differences, the modulus profile obtained by Idriss and Seed, 1968a, using iterative techniques was used to establish soil modulus values. In effect then, the linear elastic viscous damped models used with Kanai's method and the finite element method were peak strain compatible equivalent linear models.

Because of the imperfect nature of the sites considered, the sandy soils at the San Francisco State Building site contained significant amounts of clay, silt, and gravel and the clay soils at the El Centro site contained sands and silts--there is a need to modify the properties of the simple soils to compensate for the imperfections. Because of differences of opinion regarding such

corrections and because of continuing improvement in understanding soil behavior, the properties ascribed to these two sites at various times and by various investigators have differed. In the following comparisons two different sets of properties have been used for the San Francisco site.

Results of the calculations are compared on the basis of peak surface acceleration, acceleration and pseudo-velocity response spectra prepared from the computed surface motion, and ratios of surface acceleration and pseudo-velocity response spectra to bedrock response spectra (approximate amplification spectra). Such comparisons allow an easy assessment of the impact of model differences on design requirements for the specific input considered, as well as a picture of the response characteristics of the soil deposit which are less dependent on the detailed properties of the bedrock shaking. All response spectra are based on 2.5 percent of critical damping.

The ground surface motions obtained from lumped mass and continuum shear beam models are compared in Figs. 2.28(a) and (b). The acceleration response spectra shown in Fig. 2.28(a) are in good agreement. The differences in peak ground surface acceleration--0.112g for the lumped mass model versus 0.124g for the continuum model--is apparently due to the slight differences in response for periods near 0.25 sec. These differences may in part be due to the differences between the number of masses, 22, and the number of continuum layers, 8, used to represent the site. The velocity spectra shown in Fig. 2.28(b) which tend to amplify differences between model behavior at longer periods, also show very good agreement.

Response spectra for ground surface motion obtained from continuum and finite element shear beams are compared in Figs. 2.28(c) and (d). The soil properties used for these problems are more properly those of sands, in that the additional damping and increased stiffness normally introduced to account for the presence of clay and gravel have not been taken into account. Primarily as a result of the lower damping, larger strain amplitudes

(a) ACCELERATION RESPONSE SPECTRA

(b) VELOCITY RESPONSE SPECTRA

FIG. 2.28. COMPARISON OF GROUND SURFACE MOTION FROM LUMPED MASS AND CONTINUUM

(c) ACCELERATION RESPONSE SPECTRA

(d) PSEUDO VELOCITY RESPONSE SPECTRA

FIG. 2.28. (CONTINUED)

are encountered and the equivalent linear modulus is decreased. This is reflected by an increase in the fundamental period from about 0.75 sec (Fig. 2.28(b)) to 0.9 sec (Fig. 2.28(d)) for the continuum model. The agreement between the continuum and finite element methods although quite satisfactory is not as good as that between the lumped mass and continuum shear beams. The peak surface accelerations are quite different--0.144g versus 0.100g--and the acceleration response spectra differ more noticeably for periods less than 0.7 sec. The pseudovelocity response spectra in Fig. 2.28(d) show the same differences for periods below 0.7 sec and make the small differences at larger periods more noticeable. Since the input signal has strong components in the period range where the differences are larger, the difference in peak surface accelerations is not surprising.

Also shown in Figs. 2.28(c) and (d) are response spectra for ground motion calculated for the variable modulus hysteretic finite element soil model described in Section 2.2. This model provides a means of directly calculating the response of a nonlinear material. In the iterative technique used by Idriss and Seed, 1968a, some consideration must be given to the time history of soil response or bedrock shaking in establishing the rule relating the average operating condition to the peak strain. With the nonlinear hysteretic model such a step is not required. However, the need to use a step-by-step solution procedure exacts a penalty in terms of the computer time required to obtain solutions.

For the particular earthquake input considered in the examples shown the iterative procedure and rule of equivalence used by Idriss and Seed, 1968a, and the finite element nonlinear hysteretic model produce surface ground motion response spectra that are in reasonably good agreement. However, the peak surface accelerations are quite different--0.144g for the equivalent linear lumped mass model versus 0.084g for the nonlinear hysteretic finite element model. Some indication of the model sources of such different peak accelerations can be obtained from the input motion pseudovelocity response spectrum and the response spectra

ratios shown in Figs. 2.29(a) and (b).

In view of the differences in calculated peak surface accelerations noted above, it is interesting to compare the pseudo-velocity response spectra ratios calculated from the four combinations of Golden Gate Park and State Building horizontal components obtained from 1957 earthquake records (see Fig. 2.30). In each case, the appropriate Golden Gate record was amplitude-scaled (multiplied by the factor 0.8) to simulate further attenuation commensurate with the increased distance from the epicenter. The differences between the spectra ratios indicate the degree of uncertainty in using either scaled Golden Gate record as an estimate of the bedrock motion and in determining the accuracy of the calculated surface motion. In the present case the differences for periods near 0.3 and 0.5 sec are of particular interest since the major differences between the various models appeared in the same frequency ranges. The differences seen in Fig. 2.30 prevent an objective evaluation of the relative merits of various computational models of the San Francisco State Building site subjected to strong motion shaking. In spite of the differences which such uncertainties or inaccuracies in describing input motion may introduce, there is considerable value in comparative studies of alternative sites conducted with any one of the models and soil response calculations and surface motion predictions should be undertaken. The choice of method to be used can, for the present, only be based on such factors as computing costs or flexibility in describing complex soil behavior.

2.4 SENSITIVITY STUDIES WITH LINEAR ELASTIC MODEL

A number of surface motion analyses were made using a finite element shear beam with a linear elastic soil model to determine the effect of changing certain soil parameters on response predictions for both the State Building and El Centro sites. Variations in the damping (uniform increases or decreases of proportional damping) and the shear modulus (uniform stiffening or softening of the soil profile) used in this study are shown in

(a) PSEUDOVELOCITY RESPONSE SPECTRUM OF INPUT SHAKING

(b) COMPARISON OF AMPLIFICATION PROPERTIES

FIG. 2.29. COMPARISON OF INPUT MOTION RESPONSE SPECTRUM AND AMPLIFICATION PROPERTIES OF TWO SITE MODELS

FIGURE 2.30. PSEUDO VELOCITY AMPLIFICATION SPECTRA, SAN FRANCISCO STATE BUILDING

Tables 2.3 and 2.4. The baseline values of shear modulus are shown in Figs. 2.21 and 2.22. In the elastic finite element analysis utilizing a step-by-step integration approach, the viscous damping was further idealized and appears as uniform proportional damping, i.e.,

$$[M] \{\ddot{\chi}\} + (\alpha[M] + \beta[K]) \{\dot{\chi}\} + [K] \{\chi\} = \{P\} \quad (2.12)$$

$$\lambda = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \quad (2.13)$$

where α, β are the coefficients of mass and stiffness proportional damping, λ is the damping ratio (or fraction of critical damping), and ω_n is the natural frequency of the n^{th} mode. Since an iterative procedure which could adjust the definition of properties to retain compatibility with the computed strain values was impractical for use with direct integration procedure, the effects of altering the modulus and damping values are an exaggeration of those that would be observed in a more realistic model of soil behavior.

Ten cases are reported: six for the State Building site, and four for the El Centro site. Input motions were the scaled S80E Golden Gate Park record of the 1957 San Francisco earthquake and the scaled NS component of the 1940 El Centro earthquake, used for the comparisons in the preceding section of this report. An integration time step of 0.005 sec was used for the first five cases for the State Building site. In all other cases, an integration time step of 0.01 sec was used. Only the first 10 sec of the scaled Golden Gate Park record were used for studies of the State Building site, while for the El Centro site only the first 5 sec of the scaled El Centro record were used.

Acceleration and pseudo velocity response spectra and spectra ratios for the State Building site model are compared in Figs. 2.31, 2.32, and 2.33. Shear stress histories at the mid-height of the soil columns are shown in Fig. 2.34.

TABLE 2.3

INPUT DATA USED IN SAN FRANCISCO STATE BUILDING
SITE GROUND SURFACE RESPONSE CALCULATIONS

Case No.	Site Description	Elastic Modulus	Damping			Remark on Damping	Input Motion	No. of Different Materials
			λ	α	β			
1	San Francisco State Building (Sand)*	G^*	0	0	0		0.8 x Golden Gate	20
2	San Francisco State Building (Sand)*	G	8%	2.5	0	Proportional to ω_2	0.8 x Golden Gate	20
3	San Francisco State Building (Sand)*	G	8%	1.25	0	Proportional to ω_1	0.8 x Golden Gate	20
4	San Francisco State Building (Sand)*	1.25 G	8%	1.4	0	Proportional to $\omega_1' = \sqrt{1.25} \omega_1$	0.8 x Golden Gate	20
5	San Francisco State Building (Sand)*	1.7 G	8%	1.6	0	Proportional to $\omega_1'' = \sqrt{1.7} \omega_1$	0.8 x Golden Gate	20
6	San Francisco State Building (Sand)*	1.7 G	6%	1.2	0	Proportional to $\omega_1''' = \sqrt{1.7} \omega_1$	0.8 x Golden Gate	20

*See Fig. 2.21.

TABLE 2.4

INPUT DATA USED IN EL CENTRO SITE
GROUND SURFACE RESPONSE CALCULATIONS

Case No.	Site Description	Elastic Modulus	Damping			Remark on Damping	Input Motion	No. of Different Materials
			λ	α	β			
1	El Centro (Clay)*	G^*	10%	0	0.008	Proportional to ω_3	Scaled El Centro Surface Record	7
2	El Centro (Clay)*	G	10%	2.6	0	Proportional to ω_1		7
3	El Centro (Clay)*	G	13%	3.4	0	Proportional to ω_1		7
4	El Centro (Clay)*	0.81 G	7.5%	1.75		Proportional to $\omega_1 \sqrt{0.8}$		7

*See Fig. 2.22.

(a) COMPARISON OF THE ACCELERATION RESPONSE SPECTRA

(b) ACCELERATION RESPONSE SPECTRA RATIOS, SAN FRANCISCO STATE BUILDING

FIG. 2.31. INFLUENCE OF SOIL MODEL VARIATIONS ON GROUND SURFACE RESPONSE

FIG. 2.32. INFLUENCE OF SOIL MODEL VARIATIONS ON GROUND SURFACE RESPONSE

FIG. 2.33. COMPARISON OF ACCELERATION AND PSEUDO VELOCITY RESPONSE SPECTRA FROM CASES 5 AND 6, SAN FRANCISCO STATE BUILDING SITE

FIG. 2.34. COMPUTED SHEAR STRESS TIME HISTORIES AT MIDPOINT OF THE SOIL COLUMN FOR SIX DIFFERENT CASES

Case 1 for the State Building site was used to establish natural frequencies of the first few modes for use in assigning damping ratios in the other calculations. With the finite element time marching solution procedure used, proportional damping is specified, whereas in other techniques, or with a modal superposition procedure, damping can be separately specified for each material or each mode. Comparisons between models using proportional damping and modal damping thus require the selection of a particular mode for describing the damping ratio or fraction of critical damping.

As indicated in Equation 2.12, two damping constants, α and β , appear. In the case $\alpha = \text{constant}$, $\beta = 0$, damping is strongest in the fundamental or longest period mode; higher frequency (shorter period) signals propagate through the soil with little degradation. In the case $\alpha = 0$, $\beta = \text{constant}$, the high frequency (short period) signals are most strongly affected. If both α and β damping are used, signals of intermediate period will be least affected and both short and long period signals will be damped out. These characteristics are those exhibited, respectively, by the Maxwell, Kelvin-Voigt and standard linear viscoelastic solid models shown schematically in Fig. 2.6. The use of $\alpha = \text{constant}$, $\beta = 0$ damping has been interpreted as producing energy losses without the development of local strains; while $\alpha = 0$, $\beta = \text{constant}$ damping has been shown to unrealistically penalize the higher modes. The use of proportional damping must therefore be viewed as an artifice which can be used to modify the frequency dependence of model damping characteristics to improve correlation with observed behavior.

When values of α and β are assigned, damping ratios are automatically specified for all modes, as indicated by Equation 2.13. In Cases 2 through 6 for the State Building site attention was focused on the effects of damping longer period modes by selecting $\alpha = \text{constant}$, $\beta = 0$. This is in satisfactory agreement with observed behavior and does not contradict the single-degree-of-freedom interpretation of damping tests. Cases 2 and 3 allow

an evaluation of the relative importance of the first and second modes. Reference to Equation 2.13 indicates damping in the first mode in Case 2 is twice that specified in Case 3. As a result, the peak spectral amplitudes for Case 2 should be about half those for Case 3 as observed in Figs. 2.31 and 2.32. In addition the peak surface acceleration for Case 3--0.100g--is larger than that for Case 2--0.076g. Cases 3, 4 and 5 illustrate the effect of uniformly increasing the site stiffness while retaining the same fraction of critical damping in the first mode. Because of the similarity of response spectra from Cases 5 and 6 noted below and to improve the clarity of the figures, the spectra for cases 5 and 6 are shown as a single line. The value of the damping constant must be increased to account for the increase in the first mode frequency which is directly proportional to the square root of the ratio of soil stiffnesses or shear moduli. The frequency shift corresponding to a 25 percent increase in stiffness (≈ 11 percent) is difficult to detect in Figs. 2.31 and 2.32 because of the numerical technique used to determine response and amplification spectra; the change in period corresponding to the 70 percent increase in stiffness is, however, easily detected and accurately recorded. The peak surface accelerations increase slightly--0.111g for Case 4 and 0.115g for Case 5. Cases 5 and 6 illustrate an attempt to adjust the specification of damping to account for numerical "damping" introduced by increasing (doubling) the integration time step and thus reducing the time resolution of response peaks. The similarity of the surface motion obtained from these two cases is illustrated by the response spectra shown in Fig. 2.33. The peak acceleration is reduced to 0.096g however, because of the loss of resolution. Differences in frequency content in the six cases are quite evident in the shear stress histories shown in Fig. 2.34. The strong oscillations of approximately 1 sec period evident in the undamped model are of little importance in the damped models, illustrating the strong effect of α -damping on the longer period signals.

Natural periods for the El Centro site were obtained from calculations based on Kanai's method. Response spectra and

response spectra ratios are shown in Figs. 2.35 and 2.36. Suppression of the higher modes (short period modes) through the use of β -damping was examined in Case 1. To limit the rate at which the damping increased with mode number (or frequency) the nominal 10 percent damping was specified in the third mode. Peak surface acceleration was 0.239g. Comparison with Case 2, α -damping with 10 percent damping in the first mode illustrates the differences between the two types of proportional damping. Peak acceleration increased to 0.385g indicating the importance of the higher frequencies. Case 1 is less heavily damped than Case 2 in the first mode (3 percent versus 10 percent) but more heavily damped in the third mode (10 percent versus 3 percent). Both the acceleration and pseudo velocity amplification spectra, Figs. 2.35 and 2.36, respectively, show the large, nearly 3-fold, differences in spectral amplitudes expected for these two cases at the periods of the first (0.6 sec) and third (0.2 sec) modes. In Cases 3 and 4, slight adjustments of damping factors and overall stiffness were introduced in an attempt to control spectral amplitude values and the natural mode periods. Peak surface accelerations, however, varied significantly when these changes were introduced, dropping to 0.244g for Case 3 but reaching 0.441g in Case 4. The effect of substituting β -damping (Case 1) is also clearly shown in the shear stress histories shown in Fig. 2.37, with β -damping the high frequency content evident in the stress histories for Cases 2, 3, and 4 is almost completely suppressed.

The results of these two groups of calculations indicate that the stiffness and damping properties of soils play an important role in determining the amplification characteristics of a soil deposit. The models used to describe the damping characteristics must, therefore, be consistent with damping property tests and observed physical sources of energy dissipation. Since the property variations considered in the examples decoupled the definition of modulus and damping properties, their effect on ground surface motion is exaggerated. The experimentally observed dependence of damping and modulus on strain amplitudes

(a) COMPARISON OF ACCELERATION RESPONSE SPECTRA

(b) ACCELERATION AMPLIFICATION SPECTRA, EL CENTRO SITE

FIG. 2.35. INFLUENCE OF SOIL MODEL VARIATIONS ON GROUND SURFACE RESPONSE

(a) PSEUDOVELOCITY RESPONSE SPECTRA

(b) PSEUDOVELOCITY RESPONSE SPECTRA RATIOS, EL CENTRO SITE

FIG. 2.36. INFLUENCE OF SOIL MODEL VARIATIONS ON GROUND SURFACE RESPONSE

FIG. 2.37. COMPUTED SHEAR STRESS TIME HISTORIES AT MIDPOINT OF THE SOIL COLUMN FOR FOUR DIFFERENT CASES

suggests that soils respond as self-compensating systems which are therefore more tolerant of inaccuracies in defining damping and modulus values than the simple parametric studies reported here indicate. However, because of the influence of sample disturbance on modulus and a need for consistent means of converting observed damping behavior to computational model properties, continued efforts in determining in situ properties are indicated.

2.5 THE INFLUENCE OF TWO-DIMENSIONAL EFFECTS ON SOIL RESPONSE

The effects of boundary shape, of coupled vertical and horizontal motion, or of the influence of the variation in earthquake excitation over the boundary of the soil deposit can be examined by the use of two- or three- dimensional models. In the present study, two relatively simple two-dimensional problems have been solved to illustrate the importance of deviations from the idealized one-dimensional conditions treated by shear beam models. In the first problem, a model of the State Building site with steeply sloping boundaries was subjected to uniform horizontal earthquake shaking input at the base. In the second problem, a block-like horizontally layered model of the El Centro site with unconstrained vertical boundaries was subjected to vertical and horizontal components of earthquake shaking applied uniformly over its base. The results from these two problems were compared with results from one-dimensional studies of the same two sites presented in Section 2.4. It is hoped that these two-dimensional analyses will serve as a reminder that strong vertical earthquake motion may be encountered both generally and locally as a result of waves reflected from sloping boundaries or from vertical shaking input at the base of the soil deposit.

2.5.1 Analysis Method and Material Models

Since the two two-dimensional problems were intended merely to demonstrate the qualitative differences between the phenomena accounted for in one- and two-dimensional models, simple representations of the soils and soil deposit geometries were used.

Both the sands of the State Building site and the clays of the El Centro site were treated as simple elastic materials with viscous damping. The shear and shear damping properties correspond exactly to those described for Case 3 of Table 2.3 and Case 4 of Table 2.4. However, since the finite element code was formulated for two-dimensional problems and both P- and S-waves must be considered in such problems, additional soil properties data are required, even when the element mesh is constrained to respond as a shear beam. For the simple models used, specification of either the constrained modulus, Young's modulus, or Poisson's ratio was sufficient.

Application of the finite element code used for the problems reported in Section 2.4 was continued. The single column of elements constrained to respond as a shear beam was replaced by full two-dimensional arrays of elements whose interior points were free to move both vertically and horizontally. The integration time step used in both problems was 0.01 sec, which is equal to the time step used in the one-dimensional calculations for El Centro and twice the time step used in Case 3 of the State Building site one-dimensional problems.

