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SOIL BEHAVIOR UNDER EARTHQUAKE LOADING CONDITIONS

STATE OF THE ART EVALUATION OF SOIL CHARACTERISTICS FOR SEISMIC RESPONSE ANALYSES

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

January 1972

SW-AJA

A Joint Venture of

SHANNON & WILSON, INC. AND AGBABIAN-JACOBSEN ASSOCIATES

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FOREWORD

Recognizing the complexity of the problem of predicting the response of soils to earthquake motion, the United States Atomic Energy Commission (USAEC), through the technical monitorship of Union Carbide Corporation, Nuclear Division, initiated a study in 1970 with Shannon & Wilson, Inc. and Agbabian-Jacobsen Associates (SW-AJA), a joint venture, to study and develop practical and reliable methods for obtaining the necessary data and the analytical procedures needed to predict the response of soils to earthquake loading. While this study is concerned with free-field soil behavior, and therefore does not include structure-soil interaction considerations, it is part of a larger program directed toward improving the methods of site evaluation for nuclear power plants. Primary emphasis in this study is placed upon in situ methods for determination of soil properties. Where in situ methods are found unsatisfactory, or inapplicable, other procedures including improved sampling techniques coupled with improvements in field or laboratory testing are to be developed.

As an initial step in this study, the joint venture has prepared this state-of-the-art report. This is the third report prepared 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.

This report summarizes and assesses the present state of knowledge and capabilities available to the engineering profession for evaluation of soil characteristics for seismic response analyses. In addition to the basic text, references, tables and figures, the appendices at the end of this report include pertinent symbols and definitions, and a complete annotated bibliography applicable to work under this subcontract.

This report was prepared by the joint venture of Shannon & Wilson, Inc. (SW) and Agbabian-Jacobsen Associates (AJA). Dr. R. P. Miller of SW was Project Manager, assisted by Mr. Fred Brown, who was Principal Investigator for SW, and Dr. I. Arango. Dr. J. W. Workman and Dr. G. A. Young were the Project Engineers for AJA, assisted by Dr. Y. C. Eric Lee, Principal Investigator. Mr. S. D. Wilson and Dr. M. Agbabian have reviewed this report in detail and have contributed significantly to its contents. Dr. H. Bolton Seed, Professor of Civil Engineering, University of California, was special consultant to the joint venture on this project. Dr. Seed and Dr. I. M. Idriss, Assistant Research Engineer, prepared Chapter 5 of this report as principal investigators for the University of California under subcontract to the joint venture.

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CHAPTER 1

INTRODUCTION

The use of nuclear reactors in the generation of commercial electrical power continues to expand. Currently, about 100 nuclear power plants are in operation, under construction, or are in the planning stage in the United States. It is estimated that by the early 1980's the total power generating capacity will have doubled with most of the new capacity being powered by nuclear energy.

Because of their fission product inventory, nuclear power plants have always been subject to special safety requirements. To meet these requirements and to ensure the continuous generation of power should a natural disaster occur, great care is exercised in the design of nuclear power plants. As an additional safeguard, the critical subsystems which control the nuclear reactor are required to satisfy even more stringent design criteria.

Earthquake ground motions are of critical importance primarily because of the magnitudes of the vibratory motion characteristics and the resulting dynamic forces induced in critical structures and equipment of nuclear power plants. Earthquake excitations can also produce foundation and slope instability, subsidence, and less probable hazards such as surface displacements due to faulting, but all of which can seriously damage structures, dams, and piping which supplies the emergency cooling systems. The design of the facilities must give special consideration to these problems, and must be based upon reasonable and reliable estimates of the earthquake effects.

Prediction of expected earthquake motion for a proposed nuclear power plant site, and the response and behavior of the overlying soils and structures to the predicted motions, is a complex problem that requires the combined services of experienced geologists, seismologists, and soils and structural engineers. The areas of greatest uncertainty pertain to the geological and seismological studies, and to the prediction of soil behavior.

Information needed by the seismologist to relate earthquake magnitude and strength to the site and regional characteristics is often quite limited. Predictions of earthquake magnitude, therefore, frequently become somewhat subjective with a natural tendency toward conservatism. Additional study is required in this area but is not within the purview of this report.

Prediction of the response of soils to earthquake motion is limited by present knowledge of the behavior of soils under dynamic loading, which is restricted for two reasons. First, knowledge of techniques for testing soils to establish reliable dynamic characteristics is not generally available; and second, results of reliable dynamic tests, although widely distributed in the literature, have not been summarized in any useful form. This report is concerned with this area of technical deficiency and has been prepared to present reliable dynamic testing techniques, to bring together in one volume the results of significant tests that have been performed to date, and to identify areas in which further work will lead to new or improved methods for determining soil properties.

The necessity for holding this report to a reasonable volume has made it impossible to include thorough discussions of the work of many investigators who are making significant contributions. Many of these constitute a part of the state-of-the-art in a very real sense. The work of these investigators has, therefore, been referenced throughout the text, and these reference documents listed on pages 222-237 contain material which should be considered as an extension of this report for those who wish to pursue given subject areas in greater detail than is possible in this one volume. As a further extension of this report and its coverage of a very broad subject, Appendix E is an annotated bibliography of literature relating to soil properties and behavior.

To some extent, the material presented herein has been selected and organized to maintain continuity. This applies particularly to material in Chapters 2 and 5. The intent is to provide the reader with a complete evaluation procedure or process,

rather than to present a collection of individual evaluation methods with no indication of how they might lead to an overall conclusion.

The report which follows provides first, in Chapter 2, a brief introductory consideration of the problems of establishing the design earthquake for nuclear power plant facilities, and of defining the earthquake input motion for soils analyses as general background knowledge. This is followed by a brief review of analytical procedures presently used to predict the response of soils to earthquake motion. The primary objective of the chapter is to identify the soil parameters required for the various mathematical models used in prediction studies.

In the investigation and evaluation of nuclear power plant sites, it is necessary to recognize potential failure conditions as well as to predict the site response motions to earthquake loading conditions. This requires an accurate determination of the subsurface conditions, which includes establishing the presence of groundwater, providing soil classifications, determining the lateral and vertical distribution of different soil types, and establishing the engineering properties of each. Field explorations and laboratory and field tests are required to provide this information. A comprehensive review of site investigation procedures is provided in Chapter 3, which is titled, "Field and Laboratory Determination of General Site and Soil Conditions."

Dynamic soil properties are established by laboratory and field tests which have developed progressively over a period of several years. Many of the tests have been developed in conjunction with the study of foundation vibrations induced by vibrating equipment. Each dynamic test procedure must usually operate within a limited strain range. Thus, no one test is adequate for fully defining the dynamic properties required for earthquake response analyses. Chapter 4, "Determination of Dynamic Soil Properties," summarizes current laboratory and field methods used to determine these properties and discusses the limitations and adequacy of each test procedure. As a follow-on to this

discussion, Chapter 5, "Dynamic Properties of Soils," summarizes the available data determined by these tests, in a form which provides a useful guide to the selection of soil characteristics for analytical studies.

Foundations and slopes frequently experience instabilities under severe earthquake motion that can lead to catastrophic failures of structures, earth embankments, and underground utilities. Foundations for nuclear power plant facilities must be selected for which there is absolute certainty that critical instabilities will not result. Liquefaction of saturated sands represents one of the most serious types of soil instability that can result under cyclic loading. A very thorough review of the phenomena of liquefaction is presented in Chapter 6, "Liquefaction of Soils," with a comparison of laboratory test results with field performance. The subject of soil subsidence is treated in Chapter 7.

The state-of-the-art discussed in Chapters 2 through 7 is summarized in Chapter 8. Areas of uncertainty are identified, and study areas which are considered to offer the greatest promise for improving our state of knowledge, providing more reliable means for determining in situ conditions, and for evaluating the uncertainties, are described.

CHAPTER 2

RESPONSE PREDICTION PROCEDURES

Procedures presently followed in the design of nuclear power plant facilities require consideration of both an operating basis and design basis (safe shut-down) earthquake. The structure, piping, and equipment required for continuous operation of the plant are designed to remain functional for the operating basis earthquake, while those elements necessary to assure shutdown of the reactor and maintain it in a safe condition are required to remain functional for the latter. This requires the foundation to remain stable for both conditions. Studies necessary to define the magnitudes of the two design earthquakes and to predict the response of the overlying soils, structures, piping, and equipment to the resulting vibratory motions are all interrelated. The reliability of the piping and equipment analysis, for example, is obviously greatly affected by the thoroughness with which the geological, seismological, and soils investigations are conducted, even though this may not always be recognized. For this reason, a general discussion of the overall ground motion prediction problem is included in this chapter. A brief description of the mathematical models and analytical procedures used to determine the influence of soil deposits on local ground surface shaking is also included to indicate the type of soil data required. For those readers interested in the details of specific models or an evaluation of alternate approaches, attention is directed to the references cited below, to the annotated bibliography in Appendix E, and to a companion report prepared under this subcontract (SW-AJA, 1971).

2.1 DEFINITION OF DESIGN EARTHQUAKES

The study required to define the design earthquakes can be divided into three steps. First, a careful geological and seismological study of the region, and a detailed soils investigation of the nuclear power plant site is required to provide basic

information. Second, from the geological and seismological studies, the magnitudes and strengths of the design earthquakes are established. In this step, the earthquakes are defined in the terms of the geologist and seismologist, which does not always provide quantitative information that can be used directly in engineering computation. Thus, the third step is to define the characteristics of the earthquake motions in a quantitative manner. A brief discussion of these three steps follows.

2.1.1 Geological, Seismological, and Soils Investigations

Very thorough and complete geological and seismological investigations are required for the prediction of design earthquakes. These investigations are not without precedent; they are quite similar in extent and thoroughness to those required for large dams and reservoir projects. In the case of nuclear power plants, the studies must consider a large region. The objective of the geological investigation is to establish the lithology, stratigraphy, structure, and history of the general region. Tectonic structures underlying the region must be identified, as well as any physical evidence of activity during prior earthquakes. The seismological investigation is closely related to the geological investigation and requires a listing of all earthquakes of record which may have affected the general area of the site. The magnitude of the earthquakes, epicenter locations, dynamic characteristics, and durations of the resulting ground motion should be determined, or estimated. Epicenters within about 200 miles of the proposed site are of particular significance. Geological fault structures within this approximate radius, and greater than one mile in length, which are capable of causing surface faulting, must be studied if it appears that these structures could cause severe earthquake motions at the site. The detailed requirements of these studies are covered in the USAEC tentative regulatory criteria on Seismic and Geologic Siting Criteria for nuclear power plants (U.S. Atomic Energy Commission, 1971).

A thorough investigation of the subsurface soil conditions is required to provide information for design and construction as well as to provide information for evaluating the response of the soils to earthquake motion. Among the potential construction problems that must be considered in the soils investigation are slope stability, dewatering, swelling, availability of backfill materials and compaction requirements. The soils investigation must consider such structure design problems as the method of support, foundation arrangement and allowable bearing pressure to control settlement, and foundation vibrations when structures support vibrating equipment. In order to evaluate the response of the soils to seismic motion, special investigations are required to establish the susceptibility of the site to liquefaction, subsidence and slope instability, and to establish the dynamic properties of the soils for use in analytical models. Overall, a thorough soils investigation is required which includes borings, geophysical explorations, other field tests, and laboratory tests. The details of these investigations, as well as methods of establishing dynamic soil properties, are the subject of this report and are presented in Chapters 3 through 5, and Appendix A. Special problems pertaining to liquefaction and soil subsidence are presented in Chapters 6 and 7, and Appendix B.

2.1.2 Prediction of Earthquake Magnitude and Strength

Experienced seismologists are required to predict the magnitudes and strengths of earthquakes which should be anticipated for a proposed nuclear power plant site. Factors which must be considered are the length of geologic fault structures, their relationship to regional tectonic structures, and the geologic history of displacement along the structures. Unfortunately, information available to the seismologist to relate earthquake magnitude and strength to site and regional characteristics is quite limited.

Seismologists normally define a design earthquake for a proposed nuclear power plant site by predicting an earthquake intensity (I), or magnitude (M), and the strength of the motion.

The strength of the motion is normally designed by a peak horizontal ground acceleration, expressed usually in percent of acceleration of gravity (g). Although peak acceleration does not provide a good quantitative definition of the earthquake motion in an engineering sense, it is a very important parameter because of prior acceptance and wide use. The Modified Mercalli Intensity rating (I) is based upon observed damage from the earthquake and, as pointed out by Housner, 1965, is purely subjective. In more recent years, strong motions of earthquakes have been recorded, and the earthquake magnitude (M) has been expressed by the Richter scale which is an indication of the energy released by the earthquake. This is a useful rating system, since some of the characteristics of earthquakes can be related in a general way to the earthquake magnitude. When the intensity scale has been used to classify past earthquakes, the influence of the soil profile on observed damage must be considered when attempting to convert the rating to the more rational magnitude scale.

Predictions of earthquake magnitude and strength of motion require considerable judgment and experience, as well as the use of past correlations of pertinent characteristics of historical earthquakes. Some confusion and misunderstanding frequently result from the fact that some correlations are based upon peak values and provide envelope curves, while others are curves of averages. The former are more applicable to deterministic design procedures, while the latter apply to probabilistic approaches which are not yet accepted for the design of nuclear power plant facilities. Also, in defining strength of motion, it would be preferable if earthquake motions were all defined at the bedrock level, which may or may not be at the ground surface. Most earthquake strong motions have been recorded at the ground surface and variations in soil profiles at the different sites are an uncontrollable variable in most correlation studies. Examples of important correlations of earthquake magnitudes and strengths which demonstrate these variations follow to demonstrate the need for overall consistency in the response prediction procedure.

It is normally held that earthquake magnitude can be related to length of fault for shallow depth earthquakes. This is an important relationship since it assists the seismologist in assigning an earthquake magnitude to a potentially active fault zone in the proximity of a proposed nuclear power plant site. Reference is frequently made to the work of Bonilla, 1970, who provided the correlation indicated by Line A in Fig. 2.1. However, the same data have also been used by Seed, Idriss and Kiefer, 1969, to provide the average curve indicated by Line B. It is obviously important whether an average or limiting value is being specified. It is also important to note that in some regions, correlations of these types are of little value since many of the important geologic features may be deeply buried by weathered materials. In such cases, recommendations must be based largely upon judgment and subjective reasoning.

Several correlations are available to relate earthquake magnitude, distance from causative fault, and peak acceleration. An early correlation by Housner, 1965, is provided in Fig. 2.2. These curves are generally assumed to represent peak acceleration for seismic motions experienced at the ground surface. It is significant that at that time the influence of overlying soils on the peak accelerations could not be evaluated. Peak accelerations represented by these curves are not, therefore, directly applicable to the bedrock level.

The results of a study by Seed, Idriss and Kiefer, 1969, have been published in which an attempt was made to evaluate the influence of the overlying soil profile on peak acceleration, and thereby obtain bedrock peak accelerations. This study considered the work of Gutenberg and Richter, Benioff, Kanai, Housner, Esteva and Rosenblueth, and Blume. The curves given in Fig. 2.3 resulted from this study and relate peak bedrock acceleration with earthquake magnitude and distance from causative fault. These curves provide peak values which are lower than those provided by Housner (1965) for the ground surface. For example, at a distance of 25 miles from a causative fault, a peak bedrock acceleration of

(From Bonilla, 1970)

Fig. 2.1 RELATIONSHIP BETWEEN EARTHQUAKE MAGNITUDE AND LENGTH OF SURFACE RUPTURE

(From Housner, 1965)

Fig. 2.2 RELATIONSHIP BETWEEN PEAK GROUND ACCELERATION, EARTHQUAKE MAGNITUDE AND FAULT DISTANCE

(From Seed, Idriss and Kiefer, 1969)

Fig. 2. 3 RELATIONSHIP BETWEEN PEAK BEDROCK ACCELERATION, EARTHQUAKE MAGNITUDE AND FAULT DISTANCE

0.10g is indicated for an earthquake of magnitude 7 by Fig. 2.3. A comparable value for the ground surface from the curves in Fig. 2.2 is 0.27g. The difference can be attributed to two factors. First, as has been indicated above, the curves in Fig. 2.2 are for the ground surface and the influence of the soil profile is an uncontrolled variable, while the curves in Fig. 2.3 are for the bedrock level. Second, and of equal importance, the curves in Fig. 2.2 are indicated to be maximum peak values while the curves in Fig. 2.3 are an average of several correlation studies. The basis upon which the correlations are prepared should, therefore, be clearly understood by the user. In addition, as time passes and more strong motion earthquake data become available, guideline curves such as those in Figs. 2.2 and 2.3 are subject to change.

It should be evident that prediction of the earthquake magnitude and strength is not an exact science, but requires a detailed soils profile and a dynamic analysis as well as the exercise of good judgment and careful consideration of all geological and seismological information available. It is not, in the final step, a simple matter of selecting a point on published curves such as those in Figs. 2.1, 2.2, and 2.3. The depth of the fault below the ground surface, the type of faulting, the soil and rock conditions at the proposed site, and all information that may give insight into the seismicity of the region must be considered. The correlations and published works previously cited do provide guidance and help in the prediction procedure, but in the final analysis, the decision must be based upon mature and qualified judgment.

2.1.3 Characteristics of Earthquake Motions

After the design earthquake magnitudes and strengths have been defined by the geological and seismological studies, characteristics of the earthquake motions must be defined quantitatively for use in engineering studies. Time-motion records are preferred as input to analyses which predict the response of overlying soils, and for dynamic analyses of complex structures and equipment.

Simpler problems may be analyzed with a response spectrum. A very brief consideration of procedures for defining time-motion records and response spectra for input to analyses follows.

a. Time-Motion Records

Earthquake records of value to the engineer must be recorded close to the epicentral region of the earthquake by instruments of special design. The basic record is a time record of ground acceleration of which two orthogonal horizontal components and one vertical component are recorded. These basic records are then frequently integrated to provide time records of velocity and displacement. Fig. 2.4 provides an example of the acceleration record and of the integrated velocity and displacement records for one component of the El Centro earthquake of 1940. Dynamic analyses can be based on either of the three records, but acceleration or velocity are most frequently used. Since the basic record is the acceleration record, it is normally used in criteria to define the design earthquake motion with the peak value being used to indicate the strength of motion.

Important characteristics of the acceleration record are considered to be its duration, predominant period, and peak acceleration, although the root mean square (rms) of the record is considered by some to be more descriptive of the record than the peak acceleration (see Jennings, Housner and Tsai, 1969; Werner, 1970). The present practice is to scale these characteristics from an existing record to provide the design earthquake motion, keeping in mind that the magnitude of the earthquake and the peak acceleration are normally provided by the geological and seismological study. The general problem of scaling earthquake records is treated by Seed, Idriss and Kiefer, 1969. It can be briefly summarized as follows:

- 1) Since the peak accelerations of existing records are seldom of the desired strength, the first step normally is to scale the accelerations linearly to provide the design peak accelerations.

FIG. 2.4 GROUND ACCELERATION, VELOCITY AND DISPLACEMENT,
EL CENTRO, CALIFORNIA, EARTHQUAKE OF MAY 18, 1940,
N-S COMPONENT

- 2) The duration of strong motion has been related in a general way to fault length and the time required for the fault to shear, which can also be related to earthquake magnitude. Fig. 2.5 provides a relationship between duration of strong motion and earthquake magnitude developed by Housner, 1965. This is an upper bound curve and considerable deviation can be expected when the curve ordinates are compared to actual records or the average duration of several earthquakes. Segments of strong motion of an existing record can be repeated, or deleted, in order to adjust the duration to the proper time length for the design earthquake.
- 3) Fig. 2.6 provides a relationship between predominant periods of maximum acceleration, earthquake magnitude, and distance from the causative fault. It is important to note that a constant period is indicated for a given earthquake magnitude out to a distance of 25 miles from the causative fault. The work of Figueroa, 1960, indicates a very wide scatter in data out to a distance of 50 miles, which frequently encompasses the distance of interest. Only moderate scaling of predominant periods is generally justified. This is done on the individual record in linear fashion.

Scaling an individual earthquake record has the disadvantage that each record is representative of a specific event and is not typical of all earthquakes in general. As is evident in Fig. 2.7 each individual record will invariably be deficient in some response frequencies which is not desirable for a design input record. There are two alternatives. One is to utilize an ensemble of scaled earthquake records; the other is to utilize scaled artificial earthquake records that do not exhibit a specific

(From Housner, 1965)

Fig. 2.5 DURATION OF STRONG SHAKING VS EARTHQUAKE MAGNITUDE

(From Seed, Idriss & Kiefer, 1969)

Fig. 2.6 PREDOMINANT PERIODS FOR MAXIMUM ACCELERATION IN ROCK

Fig. 2. 7a ACCELERATION RESPONSE SPECTRUM

Fig. 2. 7b VELOCITY RESPONSE SPECTRUM

Fig. 2.7 RESPONSE SPECTRUM FOR EL CENTRO (1940) EARTHQUAKE

incident bias. Procedures for selecting an ensemble of real or artificial records are provided in Werner, 1970.

b. Response Spectra

Response spectra can be used as input in dynamic analyses of linear elastic systems. Response spectra provide also an effective means of comparing the characteristics of different earthquake records. A response spectrum is a plot of the maximum peak responses of a family of idealized single-degree-of-freedom damped or undamped oscillators to a specified vibratory ground motion. The simple structure in Fig. 2.9 can be characterized as a single-degree-of-freedom oscillator. The response may be expressed as acceleration, velocity, or displacement, and peak values of all three can be displayed on a tripartite plot. Individual acceleration and velocity spectra with 5 percent critical damping are provided in Fig. 2.7. A tripartite plot for the same earthquake is provided in Fig. 2.8.

A simple illustration of the use of a response spectrum is provided in Fig. 2.9. Here a simple structure has been idealized and represented by a single mass (m) supported on the ends of elastic steel columns having a stiffness K . It can be shown by mechanics that the fundamental period of the single mass system is equal to $2\pi\sqrt{m/K}$. If the response spectrum given in Fig. 2.8 is assumed to be applicable to this particular structure, and if the period is computed and found to be 0.2 sec for example, then the maximum response acceleration, velocity, and displacement can be found from Fig. 2.8 to be 1.0g, 12 ips, and 0.5 in., respectively. The maximum stress in the columns can then be computed and the adequacy of the design verified. Displacement, velocity, acceleration, or stress may control the design in different problems.

Response spectra for design can be computed from a time-motion earthquake record, or procedures can be used which do not require the prior development of a time-motion record. The two most common procedures in use are those developed by Housner, 1959, and by Newmark and Hall, 1969. Housner has provided the velocity and acceleration spectra shown in Fig. 2.10. These curves represent

Fig. 2.8 RESPONSE SPECTRUM FOR 1940 EL CENTRO EARTHQUAKE

Fig. 2.9 SINGLE - DEGREE - OF - FREEDOM STRUCTURE

Fig. 2.10a AVERAGE VELOCITY SPECTRUM CURVES

Fig. 2.10b AVERAGE ACCELERATION SPECTRUM CURVES

(From Housner, 1959)

Fig. 2.10 AVERAGE EARTHQUAKE RESPONSE SPECTRA

the averages of the response spectra for two components of ground motion of four strong motion earthquakes experienced in California. These curves can be scaled linearly by multiplying the ordinates of the curves by the ratio of the maximum predicted ground acceleration to the acceleration (4 ft/sec^2) noted in Fig. 2.10 for a period of zero seconds. Typical response spectra for operating basis and design basis or safe shutdown earthquakes developed by this procedure are shown in Fig. 2.11. It is important to recognize that Housner's procedure provides average response spectra. Newmark and Hall suggest a procedure which is based upon a standard earthquake that is taken to have a maximum ground acceleration, velocity and displacement 50 percent greater than recorded for the 1940 El Centro earthquake. These values provide the broken line bounds shown in Fig. 2.12. These standard values are then scaled to give what approaches an upper bound, or envelope spectrum. The details of scaling are provided in the reference cited. Qualified judgment and experience, and adequate field investigations are required for both procedures. Site conditions may require that certain portions of the design spectra be modified from that indicated by either scaling procedure.

2.2 PREDICTION OF SOIL RESPONSE

Earthquake vibratory motion is produced by a stress wave which emanates from a region of the earth's crust where a sudden rupture, or fault movement, has occurred. The stress wave is transmitted more rapidly and with less energy loss through bedrock than through overlying soils. Thus, the path of the strong motion is normally from the causative fault through the bedrock to the nuclear power plant site, and when overlying soils are present, through the soils to the structure and the internal piping and equipment. The response of the overlying soil to the bedrock motion can be as follows:

- 1) The overlying soils may prove stable, and the vibratory motion will then be transmitted with some modification but with no apparent damage to the foundation.

Fig 2.11a OPERATING BASIS EARTHQUAKE

Fig 2.11b SAFE SHUTDOWN EARTHQUAKE

Fig 2.11 RESPONSE SPECTRA FOR DESIGN

(From Newmark and Hall, 1969)

Fig 2.12 TYPICAL DESIGN RESPONSE SPECTRUM

- 2) Severe foundation damage may result from large scale compaction and subsidence or from a loss in shear strength due to liquefaction of saturated soils.
- 3) Slopes may become unstable resulting in slides of natural slopes and in man-made embankments, such as in earth dams.

Mathematical modeling procedures have been developed quite recently that are of value in explaining the observed response of soils to earthquake motions. These procedures can be used to advantage to predict the response of soils at proposed nuclear power plant sites. The procedures, however, require the determination of the dynamic properties for the soils. The following review of analytical procedures currently in use demonstrates the soil response studies frequently required and the dynamic soil properties which must be determined for the analyses. The latter is the major subject of this report.

2.2.1 Response of Stable Soils to Earthquake Motion

It has been recognized and documented in the technical literature since the San Francisco earthquake of 1906, that a deep deposit of overlying soils can significantly modify the earthquake ground motion at the surface from that experienced at nearby sites located on bedrock or more firm materials. Structures supported upon deep deposits of soils are frequently severely damaged. More important, the damage on some occasions has been almost selective in that buildings of a certain type (natural frequency) have been damaged while others of a different type on the same foundation materials have not.

Most recently, Rosenblueth, 1960; Whitman, 1969c; and Seed and Idriss, 1969, have provided excellent examples of the important influence of local soils on structural damage. Rosenblueth, for example, has noted that in the earthquake of Mexico City in 1957, tall structures having low frequencies were more severely damaged than relatively rigid, higher frequency, one- and

two-story buildings. This is explained by the fact that the soil conditions amplified the response accelerations for the low frequencies. Seed and Idriss, 1969, have demonstrated the influence of the soil profile on response spectra in Fig. 2.13. This figure provides response spectra recorded for the same earthquake in the same city for four sites having different soil profiles, which become progressively softer from Site A to E. It is noted that for the progressively softer sites, peak acceleration response occurred at increasingly longer periods. Structures at each site having the unique frequency of the peak response would have experienced the greatest response or inertia forces. It has also been demonstrated that soils overlying bedrock do not always amplify the bedrock acceleration but may, in some cases, actually attenuate them. In order to ensure proper design of structures and power plant equipment, it is obviously important to be able to predict the response characteristics of nuclear power plant sites which have deep soil profiles.

Three possible approaches to the development of prediction procedures for the response of soils have been noted by Seed and Idriss, 1969. One is to approach the problem empirically and attempt to accumulate strong motion records for a large number of sites having a wide range of soil conditions for earthquakes of different magnitudes and different epicentral distances. This, it is pointed out, would be an impossible task. The second procedure would be to use microtremor data recorded at the site to evaluate the effect of local soil conditions. This alone is not an acceptable procedure since the soil behaves nonlinearly and its modulus and damping characteristics are strain dependent, as shown in Fig. 2.14. In this figure the broken lines can be taken as equivalent moduli and the hatched areas are proportional to the hysteretic damping at different strain levels. It is possible for both the equivalent shear modulus and the damping to vary by an order of magnitude at different strain levels. It is, therefore, extremely difficult to predict the response of a soil deposit to strong earthquake motions from microtremor data alone.

Site A

Site B

Site C

Site E

(From Seed and Idriss, 1969)

Fig 2.13 EFFECT OF SOIL PROFILE ON PERIOD OF PEAK RESPONSE

(From Seed and Idriss, 1969)

Fig 2.14 EFFECT OF LEVEL OF STRAIN ON SHEAR MODULUS AND DAMPING

The third procedure, which is recommended, is to use analytical procedures based upon soil test data and a mathematical model of the soil profile which is subjected to the bedrock motion. This procedure can use microtremor data, and strong motion earthquake records if available for the given site, to verify the reliability of the mathematical model. This procedure is possible only because advances have been made in mathematical modeling capability quite recently. The results of the analyses, however, are dependent upon the reliability of the dynamic property values defined for the soils, as has been previously indicated. Improved procedures for determining dynamic soil properties, and for incorporating the properties into mathematical models are certain to lead to still further advances in analytical capability in the near future. However, present procedures are quite effective when carefully used and can be extremely useful in predicting the response of soils at proposed nuclear power plant sites. Discussion of mathematical models currently used for analysis follows:

a. One-Dimensional Models

It has been previously noted that earthquake motion is produced by a stress wave which is transmitted more rapidly and with less energy loss through the bedrock than through the overlying soils. When the bedrock has a horizontal surface of great extent, and the overlying soil layers are also horizontal, it is frequently assumed that the earthquake motion within the soil is produced essentially by shear waves which propagate upward through the soil from the bedrock surface. This assumption greatly simplifies the analysis since the problem can be reduced to a one-dimensional shear wave problem, for which the differential equations of motion can be written and a closed-form solution obtained for ideal (linear elastic) soil properties. This is an assumption of convenience since recorded earthquake motions at the ground surface indicate clearly that vertical components of motion of significant magnitude are also present, and that the wave transmission problem is normally more complex than can be explained by one-dimensional wave theory. However, numerical

analyses have provided computed horizontal motions for one-dimensional mathematical models of actual soil profiles that are in substantial agreement with those recorded (see Seed and Idriss, 1969; and Kanai, 1961). This has provided justification for the use of the relatively simple one-dimensional mathematical analysis for sites that have simple soil and rock geometry.

Several investigators have utilized closed-form solutions to the linear, viscoelastic, one-dimensional wave propagation problem to establish bounds for soils response (see Kanai, 1961, and Zeevaert, 1963). These solutions require that the soil be defined by its unit weight, shear modulus, and a viscous damping coefficient. However, real soils are layered and these properties may vary widely between layers. Also, the stress-strain characteristics are highly nonlinear and hysteretic, and are strain dependent, as indicated previously in Fig. 2.14. In order to introduce variations in properties in different soil layers, and more realistic stress-strain characteristics, numerical techniques have been developed (Matthiesen et al., 1964; Herrera and Rosenblueth, 1965; and Roesset and Whitman, 1969) and modified soil models examined (see, e.g. Dobry, 1970). Hagmann and Whitman, 1969, and Idriss and Seed, 1968b, have provided interesting comparisons of one-dimensional wave propagation solutions with one-dimensional discrete mass model solutions which demonstrate the validity and stability of the discrete mass model approach.

An important application of one-dimensional wave theory has been provided by Lysmer, Seed and Schnabel, 1970, in which the basic wave equation was used to study the influence of the overlying soil profile on the bedrock motion, and to study the importance of the relative rigidity of the bedrock on the soil profile motion. Two important conclusions from this study which are used to simplify dynamic analyses are these: First, the rock motions at the base of the soil deposits several hundred feet deep are not significantly different than those occurring in outcrops of the same rock formation, and second, ground surface motions are not greatly affected

by neglecting the deformability of the bedrock and considering it to be rigid. These conclusions are based upon computations for soil deposits which are underlain by a deep bed of rock having the same characteristics with depth. Based on this study, it is often assumed in one-dimensional analyses of soil profiles for earthquake motion that the bedrock is rigid and that radiant damping effects can be neglected.

One of the most versatile one-dimensional discrete mass analytical techniques currently available to the profession for analyzing the response of horizontal soil profiles to earthquake motion, is that described by Seed and Idriss, 1969, 1970a, and Idriss and Seed, 1968b, 1970. In this model, the soil profile is idealized by a series of discrete masses and springs with linear viscous dampers, as shown in Fig. 2.15. The nonlinear and hysteretic stress-strain characteristics of the soil are introduced by using an equivalent shear modulus and an equivalent viscous damping factor, both of which can vary with depth of the soil layer and the strain level within the layer. The equivalent shear modulus for a given strain level is taken as the slope of the diagonal line (average slope) drawn through the hysteresis loop, which is shown in Fig. 2.15. The average equivalent viscous damping coefficient is proportional to the area of the hysteretic loop and is computed as shown in the same figure by the procedure developed by Jacobsen, 1930. Curves have been developed from laboratory tests for some, but not all, soils which can be used to evaluate these equivalent parameters as a function of the strain amplitude. The curves provided in Fig. 2.16 are an example for sands. These curves, as well as curves developed for other soils, are discussed in greater detail in Chapters 4 and 5.

Once the soil profile has been idealized, and the soil properties determined, an iterative procedure is used with this model to introduce the proper values for the equivalent shear modulus and damping factor. Thus, the strain distribution in the soil profile is first estimated and the moduli and damping factors selected based upon the assumed strain distribution. The response

Fig 2.15a DISCRETE MASS MODEL FOR LAYERED SYSTEM

Fig 2.15b EQUIVALENT LINEAR SHEAR MODULI AND DAMPING
USED IN DISCRETE MASS MODEL (From Seed and Idriss, 1969)

Fig 2.15 ONE-DIMENSIONAL DISCRETE MASS MODEL
FOR SOIL PROFILE ANALYSIS

(From Seed and Idriss, 1970c)

Fig. 2.16 MODULI AND DAMPING RATIOS FOR SANDS

of the soil profile (discrete masses) to the base motion with the assumed characteristic, is then computed and the average strains developed within the soil profile are determined. These values are then compared with the initial assumptions and new values of moduli and damping selected for a second solution. This iterative procedure is repeated until strains are computed which agree with those assumed for the model. Fig. 2.17 provides an example of an application of the procedure to a site in Anchorage, Alaska, with a comparison of the computed and recorded response spectrum for the ground surface. The earthquake motion input in this case was not a recorded input for the site, but was scaled by procedures similar to those discussed above in Section 2.1. The soil properties which must be defined in order to use this model are its unit weight, the relative density of sands, and the shear strength of clays. This assumes curves, such as are shown in Fig. 2.16 for evaluating the equivalent shear modulus and damping ratio, are available for the soil types encountered. In the event these curves are not available, extensive cyclic simple shear or triaxial compression tests, and forced or free vibration tests of soil samples are needed to define curves for shear modulus and damping. These tests are discussed in detail in Chapter 4.

One-dimensional finite element models also can be used to analyze soil profiles of simple geometry. However, the finite element model has not had wide use. Finite element solutions are more generally used for two-dimensional models and are discussed in the next section. The same general discussion applies to one-dimensional finite element models, except the model is constrained to have one-directional motion.

b. Two-Dimensional Models

When the bedrock and/or soil layers are inclined, a one-dimensional shear wave assumption is no longer valid and a two-dimensional model must be used to introduce the more complex geometry and wave motion. It is important to note that even for a horizontally layered system, a two-dimensional model is required if other

Fig. 2.17 COMPARISON OF MEASURED SOIL RESPONSE AND RESPONSE PREDICTED BY ONE DIMENSIONAL DISCRETE MASS MODEL (Anchorage, Alaska Site)

than a simple shear wave motion is to be considered. The literature is not extensive on the use of two-dimensional models to predict earthquake ground motion. It is restricted almost entirely to applications of finite element models.

The finite element model idealizes the soil continuum as a system of finite elements interconnected at a finite number of nodal points. Either triangular or rectangular elements, depending upon the characteristics of the model, can be used. Fig. 2.18 provides an example of a finite element model used to study the effect of a sloping rock interface on soil response (taken from Seed, Idriss and Dezfulian, 1970). In most cases, the finite element model representation has assumed the soil to be equivalent linear elastic. To obtain a description of the soil response, the stiffness and mass matrices are formulated and a modal solution or a time marching integration is effected, depending upon the capability of each particular computer program. Response time histories of displacement, velocity, and acceleration can be computed for any nodal point. For this model, a shear modulus, Poisson's ratio or a bulk modulus, the unit weight of the soil, and a damping coefficient must be defined. The moduli and unit weight can be varied for different elements to represent soil or rock of different properties. While the model permits a study of more complex geometries and motions, it is a more expensive model to use because of the computer time required.

The model described above approximates the nonlinear properties of soils by using an equivalent linear modulus and equivalent viscous damping. Soil profiles can be modeled with finite element models that incorporate nonlinear elements (Isenberg, 1970), and with nonlinear finite difference models (Trulio, 1966). Use of the latter models, however, has been limited to nuclear weapon or chemical explosion induced environments, much more severe than those associated with earthquakes. They require considerably more computer time than the linear models and are, consequently, considerably more expensive to employ.

Finite Element Representation of Valley Section

(From Seed, Idriss and Dezfulian, 1970)

**Fig 2.18 FINITE ELEMENT MODEL OF VALLEY CROSS SECTION
FOR ASSESSING EFFECT OF SOIL CONDITIONS
ON BUILDING DAMAGE (Caracas Earthquake, July 29, 1967)**

2.2.2 Prediction of Foundation Damage

There are three significant types of damage that may occur to nuclear facility foundations as a consequence of vibratory earthquake motion, assuming faulting is excluded from within the foundation area. These are large-scale subsidence or compaction of loose soils, excessive lateral pressure from slides or rock movement, and complete foundation instability which may result from liquefaction of saturated sands and silts.

Unsaturated loose sands and silts may experience compaction due to earthquake vibratory motion. This, in effect, improves the load bearing capacity of the foundation. However, differential displacements due to varying depths of soil, and variations in relative density can cause rotation of structures and rupture of utility lines. Problems of this type are usually avoided by compacting the loose soil, replacing the loose soil layer with well-compacted soil, or supporting the facilities on piles. Saturated loose sands and silts, however, present an entirely different problem since liquefaction of foundation soils can result in the complete loss of shear strength and the total collapse, or overturning, of structures. This problem is given extensive attention in Chapter 6. At the present time, mathematical models do not exist that can accommodate the radical change in soil properties and the resulting large deformations that result from liquefaction. However, an example of how the one-dimensional discrete mass model, discussed in Part a. of Section 2.2.1, has been coupled with experimental tests and field data to explain the onset of liquefaction during the Niigata earthquake of June 1964, is of interest and a brief discussion of this example is warranted.

The serious consequence of liquefaction is evident from the damage description provided by Seed and Idriss, 1967, for the Niigata earthquake, which is quoted in part as follows:

"...extensive damage to engineering structures occurred as a result of liquefaction of the sandy soil on which they were supported. Many

structures settled more than 3 feet, and the settlement was often accompanied by severe tilting. ...An apartment building...rotated through an angle of 80° ...water was observed coming from the ground in the form of springs which gradually became more extensive until they covered large areas, and automobiles, houses, and other objects gradually sank into the resulting quicksand... In several cases, buried structures floated to the surface... Buildings collapsed 2,130; seriously damaged 6,200; slightly damaged 31,200..."

Following a damage survey of the structures, it was possible to divide the area into three zones based upon structure damage. In the zone of light damage, the underlying sands were denser than those in the other two zones, and the water table was at greater depth below the ground surface. In the two remaining zones the water table was at practically the same depth, but in one zone the structures experienced heavy damage while in the other only moderate damage was experienced. Field tests indicated that in the zone experiencing moderate damage slightly denser sands existed between the depths of 15 ft. and 45 ft. than in the zone of heavy damage. Otherwise, the soil profiles, topography, and depths of water table were the same.

In order to try to establish the reason for the difference in behavior between the zones of moderate damage and heavy damage, a study was conducted which involved the following steps:

- 1) The development of an estimated earthquake time-motion history for the base rock at the site,
- 2) The prediction of the response of the overlying soils to the predicted base rock motion using a discrete mass model with bilinear stress-strain characteristics,
- 3) A determination of the number of significant stress cycles and the equivalent uniform cyclic

shear stress developed at different depths during the earthquake,

- 4) A determination by laboratory tests on representative sand samples of the cyclic shear stress required to cause liquefaction of the sand in the number of significant stress cycles determined in Step 3,
- 5) A comparison of the magnitude of equivalent cyclic stress developed at various depths with the cyclic stress determined experimentally to cause liquefaction at that depth, to determine whether or not liquefaction should have occurred.

The results of the comparison for the two zones are provided in Fig. 2.19 where it will be noted that the stress intensity was sufficient to cause liquefaction for the zone of heavy damage, but not in the zone of light damage. Two important conclusions from the study were, first, a stronger earthquake would have also caused major damage in the lightly damaged zone since soils in this zone were near liquefaction, and second, a shallow depth of fill on new building sites in the heavily damaged zone would probably stabilize the site against future damage by a similar earthquake. Thus, even though mathematical modeling capability does not permit at this time a direct application to liquefaction problems, an indirect solution is possible with a relatively simple model.

2.2.3 Prediction of Slope Instability

Sites for nuclear power plant facilities frequently exhibit natural slopes, as well as man-made slopes which result from filled areas and earth embankments. A review of the literature on analytical studies, conducted to explain the failure of natural slopes and earth embankments due to earthquake motions, will reveal mathematical models of the same type as those discussed above. Several very interesting examples will be found of the use of two-dimensional finite element models combined with field

Fig 2.19a ZONE OF HEAVY STRUCTURAL DAMAGE

Fig 2.19b ZONE OF MODERATE STRUCTURAL DAMAGE

(From Seed and Idriss, 1967)

Fig 2.19 RESULTS OF LIQUEFACTION POTENTIAL ANALYSIS FOR NIIGATA EARTHQUAKE

observations and laboratory results to explain the failure of natural and man-made slopes from earthquake motion. A brief example is provided to demonstrate an application of the two-dimensional finite element model to slope stability problems.

Because of the geometry introduced by the slope, the two-dimensional finite element model is usually the more realistic model to use in the prediction of the response of earth slopes to earthquake motion. Interesting analyses of natural slopes and embankments are provided in the following references: Chopra, 1967; Idriss and Seed, 1967; Idriss, 1968; Seed, 1968b, and Dibaj and Penzien, 1969b. Idriss and Seed, 1967, for example, have provided a finite element analysis for a slope in a clay soil subjected to both horizontal and vertical earthquake motions which is based upon linear viscoelastic assumptions. The results of this study indicated that the shear strength of the clay was not exceeded. The model indicated that the maximum horizontal shear stress was distributed as shown in Fig. 2.20. It will be noted, however, that the shear stress was high beneath the slope on horizontal planes which intersect the slope. In the Alaskan earthquake of 1964, slope failure was attributed to liquefaction in a sand seam underlying the earth bank. Failure, however, was initiated well back from the slope. This was explained by Seed, 1968b, by the fact that liquefaction is more difficult to produce when the ratio of initial shear stress to normal stress is high, all other factors being equal. Thus, a finite element analysis similar to that described above was used to provide the vertical, normal and horizontal shear stress along the plane of potential liquefaction from which the variation in the ratio of these two stresses along the plane was computed. The number of cycles of cyclic shear stress required to initiate failure at different points along the sand seam were then computed. From this, it was possible to show that failure should have been initiated well back from the slope which was the observed behavior.

Finite element models have been used in similar fashion to demonstrate the mode of failure of earth dams induced by

Fig 2.20 TWO-DIMENSIONAL FINITE ELEMENT MODEL ANALYSIS RESULTS SHOWING RESPONSE OF EARTH SLOPE TO EARTHQUAKE MOTION

earthquake motion. The dynamic analysis reported by Seed, Lee and Idriss, 1969, for example, provides a logical explanation for the failure of Sheffield Dam that could not be provided by static analysis alone.

2.2.4 Summary of Mathematical Models Used to Predict the Response of Soils

The above discussion has presented a review of state-of-the-art procedures for predicting the response of soils to earthquake motion. Primary emphasis has been upon the mathematical models used in the response analyses of stable soils to identify the soil parameters which must be defined. Single applications have also been provided, for the principal models described, to an actual case where soil instability resulted. The mathematical models in these cases, however, were used indirectly in the analysis which also included laboratory tests and field tests and observations. The importance of having reliable and economical modeling procedures, and equally reliable soil parameters is evident. A summary of the models discussed above, the advantages and disadvantages of each, and the soil parameters required is provided in Table 2.1 for reference. The remaining chapters in this report describe the basic information needed and the procedures that should be followed in establishing soil parameters for dynamic analysis, as well as a more detailed discussion of the liquefaction phenomenon and soil subsidence.

TABLE 2.1 SUMMARY OF MATHEMATICAL MODELS USED
FOR DYNAMIC SOILS RESPONSE ANALYSIS

Type of Model	Required Soil Properties	Comments
One-Dimensional Continuum Models	Unit weight, shear modulus and damping factor. Shear wave velocity may be used to estimate modulus ($C_s^2 = \frac{G}{\gamma/g}$). Strain-dependence of modulus and damping factor may be accounted for through equivalent linear models. Relative density of sands and shear strength of clays may be used to obtain properties from master curves (see Fig. 2.1b). Viscous damping, because of its mathematical simplicity, is used most frequently.	Closed-form solutions are available for the response of linear soil models with discrete layers or for a class of models with continuously varying properties if excitation is simple. Numerical transform techniques allow the recovery of ground motion histories under more realistic excitation. Iterative procedure may be used with strain-dependent equivalent linear models.
One-Dimensional Discrete Mass Models		Modal superposition solutions used with iterative procedure allow treatment of strain-dependent properties. Discrete elements or layers provide variation of modulus with depth; some limitations on variation of damping unless direct integration solution is adopted.
Two-Dimensional Finite Element Models	Unit weight, shear modulus, bulk modulus or Poisson's ratio, and damping factor (as functions of current strain, previous history and loading condition if nonlinear models are used).	Can accommodate changes in soil properties in two directions (horizontal layering not required), and complex boundary geometry. Treats both shear and dilatation waves. Modal superposition and iteration to account for strain-dependence are possible. Direct integration with nonlinear soil models provides alternative means of accounting for strain-dependence. Simulated one-dimensional models are nearly competitive with other one-dimensional models; two-dimensional models require significantly longer computing times.

CHAPTER 3

FIELD AND LABORATORY DETERMINATION OF GENERAL SITE AND SOIL CONDITIONS

In the investigation and evaluation of potential nuclear power plant sites, it is necessary both to recognize potential foundation failure conditions and to predict the site response under earthquake loading for structure design. For power plants founded in soil, this requires the accurate determination of the subsurface conditions, including groundwater levels, soil classification, the stratification and lateral distribution of the soil deposit, and the engineering properties of each layer. The depth to bedrock, or dense "rock-like" soils, is also a part of this investigation.

Much of the subsurface information and data that are obtained at a site for a nuclear plant are normally required in more or less routine foundation investigations for structures under static load conditions. However, where analyses of behavior under seismic or earthquake load conditions are necessary, usually more comprehensive field explorations are required. Further, these investigations would include the determination of the dynamic soil engineering properties, and also an evaluation of the possibility of foundation failure due to soil liquefaction. While the field exploration procedures are the same, they generally include more borings to better define the extent of the soil materials involved, more complete groundwater observations, and usually geophysical explorations to further define the soil distribution and obtain data for evaluating the dynamic moduli of deformation. Following the field explorations and classification of the materials encountered, laboratory tests are performed on representative samples to determine certain physical or engineering properties affecting static load behavior, site response, and the evaluation of potential site instabilities. In addition

to the dynamic soil properties discussed in subsequent chapters, these properties include unit weight, shear strength and the void ratio or relative density.

The first section in this chapter describes briefly the field procedures available for establishing certain local soil conditions. This section is then followed by a discussion of the particular soil conditions best determined by these field procedures. The condition or parameter is identified and its pertinence to ground response or liquefaction described. Also presented, in cases where applicable, is an evaluation of the adequacy of the data obtained, or the accuracy with which the given parameter must be known. The third and final section of this chapter describes laboratory procedures that are available to determine those soil conditions which cannot be readily assessed by field methods. The methods described must utilize either disturbed or undisturbed specimens of soil recovered from borings with the general sampling techniques described in Appendix A. Because of their importance, the next two chapters, 4 and 5, are devoted to the determination of the dynamic soil properties and their magnitudes. Tests and evaluation of liquefaction potential are described in detail in Chapter 6. Ground subsidence is reviewed in Chapter 7. The state-of-the-art in all of these areas is briefly summarized in Chapter 8.

3.1 FIELD EXPLORATIONS

There are two primary field exploration methods of interest to this study. These methods include 1) drilling and sampling of borings, with some in-hole testing, and 2) geophysical explorations. Both methods can provide subsurface data as a function of depth down to bedrock, or at least to a depth of several hundred feet. Other methods of field exploration, such as test pits, hand augers, portable power drilling equipment, and related near-surface or surface tests or equipment, are also utilized to investigate subsurface conditions; however, they are

not considered applicable for this study, since they are generally limited to depths that rarely exceed 15 feet. The following sections discuss briefly the two primary methods of field exploration and generally outline engineering considerations that must be assessed when attempting to investigate a potential site.

3.1.1 Borings

In the investigation of a potential nuclear power plant site, the drilling of borings and recovery of samples is generally the most practical method of investigating the subsurface conditions. The number and depth of borings to be made at a site, under normal static load considerations, depends upon the characteristics of the structure and the competency and complexity of the soil at that specific building location. Structural characteristics that must be considered include the loads, size, type of construction, the intended use and performance criteria for the structure, and the elevations of the various foundation elements. Since a nuclear power plant usually consists of a number of structures of various sizes and loads, borings are normally required at each specific building location. At least two borings are usually necessary at the smaller and less important facilities, while as many as six or more may be required for the larger, more important structures. Also, several borings extending to bedrock, or dense "rock-like" soils, or to depths of several hundred feet may be necessary at the project site to establish soil conditions for response calculations. These deep borings generally identify, verify the competency, and establish the continuity of the deep soil deposits which are often-times not penetrated by the shallower borings made in the usual static load foundation investigation.

The number and depth of borings as well as their actual locations, taking both soil and structural factors into consideration, should be designated by the foundation soils engineer, since

he is ultimately responsible for establishing both quantitatively and qualitatively: 1) the average soil properties for use in response calculations, 2) potential failure conditions, and 3) the expected behavior of the soils under certain imposed building loads. Sampling intervals normally used for borings are discussed in Section 3.2.

The inspection of drilling and sampling operations is also a very important part of the boring program. This work is usually performed by a competent geologist, soils engineer or technician representing the engineering firm making the soil studies and recommendations. His duties, in addition to specifying sampling intervals and the depth of the bore hole, include: 1) classifying in detail each sample, 2) preparing a boring log along with pertinent field notes, 3) observing sampling techniques to assure that consistent and acceptable procedures are being practiced, and 4) observing soil cutting returns and the action of the drilling equipment, to ensure that the soil penetrated is not significantly different from the samples obtained. The soils engineer, by having experienced personnel from his firm perform this field inspection, is assured that adequate precautions are taken for obtaining the best information and for making correct interpretations from the boring explorations.

To accomplish the borings at a particular site in the most efficient manner, and to obtain the best data from this program requires careful consideration of both the drilling equipment, and the samplers to be used. Also, suitable groundwater measuring devices must be selected and installed in an appropriate manner in order to establish the groundwater conditions at the site. The selection among the different types of equipment, samplers, and groundwater measuring devices should be made, taking into consideration the general characteristics of the soils which will be penetrated. These conditions can generally be assessed sufficiently well by a soils engineer or geologist familiar with the area under consideration.

To assist the engineer who is generally not familiar with presently used drilling and sampling methods, or in-hole test procedures and available groundwater measuring devices, reference is made to Appendix A. In the first section of this appendix is a brief description of drilling equipment. Drilling methods are generally discussed, in consideration of the various soil types. The next two sections discuss the types of soil samplers most widely used and their general applicability to site investigations. The final two sections of Appendix A discuss other in-hole tests or methods which are also available as engineering tools for evaluating the subsurface conditions, groundwater measuring devices and installation procedures.