2.5.2 The Response of a Narrow Valley Model

The two-dimensional finite element valley model, shown in Fig. 2.38, consisted of a blunt V-shaped deposit of soil 520 ft wide at the surface and 120 ft wide at the base with sides sloping at 45 deg. an exaggeration of the slopes encountered in most situations. The vertical boundaries of the two rock regions were provided with horizontal roller supports and the base of the rock/soil block was subjected to amplitude-scaled shaking based on the S80E component of the Golden Gate Park record of the 1957 San Francisco earthquake. The soil was described as a linear material with uniform proportional damping of the type described in Section 2.4. The soil material input data were identical to those used in Case 3 (see Table 2.3 and Fig. 2.21) of the one-dimensional finite element analysis of the State Building site.

FIG. 2.38. NARROW VALLEY MODEL

Time histories of ground surface vertical and horizontal acceleration for a point at the center (Nodal Point 278) and a point 100 ft from the edge of the soil deposit (Nodal Point 94) are shown in Figs. 2.39 and 2.40. Pseudo velocity response spectra for the same two points are shown in Figs. 2.41 and 2.42. The amplitude of the vertical component is a strong function of position. At the center, the peak vertical acceleration is only about 1 percent of that observed 160 ft away at Nodal Point 94. The horizontal component, on the other hand, changes only slightly across the surface; the peak value at the center is about 1-1/2 times that observed 160 ft away at Nodal Point 94. It should also be noted that away from the center, the two components have comparable amplitudes, and that all of the vertical motion is the result of reflection from the inclined rock-soil interface. In the field, where a vertical component of shaking may occur, stronger vertical accelerations would be expected in a steep walled narrow valley of this type.

The histories of normal stress (σ_x and σ_y) and shear stress (τ_{xy}) for an element at the center and 100 ft below the ground surface (Element 282) and for an element adjacent to the inclined rock boundary at the same depth below the ground surface (Element 116) are given in Figs. 2.43 and 2.44, respectively. The maximum normal stresses in the element at the center of the model (see Fig. 2.43) are comparatively small; the peak is less than 20 percent of the maximum shear stress in the same element. For the element near the rock boundary, the maximum normal stress in the x-direction (σ_x) is approximately three times greater than the maximum shear stress (τ_{xy}) and the normal stress in the y-direction has the same magnitude as that of shear stress (see Fig. 2.44). Such an increase of normal stresses is caused by the reflection of waves at the inclined boundary. The shear stress time histories of these two elements indicate that the sloping boundary tends to reduce the magnitude of shear stress locally. When sloping boundaries are present, stress patterns near the boundary may therefore be different from those predicted for

(a) VERTICAL ACCELERATION

(b) HORIZONTAL ACCELERATION

FIG. 2.39. ACCELERATION HISTORIES FOR A SURFACE POINT AT THE CENTER OF THE NARROW VALLEY SOIL DEPOSIT (NODAL POINT 278)

(a) VERTICAL ACCELERATION

(b) HORIZONTAL ACCELERATION

FIG. 2.40. ACCELERATION HISTORIES FOR A SURFACE POINT 100 FEET FROM THE EDGE OF THE NARROW VALLEY SOIL DEPOSIT (NODAL POINT 94)

(a) VERTICAL COMPONENT

(b) HORIZONTAL COMPONENT

FIG. 2.41. PSEUDOVELOCITY RESPONSE SPECTRA, FOR THE NARROW VALLEY MODEL, NODAL POINT 278

(a) VERTICAL COMPONENT

(b) HORIZONTAL COMPONENT

FIG. 2.42. PSEUDOVELOCITY RESPONSE SPECTRA, FOR THE NARROW VALLEY MODEL, NODAL POINT 94

(a) HORIZONTAL STRESS

(b) VERTICAL STRESS

(c) SHEAR STRESS

FIG. 2.43. COMPUTED STRESS TIME HISTORIES AT ELEMENT 282 FROM THE TWO-DIMENSIONAL NARROW VALLEY MODEL

(a) HORIZONTAL STRESS

(b) VERTICAL STRESS

(c) SHEAR STRESS

FIG. 2.44. COMPUTED STRESS TIME HISTORIES AT ELEMENT 116 FROM THE TWO-DIMENSIONAL NARROW VALLEY MODEL

horizontally layered deposits of large lateral extent by one-dimensional models.

2.5.3 Comparison of One- and Two-Dimensional Motion

Comparisons of the ground motion calculated for one- and two-dimensional models using the State Building soil profile are based on the horizontal acceleration and velocity observed at surface points. Response spectra from Nodal Points 94 and 278 (see Fig. 2.38) are compared with response spectra for Case 3 of the one-dimensional studies of the State Building site in Figs. 2.45 and 2.46. The most notable differences between the horizontal motion in the two models are changes in the dominant periods and in peak amplitudes. As indicated in Fig. 2.45, the periods of strong accelerations for the one-dimensional model are 0.8, 0.4, 0.23, and 0.15 sec. For the two-dimensional model the period of strong acceleration is 0.42 sec. Shorter period components are present, but spectra for the two nodal points show different resonances. Peak spectral accelerations increase from 0.44g in the one-dimensional case to 1.18g at the center (Nodal Point 278) and 0.80g at Nodal Point 94 of the two-dimensional model and the peak surface accelerations are 0.100g, 0.152g, and 0.103g, respectively. As indicated in Fig. 2.46, differences between the peak spectral pseudo velocities for the one- and two-dimensional models are nearly as large--8.5 ips for the one-dimensional model versus 20.8 ips at the center (Nodal Point 278) and 21.5 ips at Nodal Point 94 of the two-dimensional model. The periods corresponding to pseudovelocity response spectra peaks for the one-dimensional model, slightly larger than those reported for acceleration peaks, are 0.85, 0.42, 0.26, and 0.17 sec. The pseudovelocity spectra for the two-dimensional model show the same dominant peak at 0.42 sec. exhibited in the acceleration response spectra.

Although the shift in dominant period from 0.85 to 0.42 sec. very nearly corresponds to that predicted by substituting P-waves for S-waves ($c_p = 2.1 C_s$) in a simple soil column resonance mode, the relative strengths of shear and normal stresses (see

FIG. 2.45. COMPARISON OF ACCELERATION RESPONSE SPECTRA FROM ONE- AND TWO-DIMENSIONAL MODELS

FIG. 2.46. COMPARISON OF PSEUDO VELOCITY RESPONSE SPECTRA FROM ONE- AND TWO-DIMENSIONAL MODELS

Figs. 2.43 and 2.44) and the variation of soil depth (See Fig. 2.38) illustrate the inadequacy of such an explanation. Although the valley model used represents an extreme case and the effect on soil response is much greater than would be expected for rock-soil boundary slopes commonly encountered, it is worth noting that the combined influence of the steeply inclined boundaries, reflections, and lateral constraint provided by the rock produced significant vertical shaking and locally altered the horizontal response of the soil deposit from that which would have occurred in a uniformly layered soil of unlimited lateral extent, even when only horizontal shaking was applied.

2.5.4 Combined Horizontal and Vertical Excitation of an El Centro Model

In the second two-dimensional problem treated, the influence of combined horizontal and vertical shaking was examined; sloping boundaries were not considered. The soil deposit was represented by a rectangular block of soil 440 ft long and 100 ft deep (see Fig. 2.47) having the composition of the El Centro site. Vertical and horizontal shaking was applied uniformly over the base. Input motion histories were obtained by applying the time and amplitude scaling factors used previously (see Section 2.3.2) to the NS and vertical records of the 1940 El Centro earthquake. The resulting vertical and horizontal acceleration input records are shown in Figs. 2.48 and 2.26, respectively. The El Centro clays were represented as linear elastic media with viscous damping. The properties assigned correspond to those used in Case 4 of the one-dimensional studies of the El Centro site (see Table 2.4 and Fig. 2.22). The finite element code and integration time step (0.01 sec) were the same as those used in the one-dimensional problems, except that the single column of elements constrained to respond as a shear beam was replaced by an unconstrained two-dimensional element array. The two-dimensional calculation was carried to a real time of 4.0 sec. versus 5.0 sec. for the one-dimensional problem treated in Case 4.

FIG. 2.47. TWO-DIMENSIONAL MODEL OF THE EL CENTRO SITE

FIG. 2.48. VERTICAL COMPONENT OF THE INPUT BASE MOTION FOR EL CENTRO SITE

Acceleration time histories and pseudo velocity response spectra for two surface points (Nodal Point 219 at the center and Nodal Point 66, 125 ft from the edge of the soil block) for both vertical and horizontal components of motion are shown in Figs. 2.49 through 2.52. As indicated in Figs. 2.49 and 2.50, the peak horizontal acceleration is amplified from 0.2g to about 0.33g at both points. The uniformity of the horizontal response along the ground surface is emphasized by the comparison of the horizontal pseudovelocity response spectra shown in Fig. 2.51. The spectra are in very good agreement for periods between 0.1 and 2.0 sec. The spectra have three strong peaks, a narrow one centered near 0.25 sec. and two broader ones covering periods ranging from 0.5 to 0.9 sec. and from 1.0 to more than 1.6 sec. This is quite different from the behavior observed in the narrow valley model subject to horizontal shaking where a single narrow peak was observed in the horizontal pseudovelocity response spectrum and significant variation between the response at comparably located surface points was observed.

The vertical acceleration at the two points, on the other hand, are only slightly affected by passage through the soil. Examination of Figs. 2.49 and 2.50 indicates slight amplification at Nodal Point 66 (from 0.17g to 0.21g) and slight attenuation at Nodal Point 219 (from 0.17g to 0.14g). The higher frequency of the vertical component of shaking evident from a comparison of the vertical and horizontal input records shown in Figs. 2.48 and 2.26, respectively, may contribute to these differences. The pseudovelocity response spectra for the vertical surface motion shown in Fig. 2.52 indicate further differences between vertical and horizontal surface motion. The vertical pseudovelocity response spectra exhibit only a single narrow peak at a period of 0.25 sec. The differences between the responses at the two points are also evident. This pattern, much stronger variation of vertical response with position than of horizontal response, was also observed in the narrow valley model. However, the causes of the variations appear to be quite different, and the sign of the change is

(a) VERTICAL ACCELERATION

(b) HORIZONTAL ACCELERATION

FIG. 2.49. ACCELERATION TIME HISTORIES FOR A SURFACE POINT 125 FT FROM THE EDGE OF THE SOIL DEPOSITS (NODAL POINT 66) FOR EL CENTRO SITE

(a) VERTICAL ACCELERATION

(b) HORIZONTAL ACCELERATION

FIG. 2.50. ACCELERATION TIME HISTORIES FOR A SURFACE POINT AT THE CENTER OF THE SOIL DEPOSIT (NODAL POINT 219) FOR EL CENTRO SITE

FIG. 2.51. VELOCITY RESPONSE SPECTRA FOR HORIZONTAL MOTION AT TWO POINTS ON THE SURFACE OF THE TWO-DIMENSIONAL EL CENTRO SITE MODEL

FIG. 2.52. VELOCITY RESPONSE SPECTRA FOR VERTICAL MOTION AT TWO POINTS ON THE SURFACE OF THE TWO-DIMENSIONAL EL CENTRO SITE MODEL

reversed. Here the larger amplitude is encountered near the end rather than at the center. The increase is attributed to the lack of constraint at the ends of the soil block.

Histories of normal stress (σ_x and σ_y) and of shear stress (τ_{xy}) for an element 50 ft below the surface of the center of the soil block (Element 196) and for an element 130 ft from the end of the soil block at the same depth (Element 68) are shown in Figs. 2.53 and 2.54, respectively. In Element 196 the normal stresses are only about half the intensity of the shear stress, but exhibit higher frequencies. The apparent increase in frequency is more closely related to the differences in vertical and horizontal base input frequency than to frequency changes which could be associated with P- and S-wave resonances. The normal stresses in Element 68 are larger than those observed in Element 196, while the shear stress is somewhat smaller. The vertical normal stress, enhanced by the general increase in vertical response (primarily a P-wave mechanism here) near the boundary, exceeds the shear stress. This is the same stress pattern observed in the narrow valley model, but the causes, as indicated above, are quite different. In the narrow valley model S-waves reflected at the sloping soil-rock interface produced the strong P-wave (normal stress) components and the constraint provided by the stiff rock "buttresses" prevented large shear strains from developing. In the present case, the P-waves are introduced by the vertical excitation of the base. Only normal reflection should occur (only vertical and horizontal boundaries are present) and no conversion from P- to S-waves or from S- to P-waves can occur. The reduction of shear stresses near the boundary relative to those at the center is apparent only for times greater than about 1 sec. suggesting that the slight attenuation is related to relief signals propagating inward from the unconstrained end boundaries.

2.5.5 Comparison of Horizontal Motion from One- and Two-Dimensional Models of the El Centro Site

The calculated response of one- and two-dimensional models of the El Centro site are compared in Figs. 2.55 and 2.56.

FIG. 2.53. COMPUTED STRESS TIME HISTORIES AT ELEMENT 196 FOR EL CENTRO SITE

(a) HORIZONTAL STRESS

(b) VERTICAL STRESS

(c) SHEAR STRESS

FIG. 2.54. COMPUTED STRESS TIME HISTORIES AT ELEMENT 68 FOR EL CENTRO SITE

FIG. 2.55. COMPARISON OF PSEUDO VELOCITY RESPONSE SPECTRA DERIVED FROM HORIZONTAL SURFACE MOTION IN ONE- AND TWO-DIMENSIONAL MODELS OF THE EL CENTRO SITE

(a) TWO DIMENSIONAL MODEL, ELEMENT 68

(b) TWO DIMENSIONAL MODEL, ELEMENT 196

(c) ONE DIMENSIONAL MODEL, ELEMENT 10 (CASE 4)

FIG. 2.56. COMPARISON OF SHEAR STRESSES AT 50 FT DEPTH IN ONE- AND TWO-DIMENSIONAL MODELS OF THE EL CENTRO SITE

In Fig. 2.55 strong similarity between the horizontal response in the two models is evident; spectral amplitudes and periods corresponding to response peaks are almost identical. Shear stresses at the middepth of the two models are compared in Fig. 2.56. The differences between the shear stress histories in the one- and two-dimensional models is slight. The one-dimensional model does exhibit some higher frequency details not evident in the two-dimensional model; these differences, however, do not affect the general similarity of the time histories or introduce large differences in peak spectral values.

Peak surface accelerations do differ, however, reaching 0.441g in the one-dimensional model but only 0.342g and 0.320g at Nodal Points 219 and 66 of the two-dimensional model, respectively. Since the linear model used provides no means of coupling the vertical and horizontal motion and the two-dimensional soil block contains no inclined boundaries which could couple the two components through reflections, the only coupling that can occur must result from the lack of constraint at the ends of the soil block, and as noted in Section 2.5.4, the resulting influence is of little significance. Thus, the analysis of the response to vertical and horizontal shaking can be decoupled for soil deposits of large lateral extent.

2.6 SUMMARY AND RECOMMENDATIONS

In the calculations reported here the ability of one-dimensional continuum, lumped mass, and finite element models equipped with equivalent descriptions of soil behavior to produce comparable results was demonstrated. Two methods of accounting for the strain dependence of modulus and damping were shown. The first method uses an iterative procedure to adjust the properties of a strain dependent equivalent linear model to reflect the calculated strain amplitudes. The procedure is repeated until the properties are in satisfactory agreement with the computed strains. The second method uses a nonlinear hysteretic model of soil behavior and directly calculates the soil response using properties

appropriate to the current state of strain, the sign of the load increment, and the previous strain history.

Two two-dimensional problems were also solved. In the first, the combined influence of the narrowness of the valley model and the steepness of the sloping boundaries altered the motion from that predicted by a one-dimensional model. The changes were much stronger than those expected from a valley model with more commonly encountered gently sloping boundaries. The second examined the response of a simple soil block to combined horizontal and vertical shaking.

The problems reported here and related problems reported by other investigators illustrate that there are situations where nonlinear behavior and the two- or even three-dimensional aspects of a soil deposit must be taken into account when determining the influence of the soil deposit on local earthquake motion. The interaction of the contributing factors is sufficiently complex that no simple rules for identifying such cases have been obtained. Careful engineering judgement must therefore be exercised in selecting models for analyzing the response of a particular site to earthquake excitation.

Although rigorously applicable under limited circumstances, the one-dimensional methods have been shown to provide accurate estimates of soil response to earthquake shaking over a much broader range of conditions. Gently sloping boundaries and more abrupt changes at some distance from the points of interest can be tolerated. Because of such demonstrations and the need for soil response analysis methods which can easily be adopted by the engineering community at large, selection of simple, economical one-dimensional methods has a number of points in its favor. However, since proper application of such methods requires an understanding of their limitations, blanket approval of their adoption at this point would be premature. Careful judgement must be exercised in their application, and in situations where uncertainty exists as to their applicability, more complex methods must be used.

For the present state of the art, use of one-dimensional models with representations of soils which account for the strain dependence of the stiffness and damping properties to conduct parametric and comparative studies of the more promising alternative sites is recommended.

Uncertainties concerning the epicentral location, magnitude, and other characteristics of earthquake shaking (see for example Fig. 2.30) must be taken into account in the parametric studies. Where sound engineering judgement suggests potential difficulty from soil deposit geometry or from unusual soil conditions, more complex methods which allow examination of their effect must be used. Because of the wide variety of conditions which may be encountered and which could have some effect, no general rules are offered; each case must be examined on its own merits.

Since three alternative one-dimensional approaches have been shown to give comparable results, selection of any one of them must be based on cost, flexibility, or personal preference.

The relative cost of the alternatives can be estimated from computer time used to perform the various calculations reported here. Data from those calculations are shown in Table 2.5. With acceptable resolution in time and space, the one-dimensional methods which account for strain dependence of the soil properties require computing times of only a few minutes (UNIVAC 1108). This represents a sufficiently low direct cost that parametric studies are economically feasible. Computing time for the two-dimensional finite element models is strongly influenced by the bandwidth of the stiffness matrix, which for simple models is proportional to the number of elements through the depth or width of the mesh, whichever is smaller. Here, a 574 element two-dimensional model with a bandwidth of 50 required an almost 100-fold increase in computer time over that for a 10 element one-dimensional model with a bandwidth of 8. However, the time and space resolution obtained in the example exceeded those normally required.

A similar basis must be used in selecting models to

TABLE 2.5

COMPUTER TIME REQUIREMENTS

<u>Method</u>	<u>Number of Elements</u>	<u>Number of Time Steps</u>	<u>Real Time</u>	<u>(UNIVAC 1108) Computer Time</u>	<u>Soil Model</u>
FE 1-D	10	500	5 sec	2 minutes	Linear
	20	1000	10 sec	2.7 minutes	
	20	2000	10 sec	5.25 minutes	
FE 2-D	574	400	4 sec	148.2 minutes	Linear
Lumped Parameter Shear Beam		Modal			Iterative
	19	Super-	15 sec	2.5*-4.0** min.	Equivalent
	9	Position	15 sec	1.6 minutes*	Linear
Continuous Shear Beam	12 layers	(Direct Determination of Amplification Spectra) (No Time History Obtained)+			Linear
			4 sec+		
FE 1-D	40	400	4 sec	38.2 minutes	Nonlinear
	10	400	4 sec	3.3 minutes	Hysteretic

* 2 Iterations

** 4 Iterations to determine strain distribution and adjust properties

+ The addition of Fast Fourier transform and inversion routines to allow calculation of time histories and an iteration procedure to obtain strain compatible properties results in computing times slightly less than those for the lumped parameter shear beam model.

represent observed nonlinear soil behavior. Equivalent linear models have been most widely used. In establishing equivalence, some knowledge of the strain states encountered during the whole shaking history is required. Experience to date indicates that a simple rule of the form $\epsilon_{ave} = C\epsilon_{peak}$ with $0.6 < C < 1.0$ provides sufficient accuracy. The values of C is based on the variation of acceleration amplitude during the period of strong motion shaking. The nonlinear variable modulus hysteretic model can be used directly with any earthquake input if soil data covering a sufficient range of strain amplitudes are available. Since no iteration is required to obtain strain compatible properties the required use of a time marching (direct integration) solution procedure is not a severe penalty.

It is recommended that, until the accuracy of the computational model is established, parametric calculations be conducted with the one-dimensional models. The ability to obtain soil properties compatible with the calculated strain amplitudes is desirable and methods incorporating iterative procedures or which directly account for the strain dependence should be selected. For the type of strain dependence observed in sand and clays, the iterative or nonlinear approaches will automatically adjust the effective modulus and damping properties and the operating strain levels to partially compensate for inaccuracies in determining the soil properties.

If two or more dimensional effects must be considered, finite element methods (or equivalent finite difference methods) are required. Additional soil data describing the dilatational behavior are then required. The additional costs involved in obtaining such data and in performing two-dimensional analyses suggests that they be reserved to treat special problems and that preliminary screening be performed on the basis of one-dimensional studies.

CHAPTER 3

STUDY OF IN SITU TESTING TECHNIQUES FOR DETERMINATION OF DYNAMIC SOIL PROPERTIES

The engineering soil properties required for analysis of site response resulting from earthquake motions are shear modulus, damping, bulk modulus (or Poisson's ratio), and unit weight. The first two of these properties must be evaluated under dynamic loading, while the latter two properties are determined either by static loading tests or measured directly in the laboratory with acceptable accuracy. Shear modulus, damping, and bulk modulus are strain dependent.

The two dynamic soil properties, shear modulus and damping, are of primary interest to this program. Useful data obtained to date on the shear modulus are available from both field and laboratory tests while data for damping have been obtained primarily by laboratory procedures. Data generated using conventional testing equipment are largely at strain levels either smaller or larger than those resulting from earthquakes. The data that have been accumulated in the levels of strain (10^{-3} to 10^{-1} percent) of interest, were obtained largely from laboratory research efforts using very specialized procedures. No meaningful success to date has been made to develop an in situ procedure to measure these properties at strains of earthquake magnitude.

Our study of methods for making in situ measurements of shear modulus and damping has been conducted in two parts. The first comprised a review and evaluation of current practices. Subsequently, considerable study was undertaken to develop conceptual improvements to existing field equipment, devices and procedures, which would permit the in situ measurement of shear modulus at strains of interest. If successful, such devices and procedures eventually may be utilized systematically in site investigations. Even though subsequent experimentation may prove these techniques to be not feasible, such equipment and procedures may still be useful as research tools for verifying the validity

of extrapolation techniques between existing laboratory and field tests.

3.1 REVIEW OF EXISTING IN SITU TESTS

Field tests for determining values of damping in situ for practical use in seismic analyses are non-existent, although some useful information has been obtained (Seed and Idriss, 1970a) by analyzing the observed response of soil layers during an actual earthquake. Also, a procedure involving the use of micro-tremors or vibration generators (Martin and Seed, 1966; Keightley, 1966) has been used in research efforts for this purpose. This latter procedure has two serious limitations, namely; 1) small strain levels, and 2) the difficulty of differentiating between material (hysteretic) damping and geometric damping from the total measured damping. Because earthquakes are unpredictable in both time and magnitude and since the above test procedure using vibration generators has these limitations, engineers presently prefer to rely either on damping values determined in the laboratory or on published values adopted by others. In the laboratory, strains of significant amplitude can be readily produced and controlled on small samples of soil.

The determination of shear modulus in situ is presently accomplished through a variety of geophysical methods and equipment. Ideally, the end result is essentially the same. In each case the primary goal of the test is to generate, identify, isolate and measure the time-rate of travel of a shear wave from a given source to various monitoring positions. Once this is accomplished, the shear modulus can be computed using elastic theory for wave propagation. However, as stated above, the problem is greatly over-simplified because at each step or phase of the test, there are significant problems which tend to cloud the end result.

Most geophysical tests used to determine the in situ shear modulus are conducted at relatively low energy levels. Consequently, the strain levels are limited to a narrow band

varying from 10^{-3} to 10^{-5} percent strain, or smaller, whereas earthquakes create higher strain levels (roughly 10^{-1} to 10^{-3} percent). Since the shear modulus decreases significantly as the strain level increases and since there are no in situ test methods which develop earthquake type strain levels below the ground surface, various researchers have summarized data based on laboratory tests to represent the in situ shear modulus for the full range of strain (Hardin and Drnevich, 1970a and b; Seed and Idriss, 1970c). One typical curve with typical test values for a sandy soil with a relative density of about 75 percent is summarized in Fig. 3.1. Superimposed on this figure are the approximate levels of strains from available in situ and laboratory tests and those strains that are expected under earthquake loading.

3.2 IN SITU TESTS FOR DETERMINING SHEAR MODULUS

There are three basic types of shear wave measurement procedures: surface, down-hole (or up-hole) and cross-hole. As discussed subsequently, each has advantages, disadvantages and limitations.

3.2.1 Surface tests

These tests are performed using one of two possible basic procedures. Usually, a vibratory energy source is placed on the ground surface and operated at a fixed frequency. The geophones are then adjusted at various intervals along the ground surface until the wave length is identified, from which the velocity may be calculated. The velocity of the wave generated is assumed to reflect the soil properties to a depth of approximately $1/2$ the wave length, based largely on empirical data. In reality a true shear wave is not generated in this test. Instead, a surface Rayleigh wave is produced and its velocity measured. Although the Rayleigh wave velocity is less than the shear wave velocity, especially for low velocity materials, the difference at low energy levels is such that little error is introduced in subsequent calculations. The primary deficiencies in this test

FIG. 3.1. SHEAR MODULI OF SANDS AT RELATIVE DENSITY OF ABOUT 75%
(FROM SEED AND IDRIS, 1970c)

are: 1) it does not measure a true shear wave, 2) refraction in layered soil systems, and 3) the validity of the empirically established half wave length theory. Also, the different shape and energy characteristics for the Rayleigh and shear waves are such that the test is applicable only for the measurement of wave velocity at low strain levels. If the near surface soils are layered, such that the various strata have considerably different seismic velocities, then considerable refraction error is likely to be introduced. As a result, the shear modulus is likely not to be representative of the average conditions at the site.

The other procedure, less familiar, utilizes a horizontal impact energy source, also located on or near the ground surface. Velocity measurements of the generated shear waves are made utilizing standard seismic refraction techniques. The only difference in the test from the standard test is that the geophones are rotated 90 degrees such that P-wave arrivals are minimized while shear wave arrivals become more apparent. The primary advantage of this test for determining shear wave properties is that the tests are relatively inexpensive and simple to conduct so long as low level energy sources are utilized. The disadvantage of testing by refraction techniques is the high rate of attenuation of shear wave energy and depth limitations. When the travel distance for the wave is large, rapid attenuation of energy makes the pattern difficult to identify even when the wave is greatly amplified. This requires that measurements be taken at closer spacings, which in turn limits the depth to which measurements are possible.

The disadvantages of surface testing are generally considered to outweigh the advantages to the point where the test is not considered practical for evaluating moduli values except in very shallow, near surface materials or reasonably uniform materials. Further, in our opinion, neither procedure can be utilized or modified to generate true shear waves at high energy levels.

3.2.2 Down-hole and Up-hole Tests

These tests are conducted in a single, cased or uncased

boring. The test name implies the direction of wave propagation. To date the most common procedure (the down-hole test) is to apply a horizontal impulse at the surface to generate a shear wave, which is received at varying depths in the boring by triaxial geophones. Up-hole tests utilize small explosive charges placed at varying depths in the drill hole, with the geophones at the ground surface.

The inherent advantages of the down-hole procedure are: 1) the ability to penetrate to significant depths, and 2) the method of shear wave measurement and identification. With current techniques, data can be obtained to depths in excess of 300 feet using a hand-held sledge hammer as an energy source. This depth is generally adequate for most foundation investigations involving earthquake engineering problems. The method of wave measurement using this technique, in our opinion, is also more positive than the other in-hole test. In all such tests, the shear wave velocity is determined by measuring the time of first arrival of the shear wave. Unfortunately, compression waves precede the arrival of shear waves, often making the distinction between the two wave patterns as well as the identification of the first arrival of the shear waves difficult. This problem is resolved in the down-hole test by reversing the direction of particle motion of the energy arrivals. When this is done, the compression waves remain unchanged, while the shear waves are recorded 180 degrees out of phase. Comparing these records, the shear waves can be clearly identified and the time of first arrival more positively established.

The primary disadvantage with the down-hole procedure is the rapid attenuation of energy with depth. Though strains of earthquake magnitude can be mobilized near or at the ground surface by using larger energy sources, at depth the strain level drops well below the range of primary interest and there are, as yet, no satisfactory in-hole shear-wave generating devices for use in either the down-hole or up-hole technique. Attempts to use small explosive impulse charges generally have either damaged

the hole and/or produced a variety of other confusing wave patterns. Acoustic methods also have been tried, but the resultant strain level thus far has been well below the desired range of 10^{-1} to 10^{-3} percent. Attempts to use large controlled in-hole vibratory sources have, however, not been tried.

Because of the rapid attenuation of energy with depth, it is less likely that this test can be developed to induce large strains in the soil unless an in-hole energy source is used. Also, the test procedure of measuring and identifying the time of first arrival of the shear wave using an impulse energy source may not prove applicable for the measurement of the velocity of high energy shear waves, since the precursor wave may travel at a higher velocity.

3.2.3 Cross-hole Tests

As the name implies, these tests are conducted such that shear waves are generated in one boring and the wave travel times recorded in adjacent borings or series of borings. Impulse energy sources (explosive charges) are normally used in this test, however, vibratory energy sources have been tried (Ballard and Leach, 1969) and proven successful.

The cross-hole method may make it possible to transmit relatively high levels of energy horizontally between closely spaced borings. Consequently, it is possible that strain levels approaching those measured in earthquakes could be developed.

The main disadvantages with cross-hole measurements are related to cost, data interpretation and horizontally layered soils. The total cost of testing using this method is considerably higher, not only because of the increased footage of borehole drilling required, but also because both down-hole sensing and wave generating equipment are required for each test. As described in the previous subsection, there are also great difficulties in defining shear waves from compression waves and thus delineating the time of first arrival of the shear wave. The interpretation of such data is further complicated when relatively thin horizontally layered soils are being tested. These layers may cause

refracted energy to interfere with or precede the arrival of the direct shear wave energy. Further interference may result from refracted-reflected energy arrivals, diffracted energy arrivals and the like. This latter complication can partially be reduced by using closely spaced boreholes.

Because it is possible to more readily induce higher strains in the soil by the cross-hole technique, this aspect of the test appears to have the most promise for use in the development of a new in situ test.

3.3 EQUIPMENT AND PROCEDURAL DEVELOPMENTS

After evaluating the various existing techniques for obtaining shear wave measurements, we have concluded that the cross-hole technique has the best potential for development of an improved test. Moreover, if a suitable in-hole energy source can be developed for use with the down-hole procedure, this test also shows some potential promise. Consequently, our approach has been to attempt to identify the major potential for improvement of these two tests and then to develop a flexible test program to answer unknown questions and establish experimentally if the development of such a test is possible. Emphasis of the entire program will be directed toward inducing higher energy to the soil, and measuring the resulting strains and shear wave velocity.

The following subsections describe various conceptual design features and advanced procedures, which we believe could lead to an improved new in situ technique involving larger strains than presently possible. None of these potential concepts has actually been tested.

3.3.1 Cross-hole Technique Improvements

The most desirable feature of this procedure is the ability to induce large strains in situ. To improve this feature of the cross-hole technique, it will be necessary to provide good coupling of the induced energy to the bore hole walls and to provide good control of the energy source itself. This requires

careful consideration of anchoring equipment and energy types.

a. Anchor Systems

Several different types of anchors appear feasible for use with the cross-hole technique. WES has developed and used a small, expanding vibro-packer anchor. Though this apparently is an effective piece of equipment, we believe it is much too small and fragile to be used with the larger sources of energy being considered in this study.

Though it may yet prove feasible to design a large controlled in-hole energy source, (not explosives) we are presently working on the assumption that the energy source will be located at the ground surface and be connected to the anchor at the desired test depth by a drill stem or pipe. Unfortunately, when the depth becomes large (i.e, on the order of 100 feet), such a connecting rod becomes flexible and loses much of its energy. The following discussion presents various types of anchors considered.

Possibly the simplest anchor for use in the cross-hole test would be a section of solid-stem auger (Fig. 3.2). This would vary in length from about 2 to 10 feet depending on the soil being tested. The diameter also could vary depending on the available drilling equipment. Approximately a 10-inch diameter section would be a conventional size, which could be inserted through a 12-inch drill casing. In cohesive soils, drill casing may not be needed. Under most soil conditions we visualize the following as one possible step by step sequence for installing the anchor prior to testing:

- 1) Advance boring to a depth just above first test location and case the hole if necessary.
- 2) Drill a pilot hole the same length as the auger anchor. This pilot bore should have a diameter approximately half of that of the auger anchor.
- 3) Screw the anchor section of auger into the pilot hole. To assure the anchor is fully confined and

FIG. 3.2. SCHEMATIC OF IN SITU TEST PROCEDURES--CROSS-HOLE TECHNIQUE

tightly fixed, a few feet of the hole above the anchor should be backfilled using washed sand or pea gravel.

- 4) Couple the energy source to the anchor connecting rods and conduct the test.

Following testing, the anchor is removed and the boring advanced to the next test depth where the above sequence of operation is repeated. Without field testing it is impossible to say for certain, but we believe that an auger type anchor would be: rugged, easy to use (no moving parts), effective, relatively inexpensive and could be fabricated in a wide range of sizes. One problem with this approach would be screwing the anchor into very hard soils, such as glacial till. This problem, however, could be overcome by over-drilling past the test depth and then inserting a reduced size auger in this interval and packing it in place with a gravel backfill. As far as we can tell, an up-down motion (impulse or vibratory) through the rods to the auger anchor would generate shear waves, which would radiate outward from the boring such that they could be detected in adjacent borings.

A cast-in-place anchor also may be feasible where time is not too critical. In this case the test hole would be drilled to its full depth prior to any testing. Then at the deepest test depth a high early strength concrete or epoxy plug would be cast to any predetermined length. An expendable connector would be cast into the anchor such that rods extending to the ground surface could be readily attached and removed. After testing at the deepest depth, the rods would be removed, the hole backfilled to the next test depth and a new anchor cast. The most significant drawback to this anchor system is the time required for each anchor to cure prior to testing, though the new regulated set cements may soon overcome this shortcoming. As an alternate to using a concrete anchor plug, a gravel packed plug may serve the same purpose. The use of a gravel plug would allow the test to be performed more quickly at specific intervals as the bore-hole is advanced.

Other anchor systems have been considered. Mechanical, pneumatic and hydraulic devices, such as heavy duty versions of WES's Vibro-packer, were studied. These systems probably would be feasible, although we believe the cost and complexity of operation make them less desirable than the relatively simple auger anchor.

It is presently planned that during field testing both the auger and cast-in-place anchors could be tried. In the final design stages, for extra strength against buckling, it may ultimately become necessary to use a section of pipe attached to the anchor. We have no plans at this time for fabricating or testing an expandible type anchor, unless the less complex systems prove unsuccessful during field testing.

b. Energy Sources

In the cross-hole test energy could be either steady-state or impulse in nature. We have reviewed a considerable number of different energy sources, which are available commercially, as well as considered the possibility of designing new equipment. Since in each case considered the larger energy source is quite bulky, we have anticipated that it would be located at the ground surface and connected via rods to some form of anchor as discussed previously.

Impulse energy for the cross-hole test may be generated through a number of heavy, but relatively simple and inexpensive hammer systems (drop hammers, piston air or hydraulic driven hammers, pile hammers or electromagnetic hammers). The primary criteria for such, other than operating at high energy levels, is that the direction of impact be reversible (i.e., up as well as down or laterally in two opposite directions). This condition will allow the resulting shear waves to be identified from compression waves as in the down-hole technique. Controlled impact energy waves at higher energy levels, although readily generated, will not be as easily identified as in existing low energy geophysical tests and thus it is more difficult to measure wave velocity. Faster traveling, low energy (precursor) waves will tend to distort the higher energy waves. Therefore it may prove

more practical to use a vibratory energy source relying on impact sources only as a second alternate.

Several vibratory sources of energy (Table 3.1) are also available, although not presently used in any standard geophysical tests. One such field machine adopted for generating high energy, steady-state adjustable vibrations is a hydraulic powered, vibratory pile driver known as the Vibro-Driver (Model 2-40H), which is supplied by the L. B. Foster Co. Table 3.2 presents the specifications as listed in the 1970 brochure of that company.

Though this equipment is quite large and would require a crane for handling it, the test procedure should be relatively easy to accomplish. As considered, once the anchor is in position in the boring, the crane would position the vibrator over the hole, Fig. 3.2, and the operator would actuate the hydraulic clamp to join the connector rods to the vibrator. Then the test would be ready to begin.

Vibroseis equipment developed by Continental Oil Co. (Duke, 1969) is also a potential high energy vibratory source which could prove useful in earthquake evaluation studies in the future. This equipment has the advantage of being able to change frequencies rapidly at a predetermined rate. Its very high cost will, however, probably limit research and use of this device.

From Chapter 4 and the discussion of control systems for use in the application of vibratory loads to shake tables, various load actuator systems can also provide the necessary energy requirements. The closed loop electrohydraulic system is especially suited for producing large force outputs at both large and small displacements. Also, it is ideally suited for programming various irregular or uniform motion patterns within a reasonable frequency range. This system because of its flexibility, in our opinion, is especially suited for use in producing controlled steady-state shear waves. Because, however, of its high cost, the purchase of such complex equipment for our use is not warranted until the basic test procedure is tried and the equipment capabilities and requirements established.

TABLE 3.1

SHEAR WAVE GENERATING DEVICES

<u>ENERGY SOURCE</u>	<u>TYPE</u>	<u>SOURCE LOCATION</u>	<u>RELATIVE ENERGY LEVEL</u>	<u>REMARKS</u>
*Explosive charge	Impulse	In-hole	Low	Difficult to filter out other wave types. Damages bore hole
*Explosive charge	Impulse	Surface	High	Difficult to filter out other wave types
*Hammer (Horizontal)	Impulse	Surface	Low	Reversible impact direction
*Falling weight	Impulse	Surface	Variable	Rayleigh waves primarily
115 Mechanical impulse ("gun")	Impulse	Surface	High	Power: explosive, electromagnetic, hydraulic, air or mechanical
Vibro-driver	Steady State	Surface	High	High strain levels but costly
Vibroseis (Continental Oil Co.)	Steady State	Surface	High	Developers claim shear wave penetration to 10,000 feet
Electromagnetic	Steady State	In-hole	Low	Very low strain levels. Prototype experimentation by Corps of Engineers (Vibro-packer)
Variable mass vibrator	Steady State	Surface	High	Used by Corps of Engineers for determining the velocity of Rayleigh waves
Load actuator system	Steady State	Surface	High	Very flexible but costly system
Rotating eccentric shaft	Steady State	Surface	High	Inexpensive in relation to the other energy sources

* Methods or equipment most commonly used by commercial geophysical firms in the U.S.A.