3.1.2 Geophysical Explorations

For many important structure foundations, geophysical explorations usually consisting of seismic refraction velocity measurements are desirable to determine the depth to bedrock and delineate the general distribution and competency of the overburden strata. These measurements generally supplement information provided by borings which identify the materials indicated by the variations in velocity. The seismic refraction survey procedure consists of placing geophones along the ground surface at a given interval (usually 25, 50 or 100 feet) in a line and, at several selected locations along the same line, firing explosive charges and recording simultaneously with an oscilloscope the arrival time of the resulting wave patterns. The total length of the survey line and the size of charge govern the depth of penetration of the seismic waves; and the spacing of the geophones control the detail with which the overburden strata can be delineated. A short line may be only 500 feet, while a long line may be 2500 feet. With the longer lines, bedrock can be detected with reasonable accuracy down to depths approaching 1,000 feet.

Data obtained in a seismic refraction survey consist principally of the time required for a compression wave to

travel from the explosive source to its point of measurement (each geophone). Data are plotted as distance versus travel time. The slope of a line connecting any two points indicates the average velocity of the compression wave through the subsurface media encountered between those points. A change in the slope of the line indicates that the wave has passed through an interface between two soil layers with different velocities. The depth of this change is calculated using the appropriate mathematical relationships. By repeating the calculation for each geophone and each shot, a two-dimensional profile of average wave velocity versus depth can be constructed. The wave velocity within a given layer or depth interval is an indication of the competency of the material (i.e., the more competent the material, the higher is the velocity of the waves) and thus becomes an important property to measure for site subsurface investigations, at a relatively inexpensive cost. Since average velocities are determined from the investigation and the detail is based on the spacing of the geophones, the delineation between materials of similar competency is often difficult and in some cases relies heavily on interpretation and on correlation with nearby borings. Materials of higher velocity often mask out underlying materials of lower velocity. The method does, however, provide a good overall picture of the uniformity of the soils at a site, greatly extending the detailed information provided at each boring location.

Other more specialized seismic survey methods are used to obtain specific soil properties for dynamic response analysis. These methods are in-hole procedures (conducted inside of a boring) and consisting of the up-hole, down-hole and cross-hole techniques. These techniques are discussed in Section 4.2.1.

The electrical resistivity method is the only other geophysical method of significance to civil engineering projects. The resistivity method is useful for defining the boundaries between soft clays and organic deposits of low resistivity, and materials of higher resistivity such as sands, gravel or bedrock.

Materials having low resistivities can be detected even if they underlie those of high resistivities. Unfortunately, boundaries between soft clay and an organic soil, or between stiff clay and soft clay shale, or between loose sand and coarse-grained sandstone, cannot usually be detected.

The most significant limitation of geophysical exploration methods is that they do not provide a physical description of the underlying soil, and, therefore, borings are advisable with each geophysical program. Attempts to determine soil types based solely on the compression wave velocity or electrical resistance without verification by borings can often be misleading.

Equipment for geophysical methods of exploration is readily available commercially. The average cost of performing such an investigation generally ranges between \$800 and \$1000 per field crew day, which includes field work, reduction and analysis of data, and report preparation. In an average field crew day, refraction survey lines totaling about 1,000 to 3,000 feet can be completed, depending generally upon the roughness of the terrain and the complexity of the subsurface conditions encountered.

3.2 SUBSURFACE CONDITIONS DETERMINED BY FIELD PROCEDURES

For ground response analyses and the evaluation of liquefaction, it is necessary to determine certain local soil conditions which can best be evaluated by field techniques. In addition to field identification and classification of the soils, these conditions or parameters include 1) the distribution and layer thicknesses of the soils at the site, 2) the in situ relative density of granular deposits, 3) the groundwater conditions, and 4) the bedrock depth. Of these conditions, all are more or less pertinent to ground response, while only conditions 2) and 3) are of particular significance to the evaluation of liquefaction. The following sections discuss these four subsurface conditions in detail and, where applicable, provide an assessment of the accuracy with which each must be known or can be determined.

3.2.1 Soil Distribution and Layer Thickness

The layer thicknesses within a deposit can be defined within the desired accuracy for most practical purposes, using existing standard drilling and sampling procedures. To define successive layer thicknesses within a deposit for both response calculations as well as for most normal foundation investigations, sampling in the upper 50 feet of soil is usually accomplished at either 2-1/2 or 5-foot intervals enabling the accuracy of the profile to be defined within about 1 to 3 feet. Below this depth down to about 100 feet, a 5-foot interval is usually desirable; while below 100 feet either 5 or 10 feet is often adequate, depending upon the complexities of the soils encountered.

When considering potential instabilities such as liquefaction or slope failures, thin layers of weak and unstable materials enclosed in a more competent zone must sometimes be identified. This may be accomplished by decreasing the sampling interval to less than that indicated above, possibly even to the extent of sampling continuously. This procedure will prove to be slightly more costly, but is considered both justified and necessary in many cases.

If the materials are interbedded in relatively thin layers, the average properties of these materials must be grouped within an arbitrarily assumed depth interval. The precise interval utilized can be quite variable, and depends upon the complexity of the soil mass as well as the approximate depth of the zone. Below a depth of 100 feet, an accuracy of between 5 and 10 feet is generally adequate; while above this depth, usually an accuracy of 2 to 5 feet may be desirable.

In evaluating a potential nuclear power plant site in the preliminary stages of an investigation, the complexity of the subsurface conditions and their lateral distribution can generally be assessed from only a few borings. The conditions observed from these borings will usually dictate the approximate number of additional borings required for an adequate evaluation

of the average thickness, distribution, and the necessary soil properties of each soil layer within a particular deposit.

Generally, detailed engineering soil property data to a depth of several hundred feet are necessary for response analyses in the evaluation of nuclear power plant sites, though the general nature of the foundation soils must be known to the depth of bed-rock or rock-like materials. In some cases, satisfactorily accurate analyses of response may be made from a general knowledge of the deep soil conditions (Kiefer, Seed and Idriss, 1970), since average properties can often be estimated sufficiently well such that changes in variables cause only a minor variation in the response output. Geophysical explorations, geological considerations and results of existing nearby oil well logs, if available, are often used to estimate the soil types, and layering at great depths.

3.2.2 In Situ Relative Density

The in situ relative density or state of compactness of cohesionless granular soil with respect to the loosest and densest states at which it can be placed by specified laboratory procedures, must be evaluated as a function of depth in granular soil deposits underlying potential nuclear power plant sites. In response analyses this property (or the void ratio), as demonstrated in Chapter 5, has a significant influence on the magnitude of the dynamic shear modulus and therefore indirectly affects the response output. For the evaluation of liquefaction, this property is of particular significance in the upper 50 to 60 feet of a deposit (see Section 6.2), especially one that is saturated or is likely to be saturated during the life of the structure. This property must be assessed in order to duplicate the field conditions with test specimens recompacted in the laboratory for cyclic loading, or to compare with data from other sites where liquefaction under similar conditions has or has not occurred.

Ideally, the best method for determining field relative density with depth is to measure the unit weight of undisturbed

specimens removed from borings and compute the relative density based on the laboratory procedures described in Section 3.3. Unfortunately, because of sample disturbance, very dense sand specimens tend to loosen and loose specimens tend to compact in the sample tube, leading to erroneous values of relative density determined in the above manner (Waterways Experiment Station, 1962). Materials that are medium dense will produce a wide scatter of data, also depending upon the degree of disturbance as well as upon other variables in the deposit itself. Measured values may show a scatter of 30 to 40 percent as illustrated in Fig. 3.1a. This scatter of data is considered typical of that obtained using careful drilling and sampling procedures. Relative densities measured in the laboratory on undisturbed specimens should therefore be used to establish average trends, since any one measurement (no matter how carefully the test is performed), may approach an upper or lower bound value and give misleading results of the average condition at the site.

Because the degree of compactness or relative density of granular materials cannot be evaluated with complete confidence from "undisturbed" samples, various types of penetration resistance tests have become widely used as an indirect method to determine this important property in situ. The distinct advantage of the penetration-type tests lies in their simplicity, allowing a large number of tests to be performed as a part of the boring operation, at a minimum cost. The resistance to penetration is generally measured using either static or dynamic cone penetrometers, or the standard penetration test, which is also a dynamic procedure. The details of each test method, their limitations and the basic equipment used in these tests are described in Appendix A.

The most widely used penetration resistance test in the United States is the standard penetration test (SPT). It is also the subject of considerable discussion within the soil engineering profession, as evidenced by the special session and state-of-the-art paper specifically devoted to this test in the Fourth Pan-American Conference on Soil Mechanics and Foundation Engineering,

a. LABORATORY MEASUREMENTS

b. FIELD STANDARD PENETRATION DATA

c. COMPARISON BETWEEN MEASURED RELATIVE DENSITY AND THAT DETERMINED BASED ON THE BLOW COUNT

d. COMPARISON OF AVERAGE PENETRATION CRITERIA AND ACTUAL MEASURED DATA

Fig. 3.1 RELATIVE DENSITIES FROM FIELD PENETRATION TESTS & LABORATORY MEASUREMENTS

- + Measured directly from Osterberg Samples
- ⊙ Calculated by Gibbs & Holtz method for dry and partially wetted sands
- × Calculated by Bazaraa method
- Standard Penetration Resistance
- ▽ Water Table

1. Data were obtained from Woodward - Clyde - Sherard & Associates, 1968 (a & b)
2. Material was a natural saturated deposit of medium dense, gray, fine to medium SAND

San Juan, Puerto Rico in 1971 (refer to de Mello, and others, 1971). Earlier papers have also dealt with the same subject. When using the SPT as the "standard", resistances to penetration by other equivalent methods may be approximately correlated with the blow count, N , determined from this test. The relationship among the results of different penetration test methods for fine sands is approximately as follows:

$$N \approx Q_c/4 \text{ to } 4.5 \approx 1/2 N_c \dots \dots \dots (3.1)$$

where N = standard penetration test resistance, blows/ft.

Q_c = static cone penetration resistance, Kg/cm^2

N_c = dynamic cone penetration resistance, blows/ft.

For foundation analysis and design, penetration test N -values which are, in reality, indices of the denseness of a granular deposit, must be evaluated in terms of soil properties, primarily the relative density. In order to convert the penetration resistance (N) to relative density, correlations have been developed and presented by various investigators (Gibbs and Holtz, 1957; Schultze and Melzer, 1965; Bazaraa, 1967). Unfortunately, these correlations may give considerably different results, as illustrated by the two sets of curves in Fig. 3.2. Some of the reasons for these differences and criticisms of these correlations are presented by Holtz and Gibbs, 1969; Peck and Bazaraa, 1969; and D'Appolonia and D'Appolonia, 1970.

Based on both literature studies and discussions with engineers in the soils and foundation engineering profession, the Gibbs and Holtz correlations (Gibbs and Holtz, 1957) are widely accepted for average representations of relative density, with due consideration given to their applicability and limitations. Various studies have been made and are continuing by engineers to improve confidence in such correlations and in relative density determinations of sand deposits from standard penetration test

Fig. 3.2 RELATIVE DENSITIES CALCULATED FROM STANDARD PENETRATION RESISTANCES
 (From Holtz & Gibbs, 1969)

data. Typical test data from a foundation investigation for a power plant site were reviewed for this purpose in this present study. These data are reproduced in graphical form in Fig. 3.1, and illustrate the typical scatter that may be expected, even with acceptable care in performing the tests.

In Figs. 3.1a and 3.1b, the laboratory measurements of relative density and the field penetration resistances are summarized from 10 borings made on the shores of the Missouri River for the proposed Cooper Nuclear Station, Brownville, Nebraska. The material penetrated was a 70-foot thick deposit of medium dense, fine to medium sand (Woodward-Clyde-Sherard & Associates, 1968a and b). By converting the penetration resistance of all field tests to relative density, using the Gibbs and Holtz criteria, and superimposing the data on the values measured in the laboratory, the plot in Fig. 3.1c is obtained. While there is considerable scatter in both field and laboratory data, average values of the laboratory data when compared with average values obtained using the Gibbs and Holtz correlation show reasonable agreement.

Further, by using the average penetration resistance curve (Fig. 3.1b) and converting average penetration values into percent relative density by using both Bazaraa and the Gibbs and Holtz curves, a comparison of relative density from these two criteria and the values measured from laboratory tests is obtained as shown in Fig. 3.1d. Of the two procedures, the Gibbs and Holtz criteria are shown to be closer to the measured laboratory values. The Gibbs and Holtz correlations also produce average values which are slightly lower than the measured average values, thus suggesting for this particular case a conservative estimate of the measured relative density by this procedure. This is consistent with observations of others, and also with more recent data presented by Gibbs, 1971. The correlation by Bazaraa appears to provide a more conservative estimate of the relative density, with most of the average computed values falling much lower than

the values measured in the laboratory (Fig. 3.1d).

In the evaluation of a site for potential liquefaction, it is most desirable to compare both field and laboratory results using as closely similar values of relative density as possible. For this purpose, the standard penetration resistance test (SPT) has been a useful supplement to undisturbed sampling (and frequently is the only source of information) for providing data from which to evaluate the relative density of cohesionless granular soil deposits. Although it is considered to be a crude test, and the relationship between engineering properties and the dynamic penetration resistance poorly defined, nevertheless its simplicity and practicality make the SPT widely accepted among soils and foundation engineers as an indicator of relative density when proper consideration is given to the effective overburden stress. Furthermore, while there has been much controversy over the ability of the Gibbs and Holtz correlation, developed in the laboratory, to estimate accurately the in situ relative density from SPT data, experience and data accumulated since its development in 1957, indicate that the Gibbs and Holtz criteria as a general rule provide the more reliable basis for computing relative density particularly in the range of less than 90 percent. This correlation is primarily useful to indicate average trends and ranges of values. A further assessment of existing practice for evaluating in situ relative density is given in Chapter 8.

3.2.3 Groundwater Conditions

For evaluating the water condition at a potential plant site, an acceptable procedure would be to install water measuring devices in at least half of the exploratory borings as described in Appendix A. For response calculations, these data enter into analytical studies, indirectly from their influence on the effective confining pressure, which generally affects both the damping and stiffness (modulus) properties of the soil.

For a site to be subject to liquefaction under cyclic earthquake stresses, one primary requirement is that groundwater must be present and be located at a reasonably shallow depth below the ground surface. In general, those sites which are most susceptible to liquefaction have groundwater tables generally located within about 10 feet of the ground surface or less, although sites with deeper water tables approaching 25 feet are potentially liquefiable. Since adequate field installation methods are readily available (Appendix A) to accurately measure the water table at a minimum cost, in granular soils this condition can be evaluated probably more accurately than most of the other subsurface conditions pertinent to this problem.

3.2.4 Bedrock Depth

The depth to bedrock or to rock-like materials at a particular site is of primary interest for use in response calculations. This parameter is determined quite readily from borings down to a depth of at least 200 to 300 feet. Below this depth, geophysical refraction surveys have been found to be an acceptable procedure to determine the depth to rock. Results of such surveys should, however, be verified with reported nearby water or oil-well drill logs, if available; and also with geological records, maps or profiles through the area. It is at this lower boundary that the rock or firm ground input motions are assumed to occur in the analyses and the resulting seismic waves are propagated upward through the soil to the ground surface. For purposes of response calculations, the following approximate accuracies are desirable:

<u>Approximate Bedrock Depth Zone, ft.</u>	<u>Approximate Accuracy, ft.</u>
0-100	5
100-200	5 to 10
200-500	20 to 50
500-1000	≈ 100
>1000	± 200

3.3 LABORATORY INVESTIGATIONS

In the laboratory, certain measurements may be performed on both undisturbed and disturbed samples of soil recovered from borings to further provide the engineer with the classification and physical and engineering properties of soils underlying a particular site. The following sections identify and briefly discuss those tests more or less routinely performed to establish the basic foundation soil properties necessary for site evaluation, whether for static load considerations, or evaluation of ground response or liquefaction potential. Also, in some cases the accuracy with which these properties can be determined, or typical values, are included. In addition to the classification and index properties, these properties include unit weight, shear strength, and relative density. Special tests to establish properties specifically required for ground response analyses and liquefaction behavior are discussed separately in Chapters 4, 5 and 6. These properties include shear modulus, damping, Poisson's ratio, bulk modulus, and strength under cyclic loading.

3.3.1 Soil Classification and Index Properties

The first step in laboratory testing for foundation soils investigations is to visually identify and classify all of the samples of soil obtained, including such descriptions as color, density or stiffness, texture, plasticity, and grain size. Physically, most soils are classified as either gravel, sand, silt or clay, depending essentially upon the predominant size of the particles. Of the several grain-size classification systems presently used, the most common grain-size ranges are as follows (Lambe and Whitman, 1969):

<u>Soil</u>	<u>Equivalent Grain Diameter</u>
Gravel	100 to 2.0 mm
Sand	2.0 to 0.06 mm
Silt	0.06 to 0.002 mm
Clay	< 0.002 mm

In addition, the soil may contain organic materials in the ground. Where organic materials are encountered in large quantities such as to comprise a "deposit", it is usually referred to as peat. This material is easily distinguished by its distinct odor, dark color, and by the presence of fibrous particles of vegetable matter in varying states of decay.

From a mineralogical standpoint there is no difference between gravel and sand, since all such particles result from a mechanical breakdown of parent rock materials. These two soil types are usually satisfactorily classified on a visual basis, although a grain size gradation test (ASTM Designation: D422-63) is available for more specific identification. In this test, the particles are segregated into various sizes by means of sieves, and classified according to the percentages of material by dry weight passing each of the standard sieves.

The clay and silt particles (commonly referred to as fines) can likewise be classified according to particle size; however, other index properties are required to describe more fully their physical nature. Since sieves cannot be made for particle sizes in this range, a hydrometer test is performed to estimate grain sizes, (also ASTM Designation: D422-63). This test, which distinguishes particle size by the average rate of particle settlement in water, and the sieve analyses for coarse-grained soils, are both described in most basic soil mechanics text books.

As particles become increasingly smaller, the surface area of the particle increases sufficiently in comparison with the volume of the particle itself such that surface forces between the particles increase in greater proportion than gravitation forces. In the presence of water, these surface forces are further increased tending to actually bond together surrounding soil particles. Under this condition the soil develops a plastic character. Since clay particles have not only the finest

grains but also are plate-like in shape, they develop large surface forces, and a larger degree of plasticity, and therefore it becomes convenient to describe soils in this range according to their plasticity characteristics.

A system of tests referred to as the Atterberg limits (ASTM Designation: D423-66 and D424-59), essentially defines the plasticity characteristics of clay and silt soils in terms of water content, and generally provides a more reliable basis for classification than actual grain size. Atterberg limit values are generally used to classify fine-grained soils into one of the four basic categories listed as follows:

Low Plasticity Clay

High Plasticity Clay

Silt of Low Compressibility

Silt of High Compressibility

From this physical description based on quantitative data, along with the visual appearance, the soil engineer obtains an initial understanding of not only what the soil is and how it looks physically, but also how surface forces (cohesion) may affect its behavior.

One other simple test which is performed as a matter of routine on even the smallest projects, is the determination of the natural water content of all samples. The water content is the ratio of the weight of water in a sample of soil to its dry weight (ASTM Designation: D2216-66). Sands and gravels typically have water contents of less than 25 percent while clays and silts can range from values of the order of 30 percent to as high as several hundred percent. The water content in relationship to the Atterberg limits of a clay sample is also an indication of its expected compressibility characteristics.

Once these basic classification characteristics are established, the engineer is in a position to assess soil properties determined by other more specialized tests. An experienced engineer, by knowing the basic classification characteristics of the soil being tested, can usually recognize whether the results obtained from the laboratory are reasonable. Disturbance due to sampling, errors in laboratory computations, or natural conditions, such as the presence of slickensided surfaces or an unusual grain structure, can often produce misleading results in the laboratory which may lead to erroneous conclusions. For response calculations, classification tests are used indirectly to assist the engineer in assessing the overall problem as well as to determine if necessary properties under dynamic loading conditions are reasonable or need to be adjusted by correction factors as discussed in Chapter 5.

For the assessment of the liquefaction potential of a site, classification characteristics of gradation, cementation and cohesion are of particular significance. As a result of proper classification and determination of gradation characteristics, it is possible to partially assess, qualitatively, whether a soil is susceptible to liquefaction. Based on experience with field cases of liquefaction (Seed and Idriss, 1967; Kishida, 1969a) and laboratory studies (Lee and Fitton, 1968), medium, fine and silty sands are considered to be the most susceptible to this phenomenon; while coarse gravels and clays are, for all practical purposes, not critical. More detailed discussion of the gradation characteristics and the grain sizes most susceptible to liquefaction is presented in Section 6.2.

Both cementation and cohesion are internal factors which tend to resist deformation and liquefaction under applied cyclic forces. The only way the effect of these two factors can be investigated, quantitatively, is in the laboratory by testing undisturbed samples.

Of the two factors, the effect of cementation is more difficult to evaluate than cohesion, since samples of weakly

cemented cohesionless soils are difficult to: 1) recover from borings, 2) extrude from the sampler, and 3) trim and place in a device for strength testing, without collapse or excessive disturbance. For practical applications, if the cementation is very weak, it is best to temporarily ignore these effects during the testing program, by loading cyclically recompacted specimens instead of undisturbed samples. With this procedure, relative densities for a number of specimens can be more closely controlled, allowing more consistent data, relating stress vs. number of cycles to failure, to be generated. Having established the appropriate correlations of other factors as discussed in Section 6.2, the added benefits of cementation can then either be added, based on the engineer's judgement, or neglected entirely, recognizing, however, that an added margin of safety is present within the calculation.

Testing a number of weak cohesionless samples in the undisturbed state is the alternate procedure for establishing all factors including cementation. However, variations within the different samples, changes in relative density due to disturbance, and other factors, produce a significantly wider scatter of data than the first approach. This makes a comparison and correlation of a number of tests subject to considerable interpretation. For this reason, this latter approach is not usually recommended.

When cohesion is present in a specimen, for example, as in a very silty sand, there are usually enough plastic fines and moisture to bond together the grains within the specimen in an undisturbed state a sufficient length of time for testing. In this case, it is generally more advantageous to test an undisturbed sample, as a remolded specimen of sand with cohesion will usually exhibit an entirely different behavior under identical cyclic loading conditions. Whether higher or lower cyclic loading strengths will be obtained from a remolded sample, as compared to an undisturbed sample, will depend to a large extent on the distribution of the fines throughout the specimen.

3.3.2 Physical and Engineering Properties

a. Unit Weight

There are a number of methods available to determine the unit weight of soil in a deposit, and its variation with depth. It can be measured directly or it can be estimated. The most commonly used method for cohesive soils is to measure the weight and volume of soil specimens removed from bore holes (obtained with normal undisturbed sampling techniques). In some cases, the unit weight can also be obtained in relatively clean sands, if there is sufficient cohesion within the soil to allow undisturbed samples to be recovered from the boreholes. Considerable difficulty is encountered in sampling clean sands from below the water table. Using the above method, unit weights can be determined reliably to within 1 or 2 pounds per cubic foot.

For gravels and clean sands, where undisturbed sampling techniques are not usually employed, penetrometers are used as an indirect method for estimating the unit weight. Based on the resistance to penetration (in terms of standard blow count) and the general classification of the disturbed material removed in the sampler, with experience, unit weight may be readily estimated. The unit weight by this procedure can usually be estimated to within 5 pounds per cubic foot.

The unit weight or mass density is used directly in response analyses and average values must be assigned to each layer within the soil deposit. Because variations in this property of ± 5 pounds per cubic foot, produce a negligible change in the resulting output, either method for determining unit weight is considered adequate. For estimating purposes, typical ranges in total unit weights for the six soil types considered of interest to this study are listed in Table 3.1. These values represent average ranges, but values extending beyond these limits will normally occur only in unusual cases.

TABLE 3.1 APPROXIMATE UNIT WEIGHTS OF SIX BASIC SOIL TYPES

	<u>UNIT WEIGHT * (pcf)</u>			
	Poorly Graded Soil		Well Graded Soils	
	<u>Range</u>	<u>Typical Value</u>	<u>Range</u>	<u>Typical Value</u>
Loose Sand	105-120	110	110-125	115
Dense Sand	120-132	125	125-137	130
Soft Clay	100-120	110	100-120	110
Stiff Clay	120-140	130	120-140	130
Silty Soils	100-125	110	100-125	110
Gravelly Soils	120-140	130	125-145	135

*Unit weights are representative of moist sands and gravels and saturated silts and clays. To obtain the submerged unit weight, subtract 62.4 pcf from each of the above values.

b. Shear Strength

Shear strength is the only soil property noted herein which is not used directly in free field response analyses. However, it is one of the most important engineering properties in foundation design. Determination of the shear strength is normally necessary for slope stability and lateral earth pressure evaluation, for the determination of the allowable bearing pressures for footings, and for the load capacities of piles. Also, it can be used to delineate variations in properties within a thick deposit particularly when the soil classification data are otherwise about the same. In the laboratory, shear strength can be determined in a number of ways depending largely upon the classification of the soil.

For cohesionless soils (i.e., clean sands or gravels, and possibly some silts, the shear strength is referred to in terms of the angle of internal friction, ϕ . This shearing resistance property is generally evaluated either by direct shear tests or triaxial compression tests, usually on samples recompacted

to the estimated field conditions, or on undisturbed samples if they can be satisfactorily obtained. In the direct shear test, a drained test (i.e., performed slowly so that no pore water pressures are allowed to build up during the test) is usually conducted; while in the triaxial test, either a drained or undrained test can be performed. In the latter case, pore pressure measurements may be made and recorded.

For cohesive soils, the shear strength is influenced by the magnitude of the bonding forces between the individual particles as well as frictional effects. The shear strength at the natural water content of a soil is usually measured by failing a cylindrical specimen, axially, in unconfined compression (unconfined compression test), in a "quick" triaxial compression test (unconsolidated-undrained), or in a direct shear test. Other tests also available to measure or estimate the shear strength include: 1) the torsional shear test, 2) the torvane, and 3) the pocket penetrometer. Selection of the test to use depends upon the intended use of the results.

For quick, inexpensive preliminary values of shear strength, either the torvane or the pocket penetrometer may be used. The unconfined compression test and triaxial compression test are used on a routine basis where more precise data are required. To obtain a complete measure of the shearing resistance of a cohesive soil, only the two tests previously mentioned for cohesionless soils are available. For drained conditions, there are the direct shear test and the triaxial compression test; however, for undrained conditions only the triaxial test method is applicable. For cohesive soils, both tests should be performed under consolidated conditions for meaningful results of shear strength to be obtained directly.

A complete discussion of these and other tests, the factors affecting them, together with approximate values for typical soils is given elsewhere (see for example Sowers, 1963; Terzaghi and Peck, 1967; Lambe and Whitman, 1969). Most of these

tests are routinely performed by soil testing laboratories throughout the United States.

In relation to ground response, the undrained shear strength for clay soils is of particular interest since it provides a convenient parameter for normalizing the dynamic shear modulus when expressed as a function of strain (Seed and Idriss, 1970c). Its justification and use for this purpose is described and illustrated in Section 5.4. Typical values for clays may be measured in situ with depth, using the vane test or by testing undisturbed samples using any of the described conventional shear test devices in the laboratory. Years of research on the various laboratory test shearing procedures, and relation of the results to field measurements, have led soils engineers to have a better understanding of how test results should be interpreted or correlated to determine adequately the actual field shear strength.

For cohesionless soils, variations in strength, expressed as the effective angle of internal friction, ϕ' , have been found to have only a minor influence on the dynamic soil properties, and thus also on the computed ground response (see Section 5.2). Rather than strength, the in situ compactness or relative density, discussed previously, has a much more significant influence on ground response. Relative density and the adequacy of present methods for its determination and control in the laboratory are discussed in the next section.

c. Relative Density or Void Ratio

The relative compactness of a sand deposit influences its stress-strain and volume change characteristics, as well as being an important factor in controlling the potential for a soil deposit to liquefy. The void ratio, or ratio of the void volume in a soil sample to the volume of the solids, is a quantitative measure of the compactness of the soil. The relative density is both a qualitative and quantitative measure of the compactness of a specific sample of soil relative to the same soil in its

densest state. Either of these two measures of compactness is used to control the density of specimens prepared for testing under static, cyclic, or vibration loading conditions.

In the laboratory, the void ratio of an undisturbed sample is determined by first measuring the unit weight, water content, and specific gravity of solids, from which it is then calculated by using the following relationship:

$$e = \frac{G\gamma_w}{\gamma_d} - 1 \dots \dots \dots (3.2)$$

In which, e = void ratio

G = specific gravity of solids

γ_w = unit weight of water

γ_d = dry unit weight of sample

To determine the relative density requires, in addition, the determination of the maximum and minimum densities of the specific sample of sand. The minimum density is normally determined by pouring oven dry material gently through a funnel into a mold. The most commonly used method is described in ASTM Designation: D2049-69. For reasonably clean sands, results by this method are generally consistent, even when repeated by different investigators.

The maximum density is normally determined by one of a number of methods with results often varying by several pounds per cubic foot or more, depending upon the method used. To obtain the maximum density, compaction is achieved by either an impact method or by a vibration method. Test variables within the two densifying procedures include the surcharge load during compaction, the frequency of vibration, the compactive energy, and the mold size. A comprehensive comparison of these variables, using vibration procedures (D'Appolonia and D'Appolonia, 1967), showed that the acceleration of vibration and the surcharge had the greatest influence on the compacted density. The best method is,

of course, the one that achieves the larger density, without causing a breakdown of grains. An impact method used by Casagrande (Castro, 1969), although tedious, generally satisfied the above requirements.

Often to determine the maximum value for a given material several methods are used, the results compared, and the test with the maximum density selected. No one method will produce the maximum density for all materials. As a rule, for uniform sands with few fines, a vibration technique is generally satisfactory, while impact methods are better for sands with more fines. If the percentage passing the 200-mesh sieve exceeds approximately 15 percent, the laboratory relative density determination is of doubtful validity. Vibration and impact procedures generally used are described in ASTM Designations D2049-69 and D1557-70, respectively. The latter procedure is the modified AASHTO Compaction method.

Having measured the maximum, minimum and in-place densities or void ratios, the relative density is computed using the following equation:

$$D_r (\%) = \frac{\gamma_{\max} (\gamma - \gamma_{\min})}{\gamma (\gamma_{\max} - \gamma_{\min})} \times 100 = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \times 100 \quad (3.3)$$

In which, D_r = relative density

$\gamma_{\max}$ and $\gamma_{\min}$ = maximum and minimum dry density

$e_{\max}$ and $e_{\min}$ = maximum and minimum void ratio

γ and e = natural (in situ) unit weight and void ratio

Using the same equation, a disturbed sample of sand can be recompacted in the laboratory for testing to any desired relative density. This density can be controlled such that an accuracy within a few percent of the desired relative density can be obtained.

d. Other Properties

Other laboratory tests are also performed in soil and foundation investigations to evaluate such soil characteristics as consolidation and permeability. These tests are adequately described in soil mechanics texts (see Terzaghi and Peck, 1967; Lambe and Whitman, 1969). Because these properties are not directly utilized in earthquake response analyses or evaluation of liquefaction potential, they are not discussed herein.

CHAPTER 4

DETERMINATION OF DYNAMIC SOIL PROPERTIES

Much progress has been made in recent years in the development of analytical procedures for evaluating the response of soil deposits under earthquake loading conditions. Successful application of these procedures to the determination of ground response in specific cases, however, requires the use of representative soil properties in the analyses. Thus, considerable effort has been directed toward the development or improvement of methods to determine the required dynamic soil properties for use in these analytical procedures. As a result of efforts in the past 40 years to solve other vibration problems involving soils, several field tests and a number of laboratory tests have been developed. Since each test developed could either be designed only to operate within a limited strain range, or had inherent limitations in the apparatus, no one test has been found adequate for fully defining the dynamic properties required for most earthquake vibration problems.

In cases where there are no permanent soil displacements, the response at the ground surface to a specific input motion at depth is determined primarily by the shear modulus and damping of the soil. Also, in the solution of two-dimensional finite element models, the compression (elastic) modulus and either Poisson's ratio or the bulk modulus must be defined. Fortunately, for practical problems the compression modulus varies directly with the shear modulus at comparable levels of strain and, therefore, can be computed by a simple relationship from the theory of elasticity, if Poisson's ratio is known. (See Equation 4.5, described subsequently.) Since the compression and shear modulus can be interchanged by this simple equation, only one of these parameters need be determined. For the purposes of this report, emphasis is placed on the determination of the shear modulus rather than the compression or elastic modulus.

The following three sections introduce the shear modulus and damping ratio parameters, and summarize the available dynamic field and laboratory tests or methods used to determine their magnitude. Also, the limitations of each test and the adequacy of the procedures and test results obtained are discussed relative to their applicability to site response analyses under earthquake loading conditions. Although not considered to be dynamic soil properties, as such, Poisson's ratio and bulk modulus are discussed in the fourth section, together with methods used to determine the magnitude of these parameters, and typical values for use in response calculations. These parameters have been placed here, because they are used primarily in certain dynamic response prediction models. The final section of this chapter provides general conclusions regarding the present state-of-the-art for the determination of the above-mentioned soil properties.

As a follow-on to the extensive discussion of the determination of the shear modulus and damping ratio in Chapter 4, Chapter 5 summarizes the available data on these two properties for soils determined by these various methods. These data are presented subsequently in a form which can provide a useful guide in the selection of soil characteristics for analysis purposes.

4.1 GENERAL

The stress-strain characteristics for soils are nonlinear and, thus, may broadly influence the response of a site to earthquake-induced cyclic shear loading. In general, successive wave pulses are characterized by shear stress reversals at varying amplitudes and frequencies, as illustrated by the stress records in Fig. 4.1. A single cycle of simple shear loading and unloading produced by an earthquake on a soil deposit, assuming zero initial stress, is thought to follow a curvilinear stress-strain curve having the form of a long, narrow, closed loop (hysteresis loop) as shown typically in Fig. 4.2. Similar loops, obtained in the laboratory by subjecting soils to various types of cyclic axial and torsional shear loading, show not only that soil

**Fig. 4.1 SHEAR STRESS VARIATION DETERMINED
BY RESPONSE ANALYSIS FOR THE
NIIGATA EARTHQUAKE**

(From Seed and Idriss, 1967)

Definitions

Shear modulus, $G = \frac{G_2}{1}$

Damping ratio, $\lambda = \frac{A_L}{4\pi A_T}$

A_L = Area of Loop

A_T = Area of Triangle

Fig. 4.2 HYSTERETIC STRESS-STRAIN RELATIONSHIPS AT DIFFERENT STRAIN AMPLITUDES.

is non-linear but also that it is capable of absorbing a considerable amount of energy, especially at larger levels of strain. At extremely small strains, the fact that the stress-strain characteristics are nearly linear and elastic is also evident.

For ground response analysis, the two dynamic soil properties, shear modulus and damping, as defined in Fig. 4.2 must be determined to fully describe the characteristics of each hysteresis loop for each earthquake wave pulse. The effective shear modulus of the soil controls the velocity of the propagating shear waves and is expressed as the equivalent secant modulus, determined by the slope of a line passing through the ends of the hysteresis loop at the peak stress and strain after each cycle of load. By using this definition for shear modulus, rather than the slope of the actual stress strain curve (tangent modulus), the non-linear system has a parameter consistent with that parameter normally used to define an equivalent linear viscoelastic system. The shear modulus, expressed in this form, can be more easily incorporated into mathematical analyses than other expressions required to define the complex curvilinear loading curve.

The damping ratio provides a measure of the energy absorbing characteristics of a soil. Under earthquake vibratory loading, damping arises primarily from non-linear frictional effects, known as hysteresis, as mineral particles slide upon adjacent particles. The strain energy released during unloading is less than the strain energy stored during loading. Hence, the extent of slippage of individual particles in a soil mass affects the amount of energy absorbed. The greater the movement or straining within the soil below the failure load, the greater is the damping. This property is normally represented by the area inside the hysteresis loop in Fig. 4.2. In treating the non-linear stress-strain characteristic of the hysteresis loop also as an equivalent linear viscoelastic model, the damping ratio, λ is expressed as follows:

$$\lambda = \frac{A_L}{4 \pi A_T} \quad \cdot \cdot \cdot \cdot \cdot \cdot \quad (4.1)$$

Refer to Fig. 4.2 for meaning of symbols.

From the above discussion, it is apparent that both the shear modulus and damping properties will depend on the magnitude of the strain for which the hysteresis loop is determined, and hence must be evaluated in terms of the induced strain in a soil specimen or soil deposit. Since analytical calculations of response under earthquake loading conditions apply to pre-failure conditions, discussions in the following sections refer to modulus and damping values in terms of stresses and strains prior to failure.

4.2 SHEAR MODULUS

4.2.1 Field Tests

Three field test methods that have been utilized to determine the in situ dynamic shear modulus and its variation with depth are: 1) geophysical tests, 2) surface vibrator tests, and 3) plate bearing tests (See Table 4.1). Sketches of each test procedure are shown in Fig. 4.3. Each method, except the static plate bearing test, consists of propagating low energy waves through a soil deposit and either measuring the shear-wave velocity directly, or deducing it indirectly from frequency and wave length measurements. In an assumed elastic medium, in this case soil, the elastic modulus, E , and the shear modulus, G , can be computed from the velocity of compression (V_c) and shear waves (V_s), using the following relationships:

$$E = \rho V_c^2 \frac{(1 + \mu)(1 - 2\mu)}{(1 - \mu)} \quad \dots \dots \dots (4.2)$$

$$G = \rho V_s^2 \quad \dots \dots \dots (4.3)$$

in which, ρ = mass density
and μ = Poisson's ratio

In the static plate bearing test, a repeated load procedure is employed on an exposed surface of soil, and the compressive load-deformation relationships are directly measured. Using these relationships and theories of stress distribution, the modulus, E , at the surface is first computed by Equation 4.4 and then

TABLE 4.1 FIELD TESTS TO DETERMINE SHEAR MODULUS IN SITU

<u>Test Method</u>	<u>General Procedure*</u>	<u>Approx Strain Range %</u>	<u>Reference</u>
1. Geophysical Test	Direct shear-wave velocity measurement w/depths	10^{-3} to 10^{-5}	Swain, 1962
2. Surface Vibrator Test	Measurement of frequency & wave length of surface waves	10^{-3} to 10^{-5}	Ballard, 1964
3. Plate Bearing Tests			
a. Vibratory Test	Measure natural frequency of small vibrator	10^{-3} to 10^{-5}	
b. Static Test	Slow repeated load - settlement measurements	0.08 to 5	

*Tests can be performed generally on all soil types

a. GEOPHYSICAL TESTS

b. SURFACE VIBRATOR TEST

c. PLATE BEARING TESTS

Fig.4.3 FIELD TESTS TO EVALUATE SHEAR MODULUS

converted to a shear modulus, G, using Equation 4.5. In both equations, a value must be assumed for Poisson's ratio, μ .

$$E = \frac{P (1-\mu^2)}{r \pi w} \dots \dots \dots (4.4)$$

$$G = \frac{E}{2 (1 + \mu)} \dots \dots \dots (4.5)$$

where r = radius of plate
 w = deformation
 P = load

All of these tests except the static plate bearing test subject the soil to somewhat lower levels of strain (estimated to be between 10^{-3} to 10^{-5} percent) than normally caused by an actual earthquake (see Fig. 4.4). Generally, strains in the soil of less than 10^{-3} percent at typical earthquake motion frequencies (1-5cps) are not noticeable to most persons. These vibration or geophysical field tests do not generally impose the higher strain levels of the order produced by strong motion earthquakes, because the energy sources required to induce such high internal strains are not practical. Thus, the shear moduli determined by these procedures at the lower energy levels generally overestimate the shear modulus of interest to earthquake design. For response calculations, appropriate scaling down of the measured shear modulus obtained from such field tests must therefore be accomplished to obtain more realistic shear modulus values. The static plate bearing test is performed at higher strains, however, the procedure is not readily applicable to earthquake design as discussed subsequently.

a. In-hole Geophysical Tests

In-hole geophysical techniques are the most widely used of the three field tests described to evaluate the in situ shear modulus. In this procedure, a complex pattern of waves is generated by an explosive charge or a hammer, and the time of first arrival of the shear wave traveling through the soil media from the energy source to one or more geophones is measured.

a. FIELD TESTS

* Note: Range of shear strain denoted as "Earthquakes" represents an extreme range for most earthquakes. "SM-EQ" denotes strains induced by strong motion earthquakes.

b. LABORATORY TESTS

Fig. 4.4 FIELD AND LABORATORY TESTS SHOWING APPROXIMATE STRAIN RANGES OF TEST PROCEDURES

The measured shear or S-wave velocity, can be used directly in elastic wave theory to compute the shear modulus (see Equation 4.3). However, faster traveling, compression or P-waves are also generated, in addition to the shear waves, and must be filtered from the wave patterns to delineate the true shear wave velocity. Difficulties in clearly defining the first arrival of the S-wave from the faster P-wave records pose difficulties in interpreting the results of geophysical procedures for evaluating the shear modulus.

One approach that may be considered applicable is to measure the velocity of the easily distinguished P-wave, compute the elastic modulus and convert these values to shear modulus. For foundation design, this procedure is not usually accomplished or accepted because the presence of water in soil (i.e., a water table) greatly influences the velocity of the P-wave measured. The presence of water does not, however, significantly influence the velocity of the shear wave. Thus, in most cases, it is usually preferable to measure the shear-wave velocity directly.

As shown in Fig. 4.3a, three basic techniques are employed in geophysical tests. The first technique (cross-hole technique) involves generating short bursts of energy at various depths in one bore hole and measuring the time of arrival of the induced waves at equivalent depths with geophones in other nearby bore holes. This procedure is commonly utilized and is very useful, especially in the more homogeneous soil deposits.

The more homogeneous and less stratified the soil deposit, the easier it is to delineate shear waves from compression waves since each respective wave is traveling at a relatively uniform velocity. On the other hand, the delineation of waves in thinly bedded or stratified materials is very difficult and their interpretation should be viewed with caution. Not only is a complex pattern obtained from the waves traveling at various velocities, depending upon the competency of each soil layer, but also the first arriving waves may be indicative of the more competent higher velocity layers instead of an average value for the

overall mass, potentially resulting in higher computed shear modulus values than actually present.

The other two geophysical techniques (up-hole and down-hole tests) involve generating waves with an impulse source of energy at the surface or at depth in a boring, and measuring the average shear-wave velocity between each geophone and the energy source. In the up-hole technique, the waves are generated at various depths in a boring with geophones located along the ground surface. In the down-hole technique, the waves are generated at the ground surface with the geophones located within the boring. In these techniques, both acceptable procedures, average velocities are recorded for the soil between the energy source and the location of the geophone. Hence, velocities may be determined for different zones or layers in the soil deposit by locating the geophones or energy source at appropriate depths, in conjunction with boring information.

The major difficulty of picking out the first arrival of the S-wave from the P-wave records is satisfactorily resolved in down-hole techniques by reversing the polarity of the source generating the wave pattern. In this process, measurements of the wave pattern using an impulse source of energy (usually a horizontally directed sledge-hammer blow on a firmly embedded post) are made twice. The post is struck first in one direction (i.e., parallel to the ground surface) and then a second time in the same manner, except the blow is made 180 degrees out of phase (i.e., in the opposite direction). By reversing the direction (or polarity) of the energy blow, the shear-wave pattern is recorded in the reverse direction while the compression wave pattern is essentially unchanged, allowing a distinction between the compression and shear wave patterns to be made.

Although the up- and down-hole techniques allow wave types to be separated more rationally than the cross-hole technique, local bore hole effects can also provide equally misleading results. Care must be taken so that these effects do not mask out the wave signals traveling through the soil. For instance, where

casing is required in a hole to prevent caving, the velocity and strength of signals traveling through the casing may mask out the slower and weaker signals in the soil. Resolution of such problems in each of the three techniques is, however, routinely overcome by experienced geophysicists.

One shortcoming of all geophysical techniques is that presently used energy sources generate only a single short impulse of energy. This produces one massive radial pattern of waves which is difficult to duplicate, since the magnitude of the energy itself is difficult to couple to the bore hole wall, especially when explosive charges are used. In recent years, the Waterways Experiment Station (WES) has developed a vibropacker system for inducing steady-state polarized shear and compression waves at depth for cross-hole measurements (Ballard and Leach, 1969). While the reported system has apparently been used only in feasibility studies, it shows promise for in situ measurement of shear-wave velocity. One advantage, in addition to its ability to apply a controlled energy source over a period of time, is that this method provides constant coupling of the energy source to the bore hole wall.

Of the three basic geophysical tests, the down-hole and cross-hole techniques, are the two most accepted methods in practical use for determining the in situ shear modulus with depth, although both have different limitations. The choice of one procedure over the other is largely one of preference of the engineer or by the geophysical firm making the measurements. Many engineers involved in design of nuclear power plants have in the past used cross-hole techniques rather than down-hole techniques, possibly because it appears to involve more of the average soil conditions under the site in each measurement, rather than at individual boring locations as in the down-hole technique. Although no known studies have been made comparing the results of the two techniques at given sites, shear-wave velocity measurements (and thus shear moduli) of comparable soil types and conditions using either method are not noticeably different. Such tests are not limited to

any particular type of soil and, theoretically, can be conducted to almost any depth that a boring can be advanced. Under practical conditions, however, 200 to 300 feet is rarely exceeded with reliable definition.

Of these techniques, the cross-hole procedure is likely to be more costly because of the additional expense of drilling extra bore holes, although these holes can be planned in the normal foundation investigation to also satisfactorily define the subsurface conditions. Commercially, the approximate cost of performing these tests at appropriate depth intervals (generally about 10 feet) to a depth of about 200 feet, excluding the cost of making the bore holes, is in the order of \$800 per crew day. Approximately 1 to 5 crew days may be required at a particular site for this purpose, although this estimate may vary considerably, depending on specific site requirements.

Equipment for performing these tests is relatively inexpensive and is readily available in the United States. Because of the nature of geophysical explorations, commercial firms specializing in this work maintain a high degree of mobility for moving both personnel and equipment into even the remotest areas, on relatively short notice.

b. Surface Vibrator Test

Another in situ procedure for measuring the shear modulus of all types of soil employs the use of a surface vibrator as developed by the Waterways Experiment Station (WES) in 1963 (refer to Ballard, 1964; Maxwell and Fry, 1967). In this procedure, an electromagnetic oscillator is used at high frequencies of vibration (30 to 1,000 cycles per second, cps) and a rotating mass type oscillator is used for producing low frequencies (below 30 cps). These surface vibrators generate Rayleigh waves which, for all practical purposes at small strains, have a velocity very close to the shear-wave velocity (Richart, 1960; WES, 1963). The velocity is computed from the wave length measured with geophones along the ground surface (Fig. 4.3b) and the frequency of vibration at the

source, using the following equation:

$$V_s \approx V_r = \lambda f \dots \dots \dots (4.6)$$

where λ = wave length
f = frequency of vibration

The Rayleigh wave velocity, V_r , measured between adjacent points along the ground, has been assumed empirically to correspond to the velocity through the layer of soil located at an average depth equal to one-half of the wave length. By varying the frequency of the source, and hence changing the wave length, it is then possible to obtain the variation of shear velocity with depth. The low frequency oscillator may be utilized to determine low strain dynamic shear moduli to depths on the order of 100 feet or more, depending upon site conditions and soil materials. Values down to depths of 150 feet may be obtained in materials with higher values of shear modulus.

The fact that the measured velocity at the surface corresponds closely to the shear-wave velocity of the soil at a depth of one-half wave length was first established empirically by studying the properties of pavement subgrades (Heukelom and Foster, 1960). There is, as yet, no rigorous justification to support this conclusion other than by indirect verification. More recent studies with theoretical relationships for layered systems have produced mathematical equations for determining the interfaces between successive layers more accurately than the half wave length concepts (Baladi, 1970). The equations are based on the assumption that each layer within a deposit exhibits a constant velocity. In reality, it is not known whether the velocity determined by this field procedure represents the average velocity of the layer or an average velocity between the ground surface and the computed depth. For relatively homogeneous materials, the half wave length criterion has been shown to give satisfactory modulus results, as a function of depth, when correlated with other field and laboratory procedures performed at equivalent low strain levels. For thick layered materials, where waves are refracted several

times, application of the mathematical relationships appears to be an improved approach, although this method has only been verified in a few tests. For multilayer deposits, especially where data must be determined at significant depths, the assumed interpretation and validity of either approach will probably lead to quite different values of shear modulus. For such conditions, this test is as yet, of doubtful validity.

Equipment for this test is not readily available outside of WES and a few universities, as the commercial need for this test is limited primarily to vibratory foundation problems, and large construction projects located in seismically active areas. However, even for these purposes, less expensive geophysical procedures have in the past been found to be adequate for determination of the dynamic soil properties in situ, subject to the limitations presented in the previous section. Federal agencies, on the other hand, have had a greater demand for this type of device in that they have needed to evaluate foundation soil behavior in planning facilities for launching spacecraft, radar tracking towers, various heavy weapons, and delicate electronic equipment.

c. Plate Bearing Tests

The modulus of soil can also be determined, under undrained conditions, by either applying slow repeated loads to a small plate and measuring the slope of the load settlement curve; or by determining the resonant frequency of a smaller vibrator placed on the soil in situ. Both of these procedures, shown schematically in Fig. 4.3c, allow determination of modulus values for the soil directly beneath the plate. The most undesirable features of these test procedures are the sources of error introduced by seating irregularities, the lack of confinement, the fact that the modulus determined is largely a function of the size of plate used, and they are generally limited to near-surface soil properties. These methods were developed primarily for the design of footings subjected to rocking vibrations, such as those induced

by rotating towers and antennas. Because modulus values as a function of depth are required for site response analysis, plate bearing tests are not practical nor really applicable for this purpose.

4.2.2 Laboratory Tests

There are five basic cyclic loading or vibratory tests presently available to measure the dynamic shear modulus in the laboratory for strain levels of interest to ground response analyses. These tests include: 1) the cyclic triaxial test, 2) cyclic simple shear test, 3) cyclic torsional shear tests, 4) resonant column tests, and 5) shake table tests. All of the specimens in these five tests are subjected to one of the three general loading procedures given in Table 4.2. In the repeated loading procedures, the stress-strain relationships can be measured directly; while in the forced or free vibration procedures either a resonant frequency or the frequency in free vibration must be measured in order to compute the shear modulus. In general, the repeated loading test procedures are useful at relatively high strains, while the forced and free vibration tests are better used at low strain levels. The approximate strain range for each test and the types of soil for which the test is best suited are given in Table 4.2.