TABLE 3.2

VIBRATORY ENERGY SOURCE

Vibrator	L. B. Foster Co. Vibro-Driver
Eccentric moment	1,740 in.-lbs.
Frequency range	0 - 30 cycles/sec.
Amplitude	5/16 to 1 inch
Horsepower	114
Suspended weight	9,700 lbs,
Clamping force	80 tons
Approx. size	103 inch x 34 inch x 106 inch
Rental rate	\$3,500/wk. F.O.B. New Jersey \$10,000/mo. includes operator
Remarks	Need crane to handle and hold during testing.

Other less expensive vibratory energy sources include the variable mass vibrator, a rotating eccentric shaft vibrator or an electromagnetic vibrator. Of these systems, the first two vibrators produce the largest energy and could serve as potential prototype energy sources in the development stages of this test to check the validity of the proposed test.

c. Sensing Equipment

In the cross-hole test sensing equipment is located in adjacent borings at a depth level approximately equal to that of the wave generating anchor. This equipment should be capable of identifying the shear wave velocity by means other than using the time of first arrival. This therefore requires either refined or new procedures for making the necessary measurements. Such refined methods could include: 1) refinements to the down-hole method of measurement using impulse energy sources, or 2) measurement of phase shift, amplitude and wave length characteristics of a steady-state induced wave pattern. Neither procedure to our knowledge, has been used at high strain levels. With respect to sensing equipment in general, we believe additional development is not needed as the electronics of timing and defining wave characteristics are well developed. Sensing equipment is commercially available in many different forms depending on whether it is to be located on the ground surface or locked into a bore-hole. Basically, the sensing elements include one of three primary components; accelerometers, velocity transducers (geophone) or displacement transducers. Of these, the velocity type geophone is the most commonly used, because of its availability and relatively low cost. Depending on the information desired, these geophones may be used individually or mounted in groups of two, three or more units to detect wave motion in any desired orientation. In recent years, the development and use of accelerometers has progressed in the field of geophysics. However, since velocity transducers are the least expensive, and most readily used devices in geophysics, they are presently considered adequate for those studies to be conducted in future work efforts.

In-hole sensing devices need improvement and standardization to develop practical methods of orienting and clamping the units at predetermined depths in both cased and uncased holes of varying diameters. Orientation is often provided in cased holes by lowering geophones in slots cut in the casing prior to its placement. Orientation has been accomplished in uncased holes by lowering triaxial geophones and adjusting the orientation by observing the magnitude of the induced waves. Various simple spring actuated, inflatable membranes, or air piston systems have or are presently being used to fix sensing devices to the bore-hole walls. During the test phases, any of the coupling devices may be used since the fixity may be visually checked in shallow borings. Also, orientation can be maintained by mirrors or lights in these holes. For a final part of design, this aspect of the test equipment must be studied further, especially orientation, to establish the best system or systems.

d. Recording Equipment

Recording equipment for receiving data from the sensors can be about as elaborate and precise as cost permits. It has been found that an experienced geophysicist can hand process and evaluate data on a small research type project with a minimal amount of recording instruments (analog chart recorders or oscilloscopes). On the other hand, for larger projects requiring more extensive studies, a considerable amount of equipment can be acquired to filter, amplify, record, compare, compute and plot the results such that little or no manual work would be required. Though the development of more sophisticated recording equipment would be a benefit to the various geophysical firms, we cannot justify the research and development cost that would be required to further automate the recording and data processing procedures. This latter development may be justified in due course of time, but not until other aspects of the test procedures and equipment are improved.

3.3.2 Down-hole Technique Improvements

Though we presently visualize no individual, major improvements in the down-hole(or up-hole) technique, it is possible that a series of minor developments could combine to result in a significant overall improvement. The following discusses a few aspects of this test which warrant further study and evaluation.

a. Surface Energy Sources

Surface sources which generate shear waves that travel down-ward, are presently available in a number of different forms. That most commonly used is a hand-powered sledge hammer striking a buried post or plate. The use of surface vibrations for the horizontal propagation of Rayleigh waves has also been studied to quite an extent at WES. Since these devices and other surface devices summarized in Table 3.1 can be developed to create large energy forces, but generally not at depth, their use is not contemplated in this study. Because of this energy limitation, the down-hole test procedure appears less desirable for the new test than the cross-hole technique.

b. In-hole Energy Sources

Energy sources, placed in a bore-hole (other than explosives), are presently nonexistent. The myriad of different wave patterns, failure to obtain repeatable results and damage to the hole and other instruments greatly limit the effectiveness of explosive materials as an in-hole energy source for either the up-hole or down-hole test.

The most promising area for development of an in-hole energy source causing an upward or downward propagating wave is in the field of electrohydraulic systems. Other systems would involve inducing lateral energy against the side of the bore-hole by the motion of a heavy mass. In a bore-hole, the mass must be small thus limiting the force output of the system. An electrohydraulic system, on the other hand, can develop large forces through hydraulic pressure and a load actuator. Such an in-hole

pulsating or jacking system could probably be developed for inducing an upward propagating wave. The test set up, as envisioned may be as shown in Fig. 3.3 This test would be especially useful when testing layered soils.

Because such a system would prove to be very costly, its consideration is only justified after the basic principles of the new test are checked and the problems resolved. For establishing the validity of the new test, an experimental test program is planned using the cross-hole technique and less expensive vibrators for the propagation of shear waves. The details of this program are described in the next section.

c. Sensing and Recording Equipment

Equipment for use in either the down-hole or up-hole test will require essentially the same development effort as in the cross-hole test discussed previously. In general, this likewise involves improving methods for making wave measurements as well as improving methods for orienting and clamping sensing units into the walls of the boring.

3.4 FUTURE TESTING, DEVELOPMENT AND EVALUATION

3.4.1 Damping

As described previously, there are no field tests available to measure meaningful values of damping for use in earthquake problems. One concept originally considered a likely possibility was to determine material damping properties in situ under static loading conditions. This idea consisted of statically applying torsion to an in-hole device and measuring the resulting deformation. By knowing the load deformation characteristics, hysteretic damping could be computed. Although the measurement of damping appears feasible using this approach, further studies have posed a number of questions or problems, which lead us to conclude that at this time such a device cannot be developed to produce meaningful data. Therefore further efforts to measure this

FIG. 3.3. SCHEMATIC OF IN SITU TEST PROCEDURE--DOWN-HOLE TECHNIQUE

property in situ are not contemplated.

3.4.2 Shear Modulus

Our studies to date have clearly indicated that there is no well established "state-of-the-art" with respect to in situ or geophysical testing to determine the shear modulus of soils. Considerable data are available for many sites including most nuclear power plant sites. Also, much has been published in North American literature and apparently even more in European literature. Unfortunately, the information published presents only the results of investigations performed. Actual wave records or information on different test procedures are seldom presented with the results by the few firms that conduct such tests. Since backup data are not available, it is not possible to assess the procedures used, or the results obtained, except in those cases where we have personal knowledge. Though the Corps of Engineers (W E S) has contributed significantly to the recent literature, none of their work ties the different aspects of in situ testing together nor does it attempt to relate them to actual earthquake induced motions.

With no established state-of-the-art from which to work, but with a number of potentially sound ideas and concepts to experiment with, we plan to undertake a program of testing, study, design and evaluation, which conceivably will, tie a number of loose ends together. The first step will be to conduct a simple field testing program at a site with uniform soil conditions and fairly well-established soil parameters. This program will include drilling shallow holes, performing tests and studying the results to resolve the following eight, as yet undefined factors:

- 1) The energy requirements (both magnitude and type).
- 2) The detector spacing required to record strains in the earthquake range.
- 3) The method of time-distance measurements for velocity determinations.
- 4) The clarity of the induced wave motion.

- 5) Anchorage and bore-hole coupling requirements.
- 6) The feasibility of measuring or computing shear strains.
- 7) The overall feasibility of developing such a device or procedure.
- 8) The test limitations.

Based on our review of the various methods described previously, it is our opinion that most of these factors and thus the resulting wave measurements can most efficiently be resolved using the cross-hole procedure, a vibrating energy source and velocity transducers. Using this technique, generally as shown in Fig. 3.2, the primary objective will be to generate continuous relatively clean shear waves, and determine their velocity, amplitude, and wave length for a known relatively low frequency. From these determinations, it is then theoretically possible to calculate both the shear modulus and the corresponding strain level.

Techniques and equipment for making these measurements and determinations are not currently available. For the generation of shear waves we plan to use a rotating eccentric arm connected to a down-hole anchor, since it, in our opinion, is most likely to produce a fairly clear, sine wave at low frequencies (less than 50 cps) of sufficient amplitude. The maximum force output for this field program is expected to be in the order of 3-5,000 pounds. Experimentation with this moderately large energy device should be sufficient to establish hole spacing requirements, wave measurement and accuracy requirements and relative strain levels. If sufficient strain levels can be induced only with very close hole spacings, thus reducing the accuracy of velocity measurements, this vibrator would serve as a prototype model to establish the base requirements for the design of larger test equipment and/or more accurate recording equipment. Such equipment, especially hydraulic high energy load actuators, are commercially available, but costly.

Velocity determinations are planned by measuring the phase shift of a given wave passing sequentially through two

detectors. There are at least two general methods for making these measurements, both being dependent upon the generation of a relatively pure sine wave. One method involves measuring the phase relationship on a dual-trace oscilloscope or multi-channel oscillograph, while the other procedure involves projecting the output of the two pickup devices simultaneously on the vertical and horizontal trace on the face of an oscilloscope, producing either a straight line, circle or ellipse. Placing a mask with angular relationships over the face of the oscilloscope permits the phase shift to be read directly. These methods of measurement and the use of a coupled eccentric arm energy source are different from those normally used in geophysics. Normally, wave patterns are generated by impulse loadings and the velocities determined by recording the time of first arrival of the wave traveling between two points. When dealing with larger energy waves, a precursor wave pattern (faster traveling low energy wave pattern) may precede as well as distort the first arrival of the larger energy wave. Results using impulse energy sources may lead to misleading velocity determinations and therefore will be considered only as a second alternative.

Assuming that the shear wave velocity and thus the shear modulus may be established by measuring the phase shift of the steady-state wave, the next step is to establish the corresponding strain level. We plan to accomplish this by recording the time history of velocity at each pickup at a given frequency. This record will permit displacement amplitudes to be determined. From these data and the following relationships, the peak strain level may then be determined:

$$V_s = f\lambda \quad (3.1)$$

$$\gamma = \frac{\pi\Delta}{\lambda} \quad (3.2)$$

where V_s = velocity of shear wave at generated (computed) strain level, f = frequency of generated shear wave, λ = wave length of generated shear wave, Δ = peak to peak particle displacement at point of detection (measured) and γ = the strain level.

Once a representative collection of data is available, the test program can then be assessed and appropriate modifications in anchor systems, testing equipment or procedures made. Also it would then be possible to relate energy levels and corresponding bore-hole spacings used in the above program to other possible larger energy systems or schemes. Subsequently, our effort would bring together and resolve all eight of the unknown factors regarding the art of in situ testing at strains approaching earthquake magnitudes.

In order to accomplish this end, it is important that this initial program be kept quite flexible so as to permit changes to be made to both the apparatus and the technique as the tests are in progress.

Following the completion of this initial simple field program and an evaluation of the data and procedures, the next step would consist of a period of design, fabrication and bench testing of the more refined equipment, where found necessary. This step would then be followed by an extensive field testing program, aimed at collecting data in situ with both existing geophysical tests and the newly developed test, and also laboratory data at select sites for the soil types of interest. Both laboratory and field data would then be compared and the results assessed.

After a number of tests are performed in different soil types, the problems involving installation, measurements and instrumentation should be reasonably apparent. Also, the limitations of the test should be apparent. From experience gained with these problems by actual testing, the procedure and its applicability to actual site selection can be assessed. Also, any refinements to the equipment or procedures can be made to make the procedure more standardized and/or sophisticated.

At the completion of these tests, a detailed report would be prepared to assess the state of the art of this new testing technique. This report would conceivably be the basis for instruction of persons wishing to perform this test at a potential

power plant site. If the performance of this test proves too costly for more or less routine site investigations, the results obtained from the extended field test program for the different soil types would probably provide considerable research data to allow a correlation with the existing less costly laboratory tests and in-situ tests to be made.

During the second year of subcontract work, the joint venture plans to advance development of the various techniques through a simple field testing program to resolve those questions which can best be answered through actual field experimentation. Then, in so far as funds permit, initial efforts will be directed toward fabrication or assembly of the test hardware. Details of this program are described in Chapter 6.

CHAPTER 4

SHAKE TABLE FEASIBILITY STUDIES FOR SOIL LIQUEFACTION

The application of cyclic loads or vibrations to either natural or man-made soil deposits has created unstable soil conditions (primarily liquefaction or subsidence in granular materials, and subsidence and lateral displacements in clayey soils) and in some cases excessive distress or damage to buildings founded, within this deposit. Since it is not possible to predict when and where an earthquake will occur, researchers are forced to direct their efforts toward earthquake simulation to predict the effects of these vibrations on soil deposits, earth structures and buildings. One approach to this problem is to study the behavior of a given system by performing large scale tests in the laboratory using shake table facilities.

As defined in the "Revised Program Definition" report, this study is oriented toward the feasibility of utilizing shake tables as a means for studying soil liquefaction. By testing large scale specimens of soil, it is hoped that many of the deficiencies in the smaller scale cyclic simple shear and triaxial tests can be eliminated. From the more recent work by others and our current studies, it is apparent that the use of this test method for studying liquefaction is likely to produce meaningful results. It is also apparent that the state of knowledge regarding the beneficial uses of shake table facilities is not well known. The relatively high cost, its specialized nature and thus the infrequent performance of such tests have prevented engineers from gaining the optimum knowledge from this potentially fruitful source of experimental data. For this reason, the information provided in the following sections is intended to accomplish a two-fold purpose. First, as the title implies, is a general feasibility study of shake tables. This part, contained in the next four sections, was prepared to describe and evaluate the information that may be or has been derived from shake tables. Also included is a brief historical review, a summary of available shake tables on the west coast, and a general

evaluation of existing or potential systems for supporting and applying vibratory motion to a shake table.

The second part of this study, in the last section, contains the results of studies to identify future ways in which liquefaction may be studied further. From this evaluation, conclusions are presented along with our recommendations for the most promising paths for advancing the state of knowledge in this area.

4.1 GENERAL INFORMATION

A shake table is a laboratory device which can support a soil system, structural system or a combination of the two and yet move in one, two or three directions so as to impart shear stresses and strains dynamically to the supported system. On many shake tables this motion may be applied as an impulse force, a steady state motion (sinusoidal, triangular, ramp, rectangular, etc.), or in a specific controlled irregular pattern as encountered during actual earthquakes. In soil dynamics, the first two forms of motion are well suited for studies involving the effects of explosions (shock vibrations) and machine vibrations (steady state vibrations) respectively while the last complex pattern is most applicable for accurate earthquake simulation. However, because of the complexity of analyzing random stress patterns, the sinusoidal loading pattern has more often been used in shake table studies as a simplified form of motion for earthquakes.

Other motion characteristics of a shake table such as frequency, displacement, velocity and accelerations will also govern the limitations of tests that can be performed. The magnitude or range of these characteristics depends upon the approach taken to the investigation being performed. For example there are two different approaches to the investigation of the response of a soil system to base motion using a shake table. One approach is to build a small earth structure and subject it to a prescribed base motion. The response of the structure is then used to check the applicability of theoretical methods for predicting response. With this philosophy, the test structure is not a model but is analyzed as a structure in its own right.

The other approach is to build a "model" of a real soil structure and to subject it to suitably scaled earthquake type motions. With this approach the scaling factors have to be taken into account. An earthquake is then selected with a time scale such that the natural frequency of the specimen is within the predominant frequencies of the earthquake spectrum.

Freeman, 1932, recognized the importance of such research as the following excerpt indicates. Referring to earthquake research using shake tables, he writes: "No line of attack on the dynamics and kinematics involved in so designing a structure that it shall be earthquake resistant, now presents greater promise of valuable results." Since 1932 and before, shake tables have gradually increased in size from small boxes to one presently being considered having dimensions of 100 feet by 100 feet. Details regarding this historical development are described briefly in the next section.

4.2 DEVELOPMENT OF SHAKE TABLES

Many studies employing vibrating platforms or shaking tables have been made in the past to obtain a qualitative idea regarding the response of soil masses to dynamic loads. As technology developed, more sophisticated shaking tables and instrumentation became available and investigators obtained quantitative data on the response of soil masses.

The first reported tests using a shake table were carried out by F. J. Rogers, 1906, of Stanford University as part of the research activity which followed the San Francisco earthquake of 1906. The shake table consisted of a small carriage on steel rollers, on which was placed a 40-in. by 30-in. by 12-in. deep box containing the soil. The table was driven in one horizontal direction by means of a connecting rod and a crank of variable throw. The power was supplied by an electric motor.

In 1929, Willis designed a 10-ft. by 12-ft. shake table for Stanford University which could be used with two distinct types of uni-directional motion. The forcing functions were: 1) damped

free vibration resulting from elastic impact of a pendulum on the table, and 2) a continuous forced vibration set up by a revolving unbalanced fly wheel of variable speed and variable eccentric balance, mounted on the table. The table was restrained by large springs attached to heavy concrete piers.

In 1958, Clough and Pirtz reported model test results on rockfill dams using a 7-ft. by 10-ft. shake table at Berkeley. This table had an 8-inch thick concrete slab base which was supported by flexible steel legs. Motion was imparted to the table by a 150-pound pendulum striking against a buffer spring at one end of the table. The other end of the table was anchored by a heavy spring which controlled its frequency of vibration.

A considerable amount of dynamic model testing has been conducted at the Experimental Institute for Models and Structures in Bergamo, Italy. A report of facilities in 1960 described a shaking platform, 10-ft. by 15-ft. in plan, suspended from a rigid steel frame by means of springs. With this configuration it is possible to apply both horizontal and vertical vibrations.

In 1960, the Electric Power Development Company of Tokyo built a 5-m. by 5-m. shake table for studies on dams. The table can be driven either horizontally or vertically using an eccentric rotating mass drive with a frequency range of 7 to 50 cycles per second and a maximum exciting force of 112 kips.

In 1965, Rocha reported on the facilities for carrying out earthquake studies on model dams at the National Laboratory of Civil Engineering, Lisbon. Vibrations were produced by an electromagnetic shaker with a frequency range from 5 to 5,000 cycles per second. The power output of the shaker limited the system to the testing of small models. The shaker could be driven by either a sine wave generator or a random voltage generator.

In the past few years universities and several governmental agencies have constructed various size shake tables for research studies. On the west coast, three are presently available, one is under construction and a large one (100-ft. by 100-ft.) is still being considered. The following paragraphs briefly describe the capabilities of these existing or proposed shake tables.

The most familiar shake table on the west coast is the one located at the University of California, Richmond Field Station. This facility reported by Kovacs, 1968; Silver & Seed, 1969; and Arango, 1971; consists of a concrete slab 7 ft. wide and 10 ft. long. It is supported by four cantilever leaf springs that allow free horizontal movement, but prevent longitudinal or rotational vibrations. Where necessary, plywood and aluminum channel sides have been used to contain the water or soil of the test specimen. A structural steel beam spans across the table to provide instrument support. The driving force for the table is supplied by a closed loop servo-hydraulic system acting through a 10-kip hydraulic piston which is connected to the shaking table. The system can be programmed so that various types of periodic motion can be applied to the table as either controlled displacement or acceleration. In addition, a curve-following device is available for supplying random wave forms such as records from earthquakes over a frequency range from 0.001 to 40 cycles per second; however, the leaf spring supports restrict the horizontal table movement to approximately $\pm 1/2$ -in., thus limiting the maximum acceleration that can be achieved at the lower frequency levels.

Also at the University of California, a 20-ft. by 20-ft. reinforced and prestressed concrete shaking table (Stephen et al, 1969) has recently been completed and is now operable. This table essentially serves as a model of the large table described in a subsequent paragraph. This working model is capable of accommodating a weight up to 120 kips and can simulate ground motions with accelerations of $2/3$ the force of gravity up to a maximum velocity of 25 inches per second in the horizontal direction, and a corresponding $2/9$ the force of gravity and 10 inches per second in the vertical direction. During operation, the table floats on a cushion of compressed air and is driven by a system of hydraulic actuators, four oriented vertically and three horizontally. The resultant horizontal and vertical motions are regulated by an electrohydraulic control system to simulate the motions recorded during past earthquakes.

The University of British Columbia shaking table, (Finn, Campanella, and Aoki, 1969) is 6 ft. wide and 9 ft. long. It is constrained to move in one horizontal direction by four horizontal, V-slotted needle bearings. The table is actuated by a hydraulic ram mounted on the fixed base frame and connected to the table through a rigid link. A 3 gal/min. pump supplies oil at 3000 psi to the ram through an electronically controlled servo-valve. The ram is a double-acting, double-ended hydraulic piston with a ± 2 in. stroke, and has a dynamic capacity of 25 kips., and a maximum velocity of about 17 in/sec. at full load. Specified motions are transmitted to the table under the control of an MTS earthquake simulator control console.

Under construction at the University of Washington is a shake table with dimensions of 8 ft. by 10 ft. The table will have one degree of freedom in the horizontal plane. The driving force for the table is supplied by a closed loop servo-hydraulic system acting through a 35-kip hydraulic piston, which is connected to the shaking table. The motion of the table will be programmable such that irregular motions can be followed. The table will be supported by four air bearings at the corners. The major advantage of this system is that it will provide large force outputs as well as large or small displacements.

A shake table much larger than any existing table is being considered for construction at the University of California, Richmond Field Station. This table will have dimensions of 100 ft. by 100 ft. and will weigh about 2,200 kips. A detailed feasibility study has been carried out at the University of California, Berkeley, (Penzien et al, 1967) and the working model described above, was subsequently constructed to establish the requirements for such a table. This big table will have three degrees of freedom and its motion will be governed by closed loop electrohydraulic control systems. The steel table will be driven by twenty 200-kip actuators in each of the two horizontal directions, and by twelve similar actuators in the vertical direction. Displacements will be limited to ± 6 inches horizontally and ± 2 inches vertically.

4.3 SHAKE TABLE SYSTEMS

Each of the presently designed shake tables has special features which makes it more versatile for some uses than for others. For accurate earthquake simulation, a shake table should have three degrees of freedom. Unfortunately, the development of such a system involves enormous expenditures (Penzien et al, 1967). Thus research to date has been conducted using single degree of freedom simulators. These simulators, although not as desirable as a three degree of freedom system, have and will in the future provide valuable results in studying soils subjected to earthquake-like vibrations.

Design features, which make single degree of freedom shake tables different from each other, are their force capacities, their motion control systems, and their vertical support systems. To familiarize the reader with the different shake table hardware and their limitations, each support and control system is briefly identified in the following paragraphs.

4.3.1 Motion Control Systems

At least four methods of controlling shake table motions exist. Each system with its general limitation is discussed in the following paragraphs.

a. Eccentric Rotating Mass

As the name implies, table motions are achieved merely by rotating an eccentric mass. This is one of the simplest and first used methods of vibrating a shake table but is limited in that only sinusoidal motion can be produced.

b. Pendulum-spring System

In such a system, the horizontal motion of the table is restricted by springs, and the table is struck by a pendulum. This causes damped horizontal vibration of the table. This method is limited with respect to the nature of the motion.

c. Electromagnetic Vibrator

An electromagnetic vibrator has the advantage that it can be programmed for random vibrations within a very wide range of frequencies. It has, however, several severe limitations. It is not suited to producing high force output, and so would limit testing to small light models. Also, it is only suitable for small displacements.

d. Closed Loop Electrohydraulic System

Such a system does not have the very large operating frequency range of the electromagnetic vibrator but is much more flexible in other aspects. It can produce large force outputs and can be used for both large and small displacements. A closed loop system is ideally suited to the programming of irregular motions. Such a system is considered to be the most suitable method for driving a shake table. The basic philosophy of such a system is the comparison of the desired condition of a controlled parameter with the actual condition of the parameter, and the generation of correction signals which cause the actual to equal the desired condition.

One disadvantage is that the maximum displacement attainable by shake tables decreases as the frequency increases. This does not represent a serious limitation because for earthquake motion (for example, the El Centro earthquake) the large displacements correspond to the low-frequency components and the small displacements to the high-frequency components.

4.3.2. Supporting Systems

In order to test heavy systems (such as a soil bed) the vertical supporting system must be capable of both supporting the model as well as permitting its lateral motion with a minimum amount of resistance. The following paragraphs discuss briefly the six potential or presently available systems.

a. Flexure Plate System

This method, as shown in Fig. 4.1 involves supporting the table on plates which are flexible enough to permit the required horizontal displacements. Such a scheme is only feasible for small displacements. The flexure plate system is being used at the University of California, Richmond Field Station, and also at the University of Illinois. The advantage of this system is that it has zero friction and is a relatively inexpensive approach.

b. Suspended System

In such a system, the table is suspended from a rigid framework using high tensile wire and springs as illustrated in Fig. 4.2. The shake table at the Experimental Institute for Models and Structures in Bergamo, Italy, operates on this principle. This method is useful if the table is to be subjected to motion simultaneously in three orthogonal directions. Such a system presents difficulties if it is desired to study linear motion in one horizontal direction, since guides would be necessary to prevent undesirable oscillations of the table.

c. Air Cushion Support System

It is possible to support the dead weight of the table and test structure by means of compressed air acting on the underside of the table. A flexible seal around the periphery of the table would prevent air loss and still allow table movement. A stabilizer system of hydraulic jacks is necessary to prevent table rotations in the vertical plane due to overturning moments, and to prevent twisting about a vertical axis. Such a system is being used for the 20-ft. by 20-ft. shake table at the University of California, and has been proposed for use with the even larger 100-ft. by 100-ft. table.

d. Linear Motion Bearing System

Such bearings limit the table movement to linear motion in one horizontal direction. The bearings could be of roller

FIG. 4.1. TABLE ON FLEXURE PLATES

FIG. 4.2. TABLE SUSPENDED FROM A RIGID FRAMEWORK

bearing assemblies. This technique requires precise control in placement of the bearings in order to keep friction to a low level.

e. Hydrostatic Bearing System

Hydrostatic bearings provide a low level of friction but, depending on stroke and overturning moments, they may be the most expensive.

f. Air Bearing System

This bearing system, planned for use on the new University of Washington shake table, has the advantage of being inexpensive, has zero friction, and does not transmit any vibration from the actuator system to the table. It has the disadvantage that the table floating on air has six degrees of freedom and to keep its motion in one direction will require a side control.

4.4 INFORMATION GENERATED FROM SHAKE TABLES

Although shake tables have not yet made significant contributions to the study of soil mechanics, at least not quantitatively, they offer the possibility of being useful in soil dynamics research, especially for generating information concerning soil behavior under simulated earthquake loading conditions. Generally, three major areas would benefit, namely: 1) soil properties, 2) response studies, and 3) soil behavior characteristics. Within these areas listed in Table 4.1, are several factors about which information may be derived from shake table tests either by direct measurement, by study and evaluation, or by comparison with analytical techniques or data used for determining the behavior of real systems under actual earthquake loading conditions.

In general, the shake table is used as an experimental tool to determine soil properties and/or to support, compare, verify or evaluate new or proposed theoretical analytical techniques, or empirical procedures for predicting the behavior of a soil or structural system under dynamic vibrations. Thus far, the relatively high expense of testing large scale specimens has allowed only

a limited amount of testing to evaluate any one of these factors.

TABLE 4.1

SUMMARY OF INFORMATION GENERATED FROM SHAKE TABLES

1. Soil Properties
 - Shear Modulus
 - Damping Ratio
2. Response Studies
 - For Natural Soil Deposits and Earth Structures
3. Soil Behavior Characteristics
 - Settlement
 - Liquefaction

Also, such factors as modeling, scaling, sample preparation, the elimination of boundary conditions, saturation effects, surcharge pressures, the measurement of internal stresses and strains, and others have not been given sufficient consideration to allow meaningful experimental shake table tests to be made in certain of these areas. The following subsections summarize the present state of accomplishment for obtaining information regarding these factors and provide an evaluation of the overall need or lack of need for additional experimental tests or studies. The general discussions presented herein point out the usefulness of shake tables for this purpose.

4.4.1 Dynamic Shear Modulus

The shear modulus defines the stress strain relationship, or "stiffness", of a soil deposit prior to failure. Based on reported laboratory data, the magnitude of the shear modulus is sensitive to strain and may decrease significantly during a large earthquake, at say strains on the order of 0.1 percent, as compared to the modulus during a smaller earthquake, in which strains may be of the order of 0.01 percent or less. Because the modulus is

variable and affects the magnitude of the computed ground response and the computed settlement (Silver and Seed, 1969) it must be determined for the depths and strains at levels of concern.

Dynamic shear modulus is presently determined by in situ field tests using various geophysical techniques or in the laboratory from resonant-column tests, cyclic triaxial compression tests or several of the more specialized loading tests. These tests have been discussed and the method of computation explained in SW-AJA (1972). Shear modulus values for cyclic loading conditions may also be computed from shake table tests, by either the free vibration method or the forced vibration method.

The free vibration procedure involves initially exciting the base of the specimen and subsequently measuring the vibration responses when the shake table excitation is shut off. From the theory of transverse vibration of shear beams (for a soil layer free of contact with the walls), it is possible to compute the shear modulus for a known period or frequency of free vibration.

The forced vibration procedure for determining the shear modulus involves varying the frequency of vibration of the specimen until a resonant condition is obtained. The shear modulus is then calculated from the observed resonant frequency and the theory of elasticity. Both techniques are expensive and so far as we are aware have only been performed and subsequently reported for clay materials (Goodman and Appuhn, 1966; Kovacs, Seed and Chan, 1971; Arango, 1971).

A comparison of the shear moduli, as determined by cyclic simple-shear tests and shaking table tests, is shown in Fig. 4.3. From the figure we can see that the overall relationship between the shear modulus and shear strain, determined by the various test procedures, appears to be satisfactory despite experimental scatter.

As indicated above, less expensive small scale tests are available to determine reliable values of the shear modulus in the laboratory, hence we can state that the performance of such tests together with sound engineering judgement appear to provide

FIG. 4.3 COMPARISON OF THE SHEAR MODULUS/UNDRAINED SHEAR STRENGTH--SHEAR STRAIN RELATIONSHIPS (FROM KOVACS, SEED & CHAN, 1971)

FIG. 4.4 COMPARISON OF DAMPING VALUES FROM FREE VIBRATION AND CYCLIC SIMPLE SHEAR TESTS (FROM KOVACS, SEED & CHAN, 1971)

a basis for establishing reasonable values of shear modulus for most practical engineering problems. Therefore, further efforts in this area should be directed toward the development of new or improved tests for determining the shear modulus in situ at higher strain levels than presently possible with geophysical procedures. Chapter 3 presents a discussion of the feasibility and preliminary plans for accomplishing this goal.

4.4.2 Damping Factor

Ground response involving no permanent soil displacements is determined mainly by the shear modulus and damping characteristics of the soil system. The mathematical formulation of expressions for the damping forces in a soil system subjected to dynamic loading poses a difficult problem that still requires extensive research. Unlike mass or stiffness, damping is not necessarily an inherent property of the system. Damping forces depend not only on the oscillating system but also on the surrounding medium. Some of the energy which will be delivered to a system by shaking will be dissipated by energy consumed internally by hysteretic stress-strain phenomena in the material itself, and externally (called radiation or geometric damping) through energy dissipated by radiation into the surrounding medium.

A convenient concept for attempting to measure the rate of energy dissipation is the concept of equivalent viscous damping. This concept utilizes imaginary viscous dashpots attached to the system to dissipate the same amount of energy per cycle as is actually dissipated in the system by other means. The main reason for choosing viscous damping (force proportional to the first power of the velocity) is that it gives a reasonable approximation and leads to the simplest mathematical treatment.

From preliminary studies of laboratory data, the damping ratio for most soils expressed as a percentage of critical damping may range from 2 percent for small strains induced by earthquakes of small magnitude to 20 percent for very large strains, which may exceed even the range of interest for large earthquakes.

This property, like the modulus, is sensitive to strains and has an effect on the magnitude of ground response. Damping can be measured from resonant column tests, cyclic simple shear tests, cyclic triaxial compression tests, torsional column loading tests and shake table tests. Of these tests, only the cyclic triaxial test is performed routinely in conventional testing laboratories. The following paragraph will show briefly how to determine the equivalent damping ratio by shaking table tests. The other tests have been discussed by SW-AJA, 1972.

Damping in a shake table test may be measured either by the forced vibration method or by the free vibration method. Using forced vibrations, described in the previous section, the damping ratio may be found from resonance curves in the normalized frequency response curve by the formula:

$$\lambda = \frac{\Delta f}{2f_o} \quad (4.1)$$

where λ = the damping factor, f_o = the resonant frequency, and Δf = difference in frequency of the two points on the resonance curve, with amplitude of $1/\sqrt{2}$ times the resonant amplitude.

Using free vibrations, a time response may be obtained by vibrating the system at any convenient frequency and recording the resulting transient vibration when the machine is stopped. The damping ratio is then computed using the following relationship:

$$\lambda = \frac{1}{2\pi m} \ln \frac{A_n}{A_{n+m}} \quad (4.2)$$

where A_n = the amplitude ordinate of the n^{th} cycle and A_{n+m} = the amplitude ordinate of the $(n+m)^{\text{th}}$ cycle.

A comparison of the damping factors as determined by cyclic simple shear tests, with the free vibration shaking table tests was made by Kovacs, Seed and Chan, 1971. Their results, summarized in Fig. 4.4, show that the values of damping determined from the

simple shear tests are several percent lower than those determined by free vibration shake table tests. This difference appears small when considering the overall scatter in the data used to establish these curves. Since reasonably comparable results are obtained when testing smaller scale specimens, it seems likely that these less expensive tests will continue to be performed for use in practical engineering problems. The role of shake tables in future studies to measure damping is therefore likely to diminish for three reasons: 1) less expensive tests can be used to obtain nearly the same results, 2) the effects of sample disturbance in large specimens are likely to be as significant if not more so than errors associated with some of the smaller scale tests, making a correlation of results with field behavior even more questionable, 3) sample preparation techniques presently allow only very soft, specially prepared clays or clean sands to be tested. Of these soils, only sands of various densities are normally encountered in practical engineering problems. Therefore any future work to measure this property on shake tables is likely to be concentrated on sand deposits.

4.4.3 Response of Soil Deposits

Comparisons of earthquake records made in bedrock or a base level with those made at the ground surface show that the motion at the top of the soil deposit differs generally from the motion at the base. Likewise, when the ground surface is irregular, consisting of an earth slope, dam, embankment, or the like, the records of motion at various points on the irregular surface also may vary considerably with respect to each other as well as the base motion. This modification of the input motion characteristics by the local soil condition of a particular site is termed "soil amplification." Amplification of a given input motion is a function of the modulus and damping of the soil deposit, the thickness of the deposit and, where the surface or bedrock topography is irregular, its geometry. The result is that the amplitude is increased in some ranges of frequencies and decreased in others.

In general, the softer the soil through which the shear waves are propagated, the more the high frequency waves are filtered out, and the greater is the amplification of wave motions that have frequencies corresponding to the relatively low natural frequencies of the soil structure.

Several analytical methods are being used in predicting ground response. Presently, these methods include wave propagation analysis, finite element analysis, lumped mass analysis and classical closed form solutions. When irregular surface or base rock configurations or features are being considered, a two dimensional method of analysis is required. Of the above methods, the two dimensional finite element procedure has been found to be especially useful for these studies. Unfortunately, all analytical methods require the idealization of the soil system into some mathematical model, and in many cases the extent to which this model reflects the pertinent characteristics of the site is not known. To test the accuracy of the various analytical procedures it would be desirable to locate a site for which extensive soil property data and both strong bedrock and surface motion records were available. Since such field data are not readily available, one method of obtaining these data, is by testing large soil masses utilizing shake tables to study soil response by performing controlled experimental tests on both beds of soil as well as sloping banks of soil.

One of the first to use a shaking table to investigate experimentally the response of dry sand subjected to harmonic motions of variable displacements and frequencies was Rogers, 1906. Rogers found that the motion of the sand relative to the table increased from the bottom to the top of the soil mass.

Clough and Pirtz, 1958, used a shaking table test to investigate, qualitatively, the seismic behavior of rock fill dams with sloping clay cores. Prototype structures were analyzed by models using similitude considerations. Accelerations and resultant displacements were measured. Other similar shaking table studies on rock fill dams were reported by Seed and Clough, 1963,

for the Portage Mountain Dam.

Shaking table studies on purely cohesionless embankments were made by Seed and Goodman, 1964, in which the effects of ground accelerations on the stability of slopes were investigated.

Bustamante, 1965, also conducted shaking table experiments on model embankments composed of cohesionless materials. Similar conditions, the influence of the slope inclination on behavior, and the determination of the deformation mechanism were investigated.

Goodman and Appuhn, 1966, undertook a laboratory test program on model soil-filled basins subjected to harmonic shaking. The shaking table was 3 in. deep, 6 in. wide and 24 in. long, with a rigid base and walls. A special soft clay mixture of 25 percent montmorillonite and 75 percent kaolinite was used. The water content of the clay material was varied to give a range of soil properties. It was found that the magnification of the ground motion and the natural frequency of the soil mass were dependent on the magnitude of input base motion and that the soil responded much more to horizontal motion than to vertical motion. As a more recent follow-on to their efforts, Arango, 1971, studied and reported the response of embankments 10-feet long by 5-feet wide and 0.5-feet high, built with the same clay mixture, but at a higher water content. Table accelerations up to 0.5 g and frequencies between two and six cycles per second were used in that investigation.