There are also a number of static shear loading tests used in foundation studies primarily to determine soil shear strength or, less directly, to estimate the shear modulus. These tests include the triaxial compression test, the direct shear test, the torsional shear test, the vane shear test, the cone penetrometer test, the soil box test, and the squeeze test (Sowers, 1963). The shear modulus calculated by these methods usually represents the tangent of the assumed initial linear elastic part of the stress-strain loading curve rather than the secant modulus (a parameter consistent with an equivalent linear viscoelastic system) defined previously for use in dynamic studies. Each of these tests, described elsewhere (i.e., Sowers, 1963) determines modulus values at strain levels generally greater than most earthquakes (0.1 percent) and is, therefore, generally not used in

TABLE 4.2 LABORATORY TESTS TO MEASURE SHEAR MODULUS AND DAMPING

<u>General Procedure</u>	<u>Test Method</u>	<u>Approx. Strain Range</u>	<u>Properties Determined</u>	<u>Undis- turbed Cohesion- less</u>	<u>Soil Types Disturbed Cohesion- less</u>	<u>Undis- turbed Cohesive</u>	<u>Dis- turbed Cohesive</u>	<u>References</u>
06	1. Repeated cyclic loading (allows direct determination of hysteretic stress strain relationships)	1. Cyclic Triaxial	10^{-2} to 5%	MD*	+	+	+	Seed & Lee, 1966 Lee & Seed, 1967c
		2. Simple Shear						
		A. Small Scale	10^{-2} to 5%	MD*			+	Thiers & Seed, 1968b
		B. Large Scale	10^{-2} to 4%	MD*	+	+	+	Silver & Seed, 1969 Kovacs, 1968 Kovacs et al, 1971
		3. Torsional Shear						
		A. Plate & Cone	$.2 \times 10^{-1}$ to 1.6%	MD*			+	Krizek & Franklin, 1968 a&b
		B. Hollow Col.	$.2 \times 10^{-2}$ to 5%	MD*	+		+	Hardin & Drnevich, 1970a
	2. Forced vibration	1. Resonant Column						
		A. Solid Column	10^{-2} to 10^{-4} %	Modulus	+	+	+	Wilson & Dietrich, 1960 Hardin & Richart, 1963 Hardin, 1965 Drnevich, et. al. 1967
		B. Hollow Col.	10^{-2} to 10^{-4} %	Modulus			+	
	3. Free vibration	1. Resonant Column						
		A. Solid Column	10^{-2} to 10^{-4} %	Damping	+	+	+	Richart et. al., 1970
		B. Hollow Col.	10^{-2} to 10^{-4} %	Damping			+	Drnevich et. al, 1967
		2. Shake Table	.005 to .05%	MD*			+	Kovacs, et. al. 1971
		3. Torsional Shear						
		A. Solid Column	.2%	Modulus	+	+	+	Zeevaert, 1967
		4. Simple Shear						
		A. Large Scale	10^{-2} to 4%	MD*	+	+	+	Kovacs, et. al 1971 Kovacs, 1968

*MD - Modulus and Damping

dynamic analyses. Also, shearing does not occur cyclically (simple shearing with stress reversal) as in actual earthquakes; but directly, under a continuous static compression or shearing force.

For many soils, the major shortcoming of all laboratory tests, whether for determining the shear modulus, damping or any other parameter, lies in the fact that undisturbed samples are difficult to obtain and test. In general, the greater the degree of sample disturbance, the lower the shear modulus and the higher the damping. Thus, the amount that these values must be adjusted depends upon the degree of disturbance of the specimen tested. The soils most susceptible to sample disturbance are very sensitive, soft or medium stiff, clays and silts. Since the degree of disturbance cannot be determined accurately, especially for the more sensitive soils, the engineer must often use judgment based on past experience. This effect can result in values of shear modulus in the laboratory which are two to three times lower than would be determined in situ, as illustrated in Section 5.4. The necessity for large correction factors to allow for the effect of disturbance is one of the primary reasons for development of a more direct test technique for determining these properties in situ.

Several of the existing dynamic tests have, over the years, been modified or refined by various investigators to extend the capabilities of these tests to use in the solution of most vibration problems. Descriptions of the basic tests, the limitations of each test and the applicability of each test for use in earthquake soil response studies are given in the following paragraphs. In the descriptions, references are made to Table 4.2 and Fig. 4.4. In Table 4.2, the existing dynamic tests are listed along with the approximate strain range of each test, and the types of soil on which the test can be performed. The strain range of each test is illustrated graphically in Fig. 4.4, in comparison with strains encountered in strong earthquakes.

a. Cyclic Triaxial Test

The cyclic triaxial test is a repeated compression

loading test which can be used to determine the compression (or Young's) modulus of soil and, indirectly, the shear modulus. In this test, thoroughly described in the literature (Seed and Lee, 1969; Schroeder and Schuster, 1968a), longitudinal compression and extension is applied to a solid cylindrical-shaped specimen (Fig. 4.5) and the resulting compressive stress-strain characteristics measured directly. Since the hysteretic stress-strain loading curve is determined in compression instead of shear, the compression modulus, determined directly, must be converted to shear modulus, using Equation 4.5. For each loading cycle, a hysteresis loop can be defined; and hence, under a controlled stress, either shear or compression modulus can be calculated for each successive cycle and each corresponding level of strain up to single amplitude shear strain values approaching 5 percent.

The triaxial test method, although readily used in engineering practice because of its versatility to different stress conditions, has several limitations which permit only an approximate duplication of the field stress conditions. In the field, the major principal stress is initially vertical but rotates through some small angle as cyclic shear stresses (under earthquake loads) are symmetrically applied to the soil. In the laboratory, the stresses are not symmetrically applied, since the intermediate and minor principal stresses are equal in axial compression but are shifted and become equal to the major principal stress during axial extension. This 90-degree reversal of principal stresses during each load cycle is not the same as in the field.

Secondly, in the field, a horizontal soil deposit subjected to free-field stress conditions is anisotropically consolidated. To achieve initial zero shear stresses in a laboratory specimen, the specimen must be isotropically consolidated.

Still another condition which prevents correct field simulation is the fact that the laboratory triaxial specimen undergoes deformation in each of the three principal stress directions. Presumably, the soils in the field under earthquake motions are deformed primarily in simple shear, or unidirectionally.

Fig. 4.5 CYCLIC TRIAXIAL TEST
(Cyclic Axial Loading)

Within the laboratory equipment itself, there are also shortcomings which permit only approximate duplication of field conditions. Rigid plates at the ends of the soil specimen cause non-uniform stresses and strains within the specimen even before axial loading is applied. These edge conditions and friction developed between the plate and soil specimen cause localized stresses at points which often lead to a premature progressive type failure, rather than a failure wherein all shear stresses are uniformly mobilized. This shortcoming can be assumed as a minor influence if reasonably long cylindrical specimens are used, having length to diameter ratios of about 2.5. Techniques are also available to minimize these end effects.

Although this test has a number of shortcomings, most of these conditions are also present in many of the other test procedures. The cyclic triaxial test, as noted in Table 4.2, is versatile and does have the advantage of being adaptable to the preparation and testing with ease of all types of disturbed and undisturbed soils (except gravels, unless large, more costly apparatus than presently available in most laboratories is used). Also, precise control of stresses and strains and the ready availability and familiarity of numerous laboratories with the equipment are strong advantages of this test.

b. Cyclic Simple Shear Test

Three basic cyclic simple shear tests have evolved in the past five years for studying dynamic soil properties as well as liquefaction characteristics. Two of these tests were developed for testing small scale specimens, while the last and most recent test was performed with large scale specimens of clay. The following paragraphs discuss in general each of these tests.

1) Small Scale Tests

Both of the small scale simple shear tests apply actual cyclic shear stresses and strains to the soil specimen and, thus, the shear modulus is

determined directly. In one test, the specimen shape is cylindrical (Fig. 4.6a) while the other test is performed on a rectangular specimen (Fig. 4.6b). Results of such tests performed on the rectangular shaped specimen were first published in 1968 (Peacock and Seed, 1968); and the test procedure was proposed as a means for more closely duplicating cyclic simple shear conditions in the laboratory and evaluating soil liquefaction better than with the cyclic triaxial test. The second simple shear device, introduced as early as 1961 for studying the hysteretic behavior of soils (Converse, 1961), was somewhat modified and improved in more recent years (Thiers and Seed, 1968b).

These simple shear tests employ either a uniform shear stress apparatus (modified from the Norwegian Geotechnical Institute shear apparatus, (Kjellman, 1951) or a uniform shear strain box (modified from the original design by Roscoe, 1953). The difference in the two types of simple shear apparatus, shown schematically in Fig. 4.6, is described, for example, in Sowers, 1963.

Although the simple shear test does very nearly duplicate simple shear conditions, and is generally applicable for all soil types, it nevertheless has a few conditions and shortcomings in the apparatus which again make duplication of field conditions only approximate. Boundary effects may cause either local stress concentrations near the corners or the edge surfaces, or nonuniform strain conditions, each of which may lead to a progressive and premature type of failure and, thus, to lower values of shear modulus.

Of the two small scale simple shear tests,

SPECIMENS APPROX.
3" diam. X 0.4 " high

LOADING A

LOADING B

a. UNIFORM STRESS IN SIMPLE SHEAR
(Cylindrical Shaped Specimen)

LOADING A

LOADING B

(Apparatus adapted from the original design by Roscoe, 1953.)

b. UNIFORM STRAIN IN SIMPLE SHEAR
(Rectangular Shaped Specimen)

Fig.4.6 CYCLIC SIMPLE SHEAR TESTS
(Cyclic Shear Loading)

the cylindrical-shaped apparatus with the flexible wire sides is the more desirable test for measuring shear modulus, in that undisturbed soil can be more easily trimmed and placed than in the rigid rectangular box. The cylindrical apparatus, nevertheless, has the important limitation in that it cannot be used for testing saturated sands (a very commonly encountered natural condition); however, the rigid box apparatus is particularly suitable for this purpose.

Because undisturbed saturated samples of sand are difficult to obtain, much less trim into a rigid box, the influence of boundary effects and disturbance is believed to have a serious effect on values of the shear modulus. This effect is eliminated by testing remolded samples which are prepared directly in the testing apparatus. For practical purposes, for sand soils the use of remolded samples prepared directly in this apparatus probably provides a reasonably good measure of actual soil properties, if density conditions are duplicated, within reasonable limits to field values. As will be discussed subsequently in Chapter 6, boundary effects are even more significant factors or limitations in evaluating the liquefaction potential of a sand soil.

Although the cyclic simple shear test does have boundary shortcomings, it is a good laboratory test procedure for most nearly duplicating the field conditions for soil subject to strong earthquake motion (large strains). However, measured modulus values with this test may underestimate the field modulus at comparable strains, largely because boundary stress effects cause a reduction in the soil stiffness and thus produce a slightly lower shear modulus.

Equipment for this test is not commercially available but is used in at least three universities, mostly for research. Since this test (especially the model adapted from the Norwegian Geotechnical Institute apparatus) does provide a direct measurement of shear modulus (Silver and Seed, 1969) it may, in the future, find greater usage. Equipment for the cylindrical shape uniform stress test can be obtained from Geonor (commercial marketing organization for test equipment developed by Norwegian Geotechnical Institute) at an approximate cost of \$3,000, and may be converted for determining the dynamic shear modulus for undisturbed samples of most cohesive and dry, cohesionless soils. Equipment for the rectangular-shaped uniform strain test has usually been fabricated by the respective research organization conducting the test.

2) Large Scale Test

As a follow-on to the small scale simple shear test, this larger apparatus was developed for measuring the dynamic properties of soft clay (Kovacs, 1968). The device shown in Fig. 4.7 utilizes a vibrating actuator to provide excitations at the top of the block-shaped specimen, while the base is held stationary. Remolded clay specimens with dimensions of 12 inches by 12 inches by 6 inches high were used in research studies by Kovacs.

Soils tested by this technique may be either freely vibrated or strained under repeated cyclic loading conditions. The free vibration method is conducted by first straining the unsupported soil block in a horizontal direction to some predeter-

(From Kovacs, 1968)

Fig. 4.7 LARGE SCALE SIMPLE SHEAR TEST

mined amount, and then releasing the soil block so that it vibrates freely. The decay of displacement or acceleration with time is measured using an LVDT or an accelerometer. Using the theory of transverse vibration of shear beams, the shear modulus for a known period of free vibration can be computed by rearranging a relationship presented by Jacobsen and Ayre, 1958, and solving for G. The modulus may be determined from the first mode of vibration by the following equation:

$$G = \frac{24 \gamma H^2}{g T^2} \quad (\text{psf}) \quad (4.7)$$

where γ = unit weight of soil, lb/ft³

H = height of soil block, ft.

g = gravity constant, 32.2 ft/sec²

T = period of free vibration, sec

A repeated loading method may also be used to determine the shear modulus. This method of testing is conducted by attaching an actuator to the upper surface of the unsupported soil block and then oscillating the soil in a horizontal direction with the actuator for the desired number of cycles. The shear force is measured with a load cell and the displacement with a LVDT. Shear moduli are determined from the slope of the stress versus strain curve for each loading cycle in a similar manner to the smaller simple shear tests.

Strain levels for these tests can also be extremely variable and will depend largely on the stiffness and height of the soil. Strains approaching 4 percent were obtained by Kovacs (Table 4.2) and it is reasonable to assume that strains in the order of 10⁻² percent could be used, providing linear measurements can be taken to

this accuracy.

As with shake table tests, to be discussed subsequently, this technique is limited to research studies, as blocks of soil with dimensions in the order of 12 inches cannot be readily obtained with depth in an undisturbed state. Thus, only shear moduli of disturbed or remolded specimens can be determined by this technique. In addition other limitations, such as those discussed for shake tables, also exist for this technique.

c. Cyclic Torsional Shear Tests

Three different types of cyclic torsional shear tests have been developed in the last 5 years to determine the shear modulus of soil. In two methods, cyclic torsional loads are applied to the soil, while in the third method the sample is allowed to vibrate freely after an initial torsion is applied and then released. A sketch of the basic specimen shapes and stresses applied in each torsional shear test is given in Fig. 4.8.

The torsional column loading test, shown in Fig. 4.8a, was developed by Hardin and Drnevich, 1970a, and is the most refined of the torsional tests as it measures not only very small shear strains but also the larger strains in the earthquake range. In this test, a long hollow cylinder of soil about one half inch thick is slowly twisted at 1/12 cps at the desired strain level, and the resulting load deformation relation measured directly. The shear modulus can be determined directly for the peak to peak slope of the resulting hysteresis loop (Fig. 4.2). The low frequency was found desirable in order that the inertial forces of the sensitive loading system would not significantly affect the measured load deformation relation. This repeated loading test procedure, based on a study of the various parameter effects (Hardin and Black, 1966 and 1968) is a useful test procedure, as this condition closely resembles one of pure shear. Undisturbed samples are difficult if not impossible to prepare; therefore, the test is

a. TORSIONAL COLUMN LOADING TEST
 (Torsional Cyclic Loading at Low Frequencies)

b. TORSIONAL COLUMN LOADING TEST
 (Torsional Cyclic Loading in Free Vibration)

c. TORSIONAL PLATE AND CONE TEST
 (For Soft Clays Only)

Fig. 4.8 TORSIONAL SHEAR TESTS

usually performed on remolded specimens prepared in the laboratory.

Equipment for this torsional test is not readily available for commercial testing as this device has only been recently developed. Its use has also been largely restricted to basic research.

The second torsional column loading test, shown in Fig. 4.8b, was developed by Zeevaert, 1967, as a simple, practical test for readily measuring the shear modulus of all soil types in which undisturbed samples could be obtained. In this test, a solid cylindrical column of soil is initially twisted at one end, then released and allowed to decay in free vibration. A heavy mass placed on the end of the cylinder creates a single-degree-of-freedom system, with the stiffness provided by the soil and the inertia provided by the mass. The resulting frequency of the specimen in free motion is then measured and the shear modulus calculated using the following equation for the natural period of a single-degree-of-freedom system:

$$G = \frac{W_{nc}^2 K}{1 - \frac{W_{nc}^2}{W_{ni}^2}} \quad \dots \dots \dots (4.8)$$

in which $W_{nc} = \frac{W_{dc}}{1 - D_c^2}$; $W_{ni} = \frac{W_{di}}{1 - D_i^2}$

and where K = a constant characteristic dependent upon the geometry of the system

W_n = the natural frequency of either the apparatus (i) or the entire system (c)

D = the critical damping ratio of either the apparatus (i) or the entire system (c).

For further explanations of the theory, the procedure and the use of the above equation, reference should be made to a paper by Zeevaert, 1967. This procedure is perhaps the simplest of all the laboratory tests but since it utilizes the conventional triaxial cell, it has many of the same limitations in boundary and

stress conditions as discussed for the cyclic triaxial test. The few tests reported by Zeevaert were performed at shear strains of about 0.2 percent as summarized in Section 5.4. Strains of this magnitude are compatible with large earthquakes, however, values reported are generally lower than would be reported using other methods of testing at equivalent strain levels.

The third type of torsional shear test, which is used only for soft clays, consists of a modified Weissenberg rheogoniometer (Krizek and Franklin, 1968b). The drive unit, as noted in Table 4.2, can cover a wide range of strains as well as a wide range of frequencies (9.5 to 0.0095 cps). A cone-and-plate sample holder is used as shown in Fig. 4.8c.

In this test, the specimens are subjected to steady oscillatory torsional shear deformations, while the moment response is measured. With known specimen geometry and assuming homogeneity of stress and strain throughout the sample, recorded values can be converted directly to shear stress and strains, and thus shear modulus. If an oscilloscope is also used to measure the phase angle at corresponding points in time on the stress and strain waves, the hysteretic damping characteristics can be defined. In assuming homogeneity of stresses in the specimen, one must assume that dilatational effects are negligible. This condition results only when the clays are highly impermeable and strains are small. Inertial forces are neglected which is justified by keeping the angle (6 degrees) between the cone and plate very small, especially at low amplitudes and frequencies of oscillation. In many cases, where the frequency is extremely low (<0.1 cps), creep may cause the modulus value to be lower than would be expected at higher earthquake frequencies. The major shortcoming of the test is that it is limited to soft clays. Also, difficulties in preparing the specimens of clay to shape, control of moisture content of a small specimen, and the inability to apply a confining pressure to the cone specimen, are among the disadvantages of this test procedure for use in commercial soil testing laboratories. Equipment for this test is not available on a commercial basis, and to our knowledge its use has been restricted

largely to basic research.

d. Resonant Column Tests

The resonant column test, noted in Table 4.2, utilizes forced vibratory procedures to determine the shear modulus. In this test, a cylindrical column of soil is vibrated at small amplitudes on one end, either torsionally or longitudinally (Fig. 4.9). By varying the frequency of forced vibration until a resonant condition is attained, the shear or compression modulus for a fixed-free system can be determined from the dimensions of the specimen, using the following relationships (Wilson and Dietrich, 1960):

$$E \text{ or } G \text{ (psi)} = 2.39 f^2 H^2 \gamma \times 10^{-5} \quad (4.9)$$

where f = resonant frequency in longitudinal vibration for
E, and torsional vibration for G, cps

H = height of soil cylinder, in.

γ = unit weight of soil, pcf

Other factors may need to be applied to correct for inertia effects of the cap and membrane. The shear modulus, G , is computed from the resonant frequency when torsion is applied to the soil column; whereas, longitudinal impulses allow computations of the compression modulus, E . At these small strains, the two moduli are also related by the Equation 4.5, and if they are evaluated separately, Poisson's ratio may be computed. Values of Poisson's ratio determined by this process, however, are usually low as are the corresponding strains and not applicable for most earthquake design problems.

Various arrangements and specimen configurations are commonly used for the resonant column test. For torsional loading, specimens are either solid or hollow cylinders and have either "free-free" end conditions or "fixed-free" end conditions. The free-free end condition is the least desirable of the two, because the actual degree of freedom is not easily defined since neither end is entirely free.

Of the two specimen shapes, the hollow cylinder is the

a. SOLID CYLINDER

b. HOLLOW CYLINDER

Fig. 4.9 RESONANT COLUMN TESTS

more desirable, since the applied stresses and resultant strains are, thus, more representative of a simple shear condition. Torsional stresses applied to the end of a solid cylinder of soil result in a non-uniform distribution of stress over the cross-section with high stresses near the edge and nearly zero stress in the center of the cylinder. Since the results obtained using the two shapes are nearly identical, (Richart, Hall and Woods, 1970), provided that average strains across the width of the sample in each test are used, the solid cylinder shape has become more routinely used in most soil laboratories, since it is easier to prepare.

Although this test is perhaps the most practical laboratory technique for directly measuring dynamic shear modulus, and has been used extensively when studying soil parameter effects on the shear modulus, it has the disadvantage in earthquake response studies because the levels of strain of interest during strong motion earthquakes are generally larger than can be applied with this equipment. The maximum strain amplitude of the equipment, as noted in Fig. 4.4, is in the order of 10^{-2} percent. Again as in the field tests, appropriate scaling of the modulus must be accomplished to account for the different levels of strain.

The main advantages of the test include its simplicity, the low cost of the equipment, the ability to use the apparatus for almost all types of soils, and the ability to test relatively undisturbed cylinders of soil which can readily be obtained at depth from borings. More details of the various components of equipment used for this test are described in the following articles: Wilson and Dietrich, 1960; Hardin and Richart, 1963; Hardin and Music, 1965; Stevens, 1966; Wilson and Kaarsberg, 1966; and Drnevich, Hall and Richart, 1967.

The laboratory equipment for this test is available in a number of testing laboratories and universities in the United States. It is presently performed almost routinely for the solution of many foundation vibration problems. Cost of the test equipment, which is usually fabricated on special order, ranges

from about \$4,000 to \$10,000, excluding the oscilloscope. The approximate cost of commercially preparing and testing a specimen in torsion and compression using this apparatus is generally on the order of \$250 per test.

e. Shake Table Tests

The dynamic shear modulus of larger specimens of soil can likewise be determined with the use of shake tables and employing free vibration techniques. Very few tests have been performed to date other than for research purposes, largely because of the size of the soil specimen required and the resultant expense. Typical specimen sizes and shapes for this test are shown in Fig. 4.10.

Shake table techniques have been used primarily for determining the shear modulus of large specimens of only clay materials (Kovacs, Seed and Chan, 1971). The common procedure involves initially exciting the base of the specimen and subsequently measuring the vibration response when the shake table excitation is shut off. The frequency of the first mode is then measured and the shear modulus computed using the following equation:

$$G = \frac{16 \gamma H^2 f^2}{g} \quad \dots \dots \dots (4.10)$$

The shear strain corresponding to this modulus is determined as the soil is vibrated in the steady state condition prior to stopping the shake table action. During this period, the frequency of oscillation and acceleration is measured and shear strain computed using the following equation from a modification of Kovacs, Seed and Chan, 1971:

$$\gamma = \frac{81.7a}{f^2 g H} \quad \dots \dots \dots (4.11)$$

where γ = shear strain, %
 a = acceleration, ft/sec²
 g = acceleration of gravity, 32.2 ft/sec²
 H = thickness of soil layer, ft.
 f = frequency of oscillation, cps

Fig. 4.10 SHAKE TABLE TEST

Results of tests using this approach (Kovacs, 1968) are summarized in Chapter 5.

In order to eliminate boundary effects of the shake table and sample, a large length to thickness ratio is desirable. For example, if it is required to test a 1-foot thick bed of soil, then a table with a length in the order of 10 feet is desirable, especially if rigid sides are used to provide lateral support to the specimen. Such a sample would weigh several tons, in which case a relatively large amount of energy would be required to shake the table. Preparation of a sample of this size to a required uniform density presents additional problems. Also, it is difficult to saturate the specimen and maintain it in a saturated state for this test.

A further problem with the shake table test is that, at present, only small confining pressures can be applied to the sample. Such pressures are usually applied to the specimen in the form of a vacuum (Silver and Seed, 1969) and thus the stresses which are applied to the soil are only representative of stresses within the upper 5 feet of a natural soil deposit.

Strain levels for these tests are variable and depend on such factors as the shaking capacity of the table and the density of the soil. Strains in the range of 0.005 to 0.05 percent have been developed (Silver and Seed, 1969; Kovacs, Seed and Chan, 1971). Lower strain levels could be obtained, however, the actual measurement of strains to the required accuracy becomes more difficult.

While this procedure is suitable for research, it is not practical for usual engineering applications, and it is not normally performed by commercial testing laboratories.

4.3 DAMPING

4.3.1 Field Tests

Theoretically, damping can be measured in the field by observing the rate of decay of amplitude of wave motion with distance from an energy source. The most direct field procedure

is to instrument a deposit of soil with accelerometers at several locations with depth, and compute the moduli and damping factors for the various layers from the measured response during subsequent earthquakes. Although this procedure has yielded some information (Seed and Idriss, 1970a), it has not yet been a major source for earthquake ground response data. While this procedure provides data on the combined effects of the shear modulus and damping, certain assumptions are normally necessary to evaluate the contribution of each. Further, earthquakes are unpredictable in both time and magnitude. Present procedures for obtaining earthquake ground response data should be critically studied to take fullest advantage of this potentially fruitful source of information.

Excitations from nuclear explosions (underground or cratering) also offer a potential field energy source for vibrating soil deposits in situ. They provide a large source of energy and can be controlled somewhat in both magnitude and time of occurrence. Although waves from this source are propagated through the earth similar to earthquake motions, blast-generated waves differ from earthquake-generated waves in that the former come from a compact or point source, while the latter come from an essentially wide spread source. Thus, decay caused by geometrical spreading of the waves would be larger than the internal damping which is of interest in earthquake response analyses. Near their source, ground motion characteristics from each of these causes are quite different; however, for waves traveling longer distances through the earth, the differences become almost indistinguishable (Newmark, 1967b).

At a predetermined distance from a nuclear explosion, a soil deposit suitably instrumented with a number of accelerometers, each located at various depths in the soil, could conceivably produce data from which in situ values of damping could be computed with energy dissipation characteristics and strain levels closely resembling values that would be obtained from an earthquake at some distance. To date, such tests have not been

conducted, largely because of their cost.

A third procedure for the in situ evaluation of damping involves the use of forced microtremors usually from vibration generators or other local, smaller energy resources. This field vibration procedure has been used in research studies of the dynamic behavior of several dams (Martin and Seed, 1966; Keightley, 1966). The test consists of varying the frequency of ground surface vibrators at low values (less than two cycles per second) and measuring the resulting accelerations at different depths in the soil. The viscous damping ratio in the first few modes is then determined at each depth for a linear single-degree-of-freedom system from the following equation (Martin and Seed, 1966):

$$\lambda = \frac{\Delta f}{2f_n} \dots \dots \dots (4.12)$$

where Δf is the width of the resonant peak (in frequency units) at an amplitude of .707 times the resonant amplitude, and f_n is the resonant frequency.

The values determined by this method cannot be used directly in soil problems resulting from strong-motion earthquakes for two reasons:

- 1) Shear strains and accelerations during an earthquake are considerably greater than during the forced vibration tests. Strain in these tests at the fundamental frequency is only of the order of 5×10^{-5} percent, corresponding to a maximum acceleration of approximately 10^{-3} g (Martin and Seed, 1966).
- 2) Energy dissipation from the vibrators occurs from geometrical damping as well as by internal damping. Unfortunately it is not possible to distinguish that dissipation of energy occurring only from internal damping (the predominant energy decay during earthquakes).

As an alternate to large expensive field tests at small strains, design engineers presently prefer to rely on damping values determined in the laboratory, where strains of significant amplitude can be readily executed on small scale samples of soil, and the results adjusted to a level of strain consistent with the strain range of earthquakes (see Fig. 4.4). A compilation of data obtained by others, as given in Chapter 5, is a useful guide for this purpose. This approach is used in the solution of many practical problems, and may prove to continue to be an acceptable source for this information; particularly, since in some cases variations in damping do not influence response calculations significantly. However, there are other cases in which the influence may be significant. Laboratory tests for this purpose are discussed in the next section.

4.3.2 Laboratory Tests

With a few exceptions or minor test modifications, the same general procedure used to determine shear modulus can also be used to determine internal damping. The only general method of the three, noted in Table 4.2, which cannot be used to determine damping, employs forced vibrations. The only test in this category is the resonant column test. This test can, however, be altered and by using free vibrations damping can be determined.

The basic procedure using the repeated loading method is the same as used to measure shear modulus, ie, it consists of applying forced cyclic stresses to a soil specimen and measuring the hysteretic stress-strain properties of the soil. The damping is then computed by measuring the area inside the hysteresis loop and using equation 4.1. As noted in Table 4.2, the cyclic triaxial test, the simple shear test and the torsional shear test fall into this category. Since the procedures to obtain the stress strain hysteretic characteristics and the limitations of the test procedures are the same as discussed for determining the shear modulus, reference is made to a previous section (Section 4.2.2).

An alternate procedure utilizes free vibration in the

same manner as used to determine the shear modulus. This procedure consists of applying initially a steady-state forced vibration, and then shutting off the driving force, or applying a static force, and then releasing it, and in both cases allowing the response of the sample to decay with time under free vibrating conditions. The damping ratio is then computed, using the following relationship:

$$\lambda = \frac{1}{2\pi} \ln \frac{A_n}{A_{n+1}} \dots \dots \dots (4.13)$$

where λ = initial total damping, %
 A_n = the single amplitude ordinate of the nth cycle
 A_{n+1} = the single amplitude ordinate of the n+1th cycle

A similar relationship to obtain the average damping for a number of cycles is described in Richart, Hall and Woods, 1970.

Those test methods which can be used, include either the resonant column test or shake table tests. In the resonant column test, the sample is first set into a steady-state forced torsional vibration, then the power is shut off and the resulting decay with time is recorded on an oscilloscope. Typical free vibration-decay records from this test, using a sample of Ottawa Sand, are shown in Fig. 4.11.

Utilizing a shake table to place large specimens of soil in free vibration is another method for obtaining damping in the laboratory. This procedure is, however, very expensive and generally not practical for use in most engineering problems. For this reason, very few data have been obtained from this source. One of these methods (using a shake table) consists of shaking a large bed of soil, then shutting off the power and measuring the time history of decay of acceleration (Kovacs, Seed and Chan, 1971; Arai and Umehara, 1966). Damping is then computed using equation 4.13. This method produces damping values for shear strains ranging from as low as 0.005 percent to as high as about 0.05 percent. Other conditions make the practicality and usefulness of this procedure for obtaining data limited, as discussed in Section 4.2.2.

Fig. 4.11 AMPLITUDE - TIME DECAY RECORD
FROM RESONANT COLUMN TEST

(From Hall, 1962)

A similar large-scale test for clays consists of applying a lateral force to a 12-inch square by 6-inch high block of soil, then releasing it suddenly and recording the rate of decay of the displacement. Damping is again computed using equation 4.13. This method was introduced by Kovacs, Seed and Chan, 1971, in research studies. This test, its limitations and applicability are also discussed in Section 4.2.2.

Each of these methods appears to provide reasonably consistent values for internal damping of soil, as discussed in a subsequent section, if corresponding confining pressures and levels of strains are consistent. In a comparison of simple shear tests with a soil block subjected to free vibration, Kovacs, Seed and Chan, 1971, found that slightly lower but consistent values of damping were obtained with the latter method than with the simple shear tests. The difference in the results, although small, cannot yet be explained.

As a point of interest, the torsional shear test, employing free vibration and noted in Table 4.2 (Zeevaert, 1967), cannot yet be used to measure damping of the soil. The measured strain amplitude change, or resulting decay, in this test comes from the entire loading system, not only the soil, because of undesirable frictional forces induced in the instrument during vibration.

4.4 POISSON'S RATIO AND BULK MODULUS

In the solution of two-dimensional finite element models, either Poisson's ratio or the bulk modulus must be defined along with the two dynamic soil properties previously described. Ideally, it may be assumed that the best approach would require that these properties likewise should be determined under dynamic loading conditions. Considering the limitations of geophysical tests (i.e., low stress level, influence of water table, etc.), for practical purposes there are no direct field procedures to measure these two properties. Furthermore, except for sonic or vibration tests (again at low stress levels), laboratory tests to measure

these properties are normally performed only under static loading conditions, because it is impractical to measure the axial and lateral strain for Poisson's ratio and volumetric strain for bulk modulus under dynamic load conditions. The following sections therefore describe the laboratory tests available to determine these properties under static load conditions, which, in practice, are considered satisfactory for dynamic analysis.

4.4.1 Poisson's Ratio

The laboratory test methods which may be used to measure Poisson's ratio, if it becomes desirable, are the triaxial test, soil box test, oedometer test, and sonic test techniques. From these various tests, typical values of Poisson's ratio obtained at low stress levels, and also at stress levels approaching failure, are summarized in Table 4.3.

From the results of the triaxial tests presented herein, it is demonstrated that Poisson's ratio generally increases with strain to values of 0.5 for a near-failure condition. Other tests, shown in Table 4.3, appear to indicate that the value measured is largely influenced by the method used to obtain it. For this reason, it may be generally concluded that no reliable field or laboratory method of the direct measurement of Poisson's ratio has yet been developed. Thus, the present state-of-the-art for determining Poisson's ratio for use in engineering calculations is to estimate and treat it as a constant, since it has a relatively small effect upon the resulting prediction for most civil engineering projects (Lambe and Whitman, 1969). Whitman, 1970a, suggests that, "For fully saturated soils, Poisson's ratio may be taken as 0.5. For soils of low degree of saturation, 0.35 is a reasonable value. Intermediate values may be selected for intermediate cases."

4.4.2 Bulk Modulus

Bulk modulus is a soil property not normally measured in usual foundation problems. Most practicing engineers prefer to avoid its determination altogether by estimating a value for

TABLE 4.3 POISSON'S RATIO AS DETERMINED BY
TESTING ALL TYPES OF SOILS

<u>Test</u>	<u>Loading Condition</u>		<u>Reference</u>
	Early Loading	Final Loading or Near Failure	
1) Triaxial Test			
Static Loading	0.1 to 0.2	0.5	Lambe & Whitman, 1969
Cyclic Loading	0.3 to 0.4		Lambe & Whitman, 1969
2) Soil Box Test			
(Zero Lateral Strain Test)	0.5	0.3	Ko & Scott, 1967a
3) Oedometer Test			
(Zero Lateral Strain Test)	0.30 to 0.35		Domaschuk & Wade, 1969
4) Sonic Tests	0.42 to 0.47 0.42		Domaschuk & Wade, 1969 Martin & Seed, 1966

Poisson's ratio instead. Typical values of Poisson's ratio to use as constants in dynamic analyses are described in the previous section.

Since the bulk modulus could feasibly be used as a substitute for Poisson's ratio for finite element problems, the following paragraphs are devoted to a brief description of its determination. For soils located below the water table (saturated), undrained modulus values are required. Undrained saturated specimens in hydrostatic compression exhibit the stress deformation behavior of the pore water, since water is virtually incompressible. As a result of this condition, bulk modulus values approaching that of water (i.e., approximately 320,000 psi) should be used in analyses. Values of this order of magnitude would be obtained also in the laboratory if it could be accurately measured. However, because the resulting volume changes of a virtually incompressible material are not measurable for practical purposes, no accurate method for measuring bulk modulus of soil under undrained conditions exists.

For the determination of the bulk modulus of soils located above the water table, four test methods are available. These tests are shown schematically in Fig. 4.12. In these tests, a static type of loading and unloading is used, since volume changes are extremely small and difficult to measure readily. Of these four methods, the triaxial compression test is probably the most readily available equipment for applying hydrostatic compression to the soil. Using this apparatus, one of three simple procedures can be used. One procedure consists of increasing the cell pressure in increments and measuring under drained conditions the resulting volume of water squeezed from the inside of the sample.

The second procedure consists of increasing the cell pressure in increments and measuring the total volume of fluid pumped into the cell. By knowing the calibration of the apparatus, the volume change of the unsaturated soil can be determined. One of the major shortcomings of this test is the fact that the total volume change is small, and that a large portion of this volume

a. TRIAXIAL TEST

(From Lambe and Whitman, 1969)

b. OEDOMETER TEST

(From Lambe and Whitman, 1969)

c. SOIL BOX TEST

(From Ko and Scott, 1967 c)

d. MIT CHAMBER DEVICE

(From Whitman, 1962 a)

Fig. 4.12 LABORATORY TESTS TO MEASURE BULK MODULUS

change constitutes deformations in the apparatus. Of the two procedures, the latter method is more versatile; however, it requires additional effort to calibrate the apparatus satisfactorily.

The third procedure is to hold both the cell pressure and the pore pressure constant, and measure the volume change of a saturated specimen, which results from an increase in the vertical stress only. The resulting volume change is multiplied by three to compute a bulk modulus. No corrections are necessary but the method is applicable only to elastic materials at low principal stress ratios.

The oedometer, a one dimensional compression test, has been used extensively for measuring the constrained modulus which, theoretically, is numerically equal to the bulk modulus for a Poisson's ratio of 0.5. In this test, developed primarily for use in the study of consolidation, stress is applied to the soil in the vertical direction, while lateral strain is prevented. Since volume change occurs in only the vertical direction, axial strain is equal to volumetric strain. The major advantage of this test is its simplicity to perform. The major shortcomings of this test involve experimental errors, the major one being side friction, i.e., stresses between the surface of the sample and the oedometer ring. Side friction not only introduces non-uniform stress patterns and shear stresses which lead to larger volumetric strains, but also lower actual vertical stresses. All of these factors produce a much lower bulk modulus than would normally be reported.

The two other test devices available to evaluate this property, the soil box test and the MIT Chamber Device, are not extensively used, nor are they devices that are familiar to commercial testing laboratories. In the soil box test (Ko and Scott, 1967c), a soil cube is compressed isotropically by injecting fluid between the membranes and the rigid metal walls surrounding the sample. The amount of fluid injected provides a measure of the resulting volume change, while the pressure indicates the volumetric stress.

The MIT Chamber Device test, for obtaining the bulk modulus, is performed in much the same way. This device, developed at MIT in 1961 (Whitman, 1962a) comprises a procedure which consists of coating a block sample of soil with a rubber skin (hardened liquid latex skin), placing it in a relatively rigid container and pressurizing an oil fluid surrounding the sample. The volume change of oil pumped into the chamber is recorded as the pressure is slowly applied to the fluid. By suitably calibrating the apparatus, the volume change of the soil can be determined at each stress level and the bulk modulus defined. Both this test and the soil box test have the same major shortcomings as the triaxial test (second procedure).

Typical values of bulk modulus as a function of stress, for hydrostatic compression tests performed on dry Ottawa sand at three different densities (Ko and Scott, 1967a) are summarized in Fig. 4.13. Although no data for cohesive soils has been developed to our knowledge, similar relationships to sands would also be expected for clay soils. The apparent lack of bulk modulus data and reliable methods for their determination verifies that this property is relatively unimportant for engineering design applications.

4.5 CONCLUSIONS REGARDING UTILIZATION AND ADEQUACY OF TEST PROCEDURES

In the previous sections of Chapter 4, existing tests or methods used to determine the dynamic shear modulus, damping characteristics, Poisson's ratio and bulk modulus have been described. Also included for shear modulus and damping have been a general description of the limitations of each test, its applicability, the availability of test equipment and, in some cases, the general cost of performing such a test. A number of these tests, for various reasons, are more suited for determining the dynamic soil properties for use in response calculations than other tests. The following two sections are included to briefly identify those methods now in practical usage and to point out those areas in

Fig. 4.13 HYDROSTATIC COMPRESSION TESTS ON SAND

(From Ko and Scott, 1967a)

which data are lacking.

4.5.1 Shear Modulus

The shear modulus for use in response calculations is presently determined in many cases at low strain levels by measuring the velocity in the field of a low energy seismic shear wave, propagated through the soil. Reduction factors are then applied to the measured modulus to obtain values consistent with the larger strain levels of earthquake magnitude. For the evaluation of nuclear power plant sites, geophysical methods employing in-hole techniques are presently most utilized for these in situ measurements; while in the laboratory, the resonant column test and the cyclic triaxial test are undoubtedly the predominant methods. The in-hole field geophysical methods are the most applicable field procedures for the following reasons:

- 1) They involve a large mass of soil in the determination of the shear modulus in situ.
- 2) The cost of performing the tests is reasonable.
- 3) The equipment is more available than for the other tests.
- 4) Shear modulus determinations may be made as a function of depth, allowing this property to be defined for each major soil deposit.
- 5) They can be performed on most types of soil.

Field values determined from in-hole geophysical tests are usually supplemented and compared with values determined from other tests, largely from resonant column tests and cyclic triaxial tests, with consideration given to the difference in strain levels of each method. Since geophysical field tests and the laboratory resonant column test are indicative of moduli at low strain levels, while the cyclic triaxial test establishes values at high strain levels, it is possible by extrapolation to determine moduli values at intermediate strain levels corresponding to values encountered during actual earthquakes. It is these intermediate or extrapolated values of shear modulus that are normally of interest in the response calculations.

The two aforementioned laboratory tests are the most commonly used procedures likewise for reasons 2), 4) and 5) for the resonant column test, and reasons 2), 3), 4) and 5) for the cyclic triaxial test. Several other reasons for their predominance are listed as follows:

- 1) Modulus determinations from the resonant column test have been found in many cases to closely agree with values obtained in situ, using geophysical methods at comparable levels of strain (especially for sandy soils).
- 2) Relatively undisturbed specimens in a solid cylindrical shape can be readily obtained and tested in many cases.
- 3) A number of measurements can be performed on different specimens at a minimal cost to verify the uniformity of the soils throughout the site, since field in situ procedures can, for economic reasons, be made only at a few select locations.

The most serious limitation of these field and laboratory procedures lies in the fact that they are either performed at energy levels higher or lower than most earthquakes, requiring the use of extrapolation procedures to adjust the modulus to the strains of interest for evaluating ground response under earthquake loading. If a suitable in situ test or procedure could be developed to vibrate an element or zone of soil in the same manner and at strains approaching earthquake magnitudes, this test could greatly improve existing methods for the in situ determination of the shear modulus. Even if this in situ test were to prove much more costly than existing techniques, its use in a few cases, supplemented also by field instrumentation during actual earthquakes, would serve to verify the validity of presently used extrapolation procedures. This verification would also create more confidence in existing laboratory methods and thereby reduce some of the conservatism presently associated with their use.

In the laboratory, as noted in Fig. 4.4, other tests including the cyclic simple shear test, shake table tests, and

torsional shear tests operate at least partially within strains of earthquake magnitude. Of these tests, the cyclic simple shear test (small scale) seems to offer the greatest promise for providing accurate data with practical test procedures. Its present limitations, which include its lack of availability and familiarity, and difficulties of preparing undisturbed specimens, in our opinion can be overcome as the test becomes more standardized. The simple shear test, combined with the presently used resonant column test, should produce sufficient shear moduli data for design, at appropriate strains of earthquake magnitude.

4.5.2 Damping

The soil damping properties for use in response calculations are presently determined using laboratory techniques, as there is as yet no test for determining usable data in situ. The resonant column and cyclic triaxial tests are presently the most commonly used of the various techniques described. Using these two tests and extrapolation procedures, similar to those described for shear modulus in the previous section, the appropriate damping values for the intermediate strains of earthquake magnitude are obtained.

In addition to these two tests, the cyclic simple shear test (small scale), although not presently utilized to any great extent, shows the greatest promise for future practical development in the determination of damping for earthquake engineering problems. Further, this test, performed at comparable strains with the cyclic triaxial test (Fig. 4.4), has the distinct advantage over the triaxial test in that it more nearly duplicates earthquake loading (simple shear) conditions. This test, together with the resonant column test, may provide a measure of damping for those strain ranges of interest (Fig. 4.4).

Since no field tests are readily available to measure damping, the correlation between in situ and laboratory values is the largest area in which data are lacking. At present, apparently the only meaningful ways in which field values of damping at strain

levels of earthquake magnitude may be obtained for comparison with laboratory values is by recording, at different depths, the strong ground motions created from actual earthquakes, or by explosions at an underground nuclear test site.

In the past, a number of ground motion records have been obtained during strong motion earthquakes. Unfortunately, the data collected in most areas were usually recorded at one point in the soil (at the ground surface) and not at two or more locations to permit a computation of material characteristics. Occasionally, where more than one record was present, either the soil types and their dynamic properties were not accurately known, or bedrock motions were measured at a rock outcrop, requiring that these same motions be adjusted to equivalent motions under a considerable thickness of overburden. In each case, assumptions usually must be made concerning rock motions in order to define both dynamic soil properties and corresponding ground motions as a function of depth. A suitable program set up to instrument both earthquake and soil characteristics at one or two selected sites, using underground nuclear explosions as the energy source, could help considerably to define field values of damping, should this property be found to significantly influence the analytical response calculations.

The results of calculations reported by SW-AJA, 1971, indicate that damping (and modulus) properties of soils play an important role in determining the amplification characteristics of a soil deposit. However, the dependence of damping (and modulus) on strain amplitudes suggest that soils respond as self-compensating systems which may be more tolerant of inaccuracies in defining damping (and modulus) values than simple parametric studies may indicate.

4.5.3 Poisson's Ratio and Bulk Modulus

As described previously, the present state-of-the-art for the determination of these properties for response calculations

is to estimate Poisson's ratio as being somewhere between 0.35 and 0.5, and to avoid the use of bulk modulus. This approach is justified since practical solutions to engineering problems are relatively insensitive to variations in Poisson's ratio. For this reason, it is our opinion that additional research effort in this area to more closely define these properties under either static or dynamic loading conditions is not warranted.

CHAPTER 5

DYNAMIC PROPERTIES OF SOILS

Considerable data have become available in recent years from which to establish values of dynamic properties for various types of soils. In cases of free-field ground response under seismic, or earthquake, loading conditions involving no residual soil displacement, the response is governed primarily by the dynamic shear moduli and damping characteristics of the soils. Most of the data generated from either engineering projects or research efforts concerned with cyclic loading conditions have been for sands and saturated clays of various strengths and stiffnesses, although limited data are also available for gravelly soils and peats. Because silty soils are rarely encountered in a pure form, but appear very commonly with large amounts of either sands or clays, they have been grouped with one or the other soil types generally according to their plasticity and/or gradation characteristics.

The methods used to determine the dynamic shear moduli and damping ratios for soils have been described in the previous chapters. These methods generally include field geophysical tests, together with forced and free vibration tests and repeated loading tests in the laboratory. In addition to these methods, modulus and damping values have been determined in a few cases from the results of ground response analyses from actual earthquake data. In these cases, where motions have been determined at different depths in a soil profile during an earthquake, computations have been made to determine the corresponding values of the two primary dynamic properties controlling the response of the deposit.

The following sections summarize available data on the dynamic shear moduli and damping factors for soils under symmetrical cyclic loading conditions similar to those illustrated in Fig. 4.2, and present the results in a form which will provide a useful guide in the selection of soil characteristics for analysis purposes. All of these data have been compiled under subcontract by the

University of California at Berkeley. The principal investigators in that subcontract were Dr. H. Bolton Seed and Dr. I. M. Idriss, who presented the data in a report entitled, "Soil Moduli and Damping Factors for Dynamic Response Analyses." The following Sections 5.1 through 5.8 of this chapter are quoted directly from that report.

5.1 PREVIOUS STUDY BY HARDIN AND DRNEVICH

A comprehensive survey of the factors affecting the shear moduli and damping factors of soils and expressions for determining these properties have recently been presented by Hardin and Drnevich, 1970a and b. In this study it was suggested that the primary factors affecting moduli and damping factors are:

Strain amplitude, γ

Effective mean principal stress, σ'_m

Void ratio, e

Number of cycles of loading, N

Degree of saturation for cohesive soils, S

and that less important factors included:

Octahedral shear stress

Overconsolidation ratio, OCR

Effective stress strength parameters, c' and ϕ'

Time effects

Relationships were presented to determine the values of maximum shear modulus (at essentially zero strain) and the variations of modulus values with strain for all soils. The expression for evaluating the maximum shear modulus is:

$$G_{\max} = 14760 \times \frac{(2.973 - e)^2}{1 + e} (\text{OCR})^a (\sigma'_m)^{1/2} \dots (5.1)$$

where $G_{\max}$ = maximum shear modulus in psf,

e = void ratio

OCR = overconsolidation ratio

a = a parameter that depends on the plasticity index (PI) of the soil, and

σ'_m = mean principal effective stress in psf.

The value of a can be obtained from the following table:

PI	a
0	0
20	0.18
40	0.30
60	0.41
80	0.48
≥100	0.50

The modulus value, G , at a strain level, γ , is then evaluated from the relationship:

$$G = \frac{G_{\max}}{1 + \gamma/\gamma_r} \dots \dots \dots (5.2)$$

where

$$\gamma_r = \frac{\tau_{\max}}{G_{\max}} \dots \dots \dots (5.3a)$$

$$\tau_{\max} = \left\{ \left(\frac{1 + K_o}{2} \sigma_v' \sin \phi' + c' \cos \phi' \right)^2 - \left(\frac{1 - K_o}{2} \sigma_v' \right)^2 \right\}^{1/2} \dots (5.3b)$$

K_o = coefficient of lateral stress

σ_v' = vertical effective stress, and

c', ϕ' = static strength parameters in terms of effective stress

Similar relationships were also presented for evaluating the damping ratio. The damping ratio, λ , at a strain level, γ , is given by:

$$\lambda = \frac{\lambda_{\max} \cdot \gamma/\gamma_r}{1 + \gamma/\gamma_r} \dots \dots \dots (5.4)$$

where $\lambda_{\max}$ is the maximum damping ratio corresponding to very large strains. For clean sands, $\lambda_{\max}$ (in percent) is evaluated by:

$$\lambda_{\max} = D - 1.5 \log_{10} N \dots \dots \dots (5.5a)$$

where $D = 33$ percent for clean dry sands or $D = 28$ percent for clean saturated sands and N = number of cycles. For saturated cohesive soils, $\lambda_{\max}$ is given by

$$\lambda_{\max} = 31 - (3 + 0.03f) (\sigma_m')^{1/2} + 1.5 f^{1/2} - 1.5 \log N \dots (5.5b)$$

where f = frequency of applied cyclic load in cycles per second,

and

σ'_m = mean principal effective stress in kg/cm^2 .

The significance of the factors involved in these relationships is discussed in the following section.

5.2 SHEAR MODULUS VALUES FOR SANDS

All investigations have shown that modulus values for sands are strongly influenced by the confining pressure, the strain amplitude and the void ratio (or relative density) but not significantly by variations in grain size characteristics. It has been found that in general, the shear modulus and confining pressure are related by the equation:

$$G = 1000 K_2 (\sigma'_m)^{1/2} \dots \dots \dots (5.6)$$

so that the influence of void ratio and strain amplitude can be expressed through their influence on the parameter K_2 .

The influence of other factors on K_2 , may be illustrated by the results in Fig. 5.1 which were computed using the relationships suggested by Hardin and Drnevich. Plots are presented to show the influence of ϕ' , effective vertical stress (σ'_v), K_0 , and void ratio on the computed relationships between K_2 and strain amplitude. It may be seen that:

- 1) At very low strains ($\gamma \leq 10^{-3}$ percent), K_2 depends only on the void ratio, e .
- 2) At intermediate strains ($10^{-3} < \gamma < 10^{-1}$ percent) the variation of K_2 with strain is only slightly influenced by the vertical stress, and very slightly by variations in ϕ' and K_0 . The values of K_2 are still influenced strongly by the void ratio however.
- 3) At very high strains ($\gamma > 10^{-1}$ percent), the values of K_2 are slightly influenced by the vertical stress but they are essentially independent of K_0 , ϕ' and e .

Thus for practical purposes, values of K_2 may be considered to be determined mainly by the void ratio or relative density and the

Fig. 5.1 INFLUENCE OF VARIOUS FACTORS ON THE SHEAR MODULI OF SANDS.
(based on Hardin and Drnevich expressions)

(From Seed and Idriss, 1970 c)

strain amplitude of the motions.

A number of investigators, using different laboratory testing procedures, have presented data on the relationships between these factors. The test conditions used in these investigations are summarized in Table 5.1 and the results are presented in Fig. 5.2, for samples having a relative density of about 75 percent, and in Fig. 5.3, for samples having a relative density of about 40 percent. Average relationships between K_2 and strain for these two relative density conditions are shown in Figs. 5.2 and 5.3, and they are compared in Fig. 5.4. Values of K_2 at other relative densities can be estimated by interpolation as shown in Fig. 5.4.

It may be seen that for relatively dense samples, the values of K_2 determined at very low strains for laboratory test specimens are typically in the range of 50 to 75. The results of a number of determinations of shear moduli for sands at very low strain levels by means of in situ shear wave velocity measurements are summarized in Table 5.2; the six investigations for dense to extremely dense sands (excluding clayey and partly cemented sands) give values for K_2 ranging from 44 to 86. Thus there appears to be good general agreement between the results of laboratory and in situ investigations.

For purposes of comparison, representative values of the relationship between K_2 and strain at different void ratios determined by the Hardin-Drnevich relationship for an effective vertical stress of 3000 psf, $K_0 = 0.5$ and $\phi' = 36^\circ$ are plotted in Fig. 5.5.

The good agreement between the results in Figs. 5.4 and 5.5 indicates that reasonable values for the shear moduli of sands may be obtained either by use of the curves in Fig. 5.4 or by use of the Hardin-Drnevich equations. Where field data are obtained in terms of the standard penetration resistance, the data in Fig. 5.4 are likely to be more convenient but for other purposes, direct computation from equations 5.1, 5.2 and 5.3 may be desirable.

TABLE 5.1 SUMMARY OF LABORATORY INVESTIGATIONS OF SHEAR MODULI
AND DAMPING RATIOS FOR SANDY SOILS.

(From Seed and Idriss, 1970c)

Type of Test	Soil Tested	Range of Strain		Range of Confining Pressure	Reference
		Shear	Axial		
Forced Vibration: Longitudinal Vibration	Sand		$<5 \times 10^{-3}\%$	600 to 7200 psf	Richart, Hall and Lysmer (1962)
	"		"	450 to 7500	Hardin and Richart (1963)
	"		"	600 to 7400	Hall and Richart (1963b)
	"		"	600 to 7200	Hardin (1965)
	Sand, silty sand and clayey sand		"	1000 to 3500	Donovan (1968, 1969)
Forced Vibration: Torsional Vibrations Solid Sample	Sand	$<10^{-2}\%$		600 to 7200	Richart, Hall and Lysmer (1962)
	"	"		600 to 7400	Hall and Richart (1963b)
	"	"		450 to 7500	Hardin and Richart (1963)
	"	"		600 to 7200	Hardin (1965)
	"	"		600 to 8500	Drnevich, Hall and Richart (1966)
Forced Vibration Torsional Vibration Hollow Sample	Sand "	10^{-3} to $6 \times 10^{-2}\%$ "		600 to 8500 500 to 1800	Drnevich, Hall and Richart (1966) Hardin and Drnevich (1970)
Free Vibration: Cylindrical Sample	Sand		10^{-3} to $10^{-4}\%$	400 to 6400	Kishida and Takano (1970)
Triaxial Compression	Sand and gravel, silt and sand		2×10^{-3} to $5 \times 10^{-3}\%$	400 to 1800	Weissman and Hart (1961)
	Sand, silty sand and clayey sand		5×10^{-3} to 0.1%	1000 to 3500	Donovan (1968, 1969)
	Sand		10^{-1} to 1%	3000 to 3400	Matsushita, Kishida and Kyo (1967)
Simple Shear	Sand "	3×10^{-2} to 0.5% 10^{-2} to 0.5%		2000 500 to 4000	Seed (1968a) Silver and Seed (1969)

Fig. 5.2 SHEAR MODULI OF SANDS AT RELATIVE DENSITY OF ABOUT 75 %.

(From Seed and Idriss, 1970 c)

Fig. 5.3 SHEAR MODULI OF SANDS AT RELATIVE DENSITY OF ABOUT 40%.

(From Seed and Idriss, 1970c)

Fig. 5.4 SHEAR MODULI OF SANDS AT DIFFERENT RELATIVE DENSITIES.

(From Seed and Idriss, 1970 c)

TABLE 5.2 SHEAR MODULI* OF SANDS BASED ON IN SITU
SHEAR WAVE VELOCITY MEASUREMENTS

(From Seed and Idriss, 1970c)

<u>Soil</u>	<u>Location</u>	<u>Depth Ft.</u>	<u>K₂</u>
Loose moist sand	Minnesota	10	34
Dense dry sand	Washington	10	44
Dense saturated sand	So. California	50	58
Dense saturated sand	Georgia	200	60
Dense saturated silty sand	Georgia	60	65
Dense saturated sand	So. California	300	72
Extremely dense silty sand	So. California	125	86
Dense dry sand (slightly cemented)	Washington	65	166
Moist clayey sand	Georgia	30	119

*Shear modulus, $G = 1000 K_2 (\sigma'_m)^{1/2}$ psf

Fig.5.5 SHEAR MODULI OF SANDS AT DIFFERENT VOID RATIOS.
 (based on Hardin - Drnevich expressions)

(From Seed and Idriss, 1970 c)

It may be noted that if each of the relationships shown in Figs. 5.4 and 5.5 is replotted to show the variation with shear strain of the ratio of shear modulus at strain γ to shear modulus at a shear strain of 10^{-4} percent, the results fall within the relatively narrow band shown in Fig. 5.6. Thus a close approximation to the modulus vs. shear strain relationship for any sand can be obtained by determining the modulus at a very low strain level, say by wave propagation methods in the field, and then reducing this value for other strain levels in accordance with the results indicated by the average (dashed) line in Fig. 5.6.

5.3 DAMPING RATIOS FOR SANDS

From their study of factors influencing the damping ratios of sands, Hardin and Drnevich concluded that shear strain, effective mean principal stress (or σ_v' and K_0), void ratio and number of cycles were very important, while octahedral shear stress, angle of friction and degree of saturation had lesser effects. As in the case of moduli, the effects of variations in grain size characteristics were considered to be relatively insignificant.

Computations of the effects of the above factors on the relationship between damping ratio and shear strain amplitude, as determined by the Hardin-Drnevich relationships are shown in Fig. 5.7. It is apparent that the effects of ϕ' , K_0 , void ratio and degree of saturation are relatively minor, and it can readily be seen from the equation for maximum damping ratio

$$\lambda_{\max} = 30 - 1.5 \log_{10} N \dots \dots \dots (5.7)$$

that if values of λ are determined for about $N = 5$ cycles, values for other numbers of cycles in the range of interest (say 5 to 30) will not be significantly different.

Thus the main factor affecting the relationship between damping ratio and shear strain is the vertical confining pressure σ_v' . The influence of this factor, as determined by two studies is shown in Fig. 5.8. For pressures less than about 500 psf, the effect of pressure changes may be significant but excluding these very low pressures, which represent conditions in the top few feet

Fig. 5.6 VARIATION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SANDS.

(From Seed and Idriss, 1970 c)

Fig. 5.7 INFLUENCE OF VARIOUS FACTORS ON THE DAMPING RATIOS FOR SANDS.
 (based on Hardin and Drnevich expressions)

(From Seed and Idriss, 1970 c)

Fig. 5.8a INFLUENCE OF CONFINING PRESSURE ON DAMPING RATIO OF SATURATED SAND.
(based on Hardin and Drnevich expressions)

Fig. 5.8b INFLUENCE OF CONFINING PRESSURE ON DAMPING RATIO OF DRY SAND.

of soils, the effect of variations in pressure is very small compared with the effect of shear strain, and an average damping ratio vs shear strain relationship determined for an effective vertical stress of 2000 to 3000 psf would appear to be adequate for many practical purposes. Considering the potential scatter of test data for damping ratios, even those obtained by the same investigator using the same test procedure, the adoption of such an average relationship may be even more justified.

A list of previous investigations of damping ratios of sands is presented in Table 5.1, and the results of these studies are summarized in Fig. 5.9. Approximate upper and lower bound relationships are shown by dashed lines and a representative average relationship for all of the test data is shown by the solid line. This average relationship is likely to provide values of damping ratio with sufficient accuracy for many practical purposes.

The curves in Fig. 5.9 also provide a basis for evaluating the relationship between damping ratio and strain for particular sands for which limited test data is available. If the value of damping ratio at a strain level of 0.1 to 0.5 percent is determined, the probable damping ratios at other strains can be approximated by drawing a line through the known data point parallel to the curves shown in Fig. 5.9.

5.4 SHEAR MODULUS VALUES FOR SATURATED CLAYS

Accurate determination of the shear moduli of saturated clays is enormously complicated by the large effects of strain amplitude and sample disturbance on modulus values. In situ measurements eliminate the problems raised by sample disturbance but to date no techniques have been developed for inducing large controlled strain amplitudes in natural deposits and thus moduli can only be determined at very small strain levels. In the laboratory, on the other hand, samples may be tested under a wide range of strains but for test specimens from natural deposits, the moduli determined will inevitably be influenced by the effects of sample disturbance.

Fig. 5.9 DAMPING RATIOS FOR SANDS.

(From Seed and Idriss, 1970 c)

The joint influence of these effects is illustrated by the data presented in Figs. 5.10 and 5.11. Fig. 5.10 shows values of shear moduli for San Francisco Bay mud at a depth of about 25 ft. determined by in situ shear wave velocity measurements by Aisiks and Tarshansky, 1968, and values determined by cyclic loading simple shear tests on undisturbed samples by Thiers. Projecting the laboratory test data to the strain level corresponding to the field test conditions, it may be seen that the laboratory test values are only about 40 percent of those for the in situ clay. This result is not surprising in the light of previous studies of the influence of disturbance on the moduli of natural clays (Ladd, 1964) and it emphasizes the magnitude of the correction which may have to be made for this effect.

The influence of strain amplitude on shear modulus is also apparent from the data in Fig. 5.10, the values at strains of about 0.5 percent being only about 12 percent of those corresponding to strains of the order of 10^{-3} or 10^{-4} percent.

Fig. 5.11 shows similar data for Union Bay clay. In this case values of shear moduli for in situ conditions were determined from seismic wave velocity measurements and from observations of the response of the clay during an earthquake; modulus values for undisturbed samples were determined by resonant frequency tests and cyclic loading tests in the laboratory. Again the in situ moduli are two or three times greater than the laboratory test values at comparable strains, and the modulus decreases enormously with increasing strain amplitude.

In addition to the effects of strain amplitude and disturbance, the shear moduli of different clays will clearly depend on their relative strengths and stiffnesses. Hardin and Drnevich express these effects in terms of the effective mean principal stress, void ratio, overconsolidation ratio and effective stress strength parameters, but the resulting relationships do not always provide reasonable evaluations of shear moduli for in situ conditions, as evidenced by the results shown in Figs. 5.10 and 5.11.

However, in view of the facts that (1) stiffness increases

Fig.5.10 SHEAR MODULUS DETERMINATIONS FOR SAN FRANCISCO BAY MUD.

Fig.5.11 SHEAR MODULUS DETERMINATIONS FOR UNION BAY CLAY AT DEPTH OF ABOUT 80 FT.

in general with soil strength and (2) for static load conditions, the ratio E/s_u for saturated clays does not vary widely from one soil to another, it seems reasonable to expect that variations in clay characteristics might be taken into account with a reasonable degree of accuracy by normalizing the shear modulus, G , with respect to the undrained shear strength, s_u , and expressing the relationship G/s_u as a function of shear strain.

Test data obtained by a number of investigators and expressed in this form are summarized in Table 5.3 and plotted in Fig. 5.12. For test data obtained in laboratory tests under unconsolidated-undrained test conditions, the measured moduli were multiplied by a factor of 2.5 to make an approximate allowance for sample disturbance. Clearly the effects of disturbance will vary from one study to another but in the absence of detailed information on sampling and testing conditions it was considered that a factor of 2.5 would represent a reasonable average correction factor for these effects. For in situ and laboratory consolidated-undrained test conditions, no correction was applied to the test results.

While there is considerable scatter in the data, most of the test results fall within the dashed lines in Fig. 5.12, that is, within $\pm 50\%$ of the average values shown by the solid line in the figure. Thus the average values are likely to provide reasonable estimates of the in situ moduli for many clays.

Alternatively the data in Fig. 5.12 might be used to assess the influence of strain amplitude on the shear modulus of natural clays, by expressing the ordinates in terms of the ratio of shear modulus at shear strain γ to shear modulus at a shear strain of 3×10^{-4} percent. This ratio for the average values shown in Fig. 5.12, is plotted as a function of shear strain in Fig. 5.13. Reasonable estimates of the shear modulus of a clay at any strain amplitude can be obtained by determining the in situ value at strains of the order of 3×10^{-4} percent by means of shear wave velocity measurements and applying the reduction factors shown in Fig. 5.13 to determine values at other shear strains.

Fig.5.12 IN-SITU SHEAR MODULI FOR SATURATED CLAYS.

* Data added subsequent to the referenced publication

(From Seed and Idriss, 1970c)

Fig.5.13 TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SATURATED CLAYS.

(From Seed and Idriss, 1970 c)

5.5 DAMPING VALUES FOR SATURATED CLAYS

Test data for damping ratios for saturated clays are so limited and the results vary to such an extent that it is difficult to determine the main factors influencing the damping ratios of saturated clays. A list of previous investigations of damping ratios for saturated clays is presented in Table 5.3 and the results of these studies are summarized in Fig. 5.14. Approximate upper and lower bound relationships between damping ratio and shear strain are shown by the dashed lines and a representative average relationship for all of the test data is shown by the solid line. This average relationship may well provide values of damping ratio with sufficient accuracy for many practical purposes.

The curves in Fig. 5.14 also provide a basis for evaluating the relationship between damping ratio and strain for any particular clay. If the value of damping ratio at a strain level of 0.1 to 0.5 percent is determined, the probable damping ratios at other strains can be estimated by drawing a line through the known data point parallel to the curves shown in Fig. 5.14.

Further studies are required of the factors influencing the damping ratios of saturated clays to permit more detailed assessments of this characteristic for analysis purposes.

5.6 SHEAR MODULI AND DAMPING RATIOS FOR GRAVELLY SOILS

Probably because of the large diameter of test specimens required, there do not appear to have been any laboratory investigations of the shear moduli and damping ratios for gravelly soils. The results of a limited number of moduli determinations for these types of soils, based on in situ shear wave velocity measurements are summarized in Table 5.4, from which it may be seen that at small strain levels, modulus values are between 1.25 and 2.5 times greater than those for dense sands.

At higher strains, it seems likely that moduli for gravelly soils will decrease in a manner similar to that for sands. Thus by applying the moduli reduction factors shown in Fig. 5.6 to the data in Table 5.4 variations in shear moduli with strain might be

TABLE 5.3 SUMMARY OF LABORATORY INVESTIGATIONS OF SHEAR MODULI
AND DAMPING RATIOS FOR SATURATED CLAYS.

(From Seed and Idriss, 1970c)

Type of Test	Soil Tested	Range of Strain		Range of Shear Strength	Data Correction Factor*	Reference
		Shear Strain	Axial Strain			
Field shear wave velocity measurements	S.F. Bay mud	<10 ⁻³ %	<10 ⁻³ %	200 to 500 psf	1.0	Aisiks and Tarshansky (1968)
Field compression wave velocity measurements	Union Bay clay				1.0	Shannon and Wilson (1967)
Lab. Free Vibration Tests: Longitudinal Vibrations	Elkhorn Slough silty clay		3x10 ⁻² to 2%	300 to 1100 psf	2.5	Parmalee et al. (1964); Idriss (1966)
Lab. Free Vibration Tests: Shear Vibrations	S.F. Bay mud Kaolinite/Bentonite mixture	2x10 ⁻² to 0.5% 5x10 ⁻² to 2%		300 psf 44 to 85 psf	2.5	Kovacs (1968) Kovacs (1968)
					2.5	
Lab. Forced Vibration Tests: Longitudinal Vibrations	Cambridge clay Mississippi gravels		≈2.5x10 ⁻³ % ≈2.5x10 ⁻³ %	1080 psf 520 psf	2.5	Wilson and Dietrich (1960) Wilson and Dietrich (1960)
					2.5	
Lab. Forced Vibration Tests: Torsional Vibrations	Birch Bay clay Montana clay	≈2.5x10 ⁻³ % ≈2.5x10 ⁻³ %		1000 to 2420 psf 6000 psf	2.5	Wilson and Dietrich (1960) Wilson and Dietrich (1960)
					2.5	
Lab. Forced Vibration Tests: Torsional Vibrations (consol. samples)	Whidbey Bay clay	≈2.5x10 ⁻³ % 0.125% ≈2.5x10 ⁻³ %		230 to 1800 psf	1.5**	Wilson and Dietrich (1960)
	Silty clay			800 to 1500 psf	1.0	Zeevaert (1967)
	Edgar Plastic Kaolin			1400 to 1800 psf	1.0	Hardin and Black (1968)
Lab. Triaxial Comp. Tests	Ardmore clay		0.1 to 0.5%	-	-	Taylor and Menzies (1963)
	Ardmore clay		0.5 to 1%	-	-	Taylor and Hughes (1965)
	Union Bay clay		3x10 ⁻³ to 0.3%	200 to 880 psf	2.5	Shannon and Wilson (1967)
	Silty clay		10 ⁻² to 0.1%	-	-	Donovan (1969)
	Webb Mark IV clay		0.2 to 1%	-	-	Taylor and Bacchus (1969)
Lab. Torsional Shear Tests	Georgia Kaolinite	3x10 ⁻² to 0.2%		-	-	Krizek and Franklin (1967) Hardin and Drnevich (1970)
Lab. Simple Shear Tests	S.F. Bay mud	0.2 to 4%		300 to 400 psf	2.5	Thiers (1965), Thiers & Seed (1968a)
	Kaolinite/Bentonite mixture	0.1 to 2.5%		44 to 85 psf	2.5	Kovacs (1968)
	S.F. Bay mud	0.1 to 3%		300 psf	2.5	Kovacs (1968)

*Applied to modulus values to allow for sample disturbance.

**Sample disturbed slightly after consolidation.

TABLE 5.4 SHEAR MODULI* OF GRAVELLY SOILS BASED ON
IN SITU SHEAR WAVE VELOCITY MEASUREMENTS

(From Seed and Idriss, 1970c)

<u>Soil</u>	<u>Location</u>	<u>Depth Ft.</u>	<u>K₂</u>
Sand, gravel, and cobbles with little clay	Caracas	200	90
Dense sand and gravel	Washington	150	122
Sand, gravel and cobbles with little clay	Caracas	255	123
Dense sand and sandy gravel	So. California	175	188

*Shear modulus $G = 1000 K_2 (\sigma'_m)^{1/2}$ psf

Fig. 5.14 DAMPING RATIOS FOR SATURATED CLAYS.

* Data added subsequent to the referenced publication

(From Seed and Idriss, 1970c)

estimated as shown in Fig. 5.15. Additional data on modulus values and damping ratios for gravelly soils are badly needed; however, approximate values for use in some types of response analyses can be estimated by the procedure shown in Fig. 5.15 and the assumption can be made that damping is approximately the same as for sandy material.

5.7 SHEAR MODULI AND DAMPING RATIOS FOR PEATS

The results of several investigations which have provided data on the shear moduli and damping ratios for peats are summarized in Fig. 5.16. Shear moduli are shown as the ratio of G/s_u , with values determined by in situ seismic wave velocity measurements, evaluation of ground response to earthquake excitation, and laboratory investigations. As in the case of clays, moduli determined by laboratory unconsolidated-undrained tests were multiplied by a factor of 2.5 before being plotted in Fig. 5.16. While the nature of peaty soils is likely to vary considerably from one location to another, the data in Fig. 5.16 may provide some indication of the dynamic characteristics of this type of soil.

5.8 CONCLUSION

In the preceding pages, an attempt has been made to summarize in a convenient form, the available data concerning the shear moduli and damping ratios for soils. Clearly more data on these dynamic characteristics are required, particularly for silts, clays and gravelly soils. However it is hoped that the data presented will serve as a useful guide in the selection of soil properties for dynamic response analyses and that other engineers might be encouraged to make available any additional data which would supplement those presented above.

Fig. 5.15 MODULI DETERMINATIONS FOR GRAVELLY SOILS.

(From Seed and Idriss, 1970 c)

Fig. 5.16 MODULI AND DAMPING DETERMINATIONS FOR PEATS.

(From Seed and Idriss, 1970 c)

CHAPTER 6

LIQUEFACTION OF SOILS

The application of cyclic loads as produced by earthquakes, to relatively loose deposits of fine sands below the water table may result in partial or complete loss of supporting capacity of the deposit. If all strength is lost, at least temporarily, water flows upward to the surface creating sand boils, buried pile lines and tanks become bouyant and may float to the surface; and structures supported above or within the deposit undergo significant settlements and tilting. This phenomenon is referred to as "liquefaction" and is obviously a condition to be avoided in any type of major construction.

Relatively dense deposits of sand require more repetitions of cyclic loads or cyclic loads of greater intensity before they develop loss of supporting capacity as described above. However, severe damage can result from settlement or displacements that develop even when there is only a partial loss of strength. Various investigators have referred to this latter phenomenon as "partial liquefaction", "cyclic mobility", or other similar terms. There are no clear-cut definitions for these terms and no specific boundaries between events of varying magnitude. In this report, the general reference to liquefaction potential relates to any of the entire spectrum of events resulting from the application of cyclic loads to granular deposits below the water table, that could result ultimately in unacceptable distress to a structure founded thereon.

Extensive field and laboratory investigations have been performed in the last decade with the specific purpose of evaluating the liquefaction potential of soil deposits during earthquakes. Investigations have included:

- 1) Studies of the in situ soil conditions at various sites that have been subjected to earthquakes of known intensities and identification of soil

properties which differ between those sites that have developed liquefaction and those that have remained stable.

- 2) Studies of the behavior of representative soil elements in the laboratory when subjected to the same stress reversals as they would encounter in the field, and extrapolation of the results into meaningful predictions of probable field performance.

It is obvious that in order to establish the reliability of either approach, the end results of the two general methods must dovetail with each other.

This report summarizes primarily the state-of-the-art of the second approach, that of laboratory testing, but also includes a section on the comparison of laboratory results with field performance.

6.1 BACKGROUND

Liquefaction was first observed to take place under single or multiple load applications which occurred naturally on earth slopes due to a gradual rise in the water table or tidal fluctuations resulting in excess seepage pressures, or sudden shock from explosions or earthquakes. The end result was generally a massive flow slide, in which the soil came to rest only when the slope had been reduced to a few degrees, depending upon the viscosity of the fluid. In all cases, the sands were excessively loose when a small change in static equilibrium of the stresses would result in the sands becoming unstable or liquid. To explain this phenomenon and predict its occurrence, the "critical void ratio" concept was advanced by A. Casagrande, 1936. Subsequently, following extensive studies of numerous flow slides along the banks of the Mississippi River, empirical rules were developed by the Corps of Engineers to predict the likelihood of occurrence of such flow slides. These rules, and other empirical methods as well as Casagrande's concept, were also derived largely from the physical characteristics of the sands and included such parameters as grain size

characteristics, relative density, and the thickness of stable deposits overlying the liquefiable sands.

In recent years, the term "liquefaction" has been extended even further to include soil behavior under continuous earthquake vibrations or cyclic loading conditions. While the end result of instability is essentially the same, whether caused by either static or dynamic loading, the stresses and pore-pressure changes leading to liquefaction are quite different in the two cases. Under static or single-stress (monotonic) loading conditions, the stresses required to cause liquefaction are much larger than those required to cause liquefaction under continuous cyclic loading conditions. Under continuous vibrations the smaller cyclic stresses, none of which would cause failure by itself, cause a steady build-up of pore pressure within the soil mass which in turn reduces progressively the effective stresses between the soil grains leading ultimately to large strains and/or liquefaction.

The most comprehensive studies to date of liquefaction and of related events, including ground response to earthquakes, have been accomplished by various investigators at the University of California under the direction of Dr. H. Bolton Seed. These studies have concentrated on the behavior of sand specimens under cyclic loading conditions in the laboratory and their relationships to conditions known to have produced liquefaction during earthquakes in the field. These procedures lead to results which are somewhat different from those indicated either by the empirical approach or by the critical void ratio concept with respect to the susceptibility of the denser sands to liquefaction. Consequently, some engineers have questioned whether these procedures are applicable to site evaluation investigations for major structures, including nuclear power plants, feeling that the results may tend toward overly-conservative design, particularly for the denser sand sites.

The following sections of this state-of-the-art report are intended to provide a better understanding of the basic concept of each approach and the applicability of each method to evaluate liquefaction effects under earthquake loading conditions.

6.1.1 Critical Void Ratio Concept

The critical void ratio concept was derived and developed largely from the volumetric changes of soils under axial loading (fully drained) in the triaxial cell. It was concluded that if a saturated soil specimen was not allowed to change its total volume (either because the rate of loading was too rapid or the soil too impervious), shear stresses applied to the soil would result in an increase or decrease in the pore-water pressure depending on the tendency for volumetric change of the soil skeleton. A loose specimen with a high void ratio would try to become denser during application of load. This tendency to decrease the volume of the soil would increase the pore water pressure and thus decrease the intergranular pressure such that the total volume remained constant. Provided the pore pressure kept increasing until it became equal to the confining pressure, the effective stress between the particles would become zero, creating a fluid condition. On the other hand, a dense specimen with a low void ratio would tend to increase its volume during shearing. As the granular structure expanded, the specimen would tend to dilate and the pore pressures would decrease, thus increasing the effective stresses and causing the sand to become more stable. It appeared obvious that at some intermediate density between the two extremes in density, there would be some initial void ratio, termed the "critical void ratio," for which no volume change during drained shear, and correspondingly no pore-pressure changes during undrained shear, would occur. Thus the critical void ratio was intended to define the boundary between sands that would and would not liquefy under static loading conditions. The ratio of the critical void ratio to the in situ void ratio thus defined the liquefaction potential of a site (i.e., if the void ratio were above the critical value, it could liquefy; and if it were below the critical value, it would not liquefy).

It is now generally recognized that all sands, including even relatively dense sands, tend to decrease in volume during the initial stages of loading and it is not until the shear stresses

become appreciable that the effects of shear displacements between the grains tend to produce a volume increase for dense sands. As a result, the initial concept of critical void ratio has subsequently been modified by Castro, 1969, to be that void ratio at a specific confining pressure which will permit large strains to occur at failure without further change in volume. As such, this definition is of interest to liquefaction phenomena relating to flow slides but is not directly applicable to the phenomena relating to cyclic loading. As an example, laboratory tests demonstrate that the critical void ratio for a given sand decreases as the confining pressure increases. This result would lead one to believe that a sand at a given density or void ratio is less stable under a high confining pressure than under a low confining pressure, and this is true once failure has been initiated. Under cyclic loading conditions, however, and assuming that applied shear stresses remain constant, the number of stress cycles required to develop liquefaction increases rapidly as the confining pressure is increased.

The critical void ratio concept is furthermore not applicable for predicting soil behavior under cyclic loading, because both volume changes and stress conditions leading to failure are quite different from those under monotonic loading conditions. In the latter instance, the critical void ratio pertains to a failure condition with no change in volume; whereas during cyclic loading, the applied stresses are significantly lower than static failure stresses.

The significant difference between a static test and a cyclic load test is the relationship between pore-pressure change and applied shear stresses during loading. In a static or monotonic test, a dense sand will tend to increase its volume prior to failure, thus causing a drop in pore pressure and an increase in effective stress. In a cyclic load test, each load cycle results in a residual pore pressure, which means that the effective stresses gradually become smaller until a failure condition is reached. If at this stage of the test, the void ratio is above

the critical void ratio (which varies with the effective stresses) failures will be abrupt. On the other hand, if the void ratio at failure is less than the critical void ratio, continued application of cyclic loads will result in increasing strains. These strains stop as soon as the cyclic loading ceases.

6.1.2 Field Observations

The development of empirical procedures to evaluate the liquefaction potential of soils began in the late 1940's and early 1950's by the Waterways Experiment Station (WES) during investigations of the stability of embankments along the Mississippi River. The conditions leading to the flow failures, commonly termed "spontaneous liquefaction", apparently occurred on loose point bar deposits when a build-up of pore pressures resulted from large river fluctuations. The point bar deposits generally underlay more competent cohesive materials and comprise the lower portion of the river embankment. In addition to flow failures, caving also occurred from scour. As a result of observations of numerous embankment failures, and subsequent exploration programs, the empirical criteria established as early as 1952 were continuously modified as more data became available and led to the conclusions that sites were potentially unstable when the following conditions existed:

- 1) The thickness of potentially liquefiable sands is at least 25 feet.
- 2) The ratio (R) of the thickness of the overlying stratum (which is not likely to liquefy) to the thickness of the stratum to be liquefied is less than 0.85.
- 3) The percent fines (passing #200 sieve) of the liquefiable zone are less than 10 percent, except for occasional thin cohesive lenses.

The above empirical criteria bypass difficulties in performing reliable tests for determination of volume changes and densities necessary using the critical void ratio concept, and thus still have merit with respect to flow slides.

One other empirical approach, developed by Florin, and widely used in the Soviet Union to study the liquefaction potential of natural and hydraulically filled river banks, includes blasting in the field (Florin and Ivanov, 1961). In this test, the upper 30 feet of soil is investigated by placing and firing successively three standard weight charges (5 kg) at a given depth (approximately 15 feet) and then measuring after each shock the average settlement of the surface of the soil within a given radius (approximately 15 feet) from the charge. The size and depth location of the charge is selected to ensure that there will be no expulsion of soil during blasting. If the average settlement is less than 3 to 4 inches and the ratio of settlement between successive shots is less than 0.6, there should be no need to provide measures against spontaneous liquefaction. This approach, as well as a similar study described by Kummeneje and Eide, 1961, again relies indirectly on only the density characteristics of the soil, since dense soils will normally not settle as much as the looser, more liquefiable soils. The main advantages of these empirical methods are their simplicity and the general ease of being able to cover thoroughly a large area at a reasonably low cost.

Although conditions leading to liquefaction of point bar deposits do not apply to cyclic loading conditions as in earthquakes, the end result may be the same for sands which are susceptible to spontaneous liquefaction. However, the listed criteria are not applicable to evaluation of liquefaction potential under earthquake loading.

Along the same lines as those undertaken by WES, and possibly the Soviet Union, to develop empirical rules for predicting liquefaction of embankments, recent attempts have been made by two Japanese investigators to also make empirical rules for predicting liquefaction caused by earthquakes, based on case histories in Japan.

Kishida, 1969a, and Ohsaki, 1969, conclude that sandy soils, having the following gradation characteristics, are susceptible to

liquefaction if the groundwater table is near the ground surface:

$2\text{mm} > D_{50}$ $U < 10$ (Kishida)

$2\text{mm} > D_{60} > 0.20\text{mm}$ $D_{10} > 0.074\text{mm}$ $U < 5$ (Ohsaki)

Kishida further concludes that the above soils are likely to liquefy provided:

- 1) The thickness of the non-liquefiable soil stratum over the liquefiable stratum is less than 8m., and
- 2) The ratio of the thickness of the non-liquefiable soil stratum to the liquefiable zone is less than unity.

Kishida also concludes that the above soils are not likely to liquefy provided:

- 1) The effective confining pressure exceeds 2kg/cm^2 , or
- 2) The relative density exceeds 75%

Ohsaki concludes that soils susceptible to liquefaction are not likely to liquefy if the standard penetration resistance (blows/ft) is greater than $2Z$ where Z is the depth in meters.

While the above criteria are useful as guidelines for preliminary site evaluation, they are not suitable for general adoption for determining the liquefaction potential of a nuclear power plant site for the following reasons:

- 1) They are based largely on the 1964 Niigata Earthquake and do not consider other earthquakes and other soils.
- 2) They are too general and do not include either shaking intensity or duration of shaking.
- 3) They provide no indication of the margin of safety with respect to a design earthquake.
- 4) No differentiation is made between total liquefaction and tolerable settlements.

6.1.3 Cyclic Loading, Laboratory Tests, and Analyses

The mechanics of progressive pore-pressure build-up leading to liquefaction, as a result of cyclic loading, are different from static loading. Since the effects are unpredictable from theoretical considerations, they must be investigated by experimental

procedures. Laboratory tests permit such effects to be studied under controlled conditions so that the variables can be independently evaluated, provided samples representative of in situ conditions can be prepared. Although liquefaction can be developed by shaking a small amount of saturated sand in the palm of the hand, the results cannot be evaluated, since boundary conditions, confining stresses, deformations and pore pressures cannot be controlled or measured. However, the development of two laboratory tests, the cyclic triaxial test (Seed and Lee, 1966; Lee and Seed, 1967a) and the cyclic simple shear test (Peacock and Seed, 1968), makes it possible to study the parameters affecting the events leading to liquefaction.

The results of a number of tests using these testing procedures, indicate that even though the relative density controls to a large extent the susceptibility of sand to liquefaction, other factors such as the confining pressure, the number of stress or strain cycles, and the magnitude of applied stresses are also important factors. More details about these and other test procedures and the influence of each of these important factors and others are given in later subsections.

Based on laboratory tests, together with correlations of the various factors, and a number of simplified assumptions about the stress conditions resulting from actual earthquakes, methods of analyses to evaluate the liquefaction potential of a site have been developed (Seed and Idriss, 1967, 1970b). In general, after establishing the soil conditions and the design earthquake, three separate phases are required. One involves performing a detailed ground response analysis of the prospective site. The choice of method of analysis or analytical model, as discussed in Chapter 2 of this report, depends on the topography of the area, the uniformity of the soil conditions and the depth and slope of the underlying bedrock surface. The response study develops a time history and magnitude of the shear stresses induced in the soil by an assumed earthquake. These stresses must be determined for those selected depths or soil layers that are most likely to liquefy (generally in the upper 50 feet).

The second phase is to determine the relationship between the magnitude of shear stress and the number of load repetitions that must be applied to the soil to cause liquefaction. This is accomplished in the laboratory during either the cyclic triaxial test or the cyclic simple shear test. Since neither of these two tests simulate exact simple shear conditions as encountered under earthquake loading, the results must be corrected empirically to agree with observed field data. These corrections are discussed in a subsequent section of this chapter.

Having established both the probable ground response of a specific site and the behavior of laboratory specimens of foundation soils subjected to repetitive cyclic loads of varying magnitude, the third phase is to compare the two sets of data and thus evaluate the liquefaction potential of the site.

This procedure is based on a number of simplifying assumptions since it is necessary to correlate the complex time history of stresses from an assumed earthquake with a uniform sequence of cyclic stresses. The more important assumptions are as follows:

- 1) The critical design time history of accelerations is reasonably well known.
- 2) The prescribed time history of acceleration can be normalized by averaging groups of loading cycles to conform to a uniform sinusoidal pattern of significant stress cycles.

The next Section 6.2 discusses the various factors pertinent to the soil liquefaction problem. Tests to study liquefaction in the laboratory are discussed in Section 6.3. Sections 6.4 and 6.5 describe the methods for evaluating soil liquefaction potential. A recent paper by Seed and Idriss, 1970b, entitled, "A Simplified Procedure for Evaluating Soil Liquefaction Potential," which provides a comprehensive picture of the current (1972) state-of-the-art for predicting liquefaction potential, is reproduced in Appendix B. Further comments, conclusions, and an evaluation of procedures presently used to determine soil liquefaction potential are contained in Section 6.6.

6.2 FACTORS PERTINENT TO THE SOIL LIQUEFACTION PROBLEM

Based on cyclic loading tests performed in the laboratory, the occurrence of liquefaction of cohesionless soils involves a direct relationship among the three factors: 1) void ratio (relative density), 2) the ratio of cyclic shear stress to initial effective confining pressure, and 3) the number of stress cycles. Also less directly involved are the grain characteristics (particle size, shape and gradation) and cohesion or cementation within the soil mass. All of these factors not only reflect the physical properties of the soil but also the initial stress conditions in the ground and the anticipated stress and duration characteristics of the earthquake. In the laboratory, the three important parameters are correlated by subjecting test samples of known density and confining pressure to cyclic shear stresses and measuring the number of cycles required to develop liquefaction.

In the field, the evaluation of the three corresponding factors is not so easily defined, because the density characteristics of the soil are not as accurately measured as in the laboratory, and the magnitude, frequency, and duration characteristics of the earthquake (input motions) must be determined. From a practical standpoint, the shear stress in the soil and the number of significant cycles determined for each soil layer from response calculations are fixed by design criteria, backed up by judgment. The following paragraphs discuss the effect of each factor on the liquefaction potential and the way in which each is evaluated by cyclic loading tests in the laboratory and, correspondingly, in the field under the design earthquake loading condition.

6.2.1 Void Ratio or Relative Density

Laboratory tests have, of course, verified that the lower the relative density (or the higher the void ratio of a soil) the more easily a sample will liquefy. This result is verified not only by cyclic triaxial tests with typical relationships as shown in Fig. 6.1a (Lee and Seed, 1967a), but also by shake table tests (Whitman, 1970c), cyclic simple shear tests (Peacock and Seed, 1968),

a. EFFECT OF VOID RATIO

b. EFFECT OF CONFINING PRESSURE

Fig. 6.1 INTERRELATION BETWEEN VOID RATIO, CONFINING PRESSURE, PEAK PULSATING DEVIATOR STRESS, AND NUMBER OF CYCLES

(From Lee and Seed, 1967a)

and by comparisons of field case histories where liquefaction did and did not occur (Whitman, 1969a; Seed and Peacock, 1970).

The most common methods used in the field to determine relative density are either to estimate it, based on penetration tests in borings utilizing standard correlations, or else to actually measure it from undisturbed samples. The details and limitations of the field methods are described in subsection 3.1.1 and Appendix A; while the accuracy with which correlations and measurements are made is discussed and illustrated by actual test data in subsection 3.2.2. In the laboratory, recompacted specimens can be controlled to a desired relative density within an accuracy of about one to two percent. The laboratory procedure used to measure void ratio and then to convert this term to relative density is discussed in subsection 3.3.2.

6.2.2 Ratio of Shear Stress to Initial Effective Stress

The ratio of cyclic shear stress to the initial effective confining stress (τ/σ'_c or $\sigma_p/2\sigma'_c$) has proven to be a convenient expression for presenting data for the study of liquefaction (Seed and Idriss, 1967; Peacock and Seed, 1968; Finn, Pickering and Bransby, 1969; and Whitman, 1969a). In this expression, the effective confining stress is a parameter which can be determined within the desired accuracy both in the laboratory and the field. In the field, it is computed from the measured or estimated unit weight of the soil; while in the laboratory, it is controlled directly by external pressure within a confining chamber surrounding the soil specimen, or by appropriate surcharge loads to the top of the specimen.

From field observations it has generally been concluded by a number of investigators that even in a saturated sand deposit below a depth of 50 to 60 feet, sands are not likely to liquefy. These depths are in general agreement with Kishida, 1969a, who states that a saturated sandy soil is not liquefiable if the value of the effective overburden pressure exceeds 2 Kg/cm^2 ($2 \text{ Kg/cm}^2 \approx 60 \text{ ft. of soil below the water table}$).

In the laboratory, the effect of confining pressure is best illustrated by performing a series of cyclic loading tests and plotting the results in the form shown in Fig. 6.1b. From many such tests it is well established that the effective pulsating deviator stress required to cause liquefaction in a specific number of cycles increases as the effective confining stress is increased. Further, the pulsating deviator stress, and thus the shear stress has been found to increase linearly with the confining pressure for a constant relative density (up to a value of 80 percent) and number of cycles (Seed and Lee, 1966; Peacock and Seed, 1968). This linear relationship, with two factors held constant, thus makes the effective stress a convenient value to normalize the shear stress.

The shear stress term in the ratio, τ/σ'_c , is included to reflect the magnitude of a potential earthquake. In the laboratory, the shear stress is expressed as one half the peak pulsating deviator stress. Typical records of pulsating load tests, with a uniformly applied pulsating stress, are shown in Figs. 6.2 and 6.3. The interrelation of this factor with the other factors under cyclic loading conditions is summarized in Fig. 6.1.

Under actual field conditions, the time history of shear stresses must be calculated for each significant soil layer, to include the accumulative effect of each significant cycle of shear stress. This can be accomplished using either ground response analyses or a more simplified procedure (Appendix B). Because the time history of earthquake induced stresses is random in nature and too complex to analyze directly using existing techniques, certain simplifying assumptions must be made. In view of the fact that the characteristics of the design earthquake itself are semi-empirical, it appears that simplifying assumptions or adjustments in the time stress history likewise are justifiable at this time. Simplifying the time history record of shear stresses involves normalizing the random peak shear stresses and the number of cycles to equivalent uniform values which can be compared with the uniform peak stress cycles applied in the laboratory. The adjusting of

$D_r = 55\%$

INITIAL EFFECTIVE CONFINING PRESSURE = 3 kg per sq cm

INITIAL PORE WATER PRESSURE = 100 psi

EFFECTIVE PULSATING DEVIATOR STRESS = ± 1.6 kg per sq cm

Fig. 6.2 RECORD OF PULSATING LOAD TEST ON LOOSE SAND

(Unpublished Shannon & Wilson data)

$D_r = 78\%$

INITIAL EFFECTIVE CONFINING PRESSURE = 3 kg per sq cm

INITIAL PORE WATER PRESSURE = 100 psi

EFFECTIVE PULSATING DEVIATOR STRESS = ± 2.2 kg per sq cm

Fig. 6.3 RECORD OF PULSATING LOAD TEST ON DENSE SAND

(Unpublished Shannon & Wilson data)

random stress conditions to obtain an equivalent uniform pattern presently involves a non-linear adjustment in accordance with the laboratory stress versus number of cycle curve shown typically in Fig. 6.1.

Once the field stresses are normalized, they can be compared numerically with the uniform cyclic stresses applied in the laboratory provided an adjustment factor is used to make the laboratory results consistent with the field performance.

6.2.3 Number of Cycles

The number of cycles is the last of the three interrelated factors of major interest in evaluating the soil liquefaction potential of a site. Under field conditions, the number of significant cycles is dependent upon the frequency and duration of loading of the earthquake. Typical earthquake characteristics are approximately as follows:

Frequency of vibrations	.5 to 5 cps
Duration of strong shaking	5 to 30 sec
No. of significant stress cycles	5 to 30

Based on these data, the relationship of the number of cycles in the laboratory must generally be correlated with the relative density, the confining pressure, and the peak cyclic shear stress or deviator stress between the ranges of 5 and 30 cycles. As an example of the relationship between these various factors, determined from cyclic triaxial test data, reference is made to Fig. 6.1. In this figure, it is seen that the number of cycles to cause failure increases with a decrease in the pulsating stress. Similar relationships can also be determined from the various other cyclic loading laboratory tests.

6.2.4 Grain Characteristics

The grain characteristics including particle size, shape and gradation, influence the susceptibility of soil to liquefaction (Lee and Fitton, 1968). The effect of particle size is best illustrated in Fig. 6.4. Based on the tests summarized in this

a. GRAIN-SIZE DISTRIBUTIONS

b. EFFECT OF GRADATION ON CYCLIC STRENGTH

(From Lee and Fitton, 1968)

Fig. 6.4 GRAIN-SIZE & STRENGTH CHARACTERISTICS OF MOST LIQUEFIABLE SOILS

figure, the weakest soils are the very fine sands and silty sands. As the particle size increases to the gravel range the stress required to create liquefaction of the looser deposits nearly doubles over that required to liquefy the silty fine sands. The presence of clay fines, however, causes an even larger increase in the stress level required to create liquefaction. Such high pulsating stresses illustrate that insensitive clays and gravels are, for all practical purposes, not susceptible to liquefaction.

Based on a number of grain size tests on sands that did liquefy in Japan under actual earthquakes, including the Niigata earthquake in 1964, it has been further verified that silty sands are the most susceptible soils to liquefaction. An envelope of these grain size curves superimposed on the envelope of soils found most likely to liquefy in the laboratory, is presented in Fig. 6.5.

The effect of grain shape on liquefaction, has been investigated to a lesser extent by Lee and Fitton, 1968, and Rocker, 1968. Lee and Fitton tested 10 sands at different sites of similar grain size, but of varying shapes, and concluded that there was no significant difference in effect as all values showed a maximum deviation in pulsating stress of only 12 percent at a constant relative density. Although there are discrepancies in the reported data, the actual effect of grain shape is believed to not be significant. Fig. 6.6 (Seed and Peacock, 1970) summarizes the stress conditions causing failure in 10 cycles as a function of mean grain sizes, based on published data from 10 different organizations.

The effects of gradation or changes in the uniformity coefficient, U , have likewise not been studied extensively or reported in the published literature. In general, there are no data available to indicate that variations in the gradation characteristics have a significant effect on the susceptibility of a soil to liquefaction. Although intuition would favor a well-graded soil as being more resistant to liquefaction than a poorly-

PER CENT FINER BY WEIGHT

Fig.6.5 SOILS SUSCEPTIBLE TO LIQUEFACTION

Fig.6.6 STRESS CONDITIONS CAUSING LIQUEFACTION IN 10 CYCLES - TRIAXIAL COMPRESSION TESTS

(From Seed and Peacock, 1970)

graded soil, at a constant relative density, the limited data that are available apparently do not support this concept.

Although the effect of grain shape and gradation have not been thoroughly investigated and are not yet clear, the general grain size and cyclic load data in Figs. 6.4, 6.5 and 6.6 allow the engineer to assess empirically the general liquefaction potential of many soils. Empirical comparisons of other data with these data should, however, be used only as a preliminary guide, as deviations due to such factors as sensitivity, cohesion and particle cementation, will surely exist.

6.2.5 Soil Types

a. Cemented and Slightly Cohesive Soils

Cementation and cohesion are soil characteristics which tend to resist deformation and liquefaction under cyclic loading. When evaluating potentially liquefiable soils, both of these properties are difficult to assess quantitatively because of the problems involved in testing undisturbed samples. These problems are explained in subsection 3.3.1.

As shown on Fig. 6.5, silty fines are susceptible to liquefaction; however, if the fines are plastic (developing cohesion), the tendency to liquefy is greatly reduced. Clay fines are cohesive and are not susceptible to liquefying. The amount of cohesive fines has been used as one limit for empirically determining if a soil is susceptible to liquefaction. Based on experience from laboratory cyclic triaxial tests (Lee and Fitton, 1968), Japanese field observations of actual earthquakes (Kishida, 1969a), and data from the 1964 Alaska earthquake (Shannon & Wilson, 1964), those soils most liquefiable have D_{10} grain size values generally larger than 0.01 mm (Fig. 6.5). WES also reports that those sands most susceptible to liquefaction in their river bank studies along the Mississippi River have less than 10 percent fines passing the 200 mesh sieve (WES, 1956).

b. Cohesive Soils

Major slide movements during earthquakes have occurred in clay deposits. There is no evidence that clays liquefy during earthquakes in the sense that an extensive zone of clay soil is converted into a fluid condition. However, some types of clay may lose a proportion of their strength and the additional forces induced by an earthquake may cause slope failures with resulting slumping. In addition, clay deposits often contain sand lenses and liquefaction of these lenses may lead to significant landslide development.

The behavior of homogeneous clay samples has been studied in the laboratory primarily by means of cyclic triaxial and simple shear tests. Thiers and Seed, 1968b, for example, found that at least for the soils tested, reductions in strength of the order of 20 percent were obtained after the application of 200 cycles of a deviatoric strain equal to approximately one half of the failure strains for such materials.

Shake table studies on clays have been carried out primarily with the purpose of obtaining soil response values (Appuhn, 1964, Kovacs, 1968). More recently, Arango, 1971, studied the seismic stability of slopes in saturated clays by use of a shake table and a simple shear device. The following paragraphs summarize the findings of Arango's investigations:

"The shaking table studies were used to obtain a mechanistic picture of the deformations of the slopes of clay banks during earthquakes, the extent of such deformations and the factors influencing them. The simple shear tests were performed to compare the soil behavior under cyclic loading.

Test results and comparative analysis of the two series of tests showed that in large shaking table tests there is a peak or yield acceleration which

might be developed within the clay banks during shaking. The development of this acceleration determined whether or not the slope would be greatly deformed under cyclic loading conditions. The value of the yield acceleration was found to be related to the geometry of the potential yielding soil mass and to the available maximum strength of the clay when subjected to the given seismic loading.

It was also found that the maximum strength shown by clay banks under seismic loading was always lower than the transient strength of the material. The actual value of the strength may be either higher or lower than the static undrained strength of the clay, depending on such factors as the amplitude of ground motion and the rate of strain. The deformation of the slope under seismic loading was found to be a function of the residual strength of the clay and the duration of the seismic motion."

6.3 LABORATORY TESTS TO STUDY SOIL LIQUEFACTION

Laboratory tests which have been used to study the phenomena leading to liquefaction under vibratory or cyclic loading conditions consist of shake table tests, cyclic triaxial tests, cyclic simple shear tests, soil box tests and cyclic torsional column loading tests. All of these tests except the shake table procedures have been developed within the past 5 to 7 years (since 1964), largely as a result of the recent concern for behavior of saturated soil under earthquake loading conditions. This concern probably became most evident in 1964, the year in which two great earthquakes, one in Japan near Niigata and the other in Alaska, created soil liquefaction which resulted in widespread damage.

Before 1964, other case histories had led investigators to study the soil characteristics largely through shake tables. However, because of geometrical and boundary conditions of the container, the results were difficult to relate to field conditions. These difficulties caused other researchers to turn to small scale tests in which stresses, strains, pore pressures and relative densities could be more closely controlled or measured. These test devices were essentially monotonic shear devices, converted or modified to apply cyclic loading conditions representing those induced under actual field conditions. While these tests, especially the cyclic triaxial and simple shear tests, have led to a better understanding of the factors affecting liquefaction, none of the small scale tests exactly reproduces the field stress conditions, thus prohibiting direct extrapolation of the results to field conditions. This section describes the tests, including their limitations and availability of the equipment and general cost of each test, so that practicing engineers may understand the uses of the tests and the state-of-the-art in soil testing methods for evaluating liquefaction under cyclic loading conditions in the laboratory.

6.3.1 Shake Table Test

The first laboratory investigation of liquefaction under vibratory loading conditions employed various sizes of shake tables (Maslov, 1957; Tanimoto, 1967; Yoshimi, 1967; Whitman, 1970c). These tests generally consisted of placing saturated sand in a box or tank on a shake table and measuring the horizontal table acceleration at which liquefaction occurred. These values were related directly to field measured accelerations. From the known density characteristics of the specimen and the measured acceleration, Whitman further computed the peak shear stress causing liquefaction, using the following relationship (Whitman, 1970c):

$$\tau = \frac{\gamma a}{\gamma' g} \sigma'_v$$

where γ = wet unit weight
 a = shake table acceleration to cause liquefaction
 g = acceleration of gravity
 σ'_v = vertical effective stress at depth under study
 γ' = submerged unit weight

By means of the above relationship, data from this test could also be compared with the shear stresses measured directly in the smaller scale tests, described subsequently.

Although the above reported approaches appear to be simple, providing a direct method for creating liquefaction under simple shear loading conditions, certain factors of the apparatus which generally affect the results are not measured or considered, making extrapolation to field conditions presently difficult, and in some cases misleading. Such factors that affect the test results or influence their interpretation include the following:

- 1) Frequency of vibration
- 2) Duration of vibration
- 3) Size and geometry of container
- 4) Deformation characteristics of container
- 5) Method of sample placement
- 6) Drainage control
- 7) Strain measuring devices
- 8) Confining pressure

The frequency must be selected so that the shear stresses are applied as uniformly as possible throughout the specimen in the container. For this purpose, frequencies much smaller than the resonant frequency are desirable. Also, the frequency must be selected so that the amplitude of vibration is compatible

with both the desired acceleration level and the desired shear strain.

Somehow the duration of shaking must be reflected in these tests since, as noted for small scale tests, it is an important factor. The longer a specimen is shaken, the greater is the possibility for the pore pressure to build up to the confining pressure creating a liquid condition and, as for small scale tests, the number of cycles to create liquefaction is probably the best expression of this factor. In previously reported investigations, which have expressed liquefaction in terms of the acceleration required to cause it to occur, this important factor has been omitted from consideration.

The size and geometry of the container must be selected so as to satisfy free field soil behavior, i.e., to eliminate non-representative boundary restraints. Results of recent investigations of clay embankments (Arango, 1971; Kovacs, Seed and Chan, 1971) indicate that a length to thickness ratio of at least 12 is necessary in order to assure an adequate representation at a point in the center of the soil specimen. It is anticipated that this length to thickness ratio is also reasonably applicable for sands. This factor has largely been neglected in most of the previously reported investigations of liquefaction using shake table facilities.