Kovacs, 1968, studied the response of clay embankments to applied base motions. A number of small scale clay embankments were constructed on the shaking table and small unbonded strain gage accelerometers were placed at selected locations to measure acceleration on the surface of the embankments. The frequency and amplitude of the base acceleration, and the geometry of the embankments were varied during the testing program. The dynamic properties of the clay material obtained from cyclic simple shear tests were used in a finite element analysis to compute the response over the length of the embankment. The comparison of the

experimental response with the theoretical values showed that the agreement was highly satisfactory in the regions behind the crest and beyond the toe of the slope for the cases studied.

Finn, Campanella and Aoki, 1969, ran experiments on a large level bed of sand to determine the fundamental frequency of the layer. The ratio of the acceleration at the surface of the model to that of the base was plotted over a wide range of frequencies, and a pronounced magnification of the ratio was noted at a frequency of approximately 50 cycles per sec. The boundary effects on the response were also studied.

In Silver and Seed's 1969 investigation of the applicability of a suggested method for calculating the settlement of sand layers subjected to horizontal accelerations, they found that there was reasonable agreement between the computed surface accelerations and the surface accelerations measured in the shaking table tests. Agreement in the range of 15 to 30 percent was obtained.

Similar studies involving the direct measurement of the response of earth embankments or deposits in the field and their comparison with results predicted by analytical approaches are highly desirable. Since it would be somewhat costly to excite such a large prototype structure and measure the response in situ, a laboratory study with shake table facilities would still appear to provide one expedient method of partially fulfilling this need. More research and thus information in the near future is likely to be forthcoming mostly from those universities that possess or are constructing shake table facilities.

4.4.4 Settlement Characteristics

Ground vibration induces compaction of cohesionless sand deposits which in turn produces settlement of the ground surface. Excessive settlement may cause damage to structures supported by this material. Engineers concerned with seismic disturbances therefore should be able to predict the volume change potential that might be expected to develop from ground motions of known form and characteristics. Of the various methods available,

shaking table tests are probably the most convenient means for actually measuring the settlement of sand layers under dynamic loading.

D'Appolonia and D'Appolonia, 1967, and Whitman and De Pablo, 1969, recognized from laboratory studies of vibrational compaction of sand, that the combination of vertical accelerations accompanied by repeated loads on the sand surface may be responsible for the increased density observed in maximum density tests where dead weight surcharge is used. However, these factors have little influence on the settlement of sands during earthquakes where cyclic shear strains are the primary mechanism for deforming the soil particles and causing them to move into a denser packing.

As the simple shear test closely duplicates the loading imposed on a soil "element" during a seismic event, it should be possible to predict the settlement of a deposit of cohesionless soil due to seismic shaking, by noting the time history of settlement of samples in cyclic simple shear tests conducted under various conditions. From the time history of vertical deformation of a specimen tested in simple shear, the vertical strain due to compaction can be calculated as the total vertical deformation taken at the end of any strain cycle divided by the initial height of the specimen.

To be able to predict the ground surface settlements in deposits of cohesionless soil caused by ground shaking, Silver and Seed, 1969, developed an analytical tool that utilizes existing computer techniques and material properties obtained from simple shear tests to determine ground surface settlements in horizontal deposits of dry sands caused by ground shaking. To check the accuracy of their method, calculated settlements were compared with settlements observed in shaking table tests. From the comparison, they noted that while in any one case differences between actual and computed settlements may be as much as 50 percent, the overall degree of agreement was remarkably good. Silver and Seed, 1969, stated: "it would be reasonable to assume therefore, that the proposed technique might be used as a first approximation for

estimating the volume change tendencies and the ground surface settlements induced in deposits of dry cohesionless soil caused by seismic shaking."

We can conclude that the shake table has been important in past work in assessing the potential settlement of deposits of cohesionless soils during ground shaking. However, at present it seems doubtful that further progress will be forthcoming in this area for some time, because of the difficulties encountered in preparing and testing large samples other than dry sands. In nuclear power plant design, sites, where very loose materials are encountered, are usually found unacceptable from static foundation conditions, namely settlement. These sites can usually be treated satisfactorily by removing and replacing or recompacting the on-site soils to a denser state.

4.4.5 Liquefaction

During seismic shaking, a soil element is subjected to a series of random motions. Cyclic shear strains deform the element causing rearrangement of soil particles in granular deposits resulting in volume change of the soil. If the soil is saturated and drainage cannot occur, these volume change tendencies will lead to changes in the pore water pressure. Thus if the cyclic motion causes densification, a rise in pore pressure will occur and, if large enough, may reduce the shearing strength of the sand soil to the point where the soil may either undergo excessive strain or lose its supporting capacity. If the pore water pressure becomes equal to the overburden pressure, the effective stress becomes zero, and liquefaction occurs. The term "liquefaction" has been applied to the process by which a saturated mass of soil is caused by external forces to suddenly lose its shearing strength and behave as a liquid. Much of the damage which occurred in the Chilean earthquake in May, 1960, and in Niigata, Japan, and Alaska during major earthquakes in 1964, was caused by the liquefaction and resulting subsidence of foundation soils, or by liquefaction-induced landslides. This damage has focused attention on research to determine the significant parameters affecting the liquefaction

potential of saturated sands.

Several researchers who have recently reported the results of liquefaction studies on large scale samples in shake table tests include Nunnally, Yoshimi, Whitman, and Finn et al. Nunnally, 1966, ran tests on four sands having different gradation, shape, and roughness characteristics to make comparisons of the effect of these factors. The apparatus consisted of a rectangular tank with a length and width of 15 cm. on each side and a height of approximately 100 cm. Loose sand was placed in the tank, saturated, surcharged with lead blocks, and subjected to vibrations with blows of a hammer weighing approximately 2 pounds and free-falling a given distance. The magnitude of shock applied was determined by the peak acceleration measured by an accelerometer attached to the base of the tank. Pore pressures were also measured at four points on the side of the tank.

Yoshimi, 1967, ran experiments in Japan to study qualitatively the factors that influence the conditions producing liquefaction in a saturated mass of loose sand within impervious boundaries. Loose saturated sand was placed in a rigid box, covered with an impervious membrane, and subjected to horizontal vibration while a surcharge was applied on the membrane. The box was of welded steel construction with interior dimensions of 50 cm. by 25 cm. by 27 cm., having a porous plate in the bottom and a glass window in a side wall. The box was subjected to horizontal vibration in the longitudinal direction of the box, and the pore pressure and acceleration were recorded on the oscillograph.

Whitman, 1970c, reports results on a shaking table study performed in Chile. The table was about 55 in. long by about 35 in. wide. The sand has a depth of about 15 in. Steady state vibrations, with a frequency of 4.4 cps, were applied over an interval of 4 minutes. Settlements of the surface of the sand were measured following each run. Pore pressures were measured during the run by standpipes at the side of the container, at various depths below the surface of the sand.

In each case the results are somewhat questionable in view

of the measurement procedures and equipment used. In the tests by Nunnally and Yoshimi, the dimensions of the specimen boundary conditions and the location of pressure measuring devices at the edge of the sample were something less than desirable. The tests performed in Chile likewise had the same undesirable features as well as very small surcharge loads and no provision for preventing water from draining to the top of the specimen, thus relieving any tendency for the pore pressure to build up.

Aware of these potential problems, Finn, Emery, and Gupta, 1970, conducted tests on a saturated sand bed 1-1/2ft. wide by 8 ft. long and 6-1/4 in. deep. The saturated sample was sealed during testing by a rubber membrane. The pore pressure in the model during testing was measured by two pore pressure probes located in the middle of the body of the sample. The rubber membrane was attached along its edges to a rigid cover plate, and air from a constant pressure regulator was introduced into the enclosed space to provide surcharge pressure on the sand sample. With the surcharge, the soil layer corresponds to the field condition of a layer of sand between two impervious boundaries. The specimen was subjected to a sinusoidal acceleration of .25 g and a frequency of 2 cycles per second (Hz).

Finn, Emery, and Gupta observed that when large specimens of saturated sand are vibrated on a shaking table, the pattern of pore water pressure response is similar to that noted in cyclic loading triaxial and simple shear tests, and the resistance to liquefaction failure of a given sand under the excitation of a simulated earthquake depends very much on the surcharge pressure on the sand layer. Even though several of the limitations of the three previously performed shaking table investigations were at least partially taken into account, surcharge pressures (1 to 2 psi) were much less than those present in the field. This has been one of the major limitations in using shake tables to study this phenomenon in a meaningful way.

In assessing the preliminary test data reported by Finn et al., the procedures appear sound; however, the results indicate

that the intensity of shaking required to cause liquefaction was higher than that observed under field conditions or in either the cyclic simple shear or the cyclic triaxial tests. The reason for this significant variation cannot be explained from the data presented.

While shake table tests have not yet been particularly fruitful in the study of soil liquefaction, investigators are gradually developing improvements to minimize the limitations in the apparatus and procedures. The major handicaps are the control of saturation or drainage effects and the application of surcharge pressures, as previously indicated. The work reported by Finn, Emery and Gupta, 1970, has demonstrated important progress in these areas. As these limitations are eliminated and as more sophisticated shaking tables become available, their role in future research on liquefaction is likely to be significant.

4.5 CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDIES OF LIQUEFACTION

Both laboratory investigations and observations of field performance have shown that the liquefaction potential of a soil deposit to earthquake motions depends on the characteristics of the soil, the initial stresses acting on the soil, and the characteristics of the earthquake involved. In addition to saturation, the significant factors include soil type, relative density or void ratio, initial confining pressure, intensity of ground shaking and duration of ground shaking. At the present time, the liquefaction potential of a site is evaluated by: 1) using empirical rules based on the observed field performance, 2) comparing the local soil conditions of the site and design earthquake characteristics with a similar site that has liquefied (usually Niigata), or 3) using the general or simplified procedure described by Seed and Idriss, 1967 and 1970b, respectively. To check the applicability of the above, and of potential new methods to improve our understanding of liquefaction, two avenues of approach show promise for future investigations, namely:

- 1) Field Investigations - A compilation of field data at various sites that have been subjected to earthquakes of known intensity and comparison of soil properties which differ between those sites that have developed liquefaction and those that have remained stable.
- 2) Laboratory Investigation - Continued studies of the various factors and behavior of large size samples in the laboratory when subjected to the same stress reversals as they would encounter in the field.

The following two sections describe the merits of each approach. The final section describes the plans of the SW-AJA joint venture toward advancing the state of knowledge in this area.

4.5.1 Field Investigations

Probably the most logical area for meaningful improvement in the evaluation of soil liquefaction potential lies in increased knowledge obtained from the actual performance of real sites. This additional information comes slowly, especially in the strong motion earthquake range. Actual sites must be instrumented, the distribution and engineering properties of the soils determined by field explorations, and the site subjected to significant earthquakes. This is costly in terms of both time and money. Until more reliable information of this type is obtained to fix more clearly the differences between soil deposits that have or have not liquefied, the results can only be used to empirically assess the liquefaction potential of any site. The collection of these data does, however, provide a basis or rough guide for assessing the results obtained in laboratory studies.

The largest effort to date in compiling these data is reported by Seed and Peacock, 1970. Their investigation reports data from 35 sites in which liquefaction either did or did not occur during earthquake shaking. The authors note that considerable judgment was involved in evaluating 27 of the sites, since data were lacking and had to be estimated. It is likely that those

researchers involved in liquefaction studies will continue to update and refine these and future data as more earthquakes occur and field cases become evident.

4.5.2 Laboratory Investigations

Since the smaller scale cyclic triaxial and simple shear tests presently used to evaluate liquefaction characteristics, have a number of limitations which are known but cannot be readily resolved, because of boundary condition restraints, the testing of larger size specimens appears very desirable. Here boundary effects can be minimized and other variables studied more easily (for example, the pore pressure distribution in a large mass of saturated sand during liquefaction may be observed.)

In considering the design of a large apparatus, previous efforts have shown that the variables associated with the inability of controlling three major factors have limited the usefulness of data obtained in shake table tests. These limiting factors include:

- 1) Drainage effects
- 2) Boundary condition effects
- 3) Surcharge effects

The means by which these three factors can best be improved depends upon the method by which cyclic or vibratory loading is applied to the specimen. At this time, there are two alternate approaches that appear most feasible for applying cyclic loads to a large scale specimen in the near future. Although it is not a routine type of test, the most conventional means is with the use of shake table facilities. Also, since the simple shear test more readily duplicates field conditions than any of the other small scale tests, a similar shear loading device and confining box or container designed to accommodate large size specimens may prove desirable. The following paragraphs discuss the merits of each approach.

a. Shake Table Tests

The shake table test, as yet, has produced little useful

quantitative data for studying liquefaction. Although sample preparation is difficult and shake table tests are comparatively costly, lack of consideration of the inability to resolve the three major limitations listed above have been the major reasons for this scarcity of test data. Recent efforts, however, by Finn, Emery and Gupta, 1970, using a specially designed sample container mounted on the shake table have produced a significant advancement toward minimizing the first two of the three major limitations. Drainage was prevented by enclosing the specimen in a membrane while boundary effects were minimized by testing reasonably long and wide test specimens in relation to their height. Further review of the test procedures and apparatus used in these tests lead us to believe that using a slightly modified (more rigid) apparatus, the third limitation may be resolved. This accomplishment would provide a broader range of capability for shake table testing and make possible additional areas for research in soil liquefaction.

An appropriately designed apparatus could be used as an improved test to compare and assess the effect on liquefaction of the various parameters reported for the smaller scale cyclic simple shear and triaxial tests. These parameters include shear stress, pore pressure, relative density, confining pressure, and number of cycles. Comparisons could also be made with field performance to refine correction factors presently applied to the smaller scale cyclic tests. Individual factors such as grain size, shape, and gradation effects could also be studied.

Presently, application of shear stresses in the laboratory to study liquefaction in all tests has only been accomplished by one directional loading. Earthquakes apply many directions of motion to the soil, the dominant motion usually considered to be horizontal. With the recent completion of the new 20-ft. by 20-ft. shake table in Berkeley, vertical motion can now be applied to a test sample simultaneously with one horizontal direction. Using this new table and a suitably designed sample container, studies of the effects of different directions of motion could be conducted. This totally new type of test program would be the next step toward

duplicating field earthquake loading conditions. Those questions which may be answered from such a study might include the following:

- 1) Would the application of one horizontal and one vertical direction of motion (at an appropriate stress level) cause liquefaction to occur in half as many cycles as existing laboratory tests?
- 2) How do the different directions of motion affect the rate of pore pressure build up?
- 3) What effect does shifting the loading phases of two directions of motion have on the rate at which liquefaction occurs?

One major disadvantage of using this approach involves the expense of performing these tests. This, however, is not considered serious for the purposes of this project, since any laboratory device designed to handle and load large specimens will prove costly. In fact, since shaking facilities are available, the expense of fabricating new test apparatus with simple shear loading systems (the most expensive part) makes shake tables more advantageous at this time. The major improvement required for such tests is a suitably designed sample container to overcome the major handicaps previously discussed.

b. Large Scale Cyclic Simple Shear Tests

Because the existing cyclic simple shear test device is so small (2 inches by 2 inches by 1-1/8 inches), it is not possible to fully assess its limitations nor eliminate them. Therefore one possible improvement is to design a larger scale device. Of the previously described considerations the major limitations of the existing equipment include boundary effects such as: 1) end effects, 2) corner effects, and 3) side friction effects. Further, the effect of the intermediate principal stress is not well understood. As presently envisioned, a larger scale device could possibly minimize, but not entirely eliminate these effects as well as provide a more improved test for studying the various parameters which lead to liquefaction than the smaller scale tests.

Also, once developed, tests using this device would probably prove to be less expensive and less cumbersome to perform than shake table tests. (However, the development cost may actually exceed the cost of improvements necessary to shake table sample containers).

The end plates of the simple shear device, because of their rigidity, in effect, create a non-uniform distribution of stress on the contact soil face. This undesirable effect obviously would be very difficult to eliminate entirely in a large test device; however, it could be minimized by testing reasonably long samples in relation to their height.

In the small test device, the corners of the specimen are subjected to much larger localized stresses and, therefore, are likely to liquefy much faster than the rest of the sample. This localized liquefaction in turn, migrates to the center and thus causes the sample to fail prematurely. By testing larger specimens, this effect would automatically be minimized. Also on a large sample, pore pressure measurements made near these corners and at other locations could be compared to assess this effect within reasonable limits.

The final boundary condition limitation, side friction effects, occurs because the sides of the small test device are fixed. This frictional resistance, in effect, creates a non uniformity of shear stresses across the width of the specimen. In the shake table, this effect is minimized by testing specimens with a large width to height ratio. With the large scale simple shear device, this effect could possibly be minimized by providing moveable sides and/or sliding side panels. (This latter provision has, in reality, been found to be a very difficult task).

While this new device would not yet provide a "perfect" test, it would, in our opinion, be a significant improvement over the smaller scale cyclic simple shear test. Once developed such a device, if appropriately designed for broad flexibility of usage could be utilized to: 1) refine the selection of the correction factor as used in the small cyclic load simple shear and triaxial tests, 2) determine the effect of specimen size on the factors

which cause liquefaction, 3) shed more light on the effects of test procedures on pore pressure build-up, 4) study boundary condition effects more closely than presently possible in the smaller tests, and also, 5) fill in several gaps in available data.

While this new device can provide a valuable contribution to the state of knowledge of liquefaction using loading in one direction, the performance of such tests like the shake table, as well as the design and fabrication of the equipment, will prove to be costly.

4.5.3 Suggested Approach for Future Studies

Since reliable field data on liquefaction will only be obtained when strong motion earthquakes occur in well instrumented areas, further concentrated study in this area cannot be widely undertaken until these earthquakes take place. Therefore, further advances in this area in the meantime may need to continue to rely on laboratory methods of investigation.

For such investigations, two large scale devices, the shake table and a large scale simple shear device, have been suggested as possible methods for studying liquefaction. Of these two methods, both can in effect develop valuable information for advancing the state of knowledge of liquefaction. Initial considerations earlier in the subcontract indicated that use of the simple shear device appeared to have certain advantages over the use of shake tables. However, as a result of recent work, reported by Finn, Emery and Gupta, 1970, the major limitations of shake table tests appear to be at least partially resolved. It further appears that by designing a more rigid and wider sample container for the shake table, together with other modifications not specifically identified at this time, all three of the major handicaps presently limiting the acquisition of useful test data, can be kept to a minimum. Because of these recent improvements, it now appears that at this time the shake table offers a better method for studying liquefaction than the large scale cyclic simple shear device for the following reasons:

- 1) Design of Equipment - It is much more difficult to design a test device with moving parts, especially side panels as in the suggested cyclic simple shear device. Efforts completed thus far on Task 7 have found this to be the case. The design for the shake table container is envisioned to have no moving parts and will basically consist of a rigid box container which can be mounted directly to the shaking table. (This will require further study).
- 2) Cost of Fabrication - Since shake tables are available, the method of providing the vibrational forces is already established. The cost for the load applicator for the simple shear device is expected to be of the order of \$80,000. The construction of a device with no moving parts (shake table container) would require less effort than one with numerous moving components (simple shear device).
- 3) Flexibility of Testing - Shake table tests can be performed under one directional loading as well as multi-directional loading. The simple shear device is limited to one directional loading only. When conducting research studies, the more flexible apparatus is more likely of achieving success.
- 4) Experience with Testing - Experience gained from previous shake table tests has been reported and thus can serve as a guide in sample preparation and testing. A new device such as the proposed simple shear test will require at least some initial experimentation to resolve unknown questions which may have already arisen and been answered in shake table studies. For example, Finn, Campanella, and Aoki, 1969, studied the effects of different types of instrumentation to establish the best for their shake table studies.
- 5) Simplicity of Testing - The results can be more

easily interpreted in the shake table tests as the mechanics of the test are less complex and more easily understood.