The deformation characteristics of the container also affect the liquefaction potential of a tested soil, since a container with rigid sides will cause the corners and sides of the sand specimen to move with the table thus resulting in smaller shear strains within the specimen.

As the size of the specimen is increased, uniform density is more difficult to attain. If a uniform density is not maintained throughout the specimen, the looser areas will tend to liquefy more readily.

In studying liquefaction, specimens must be tested under undrained conditions in order for pore pressures to build up and

thus for failure to occur. To preserve this condition on a large specimen of sand, which may be only inches thick as in a shake table, drainage from the top or bottom of the specimen must be prevented. This problem can be overcome by covering the specimen with an impervious rubber membrane; however, with large specimens, this task is cumbersome.

In shake table tests performed to date, instrumentation has not been installed in the soil sample to measure the strains which occur within the soil prior to the onset of liquefaction. Such measurements would also be desirable, to investigate the influence of boundary conditions and effects of loose zones on the liquefaction potential of the soil mass. Suitable instrumentation has not yet been developed specifically for measuring these internal strains.

Developing sufficiently large confining pressures (or vertical effective stresses, σ'_v) with a shake table, to more closely simulate field conditions, presents one of the most significant problems with the shake table tests. At realistic confining pressures, the shear stresses required to cause liquefaction are so large that they must be produced either by the oscillation of a large overburden mass on the surface of the soil specimen, or by a separate actuator attached to the soil surface rather than by the shake table.

Aware of many of the above limitations, Finn, Emery, and Gupta, 1970, conducted shake table tests to study liquefaction on a saturated sand sample to which a small surcharge air pressure was applied. Although all of the problems were not resolved, they demonstrated important progress in this area. For a further discussion and assessment of their research, reference is made to a report of their work directly, or to SW-AJA, 1971.

In summary, shake table tests have not produced particularly useful results on liquefaction behavior to date because of the following limitations:

- 1) A thin layer only is tested, and in most tests this would represent the upper 1- to 3-foot layer

of ground. Thus there is no indication of confining pressure effects, which are known to be significant.

- 2) The geometry of the table has usually been such that the measured accelerations represented the motion of the entire system, and not that of the soil. Hence, the computed shear stresses to cause liquefaction may, in fact, be much larger than those actually created in the soil.
- 3) Difficulties in measuring stresses and strains within the soil mass have required values for these factors to be determined indirectly from computations, with unknown errors.

It is unlikely that meaningful data will result from additional shake table tests in their present form. However, as the various limitations are overcome, as accomplished in part by Finn, Emery and Gupta, and as more sophisticated shaking tables become available, their role in future research on liquefaction is likely to be significant.

6.3.2 Cyclic Triaxial Test

The cyclic triaxial test was developed (Seed and Lee, 1966) as an inexpensive method by which to study under cyclic loading the various factors leading to the onset of liquefaction. The ease of preparing and testing specimens of sand compacted to desired relative densities, the availability of the equipment, and the precise control of most factors considered of interest, has resulted in extensive use of this procedure.

In this type of test, a specimen of the sand to be tested is first compacted in the laboratory to a given density and then encased in a rubber membrane and subjected to an initial all-around hydrostatic confining pressure. (Undisturbed sand samples are seldom used because of the difficulty of obtaining and handling samples of non-cohesive soil.) Next, the specimen is saturated using standard laboratory procedures. The saturated sample is then consolidated under the desired ambient pressure which produces the stress condition as shown by Condition 1 in

Fig. 6.7. In order to assure complete saturation (a requirement for this type of test), the pore pressure is increased initially to at least 30 psi, and preferably to about 90 psi, which requires an equal increase in the confining pressure.

After the desired initial state of stress is attained, the drainage valves are closed to prevent further drainage and sensors are connected to measure the pore pressure. The axial stress is then alternately increased and decreased by σ_{dc} at frequencies of the order of one cycle per second, and at the same time the applied stresses and the resulting vertical strain and pore-pressure changes are recorded.

The above test procedure produces the same effective stress changes as would occur were the axial stress to be increased by $\sigma_{dc}/2$ and simultaneously the lateral stress reduced by the same amount (Seed and Peacock, 1970). If this were done, the normal stress on a 45° plane in the sample would remain unchanged and the shear stress developed on the plane equal to $\sigma_{dc}/2$. Reversing the stress change would then reverse the direction of shear stress application on the 45° plane, all as shown for Condition 2, Fig. 6.7. This means that on this plane the normal stress remains constant but cyclic shear stresses equal to $\sigma_{dc}/2$ are applied as the test proceeds. The conditions on the 45° plane are thus similar to those on the horizontal plane for the field loading condition (Seed and Lee, 1966).

The results of triaxial compression tests of this type are usually expressed in terms of the ratio of the applied shear stress $\sigma_{dc}/2$ and the initial ambient pressure σ_a which causes liquefaction to occur for any given sample and test condition; that is, in terms of the ratio τ_{max}/σ_a or $\sigma_{dc}/2\sigma_a$.

The applicability of cyclic loading triaxial tests to the evaluation of liquefaction potential under field loading conditions is affected by a number of factors. In the first place, the stress conditions are quite different since in the triaxial

Fig.6.7 CYCLIC LOADING TRIAXIAL COMPRESSION TEST

(From Seed and Peacock, 1970)

test it is only possible to produce the desired stress changes by consolidating the sample initially under an ambient stress condition corresponding to $K_0 = 1$, whereas for the normal field condition $K_0 =$ approximately 0.4. Secondly, the shear stress involved in the ratio $\sigma_{dc}/2\sigma_a$ is the maximum shear stress on the sample, whereas in the field condition the shear stress in the ratio τ_{max}/σ_a is somewhat less than the maximum.

In addition to the above differences in stress conditions, the accuracy of cyclic loading triaxial tests is also affected by the following differences as quoted directly from Seed and Peacock, 1970:

- "1) Under field loading conditions the direction of the major principal stress will vary from about 0 to 40° on each side of the vertical but in the triaxial compression test, the direction of the major principal stress will necessarily rotate through an angle of 90° on each stress cycle.
- 2) Under the idealized field loading condition or in the simple shear test, the intermediate principal stress has a value corresponding to plane strain conditions; in the triaxial compression test, however, the intermediate principal stress has a value equal to the major principal stress during one half of the loading cycle and equal to the minor principal stress during the other half of the loading cycle.
- 3) Unless special precautions are taken friction between the cap and base of the test specimen can cause stress concentrations which can lead to premature failure of the specimen (Castro, 1969).
- 4) During the lateral compression part of the stress cycle, the sample will tend to neck as failure approaches, and it is difficult to determine

the stress conditions in the sample once this occurs. The effect is not likely to be very significant for relatively loose samples in which liquefaction and the development of large strains occur almost simultaneously but it can have a significant effect on the behavior of dense samples, again leading to apparently larger deformations than would otherwise be the case.

The effects of these limitations of the test procedure are likely to cause test specimens to liquefy or develop failure strains at stress levels lower than those which would be required under the corresponding field conditions. On the other hand, the use of a $K_0 = 1$ condition in the test procedure together with a failure criterion expressed in terms of the ratio $\sigma_{dc}/2\sigma_a$ will cause the apparent stresses inducing failure or liquefaction in triaxial compression tests to be higher than those inducing failure in the field. Since the effects of these latter factors will normally outweigh other sources of error, the stresses causing liquefaction or failure in triaxial compression tests will normally need to be reduced before they are used to evaluate liquefaction under field conditions."

Recent publications describing results of extensive cyclic triaxial test programs conducted to study the factors affecting liquefaction include: Castro, 1969; Lee and Seed, 1967a; Lee and Fitton, 1968; Rocker, 1968; Seed and Lee, 1966; and Seed and Peacock, 1970.

The definition of failure by liquefaction is not well established by most tests. For loose sands this is not a problem as liquefaction occurs abruptly when the effective stress equals the pore pressure, as shown in Fig. 6.2. For medium dense or dense sands, however, both conditions do not occur simultaneously (Fig. 6.3). In these cases, straining may begin many cycles

before the pore pressure has reached the effective confining pressure. One approach (Seed and Lee, 1966) is to define liquefaction as occurring when the number of stress cycles reaches a given double amplitude strain of, say, 5, 10 or even 20 percent. Since such large strains will cause failure of structures which derive their support from such soils, this phenomenon is still properly classified as one of liquefaction, but because of the differing definitions of liquefaction under cyclic loading, the exact conditions assumed should always be specified for the benefit of those reviewing the work.

With due consideration given to the limitations of the test results and the care required in applying them to field problems, the cyclic triaxial test is the most useful laboratory procedure for the study of liquefaction available at the present time (1972).

6.3.3 Cyclic Simple Shear Test

The cyclic simple shear test was developed with the anticipation that it would better duplicate simple shear conditions than the triaxial test. Since the initial stress conditions in the ground are anisotropic, unlike those of the triaxial test, results were expected to be more nearly representative of the field case. However, because of local boundary conditions and other limitations discussed in subsection 4.2.2, the laboratory specimens tend to liquefy prematurely and the results thus underestimate the shear stresses required to cause liquefaction in the field.

This condition was observed by Peacock and Seed, 1968, the developers of this test. These two investigators suggested that this underestimate of shear stress empirically may be as much as 40 to 50 percent in some cases. Of the various boundary effects, the presence of small voids in the corners has been found to account for about 60 to 75 percent of this difference

(Peacock and Seed, 1968). These voids are generally present, due largely to the practical difficulties in preparing and placing test specimens in the shearing chamber.

Another important effect on the interpretation of results obtained from simple shear tests, appears to be the fact that shear stresses are only partially mobilized on the top and bottom plates. Studies of various roughnesses of these plates to better mobilize these stresses (Finn, Pickering and Bransby, 1969), showed that the use of ribbed plates resulted in 15 percent higher shear stresses to cause liquefaction.

The cost of the cyclic simple shear test apparatus and the practical difficulty of preparing samples for testing (especially undisturbed samples), together with other test deficiencies, have restricted its practical use in most testing laboratories. Results of research efforts on sands using this apparatus are only present in four articles; Peacock and Seed, 1968; Seed and Peacock, 1970; and Finn et al., 1969 and 1970. Further study using the simple shear device has been under consideration by Dr. Kovacs, at Purdue University. It is understood that he plans to study the liquefaction phenomenon and the moduli and damping properties of sand, using a larger shear box with a volume of $1/2$ cubic foot. Existing models are generally only about 0.0026 cubic foot (4.5 in.^3).

6.3.4 Soil Box Test

The soil box was developed originally in 1966 for measuring the static behavior of soil in hydrostatic compression (Ko, 1966; Ko and Scott, 1967c). This device has recently been used on a preliminary basis to study liquefaction in the laboratory, but in private communications with the authors, these data are not yet in a form for study and comparison with data from other test procedures.

6.3.5 Cyclic Torsional Column Loading Test

The cyclic torsional column loading test, although developed primarily for measuring shear modulus and damping, is presently being extended by Drs. Hardin and Drnevich of the University of Kentucky to study liquefaction in the laboratory. This application is still in the development stages and as yet no data are available for comparison purposes with the other test procedures.

6.4 GENERAL METHOD FOR EVALUATING SOIL LIQUEFACTION POTENTIAL

Since there are many factors that strongly influence the liquefaction potential of any soil deposit, any general method for evaluating such potential must take these factors into account. The general method proposed by Seed and Idriss, 1970b, from which the following is quoted, is believed to be the present state-of-the-art:

- "1) After establishing the soil conditions and the design earthquake, determine the time history of shear stresses induced by the earthquake ground motions at different depths within the deposit.
- 2) By appropriate weighting of the stress levels involved in the various stress cycles throughout the earthquake, convert the stress history into an equivalent number of uniform stress cycles and plot the equivalent uniform stress level as a function of depth as shown in Fig. 6.8. By this means the intensity of ground shaking, the duration of shaking and the variation of shear stress with depth within the deposit are taken into account.
- 3) By means of laboratory soil tests on representative samples, conducted under various confining pressures, determine the cyclic shear stresses which would have to be developed at various depths to cause liquefac-

Fig.6.8 METHOD OF EVALUATING LIQUEFACTION POTENTIAL.

(From Seed and Idriss, 1970b)

tion in the same number of stress cycles as that determined in step (2) to be representative of the particular earthquake under consideration. Either cyclic load triaxial compression tests or cyclic load simple shear tests may be used for this purpose and the interpretation of the test data to determine the stress conditions causing liquefaction in the field has been discussed elsewhere (Seed and Peacock, 1970). However in this way, the soil type, the in-place condition and the initial effective stress condition may be appropriately taken into account; the stresses required to cause failure may then be plotted as a function of depth as shown in Fig. 6.8.

- 4) By comparing the shear stresses induced by the earthquake with those required to cause liquefaction, determine whether any zone exists within the deposit where liquefaction can be expected to occur (induced stresses exceed those causing failure)."

6.5 A SIMPLIFIED PROCEDURE FOR EVALUATING SOIL LIQUEFACTION POTENTIAL

The present state-of-the-art procedure for evaluating soil liquefaction potential was summarized in the preceding paragraphs. Application of this procedure may require ground response analyses and the conduct of cyclic load tests on representative soils. However, in many cases a simplified procedure recently proposed by Seed and Idriss, 1970b, may prove adequate. Since it provides a comprehensive picture of the current state-of-the-art for predicting liquefaction potential, the paper by Seed and Idriss is reproduced as Appendix B to this report.

6.6 CONCLUSIONS AND EVALUATION OF PROCEDURES PRESENTLY USED TO DETERMINE SOIL LIQUEFACTION POTENTIAL

In order to have confidence in procedures for evaluating the liquefaction potential of soils, it is necessary to correlate the results of laboratory investigations with studies of actual sites where liquefaction has or has not developed under known stress conditions of actual earthquakes. Several investigators have undertaken such correlative studies, although the most comprehensive study is that by Seed and Peacock, 1970.

Table 6.1, from the Seed and Peacock report, summarizes the site conditions and earthquake data for some 35 known cases of liquefaction and non-liquefaction where the probable ground motion intensity was either measured or could be computed approximately. Fig. 6.9 from the same report, based on the data shown in Table 6.1, represents the relationship between the stress ratio $(\tau_{hv})_{av}/\sigma'_o$ and relative density for conditions which did and did not cause liquefaction. It is important to note that in some cases the plotted points are themselves a lower bound; i.e., the point represents the highest relative density at which sand at a particular site liquefied. The solid line in this figure thus represents the lower envelope of the relationships between the applied shear stress/initial normal stress ratio and the in situ relative density for sands that have liquefied and sands that have not.

In compiling available field data, it is necessary in many cases to estimate the ground accelerations at the various sites from a knowledge of the magnitude and epicentral distance of the earthquake. Thus in Fig. 6.9, of the 35 points plotted, there are only four cases where liquefaction occurred and the maximum ground acceleration was recorded: and similarly, four points where liquefaction did not occur and the maximum ground acceleration was recorded.

TABLE 6.1 SITE CONDITIONS AND EARTHQUAKE DATA FOR KNOWN CASES
OF LIQUEFACTION AND NON-LIQUEFACTION.

(From Seed and Peacock, 1970)

Earthquake	Date	Magnitude	Site	Distance from source of energy release -miles	Soil Type	Depth of water table -feet	Critical depth -feet	Av. penetration resist. at critical depth N	Relative density -%	Max. ground surface acceleration -g	$\frac{T_{av}}{T_0}$	Duration of shaking -secs	Field Behavior	Reference
Niigata	1802	6.6	Niigata	24	Sand	3	20	6	53	0.12	0.135	=20	No liquefaction	Kawasumi (8)
Niigata	1802	6.6	Niigata	24	Sand	3	20	12	64	0.12	0.135	=20	No liquefaction	Seed & Idriss (22)
Niigata	1887	6.1	Niigata	29	Sand	3	20	6	53	0.08	0.09	=12	No liquefaction	Seed & Idriss (22)
Niigata	1887	6.1	Niigata	29	Sand	3	20	12	64	0.08	0.09	=12	No liquefaction	Seed & Idriss (22)
Mino Owari	1891	8.4	Ogaki	20	Sand	3	45	17	65	=0.35	0.39	=75	Liquefaction	Kishida (10)
Mino Owari	1891	8.4	Gisan West	20	Sand	6	30	10	55	=0.35	0.37	=75	Liquefaction	Kishida (10)
Mino Owari	1891	8.4	Unuma	20	Sand & Gravel	6	25	19	75	=0.35	0.35	=75	No liquefaction	Kishida (10)
Mino Owari	1891	8.4	Ogane Pond	20	Sand	8	20	16	72	=0.35	0.35	=75	Liquefaction	Kishida (10)
Santa Barbara	1925	6.3	Sheffield Dam	7	Sand	=15	25	-	40	=0.2	0.16	15	Liquefaction	Seed et al (26)
El Centro	1940	7.0	Brawley	5	Sand	=15	=15	-	58	=0.25	0.155	30	Liquefaction	Ross (20)
El Centro	1940	7.0	All-Am. Canal	5	Sand	=20	=25	-	43	=0.25	0.155	30	Liquefaction	Ross (20)
El Centro	1940	7.0	Solfatara Canal	5	Sand	5	=20	-	32	=0.25	0.26	30	Liquefaction	Ross (20)
Tohankai	1944	8.3	Komei	100	Sand	5	13	4	40	=0.08	0.08	=70	Liquefaction	Kishida (10)
Tohankai	1944	8.3	Meiko St.	100	Silt & Sand	2	8	1	30	=0.08	0.09	=70	Liquefaction	Kishida (10)
Fukui	1948	7.2	Takaya	4	Sand	11	23	18	72	=0.30	0.30	=30	Liquefaction	Kishida (10)
Fukui	1948	7.2	Takaya	4	Sand	3	23	28	90	=0.30	0.32	=30	No liquefaction	Kishida (10)
Fukui	1948	7.2	Shonenji Temple	4	Sand	4	10	3	40	=0.30	0.29	=30	Liquefaction	Kishida (10)
Fukui	1948	7.2	Agr. Union	4	Sand & Silt	3	20	5	50	=0.30	0.33	=30	Liquefaction	Kishida (10)
San Francisco	1957	5.5	Lake Merced	4	Sand	8	10	7	55	=0.18	0.13	18	Liquefaction	Ross (20)
Chile	1960	8.4	Puerto Montt	=70	Sand	12	15	6	50	=0.15	0.15	=75	Liquefaction	Lee (12)
Chile	1960	8.4	Puerto Montt	=70	Sand	12	15	8	55	=0.15	0.15	=75	Liquefaction	Lee (12)
Chile	1960	8.4	Puerto Montt	=70	Sand	12	20	18	75	=0.15	0.15	=75	No liquefaction	Lee (12)
Niigata	1964	7.5	Niigata	32	Sand	3	20	6	53	0.16	0.175	40	Liquefaction	Seed & Idriss (22)
Niigata	1964	7.5	Niigata	32	Sand	3	25	15	70	0.16	0.175	40	Liquefaction	Kishida (9)
Niigata	1964	7.5	Niigata	32	Sand	3	20	12	64	0.16	0.175	40	No liquefaction	Seed & Idriss (22)
Niigata	1964	7.5	Niigata	32	Sand	12	25	6	53	0.16	0.16	40	No liquefaction	Seed & Idriss (22)
Alaska	1964	8.3	Snow River	60	Sand	0	20	5	50	=0.15	0.18	180	Liquefaction	Ross et al (21)
Alaska	1964	8.3	Snow River	60	Sand	8	20	5	44	=0.15	0.15	180	Liquefaction	Ross et al (21)
Alaska	1964	8.3	Quartz Creek	70	Sandy Gravel	0	=25	40-80	100	=0.12	0.145	180	No liquefaction	Ross et al (21)
Alaska	1964	8.3	Scott Glacier	55	Sand	0	=20	10	65	=0.16	0.185	180	Liquefaction	Ross et al (21)
Alaska	1964	8.3	Valdez	35	Sand & Gravel	5	=20	13	68	=0.25	0.25	180	Liquefaction	Coulter & Migliaccio (2)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	3	12	14	78	0.21	0.23	45	No liquefaction	Ohsaki (15)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	3	12	6	58	0.21	0.73	45	Liquefaction	Ohsaki (15)
Tokachioki	1968	7.8	Hachinohe	45 to 110	Sand	5	10	15	80	0.21	0.185	45	No liquefaction	Ohsaki (15)
Tokachioki	1968	7.8	Hakodate	100	Sand	3	15	6	55	0.18	0.205	45	Liquefaction	Kishida (11)

Fig.6.9 RELATIONSHIP BETWEEN $(\tau_{hv})_{av}/\sigma'_0$ AND RELATIVE DENSITY FOR KNOWN CASES OF LIQUEFACTION AND NON-LIQUEFACTION.

(From Seed and Peacock, 1970)

In Fig. 6.10, Seed and Peacock compare the stress ratios causing liquefaction in 25 cycles during laboratory simple shear and triaxial compression tests, on sands having a relative density of 50 percent, with the field results shown in Fig. 6.9. They show that there is substantial agreement among the data, provided the stress ratios in cyclic triaxial tests are multiplied by a factor of about 0.6, as indicated by considerations of the test limitations.

However, the position of the lower bound line in Fig. 6.9 is strongly influenced by the recorded data in the 1964 Niigata earthquake, in which it is reliably established that sand of a relative density of 70 percent liquefied at a computed stress ratio of 0.175, based on a recorded surface acceleration of 0.16g. Furthermore, the lower bound line in Fig. 6.9 must pass through the origin. Thus, although it is possible to question the precise location of the data points, and perhaps raise the lower bound line slightly, it is unlikely that its position could be changed significantly. Accordingly, it would appear to provide justification for the recommendation that triaxial cyclic load test data should be multiplied by a factor of the order of 0.6, as shown in the comparative data plots in Fig. 6.10.

A detailed analysis of the reasons why such a factor should be used has been presented by Seed and Peacock, 1970, based on an evaluation of the theoretical and practical deficiencies of the triaxial and simple shear test procedures. The authors note that considerable judgment is involved in making such an evaluation. While additional research of this type may lead to a better understanding of the deficiencies in the test procedures, and provide an improved guide for judgment, it is not likely to eliminate them.

Larger triaxial and simple shear samples and improved testing devices, supplemented by in situ testing, are not likely to prove feasible for routine testing in the near future. However, the elimination of previously known limitations in large

Fig.6.10 STRESS CONDITIONS CAUSING LIQUEFACTION.

(From Seed and Peacock, 1970) .

shake table devices is expected to open up an expanded research potential for laboratory studies of soil liquefaction phenomena.

The best hope for significant 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. Actual sites must be instrumented, the distribution and engineering properties of the soils determined, and the sites subjected to significant earthquakes. This is costly in units of both time and money. Until more reliable information of this type is obtained, a detailed site investigation combined with response studies, a laboratory investigation including cyclic load test procedures and an analysis of the type described previously in this chapter must be completed; and the results reviewed by competent engineers who can supply the necessary element of judgment, before a responsible decision can be made.

CHAPTER 7

SOIL SUBSIDENCE

Compaction or subsidence of a dry or unsaturated sand deposit, subjected to cyclic shear strains during earthquake loading may result in settlement distress to a structure founded on such a deposit. In practice, sandy soils are likely to be the most susceptible to this type of instability. In our experience, gravel deposits are usually dense in their natural state in the field and hence are not likely to be subject to vibration compaction. Because of their cohesive nature, clay soils that are acceptable for foundation support from other considerations, are not susceptible to compaction from vibrations. Creep movements may be initiated in sensitive clays by a series of vibrations or stress pulses, but such soils are not acceptable for other reasons.

Rearrangement of soil particles during seismic shaking of a sand deposit may result in volume changes in the soil deposit. If the soil is sufficiently loose there is a tendency for a volume decrease as the particles move into a dense packing. As discussed in the previous chapter on liquefaction, if the soil is saturated and drainage cannot occur during a reduction in volume, positive pore pressures will develop with an accompanying decrease in shear strength, potentially resulting in ultimate failure of foundation support. If, on the other hand, the soil is dry or sufficiently pervious such that excess pore water pressures can dissipate by drainage, a decrease in volume may cause excessive ground surface settlements. If the site is low and near a body of water, inundation could occur as a result of the settlement, in addition to unacceptable distress to structures founded on the site.

The factors of primary importance to a volume change tendency of a given cohesionless material have been previously pointed out in Chapter 6 to include: 1) relative density (or void ratio), 2) confining pressure, 3) cyclic shear stress or amplitude of shear strain, and 4) number of stress cycles (or duration of shaking).

Several researchers have reported on their investigations of some of these factors and on the general subject of sand densification during vibrations. In studies of densified sands in a vertically vibrating system with almost constant stresses, Whitman and Ortigosa, 1969, conclude that when dynamic stresses are small there is no densification at accelerations of less than 1.0g for confining stresses near zero. Youd, 1967, confirmed phenomena previously observed by other investigators (ie, Barkan, 1962) regarding the behavior of granular materials subjected to horizontal, steady-state vibrational displacements. Among the more important conclusions are that vibration reduces the coefficient of friction (hence, shear strength); and vibrational densification commences only after a certain threshold acceleration is exceeded.

Silver and Seed, 1969, studied the volume change characteristics of dry samples of medium quartz sand under cyclic loading simple shear conditions. The results have been used to develop a simple method for predicting vertical settlements due to compaction in layers of dry cohesionless soil subjected to seismic loading conditions, and the settlements computed by this method have been shown to be in reasonably good agreement with settlements observed in layers of sand subjected to horizontal base motions in shaking table tests.

Silver and Seed state that, "Measurements of the volume change behavior of sands under cyclic loading simple shear conditions showed that the vertical strain due to compaction is not significantly affected by values of vertical stress and depends only on the shear strain amplitude induced in the sample at shear strains exceeding 0.05 percent. This indicates that cyclic shear strain, which deforms the sample allowing particles to move into denser packing, may well be a fundamental parameter in determining the volume change behavior of cohesionless soil under dynamic loading conditions. On the other hand, it was found that as the vertical stress increases, the cyclic shear stress necessary to cause any degree of compaction also increases, a result in agreement with studies of the liquefaction potential of sands subjected

to cyclic loading."

Curves showing the relationship between vertical settlement and shear strain amplitude for a variety of conditions are given in Figs. 7.1, 7.2 and 7.3 from Silver and Seed. Figure 7.1 shows the influence of relative density on the vertical settlement of sand in 10 cycles of loading. In the upper range of cyclic strain of strong motion earthquakes (0.1%), a reduction in relative density from 80 to 45 percent increases the vertical strain due to compaction from 0.1 percent to 0.4 percent. Apparently the compaction is insignificant, even at a relative density of 45 percent in the lower ranges of cyclic shear strains (0.02%) as produced by strong motion earthquakes. Silver and Seed demonstrated that as much vertical settlement due to compaction occurs in the first 10 cycles of loading as occurs in the next 40 cycles, indicating the importance of the first few strain cycles in determining the total vertical strain.

In Figs. 7.2 and 7.3 are shown the influence on vertical settlement due to compaction, of variations in shear strain amplitude, vertical stress, and number of stress cycles for medium dense sand ($D_r=60\%$). These data show the significant influence of shear strain amplitude on the vertical strain and the relatively minor influence of variations in vertical confining stress at a given strain amplitude. The data show that across the full range of strain considered for strong motion earthquakes, the vertical compaction may increase one order of magnitude (from 0.01% to 0.1%) for a medium dense sand. Figs. 7.2 and 7.3 again illustrate the importance of the first few cycles of loading on the total amount of compaction, or subsidence.

A method for calculating the settlements of sand layers subjected to horizontal accelerations was also suggested by Silver and Seed, 1969, together with details of a typical analysis. For a variety of test conditions, they found calculated settlements to agree reasonably well with measured settlements of one foot high layers of sand subjected to horizontal shaking in shaking table tests. The authors further suggested that the proposed technique

Fig. 7.1 EFFECT OF RELATIVE DENSITY ON SETTLEMENT
IN TEN CYCLES

(From Silver and Seed, 1969)

Fig. 7.2 VERTICAL SETTLEMENT - SHEAR STRAIN RELATIONSHIP FOR DIFFERENT STRESS CYCLES ($D_r = 60\%$)

(From Silver and Seed, 1969)

Fig. 7.3 SETTLEMENT-VERTICAL STRESS RELATIONSHIP FOR MEDIUM DENSE SAND AT DIFFERENT SHEAR STRAIN LEVELS ($D_r = 60\%$)

(From Silver and Seed, 1969)

might be used as a first approximation for estimating the volume change tendencies and ground surface settlements induced in deposits of cohesionless soil caused by seismic shaking.

In view of the foregoing discussion, ground subsidence, due to compaction from earthquake vibrations, may be an important consideration for sites subjected to very strong motion earthquakes. There is significantly more volume change at higher strain amplitude, and for sands at relative densities below about 60 percent, than at lower and higher values, respectively. On the other hand, from a practical standpoint, to satisfy the static load requirements imposed by the major structures of nuclear power plants, loose soils are generally removed and recompacted to a greater density. This procedure, to a large extent, would eliminate the ground subsidence problem due to vibrations, at least for the important structures. While this does not eliminate the need to consider this potential problem in design, it is likely that other design requirements will automatically minimize damage resulting from this cause in many cases.

CHAPTER 8

SUMMARY AND RECOMMENDATIONS

In attempting to determine the influence of local soil conditions on earthquake-induced ground motion and to predict soil behavior under seismic loading, several complex factors must be studied:

- 1) Determination of the magnitude and characteristics of the earthquake to be considered.
- 2) Determination of local site and soil conditions.
- 3) Determination of dynamic soil properties.
- 4) Selection of a method of analysis.
- 5) Selection of models of the soil deposit.
- 6) Selection of models of soil behavior.

In this report, attention has been focused primarily on the determination of soil properties for use in modeling and analyzing ground response to earthquake shaking. Although empirical correlations have been used in the past to predict ground shaking, new more rational procedures consisting of a variety of analytical methods and models of soil behavior are now being applied to the problems of interest. As indicated in Chapter 2, these methods have in common basic input data requirements; in addition to a characterization of the earthquake excitation and a description of the soil deposit, the models require certain soil properties. One-dimensional models require descriptions of the shear behavior, the dissipative (damping) properties, and the density (unit weight). The more complex models may account for the dependence of these properties on confining pressure, strain amplitude, and previous loading history, but this complexity represents only a more detailed description of the same basic properties. When extended studies which account for two- or three-dimensional effects are undertaken, (or when two-dimensional methods are used to examine one-dimensional situations), a description of

the dilatational behavior not required by the one-dimensional models must also be obtained.

Since approximations are introduced at many points in the process of defining site conditions, soil properties, and models for analyzing a soil deposit, the uncertainties resulting from these approximations must be determined. Some of the areas of major concern are the determination of in situ soil properties for seismic response analyses, and improved understanding of liquefaction and the parameters controlling its occurrence. In the following sections, the state-of-the-art discussed in Chapters 2 through 7 is summarized, areas of uncertainty are identified, and study areas which are considered to offer the greatest promise for improving our state of knowledge, providing more reliable means for determining in situ conditions, and for evaluating the uncertainties, are described.

8.1 SOIL MODELS AND ANALYSIS TECHNIQUES

Although the objective of this report is to provide a summary of the state-of-the-art of subsurface site investigation procedures, laboratory and in situ measurement of static and dynamic soil properties, and the determination of soil behavior under conditions leading to large strain phenomena such as liquefaction and subsidence, an introductory discussion of the factors involved in determining earthquake shaking environments for reactor facility design purposes has been included. A more detailed examination of the similarities and differences between selected analysis methods is reported in SW-AJA, 1971.

The mathematical analysis can be considered as a three-step process: 1) describe the soil deposit (e.g., continuum, lumped parameter, or finite element), 2) describe the behavior of the soil (e.g., equivalent linear elastic with equivalent viscous damping, elastoplastic, nonlinear hysteretic, et cetera), and 3) obtain a description of the surface motion induced by specified input motion by applying an appropriate solution

technique (e.g., modal superposition, direct integration, integral transform and inversion). Using this view, the need to control a large number of variables in order to make valid comparisons is evident. A comparison of three soil deposit model/solution techniques using equivalent descriptions of soil behavior and identical input motions has shown that lumped parameter/modal superposition, continuum/integral transform, and finite element/direct integration schemes, with comparable resolution of the soil deposit layering, produce comparable results when one-dimensional models are considered.

The relative merits of several alternative descriptions of the energy dissipation or damping characteristics of soil are difficult to establish at this time because of the lack of strong motion records of soil response at two or more levels at a single location in a soil deposit. Accordingly, selection of models and analysis techniques must for the present be based on the engineer's preferences, or the relative cost of applying alternate methods. Under selected conditions, all are capable of providing satisfactory results.

The need for reliable descriptions of the earthquake excitation and of the properties of the soils comprising a prospective reactor site is also clear. The sensitivity of calculated ground motion to changes in soil model parameter values has been demonstrated when strain dependence of both modulus and damping properties is taken into account, either directly in a nonlinear model or indirectly with an iterative procedure and equivalent linear models, and when it is ignored in simpler models. However, establishment of accuracy requirements for soil property measurements demands that a program of calculations utilizing both simple models and models which account for the observed complex behavior of soils should be undertaken to help establish the degree of simplification which can be introduced without altering significant aspects of the computed soil response. Recordings from multilevel accelegraph stations or from a suitable laboratory simulation

study would also help considerably in this regard.

8.2 GENERAL SITE AND SOIL CONDITIONS

In the investigation and evaluation of power plant sites having soil overburden, it is necessary to recognize potential failure conditions as well as to predict the site response under earthquake loading conditions. To do so requires the accurate determination of the general subsurface conditions normally obtained using more or less routine foundation investigation procedures for structures under static load conditions. In general, these conditions include groundwater levels, soil classification, the stratification and lateral distribution of the soil deposit and the engineering properties of each layer down to bedrock or rock-like materials. The engineering properties determined for static considerations, but also used in dynamic studies, include the unit weight, the shear strength and where applicable, the relative density. All of these conditions and/or properties, with the exception of the relative density, can be determined with assurance and sufficient accuracy from an adequately planned field exploration and laboratory testing program. This program usually consists of making borings and geophysical surveys, taking samples and performing the appropriate laboratory tests and/or field measurements. The spacing of borings and the sampling interval is adjusted to achieve the accuracy necessary to define the geometry of the layers and of the entire deposit. Details of the procedures used to evaluate each of these conditions are treated more fully in Chapter 3.

The relative density of granular soils, used indirectly in response studies and as a primary parameter in liquefaction and subsidence investigations, is determined as a function of depth using procedures that have been the subject of considerable discussion in the soil engineering profession. Efforts to measure the density of granular soils by testing "undisturbed" samples in the laboratory indicate that disturbance effects seriously

limit accuracy, producing a significant scatter of data (Fig. 3.1a), allowing only average correlations to be established. The in situ measurement of compactness using penetrometers has been found to be a simpler and faster method providing reasonable evaluations of relative density for most deposits (Fig. 3.1c).

Some engineers believe that the static cone penetrometer gives a better indication of the engineering properties of sand than a dynamic penetration test such as the standard penetration test. However, the cone penetration test has one major limitation in that a sample is not retrieved for visual inspection. With this approach, small variations in sand density and grain size often produce very large changes in penetration resistance, masking average trends and making it more difficult to interpret the general engineering properties of the in situ deposit.

The standard penetration test also has limitations. The introduction of human factors and nonstandardized procedures or equipment in this test, and the fact that the penetration resistance varies significantly with the effective overburden pressure and the grain size characteristics, as well as the density, are among the major factors which influence the accuracy of this approach.

Since all penetrometer tests have similar limitations, it is our opinion that regardless of the method used to obtain the relative density of a given soil deposit only approximate values can be obtained. Because the standard penetration test is conducted in conjunction with routine subsurface explorations (where visual inspection of each sample is possible) and has been the subject of greatest study for evaluating the relative density of a soil deposit, we further conclude that this is the most practical method at the present time and will continue to remain a preferred procedure, at least in the foreseeable future. However, to assure that representative data are obtained, and that the test procedure is not drastically altered in actual field practice, as demonstrated by Casagrande and Casagrande, 1968, adequate field inspection by soils engineers is required.

Numerous correlations between relative density and penetration resistance, as described in Chapter 3, have been proposed for use in design. Of the various correlation techniques, the authors at this time support the Gibbs and Holtz correlation as the most representative for practical application.

Penetration tests should be supplemented by laboratory tests on undisturbed samples, to serve as a check against the relative density determined by penetration procedures. Finally, all of the test data should be reviewed and evaluated by the soils engineer as a basis for a final judgement as to the best values of relative density to be used in design. While it is likely that no two engineers will arrive at the same value of relative density for a given soil deposit, the range of values can be narrowed down to acceptable limits if sufficient determinations are made. An experienced foundation and soils engineer who is familiar with the equipment and procedures used can probably estimate the average relative density of a granular soil deposit with confidence within about ± 5 to 10 percent, which is adequate for many practical purposes.

For evaluating potential power plant sites, the use of average values of relative density rather than lower bound values in calculations is considered desirable because it allows engineers to assess the problems more realistically. To provide the necessary conservatism in the design for density variations from the average, an appropriate factor of safety or adjustment is best applied after an evaluation of the conditions causing liquefaction has been made. To improve the accuracy of relative density determinations, further studies are obviously needed in this area. Such work could consist of performing numerous tests using each available method and various types of equipment on a number of granular soil deposits, to compare and assess the effects of the pertinent variables of each test. A complete study of the variables in the standard penetration test itself is also considered in order. As an example, the effect of using

different weight drill rods could be evaluated. Also, the effect of hole-diameter using identical drilling procedures could be studied by varying the sizes of the cutting bits or augers.

If the non-standard variables (Appendix A) in this test are studied and assessed quantitatively for a given soil, in our opinion, design engineers would be more inclined to demand more standardized procedures and equipment than presently utilized. Since an experimental study of this magnitude can best be accomplished systematically, after careful planning, by a large government agency experienced in soil testing, we would recommend that the Waterways Experiment Station of the Corps of Engineers or the U. S. Bureau of Reclamation consider undertaking such a study.

8.3 DYNAMIC SOIL PROPERTIES

The two primary dynamic soil properties used in seismic response calculations are the shear modulus and the damping ratio. The shear modulus is usually determined by either in-hole field geophysical methods or by tests on specimens of soil in the laboratory. Meaningful values of damping can presently be determined only in the laboratory. Existing test procedures, their limitations, the availability of equipment, and the cost (where possible) of performing the tests for determining these properties are identified or described in Chapter 4. Both the shear modulus and the damping ratio are strain dependent as demonstrated by the data summarized in Chapter 5.

Confidence in our ability to determine the above properties adequately is strengthened by the general agreement between values for typical soils at similar strains found and reported by different investigators using different laboratory test procedures. Test results presented in Chapter 5 point out (where available) the influence of various factors on the values obtained. The range or scatter of test data for several of the soil types of interest is also presented to illustrate typical

values, as well as to indicate the accuracy with which values can be obtained by laboratory testing.

For practical problems, most of the conventional test procedures used for studying ground response under earthquake loading, are performed at strain levels either higher or lower than those of most earthquakes, requiring extrapolation of the data to adjust the modulus to the strains of interest (10^{-3} to 10^{-1} percent).

There exists a need for the development of an in situ test to determine the shear modulus at strains approaching earthquake magnitudes. Most in situ measurements now are only at very small strains using geophysical techniques. If a suitable in situ test or procedure to vibrate a zone of soil in the same manner and at strains approaching earthquake magnitudes could be developed, it would represent a great improvement over existing in situ methods. A modification of the cross-hole geophysical technique, using a large vibratory energy source with velocity transducers in closely spaced holes, appears to offer the most promising approach to the development of a new test. Development of a shear strain measurement technique in such a test is also essential. If a large strain field test were to prove too much more costly than existing techniques, its use in a few cases would serve to validate the present extrapolations from laboratory data. Verification by field measurements during actual earthquakes would also create confidence in existing laboratory methods.

There also exists a need for a conventional (or standardized) laboratory test for determining the shear modulus at relatively high strains using a device which most nearly duplicates earthquake loading conditions. The resonant column test and the cyclic triaxial test are currently used to evaluate the modulus for response calculations. In our opinion, the simple shear test offers even greater promise; it can provide more direct and more accurate modulus data with practical test procedures. Its present

major limitations, described in Chapter 4, can be overcome and a standardized test apparatus and test procedure developed.

As previously noted, there are, as yet, no tests for determining damping properties in situ. The only way in which meaningful field values of damping at strain levels of earthquake magnitude may be obtained for comparison with laboratory values is by recording, at different depths, ground motions from actual earthquakes or from large explosions such as an underground nuclear test. Such data are not available. A suitable measurement program, with recording stations set up at several selected sites near underground nuclear explosions could help considerably in defining field values of damping in soil.

A large vibrator or high energy impact source could also be used to obtain damping values; however, separation of signal decay due to radial propagation from that due to the dissipative behavior of the soil may be difficult. Material damping in soil is considered to be primarily a hysteretic process resulting from intergranular friction. When using localized explosive or vibratory sources of energy, hysteretic damping probably accounts for only a very small percentage of the total signal decay measured. Since the material damping is expected to be small and could only be obtained in such tests as the difference of relatively large numbers, the accuracy of such a procedure would be unsatisfactory. Development of methods for measuring damping in situ, therefore, does not appear entirely feasible at this time.

8.4 LIQUEFACTION

Much information about liquefaction of sand soils under earthquake loading conditions has been obtained in the past decade. During strong motion earthquakes, the steady buildup of pore pressure resulting from cyclic loading, is considered as the primary causative mechanism. Both laboratory investigations and field observations indicate that the liquefaction potential of a saturated sand deposit under earthquake excitation depends on

the relative density and other characteristics of the soil, the initial stresses acting on the deposit, and the strength and duration of the earthquake involved. Gravelly or clay soils are generally not susceptible to liquefaction.

Existing laboratory tests used to observe the liquefaction phenomenon, and to study the factors influencing its occurrence are identified in Chapter 6. Procedures presently available to the engineering profession for evaluating liquefaction potential at specific sites are described. These procedures include 1) the use of empirical rules based on observed field performance, 2) comparison of the site soil conditions and design earthquake characteristics with a similar site that has liquefied under known earthquake excitation, or 3) use of the general or simplified procedures described by Seed and Idriss, 1967 and 1970b.

Occasionally, a site may be found which so clearly satisfies all empirical rules or where both soil and seismicity conditions are so close to a known case of liquefaction that no further studies may be required. In other cases, however, the general or simplified analyses procedures provide the most suitable means for evaluating the liquefaction potential of a site.

Since more 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 undertaken until earthquake records in these areas are available. Therefore, advances in understanding and predicting the occurrence of liquefaction must, in the immediate future, rely on laboratory investigations.

For such investigations, two types of large scale laboratory tests--shake tables and large scale simple shear devices--are recommended as possible ways for studying liquefaction. A discussion of the problems associated with both types of equipment is presented in SW-AJA, 1971. Both devices provide means

for advancing the state of knowledge of liquefaction, using large samples to minimize boundary effects. As a result of recent work, the major limitations of shake table testing appear to be essentially resolved; it now appears that the shake table offers a better method for studying liquefaction than the large scale cyclic simple shear device. Thus it appears that future efforts in large scale laboratory tests for soil liquefaction should be conducted with shake tables. Should continued study and research demonstrate the need for development of a large scale simple shear device, further consideration should also be given to this alternate approach.

8.5 SOIL SUBSIDENCE

Ground vibration can induce compaction of cohesionless sand deposits and, consequently, settlement of the overlying ground surface. Excessive settlement may damage structures supported by this material and is obviously a condition to be avoided.

In most cases, sites with loose materials are usually considered inadequate for support of loads even under static load conditions. When only thin layers of loose material are encountered, the site is often renovated by removing and replacing or by recompacting the on-site soils to achieve a satisfactory density. Because of this practice, improved understanding of compaction and subsidence of loose soils is not a pressing requirement for nuclear power plant sites. Long range plans should, however, include research efforts covering this instability phenomenon.

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APPENDIX A

BORING EXPLORATIONS

DRILLING, SAMPLING, AND TESTING METHODS

APPENDIX A

BORING EXPLORATIONS

DRILLING, SAMPLING, AND TESTING METHODS

In the investigation and evaluation of potential nuclear power plant sites, it is usually necessary to make borings, take samples, install groundwater measuring devices, and in most cases conduct special tests in the borings. To obtain the best possible data from the borings, in the most efficient manner, requires careful consideration of both the drilling equipment and the samplers to be used. Also, suitable groundwater measuring devices must be selected and installed in an appropriate manner in order to accurately establish the groundwater conditions at the site.

Because the boring phase of a field investigation is one of the first steps, and of primary importance to site selection, it is included herein as an appendix since, although it is recognized that much of this information may be familiar to many soils engineers, it may be very unfamiliar to those who are not soils or foundation engineers.

In the first section of this appendix is a brief introduction to drilling equipment currently used for making soil investigations. Drilling methods are discussed in general and an assessment given of their application to various soil types. The next two sections discuss the types of soil samplers most widely used and their general applicability to site investigations. The final two sections discuss other in-hole tests or methods as well as groundwater measuring devices and installation procedures which are also available as engineering tools for evaluating the subsurface conditions.

A-1 DRILLING EQUIPMENT

The most usual method of subsurface exploration consists of drilling holes, generally 3 to 12 inches in diameter, into the ground and extracting samples at an appropriate depth interval for both identification and testing (Hvorslev, 1949). For this purpose, powered drilling rigs mounted on truck beds are in most common use. The hole is generally advanced by either an auger (Fig. A-1), a percussion drill (Fig. A-2), or a grinding bit (Fig. A-3). Variations of these methods are available for special purposes, in which equipment has been developed for improved techniques in certain soil conditions; also, larger diameter holes may be made for direct inspection and testing.

a. Auger Drilling

Heavy powered continuous flights of either hollow or solid auger can be used for drilling holes 6 to 10 inches diameter, both through stiff cohesive soils and some dense granular soils to depths approaching 200 feet. Auger drilling is particularly well-suited for advancing borings in partially saturated materials above the groundwater level, especially when undisturbed samples are to be obtained of these materials.

Hollow-stem augers have several advantages over solid-stem augers and have been extensively used, particularly on the West Coast, since the early 1960's. Solid augers can be used only in cohesive or slightly cemented soils where caving of the hole is not a problem, since the entire auger must be removed from the hole each time before a sample can be taken. Soils most susceptible to caving are clean, uncemented sands and gravels, especially when located below the water table. Hollow augers, on the other hand, can be used in nearly all soil conditions (the exception being in very dense, coarse gravel and cobble deposits) since the auger is not removed from the hole during sampling, and the walls are automatically protected against caving. Furthermore, determination

a. DRILL RIG

b. AUGER FLIGHTS

Fig. A-1 AUGER DRILL EQUIPMENT

(From Hvorslev, 1949)

NOTE THE SPUDDING ARRANGEMENT SHOWN HERE IS ONLY ONE OF MANY IN ACTUAL USE

REGULAR MOTHER HUBBARD TWISTED

b. PERCUSSION BITS

a. PERCUSSION DRILLING

DART VALVE DISK VALVE

c. BAILERS

FIG. A-2 PERCUSSION DRILL EQUIPMENT

a. ROTARY DRILLING

TWO-BLADE BIT

THREE-BLADE BIT

FOUR BLADE BIT

TWO-CONE BIT

TRI-CONE BIT

ROLLER BIT

FISHTAIL BIT

b. ROTARY BITS

Fig. A-3 ROTARY DRILL EQUIPMENT

(From Hvorslev, 1949)

of the free groundwater level is also facilitated with the hollow-stem auger. Samples are generally removed by long drill rods which extend through the continuous hollow stem to the bottom of the bore hole. The easier to handle drill rod in the hollow auger generally allows a much faster rate of drilling and sampling than with a solid-stem auger. This factor, together with the more important fact that no casing is required, make the hollow-stem auger much more versatile than the solid-stem auger. Neither type of auger can be advanced through hard rock.

The hollow-stem auger is a fast and efficient drilling tool for a wide range of soil conditions. Special care is required in very soft soils and in granular soils below the water table, to ensure minimum disturbance and to prevent heave into the auger at the bottom of the boring. Satisfactory results for both conditions are obtained on a routine basis by experienced and careful drillers, and with thorough inspection procedures.

The commercial price for making a boring with a hollow-stem auger ranges from \$5 to \$8 per foot, or hourly between \$25 to \$50, plus mobilization charges. Special sampling techniques or unusual job requirements further increase this cost. In average soil conditions, the boring may be advanced routinely at a rate of 10 to 15 feet per hour.

It is sometimes desirable and useful to make large-diameter holes for direct inspection of the soil conditions and for in-place tests. Holes approaching 5 feet in diameter may be made with bucket, table, or disk augers quite rapidly, even in stiff and dense soils to depths in excess of 100 feet. Such holes or shafts are most effective for determining detailed information above the water table, i.e., mapping in situ soil profile, plate bearing tests, density tests and obtaining undisturbed chunk samples. For personnel safety, these shafts are usually cased with telescoping steel sections of decreasing diameter.

Equipment for drilling large-diameter holes is readily available on a competitive basis in most large metropolitan areas

throughout the United States. The commercial price for drilling these holes, including standby time for detailed inspection and in-place tests may range from \$100 to \$200 per foot, plus mobilization costs. In average soil conditions, these holes may be advanced at a rate of about 5 feet per hour, depending upon time required for inspection and tests.

b. Percussion Drilling

Percussion drilling can be used to advance borings in most types of soil or rock. It is particularly useful for borings through dense gravel, cobble and boulder deposits. For soil sampling operations, most of these drilling rigs, usually called "churn-drill" or "cable-tool" rigs, are generally capable of advancing holes to depths in excess of 200 to 300 feet. However, the average rate of drilling with this method, in dense materials, rarely exceeds 2 feet per hour. Thus it is very time-consuming, but nevertheless consistently dependable for advancing a boring through virtually all conditions.

Percussion drilling consists of alternately lifting and dropping of a chopping bit, thus essentially pounding a hole into the soil. Cuttings generated from the continuous pounding are removed from the hole with either a bailer device (churn drill method), as shown typically in Fig. A-2, or by continuously circulating water or air down the drill rod, through the end of the chopping bit and out of the hole around the outside of the drill rod. This latter method (wash boring method, see Terzaghi and Peck, 1967), employs the same circulating techniques as the rotary method described in the following section. In materials where caving is apparent, pipe casing is advanced as drilling progresses.

Except at sites where dense gravels and cobbles are anticipated, percussion drilling methods are not normally selected for foundation exploration purposes. This is due primarily to the slow rate of drilling, and potential disturbance caused by advancing the hole. The heavy pounding action from the string of tools,

which may weigh 1000 to 2000 pounds, dropping on the drilling bit is likely to disturb and compact the soil at the bottom of the hole, especially in sands and gravels, which may make the reliability of data obtained from subsequent "undisturbed" samples taken from the bottom of the bore hole somewhat questionable. (In dense soil deposits, this problem is not likely to be significant.) Also, largely because of the dense and coarse-grained nature of the soil conditions for which this method is best suited, sampling techniques are not "standardized", making comparisons with other sites more difficult.

The commercial price for borings made with a percussion drill is approximately \$5 to \$8 per foot plus mobilization costs, comparable to the hollow-stem auger. However, for conditions in which the auger is also capable of drilling, soil inspection costs are much greater for the percussion drill, possibly by 500 percent. This may be reduced considerably by using two rigs, where possible, under one inspector.