Based on the initial assessment that the large scale simple shear device is the best path for future studies of liquefaction, the SW-AJA joint venture initiated studies considering the design aspects of this device. The results of these efforts constitute Task 7 of this subcontract, and will be reported subsequently. Also, in the light of recent advancement of the concept of utilizing shake tables for studying liquefaction, some effort in Task 7 will be devoted toward the design concept of the shake table container. The scope of this task is described briefly in Chapter 6. At the present time based on cited improvements in shake table testing, future efforts in large scale laboratory tests for soil liquefaction studies should be directed toward shake table facilities. Should continued studies or research progress demonstrate the need for further development of a large-scale simple shear device, further consideration should be given to this alternate approach.

CHAPTER 5

SUPPLEMENTARY LABORATORY TESTS

5.1 STATEMENT OF PROBLEM

The upward propagation of shear stresses through a loose saturated deposit of sand as a result of earthquake motion in firmer underlying strata imposes stress changes on soil elements that are difficult to reproduce in the laboratory. The two types of small scale tests presently being used are the cyclic triaxial loading test and the cyclic simple shear test. Many of the test procedures and problems associated with each type, such as rotation of principal stresses and stress concentrations are discussed in the State of the Art report (SW-AJA, 1972) submitted also as a part of this subcontract. Because these problems affect the results in different degrees for each type of test, it is not surprising that the results differ. Seed and Peacock, 1970, conclude that the shear stresses in a triaxial test should be multiplied by a factor of about 0.60 in order to compare laboratory data with field performance based on actual earthquake experience.

The purpose of the investigation described herein was to investigate the mechanism of pore pressure buildup in a cyclic triaxial test in anticipation that this might lead to a better interpretation of the validity of such tests.

5.2 VOLUME CHANGE DURING LOADING

When a homogeneous, saturated specimen of sand is loaded hydrostatically, i.e., each of the three principal stresses is simultaneously increased, water is expelled from the interior as the individual grains move into a more tightly packed arrangement. In addition to readjustment of the grain packing, the individual grains change shape as a result of concentrated loading at the points of contact, which results further in an elastic decrease in volume.

In an idealized cubical packing of uniformly sized grains, the contact forces are everywhere normal to the contact surfaces.

In a real sand deposit composed of variable sizes and shapes of grains in a random arrangement, the contact forces are seldom, if ever, normal to the contact surfaces. Although by fundamental laws of static equilibrium the sum of the shear stresses in a given direction along any plane passing through a soil element must be zero, there still exist significant shear stresses at each contact point. As the all-around stress increases, both the normal stresses and the shear stresses at the points of contact increase, but not necessarily in direct proportion. Thus shear stress failures will occur locally at points of contact resulting in a redistribution of stress and a non-recoverable volume change. Further, repetitive loads or stress reversals are likely to cause progressive decreases in total volume for a drained sand even though all three principal stresses are changed simultaneously.

When cyclic deviator stresses are applied to a drained sample, there is likewise a progressive volume decrease after each successive cycle. If, however, the sample is fully saturated and no drainage is permitted, the tendency towards a volume decrease must be offset by an increase in pore pressure causing a decrease in effective stress and an equal tendency towards a volume increase.

By studying the progressive volume decrease during cyclic triaxial loading in drained tests and the corresponding progressive pore pressure increase in undrained tests, it was hoped to better understand the significance of the various factors causing the pore pressure buildup.

Although it is conventionally assumed that under undrained saturated conditions the volume of water expelled is equal to this true volume change of the specimen (both elastic and non-recoverable), it has been well established that additional water may be expelled as a result of volume change in the drainage lines and as the result of poor contact between the end caps and the sample. Since even medium dense sands are relatively incompressible, these errors may become significant unless properly accounted for by calibration or good technique. However, a third and even more

significant source of error is introduced especially on coarser grained soils by penetration of the rubber membrane which surrounds the specimen into the voids between the grains. The magnitude of this effect is probably most effected by the gradation of the sand increasing generally with coarser soils and thus larger spacing between individual grains. One approach in research studies to minimize this error as much as possible is by testing the finer grained sands. The magnitude of the error introduced by this apparent volume change also depends on the thickness and stiffness of the rubber membrane, the pressure difference across the membrane between the chamber fluid and the pore pressure and the dimensions of the specimen.

Roscoe, Schofield, and Thurairajah, 1963, investigated this effect for triaxial specimens of sand, and Ko and Scott, 1967a, did the same for a cubical box having rubber sides. Both found that the volume change induced by penetration of the rubber membrane between the grains was far in excess of that resulting from compression of the sample itself. Since the magnitude of the corrected volume changes to be deducted from the measured total volume change is greater than the resulting true volume change itself, the determination of the true volume change of the specimen is subject to considerable error.

5.3 LABORATORY TEST TECHNIQUES

The laboratory tests described herein were performed on specimens of Ottawa sand approximately 2.5 inches in diameter and 6 inches high. They were placed at a relative density of approximately 50 percent. They were placed dry, then saturated by first circulating de-aired water through the specimen under partial vacuum (a procedure utilizing ammonia gas was normally used to increase the degree of saturation). Finally a back pressure of 86 psi was applied and the pore pressure coefficient checked to verify that complete saturation had been attained.

In drained tests, volume changes were observed in a calibrated burette such that volume changes of as little as 0.001 cc

could be measured with precision. In undrained tests, a pressure transducer was employed to measure pore water pressure within the specimens. The transducer was connected to a Honeywell Herland, Model 712-C recorder. In order to apply vertical loads to the specimens, the piston was connected to the top cap. The loads applied to the specimens were measured by a load cell fixed onto the top of the piston. These loads were actuated by means of compressed air, applied to a double-acting bellofram air cylinder. The piston for the air cylinder is fixed onto the load cell. For drained tests and quasi-static undrained tests, the air pressure was changed manually and recorded from pressure gages. For cyclic load tests, a Modernair solenoid actuated air-loading valve was used, similar to that described by Seed and Lee, 1966.

Vertical strains were measured with a linear variable differential transducer (LVDT). The load cell and LVDT were connected to a Sanborn 321 dual-channel carrier amplifier-recorder.

5.4 SUMMARY OF STATIC TESTS

The purpose of this investigation was to study the influence of various laboratory techniques upon the development of pore-pressure buildup in cyclic load tests. Although numerous tests were performed, they were all performed on nearly identical specimens of one type of sand at one relative density and at one effective initial confining pressure. Detailed test results are therefore not presented and only generalized data are included in the form of summary plots.

Three types of static drained tests and one type of membrane test were performed as described below:

- 1) Uniaxial Load Change - In this type of test the axial load is cycled from the initial effective all-around stress of 14 psi to a higher axial stress and then to an axial stress less than the initial. For each stress change the corresponding volume change and axial strain was recorded. During this test both the chamber pressure and the neutral

stress are maintained constant, thus there is no change in stress gradient across the membrane and the membrane penetration remains unchanged.

- 2) **Biaxial Load Change.** - In this type of test the vertical effective stress is held constant, whereas the lateral effective stress is alternately increased and decreased. The neutral stress is held constant, therefore the stress gradient across the membrane is also changed, with a corresponding change in membrane penetration.
- 3) **Hydrostatic Load Change** - In this type of test all three major principal stresses are alternately increased and decreased while the neutral stress is held constant. In the vertical direction the rigid cap and base preclude any penetration whereas in the two lateral directions the membrane penetration between the grains does vary.
- 4) **Membrane Penetration Tests.** - Using a technique modified from that originated by Roscoe, Schofield and Thurairajah, 1963, the volume change resulting from membrane penetration was determined by replacing the bulk of the sand with an aluminum cylinder, leaving only a thin annular layer of sand in contact with the membrane.

Fig. 5.1 typified the results of a number of tests made to determine both the volume changes resulting from membrane penetration and from compression of the sand itself. Note that for a stress increase of 10 psi, the following changes developed:

<u>Condition</u>	<u>Volume Change - cc</u>
σ_1 increase	0.045
$\sigma_1 = \sigma_2$ increase	0.090
$\sigma_1 = \sigma_2 = \sigma_3$ increase	0.135
Membrane only	0.350

Thus the volume change resulting from membrane penetration, for Ottawa sand at a relative density of 50 percent, was more than seven times greater than the volume change resulting from an equal increase in axial load.

Fig. 5.2 shows a typical result from cyclically increasing and decreasing (at rates of about 1 cycle per minute) the axial load. In this case no membrane correction is necessary. Note the typical hysteresis effects as well as the net increase in total volume change at the end of each cycle.

5.5 INFLUENCE OF MEMBRANE PENETRATION OF PORE PRESSURE

The volume change resulting from membrane penetration exerts two significant influences upon the pore pressure changes that develop during a "constant-volume" cyclic load test. The first of these effects is illustrated schematically in Fig. 5.3. Figs. 5.3(a) and (b) represent simplified relationships assuming no membrane penetration. If the axial load is increased without permitting drainage, the vertical effective stress is increased tending to produce a volume decrease Δ_v (Fig. 5.3(a)). However, no volume change is permitted, therefore the pore pressure increases which decreases the lateral effective stresses (both minor and intermediate stresses) which produces a volume increase, Δ_h . The inclination of the volume increase line (to the left of the vertical line) is twice as great as the volume decrease line (to the right of the vertical axis). In order for Δ_v to equal Δ_h which means no overall volume change, the pore pressure increase must equal 1/3 of the applied deviator stress. Similar relationships during axial load decrease are shown in Fig. 5.3(b).

The effect of membrane penetration is to increase the inclination of the lateral stress-volume change relationship, such that the vertical effective stress changes must be significantly greater in order to offset the volume changes caused by membrane penetration. These relationships are shown in Fig. 5.3(c) and (d).

Fig. 5.3(e) compares the ranges of effective stress changes in both σ_v and σ_h during a single stress reversal in a triaxial

FIG. 5.1. VOLUME CHANGE CAUSED BY MEMBRANE PENETRATION

FIG. 5.2. VOLUME CHANGE WITH REPETITIVE CYCLES OF AXIAL LOADING

(a) AXIAL LOADING WITHOUT MEMBRANE EFFECT

(b) AXIAL UNLOADING WITHOUT MEMBRANE EFFECT

(c) AXIAL LOADING WITH MEMBRANE

(d) AXIAL UNLOADING WITH MEMBRANE

(e) RANGE OF EFFECTIVE STRESSES

FIG. 5.3. INFLUENCE OF MEMBRANE PENETRATION ON EFFECTIVE STRESSES

test, both with and without membrane penetration. Note that for the particular assumption made, the following ranges apply.

<u>Condition</u>	<u>Effective Stress Change</u>	<u>Percent of Deviator Stress</u>
	σ_H'	σ_V'
Without Membrane	± 33	± 67
With Membrane	± 13	± 87

If the sand were perfectly elastic and non-hysteretic, the pore pressure and the effective stresses would return to the initial state at the end of each cycle and there would be no residual effect of the membrane penetration. In reality, however there are two significant effects as described below.

First, the effective principal stress ratio during the unloading cycle with membrane is significantly higher than without membrane. This means that a failure state is approached more quickly and with a fewer number of cycles. Second, the range of effective stress change is greater (in the vertical direction) for the case with membrane penetration than without. Since the gradual readjustment of the grains which causes an increase in density (and therefore a pore pressure buildup at constant volume) is probably related to the range of effective stress change, the increase in this range caused by the membrane penetration is likely to result in an increased residual pore pressure at the end of each cycle. No attempt was made to quantify either of these two effects, however, laboratory tests did verify qualitatively the relationship presented herein at slow rates of cyclic loading.

At faster rates of loading, corresponding with earthquake loading rates, the relationship presented herein is complicated by permeability effects. As the pore pressure changes in response to axial loading, and the chamber pressure remains constant, small quantities of water flow to and from the interior of the sample to the interface between the sand and the membrane. This produces non-uniform stress conditions in the sample depending upon the volume change involved and the permeability of the sand. These

effects have not been analyzed.

5.6 APPLICATION TO SIMPLE SHEAR TESTS

Most simple shear tests conducted to date have involved apparatus in which the specimen is encased laterally with a rubber membrane, although the rubber membrane is further encased with a rigid steel plate. Whether or not the membrane penetration effects are as critical for this condition as for the unsupported membrane in the triaxial test has not been investigated.

In the simple shear test, the major principal stress is increased at the same time as the minor principal stress is decreased, and the intermediate principal stress is unchanged. For this condition, the stress changes are as shown in Fig. 5.4(a). The range of effective stress changes in the simple shear test as compared with those in the triaxial test is shown in Fig. 5.4(b).

5.7 DISCUSSION OF RESULTS

The tests performed as part of this investigation and as reported in this section, clearly indicate that the membrane penetration in a cyclic triaxial load test has a significant influence on the pore pressure buildup and thus on the number of cycles of stress reversal required to develop failure. Although some of the effects are self compensating, it is believed that the resultant effect of membrane penetration is to require too many cycles or higher stresses to develop failure as compared with an in situ element of soil.

The extent to which membrane penetration influences the results of simple shear tests was not established but it is believed to be not as significant as for the triaxial tests.

Seed and Peacock, 1970, recommended that the applied shear stresses in cyclic triaxial tests be multiplied by a factor of about 0.60 in order to make them agree with results of field observations, and also to make them agree more closely with results of simple shear tests. How much of this correction factor is the result of membrane penetration and how much is the result of other

(a) SIMPLE SHEAR WITHOUT MEMBRANE EFFECT

(b) RANGE OF EFFECTIVE STRESSES

FIG. 5.4. EFFECTIVE STRESSES IN SIMPLE SHEAR TEST

effects has not been established. However, it must be pointed out that the magnitude of the membrane penetration effect is a direct function of the area of the membrane to the volume of the sample, and therefore diminishes with increasing diameter of the specimen. Since most tests to date have been performed on relatively small specimens (1.4 to 2.0 inches in diameter), the correction factor of 0.60 should not be applied to the results from larger specimens without further verification.

5.8 CONCLUSION

The discussion contained herein supported by triaxial test results suggests that the membrane surrounding soil elements tested in cyclic shear or compression loading in the laboratory has some influence on the rate of pore pressure buildup and subsequent liquefaction. Its penetration between grains into the boundaries of an undrained specimen of soil in effect creates different effective stresses during loading and unloading had it not been present or when compared with the cyclic simple shear apparatus. The exact influence of this effect in relation to the other deficiencies of the test was not studied in this minimal effort.

Instead of attempting to determine the difference between the various tests empirically or through performing a large number of tests, as has been done by others (Seed and Peacock, 1970; and Finn et al, 1969), the problem was approached by studying the mechanism of pore pressure buildup and volume changes under various cyclic stress applications. Although the results obtained from this study do not answer all the questions pertaining to the problem of explaining liquefaction, they do provide interesting pieces to the overall puzzle. Also the approach taken in this study is somewhat different than that previously taken and could serve as the introduction to a possible avenue toward the development of a theoretical explanation of the phenomenon leading to liquefaction under cyclic stresses. If all of the factors affecting liquefaction in the laboratory could be separated and assessed individually in terms of magnitude, it may become possible to relate similar stress

conditions and pore pressure changes to the field case. Further studies and testing with soil elements could possibly be conducted in future investigations at research oriented universities to evaluate if this overall approach will prove fruitful to this end. The conclusions drawn out in Chapter 4 on Laboratory Shake Table Feasibility Studies also essentially could parallel this idea or general approach.

CHAPTER 6

SECOND YEAR WORK EFFORT

This chapter discusses briefly the technical approach followed and presents a proposed schedule for accomplishing the second year of subcontract work. The proposed schedule contained in Table 6.1 does not include the full scope of work described in our "Revised Program Definition" report because of limitations on funding.

Under this reduced scope five tasks, as given in Supplemental Agreement No. 2 to Subcontract No. 3354, are described in the following sections. In addition to expanding the state-of-the-art report and finalizing the first interim report, four new tasks are to be undertaken. Major work effort is to be directed toward the development or improvement of methods to measure in situ the shear modulus of soils at a higher level of strain than presently utilized in dynamic soil tests in the field. Also included is the further development of concepts for testing large scale specimens under cyclic loading to study liquefaction in the laboratory; modeling and analysis of silty and gravelly soils; and validation of earthquake response prediction techniques.

6.1 TASK 5 - EXPAND STATE OF ART REPORT AND FINALIZE FIRST INTERIM REPORT

The first objective of Task 5 (shown in Table 6.1 as Part 1) is the expansion of the State-of-the-Art report, submitted in draft form in June, 1971, to develop and include interpretations, conclusions, and recommendations indicated by the report. This expanded report, "Soil Behavior Under Earthquake Loading Conditions - State of the Art, Evaluation of Soil Characteristics for Seismic Response Analyses," will be printed in final form in January, 1972 (SW-AJA, 1972).

The second objective of Task 5 (Part 2) is to finalize the report of those accomplishments of the first year of subcontract

TABLE 6.1

SUMMARY WORK SCHEDULE FOR SECOND YEAR

WORK TASK	1971												1972	
	F	M	A	M	J	J	A	S	O	N	D		J	F
5. PART 1 EXPAND STATE OF ART REPORT	■	■	■	■	■	■								
5. PART 2 FINILIZE FIRST INTERIM REPORT	■	■	■	■	■	■	■	■	■					
6. IN SITU TEST TECHNIQUES PRELIMINARY FABRICATION & TESTING					■	■	■	■	■	■	■	■	■	
7. SHAKE TABLE STUDY DETAILED DESIGN CONCEPTS						■	■	■	■	■	■			
8. MODELING & ANALYSIS OF SILTY & GRAVELLY SOILS						■	■	■	■	■	■			
9. VALIDATION OF EARTHQUAKE RESPONSE PREDICTION TECHNIQUES					■	■	■	■	■	■	■	■	■	
DRAFT REPORTS					▲				▲	▲			▲	

LEGEND

- MAJOR WORK EFFORT
PART-TIME OR MINOR WORK EFFORT
DRAFT REPORT

work, not reported elsewhere. A summary of the work areas included in this "Interim Report No. 1" is contained in Chapter 1, herein.

6.2 TASK 6 - IN SITU DEVICE, PRELIMINARY FABRICATION AND TESTING

The major effort during this second year will be directed toward the development of a borehole vibratory test to determine more accurately and at higher levels of strain than presently used in dynamic field test procedures, the in situ shear modulus of a soil deposit. If successful, such a device or test procedure is envisioned to be useful either in actual site explorations or as a research tool for verifying the validity of extrapolation procedures presently utilized between laboratory and field determined values. The concepts presently considered are described in Chapter 3.