A special percussion drilling method which has found widespread use since about 1968, is the Becker Method, by which sand, gravel and boulder formations can be penetrated rapidly and efficiently. With the Becker drill, a double-wall drive pipe is driven by a diesel-operated pile hammer; while air or water, under pressure, is forced down the annulus of the drive pipe. The material cut by the drill bit is rapidly transported to the surface through the inside pipe by the drilling fluid. The discharged materials emerge into a cyclone where the samples can be obtained for logging the soil profile.

The Becker drill is capable of advancing a boring at rates in excess of 10 feet per hour. Disturbed samples for soil classification are easily obtained as the boring advances. Experience with this equipment indicates a practical depth limitation of 150 feet, although information provided by the manufacturer indicates that greater depths are possible with an extra compressor. In our earlier experiences with this rig, there was no capability provided

for making penetration tests or for obtaining undisturbed tube samples. However, recent literature indicates that this may now be possible.

c. Rotary Drilling

The rotary drilling method is used for advancing borings, generally 4 to 8 inches in diameter, through most types of soil and rock. The hole is made by rotating and pushing a cutting bit into the soil at the bottom, subsequently loosening this soil and grinding it into fine-grained cuttings. These cuttings, or loosened soil particles, are then removed by circulating a drilling fluid, either water, air, or drilling mud from a sump, down through the drill rods, out of the cutting bit and out of the hole around the outside of the drilling rod. The stability of the hole is maintained by either advancing steel casing within the hole or by using a heavy drilling fluid. Where pervious soils are encountered, circulation of the drilling fluid is maintained by pumping chemicals into the hole which stabilize into a gel with time; or with drilling mud, usually a suspension of bentonite of a creamy consistency and a specific gravity greater than water, coating the walls of the bore hole with an impervious membrane. Unfortunately, the use of drilling mud or gel eliminates the possibility of determining the piezometric or groundwater levels corresponding to the various pervious strata through which the hole may pass.

This method of drilling has the advantage of being able to advance holes to considerable depths at a more rapid rate than the percussion drilling method, and of being able to recover readily both disturbed and undisturbed samples. However, rotary drilling is not well suited for use in deposits containing very coarse gravel, numerous stones and boulders, or chert nodules; or very porous deposits with a strong groundwater flow, since an excessive amount of drilling fluid may be lost by seepage (Hvorslev, 1949). Under average drilling and sampling conditions in soils, the rate of drilling in a normal work day is about 40 to 50 feet using the

rotary method. The commercial price for rotary drilling is competitive with the hollow-stem auger, ranging in the order of \$5 to \$8 per foot, plus mobilization charges.

A-2 UNDISTURBED SAMPLING

Soil cuttings are generally inadequate to define any more than the general classification characteristics of soil. Thus, reasonably undisturbed samples should be obtained, where possible, at appropriate depths for detailed classification analysis and testing in the laboratory. Where this is not possible, disturbed samples may be obtained by driven samplers, discussed in the next section.

The conventional method for obtaining an undisturbed sample is to place a thin-wall steel tube on the end of the drill rods and force it, preferably in one rapid continuous motion, into the soil at the bottom of the boring (ASTM Designation: D1587-67). These tubes are typically about 2 to 3 feet long and generally range in diameter between 2 and 6 inches, with the 3-inch O.D. size being most standard. Piston samplers, and core barrel samplers, are special samplers and are used for soils difficult to sample (either very loose and soft, or of variable hardness in the case of the latter sampler), but basically they also use thin-walled tubes which are pushed into the soil at the bottom of the bore hole. Sketches of these more or less typical undisturbed soil samplers are shown in Fig. A-4a. Foil samplers are also available and use long strips of foil to encase the sample (Terzaghi and Peck, 1967). These samplers are used primarily in fine-grained silt and clay soils. The core barrel, or Pitcher sampler in Fig. A-4a is particularly useful in strata that are alternately hard and soft. In the harder soils, the Pitcher sampler cuts through strata, rather than shearing through them, allowing the sample to slide upward into an inner thin-wall tube for retention. In soft zones, the tube is forced ahead of the cutter barrel by a spring.

CONVENTIONAL
TUBE SAMPLER

PISTON SAMPLER

CORE BARREL
SAMPLER

a. TYPICAL UNDISTURBED SOIL SAMPLERS

TYPICAL CONE PENETROMETERS

SPLIT SPOON SAMPLER
FOR STANDARD PENETRATION TEST

b. PENETROMETERS

FIG. A-4 TYPICAL SOIL SAMPLERS AND PENETROMETERS

Undisturbed samples are most readily taken in silt and clay deposits, but with special samplers and careful techniques, undisturbed samples can also be obtained in sand deposits. Gravels and clean sands are often not sampled, using undisturbed techniques, because these soils either bend the edges of the thin-wall tube or fall out of the open end of the tube, upon extracting the filled tube from the bore hole, particularly below the water table. Freezing methods have been utilized with a limited degree of success in clean sands, and also special samplers like the Bishop sampler (Terzaghi and Peck, 1967) have been developed for sampling clean sands below the water table. While all of these sampling methods are used with a variety of success, no one sampler can be said to be best for all conditions. Thus, it is necessary that the soils and foundations engineer responsible for determining the engineering soil properties be given wide latitude to utilize the sampler and techniques best suited to the conditions encountered. These soil materials are highly sensitive to disturbance, as loose cohesionless materials tend to densify or compact upon sampling, and dense materials tend to loosen somewhat (Waterways Experiment Station, 1962). Thus, for example, measured densities of granular materials, because of disturbance in sampling, should be used only to establish average trends, since any one measurement may give misleading results. Fortunately, in most problems in practice, sufficiently reliable information concerning the properties of sands below the water table can be obtained indirectly from penetration tests.

A-3 PENETRATION TESTS

Because adequate measurements of the degree of compactness or relative density of granular soil materials underlying a potential site cannot be evaluated with confidence from "undisturbed" samples, various types of penetrometer, or penetration resistance tests have become substituted as an indirect method to determine this important property in situ. The distinct advantage

of the penetrometers lies in their simplicity, allowing a large number of tests to be performed as a part of the boring operation at a minimum cost. The soil density and the state of effective stress are the two most important variables influencing the results of these tests.

There are two basic shapes of penetration-type samplers in common use, namely the cone penetrometers and the hollow tube samplers, Fig. A-4b. Both types of penetrometers may be either driven with the impact of a falling hammer weight (dynamic method), or pushed by static pressure (static method) into the undisturbed soil at the bottom of a boring as drilling progresses. Using the dynamic procedure, the number of blows required to drive the sampler a given distance into the soil is recorded and, based on the corresponding vertical effective stress present at that depth, may be correlated with the relative density or approximate compactness of the soil, using available curves or equations empirically developed for that purpose (see Chapter 3).

The two most widely used dynamic penetrometer tests are the standard penetration test in which a sample is obtained, and the dynamic cone penetration test in which no sample is obtained. The standard penetration test (ASTM Designation: D1586-67) consists of driving a 2-inch O.D. split steel tube into a soil deposit with a 140-pound hammer, freely falling 30 inches. With this standardized method, the blow-count, or N-value, is obtained for one foot of penetration after an initial penetration of 6 inches at any desired test depth interval. The dynamic cone penetration test involves driving an approximately 2-inch diameter cone, 12 inches with a 120- to 160-pound hammer, again falling freely 30 inches. The standard penetration resistance is roughly found to be 50 percent of the cone penetration resistance, when the same driving energy is used.

Of the two dynamic tests, the standard penetration test is more widely used, since the cone penetrometer test does not permit visual identification of the soil penetrated. The sample

recovered from the standard penetration test is, however, disturbed and normally used only for classification purposes. In addition to the standardized variables previously noted, there are also many non-standardized factors which influence the performance of the test, and which can have important effects on the measured penetration resistance. Those variables considered important (Fletcher, 1965; Bazaraa, 1967; Ireland, Moretto and Vargas, 1970) include the following:

- 1) Upward seepage of water into the hole
- 2) Inadequate cleaning of the bore hole
- 3) Mechanism controlling drop of hammer
- 4) Weight and length of drill rods
- 5) Diameter of bore hole
- 6) Interval recorded for blow count
- 7) Details of the drop weight
- 8) Drilling method

Many of these variables rely heavily on the boring contractor, since he provides the drilling equipment and is actually responsible for advancing the boring and performing the penetration resistance tests. A significant review of the pitfalls and potential sources of error in this test, and the great influence of equipment and procedures on the penetration resistance values obtained is given by Casagrande and Casagrande, 1968. These authors further support their discussion by presenting a comparison among standard penetration resistance values measured in four borings placed in close proximity to each other. These borings were drilled with four different types of equipment and by two different reputable contractors and competent drilling foremen. The results showed large average variations in penetration resistance (by as much as 300 percent), even though the soil profiles were essentially identical.

Variations in the deposit itself can likewise produce significantly different penetration resistances. These factors

include variations in cementation, particle size, particle gradation, effective confining pressure, and density; the latter two being the most important variables as previously noted. Because of the influence of so many variables, published penetration resistance values obtained from different sources with different drilling equipment can be compared and utilized only to establish approximate correlation. A great deal of judgement is thus required of the soils engineer obtaining the data at a particular site, to correctly interpret the data in light of all of these variables.

Due to the many variables which make the results far from standard, the standard penetration test has undergone considerable criticism. However, it is reasonably well accepted that even though these variables exist, if the test is properly performed and the effects of the potential sources of error recognized, the results obtained still provide the best method for estimating the relative density or compactness of granular soils as a function of depth. Again, the results should be used only to estimate average values rather than actual values based on any one test. The most commonly used correlations to evaluate the relative density from N values are discussed in Section 3.2.

Because of its simplicity, the standard penetration test has also been used to estimate the strength of clay soils. However, the interpretation of penetration resistance data vs. strength should be considered as only a crude estimate, since the compression behavior of such soils is time dependent. Also, since undisturbed samples of clay soils can be readily obtained and tested, more reliable methods are available in the laboratory for determining the shear strength, thus eliminating the variables that may be introduced by the penetration test.

Of the various static penetrometers, the Dutch cone has been most widely used. This procedure consists of pushing a 1.4-inch diameter cone, with a 60-degree point, under a steady rate slowly into the soil and measuring its resistance to penetration.

Based on a statistical study of mostly uniform fine sands, but including some silty and medium sands, the ratio of the static cone penetration resistance, Q_c (kg/cm²) to the standard penetration resistance (blows/foot) has a mean value between 4.0 and 4.5 (Schmertmann, 1970). This ratio has been found to vary with the grain size, increasing from unity for fine-grained soils to 18 for some gravelly soils. For the fine sandy materials reported by Schmertmann, the effect of the magnitude of N-value on the correlation is as follows:

<u>Blow Count Range</u>	<u>Correlation</u>
0 < N < 10	$Q_c = 4.9N$
0 < N < 30	$Q_c = 4.1N$
All N	$Q_c = 18.3 + 2.9N$

As mentioned previously, the cone test, whether using dynamic or static loading, has the distinct disadvantage of not recovering a soil sample for identification purposes. Because the force for penetration is applied statically, however, the test does have the advantage over dynamic tests of having fewer human and mechanical variables involved in applying the dynamic impact loads. Results using the static test, therefore, will generally be more consistent (i.e., with less scatter of test data) than the other tests. Even though all penetrometer tests are empirical and have a number of variables, causing a scatter of data, they presently are the best procedures for estimating the average relative density or compactness of a soil deposit in situ as a function of depth.

Within the United States, the standard penetration test is the penetrometer test method most widely used by drilling companies, testing laboratories, and soils engineering firms. Commercially, the cost of performing this test is about \$4 to \$7 per test in addition to the cost of advancing the boring. The

dynamic and static cone tests, on the other hand, are not as readily available in the United States, but are very common worldwide.

A-4 OTHER IN-HOLE TESTS

In addition to obtaining both disturbed and undisturbed samples of soils from bore holes, other tests may be performed to determine the engineering properties of the local soils in situ. Such tests include the determination of shear strength and deformability, by means of the vane shear test and pressuremeter test, respectively. While the data thus obtained may not be of direct use to site response analyses, they are useful in the evaluation of foundation behavior for static loading requirements and further, with these properties, more precise determination of the required parameters may be made.

The vane test consists of pushing four rectangular vanes into the soil, twisting the vanes and directly measuring the shear resistance (strength) of the soil (Terzaghi and Peck, 1967). This device is generally limited only to soft or medium stiff clays. The main advantage of the test is that it gives more realistic values of the in situ shear strength than laboratory tests, especially if the soils are extremely sensitive. Its main disadvantage is that it gives accurate values only for purely cohesive materials such as plastic clays. Silty materials having both cohesion and friction may also be tested; however the results obtained generally underestimate the true shear strength.

The pressuremeter and borehole jack tests are two of several types of tests that may be performed to estimate the deformation modulus and ultimate bearing capacity (strength) of a soil deposit in situ, as a function of depth. Such tests (for example, the Goodman bore hole jack) were primarily designed for rock, but they have been successfully utilized for "soft" rock and very firm soil conditions, as well (Goodman, Van and Heuze, 1968; and

Greenland, 1964). The pressuremeter apparatus consists of a metal cylindrical probe approximately 30 inches long and 3 inches diameter, covered with a rubber membrane. The test is performed by inserting the probe into the bore hole to the test level and inflating the membrane against the test hole walls by water under pressure. The modulus and bearing capacity are derived directly from the resulting pressure deformation curve. Both of the tests provide useful data, and the cost of testing is comparable to an equivalent amount of bore hole sampling and laboratory testing. Many small tests of this type may be performed in a bore hole at considerably lower cost than that for standard plate bearing tests of small loadings at shallow depths in simple conditions. One disadvantage, however, may be the unknown condition and deformation behavior of the bore hole wall being tested. The pressuremeter test measures volumetric strain for any given pressure, which may or may not be uniform around the walls of the bore hole, while the Goodman bore hole jack is designed to measure deformation modulus in a known orientation across the bore hole.

Results from any in-hole test procedure may be influenced significantly by local conditions created in making the hole, and may bear no relationship to the correct value of the property measured. Such tests should thus be carefully supervised and interpreted by competent soils and foundation engineers.

A-5 GROUNDWATER DATA

Correct groundwater data are of basic importance to all soils and foundation evaluations. The depth to the water table can be measured accurately and directly at any location on a site at a minimal cost, by either installing a piezometer or an observation well in a boring at that location following drilling (see Fig. A-5). The actual selection of the tip assembly to be installed depends upon the permeability of the soils encountered. For thick deposits of highly permeable materials, an ordinary well-point tip or any of the common tip assemblies noted in Fig. A-6 will be adequate;

a) OBSERVATION WELL
DETAILS

b) PIEZOMETER AND
PORE PRESSURE
WELL DETAILS

Fig. A-5 GROUNDWATER OBSERVATION DETAILS

a) POROUS STONE

b) HYDRAULIC *

c) ELECTRIC *

d) AIR ACTUATED

Fig. A-6 GROUNDWATER OBSERVATION TIP ASSEMBLY
DETAILS

*(From Terzaghi & Peck, 1967)

hence making the cost the governing factor. On the other hand, for relatively impermeable materials, pore pressure devices requiring very small changes in water volume are desirable. With these devices, the water level is determined by actually measuring the pressure in the pore water; hence, the hydrostatic pressure and effective groundwater level may be established with very little volume of water required. The last three tip assemblies in Fig. A-6 are used mostly for cohesive soils, while the porous stone tip assembly is commonly used in granular soils and some silts.

In some soil formations, the water within the soil mass is not continuous, but is trapped or "perched", or is subjected to an excess hydraulic head within an aquifer, continuous with some higher source of water, creating what is known as an artesian condition. This condition generally occurs in a pervious zone when an impermeable soil lies above and below the pressure layer. If such a condition prevails, the water level in the pervious layer, or the excess pressure can also be measured accurately by installing a tip within this pervious material and sealing it off from the other zones or layers as shown in Fig. A-5b.

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APPENDIX B

A SIMPLIFIED PROCEDURE
FOR EVALUATING
SOIL LIQUEFACTION POTENTIAL

by

H. Bolton Seed and I.M. Idriss

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A SIMPLIFIED PROCEDURE FOR EVALUATING SOIL LIQUEFACTION POTENTIAL

by

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Catastrophic failures in recent earthquakes have provided a sobering reminder that liquefaction of sandy soils as a result of earthquake ground shaking poses a major threat to the safety of civil engineering structures. Major landslides (25), lateral movements of bridge supports (6,24), settling and tilting of buildings (19,26), and failure of waterfront retaining structures (1,8,30) have all been observed in recent years as a result of this phenomenon and efforts have been increasingly directed to the development of methods of evaluating the liquefaction potential of soil deposits. It is the purpose of the present paper to describe a simplified procedure for evaluating liquefaction potential and to compare the results obtained by the method with a number of cases in which liquefaction is known either to have occurred or not occurred in the field.

It should be noted at the outset that the term liquefaction as used in this paper describes a phenomenon in which a cohesionless soil loses strength during an earthquake and acquires a degree of mobility sufficient to permit movements ranging from several feet to several thousand feet. When the term was originally introduced it was intended to describe a phenomenon in which a soil could undergo large movements, as in flow slides, with little or no resistance to movements. However failures due to limited movements of several feet in recent earthquakes have been

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attributed to "liquefaction." While the term 'cyclic mobility' might be more appropriate to describe this type of soil behavior, the broader use of the term "liquefaction" is adopted in the following pages.

In addition, considerations of liquefaction are limited to cases of relatively level ground where the response of the soil to stresses induced by an earthquake is not further complicated by the presence of initial horizontal shear stresses due to the proximity of significant surface irregularities.

Factors Known to Influence Liquefaction Potential

Both laboratory investigations and observations of field performance have shown that the liquefaction potential of a soil deposit due to earthquake motions depends on the characteristics of the soil, the initial stresses acting on the soil and the characteristics of the earthquake involved. The significant factors include:

(1) The soil type

For cohesionless soils, the soil type is perhaps most easily characterized by the grain size distribution. There is some evidence to show that uniformly graded materials are more susceptible to liquefaction than well-graded materials (Ross, et al. (24); Lee and Fitton (16)) and that for uniformly graded soils, fine sands tend to liquefy more easily than do coarse sands, gravelly soils, silts or clays (16). In a study of bridge foundation displacements in the Alaska earthquake, for example, Ross, et al. noted that there were no cases of bridge damage due to this cause for structures supported on gravels but there were many cases of damage for bridges supported on sands. Again in the Fukui earthquake, Kishida (11) notes that liquefaction occurred at a site where the upper 15 ft consisted of medium sand but there was no liquefaction at an adjacent site

where the soil in this depth range was a sandy silt. These field observations are supported by the results of laboratory cyclic load tests on a wide range of materials (16).

(2) The relative density or void ratio

Since the classical work of Casagrande (2) on the volume changes accompanying shear deformations in cohesionless soils, it has been generally recognized that the susceptibility of a given soil to liquefaction will be determined to a high degree by its void ratio or relative density. In any given earthquake loose sands may liquefy but the same materials in a denser condition may not.

In the city of Niigata, Japan in 1964, for example, liquefaction was extensive where the relative density of the sand was about 50 percent but it did not develop in areas where the relative density exceeded about 70 percent. Laboratory test data of all types show the important influence of this factor on soil behavior.

(3) The initial confining pressure

There is considerable evidence to show that under earthquake loading conditions, in contrast to flow-slide susceptibility under static load conditions, the liquefaction potential of a soil is reduced by an increase in confining pressure. Laboratory tests by numerous investigators (4,5,16,17, 18,21,27) have shown that for a given initial density, the stress required to initiate liquefaction under cyclic load conditions increases with the initial confining pressure. The effect was also shown in the field during the Niigata earthquake where soil under a 9 ft fill remained stable but similar soils surrounding the fill liquefied extensively (26).

(4) The intensity of ground shaking

For a soil in a given condition and under a given confining pressure, the vulnerability to liquefaction during an earthquake depends on the magnitude of the stresses or strains induced in it by the earthquake; these in turn are related to the intensity of ground shaking. The significance of the applied stresses has been shown by many laboratory investigations but the important effect of ground shaking intensity in the field is very well illustrated by the soil behavior at Niigata in Japan. Records of earthquakes for this city extend back over a period exceeding one thousand years, and estimated values of maximum ground accelerations for earthquakes affecting the city in the past 370 years are shown in Fig. 1. These values were determined from the relationships between earthquake magnitude, distance from source of energy release and maximum ground accelerations suggested by Housner (9) with appropriate corrections for the effect of local soil conditions based on the recorded ground acceleration of 0.16g in the 1964 earthquake. Magnitudes and epicenter locations for the earthquakes affecting Niigata have been presented by Kawasumi (10).

It may be seen in Fig. 1 that although the relatively loose sands in Niigata have been shaken by 25 earthquakes in the past 370 years, historical records show only 3 occasions on which liquefaction has been reported in or near Niigata itself; on these occasions the estimated ground accelerations were in excess of 0.13g, culminating with the extensive liquefaction in 1964 when ground accelerations had their probable maximum value of 0.16g. Of special significance, however, is the fact that in 22 other earthquakes producing estimated ground accelerations ranging from 0.005g to 0.12g, there was no indication of any soil liquefaction in the city

Fig. 1 ESTIMATED MAXIMUM GROUND ACCELERATION IN NIIGATA.

(Kawasumi (10)). The intensity of ground motions must thus be considered an important factor in evaluating soil liquefaction potential.

(5) Duration of ground shaking

The duration of ground shaking is a significant factor in determining liquefaction potential since it determines in a general way the number of significant stress or strain cycles to which a soil is subjected. All laboratory studies of soil liquefaction under cyclic loading conditions show that for any given stress or strain level, the onset of liquefaction depends on the application of a requisite number of stress or strain cycles. In the field, the importance of this is perhaps best illustrated by the landslides which were triggered by liquefaction in Anchorage during the Alaska earthquake of 1964. These slides did not occur until about 90 seconds after the earthquake motions started (22) indicating the need for development of sufficient stress cycles to induce liquefaction and instability. Clearly if the duration of ground shaking had been only 45 seconds, no liquefaction or soil instability would have developed.

General Method of Evaluating Liquefaction Potential

In view of the evidence that the five factors discussed above have a significant influence on the liquefaction potential of any soil deposit, it is apparent that any method for evaluating liquefaction potential should take these factors into account. Accordingly, Seed and Idriss (26) proposed a general method for evaluating liquefaction potential involving the following steps:

- (1) After establishing the soil conditions and the design earthquake, determine the time history of shear stresses induced by the earthquake ground motions at different depths within the deposit.

- (2) By appropriate weighting of the stress levels involved in the various stress cycles throughout the earthquake, convert the stress history into an equivalent number of uniform stress cycles and plot the equivalent uniform stress level as a function of depth as shown in Fig. 2. By this means the intensity of ground shaking, the duration of shaking and the variation of shear stress with depth within the deposit are taken into account.
- (3) By means of laboratory soil tests on representative samples, conducted under various confining pressures, determine the cyclic shear stresses which would have to be developed at various depths to cause liquefaction in the same number of stress cycles as that determined in step (2) to be representative of the particular earthquake under consideration. Either cyclic load triaxial compression tests or cyclic load simple shear tests may be used for this purpose and the interpretation of the test data to determine the stress conditions causing liquefaction in the field has been discussed elsewhere (Seed and Peacock (29)). However in this way, the soil type, the in-place condition and the initial effective stress condition may be appropriately taken into account; the stresses required to cause failure may then be plotted as a function of depth as shown in Fig. 2.
- (4) By comparing the shear stresses induced by the earthquake with those required to cause liquefaction, determine whether any zone exists within the deposit where liquefaction can be expected to occur (induced stresses exceed those causing failure).

Fig. 2 METHOD OF EVALUATING LIQUEFACTION POTENTIAL

Fig. 3 DETERMINATION OF MAXIMUM SHEAR STRESS

In applying the method, the stress history at various depths in a deposit can best be determined by a ground response analysis (26), possibly conducted in steps to take into account the changing deformation characteristics of the soil as the pore pressures build up (28). In most cases this latter refinement is unnecessary since the soil properties do not change appreciably until liquefaction is imminent. Even so, a ground response analysis may involve techniques, skills and equipment which are not readily available. In addition, the conduct of cyclic load tests may present difficulties.

Accordingly in applying the method over a period of time, simplified techniques have been developed for evaluating induced stresses and soil characteristics which are sufficiently accurate for many practical purposes. These techniques are described in the following pages.

Simplified Procedure for Evaluating Stresses Induced by Earthquake

The shear stresses developed at any point in a soil deposit during an earthquake appear to be due primarily to the upward propagation of shear waves in the deposit. If the soil column above a soil element at depth h behaved as a rigid body and the maximum ground surface acceleration were $a_{\max}$, the maximum shear stress on the soil element would be

$$(\tau_{\max})_r = \frac{\gamma h}{g} \cdot a_{\max} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where γ is the unit weight of the soil, see Fig. 3(a). Because the soil column behaves as a deformable body, the actual shear stress at depth h , $(\tau_{\max})_d$, as determined by a ground response analysis will be less than $(\tau_{\max})_r$ and might be expressed by

$$(\tau_{\max})_d = r_d \cdot (\tau_{\max})_r \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where r_d is a stress reduction coefficient with a value less than 1. The variations of $(\tau_{\max})_r$ and $(\tau_{\max})_d$ will typically have the form shown in Fig. 3(b) and in any given deposit, the value of r_d will decrease from a value of 1 at the ground surface to much lower values at large depths, as shown in Fig. 3(c).

Computations of the value of r_d for a wide variety of earthquake motions and soil conditions having sand in the upper 50 ft have shown that r_d falls within the range of values shown in Fig. 4. It may be seen that in the upper 30 or 40 ft the scatter of the results is not great and for any of the deposits, the error involved in using the average values shown by the dashed line would generally be less than about 5%. Thus to depths of about 40 ft, a reasonably accurate assessment of the maximum shear stress developed during an earthquake can be made from the relationship

$$\tau_{\max} = \frac{\gamma h}{g} \cdot a_{\max} \cdot r_d \quad (3)$$

where values of r_d are taken from the dashed line in Fig. 4. The critical depth for development of liquefaction, if it is going to occur, will normally be in the depth covered by this relationship.

The actual time-history of shear stress at any point in a soil deposit during an earthquake will have an irregular form such as that shown in Fig. 5. From such relationships it is necessary to determine the equivalent uniform average shear stress. By appropriate weighting of the individual stress cycles, based on laboratory test data, this determination can readily be made. However after making these determinations for a number of different cases it has been found that with a reasonable degree of accuracy, the average equivalent uniform shear stress, τ_{av} , is about 65% of the maximum shear stress, $\tau_{\max}$. Combining this result with the above

Fig 4 RANGE OF VALUES OF r_d FOR DIFFERENT SOIL PROFILES

Fig 5 TIME HISTORY OF SHEAR STRESSES DURING EARTHQUAKE
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expression for $\tau_{\max}$ it follows that for practical purposes, the average cyclic shear stress may be determined by:

$$\tau_{av} \approx 0.65 \times \frac{\gamma h}{g} \cdot a_{\max} \times r_d \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

The appropriate number of significant stress cycles N_c will depend on the duration of ground shaking and thus on the magnitude of the earthquake.

Representative numbers of stress cycles are as follows:

<u>Earthquake Magnitude</u>	<u>No. of Significant Stress Cycles, N_c</u>
7	10
$7\frac{1}{2}$	20
8	30

The use of these values together with stresses determined from Eq. 4, provides a simple procedure for evaluating the stresses induced at different depths by any given earthquake for which the maximum ground surface acceleration is known.

Simplified Procedure for Evaluating Stresses Causing Liquefaction

Determination of the cyclic shear stress causing liquefaction of a given soil in a given number of stress cycles is normally made by a laboratory test program using cyclic loading triaxial compression tests. The results of a number of such investigations on soils with different grain sizes, represented by the mean grain size, D_{50} , and at a relative density of 50 percent are summarized in Figs. 6 and 7 (29). The results of these tests are expressed in terms of the stress ratio $\frac{\sigma_{dc}}{2\sigma_a}$ causing liquefaction in 10 and 30 cycles, where σ_{dc} is the cyclic deviator stress

Fig 6 STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 10 CYCLES

Fig 7 STRESS CONDITIONS CAUSING LIQUEFACTION OF SANDS IN 30 CYCLES

and σ_a is the initial ambient pressure under which the sample was consolidated. Although the tests were performed by different investigators, it may be seen that there is a reasonable degree of consistency in the results, suggesting that these data may be used to estimate the liquefaction characteristics of other sands from a knowledge of the mean grain size, D_{50} . The stresses required to cause liquefaction for sands at other relative densities may be estimated from the fact that for relative densities up to about 80 percent, the shear stress required to cause initial liquefaction is approximately proportional to the relative density.

Also shown in Figs. 6 and 7 are the values of the stress ratio $\frac{\tau}{\sigma_o'}$ causing liquefaction under field conditions, estimated from the results of simple shear tests; τ is the shear stress developed on a horizontal plane and σ_o' is the initial effective overburden pressure. It may be seen that the field value of $\frac{\tau}{\sigma_o'}$ is less than the corresponding value of $\frac{\sigma_{dc}}{2\sigma_a}$. However the two stress ratios may be related by the equation:

$$\left(\frac{\tau}{\sigma_o'}\right)_l = \left(\frac{\sigma_{dc}}{2\sigma_a}\right)_l \cdot c_r \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where c_r is a correction factor to be applied to laboratory triaxial test data to obtain the stress conditions causing liquefaction in the field. Detailed consideration of the stress conditions in triaxial compression test specimens and the limitations of the test itself have led to the suggestion that appropriate values of c_r vary with relative density approximately as shown in Fig. 8 (29). Thus the test data in Figs. 6 and 7 together with the values of c_r in Fig. 8 provide a simple means for assessing the stress conditions likely to cause liquefaction of different soils in the field. For a given soil at a relative density D_r , the stress ratio

Fig. 8 RELATIONSHIP BETWEEN C_r AND RELATIVE DENSITY.

causing liquefaction in the field may be estimated from the relationship

$$\left(\frac{\tau}{\sigma_o'}\right)_{D_r} \approx \left(\frac{\sigma_{dc}}{2\sigma_a}\right)_{50} \cdot c_r \cdot \frac{D_r}{50} \quad (6)$$

where the suffixes D_r and 50 denote relative densities of D_r and 50 respectively and values of $\left(\frac{\sigma_{dc}}{2\sigma_a}\right)_{50}$ are taken from Fig. 6 or 7. On specific projects it may often be desirable to perform cyclic load tests to determine the stress conditions causing liquefaction but for many purposes the use of the above expression may well be adequate.

Evaluation of Liquefaction Potential

In order to evaluate the liquefaction potential of a deposit it is necessary to determine whether the shear stress induced at any depth by the earthquake, determined from Eq. 4, is sufficiently large to cause liquefaction at that depth, as indicated by the relationship in Equation 6. For deposits in which the water table is at a depth of 0 to 10 ft, the critical depth will often be about 20 ft and for those where the water table depth is about 15 ft, the critical depth may be about 30 ft. Thus the evaluation can often be made simply for a representative element at one of these depths.

Consider for example, a deposit of sand for which $D_{50} \approx 0.2$ mm, the water table is 5 ft below the ground surface and which is subjected to ground shaking by a magnitude 7 earthquake. From Equation 4, the average shear stress induced for about 10 cycles will be:

$$\tau_{av} \approx 0.65 \times \frac{\gamma h}{g} \cdot a_{max} \times r_d$$

At a depth of 20 ft, $r_d \approx 0.95$ (see Fig. 4) giving

$$\tau_{av} \approx 0.65 \times 0.95 \times \frac{\gamma h}{g} \cdot a_{max}$$

If the shear stress required to cause initial liquefaction in 10 cycles is τ_{l10} , it follows from Eq. 6:

$$\frac{\tau_{l10}}{\sigma_o'} \approx \left(\frac{\sigma_{dc}}{2\sigma_a} \right)_{l50} \cdot \frac{D_r}{50} \cdot c_r$$

However for a soil with $D_{50} \approx 0.2$ mm, the data in Fig. 6 show that

$$\left(\frac{\sigma_{dc}}{2\sigma_a} \right)_{l50} \approx 0.24$$

giving

$$\tau_{l10} \approx 0.24 \cdot \sigma_o' \cdot \frac{D_r}{50} \cdot c_r$$

The sand will develop initial liquefaction in 10 cycles if

$$\tau_{av} = \tau_{l10}$$

$$\text{i.e. } 0.65 \times 0.95 \times \frac{\gamma h}{g} \cdot a_{max} = 0.24 \cdot \sigma_o' \cdot \frac{D_r}{50} \cdot c_r$$

$$\text{or } \frac{a_{max}}{g} \approx \frac{0.24}{0.65 \times 0.95 \times 50} \cdot \frac{\sigma_o'}{\gamma h} \cdot c_r D_r$$

$$\approx 0.0078 \cdot \frac{\sigma_o'}{\gamma h} \cdot c_r D_r$$

For a water table 5 ft below the ground surface and a soil element at a depth of 20 ft:

$$\gamma h \approx 20 \times 112 \approx 2240 \text{ psf}$$

$$\sigma_o' \approx 5 \times 112 + 15 \times 50 \approx 1310 \text{ psf}$$

$$\text{Thus } \frac{a_{max}}{g} \approx 0.0078 \times \frac{1310}{2240} \cdot c_r D_r$$

$$\approx 0.0046 c_r D_r \quad \text{B-17}$$

The relationship between $a_{\max}$ and D_r indicated by this equation may be evaluated as follows:

D_r	c_r (from Fig. 8)	$\frac{a_{\max}}{g} = 0.0046 c_r D_r$
30	0.55	0.076
40	0.55	0.100
50	0.57	0.131
60	0.60	0.166
70	0.64	0.205
80	0.68	0.250

and the relationship can be plotted as shown in Fig. 9. If, for any given value of maximum ground acceleration, the relative density of the deposit exceeds the value indicated by this relationship, liquefaction would not be expected to occur, and vice versa. Relationships computed following the same procedure for water table depths of 10 and 15 ft are also plotted in Fig. 9, indicating the lower degree of liquefaction potential resulting from a greater depth of water table.

Similar computations can readily be made for other soil types, depths of water table and number of stress cycles. Soils having the greatest susceptibility to liquefaction appear to be very fine uniform sands, with D_{50} of the order of 0.08 mm. For such materials the relationships between $a_{\max}$ and relative density for which initial liquefaction would just occur in an earthquake producing 20 stress cycles for different water table elevations are shown in Fig. 10.

In general when liquefaction of sands develops in the field, the mean grain size might be expected to range from 0.075 mm to about 0.2 mm and

Fig 9 EVALUATION OF LIQUEFACTION POTENTIAL FOR FINE SAND - 10 STRESS CYCLES

Fig 10 EVALUATION OF LIQUEFACTION POTENTIAL FOR VERY FINE SAND - 20 STRESS CYCLES

the number of stress cycles from 10 to 20. Thus the boundaries between conditions which do and do not liquefy might normally be expected to be within the range covered by the conditions:

$$D_{50} \approx 0.075 \text{ mm}; \quad N_c = 20$$

$$D_{50} \approx 0.2 \text{ mm}; \quad N_c = 10$$

The computed relationships between relative density and maximum ground acceleration for which initial liquefaction will just occur for these conditions are plotted in Figs. 11 and 12 for water table depths of 5 and 10 ft. respectively. For earthquakes exceeding about Magnitude 7 and appropriate water table depths, it might therefore be expected that combinations of relative density and maximum ground surface acceleration falling above the upper curve ($D_{50} \approx 0.2 \text{ mm}; N_c = 10$) in Figs. 11 and 12 would often cause liquefaction; combinations falling below the lower curve ($D_{50} \approx 0.075 \text{ mm}; N_c = 20$) would usually be safe against liquefaction; while combinations falling between the curves might cause liquefaction or not, depending on the particular combination of conditions involved. It is believed that charts of this type can provide a useful guide in the evaluation of soil liquefaction potential during earthquakes.

Comparison of Liquefaction Potential Evaluations with Field Behavior

The only reliable method for determining the usefulness of any method of evaluating liquefaction potential is by comparing its results with the known performance of soils in the field. Accordingly a compilation has been made of a number of cases in which sand deposits have or have not liquefied during earthquakes. The soil conditions, ground motions and field behavior in these cases are summarized in Table 1.

Table 1. Site Conditions and Earthquake Data for Known Cases of Liquefaction and Non-Liquefaction.

Earthquake	Date	Magnitude	Site	Distance from epicenter, miles	Soil type	Water table depth, ft	Depth of liquefaction, ft	Relative density, %	Max. ground surface acceleration, g	Duration of shaking, secs	Field Behavior	References
Niigata	1807	6.6	Niigata	24	Sand	3	12	64	0.12	70	No liquefaction	Kawasumi (10)
Niigata	1802	6.6	Niigata	24	Sand	3	12	64	0.12	0	No liquefaction	Seed & Idriss (26)
Niigata	1888	6.1	Niigata	24	Sand	3	12	64	0.08	12	No liquefaction	Kawasumi (10)
Niigata	1887	6.1	Niigata	24	Sand	3	12	64	0.08	12	No liquefaction	Seed & Idriss (26)
Mino wari	1891	8.4	Ogaki	6	Sand	17	17	65	0.3	75	Liquefaction	Kishida (12)
Mino wari	1891	8.4	Utsunomiya	20	Sand	10	10	55	0.35	75	Liquefaction	Kishida (12)
Mino wari	1891	8.4	Utsunomiya	6	Sand & gravel	19	19	72	0.35	75	No liquefaction	Kishida (12)
Mino wari	1891	8.4	Ogaki	6	Sand	17	17	65	0.3	75	Liquefaction	Kishida (12)
Santa Barbara	1905	6.3	Shift field Dam	7	Sand	1	1	40	0.1	15	Liquefaction	Seed et al (28)
El Centro	1940	7.0	Brawley	5	Sand	1	1	38	0.25	30	Liquefaction	Ross (23)
El Centro	1940	7.0	Alamo	5	Sand	20	20	43	0.25	30	Liquefaction	Ross (23)
El Centro	1940	7.0	Solfatara	5	Sand	5	20	32	0.25	30	Liquefaction	Ross (23)
Tohrankai	1944	8.3	Komei	100	Sand	13	13	40	0.08	70	Liquefaction	Kishida (12)
Tohrankai	1944	8.3	Meiko St	100	Silt & Sand	8	1	30	0.08	70	Liquefaction	Kishida (12)
Fukui	1948	7	Takaya	4	Sand	3	3	18	0.30	30	Liquefaction	Kishida (12)
Fukui	1948	7.2	Takaya	4	Sand	3	3	28	0.30	30	No liquefaction	Kishida (12)
Fukui	1948	7.2	Shonenji Temple	4	Sand	10	3	40	0.30	30	Liquefaction	Kishida (12)
Fukui	1948	7.2	Shonenji Temple	4	Sand & Silt	3	20	50	0.30	30	Liquefaction	Kishida (12)
San Francisco	1957	5.5	Lake Merced	4	Sand	8	1	7	0.18	18	Liquefaction	Ross (23)
Chile	1960	8.4	Puerto Montt	70	Sand	1	1	6	0.15	75	Liquefaction	Lee (15)
Chile	1960	8.4	Puerto Montt	70	Sand	12	1	8	0.15	75	Liquefaction	Lee (15)
Chile	1960	8.4	Puerto Montt	70	Sand	12	1	18	0.15	75	No liquefaction	Lee (15)
Niigata	1964	7	Niigata	32	Sand	3	0	6	0.16	40	Liquefaction	Seed & Idriss (26)
Niigata	1964	7.5	Niigata	32	Sand	3	5	15	0.16	40	Liquefaction	Kishida (11)
Niigata	1964	7.5	Niigata	32	Sand	3	5	15	0.16	40	No liquefaction	Seed & Idriss (26)
Niigata	1964	7.5	Niigata	32	Sand	12	6	53	0.16	40	No liquefaction	Seed & Idriss (26)
Alaska	1964	8.3	Snow River	60	Sand	8	0	5	0.15	180	Liquefaction	Ross et al (24)
Alaska	1964	8.3	Snow River	60	Sand	8	0	5	0.15	180	Liquefaction	Ross et al (24)
Alaska	1964	8.3	Valdez	70	Sand & gravel	25	40-80	100	0.12	180	No liquefaction	Ross et al (24)
Alaska	1964	8.3	Valdez	70	Sand	20	10	65	0.16	180	Liquefaction	Ross et al (24)
Alaska	1964	8.3	Valdez	35	Sand & gravel	5	13	68	0.25	180	Liquefaction	Coulter & Migliaccio (3)
Tokachi	1968	8	Tokachi	4 to 11	Sand	3	1	14	0.21	45	No liquefaction	Ohsaki (20)
Tokachi	1968	8	Tokachi	4 to 11	Sand	3	1	6	0.21	45	Liquefaction	Ohsaki (20)
Tokachi	1968	7.5	Tokachi	45 to 110	Sand	5	10	15	0.21	45	No liquefaction	Ohsaki (20)
Tokachi	1968	8	Tokachi	100	Sand	3	15	6	0.18	45	Liquefaction	Kishida (13)

Fig. II EVALUATION OF LIQUEFACTION POTENTIAL FOR FINE SANDS-
WATER TABLE 5 FT. BELOW GROUND SURFACE.

Fig. 12 EVALUATION OF LIQUEFACTION POTENTIAL FOR FINE SANDS - WATER TABLE 10 FT. BELOW GROUND SURFACE.

In most cases no direct measurement of relative density was made but data were available concerning the standard penetration resistance of the deposits. In these cases the in-situ relative density was determined from the relationship between relative density, penetration resistance and effective overburden pressure proposed by Gibbs and Holtz (7). This correlation is presented in Fig. 13.

In addition, measurements of maximum ground accelerations were available for only a few cases. Thus in most cases it was necessary to estimate the maximum ground surface accelerations from the approximate relationships between earthquake magnitude, distance from zone of energy release and maximum ground acceleration proposed by Housner (9).

In this way, for locations where the water table was about 5 ft below the ground surface, it was possible to determine combinations of relative density and maximum ground surface acceleration for 15 cases where liquefaction is known to have occurred and 8 cases where liquefaction did not develop. These cases are plotted in Fig. 11, larger symbols being used for cases where ground motion measurements were available and smaller symbols for cases where ground motions had to be estimated.

For locations where the water table was about 10 ft below the ground surface it was possible to determine combinations of relative density and maximum ground surface acceleration for 8 cases where liquefaction is known to have occurred and 6 cases where liquefaction did not develop. These cases are plotted in Fig. 12.

It may be seen from Figs. 11 and 12 that the liquefaction behavior of soils in the field is in reasonably good agreement with the anticipated behavior determined by the simplified procedure for evaluating liquefaction potential. It appears therefore that evaluations of this type provide not

Fig. 13 RELATIONSHIP BETWEEN STANDARD PENETRATION RESISTANCE, RELATIVE DENSITY AND EFFECTIVE OVERBURDEN PRESSURE.

only a convenient means of summarizing available knowledge of conditions causing liquefaction in the field in past earthquakes, but also of extending this information to different conditions of relative density and maximum ground accelerations.

Comparison of Liquefaction Potential Evaluations with Limiting Conditions for Liquefaction at Niigata

In the Niigata earthquake of June 16, 1964, extensive liquefaction occurred in some parts of the city but not in others. The maximum ground acceleration recorded in the city was 0.16g. Following the earthquake Japanese engineers made detailed studies to determine the differences in soil conditions between the heavy damage area where liquefaction occurred and the light damage area where liquefaction did not develop. It was concluded that the essential difference was a somewhat higher relative density, as evidenced by higher values of the standard penetration resistance, in the light damage area as compared with the heavy damage area. Independent studies by Kishida (11) and Koizumi (14) led to the delineation of boundaries, in terms of variations of penetration resistance with effective overburden pressure, between those soil conditions producing liquefaction and those for which liquefaction did not occur, (see Fig. 14).

It is interesting to compare these boundaries with that which might be determined by the simplified procedure for evaluating liquefaction potential. For the conditions at Niigata, the mean grain size for the soil ranged from about 0.1 to 0.5 mm, the water table was about 5 ft below the ground surface and the earthquake (Magnitude about 7.6) could be expected to produce about 20 significant stress cycles. Reference to Fig. 11 shows that for these conditions and a maximum ground surface

Fig. 14 ANALYSIS OF LIQUEFACTION POTENTIAL AT NIIGATA
FOR EARTHQUAKE OF JUNE 16, 1964.

acceleration of 0.16g, it would be necessary for the sand to have a relative density of about 72% to be marginally safe against liquefaction. Thus the limiting condition for no liquefaction to occur would be expected to correspond approximately with a relative density condition of about 72% in the upper 30 ft: Combinations of effective overburden pressure and standard penetration resistance corresponding to this relative density may be read off directly from Fig. 13, and they are plotted in Fig. 14. It may be seen that the boundary between zones of liquefaction and no liquefaction determined in this way is in reasonably good agreement with the boundaries determined by Kishida and Koizumi on the basis of field observations.

Charts for Evaluating Liquefaction Potential in Terms of Penetration Resistance

Both the simplified analysis procedure described previously and the data in Figs. 11 and 12 may readily be used to develop simple charts for the evaluation of liquefaction potential in terms of penetration resistance values. For example on the basis of the analytical and field data shown in Fig. 15 (reproduced from Fig. 11) it is very unlikely that combinations of maximum ground acceleration and relative density falling below line YY would cause liquefaction. Similarly it is highly probable that combinations of maximum ground acceleration and relative density falling above line XX would cause liquefaction of sands in which the depth of water table is about 5 ft. Thus for any given value of maximum ground acceleration, it is possible to designate three ranges of relative densities:

- (1) a range in which liquefaction is very likely to occur
- (2) a range in which liquefaction may or may not occur depending on the characteristics of the sand and the number of

Fig. 15 EVALUATION OF LIQUEFACTION POTENTIAL FOR SANDS -
WATER TABLE 5 FEET BELOW GROUND SURFACE.

significant stress cycles produced by the earthquake or the earthquake magnitude (zone between XX and YY in Fig. 15).

and (3) a range in which liquefaction is very unlikely to occur.

For sites consisting of sands with the water table about 5 ft below the ground surface, the results in Fig. 15 would indicate the following ranges:

<u>Max. ground surface acceleration</u>	<u>Liquefaction very likely</u>	<u>Liqn. potential depends on soil type and earthquake magnitude</u>	<u>Liquefaction very unlikely</u>
0.10g	$D_r < 33$	$33 < D_r < 54$	$D_r > 54$
0.15g	$D_r < 48$	$48 < D_r < 73$	$D_r > 73$
0.20g	$D_r < 60$	$60 < D_r < 85$	$D_r > 85$
0.25g	$D_r < 70$	$70 < D_r < 92$	$D_r > 92$

The values of relative density listed above can readily be converted to corresponding values of standard penetration resistance by means of the correlation shown in Fig. 13, thus leading to liquefaction potential evaluation charts such as those shown in Fig. 16. Charts for a water table depth of 10 ft are shown in Fig. 17 and similar charts for other depths of water table can readily be developed. However it may be seen that the range of penetration resistance values in which the liquefaction potential depends on the soil type and the earthquake characteristics is considerable; for conditions within this range it would be necessary to make individual assessments of liquefaction potential for each site using the general procedure described previously.

The limiting values of penetration resistance for which liquefaction is very unlikely, as indicated by the above table and the results in Fig. 16, are compared in Fig. 18. Also shown in this figure are the

Fig 16 LIQUEFACTION POTENTIAL EVALUATION CHARTS FOR SANDS WITH WATER TABLE AT DEPTH OF ABOUT 5 FEET

Fig. 17 LIQUEFACTION POTENTIAL EVALUATION CHARTS FOR SANDS WITH WATER TABLE AT DEPTH OF ABOUT 10 FEET.

Fig. 18 PENETRATION RESISTANCE VALUES FOR WHICH LIQUEFACTION IS VERY UNLIKELY TO OCCUR UNDER ANY CONDITIONS.

limiting values suggested by Ohsaki (20) based on experiences in the Niigata earthquake and Kishida (12), based on a study of a number of earthquakes. It seems reasonable to expect that the limiting boundary would vary depending on the intensity of shaking as indicated by the results of this study and that the limiting values suggested by Ohsaki and Kishida are perhaps somewhat conservative for low intensity shaking and somewhat unconservative for strong intensity shaking.

Conclusion

In the preceding pages an attempt has been made to identify some of the significant factors affecting the liquefaction (or cyclic mobility) potential of sands during earthquakes, to present a simplified procedure for evaluating liquefaction potential which will take these factors into account, to assemble available field data concerning the liquefaction or non-liquefaction behavior of sands during earthquakes and to compare these data with evaluations of performance made by the simplified procedure. It must be recognized that in any study of this type, a number of approximations must be made especially with regard to determinations of relative densities and maximum ground accelerations in the field. However it is believed that even the limited available field data can provide a useful guide to the probable performance of other sand deposits, that the method of representing the data provides a useful framework for evaluating past experiences of sand liquefaction, and that the simplified procedure provides a reasonably good means for extending previous field observations to new situations. Where greater accuracy is justified, the simplified liquefaction evaluation procedure can readily be supplemented by test data on particular soils or by ground response analyses to provide more

definitive evaluations. However it is believed that the simplified procedure will provide adequate assessments of liquefaction potential for many practical purposes.