In this work task, both cross-hole and down-hole techniques have been reviewed to establish which approach appears the most promising for continued development. Because it is easier to propagate high energy shear waves laterally rather than vertically, the cross-hole test procedure appears to be the more suitable for continued studies and initial field testing.

During this second year of subcontract work, we plan to conduct the limited field testing program described in Chapter 3 utilizing cross-hole techniques, a vibratory energy source and velocity transducers. This testing will be conducted only on one locally available soil (probably glacial till or hard clays in Seattle). This field program is planned to help resolve experimentally, equipment and test constraints such as: 1) energy and wave motion requirements, velocity and strain measurement requirements, 2) anchorage and borehole coupling requirements and 3) detector spacing requirements. The vibratory energy device planned for use in this program will be large enough (3000 to 5000 pounds of force output) to only establish the above requirements and to check the applicability of the proposed procedure. In the interest of scheduling and economy, we presently plan to subcontract the fabrication of this device. Commercial vibrators of the type being

considered (low frequency), are usually available only on a purchase basis and cannot be rented or leased as originally intended. Further, those devices, which could be purchased, have at least a four month period between order and delivery and are generally much more sophisticated and costly than considered necessary for this study. Other equipment such as couplings, anchor systems, loading or support frames, housing assemblies for transducers, etc. are also planned for fabrication by subcontract during this test program, if such are not commercially available.

Following this initial field program and an assessment of the test data obtained, the energy source and detector spacing required to produce local strains approaching earthquake level will be determined, and the equipment best suited can then be defined. Subject to funding limitations, initial acquisition, fabrication and/or modification of such equipment may be started; however, the actual development and completion of this equipment must be accomplished in subsequent subcontract efforts. Also, actual testing of different soil types at different sites and making comparisons of test data with other field and laboratory values would be accomplished in future investigations. The funding requirements for both the acquisition and fabrication of the test equipment and subsequent testing will depend upon performance requirements, and whether it is more practical to lease, fabricate or purchase the equipment outright. Also, the actual location of the sites selected will significantly effect the funds required.

6.3 TASK 7 - STUDY AND DEVELOPMENT OF SHAKE TABLE AND OTHER CONCEPTS FOR SOILS TESTING

General conclusions reached in Chapter 4 indicate that shake tables have not yet made significant contributions, quantitatively, to the study of soil behavior. However, the testing of large-size specimens to minimize the effects of boundary conditions for soil liquefaction studies is desirable. Although sample preparation is difficult, and shake table tests are comparatively costly, the major limitations in shake table tests are the control

of saturation conditions and the application of surcharge loads or confining pressures. Recent advances indicate that with additional development and modifications to the sample container, these handicaps may be overcome. Methods to accomplish this will be investigated further, and recommendations for modifying existing shake tables will be made.

Should modifications to overcome existing deficiencies in shake table tests prove to be impractical, use of a large scale cyclic simple shear apparatus may be an improved method for further study of liquefaction in the laboratory. In this task, a detailed conceptual study of the proposed large scale simple shear apparatus will be conducted. Problems of equipment development including size and shape limitations, instrumentation, and the usefulness of data resulting from such tests will be studied. Design concepts for fabrication and testing in future years of subcontract work will be presented.

6.4 TASK 8 - MODELING AND ANALYSIS OF SILTY AND GRAVELLY SOILS

The work undertaken in this task will parallel the computational studies performed for sands and clays, except that only one-dimensional problems will be treated. Preliminary results of studies of gravelly soils (State of the Art Report, Section 5.6) indicate no change in model format will be required. Hence, the gravelly soil modeling effort will be limited to the definition of model parameter values. Since silts have generally been treated either as sands or clays and little or no data are available for cases in which the sand-silt or silt-clay differentiation has been made, no models will be developed for silty soils. Instead, the previous practice of treating silts as either sands or clays will be followed.

The program of 1-D calculations will therefore consist of several cases based on the State Building Site and the 1957 San Francisco Earthquake. In two or three problems, one or more layers will be converted from sands to gravelly soils with a corresponding large increase in modulus. Surface response spectra will be

compared with spectra from a baseline calculation already completed. In a second subset of calculations some or all of the layers will be converted from sand to clays. Surface response spectra will be compared with baseline spectra from previous problems.

The objectives will be to demonstrate the influence of sand-gravel differentiation and sand-clay (an exaggeration of sand-silt or silty-clay) differentiation at stress levels below soil failure.

6.5 TASK 9 - VALIDATION OF EARTHQUAKE RESPONSE PREDICTION TECHNIQUES

Personnel at the earthquake engineering and seismology centers at NOAA/Coast & Geodetic Survey, and U. C. Berkeley, will be contacted. The suitability of soil data and ground motion records available from the multi-level accelerograph stations at Union Bay, Seattle, Washington; San Francisco Bay, California, and two or more locations in Japan will be reviewed. Station locations will be examined to determine if other multi level stations have been installed and the possibility of substituting outcrop-soil surface station pairs for multilevel stations will be considered. Peak accelerations recorded and reported for the multilevel stations identified have been of low amplitude. If strong motion records for a multilevel accelerograph station in a deposit composed primarily of sands, clays, and gravelly soils are not available, use of the Union Bay data will be considered as an alternate, although less satisfactory, means of testing the adequacy of the soil response calculation procedures.

6.6 REPORTS

Information resulting from the four new tasks to be accomplished in the second year of subcontract work will be submitted in draft reports as noted in Table 6.1. The first report, to be submitted in December 1971, will summarize the results of Tasks 7 and 8. The reports at the end of the second year will contain the results of Tasks 6 and 9. All reports will be submitted first in draft copies for review and comment by Union Carbide Corporation.

These reports will be revised and submitted in final form following receipt of comments on the draft report. In addition to submission of the above reports, monthly progress letters will be prepared to summarize status, and changes that appear desirable.

REFERENCES

- AISIKS, E. G., and TARSHANSKY, I. W. (1968)
"Soil Studies for Seismic Design of San Francisco Transbay Tube," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 138-166.
- ARANGO, I. (1971)
"Seismic Stability of Slopes in Saturated Clay," Ph. D. thesis, Univ. of Calif., Berkeley.
- BALLARD, R. F., Jr., and LEACH, R. E. (1969)
"Development of Vibropacker System for Inducing Polarized Shear Waves and Compression Waves at Depths," Misc. Paper S-69-30, U. S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., July.
- BUSTAMANTE, J. I. (1965)
"Dynamic Behavior of Non-Cohesive Embankment Models" Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 3, pp. IV: 144-160.
- CLOUGH, R. W. (1965)
"The Finite Element Method in Structural Mechanics," Chap. 7, Stress Analysis, O. C. Zienkiewicz & G. W. Holister, Editors, John Wiley and Sons, Ltd., London.
- CLOUGH, R. W., and CHOPRA, A. K. (1966)
"Earthquake Stress Analysis in Earth Dams," Jour. of Eng. Mech. Div., ASCE, Vol. 92, No. EM2, Apr., pp. 197-212.
- CLOUGH, R. W., and PIRTZ, D. (1958)
"Earthquake Resistance of Rock-Fill Dams," Trans., ASCE, Vol. 123, pp. 792-810.
- D'APPOLONIA, D. J., and D'APPOLONIA, E. (1967)
"Determination of the Maximum Density of Cohesionless Soils," Proc., 3d Asian Reg. Conf. on Soil Mech. and Found. Eng., Haifa, Vol. 1, pp. 266-268.
- DEZFULIAN, H., and SEED, H. B. (1970)
"Seismic Response of Soil Deposits Underlain by Sloping Rock Boundaries," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM6, Nov., pp. 1893-1916.
- DIBAJ, M., and PENZIEN, J. (1969a)
"Nonlinear Seismic Response of Earth Structures," Report No. EERC 69-2, Univ. of Calif., Earthquake Eng. Research Center, Berkeley, Calif., Jan.

- DIBAJ, M., and PENZIEN, J. (1969b)
 "Response of Earth Dams to Traveling Seismic Waves,"
Jour. of Soil Mech. and Foundations Div., ASCE, Vol. 95, No. SM2, Mar., pp.541-560.
- DIMAGGIO, F. L., and SANDLER, I. (1970)
 "Material Models for Soils," DASA-2521, Report to Defense Atomic Support Agency, Apr.
- DOBRY, R., WHITMAN, R. V., and ROESSET, J. M. (1971)
 "Soil Properties and The One-Dimensional Theory of Earthquake Amplification," Report R71-18, Mass. Inst. of Technol., Dept. of Civil Eng., May.
- DRNEVICH, V. P., HALL, J. R., Jr., and RICHART, F. E., Jr. (1966)
 "Large Amplitude Vibration Effects on the Shear Modulus of Sand," Contract Report No. 3-161, U. S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Oct.
- DUKE, C. M. (1969)
 "Techniques for Field Measurement of Shear Wave Velocity in Soils," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A5, pp. 39-54.
- FELIPPA, C. A. (1966)
 "Refined Finite Element Analysis of Linear and Nonlinear Two-Dimensional Structures," Report No. SESM66-22, Univ. of Calif., Structural Eng. Lab., Berkeley, Calif., Oct.
- FINN, W. D. L., CAMPANELLA, R. G., and AOKI, Y. (1969)
 "Seismic Testing of Soil Models," Soil Mech. Research Report Series No. 8, Univ. of British Columbia, Vancouver, Canada.
- FINN, W. D. L., EMERY, J. J., and GUPTA, Y. P. (1970)
 "A Shaking Table Study of the Liquefaction of Saturated Sands During Earthquakes," Soil Mech. Research Report Series No. 17, Univ. of British Columbia, Vancouver, Canada, Aug.
- FINN, W. D. L., PICKERING, D. J., and BRANSBY, P. L. (1969)
 "Sand Liquefaction in Triaxial and Simple Shear Tests," Soil Mech. Research Report Series No. 11, Univ. of British Columbia, Vancouver, Canada, Aug.
- FREEMAN, J. R. (1932)
Earthquake Damage and Earthquake Insurance, McGraw-Hill Book Company, Chap. 18.
- GOODMAN, R. E., and APPUHN, R. A. (1966)
 "Model Experiments on the Earthquake Response of Soil-Filled Basins," Bull., Seis. Soc. of Am., Vol. 77, No. 11, Nov., pp. 1315-1326.

- HARDIN, B. O., and DRNEVICH, V. P. (1970a)
 "Shear Modulus and Damping in Soils; 1, Measurement and Parameter Effects," Tech. Report UKY 26-70-CE-2, Soil Mech. Series No. 1, Univ. of Ky., College of Eng., July.
- HARDIN, B. O., and DRNEVICH, V. P. (1970b)
 "Shear Modulus and Damping in Soils; 2, Design Equations Curves," Tech. Report 27-70-CE-3, Soil Mech. Series No. 2, Univ. of Ky., College of Eng., July.
- HUDSON, D. E., and HOUSNER, G. W. (1958)
 "An Analysis of Strong-Motion Accelerometer Data from the San Francisco Earthquake of March 22, 1957," Bull., Seismol. Soc. of Am., Vol. 48, No. 3, July, pp. 253-268.
- IDRISS, I. M., and SEED, H. B. (1968a)
 "An Analysis of Ground Motions During the 1957 San Francisco Earthquake," Bull., Seismol. Soc. of Am., Vol. 58, No. 6, Dec., pp. 2013-2032.
- IDRISS, I. M., and SEED, H. B. (1968b)
 "Seismic Response of Horizontal Soil Layers," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM4, July, pp. 1003-1031.
- IDRISS, I. M., and SEED, H. B. (1970)
 "Seismic Response of Soil Layers," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM2, March, pp. 631-638.
- KANAI, K. (1950)
 "The Effect of Solid Viscosity of Surface Layer on the Earthquake Movements," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 28, pp. 31-35.
- KANAI, K., et al. (1966)
 "Comparative Studies of Earthquake Motions on the Ground and Underground, II," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 44, pp. 609-643.
- KEIGHTLEY, W. O. (1966)
 "Vibrational Characteristics of an Earth Dam," Bull., Seismol. Soc. of Am., Vol. 56, No. 6, Dec., pp. 1207-1226.
- KO, H. Y., and SCOTT, R. F. (1967a)
 "Deformation of Sand in Hydrostatic Compression," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM3, May pp. 137-156.
- KOVACS, W. D. (1968)
 "An Experimental Study of the Response of Clay Embankments to Base Excitation," Ph. D. thesis, Univ. of Calif., Berkeley.

- KOVACS, W. D., SEED, H. B., and CHAN, C. K. (1971)
"Dynamic Moduli and Damping Ratios for a Soft Clay," Jour.
of the Soil Mech. and Found. Div., ASCE, Vol. 97, No. SM1,
 pp. 59-75.
- LYSMER, J., SEED, H. B., and SCHNABEL, P. B. (1970)
"Influence of Base Rock Characteristics on Ground Response,"
Report No. EERC 70-7, Univ. of Calif., Earthquake Eng.
Research Center, Berkeley, Calif., Nov.
- MARTIN, G. R., and SEED, H. B. (1966)
"An Investigation of the Dynamic Response Characteristics
of Bon Tempe Dam, California," Report No. TE-66-2 to State
of Calif. Dept. of Water Resources, Univ. of Calif., Soil
Mech. and Bituminous Materials Research Laboratory,
 Berkeley.
- NUNNALLY, S. W., (1966)
"Development of a Liquefaction Index for Cohesionless Soils,"
 Ph. D. thesis, Northwestern Univ.
- PENZIEN, J., et al. (1967)
"Feasibility Study Large-Scale Earthquake Simulator Facility,"
Report No. EERC-67-1, Univ. of Calif., Earthquake Eng.
Research Center, Berkeley, Calif., Sept.
- RICHART, F. E., Jr., HALL, J. R. Jr., and LYSMER, J. (1962)
"A Study of the Propagation and Dissipation of 'Elastic'
Wave Energy in Granular Soils," Research Report, Civil Eng.
Dept., Univ. of Fla., Sept.
- ROGERS, F. J. (1906)
"Experiments with a Shaking Machine," Report, Earthquake
Inv. Comm. on California Earthquake, Vol. 1, pp. 326,
 (reprinted in Seismol. Soc. of Am., Vol. 30, 1930, pp. 147-
 159).
- ROESSET, J. M., and WHITMAN, R. V. (1969)
"Theoretical Background for Amplification Studies,"
Report No. 5, Effect of Local Soil Conditions Upon Earthquake
Damage, Research Report No. R69-15, Soils Pub. No. 231, Mass.
Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., Mar.
- ROSCOE, K. H., SCHOFIELD, A. N., and THURAIRAJAH, A. (1963)
"An Evaluation of Test Data for Selecting a Yield Criterion
for Soils," Special Technical Pub. No. 361, Laboratory
Shear Testing of Soils, ASTM, pp. 111-133.
- SEED, H. B. (1968)
 Unpublished test results.

- SEED, H. B., and CLOUGH, R. W. (1963)
 "Earthquake Resistance of Sloping Core Dams," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 89, No. SM1, Feb. pp. 209-242.
- SEED, H. B., and GOODMAN, R. E. (1964)
 "Earthquake Stability of Slopes of Cohesionless Soils," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 90, No. SM6, Pt. 1, Nov., pp. 43-73.
- SEED, H. B., and IDRIS, I.M. (1967)
 "Analysis of Soil Liquefaction: Niigata Earthquake," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM3, May, pp. 83-108.
- SEED, H. B., and IDRIS, I. M. (1970a)
 "Analysis of Ground Motions at Union Bay, Seattle, During Earthquakes and Distant Nuclear Blasts," Bull., Seismol. Soc. of Am., Vol. 60, No. 1, Feb., pp. 125-136.
- SEED, H. B., and IDRIS, I. M. (1970b)
 "A Simplified Procedure for Evaluating Soil Liquefaction Potential," Report No. EERC 70-9, Univ. of Calif., Earthquake Eng. Research Center, Berkeley, Calif., Nov.
- SEED, H. B., and IDRIS, I. M. (1970c)
 "Soil Moduli and Damping Factors for Dynamic Response Analyses," Report No. EERC 70-10, Univ. of Calif., Earthquake Eng. Research Center, Berkeley, Calif., Dec.
- SEED, H. B., IDRIS, I.M., and KIEFER, F. W. (1969)
 "Characteristics of Rock Motions During Earthquakes," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM5, Sept., pp. 1199-1218.
- SEED, H. B. and LEE, K. L. (1966)
 "Liquefaction of Saturated Sands During Cyclic Loading," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 92, No. SM6, Nov., pp. 105-134.
- SEED, H. B., and PEACOCK, W. H. (1970)
 "Applicability of Laboratory Test Procedures for Measuring Soil Liquefaction Characteristics Under Cyclic Loading." Report No. EERC 70-8, Univ. of Calif., Earthquake Eng. Research Center, Berkeley, Calif., Nov.
- SHIMA, E. (1962)
 "Modifications of Seismic Waves in Superficial Layers as Verified by Comparative Observations on and Beneath the Surface," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 40, pp. 187-259.

- SILVER, M. L., and SEED, H. B. (1969)
"The Behavior of Sands Under Seismic Loading Conditions,"
Report No. EERC 69-16, Univ. of Calif., Earthquake Eng.
Research Center, Berkeley, Calif., Dec.
- STEPHEN, R. M., et al. (1969)
"Structural Dynamic Testing Facilities at the University of
Calif., Berkeley," Report No. EERC 69-8, Univ. of Calif.,
Earthquake Eng. Research Center, Berkeley, Calif., Aug.
- SW-AJA (1972)
"Soil Behavior Under Earthquake Loading Conditions:
State of the Art, Evaluation of Soil Characteristics for
Seismic Response Analyses," A Joint Venture of Shannon &
Wilson, Inc., and Agbabian-Jacobsen Associates, Jan.
(In Press).
- TSAI, N. C. (1969)
"Influence of Local Geology on Earthquake Ground Motion,"
Ph. D. thesis, Calif. Inst. of Technol.
- TSAI, N. C., and HOUSNER, G. W. (1970)
"Calculation of Surface Motions of a Layered Half-Space,"
Bull., Seismol. Soc. of Am., Vol. 60, No. 5, Oct.,
pp. 1625-1651.
- WEISSMANN, G. F., and HART, R. R. (1961)
"The Damping Capacity of Some Granular Soils," Special
Technical Pub. No. 305, Symposium on Soil Dynamics,
ASTM, pp. 15-19.
- WHITMAN, R. V. (1968)
"Effect of Local Soil Conditions Upon Seismic Threat to
Nuclear Power Plants," Stone & Webster Eng. Corporation,
Sept.
- WHITMAN, R. V. (1970c)
"Summary of Results from Shaking Table Tests at University
of Chile Using a Medium Sand," Progress Report No. 9,
Effect of Local Soil Conditions Upon Earthquake Damage,
Research Report R70-25, Soils Publ. No. 258, Mass. Inst.
of Technol., Dept. of Civil Eng., Cambridge, Mass., May.
- WHITMAN, R. V., and DEPABLO, P.O. (1969)
"Densification of Sand by Vertical Vibrations," Proc.,
4th World Conf. on Earthquake Eng., Chile, Vol. 3, No.
A-5, pp. 29-38.
- YOSHIMI, Y. (1967)
"An Experimental Study of Liquefaction of Saturated Sands,"
Soil and Foundation (Japan), Vol. 7, No. 2, Mar., pp. 20-32.