Acknowledgments

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APPENDIX C

SYMBOLS

APPENDIX C

SYMBOLS

<u>SYMBOL</u> (Units)	<u>REPRESENTS</u>
A (ft)	amplitude of oscillation or vibration
A	constant
a (ft/sec ²)	shake table acceleration to cause liquefaction
a	constant
a _{n+1} (ft)	harmonic input wave amplitude
AMP (w)	response amplification factor
B	constant
b	constant
b	response level of a system
C	constant
C _c (lb-sec/in)	critical damping
[C]	damping matrices
[C] _t	damping matrices at time, t
c	constant
c (ft/sec)	wave speed
c (lb/ft ²)	cohesion
c' (lb/ft ²)	cohesion (effective shear stress parameter)
c _s	fraction of critical coefficient of damping
D _r (%)	relative density of cohesionless soils
D ₁₀ (mm)	diameter at which 10% of the soil is finer
D ₅₀ (mm)	mean grain size
DBE	design basis earthquake (safe shutdown earthquake)

E (lb/in ²)	modulus of linear deformation, Young's modulus
e	void ratio
e_{cr}	critical void ratio
e_{max}, e_{min}	maximum and minimum void ratios
F (lbs)	force
$F(\sigma_{ij}, \epsilon^p_{ij})$	functions
F_o (lbs)	yield force
f (cycles/sec)	frequency
f_n (cycles/sec)	resonant frequency
$f(\sigma_{ij}, \epsilon^p_{ij})$	yield condition function
$f(z)$	wave function
G, G_1, G_2 , (lb/in ²)	shear modulus, specific gravity
G_j (lb/in ²)	shear modulus of the j th layer
$G_B(w)$	power spectral density of the bedrock motions
G_{max} (lb/in ²)	maximum shear modulus at essentially zero strain
$G_s(w)$	power spectral density of the surface ground motions
G_{VL}, G_{UL}, G_{RL} (lb/in ²)	shear modulus for virgin loading, unloading and reloading
g (ft/sec ²)	acceleration of gravity (32.17 ft/sec ²)
$g(z)$	wave function
H (ft or inches)	thickness of layer of soil or height of soil sample
$H(\sigma_{ij}, \epsilon^p_{ij})$	function
$H(w)$	transfer function of the soil layers
$H_j(w)$	Fourier transform of $h(t)$
Hz (cycles/sec)	cycles per second
$h_j(t)$	transfer function for j th layer

I	intensity of earthquake on Modified Mercalli scale
i	$\sqrt{-1}$
[J]	transfer matrix
J_1, J_2'	first stress invariant, second stress deviator invariant
$\dot{J}_1, \dot{J}_2'$	rate of first stress invariant, rate of second stress deviator invariant
K (lb/in, lb/ft ²)	spring constant or bulk modulus
K_0	coefficient of lateral earth pressure at rest
K_2	constant used in defining shear modulus as a function of shear strain and confining pressure
$[K]_t$	stiffness matrices at time, t
k	constant
L (ft)	wave length
l (ft)	width of soil element
M	amplitude magnification factor, earthquake magnitude on Richter scale
$ M _t$	mass matrices at time, t
m (lb-sec ² /ft)	mass ($m = W/g$)
N (cycles, blows/ft)	no. of cycles, standard penetration resistance, wave number
N_c (blows/ft)	dynamic cone penetration resistance
OCR	overconsolidation ratio
P (lb/in ² , lbs)	volumetric stress, plate bearing test load
$P_f(b, t_d)$	probability of failure across the barrier level within the duration, t_d
$P_s(b, t_d)$	probability that the system is safe within the duration, t_d
p (lb/in ²)	wave period, pressure
Q_c (kg/cm ²)	static cone penetration resistance

$[R], [R]_t$	force vector; force vector at time, t
r (in.)	radius of plate in plate bearing test
$[r]_t, [\dot{r}]_t, [\ddot{r}]_t$	displacement, velocity and acceleration vectors relative to the base, at time, t
S (% ,lb)	degree of saturation, shear force in simple shear device
$S(t)$	random input with intensity I (t)
s_{ij} (lb/in ²)	stress
s'_{ij} (lb/sec)	rate of stress
s_u (kg/cm ²)	undrained shear strength
T (sec)	period of vibration
$t, \Delta t$ (sec)	time, time increment
t_d (sec)	duration of earthquake excitations
U	uniformity coefficient = D_{60}/D_{10}
U_j (w)	Fourier transform of u_j (t)
u, u' (in., lb/in ²)	displacement or pore pressure
$u_j(z_j, t)$	response in j th layer
V (ft ³)	total volume
V or $\dot{x}$ (ft/sec)	velocity of wave propagation
V_c (ft/sec)	velocity of compression wave
V_r (ft/sec)	velocity of Rayleigh wave
V_s (ft/sec)	velocity of shear wave
V_s (ft ³)	volume of solid particles
V_v (ft ³)	volume of voids
V_1, V_2 (ft/sec)	wave speeds
W (lb)	weight
w (in.)	displacements
x (in. or ft.)	displacement of a system relative to the ground

$Y (w)$	Fourier transform of $y (t)$
$y (t)$	input motion
<u>Greek Letters</u>	
$\alpha (1/ft)$	coefficient of attenuation
α, α_j	damping coefficient
α_i	coefficient
$\alpha_m (ft/sec)$	velocity of propagation of dilatational waves in m^{th} layer
β, β_j	damping coefficient
β_i	coefficient
$\beta_m (ft/sec)$	velocity of propagation of rotational waves in m^{th} layer
$\gamma (\%, lb/ft^3)$	shear strain amplitude or wet unit weight of soil
$\dot{\gamma} (\%/sec)$	rate of shear strain
$\gamma' (lb/ft^3)$	unit weight of submerged soil
$\gamma_d (lb/ft^3)$	dry unit weight of soil
$\gamma_{max}, \gamma_{min} (lb/ft^3)$	maximum and minimum dry density
$\gamma_w (lb/ft^3)$	unit weight of water ($62.4 lb/ft^3$)
$\gamma_{\beta m}, \gamma_{\alpha m}$	ratios of wave speeds in m^{th} layer
Δ	change in
Δ_m	solution of dilatational wave for m^{th} layer
Δ'_m, Δ''_m	constants in the dilatational wave solutions
δ	logarithmic decrement
δ_{ij}	delta function, defined as $\delta = 0 \quad i \neq j$ $\delta = 1 \quad i = j$
$\epsilon (\%)$	linear strain
$\epsilon_{ij}^E, \epsilon_{ij}^P, \epsilon_{ij}, \dot{\epsilon}_{ij} (\%)$	elastic strain, plastic strain, strain, rate of strain

$\dot{\epsilon}_{ij}^p, \dot{\epsilon}, \dot{\epsilon} \text{ (\%/sec, \%)}$	rate of plastic strain, volumetric strain, rate of volumetric strain
$\epsilon_y \text{ (\%)}$	shear strain at yielding point
$\lambda \text{ (\%, ft)}$	damping ratio or wave length
$\lambda, \lambda', \lambda_m$	Lame's constants
$\lambda_n, \lambda_{eq}, \lambda_{vis}, \lambda_{eqh} \text{ (\%)}$	damping ratio for n^{th} mode, equivalent damping ratio, viscous damping ratio, equivalent hysteretic damping ratio
μ	Poisson's ratio
$\mu, \mu', \mu_1, \mu_2, \mu_m$	Lame's constants
Λ	scalar function
$\rho, \rho_1, \rho_2, \rho_m \text{ (lb-sec}^2/\text{ft}^4)$	mass density = γ/g
ρ_{12}	correlation coefficient of σ_1 , and σ_2
$\sigma_c \text{ (lb/ft}^2)$	effective confining pressure
$\sigma_{dc}, \sigma_p \text{ (lbs/ft}^2)$	pulsating deviator stress
$\sigma_i, \sigma_n \text{ (lb/ft}^2)$	normal stress in the i^{th} direction
$\sigma_{ij}, \sigma_k, \dot{\sigma}_{kl} \text{ (lbs/ft}^2)$	stress, rate of stress
$\sigma'_m \text{ (lb/ft}^2)$	effective mean principal stress
$\sigma_n \text{ (lb/ft}^2)$	normal stress
$\sigma'_o \text{ (lb/ft}^2)$	average effective normal stress or confining pressure
$\sigma'_v \text{ (lb/ft}^2)$	effective vertical stress
$\sigma_3, \sigma_1 \text{ (lb/ft}^2)$	confining pressure or minor principal stress, major principal stress
σ_1^2, σ_2^2	response variances of displacement and velocity respectively
$\tau, \dot{\tau} \text{ (lb/ft}^2)$	shear stress, rate of shear stress
$\phi \text{ (deg)}$	angle of internal friction
$\phi' \text{ (deg)}$	effective angle of internal friction
$\phi_{N+1} \text{ (w) (degrees)}$	phase angle

ω (cycles/sec)	frequency
ω_m	solution of rotational waves for m^{th} layer
$\omega'_m \ \omega''_m$	constants in the dilatational and rotational wave solutions
ω_n (cycles/sec)	the n^{th} characteristic frequency
ω_s (rad/sec)	undamped circular frequency

APPENDIX D

A GLOSSARY OF
STANDARD NOMENCLATURE, TERMINOLOGY
AND DEFINITIONS

APPENDIX D

A GLOSSARY OF STANDARD NOMENCLATURE, TERMINOLOGY AND DEFINITIONS

This glossary has been prepared to define the usage of terms and to provide a convenient reference list on the general subject of soil behavior under earthquake loading conditions.

Several of the sources used in determining usage were the following:

- 1) American Society for Testing and Materials (1970)
"Standard Definitions of Terms and Symbols Related to Soil and Rock Mechanics," D 653-67, Part 11, 1970 Annual Book of ASTM Standards, Philadelphia, Pa.
- 2) American Geological Institute (1960)
Glossary of Geology and Related Sciences, 2d ed., Washington, D.C.
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Encyclopedia of Hydraulics, Soil and Foundation Engineering, Elsevier Pub. Co., Amsterdam, New York.
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"A Glossary of Geoplosics: The Systematic Study of Explosion Effects in the Earth," AFSWC-TN-60-20, Stanford Research Institute for the Air Force Special Weapons Center, Kirtland AFB, N. Mex.

ACCELEROGRAM

The record from an accelerometer showing acceleration as a function of time.

ACCELEROMETER

An instrument for measuring acceleration.

AMPLITUDE

Maximum deviation from mean or center line of wave.

ANGLE OF INTERNAL FRICTION

Angle between the abscissa and the tangent of the curve representing the relationship of shearing resistance to normal stress acting within a soil.

ANISOTROPIC MASS

A material having different properties in different directions at any given point.

ATTENUATION

Dying out (decay), reduction of amplitude or change in wave form due to energy dissipation or distance with time.

BODY WAVES

Waves propagated in the interior of a body, i.e., compression and shear waves, the P and S waves of seismology.

BULK MODULUS

The ratio of the change in average stress to the change in unit volume.

COHESIONLESS SOIL

A soil that when unconfined has little or no strength when air dried and that has little or no cohesion when submerged.

COHESIVE SOIL

A soil that when unconfined has considerable strength when air dried and that has significant cohesion when submerged.

COMPRESSION WAVE

Wave in which element of medium changes volume without rotation.

CONTROLLED STRESS TEST

A test in which the stress to which a specimen is subjected is applied at a controlled rate.

CRITICAL DAMPING

The minimum viscous damping that will allow a displaced system to return to its initial position without oscillation.

CRITICAL DENSITY

The unit weight of a saturated granular material below which it will lose strength and above which it will gain strength when subjected to rapid deformation.

CRITICAL FREQUENCY

Frequency at which maximum or minimum amplitudes of excited waves occur.

CRITICAL VOID RATIO

The void ratio corresponding to the critical density (see void ratio).

CYCLIC LOADING TEST

Laboratory tests in which the stress to which a specimen is subjected is reversed from extension to compression and vice versa over a number of stress applications.

CYCLIC MOBILITY

A phenomenon leading up to liquefaction. In this condition cyclic shear stresses are applied to a saturated specimen of sand. The applied cyclic shear stresses produce a gradual build-up in the pore-water pressure within the undrained specimen causing the effective pressure to reduce ultimately to zero.

DAMPING

The dissipation of energy with time or distance.

DAMPING RATIO

For a system with viscous damping, the ratio of actual damping coefficient to the critical damping coefficient.

DENSITY

Weight per unit volume.

DESIGN BASIS EARTHQUAKE

For a reactor site, that earthquake which produces the vibratory ground motion, for which those features of the nuclear power plant necessary to shut down the reactor and maintain the plant in a safe condition without undue risk to the health and safety of the public are designed to remain functional.

DEVIATOR STRESS

The difference between the major and minor principal stresses in a triaxial test.

DILATANCY

The expansion of cohesionless soils when subject to shearing deformation.

DIRECT SHEAR TEST

A shear test in which soil under an applied normal load is stressed to failure by moving one section of the soil container relative to the other section.

DISPERSION

The dependence of the propagation velocity on wave length or frequency which causes the shape of a disturbance to change continually as time goes on. In an unlimited medium there will be a continual spreading out of the disturbance into trains of waves.

DOUBLE AMPLITUDE

Total excursion or overall height of wave (peak to peak, crest-to-trough) or for sinusoidal wave twice the amplitude.

DYNAMIC SOIL PROPERTIES

Those soil properties which affect the response of soils subjected to cyclic loading conditions.

EARTH PRESSURE AT REST

The value of earth pressure when the soil mass is in its natural state without having been permitted to yield or without having been compressed.

EARTHQUAKE

Groups of elastic waves propagating in the earth, set up by a transient disturbance of the elastic equilibrium of a portion of the earth.

EFFECTIVE STRESS (Intergranular Pressure)

The average normal force per unit area transmitted from grain to grain of a soil mass. This stress is effective in mobilizing internal friction.

EPICENTER

The point on the earth's surface directly above the focus

of an earthquake.

EXTENSION TRIAXIAL LOADING

Loading in which the axial stress is reduced to some value less than the confining pressure but not necessarily below zero effective stress.

FAILURE

Failure in which movement caused by shearing stresses in a soil mass is of sufficient magnitude to destroy or seriously endanger a structure.

FAULT

A fracture or fracture zone along which there has been displacement of the two sides relative to one another parallel to the fracture.

FINES

Portion of soil finer than a No. 200 (74 micron) U. S. standard sieve.

FINITE ELEMENT ANALYSIS

An analysis which uses an assembly of finite elements which are connected at a finite number of nodal points to represent a structure and/or a soil continuum.

FORCED VIBRATION

Vibration that occurs if the response is imposed by the excitation. If the excitation is periodic and continuing, the oscillation is steady-state.

FREE FIELD

The regions of the medium which are not influenced by man-made structures or a medium which contains no such structures. It also refers to that region in which boundary effects do not influence significantly the behavior of the medium.

FREE VIBRATION

Vibration that occurs in the absence of forced vibration.

FREQUENCY

Number of cycles occurring in unit time.

GEOMETRICAL DAMPING

That component of damping which occurs due to the radial spreading of energy waves with distance from a given source.

GEOPHYSICS

The study of the physical characteristics and properties of the earth.

GEOPHONE

Sensing device used to measure electronically the rate of travel of sound or force waves transmitted through the earth from a known source.

GEOPHYSICAL EXPLORATION

An indirect method of determining structure and composition of underground geological formations. The principle involved includes a measurement of the rate of travel of sound waves or low frequency force waves, generated by explosives or hammer blows, through a soil medium.

GRAIN SIZE

A term relating to the size of mineral particles that make up a soil deposit.

GROUND RESPONSE OR MOTION

A general term, includes all aspects of motion, viz, acceleration, particle velocity, displacement, stress and strain, usually resulting from a nuclear blast or an earthquake.

GROUND WATER LEVEL

Depth below the ground surface at which the pressure in the water is zero with respect to the atmospheric pressure.

HALF SPACE

Medium of soil in which the ground surface is considered to act as flat plane surface of infinite extent.

HYDROSTATIC PRESSURE

The pressure in a liquid under static conditions; the product of the unit weight of the liquid (water = 62.4 pcf) and the difference in elevation between the given point and the ground water elevation.

HYSTERESIS LOOP

1) The stress-strain path of a material under cyclic loading conditions.

2) A trace of the lag in the return of an elastically

deformed specimen to its original shape after the load has been released.

IN SITU STRENGTH

The in-place field strength of a soil deposit.

INTENSITY

A number describing the effects of an earthquake on man, on structures build by him and on the earth's surface. The number is rated on the basis of an "earthquake intensity scale", the scale in common use in the U. S. today is the Modified Mercalli Scale of 1931.

INTERMEDIATE PRINCIPAL STRESS (See principal stress)

The principal stress whose values is neither the largest nor the smallest (with regard to sign) of the three principal stresses.

INTERNAL FRICTION

The resisting shear strength considered to be due to the interlocking of the soil grains and the resistance to sliding between the grains.

ISOTROPIC MASS

A mass having the same property (or properties) in all directions.

LINEAR VISCOELASTIC MEDIUM

A medium for which the relationship between stress and strain can be expressed as a linear relationship between stress, strain and their nth order temporal derivatives.

LIQUEFACTION

(See text for a more descriptive definition)

The sudden large decrease of the shearing resistance of a cohesionless soil. It is caused by a collapse of the structure by cyclic or other type of strain and is associated with a gradual but temporary increase of the pore water pressure. It involves a temporary transformation of the material into a fluid mass.

LOGARITHMIC DECREMENT

The natural logarithm of the ratio of any two successive amplitudes of like sign, in the decay of a single-frequency oscillation.

LVDT

Linearly variable differential transducer - an electrical device commonly used to measure the relative displacement between two objects.

MAGNITUDE

A quantity characteristic of the total energy released by an earthquake, as contrasted to "intensity," which describes its effects at a particular place. C.F. Richter devised the magnitude scale in current use, which is in terms of the motion which would be measured by a standard type of seismograph located 100 kilometers from the epicenter of an earthquake.

MAJOR PRINCIPAL STRESS (See principal stress)

The largest (with regard to sign) principal stress.

MICROTREMOR

A feeble earth tremor resulting from natural or man-made forces.

MINOR PRINCIPAL STRESS (See principal stress)

The smallest (with regard to sign) principal stress.

MODULUS OF ELASTICITY

The ratio of stress to strain for a material under given loading conditions; numerically equal to the slope of the tangent or the secant of a stress-strain curve.

MONOTONIC LOADING

Continuously increasing load in one direction.

NATURAL FREQUENCY

Frequency for which phase angle is 90 degrees between the direction of the excited force vector and the direction of the excited excursion vector.

NORMAL STRESS

That stress component normal to a given plane.

OPERATING BASIS EARTHQUAKE (OBE)

For a reactor site, that earthquake which produces the vibration ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional. The maximum vibration ground acceleration of the OBE is equal to

at least one-half that of the design basis earthquake.

OSCILLATION

The variation, usually with time, of the magnitude of a quantity with respect to a specified reference when the magnitude is alternately greater and smaller than the reference.

OVERBURDEN

The generic term applied to uppermost layers of the geologic structure, usually soil layers of low seismic velocity overlaying rock or other high seismic velocity strata.

OVERCONSOLIDATION RATIO (OCR)

The ratio of the maximum past pressure or stress to which a soil has been subjected to the computed value of vertical effective pressure or stress existing in the field at present.

P WAVE (See also compression or body wave)

Body wave in which the direction of the particle motion is the same as the direction of wave propagation. Wave velocity is commonly measured in geophysical refraction surveys to define the contact between and dynamic properties of competent layers (high velocity materials) and softer or less competent layers (low velocity materials), such as bedrock and soil overburden.

PARTICLE ACCELERATION

The time rate of change of particle velocity.

PARTICLE DISPLACEMENT

The difference between the initial position of a soil particle and any later position.

PARTICLE VELOCITY

The time rate of change of particle displacement.

PEAK STRAIN

The maximum strain applied to a soil specimen. This strain may or may not be at failure.

PENETROMETER

A soil sampling device which is pushed or driven with a hammer into the undisturbed soil at the bottom of a boring.

PERIOD

Time interval occupied by one cycle.

PLANE STRAIN (Biaxial)

A measure of strain that takes place in two directions while remaining zero in the third dimension.

PLASTICITY INDEX

The numerical difference between the liquid limit and the plastic limit. These limits are determined in the laboratory by standard tests and serve as a basis for estimating the relative plasticity of a given soil sample.

POISSON'S RATIO

The ratio of the lateral linear strain to the longitudinal linear strain within the elastic behavior of a material subjected to axial load.

PORE WATER PRESSURE

Pressure or stress transmitted through the pore water (water filling the voids of the soil).

PRECURSOR

A waveform characterized by higher than ideal propagation velocity and early arrival times. The waveform is initially observed as a double peaked waveform i.e., the precursor is followed by the main wave.

PRINCIPAL STRESS

Stresses acting normal to three mutually perpendicular planes intersecting at a point in a body, on each of which the shearing stresses are zero.

RAYLEIGH WAVE (Surface)

Dispersive surface wave in which element has retrograding elliptic orbit with one major vertical and one minor horizontal component both in plane of propagation velocity.

RELATIVE DENSITY

The ratio of: (1) the difference between the void ratio of a cohesionless soil in the loosest state and any given void ratio, to (2) the difference between its void ratios in the loosest and in the densest state.

REPEATED LOADING

Process of loading and unloading of a soil specimen.

RESONANCE

The reinforced vibration of a body exposed to vibration, at about the frequency of another body.

RESONANT FREQUENCY

A frequency at which resonance exists.

RESPONSE

The motion in a system resulting from an excitation under specified conditions.

RESPONSE SPECTRUM

The maximum response of a linear, single-degree-of-freedom spring mass system, relative to the motion of the ground, as a function of the undamped natural frequency of the spring mass system.

S WAVE (Shear wave)

Body wave in which the particle motion is at right angles to the direction of wave propagation. Wave velocity may be measured by in-hole geophysical procedures to determine the dynamic shear moduli of the materials through which the wave passes.

SAFE SHUTDOWN EARTHQUAKE

For a reactor site, that earthquake which produces the vibratory ground motion, for which those features of the nuclear power plant necessary to shut down the reactor and maintain the plant in a safe condition without undue risk to the health and safety of the public are designed to remain functional.

SATURATED SOIL

Soil with zero air voids. A soil which has its interstices or void space filled with water to the point where runoff occurs.

SECANT MODULUS

The slope of a stress-strain curve defined by a line connecting the zero stress strain axis with the point of peak stress and strain on the actual curve.

SEISMICITY

A measure of the probability of earthquake occurrence in an area.

SEISMOMETER

Instrument to pickup linear (vertical, horizontal) or rotational displacement, velocity, or accelerations.

SHEAR MODULUS (Rigidity)

The ratio between shear stress and shear strain in simple shear.

SIMPLE SHEAR

The state of stress at the point where only shearing stresses act on any two perpendicular planes.

SOIL DYNAMICS

The study of the engineering properties of soils as they are affected by transient impulsive loading.

STANDING WAVE

A wave produced by simultaneous transmission in opposite directions of two similar waves resulting in fixed points of zero amplitudes called nodes.

STEADY-STATE VIBRATION

Vibration in a system where the velocity of each particle is a continuing periodic quantity.

STRAIN

The change in length per unit of length in a given direction.

STRAIN DEPENDENT PROPERTY

That property of soil, the magnitude of which depends on the magnitude of the induced strain.

STRENGTH OF EARTHQUAKE

The peak ground acceleration recorded or predicted for a particular earthquake, expressed usually in percent of acceleration of gravity.

STRONG MOTION

Ground motion of sufficient amplitude to be of engineering interest in the evaluation of damage due to earthquakes.

UNDISTURBED SAMPLE

A soil sample that has been obtained by methods in which every precaution has been taken to minimize disturbance to the sample.

UNDRAINED SHEAR STRENGTH

The shear strength of a soil in which the pore water is not allowed to escape from the specimen during testing.

VANE SHEAR TEST

An in-place shear test in which a rod with thin radial vanes at the end is forced into the soil and the resistance to rotation of the rod is determined.

VIBRATION

An oscillation wherein the quantity is a parameter that defines the motion of a mechanical system.

VISCOELASTIC MEDIUM

A stress-strain relationship in which the stress is a function of both strain and strain-rate, though not necessarily proportional to both.

VISCOSITY

The cohesive force existing between particles of a fluid which cause the fluid to offer resistance to a relative sliding motion between particles; internal fluid friction.

VISCOUS DAMPING

The dissipation of energy that occurs when a particle in a vibration system is resisted by a force that has a magnitude proportional to the magnitude of the velocity of the particle and direction opposite the direction of the particle.

VOID RATIO

The ratio of the volume of void space to the volume of solid particles in a given soil mass.

WAVE LENGTH

Normal distance between two wave fronts with periodic characteristics in which amplitudes have a phase difference of one complete cycle.

YOUNG'S MODULUS

The ratio of unit stress to unit strain within the elastic range of a material under a given loading condition, numerically equal to the slope of the tangent of a stress-strain curve.

APPENDIX E

ANNOTATED BIBLIOGRAPHY

APPENDIX E
ANNOTATED BIBLIOGRAPHY

This is an annotated bibliography of selected references relative to the general subject of "Soil Behavior under Earthquake Loading Conditions". References included have largely been published after 1960, except for certain references considered to be particularly pertinent to this subject. In the interests of brevity we have excluded references which, in our opinion, do not deal specifically with factors important to the subject.

The references are listed alphabetically by author and where more than one article is published by the same person, the arrangement is chronological by date of the publication. The annotations included have come primarily from the following sources:

- 1) Geodex Soil Mechanics Information Service and Geotechnical Abstracts
- 2) Publications of the American Society of Civil Engineers
- 3) Engineering Index
- 4) Nuclear Science Abstracts
- 5) Nuclear Safety Information Center
- 6) Technical Abstract Bulletin
- 7) Defense Documentation Center bibliography request
- 8) U.S. Government Research & Development Reports
- 9) Geophysical Abstracts
- 10) The actual book or author's abstract

Where annotations were not readily available, generally from the article or book itself, or from other sources, they were written as a part of this study.

AISI, 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.

A design case history for the San Francisco transbay tube which takes into account structural design of the tunnel and the foundation soils with respect to nearby active faults. Considered are the problems of sand liquefaction and loss of strength of clay soils during future earthquakes among other possible design problems. In addition to geophysical surveys performed at the site to evaluate dynamic soil moduli and Poisson's ratio, undisturbed samples of both cohesive and cohesionless soil were taken for dynamic laboratory testing. Both cyclic triaxial and confined cyclic simple shear tests were made from these samples. Damping factors (expressed as a percent of critical damping) are obtained from cyclic testing. The moduli obtained from geophysical and laboratory testing are compared.

AKAI, K., and YAMAUCHI, Y. (1968)

"Response Characteristics of Saturated Clay to Impact Loading," Soils and Foundations (Japan), Vol. 8, No. 2, June, pp. 13-34.

Cylindrical samples of saturated clay were formed by mixing at a high water content, and then consolidated up to pressures of 1.0 kg per sq cm. The samples were then subjected to one of two types of dynamic loading. The first type of loading consisted of dropping a weight on the end of the sample and measuring the resulting vibrations. The size of the weights ranged from very small up to large enough to cause failure. The second type of test was similar, except that the sample was subjected to a dynamic torsion load and resulting torsional vibrations of a heavy weight mounted on the sample were measured. Values of Young's modulus, shear modulus and damping were obtained, and are summarized for various water contents and stress intensities.

AKI, K., and LARNER, K.L. (1970)

"Surface Motion of a Layered Medium Having an Irregular Interface Due to Incident Plane SH-Waves," Jour. of Geophysical Res., Vol. 75, No. 5, Feb., pp. 933-954.

A practical method is devised to calculate the elastic wave field in a layer-over-half-space medium with an irregular interface, when plane waves are incident from below. This method may be used for studying the interface shape of the M discontinuity, for example, using the observed spectral amplitude and phase-delay anomalies due to teleseismic body waves. The method is also useful for the engineering-seismological study of earthquake motions of soft superficial layers of various cross sections. The scattered field is described as a superposition of plane waves, and application of the continuity conditions at the interface yields coupled integral equations in the spectral coefficients. The equations are satisfied in the wave-number domain when the interface shape is made periodic and the equations are Fourier transformed and truncated. Frequency smoothing by using complex frequencies reduces lateral interferences associated with the periodic interface shape and permits comparison of computed results with those obtained from finite bandwidth observations. Analyses of the residuals in the interface stress and displacement, performed for each computed solution, provided estimates of the errors. The relative root-mean-square residual errors were generally less than 5% and often less than 1% for problems in which the amplitude of the interface irregularity and the shortest wave length were comparable. The method is applied to several models of 'soft basins,' 'dented M discontinuity' and 'stepped M discontinuity.' The results are compared with those derived from the flat-layer theory and from the ray theory. In addition to vertical interference effects familiar in the flat-layer theory, we observe the effects of lateral interference as well as those of ray geometry on the motion at the surface.

AKINO, K., OTA, T., and YAMAHARA, H. (1969)

"Seismic Observation of Rigid Structure on Various Soils and Its Review," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 1, No. A2, pp. 38-52.

Three identical rigid concrete boxes were constructed each on a different soil and the results of artificial forced vibration tests and recorded earthquake response of these structures is briefly analyzed. The paper assumes the form of a progress report, indicating that much more data is available than can be properly analyzed or presented at this time. The test structures were 3.5 m square in plan and 7.4 m high, buried to a depth ranging from 2.5 to 4.4 m. The soil conditions were respectively described as gravel, silty rock and loam. Detailed profiles to a depth of 20 m are presented. Forced vibration tests were performed before and after backfilling to obtain modulus and damping properties of the foundation for use in theoretical response analyses. Backfilling produced a strong notice-

able effect. Examples of accelerograms obtained from various locations in and around the structures are presented, along with analysis of the data. Their theoretical model gave good predictions of the earthquake motion provided proper values of modulus and damping were used but the analysis was sensitive to these parameters.

ALGERMISSEN, S. T. (1969)

"Seismic Risk Studies in the United States," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 1, pp. A1, 14-27.

A new Seismic Risk Map of the United States is presented, together with strain release and maximum Modified Mercalli intensity maps of the country. Frequency of occurrence of damaging earthquakes was not considered in assigning ratings to the various zones on the risk map, but studies of earthquake frequency are included as an aid in using the risk map. The Seismic Risk Map is suggested as a revision of the Seismic Probability Map prepared by the Coast and Geodetic Survey in 1947 and withdrawn in 1952.

AMBRASEYS, N.N., and SARMA, S. (1969)

"Liquefaction of Soils Induced by Earthquakes," Bull., Seismol. Soc. of Am., Vol. 59, No. 2, Apr., pp. 651-664.

The conditions under which liquefaction effects are known to have occurred are reviewed and their generic cause is discussed. Special attention is given to the delay with which liquefaction effects appear on surface deposits originally stable. It is suggested that little damage to structures on poor soil can occur as the direct result of severe earthquake shaking and that damage observed is almost entirely due to foundation failures, ground settlements, and tilting. Field evidence suggests that earthquakes are incapable of densifying originally loose deposits to a stable mass, and that such deposits neither experience nor transmit severe shaking.

APPUHN, R.A. (1964)

"A Shaking Table Experiment to Determine the Seismic Response of Soft Ground," M.S. thesis, Univ. of Calif., Berkeley.

The motion of soil filling a model bedrock basin was observed during continuous vibration on a laboratory shaking table. The following factors were varied in different experiments: consistency of the soil; thickness of the soil fill; length-to-width ratio of the basin; and frequency and intensity of "bedrock" motion.

ARAI, H., and UMEHARA, Y. (1966)

"Vibration of Dry Sand Layers," Proc., Japan Earthquake Engineering Symposium, Tokyo.

Acceleration responses of dry sand layers were obtained from the results of shaking table experiments. Using the shaking table excited by the electro-dynamic force, the authors carried out some tests in four cases of sand layers made in the box on the shaking table with 5m long, 1.5m wide and 1.5m high, i.e., one of them is 75cm thick in the dense state and the others are 80cm, 60cm, 40cm thick respectively, in the loose state. The sand is fine grained sand called Onahama Sand with the effective size of 0.13mm, the uniformity coefficient of 1.31 and the angle of repose of 32.5°. Table accelerations and the accelerations in some places of the sand layer were measured by the unbonded type wire strain gage accelerometer. By applying the shear vibration theory to the obtained acceleration responses, the authors estimate the shear wave velocities and the damping constants of dry sand layers.

ARANGO, I. (1971)

"Seismic Stability of Slopes in Saturated Clay," Ph.D. thesis, Univ. of Calif., Berkeley.

Studies have been made of the behavior of banks of saturated clay under simulated earthquake conditions. Shaking table experiments and simple shear tests were carried out using a soft clay having the desirable characteristics with regard to strength and permeability. The results of the studies have been used to develop a simple method for predicting slope deformations under seismic loading conditions.

The shaking table studies were used to obtain a mechanistic picture of the deformations of the slopes of clay banks during earthquakes, the extent of such deformations and the factors influencing them. The simple shear tests were performed to compare the soil behavior under cyclic loading in the two different conditions: large shaking table structures representative of actual field slopes and smaller samples of clay capable of rather simple testing in the laboratory.

ARCE, E.M., et al. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

BALADI, G. Y. (1970)

"Theoretical Investigation of the Half Wavelength Theory," Misc. Paper S-70-28, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Dec.

The research reported was initiated to derive a method for predicting the thicknesses of layers in a layered medium by use of induced forced-wave motion at the surface of the medium. Each layer of the medium was assumed to be homogeneous, isotropic, elastic, and in welded contact with both the layer above and beneath it. The general differential equations of motion were employed separately in each layer of the medium. The mathematical difficulties of solving n-layer system were overcome by using the condition at the interface of two adjacent layers. The solution presented herein was found to give satisfactory results and to be more accurate than the half wavelength criterion which simply states that the calculated modulus applies to the material at a depth equal to one-half the surface wavelength. It is recommended that an extensive field test on controlled layered systems be conducted and complete descriptions of the dispersion curves for a multilayer system be obtained in order to verify the solution.

BALLARD, R.F., JR. (1964)

"Determination of Soil Shear Moduli at Depths by In-Situ Vibratory Techniques, Misc. Paper No. 4-691, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Dec.

The tests described were conducted to investigate the possibility of determining by in-situ field measurements the dynamic moduli of soils at depths approaching or exceeding 100 ft. Tests were performed at several sites representative of various soil materials ranging from sand to rock. During the test program, various mechanical and instrumentation improvements were made in test equipment. Specifically, the hydraulic-powered, fixed-mass vibrator initially used was replaced by a variable-mass vibrator, and a double-amplification technique was utilized in conjunction with a low-frequency band pass filter to increase the amplitude and quality of the transducer signal.

Data were acquired utilizing the fixed-mass vibrator at three sites: Eglin Air Force Base, Fla., in sand; Waldorf, Md., in cohesive soil; and Frenchman Flat, Nev., in cohesive soil. Wave penetration at these sites extended to depths of 20, 37 and 45 ft, respectively.

After construction of the variable-mass vibrator, data were acquired with this vibrator at four sites: U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., in cohesive soil; Sacramento Peak, N.Mex., in rock; and Buckboard Mesa and Area 15 at the Nevada Test Site, Nev., both in rock. Investigational depths attained at these sites were 120, 86, 90, and 170 ft, respectively. Although the variable-mass vibrator with improved instrumentation was not tested on sand, results of tests on other materials indicate that depths greater than the 20-ft depth investigated at Eglin with the fixed-mass vibrator can almost certainly be penetrated with the variable-mass vibrator.

BALLARD, R.F., JR. (1967)

"A Vibratory Technique for Delineating Subsurface Fracture Zones Resulting From Detonations," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., Aug., pp. 891-903.

Test results of shallow geophysical surveys conducted in the immediate vicinity of blast craters indicate their feasibility in establishing zones of fractured and intact rock. The surveys ran radially outward from the crater with an additional series of perpendicular surveys. The surveys consisted of establishing the length of surface (Rayleigh) waves using a vibrator and movable pick-ups. Vibrator frequency was varied from 9 to 100 hz. The shear wave velocity of the rock was calculated for the various locations and depths. Contours of the change in shear wave velocity with depth indicate the probable limits of major and minor disturbance.

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.

This report describes the development of the U.S. Army Engineer Waterways Experiment Station (WES) borehole vibropacker system and a technique for applying the apparatus for generation of vertically polarized shear waves at any predetermined point within an existing borehole. Methods of receiver and transducer placement and orientation are also discussed. In addition, techniques of data acquisition and interpretation are presented along with comparisons of

data from in-situ surface vibrations and laboratory dynamic shear test data. When comparing shear and compression wave velocities, it was found that a good correlation exists between the three comparable test methods: the vibropacker system, surface vibrations, and laboratory testing. As a result of this feasibility study, it was determined that the vibropacker system will prove to be a useful tool for the determination of shear and compression wave velocities.

BANKS, D.C., and STROHM, W.E., JR. (1965)

"Methods of Preventing Flow Slides," Potamology Inv. Report 12-16, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Oct.

This report summarizes a study of investigations of liquefaction of sands accomplished since 1956 with the purposes of (a) updating Potamology Investigations Report 12-5, A Review of the Soils Studies, by Dr. M. J. Hvorslev, which was published in June of 1956, and (b) evaluating methods of stabilizing potentially unstable deposits along revetted reaches of the banks of the Lower Mississippi River. The ultimate objective of the study was to determine the most promising method of stabilization for field testing.

Recent literature describing liquefaction phenomena, occurrence of flow slides, results of laboratory and field tests dealing with liquefaction of sands, and methods for densifying loose sand and silt deposits is summarized in this report. The results of the literature review support and extend the findings of Report 12-5. However, no data were found which provide the information that would be obtained from the additional investigations recommended in Report 12-5 on development and progress of arcuate-shaped liquefaction failures, including information on development of excess pore pressures during such failures, and more detailed information on associated soil conditions. These recommended investigations contained in Report 12-5 are still considered desirable, but field experiments can be undertaken now utilizing currently available information to investigate the feasibility of improving the stability of loose sand deposits with respect to flow failure by densification.

The most suitable procedure is considered to be densification by the vibroflotation process using the results of an extensive soil investigation before and after densification to determine the success of the densification procedure. The use of explosives to densify loose sand is not considered suitable at present because of the large amount of preliminary experimental investigation that would be required to determine optimum charge weights, depths, spacings, and other factors for the range of conditions to be found at potentially unstable sites.

Field experiments to evaluate stabilization by densification are proposed.

BARANOV, V., and KUNETZ, G. (1960)

"Film Synthetique avec Reflections Multiples Theorie at Calcul Pratique," Geophysical Prospecting, Vol. 8, No. 2, June, pp. 315-325.

A short summary of the theory of synthetic seismograms, including all multiple reflections and a description of the method of their construction with the use of an electronic computer are given.

BARKAN, D. D. (1962)

Dynamics of Bases and Foundations, McGraw-Hill Book Co., New York.

This book includes a discussion of the elastic and vibratory properties of soils bases, the theory and practice of the design of foundations under machines with dynamic loads, questions of wave propagation from industrial sources, and problems of the effect of these waves on structures.

BARKAN, D.L. (1967)

"Developments in Soil Dynamics," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., Aug., pp. 599-606.

A concise discussion of the history, development and state of the art of soil dynamics in the USSR. All phases of foundation vibrations are mentioned; including diverse machines, dynamics of hydraulic engineering structures, dynamic properties, wave propagation, plastic settlements due to impact loadings, pile driving, blast loading, compaction.

BAZANT, Z. (1965)

"Stability of Saturated Sand During Earthquake," Proc., 3rd World Conf. on Earthquake Eng., New Zealand, Vol. 1, pp. 1:6-21.

There are many examples of liquefaction and of great subsidence in earthquake areas underlain by saturated sand. Therefore, it is of interest to know how the stability of sand is affected by the earthquake. This case is studied as the state of stress in dynamically excited soil. The acceleration of an earthquake creates either dynamic stability of sand, compaction, or liquefaction. Criteria differentiating these states are the critical acceleration and the liquefactional acceleration. An equation is given for predicting the behavior of saturated sand during earthquakes for a given dry density, height, and volume of sand layer.

BAZANT, Z., and DVORAK, A. (1965)

"Effects of Vibrations on Sand and the Measurement of Dynamic Properties," Proc., 6th Internat. Conf. on Soil Mech. and Found. Eng., Montreal, Vol. 1, pp. 161-164.

Presents a short review of some recent experimental and theoretical work in this area. As accelerations increase, sand will assume four states: dynamic stability, compaction, liquefaction and motion. Results of dynamic loading of small blocks resting on sand are used to obtain the dynamic modulus of elasticity of the foundation soil.

BAZARRA, A.R.S.S. (1967)

"Use of the Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sand," Ph.D. thesis, Univ. of Ill., Urbana.

On the basis of a literature and field test study a technique is developed for correcting the measured standard penetration N-values for the effect of overburden pressure, for using these corrected N-values to estimate the settlement of shallow foundations on dry sand and for estimating the effect of submergence on settlement. The correction technique is shown to make it possible to compute the relative density of sand with reasonable accuracy. On the basis of a correlation between the corrected N-values, the results of plate load tests at 18 different sites and the results of settlement measurements of actual structures on sand, a new settlement chart was developed for estimating the settlements of shallow foundations on dry sand. From a study of the settlement observations for 21 structures on sand, the ratio of predicted settlements to measured values was found to vary between 0.80 and 2.50, with an average of about 1.5.

BISHOP, A.W., and HENKEL, D.J. (1962)

The Measurement of Soil Properties in the Triaxial Test, 2nd ed., Edward Arnold Ltd., London.

Excellent text presenting principal features of the triaxial apparatus, details of technique and equipment for standard and special tests. In Appendices discusses effects of rubber membrane, drains, proving ring ram, friction, testing rate and air trapped between sample and membrane. Appendix 6 of 2nd Ed. reviews recent developments re-measurement of pore pressures in partly saturated soils and effect of testing rate on pore pressures.

BJERRUM, L. (1954)

"Theoretical and Experimental Investigations on the Shear Strength of Soils," Publication No.5, Norwegian Geotechnical Institute.

Theoretical investigations and laboratory tests to evaluate the relation between deformation properties and pore water pressures in clay.

BLUME, J. A. (1965)

"Earthquake Ground Motion and Engineering Procedures for Important Installations Near Active Faults," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 3, pp. IV:53-71.

Costly or sensitive buildings, structures, and facilities are being constructed in active earthquake areas throughout the world. Building codes and normal design practices are not always compatible with the inherent risks. This paper outlines procedures for site studies and site selection, evaluation of seismic probability and risk, a new approach to ground motion based upon soil and rock properties, response spectra, inelastic design criteria, and related matters, in the light of current (limited) knowledge. Hypothetical examples are based upon experience with nuclear power plants and major research facilities close to active faults. Analytical, empirical, philosophical, and practical considerations are reconciled.

BLUME, J.A., SHARPE, R.L., and KOST, G. (1971)

"Earthquake Engineering for Nuclear Reactor Facilities," JAB-101, John A. Blume & Associates, San Francisco, Calif.

Discusses state-of-the-art techniques and procedures, many of which are currently used by groups designing nuclear power plant facilities. The procedures discussed are based on extensive experience in design and analysis of structures of all types -- both in the United States and abroad -- for resistance to earthquake-induced forces.

BONILLA, M.G. (1970)

"Surface Faulting and Related Effects," Earthquake Engineering, R. L. Wiegel, Coordinating Ed., Prentice-Hall, Inc., Englewood Cliffs, N.J., pp. 47-74.

Provides a discussion of surface manifestations of faulting, record of historical surface faulting in Continental United States and adjacent parts of Mexico, geographical distribution of historic surface faults, relationship between fault length and earthquake magnitude, and discussion of the location of likely future surface faulting.

BOUTWELL, G.P., JR. (1968)

"On the Yield Behavior of Cohesionless Materials," Ph.D. thesis, Duke Univ.

The Roscoe concept of the "critical state" for failure of sand is verified by drained and undrained triaxial extension and compression tests on sands prepared at varying initial densities and tested at varying initial effective radial stresses. A comparative evaluation was made of the various approaches proposed for separating the observed shear strength into its components of friction and dilation. Three separate phases of the undrained behavior of sand are defined. An analytical relationship is established for the pore pressure parameters which describes the pore pressure changes developed in the undrained condition during yield as a function of the deviatoric stress state causing that yield. Upon unloading from undrained shear failure, a residual pore pressure remained. The causes of liquefaction were evaluated by cyclic stress reversal tests.

BRADY, A.G. (1966)

"Studies of Response to Earthquake Ground Motion," Ph.D. thesis, Calif. Inst. of Technol.

A study of the accuracy of electronic digital computer calculations of ground displacement and response spectra from strong-motion earthquake accelerograms indicates that the main errors are those involved in digitizing the original record. Standardized methods of adjustment and calculation are recommended to minimize such errors. The spread of response spectral values about their mean was investigated experimentally by Monte Carlo techniques employing a combination of analog and digital computations. A standard method of analysis for the study of comparative ground motion at different sites is recommended.

CALHOUN, C.C., JR., and FLANAGAN, C.F. (1969)

"Verification of Empirical Method for Determining Riverbank Stability, 1967 Data," Potamology Investigations Report 12-20, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Apr.

This report is the fifteenth of a series in which new data obtained from borings made for revetment construction are analyzed to determine the applicability of an empirical method for predicting riverbank stability with regard to flow (liquefaction) failure. Boring data obtained in 1967 are analyzed, and stability predictions are made for 12 new areas. Failures during 1967 at sites previously analyzed also are discussed.

Based on analyses made in 1958 of previous performance data, the classification criteria for zone A and zone B sands were modified in 1959. The failures at sites previously studied, new site predictions, and current year performance are analyzed using the modified criteria.

The modified criteria have proved reliable, with only seven exceptions, in predicting flow failure. However, many locations predicted to be unstable have not experienced flow failure, and it is possible that either the density of zone A sand may be such that flow failure will not occur or the severity of river attack has not been sufficient to initiate flow failure.

CASAGRANDE, A., and CASAGRANDE, L. (1968)

"Donald C. Cook Nuclear Power Plant, Preliminary Safety Analysis Report, Amendment 5; Appendix G, Report to American Electric Power Service Corporation on Foundation Investigations for the Donald C. Cook Nuclear Power Plant," Docket 50315-9, U.S. Atomic Energy Commission, Sept. 6.

The authors report the results of a foundation investigation for the proposed Donald C. Cook Nuclear Power Plant. Particular attention is given to the in situ and laboratory determination of the relative density of the site's subsurface materials. Different

drilling procedures are used and the penetration resistance measured using the Standard Penetration Test. The results obtained are compared and in essence point out the pitfalls, imperfections and variables of the test.

CASAGRANDE, A., and WILSON, S. D. (1960)

"Testing Equipment, Techniques and Errors," (Moderator's Report), Proc., ASCE Research Conf. on Shear Strength of Cohesive Soils, Univ. of Colo., pp. 1123-1130.

The authors point out and discuss thoroughly four errors commonly present in the triaxial test. These errors include 1) those due to piston friction, 2) those due to migration of water and air through the membrane, 3) those due to pore pressure measuring equipment and techniques and 4) those due to non-uniform distribution of stresses and water content induced in specimen during axial loading.

CASTRO, G. (1969)

"Liquefaction of Sands," Harvard Soil Mech. Series No. 81, Jan.

To imitate as closely as possible the stress-strain conditions that develop in an element within a large mass of sand, consolidated-undrained triaxial compression tests with load control and lubricated end surfaces were used. With this automatic recording apparatus it was possible to achieve systematically liquefaction of loose, saturated sand specimens, and to investigate the factors that control liquefaction. On loose specimens consolidated anisotropically the additional deviator stress which must be applied to the specimens in order to trigger the liquefaction failure is very small for principle stress-ratios after consolidation greater than 1.5. This is of particular significance in relation to the occurrence of spontaneous liquefaction in nature because sand in slopes is always consolidated anisotropically. When such a slope is subjected to a small but rapidly applied strain, additional shear stresses are induced at constant volume conditions which may be sufficient to trigger a liquefaction failure. The effect of cyclic loads was also investigated.

CAUGHEY, T.K., and O'KELLY, M.E.J. (1965)

"Classical Normal Modes in Damped Linear Dynamic System," Jour. of Applied Mech., Sept., pp. 583-588.

Necessary and sufficient conditions under which both discrete and continuous damped linear dynamic systems possess classical normal modes are determined.

CAUGHEY, T.K., and STUMPF, H.J. (1961)

"Transient Response of a Dynamic System Under Random Excitation," Jour. of Applied Mech., Dec., pp. 563-566.

This paper analyzes the transient response of a simple harmonic oscillator to a stationary random input having an arbitrary power spectrum. The application of the results of this analysis to the response of structures to strong-motion earthquakes is discussed.

CHAE, Y.S., HALL, J.R., and RICHART, F.E., JR. (1965)

"Dynamic Pressure Distribution Beneath a Vibrating Footing," Proc., 6th Internat. Conf. on Soil Mech. and Found. Eng., Montreal, Vol. 2, pp. 22-26.

A series of concentric steel rings up to 12 in. dia. were connected to a solid plate by means of SR-4 type load cells and placed on a box of Ottawa sand. A controlled frequency vibrator excited the footing in vertical vibrations. Loads were added to the footing to apply different average contact pressures. Measurements of the dynamic contact pressures and phase angles were made and provided data for correlation with theoretical formulas.

CHOPRA, A.K. (1967)

"Earthquake Response of Earth Dams," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM2, Mar., pp. 65-81.

An investigation demonstrating that the earthquake response of earth dam sections cannot satisfactorily be evaluated by the widely used shear wedge approach. A two-dimensional dynamic stress analysis technique is advocated which is based on a finite element concept, assumes homogeneous isotropic, linearly elastic material, and ignores the influence of

stored water. Frequencies, mode shapes, and responses to the El Centro earthquake are determined by both methods. It is demonstrated that the wedge method overestimates the fundamental frequency of vibration, and the deformations are shown to differ appreciably from the pure shear deformations assumed in the shear wedge theory. The finite element method reveals that stresses near the faces of the dam depend significantly on the side slopes.

CLOUGH, R.W., and CHOPRA, A.K. (1966)

"Earthquake Stress Analysis in Earth Dams," Jour. of the Eng. Mech. Div., ASCE, Vol. 92, No. EM2, Apr., pp. 197-212.

The finite element method, which has been used extensively in the static analysis of elastic continua, is extended to the dynamic analysis of two-dimensional stress systems. The method involves calculation of vibration mode shapes and frequencies, making use of a stiffness matrix evaluated by the finite element method and a mass matrix formed by lumping the mass at the finite element nodal points. The dynamic response is then calculated by the mode-superposition method, the response of each mode being obtained by step-by-step integration. The method is demonstrated by the earthquake analysis of a triangular earth-dam cross section treated as an elastic plane-strain problem. Vertical and horizontal acceleration components of the 1940 El Centro earthquake are applied; results are presented in the form of plots showing the time history of stresses at selected points in the cross section and also plots of stress contours at selected instants of time.

COFFEY, D.D. (1966)

"Methods of Advancing Boreholes and Taking Disturbed Samples," (Session Leader's Address), Proc., Site Investigation Symposium, Sydney, Inst. of Civil Eng., Australia, 1966; Civil Eng. Trans., Vol. CE9:1, Apr., 1967, pp. 5-9.

Details of methods (currently in use in Australia) of advancing test bores and taking disturbed samples. For the various types of auger, percussion, wash boring and rotary drilling, the equipment used and applications recommended are discussed and presented in tabular form. Finally, for each boring and equipment type the possible disturbed and undisturbed sampling techniques are mentioned.

CONVERSE, F.J. (1961)

"Stress-Deformation Relations for Soft Saturated Silt Under Low-Frequency Oscillating Direct-Shear Forces," Special Technical Pub. No. 305, Symposium on Soil Dynamics, ASTM, pp. 15-19.

Oscillating direct shear tests on soft saturated mud (from San Francisco Bay) indicated that increases of deformation amplitudes result in lower shearing moduli and greater energy loss per cycle under simulated earthquake conditions.

COX, J.B. (1966)

"A Review of the Vane Test," Proc., Site Investigation Symposium, Sydney, Inst. of Civil Eng., Australia, 1966; Civil Eng. Trans., Vol. CE9:1, Apr., 1967, pp. 19-30.

A review of the theory and practice of the vane test, beginning with an outline of its development and details of the vane apparatus. The theory of the vane test in $\phi = 0$ materials (i.e., "end of construction" conditions in non-fissured, saturated clays) is presented in detail and shown to be amply confirmed by laboratory and field test results, including in-situ failures. On the other hand, in $C-\phi$ materials (fissured clay, sandy clays and sands) there is a discrepancy between proposed theories and experimental vane test results, so that the use of the vane test appears to be limited in view of the change in soil conditions on inserting the vane into the soil.

COYLE, H.M., and GIBSON, G.C. (1970)

"Empirical Damping Constants for Sands and Clays," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM3, May, pp. 949-965.

The results of a laboratory investigation of the damping properties of saturated sands and clays of relatively high plasticity are reported. An effort is made to relate these damping properties to common physical properties of the soils tested. A series of dynamic (impact) and static tests were performed on a variety of sand and clay soils. The sands varied in grain size and grain shape, and the clays varied in plasticity and moisture content. Velocity of sample deformation, peak dynamic loads, and peak static loads were measured so

that damping constants for the soils could be evaluated. A mathematical model in current use which describes soil action at the point of a pile is examined. The model is used in wave equation studies of piling behavior. A modification of this model is made in order to achieve a constant damping value over the full range of loading velocities. Damping constants are evaluated for all soils tested. The damping constant is related to void ratio and effective angle of internal shearing resistance in sands, and moisture content and liquidity index in clays.

CRANDALL, S.H., CHANDIRAMANI, K.L., and COOK, R.G. (1966)
"Some First-Passage Problems in Random Vibration," Jour. of Applied Mech., Sept., pp. 532-538.

First-passage problems for a lightly damped linear oscillator excited by white noise are described for two types of initial conditions and three different types of barrier configurations. The qualitative behavior of the probability distribution of the time to first-passage is discussed. Two numerical procedures utilizing high-speed digital computers are outlined, and quantitative results for first-passage probability densities are displayed. The asymptotic behavior of this density for long mean times to failure is discussed, and a table of experimentally determined mean times to failure is included.

DASA (1968)
"News Letter," Computer Code Series No. 3, Information and Analysis Center, Oct.

This issue of the newsletter is devoted to a survey of computer codes which may be used for calculating ground shock and cratering induced by nuclear explosions. The code descriptions presented here have been compiled chiefly from reports contained in the DASIAC document collections. The list is undoubtedly incomplete, but we hope it is representative of the type and number of codes available. DASIAC should be informed of any significant omissions so that they may be presented in a future issue of the newsletter.

For presentation, the codes are grouped into two categories: (1) codes designed for specific calculations needed in ground shock studies; (2) general purpose "hydrodynamic" codes which can be adapted to ground shock as well as other types of problems. Some of the codes in the latter category may never have been used for ground shock calculations but they all appear to be sufficiently general to handle this type of problem.

Several codes provide sufficiently general capability to allow calculation of soil response to earthquake bedrock motion at intensities considerably below those considered in nuclear explosion ground shock calculations.

D'APPOLONIA, E. (1970)
"Dynamic Loadings," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM1, Jan., pp. 49-72.

Criteria for placing and improving soil sustaining dynamic loadings are presented for building and machinery foundations and for earth embankments. These criteria are compared and their limitations in practice without consideration of the response of the soil and structure to dynamic loadings are illustrated. Vibratory-induced settlement of footings supported on granular soils depends on the dynamic stresses and accelerations. For accelerations less than 1g, for most practical cases, accelerations per se do not cause sand densification. Rather, accelerations produce inertia stresses whose continued repetition results in orientation of sand grains and settlement. Confining pressure, density, grain size, dynamic stress, its magnitude and repetition, etc., influence liquefaction of granular soils. These factors are considered in appropriate design criteria for foundations. Analytical procedures developed by Seed, Ambraseys and Sarma, and Newmark for stability and deformations of earth dams in seismic regions are presented.

D'APPOLONIA, D.J., and D'APPOLONIA, E. (1967)
"Determination of the Maximum Density of Cohesionless Soils," Proceedings, 3rd Asian Regional Conf. on Soil Mech. and Found. Eng., Haifa, Vol. 1, pp. 266-268.

A study of the use of a laboratory vibration test for determining the maximum density of a cohesionless soil (air-dry dune sand). Tests in an apparatus permitting independent variation of amplitude, frequency and surcharge showed that the compacted density depended both on the peak acceleration of vibration and on the surcharge. The optimal relationship between density and peak acceleration must be derived for each soil tested in order to establish the maximum density. Typical variations are plotted.

D'APPOLONIA, D.J., and D'APPOLONIA, E. (1970)

"Use of the SPT to Estimate Settlement of Footings on Sand," Proc., Symp. on Footings on Interbedded Sands, South Perth, Australia, Oct. (Preprint? publication not verified)

Reviews and evaluates the several methods for estimating the settlement of footings on sand that employ the Standard Penetration Test (SPT). Although the SPT has many shortcomings, experience over the years has confirmed that the SPT can be used to interpret, at least qualitatively, the character of in-place sand deposits. In North America and certain other parts of the world, the SPT is widely used in the design of shallow foundations on sand. The main reason for this practice is the empirical correlation developed by Terzaghi and Peck in 1948 relating the allowable bearing stress to the blowcount from the SPT. This correlation has been used extensively and successfully. However, several recent papers have pointed out that it is overly conservative and a number of modifications have been suggested. In the past year Peck has also recommended modifications to the original Terzaghi-Peck correlation to make it less conservative. The authors have suggested a somewhat different approach for using SPT data to estimate footing settlement. This method employs a correlation between SPT and sand modulus established from field case studies together with elastic theory to estimate footing settlement.

DE GRAFT-JOHNSON, J.W.S. (1967)

"The Damping Capacity of Compacted Kaolinite Under Low Stresses," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., pp. 771-779.

An experimental study on compacted clay employing the free vibration characteristics to evaluate the shear moduli and damping properties. A sample is tested by partially loading a cylindrical sample axially with a rigid mass. The full weight of the mass is suddenly transferred to the sample. The resulting deformation and axial load produces a decay curve. The specific damping capacity is measured from the logarithmic decrement while the shear modulus is derived from the period of oscillation. Various molding water contents and masses were used along with kneading and static compaction to observe the response under these variables. The damping capacity and modulus are influenced by stress level, water content and clay structure. The damping capacity increases with molding water content. The elastic modulus decreases with increasing molding water content. These effects are much more pronounced when the samples are prepared by kneading compaction, which shows the influence of soil structure on test results.

DE MELLO, V.F.B. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971

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.

The finite element method of analysis provides a powerful tool for analyzing the response of soil deposits underlain by sloping rock boundaries. As it is customarily used, however, the mode superposition procedure for applying this analysis does not take into account the variability in the damping ratio associated with different regions of a soil deposit. Procedures are described which attempt to compensate for this shortcoming of the method. Analytical results are presented to show the influence of the inclination of the rock surface, the thickness of the soil deposit and the configuration of the deposit on the ground surface response for a range of soil types. It is shown that while the magnitude of the ground surface response in the vicinity of a sloping rock surface may be significantly influenced by the presence of the slope, the frequency characteristics are only slightly affected. Conclusions are presented concerning the effects of sloping rock surfaces on the directional ground surface accelerations in adjacent soil deposits.

DEZFULIAN, H., and SEED, H.B. (1971)

"Response of Nonuniform Soil Deposits to Travelling Seismic Waves," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 97, No. SM1, Jan., pp. 27-46.

Analytical procedures for determining the response of soil deposits underlain by sloping rock boundaries to travelling wave base excitations are described and used to compare the seismic response for base motions moving with different velocities towards and away from the rock slope. It is shown that for waves travelling away from a rock slope, response amplitudes become higher as the velocity of wave propagation increases, so that maximum response is obtained for rigid base motions. The response values for base motions travelling towards a rock slope are shown to be always higher than the corresponding values when the base motions are travelling away from the slope. Based on the results of a number of cases

analyzed it is concluded that spacial variations in base motions may be neglected in ground response analyses when the base motions propagate in a direction normal to the rock slope with a velocity of 5,000 fps or higher, or whenever the velocity of shear wave propagation in the rock or base material exceeds about 3,500 fps.

DIBAJ, M., and PENZIEN, J. (1969a)

"Nonlinear Seismic Response of Earth Structures," Report No. EERC 69-2, Univ. of Calif., Earthquake Research Center, Berkeley, Calif., Jan.

The finite element method, a step-by-step integration method, and several other numerical techniques are employed to develop an incremental procedure for nonlinear earthquake analysis of earth structures. By this procedure the total nonlinear problem is reduced to a number of successive linear problems. Each time interval is selected sufficiently small so that the resulting incremental stresses and deformations may be related by a linear constitutive law containing parameters which are changed after each time interval.

A brief discussion of the fundamental concepts of plasticity theory and its general constitutive laws are given, and suitable yield criteria for soils are selected. An extension of the Drucker-Prager yield criterion to include work-hardening effects is developed, and the stress-strain relations for materials obeying this yield criterion are derived.

A finite element formulation of the nonlinear dynamic response of earth structures of a general type subjected to either uniform or nonuniform base motion is given. Results of several nonlinear analyses are presented and compared with those obtained from linear analyses. Relative differences between the two types of solutions, i.e., linear versus nonlinear, are illustrated and it is shown that realistic response predictions of deformation can be achieved only by nonlinear analysis procedures.

DIBAJ, M., and PENZIEN, J. (1969b)

"Response of Earth Dams to Traveling Seismic Waves," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM2, Mar., pp. 541-560.

The finite element technique is employed for the study of the response of an earth dam on an elastic foundation subject to a train of earthquake waves traveling transversely across the dam. Previous analyses have assumed the base of the dam to move as a rigid block. It is shown that this approximation is only valid for small or thin dams with a base width of less than 50 to 250 ft or less than $1/4$ of the shear wave length. The response of an ideal dam (300 ft high, 2-1/2:1 side slopes, constant modulus of elasticity throughout) is presented. Contours of all stress components are shown for several instants of time, as well as time histories of stress at several key locations. These results are compared to the results obtained by a previous analysis of a dam on a rigid base. It is shown that the traveling wave solution leads to a more critical situation than the rigid base solution. The magnitude of shear stresses is higher and the zones of high shear stress are located closer to the edge for the traveling wave solution than for the rigid base case.

DIMAGGIO, F.L., and SANDLER, I. (1970)

"Material Models for Soils," DASA-2521, Report to Defense Atomic Support Agency, Apr.

Various material models of soils for use in ground shock computations are studied. The theoretical basis for models currently being used is examined in order to determine their suitability and range of validity. A new model is developed which seems to be capable of predicting observed laboratory and field behavior better than any model advanced previously, while at the same time satisfying the most restrictive requirements of classical plasticity theory. This new model is shown to fit laboratory test data for McCormick Ranch Sand from all three standard tests.

DOBRY, R. (1970)

"Damping in Soils: Its Hysteretic Nature and the Linear Approximation," Progress Report No. 8, Effect of Local Soil Conditions Upon Earthquake Damage, Research Report R70-14, Soils Publication No. 253, Mass. Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., Jan.

A review is made of the nature and mathematical modeling of the attenuation of elastic waves traveling through soil. The different definitions of damping are discussed, by studying the Voigt Solid and the one-degree-of-freedom system.

After reviewing the experimental evidence, it is concluded that the physical mechanism of

damping in soils and rocks is basically one of hysteretic dissipation of energy. This is true both for shear and longitudinal waves, and it represents a nonlinear behavior.

The problem is usually linearized by assuming a viscous coefficient inversely proportional to frequency, and postulating the superposition principle in the frequency domain for an arbitrary waveform. This is equivalent to having the damping capacity ψ and the phase angle δ , constants and independent of frequency, and the attenuation α proportional to frequency. For this case, the easiest mathematical treatment is that of the complex modulus.

On the basis of this simplified model, useful relationships between damping, modulus and frequency are given for a hysteretic type linear material, for shear, longitudinal and rod waves. The case of Rayleigh waves is also reviewed. Unlike the viscoelastic material, this is not a dispersive one, with phase velocities being now independent of frequency.

The solution of the wave equation for the hysteretic-type solid is discussed, and some applications are given.

DOBRY, R. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

DOMASCHUK, L., and WADE, N.H. (1969)

"A Study of Bulk and Shear Moduli of a Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM2, Mar., pp. 561-581.

The stress-strain relationship of a micaceous sand is investigated by separating the volumetric and deviatoric components. Isotropic compression tests are used to investigate the bulk modulus of the sand prepared at different densities and subjected to different levels of stress. The shear modulus of the sand is investigated by constant mean normal stress triaxial compression tests. The tests are performed under varying conditions of relative density, mean normal stress and deviatoric stress. Graphical and analytical solutions for the bulk and shear moduli are developed from the experimental data.

DONOVAN, N. C. (1969)

"Research Brief No. 12," Proc., Specialty Session No. 2, Soil Dynamics, 7th Internat. Conf. on Soil Mechanics and Foundation Engineering, Mexico City, pp. 154-155.

Modulus and damping values reported for sand and saturated clay samples. Results obtained using triaxial compression and torsional vibration tests.

DOUGLAS, J.D. (1966)

"Undisturbed Sampling and In-Situ Testing," (Session Leader's Comments), Proc., Site Investigation Symposium, Sydney, Inst. of Civil Eng., Australia, 1966; Civil Eng. Trans., Vol. CE9:1, Apr., 1967, pp. 5-9.

The techniques and applications for undisturbed sampling by means of thin-walled tubes, open drive samplers and rotary core samplers are outlined. The equipment, typical applications, and interpretation techniques for Standard Penetration Tests, Cone Penetration Tests, Vane Shear Tests, and the Menard Pressiometer are reviewed, but no comparisons are made. Detailed cross sections of two 3 in. air injection thin wall samplers.

DRNEVICH, V.P. (1971)

"Undrained Cyclic Shear of Saturated Sand," Tech. Report UKY 37-71-CE7, Soil Mech. Series No. 7, Univ. of Ky., College of Eng., May.

Quantitative data are presented and analyzed giving the effective confining pressure, shear modulus, damping and axial strain as a function of cycles of loading. Normalized empirical expressions based on initial conditions are developed to describe the measured parameters. The data was obtained from hollow cylinder simple shear tests of saturated Ottawa sand. Both quasi-static (1/10 Hz.) and resonance tests (30 to 100 Hz.) were performed. Cyclic shear strain amplitudes to 0.15 percent were maintained constant for the duration of each test. Dynamic pore pressures measured with a transducer mounted at the base of the specimen were less than 5 percent of the confining pressure for the largest strain amplitudes in the test program. Dilation as measured by the axial vibration of the specimen was also very small.

The basic approach used for these data should be applicable to other gradations, other cohesionless soils and to cases where confining pressure is not isotropic, but, the particular values within the empirical equations may be different.

DRNEVICH, V.P., and HALL, J.R., JR. (1966)

"Transient Loading Tests on a Circular Footing," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 92, No. SM6, Nov., pp. 153-167.

Reference is made to a previously published equation which theoretically predicts the response of a circular footing on an elastic half space subject to an oscillatory load. This paper presents experimental data which tends to confirm the theoretical equation. The tests were performed using a 1 ft dia. rigid footing on sand, dynamically loaded by dropping a 5 lb bag of sand on its surface. Numerous transducers were used to measure the distribution of dynamic load across the footing as well as the accelerations. Typical data presented shows good agreement between the theoretical and observed values over the full time history of the loading pulse. The major problem is to define the shear modulus of the sand.

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.

This report is concerned with the effect of large amplitude steady-state vibrations on the shear modulus of sands. The torsional resonant column method was employed for this research using two specially constructed devices. One device torsionally oscillates a 4 cm diameter, 27 cm long solid cylindrical specimen while the second device tests a hollow cylinder of sand having a 4 cm inside diameter, a 6 cm outside diameter, and a 30 cm length. The purpose of the hollow cylinder was to obtain a more uniform shear strain throughout the sample.

The aim of the two testing programs was to determine the shear modulus of C-190 Ottawa sand as a function of void ratio and confining pressure before and after large amplitude torsional vibration had been applied. The effect of large amplitude vibration was to increase the low amplitude shear modulus by as much as 100 percent. The increase could not be accounted for on the basis of void ratio change. Vibration amplitude (shear strain amplitude in the hollow specimen) and cycles of torsional vibration are considered in presenting the data.

The increase in low amplitude shear modulus was accompanied by an increase in damping indicating a change in the mechanical characteristics of the sample caused by the different patterns of vibration history.

DRNEVICH, V.P., HALL, J.R., JR., and RICHART, F.E., JR. (1967)

"Effects of Amplitude of Vibration on the Shear Modulus of Sand," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., pp. 189-199.

A laboratory study conducted on hollow sand specimens subjected to torsional shear oscillations of constant angular rotation. C-190 Ottawa sand used with confining pressures up to 10^4 psf. Theoretical considerations for the test situation are given along with a description of the resonant-column test device. The specimens were pre-strained at constant confining pressure by vibration for a number of cycles at various amplitudes. Then the resonant frequency at low amplitudes was determined in order to calculate the shear modulus. The shear modulus increased with the number of high amplitude pre-straining cycles when tested at lower amplitudes. The shear modulus increased significantly when tested at low amplitudes at or slightly above the pre-strain confining pressures. The shear modulus decreased as the shearing strain was increased.

DRNEVICH, V.P., and RICHART, F.E., JR. (1970)

"Dynamic Prestraining of Dry Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM2, Mar., pp. 453-469.

Dynamic prestraining is the application of vibratory shearing strains to soil that is acted upon by static confining pressure. Tests on hollow, cylindrical Ottawa sand specimens, in a torsional resonant column apparatus, demonstrate that material behavior may be significantly changed by prestraining. In general, prestraining at shearing strains between 0.001% and 0.01% can double the dynamic shear modulus and damping, strikingly reduce the static compressibility, and cause fatigue failures. Density changes cannot account for this. No changes occur for prestraining at shearing strains less than 0.001%. The effects are dependent on confining pressure, void ratio, shearing strain amplitude, and cycles of vibration. Millions of cycles may be necessary to produce these changes. Test results indicate that density or void ratio measurements may not reveal the total change in these parameters imparted to dry sands by vibratory loading. This is important in determining the dynamic response of foundations and the compaction improvement of sand soils.

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.

The shear modulus or the shear wave velocity in soil enters into many calculations of seismic response and foundation vibration problems. Unfortunately this parameter is difficult to measure accurately. A brief description of the various important body and surface waves is presented along with a short description of a number of methods which can be used to measure or compute the shear wave velocity in soil. These include: laboratory techniques, computation from P-wave measurements, computation from Rayleigh or Love wave measurements, and direct shear wave measurements. The techniques for producing and measuring these waves are also briefly described. A bibliography of 57 references provides details for the procedures which are covered in a general way in the paper.

DUNCAN, J.M., and SEED, H.B. (1967)

"Corrections for Strength Test Data," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM5, Pt. 1, Sept., pp. 121-137.

Sources of error in measuring strengths of soil in both triaxial and plane strain tests are discussed in some detail. The magnitude of errors likely to be incurred with each source is given by means of formulas and empirical constants. The sources of errors for which corrections are recommended include those caused by piston friction, membrane stiffness, filter paper strength, and side plate friction for plane strain tests. The relative importance of each error may be small, but taken together, they may be a significant proportion of the total strength. The relative importance of these corrections increases with decreasing soil strength and confining pressure, and in the extreme may be as much as 20% of the true soil strength.

EHRLER, O.M.C. (1968)

"Nonlinear Parameters of Vibrating Foundations," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM6, Nov., pp. 1119-1214.

A review paper which discusses the relative benefits of treating a vibrating soil foundation as an elastic system or as an equivalent spring-dashpot system. The essential equations of each method are presented. The effect of different amplitudes of motion on the stiffness and damping properties of the soil are discussed. It is shown that for many purposes the equivalent spring-dashpot system offers many advantages. However, for a better understanding which will lead to a better treatment of the problem, it is necessary to consider the soil foundation as an elastic half-space, possibly with non-linear properties.

FERRAZ, A.D. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

FIGUEROA, J.A. (1960)

"Some Considerations About the Effect of Mexican Earthquakes," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 3, pp. 1553-1561.

Considers seismograms from four stations each of which was located in different soil and rock conditions. The period of the shear waves was measured. Effect of distance from epicenter and focal depth was also considered in wave frequency.

FINN, W.D.L., BRANSBY, P.L., and PICKERING, D.J. (1970)

"Effect of Strain History on Liquefaction of Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM6, Nov., pp. 1917-1934.

The resistance of a saturated sand to liquefaction under cyclic loading has been assumed to depend only on the void ratio, effective stress system and intensity of cyclic loading. The effect of strain history on the resistance to liquefaction is investigated here by means of triaxial and simple shear tests. The resistance of a saturated sand to liquefaction as measured in laboratory tests is very much influenced by the previous strain history. Partial liquefaction which occurs at small shear strains greatly increases the resistance to liquefaction in subsequent tests. Cycles of large shear strains, or quasi-static shear strains $> 7.5\%$, reduces the resistance to liquefaction. The increase in resistance to liquefaction created by a very small shear strain may result from the elimination of small local instabilities in the original sand structure. The loss of resistance caused by larger shear strains is thought to be due to either the creation of a uniform metastable structure or the development of a non-uniform structure.

FINN, W.D.L., and MITTAL, H.K. (1963)

"Shear Strength of Soils in a General Stress Field," Special Technical Pub. No. 361, Laboratory Shear Testing of Soils, ASTM, pp. 42-51.

The theory of elasticity is employed to show that the stress strain curve is steeper in plane strain than under triaxial stress conditions. By assuming failure to occur at the same strain in both cases, it follows that the strength is the greater in plain strain. Literature references substantiate this theory for sand. Test results for compacted clay agree with the theory.

FINN, W.D.L., PICKERING, D.J., and BRANSBY, P.L. (1969)

"Sand Liquefaction in Triaxial and Simple Shear Tests," Soil Mechanics Research Report Series No. 11, Univ. of British Columbia, Vancouver, Canada, Aug.

In this article cyclic loading test data from triaxial test and from a modified simple shear apparatus are reported. In the modified apparatus the authors demonstrate that ribbed plates rather than smooth plates on the top and bottom of the rectangular specimen improve the boundary conditions for simple shear deformations, resulting in higher cyclic shear stresses to create liquefaction at a given number of cycles. The authors in comparing the improved simple shear device with the cyclic triaxial test concluded that there is no significant difference between the peak cyclic shear stresses in both tests which are required to cause liquefaction of similarly confined samples in a given number of cycles. Included also are the main features of the simple shear apparatus and the test procedures.

FINN, W.D.L., WADE, N.H., and LEE, K.L. (1967)

"Volume Changes in Triaxial and Plane Strain Tests," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM6, Nov., pp. 297-308.

A theoretical study with experimental supplementary data. The volume changes during shear of a dense packing of an ideal assembly composed of uniform spheres was computed. It indicated that plane strain would exhibit less dilatancy than if the specimen were tested in a triaxial test. Experimental studies on dry dense sand using a plane strain direct shear device (Bishop's) and a triaxial compression apparatus confirmed the general nature of this theoretical hypothesis. From these studies it is suggested that since drained triaxial tests show more dilatancy than plane strain, triaxial undrained tests should show lower pore pressures than plane strain tests.

FLETCHER, G.F.A. (1965)

"Standard Penetration Test: Its Uses and Abuses," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 91, No. SM4, July, pp. 67-75.

In foundation test borings, the recovery of soil samples with a 2-in. OD split sample spoon driven with a 140-lb. weight falling 30 in. has been in use for more than 30 yr. Recording the number of blows required to drive the spoon 12 in. has been called the "Standard Penetration Test." It provides an approximation of soil densities in situ. It adds little to the cost of boring operations but adds considerably to the evaluation of the results when it is properly performed and its limitations are recognized. This paper traces the history of the test, the modifications that have been introduced, the variables inherent in the test, and sets forth the factors that affect the results. Applications in granular and cohesive soils are examined.

FLORIN, V.A., and IVANOV, P.L. (1961)

"Liquefaction of Saturated Sandy Soils," Proc., 5th Internat. Conf. on Soil Mech. and Found. Eng., Paris, Vol. 1, pp. 107-111.

A review of the distribution and effect of excess pore pressures in sand acted on by vibrations. Causes and soil characteristics for quicksand.

FUNSTON, N.E., and HALL, W.J. (1967)

"Footing Vibration with Nonlinear Subgrade Support," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM5, Pt. 1, Sept., pp. 191-211.

The paper commences with a general review of the state of the art with respect to foundation vibrations. The basic relations of amplitude vs. frequency for different forcing functions are presented for the damped linear elastic system for both the elastic half space and the single degree of freedom systems. An analytical solution is then presented for a non-linear case where the stiffness decreases with displacement according to a hyperbolic relationship. Finally, some results of large scale vibration tests conducted by the U.S. Army Corps of Engineers were compared with the theories. It was found that the nonlinear theory gave much better correlation to the measured data than did the linear elastic theories.

GEISLER, H. J. (1967)

"Liquefaction in a Saturated Sand by Torsional Vibration," M.S. thesis, Univ. of Ky.

Author presents results of torsional vibration tests on solid cylinders of saturated sand performed to study the phenomenon of liquefaction. Also pore pressure build-up in the undrained specimen was studied. Liquefaction did not take place below a certain amplitude of vibration. Most tests required a large number of cycles in order to create liquefaction.

GIBBS, H.J. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

GIBBS, H.J., and HOLTZ, W.G. (1957)

"Research on Determining the Density of Sand by Spoon Penetration Testing," Proc., 4th Internat. Conf. on Soil Mech. and Found. Eng., London, Vol. 1, pp. 35-39.

Extensive testing reveals relationship between penetration resistance of split spoon sampler and the density of sand deposits. Effects of overburden pressure, length and weight of drill rod, and $w\%$ evaluated and presented in graphical form.

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.

The motion of soil filling a model bedrock basin was observed during continuous vibration on a laboratory shaking table. The following factors were varied in different experiments: consistency of the soil; thickness of the soil fill; length-to-width ratio of the basin; and frequency and intensity of "bedrock" motion.

The shear-wave velocity was measured in place, and an empirical relationship was found between velocity and soil-water content. The response spectrum - a graph of magnification factor of "bedrock" displacement versus forcing frequency - was measured in and on the basin fill as the soil was forced into vibration by various frequencies and amplitudes of bedrock displacement.

Displacements and accelerations decreased for a given set of conditions with depth below the ground surface, and the amplitude of soil displacement increased as the fill deepened.

Surface motion increased as the soil consistency was softened by addition of water; this agreed with reports of many geologists who have examined damage patterns resulting from actual earthquakes. When the soil consistency became very soft, however, further addition of water caused the reverse effect, i.e., the softer the soil, the lower the magnification of bedrock motion.

Soil displacements increased at a declining rate as the amplitude of displacement was increased in the "bedrock."

GOODMAN, R.E., VAN, T.K., and HEUZE, F.E. (1968)

"The Measurement of Rock Deformability in Bore Holes," paper presented at the 10th Symposium on Rock Mechanics, Univ. of Texas, May; reprinted by Slope Indicator Co., Seattle, Wash.

Devices for measuring rock deformability in bore holes are of three types: dilatometers, which supply an all around radial pressure to the whole of the bore hole wall; bore hole jacks, which supply a unidirectional, self-equilibrating pair of forces to opposed sectors of the wall; and penetrometers, which force a rigid die into a small portion of the wall. These devices are reviewed, and an NX bore hole plate bearing device, a new bore hole jack developed by the authors, is described.

For interpretation of dilatometer data, a simple well-known relation give E. Young's modulus, in terms of the diametral formation corresponding to each increment of pressure. An expression giving E for bore hole jacks is derived in this paper, and the influence of rock and jack characteristics on data interpretation is analyzed.

Results of bore hole jack tests conducted in rocks at three sites showed good agreement with results of other static, in situ loading tests. The results of bore hole jack tests thus definitely reflect the properties of the rock mass rather than of the rock substance. This is because the volume of rock involved in the test, about one cubic foot, is sufficiently large to embrace significant rock defects that are not usually represented in laboratory testing of samples. The tests are easy to perform and the equipment reasonably light.

GREENLAND, S.W. (1964)

"Economic Loading Tests with the Menard Pressuremeter," Proc., Symposium on the Economic Use of Soil Testing in Site Investigation, Univ. of Birmingham, pp. 3:5-9, disc. pp. 3:27-51.

The pressuremeter test is advocated for measuring the bearing capacity and modulus of deformation of soils and soft rocks. A cylindrical metal probe covered with rubber membranes is inserted into the bore hole to the test level and is inflated by water under pressure from a surface control apparatus. A direct measurement is made of the ultimate bearing capacity, and an elastic modulus is derived from the pressure-deformation curve. Four case histories illustrate the use of the pressuremeter test and present comparisons with other test results. The cost of pressuremeter testing generally is slightly less than an equivalent amount of bore hole sampling/laboratory testing and is of the order of 50-60% of the cost of plate loading tests of small loadings at shallow depths in simple conditions.

GUPTA, R.N. (1966a)

"Reflection of Elastic Waves from a Linear Transition Layer," Bull., Seismol. Soc. of Am., Vol. 56, No. 2, Apr., pp. 511-526.

A separation of P- and S-wave potentials is achieved for an inhomogeneous medium in which density is constant and Lamé's parameters, λ and μ , are assumed to vary as $\lambda/\lambda_1 = \mu/\mu_1 = (1 + bz)^2$ where λ_1 , μ_1 and b are constants. The resulting equations are solved for an arbitrary angle of incidence. Plane wave reflection coefficients are obtained for the situation when the material mentioned above forms a transition layer between two homogeneous, elastic half-spaces. First and/or second-order discontinuities in material properties are permitted at the boundaries of the transition layer. Some numerical results are given.

GUPTA, R.N. (1966b)

"Reflection of Plane Elastic Waves from Transition Layers with Arbitrary Variation of Velocity and Density," Bull., Seismol. Soc. of Am., Vol. 56, No. 3, June, pp. 633-642.

The problem of reflection of plane elastic waves is generalized numerically to an arbitrary variation, with depth, of the elastic parameters inside a transition layer between two homogeneous half-spaces. Numerical results are given for some cases of interest.

GUTENBERG, B. (1957)

"Effect of Ground on Earthquake Motion," Bull., Seismol. Soc. of Am., Vol. 47, No. 3, July, pp. 221-250.

Earthquake damage at a given location depends on properties of the arriving elastic waves, of the ground at the shaken site, and of the structures involved. To investigate effects of the ground on the motion at the surface, five identical seismographs have been operated temporarily at 25 locations within 30 miles from the Seismological Laboratory of the California Institute of Technology, Pasadena, and their records have been compared with those written by an identical instrument recording in routine at the Seismological Laboratory. The ratio of amplitudes at sites on fairly dry alluvium more than 500 feet deep to those at the Seismological Laboratory (on crystalline rock) is frequently 5:1 or more for earthquake waves having periods of 1 to 1-1/2 sec. For waves having periods of 0.1± sec. and waves having periods exceeding 10 sec. (length more than 10 miles) the corresponding differences in amplitudes are small. Amplitudes of earthquake waves and of the continuous unrest of the ground recorded at sites on water-saturated soft ground may be ten times those recorded at the Laboratory. The period of waves for which the response relative to that at the Seismological Laboratory is greatest usually decreases as the thickness of the alluvium decreases and is about 1/4 sec. at stations on alluvium 100± feet thick. At stations on crystalline rock the motion of the ground does not differ significantly from that at the Laboratory for waves having periods exceeding 1/4 sec.

At sites on alluvium, relatively strong shaking lasts several times as long as at those on crystalline rock; usually this ratio decreases with decreasing thickness of the alluvium. Ground effects may produce appreciable differences in duration and amount of shaking even at localities only a fraction of a mile apart. The importance of selecting crystalline rock or at least dry ground and avoiding water-saturated soft ground for foundations of buildings to reduce potential earthquake damage is stressed.

GUYOD, H. (1964)

"Use of Geophysical Logs in Soil Engineering," Special Technical Pub. 351, Symposium on Soil Exploration, ASTM, pp. 74-85.

Reviews the determination of physical properties of soils and rocks by continuous measurements in bore holes of electrical resistivity; natural gamma rays; density determination by the gamma-gamma method; hydrogen content by the neutron method; acoustic velocity and amplitude. Typical charts are presented for each method. The procedures are recommended as supplements to sampled test holes.

HAGMANN, A.J., and WHITMAN, R.V. (1969)

"Comparison of Methods for Analyzing Soil Deposits During Earthquakes," Research Report R69-29, Soils Pub. No. 238, Mass. Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., June.

Provides an analysis of six different uniform soil deposits by two different methods. One method utilizes a discrete mass model system while the other provides an analytical solution to the differential equations of motion. A procedure is developed for including the effect of rock damping in the discrete model solution. A comparison of both solution methods is provided.

HALL, J.R., JR. (1962)

"Effect of Amplitude on Damping and Wave Propagation in Granular Materials," Ph.D. thesis, Univ. of Fla.

Details of equipment and experiments to study the velocity of shear and longitudinal stress waves and their damping in a column of granular material. Ottawa sand, two glass bead sizes, and silt-size crushed quartz were tested. Shows the changes of shear and compressive wave velocities with amplitude of vibration and confining pressure and the variation of logarithmic decrement with amplitude.

HALL, J.R., JR. and RICHART, F.E., JR. (1963a)

"Dissipation of Elastic Wave Energy in Granular Soils," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 89, No. SM6, Nov., pp. 27-56.

Numerical values of internal damping in granular soils determined by lab tests using vibration decay method. Details of test method and damping theories. Charts of variation of logarithmic decrement (δ) with amplitude show marked effect of variations in degree of saturation and grain characteristics. $0.02 < \delta < 0.20$ in these tests. Tests on fine crushed quartz showed dependence on time of loading and stress history, and that "prestress" was removable by higher vibration amplitude.

HALL, J.R., JR., and RICHART, F.E., JR. (1963b)

"Effect of Vibration Amplitude on Wave Velocities in Granular Materials," Proc., 2nd Pan-American Conf. on Soil Mech. and Found. Eng., Vol. 1, pp. 145-162.

Laboratory tests on Ottawa sand, two sizes of glass beads, and a crushed quartz (silt-sized) indicated that longitudinal and shear wave velocities decrease with an increase in the amplitude of vibration. The finer material, the silt-sized quartz, indicated dependency on time loading and stress history. Although the wave velocity increased with the duration of the confining pressure, higher amplitudes tended to destroy this time-dependent velocity increase.

HANSEN, R. J., ed. (1970)

Seismic Design for Nuclear Power Plants, The M.I.T. Press, Cambridge, Mass.

A book of collected articles relative to the seismic design of nuclear power plants. Included are geological and seismological considerations with suggested design seismic inputs, soil amplification analyses and considerations with suggested methods for determining the dynamic soil properties, soil structure interaction and design considerations and structural response of the plant buildings.

HARDIN, B.O. (1961)

"Study of Elastic Wave Propagation and Damping in Granular Materials," Ph.D. thesis, Univ. of Fla.

Reviews theories for the propagation of elastic waves in porous and granular materials and compares them numerically for quartz sand. Tests on Ottawa sand, quartz sand and quartz silt conducted to study steady state shear waves, compression waves, free torsional vibration and damping. Relates the wave velocity and changes in confining pressure, void ratio, $w\%$, amplitude, frequency and grain shape.

HARDIN, B.O. (1965)

"The Nature of Damping in Sands," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 91, No. SM1, Pt. 1, Jan., pp. 63-97.

Theoretical studies showing that the Kelvin-Voigt model (viscous damping) can be used to describe the behavior of dry sand subjected to small amplitude sinusoidal vibrations, and thereby facilitate extrapolation of dynamic test results to other boundary conditions. Results of steady state and free vibration and static torsion tests on Ottawa sand are presented which indicate the manner in which viscosity should be assumed to decrease with frequency if viscous damping theory is applied. Theoretical solutions and test results are presented in detail.

HARDIN, B.O., and BLACK, W.L. (1966)

"Sand Stiffness under Various Triaxial Stresses," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 92, No. SM2, Mar., pp. 27-42.

Experimental data is presented to show the effect of various static states of stress that can be obtained in a triaxial cell on the dynamic shear modulus for small amplitude vibration of dry sand. The results permit an evaluation of the stiffness of dry sand, which governs vibration response in cases such as the vibration of machine foundations. A comparison with dynamic tests indicated that the stiffness of sand (as defined by the shear modulus) is independent of the deviatoric component of the initial static state of stress and of the rate of loading, and that the effect of stress history is small. Empirical equations permit the computation of the dynamic shear modulus as a function of the void ratio and the isotropic confining pressure.

HARDIN, B.O., and BLACK, W.L. (1968)

"Vibration Modulus of Normally Consolidated Clay," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM2, Mar., pp. 353-369.

Triaxial samples of remolded Kaolinite clay were anisotropically consolidated to effective confining pressures up to a maximum of 100 psi, and then subjected to small shear vibrations of less than 0.01% amplitude in order to obtain the shear modulus. The data indicated that: (1) The shear modulus was independent of the deviatoric component of consolidation. (2) The modulus increased with effective mean normal confining pressure to the $1/2$ power. (3) The modulus increased with decreasing void ratio in much the same manner as had been previously determined for sand. (4) Secondary compression caused an increase in modulus of about 10% for each 10 fold increase in consolidation time. However, this secondary compression effect was reduced or destroyed by significant changes in effective stress. Informative research data are presented which may be a useful guide to practical studies with real soils.

HARDIN, B.O., and DRNEVICH, V.P. (1970a)

"Shear Modulus and Damping in Soils; 1, Measurement and Parameter Effects," Tech. Report UKY 26-70-CE2, Soil Mech. Series No. 1, Univ. of Ky., College of Eng., July.

Based on numerous tests on a spectrum of disturbed and undisturbed soils, the shear modulus decreases and the damping ratio increases, very rapidly, with increasing strain amplitude. The rate of increase or decrease depends on many parameters, the most important of which are: effective mean principal stress, degree of saturation, void ratio, and number of cycles of loading. Ambient states of octahedral shear stress, overconsolidation ratio, effective strength envelope, frequency of loading, and time effects have a less important influence on these properties. Cohesive soils are affected differently than clean sands. The shear modulus and damping in soils are important to the analysis of all soil vibration problems. Apparatus used to measure these properties must be capable of making accurate measurements at very small shearing strains, the range being defined by practical problems in earthquake and foundation vibrations. A pseudostatic simple shear apparatus and two different resonant column apparatus were used.

HARDIN, B.O., and DRNEVICH, V.P. (1970b)

"Shear Modulus and Damping in Soils; 2, Design Equations and Curves," Tech. Report 27-70-CE 3, Soil Mech. Series No. 2, Univ. of Ky., College of Eng., July.

Equations and graphs for the determination of shear modulus and damping of soils, for use in design problems involving repeated loading or vibration of soils, are presented. These equations and graphs are based on numerous laboratory tests on both remolded and undisturbed cohesive soils and on clean sands. Comparison of the measured and computed values shows good agreement. An example problem showing how these equations and curves are used is given.

HARDIN, B.O., and MOSSBARGER, W.A., JR. (1966)

"The Resonant Column Technique for Vibration Testing of Soils and Asphalts," Conf. Proc., Instrument Soc. of Am., Vol. 21, Pt. 2, No. 16.8-3-66, pp. 1-19.

The resonant column technique has proved useful for determining pertinent soil properties for the analysis of problems involving soil vibration, such as the vibration of machine foundations. Also this technique apparently can be usefully applied in the rational analysis of flexible highway pavements. Various apparatus have been designed and used by different investigators for the resonant column testing of soils. The authors have begun a study of asphalts and asphalt mixes and have initial test results for sand-asphalt specimens. Factors such as; the stiffness and coherence of the material to be tested, which parameters such as stress or temperature are important to material response and are to be controlled during testing, and interpretation of test results, must be considered for proper design and use of resonant column apparatus. The various apparatus presently available satisfy these requirements to varying degrees.

HARDIN, B.O., and MUSIC, J. (1965)

"Apparatus for Vibration of Soil Specimens During the Triaxial Test," Special Technical Pub. No. 392, Instruments and Apparatus for Soil and Rock Mechanics, ASTM, pp. 55-74.

Details of a triaxial test apparatus that applies torsional vibration and static axial loading. The apparatus was designed to study the dynamic shear modulus or shear wave velocity and the damping characteristics of soils. The theory and analytical solution for steady-state vibration of the specimen-apparatus system is presented. The solution is programmed for an electronic computer. The effects of apparatus constants on the shear modulus of soils are calibrated. Finally, agreement between theory and experimental results is demonstrated by tests on dry sand.

HARDIN, B.O., and RICHART, F.E., JR. (1963)

"Elastic Wave Velocities in Granular Soils," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 89, No. SM1, Pt. 1, Feb., pp. 33-65.

Resonant column laboratory tests on sands and silt show the effect of various confining pressures, $w\%$, void ratios and grain sizes on the propagation of longitudinal and shear waves. Wave velocities of sand varied approximately with $1/4$ power of confining pressure and decreased linearly with void ratio independent of grain sizes and relative density. Detailed correlation chart. Test equipment diagrams.

HARDIN, B.O., and SCOTT, G.D. (1966)

"Generalized Kelvin-Voigt Used in Soil Dynamics Study," Jour. of the Eng. Mech. Div., ASCE, Vol. 92, No. EM1, Feb., pp. 143-156.

Interpretation of specimen-apparatus systems used for determination of dynamic properties of cylindrical specimens may be made by analyzing a simple model. For materials represented by the generalized Kelvin-Voigt model the general steady state solution for the forced vibration of a cylindrical shaft is presented. Its use is demonstrated by application to boundary value problems of typical dynamic test systems, and computed response values are compared to measurements on sand cylinders.

HASKELL, N.A. (1953)

"The Dispersion of Surface Waves on Multilayered Media," Bull., Seismol. Soc. of Am., Vol. 43, No. 1, Jan., pp. 17-34.

A matrix formalism developed by W. T. Thomson is used to obtain the phase velocity dispersion equations for elastic surface waves of Rayleigh and Love type on multilayered

solid media. The method is used to compute phase and group velocities of Rayleigh waves for two assumed three-layer models and one two-layer model of the earth's crust in the continents. The computed group velocity curves are compared with published values of the group velocities at various frequencies of Rayleigh waves over continental paths. The scatter of the observed values is larger than the difference between the three computed curves. It is believed that not all of this scatter is due to observational errors, but probably represents a real horizontal heterogeneity of the continental crusts.

HASKELL, N.A. (1960)

"Crustal Reflection of Plane SH Waves," Jour. of Geophysical Res., Vol. 65, No. 12, Dec., pp. 4147-4150.

An equation has been derived for the amplitude of the free surface displacement due to plane SH waves incident at any angle at the base of a layered crust. Numerical computations have been carried through for the case of a single-layered model of the continental crust. At any given angle of incidence, the surface amplitude goes through a series of minima and maxima at periods which, in the single layered case, are harmonically related. At nearly grazing angles of incidence the surface amplitude is relatively small except at periods in the neighborhood of the cutoff periods of the second- and higher-order Love-wave modes.

The results throw some doubt on the 'whispering gallery' effect as an explanation of the mode of propagation of long-period (20 to 30 sec.) S-waves at S_n velocity.

Applied to the case of an alluvial layer over a hard-rock basement, the theory appears to give an adequate explanation of the abnormally large amplitudes that occur on unconsolidated formations in the epicentral region of earthquakes.

HASKELL, N.A. (1962)

"Crustal Reflection of Plane P and SV-Waves," Jour. of Geophysical Res., Vol. 67, No. 12, Nov., pp. 4751-4767.

Expressions in terms of layer matrices have been derived for the motion of the free surface when plane harmonic P or SV waves are incident at any given angle at the base of a horizontally layered crust. A computer program has been written to calculate normalized surface displacements as functions of horizontal phase velocity (or angle of incidence) and wave period. Numerical values have been obtained for the same single-layered model of the continental crust for which the corresponding calculations for SH waves have been presented previously. The surface motion pattern is similar to that of the SH case, but it is somewhat more complex owing to PS and SP conversions at the layer boundaries. Variations in amplitude and phase of the surface displacements are particularly large and erratic in the case of SV waves that are incident at angles such that $\alpha_1 < c < \alpha_n$ where c is horizontal phase velocity and α_1 and α_n are the P wave velocities in the crustal layer and mantle, respectively. It is recommended that data from this region be avoided in studies of focal mechanisms that are based on the attitude of the plane of polarization of the S waves. For angles of incidence such that $c > \alpha_n$, the crustal transfer functions for incident SH and SV waves are sufficiently similar for conventional methods to be used in determining the angle of polarization with moderate accuracy.

HERRERA, I., and ROSENBLUETH, E. (1965)

"Response Spectra on Stratified Soil," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 1, pp. I: 44-60.

This paper concerns the probability distribution of spectral responses of viscously damped single-degree systems resting on stratified viscoelastic soil. The soil is assumed to rest on a viscoelastic homogeneous half space of rock. Motion arriving at the rock-soil interface is idealized as a stationary Gaussian process. The transfer function for the soil formation is treated independently for each vibration frequency of interest, in order to allow for dependence of viscoelastic parameters on the wave frequency; this is accomplished through use of a matrix formulation. Certain additional approximate results are included.

HERSHBERGER, J. (1956)

"A Comparison of Earthquake Accelerations with Intensity Ratings," Bull., Seismol. Soc. of Am., Vol. 46, No. 4, Oct., pp. 317-330.

The maximum accelerations recorded on 108 strong-motion records obtained in 60 earthquakes were compared with the intensity ratings assigned to the places where the records had been obtained. The results seem to show that acceleration alone cannot be used as a measure of intensity.

HEUKELOM, W., and FOSTER, C.R. (1960)

"Dynamic Testing of Pavements," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 86, No. SM1, Feb., pp. 1-28.

Equipment and techniques for nondestructive dynamic testing of pavements as developed in the Netherlands. The observed stress-strain relationships correlate with theoretical predictions.

HODGSON, J.H. (1964)

Earthquakes and Earth Structure, Prentice-Hall, Inc., Englewood Cliffs, N.J.

Describes the great earthquakes of the past and smaller ones of seismological importance, analyzes the information they provide us about the interior of the earth, reveals the role seismology plays in nuclear-test detection, and presents the latest developments in earthquake-resistant construction.

HOLTZ, W.G., and GIBBS, H.G. (1956)

"Triaxial Shear Tests on Pervious Gravelly Soils," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 82, No. SM1, Jan., pp. 867:1-22.

Summary of 183 triaxial tests on specimens ranging from 1-3/8" to 9" dia. with gravel sizes ranging from 3/16" to 3" dia. Results and recommendations on the relations between shear resistance and density, gravel content, gradation, maximum particle size, and particle shape.

HOLTZ, W. G., and GIBBS, H. J. (1969)

Discussion of "Settlement of Spread Footings in Sand," (by D. J. D'Appolonia, E. D'Appolonia and R. F. Brissette, Jour. of the Soil Mech. and Found. Div., ASCE, May, 1968), Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, SM3, May, pp. 900-905.

The authors discuss and compare their correlation of Standard Penetration Resistance vs. relative density to that correlation recently presented by Bazaraa.

HOUSNER, G.W. (1959)

"Behavior of Structures During Earthquakes," Jour. of the Eng. Mech. Div., ASCE, Vol. 85, No. EM4, Oct.

Behavior of structures during earthquakes is described by response spectrum curves which show how the response is affected by the magnitude and distance of the earthquake and by the natural period and damping of the structure. Behavior of structure depends strongly upon its ability to dissipate vibrational energy when overstressed. Desirability of limit design is indicated.

HOUSNER, G.W. (1965)

"Intensity of Earthquake Ground Shaking Near the Causative Fault," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 1, pp. III:94-115.

Data and analyses indicate an upper bound for intensity of ground shaking that is approximately 50 percent greater than at El Centro during the earthquake of 18 May 1940. Upper bounds of 50 percent g for maximum ground acceleration and 45 seconds for duration of strong shaking are indicated. The intensity immediately adjacent to a fault is not especially severe but is, in general, somewhat less than at a distance of several miles. The maximum intensity of shaking associated with a Magnitude 8.5 earthquake is less and that associated with a Magnitude 5 earthquake is greater than has been commonly supposed.

HUDSON, D.E. (1965)

"Equivalent Viscous Friction for Hysteretic Systems with Earthquake-Like Excitations," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 2, pp. II:185-201.

By equating the energy loss per cycle in a hysteretic system to the energy loss per cycle in the corresponding small amplitude linear system, an equivalent viscous damping coefficient can be defined which will permit accurate calculations of the maximum amplitude of resonant vibrations in hysteretic systems that are strongly nonlinear. The accuracy of these calculations is illustrated by comparison with digital solutions for bilinear and curved hysteresis loops. It is shown that the maximum possible value of the equivalent viscous friction for bilinear hysteresis curves of positive slope is 15.9%, and that most hysteretic systems have damping values less than this. The concept of the equivalent viscous friction is extended to earthquake-like excitations, for which the force-deflection responses are approximately sinusoidal but of varying amplitudes. It is shown by comparison with accurate digital calculations that the equivalent viscous friction for hysteretic systems under such excitations may be only of the order of 4%, even for large deformations.

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.

The San Francisco earthquake of March 22, 1957, was recorded simultaneously by accelerometers at five United States Coast and Geodetic Survey stations in the San Francisco area. Response spectrum curves were computed from the acceleration-time records, and from these response spectrum curves the spectrum intensities have been determined. From these spectrum intensities certain conclusions are drawn as to: (1) the effects of local geology on the recorded ground motions; (2) the calculation of total energy released by the earthquake from strong-motion accelerometer records; (3) possible influence of structural dynamic behavior on the accelerations recorded in building basements, and the relationship between basement accelerations and ground accelerations; and (4) the applicability of a simplified type of strong-motion earthquake instrument for investigations of local distribution effects. A general comparison is made between the present earthquake and typical Pacific Coast earthquakes.

HVORSLEV, M.J. (1949)

"Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes," Report on a research project of the Committee on Sampling and Testing, Soil Mech. and Found. Div., ASCE, ed. and printed by Waterways Experiment Station, Vicksburg, Miss., Nov.

In scope this report is an authoritative reference work on subsurface exploration and sampling of soil and rock. Its value to the practicing engineer is that it includes under one cover:

- (1) Information and general data on all methods of subsurface exploration and sampling, with a complete bibliography for detailed information on specific topics.
- (2) A clear and logical delineation of factors influencing the quality of samples, and rules covering the design and operation of equipment for obtaining undisturbed samples.

HVORSLEV, M.J. (1969)

"Needed Research on Fundamental Problems in Soil Sampling," Specialty Session No. 1, Soil Sampling, 7th Internat. Conf. on Soil Mechanics and Found. Eng., Mexico City.

Contains brief comments on design parameters and some fundamental problems in soil sampling in need of additional research.

IDRISS, I.M. (1966)

"Response of Earth Banks During Earthquakes," Ph.D. thesis, Univ. of Calif., Berkeley.

A finite element method for evaluating the response during earthquakes of semi-infinite soil layers (i.e., layers of constant depth and infinite horizontal extent) and earth banks. Closed-form solutions and a lumped-mass solution are employed, and the accuracy of these solutions is investigated. Earth banks should be idealized as a series of interconnected triangular finite elements having a horizontal side to vertical side ratio of ≤ 5 , and the bank should be divided into at least 4 sublayers. The slope influences the response values over a distance equal to 6 times the height of the bank behind the crest, and 6 times the depth of foundation material beyond the toe of the slope. Several examples illustrate the use of the solutions.

IDRISS, I.M. (1968)

"Finite Element Analysis For the Seismic Response of Earth Banks," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM3, May, pp. 142-156.

Reference is made to a previous paper by the same author describing the use of the finite element method for analyzing the response of earth banks to earthquakes. This paper elaborates on the finite element method and the various criteria which must be followed in order to obtain accurate results. It is shown that there must be at least 4 layers of elements. In areas where accurate data is desired the ratio of length to width of the elements should not exceed 5. The lengths of the level portions away from the slope should be 10 to 12 times the height or the depth ($L_v=10D$ or $L_v=10H$). If these criteria are met, reasonably accurate results will be obtained.

IDRISS, I.M., and SEED, H.B. (1967)

"Response of Earth Banks During Earthquakes," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM3, May, pp. 61-82.

The finite element method of analysis was used to find the theoretical dynamic response of a homogeneous bank of soil (assumed to obey Hooke's Law) to a given horizontal and vertical earthquake vibration. The response varied widely depending on the nature of the soil and the dimensions of the bank. There were so many significant variables that at this date it seems desirable to make a detailed analysis of each case. However, some general conclusions were possible. (1) The vertical component of ground motion affected only the vertical response significantly. (2) The maximum horizontal motions were developed at the surface. They were smaller at the toe of the slope than away from the toe. Several examples are presented showing how the period of the developed vibrations and the peak accelerations and seismic coefficients vary from place to place in the bank for various soils and bank geometry.

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.

The seismic responses of soil deposits are calculated at four sites where recordings were obtained during the 1957 San Francisco earthquake. The analyses are made utilizing an equivalent linear lumped mass method of analysis incorporating the soil characteristics at the four sites and estimates of the rock motions at each site based on the record obtained on rock during this earthquake. Comparisons of the maximum amplitudes of ground motions, the time history of ground accelerations, velocity and acceleration spectrums and spectral intensities for both computed and recorded ground accelerations indicate a reasonably high degree of agreement. The influence of the amplitude of base rock accelerations on the response of a soil deposit is also discussed and illustrated.

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.

Three analytical methods are presented for computing the response of horizontally bedded soil deposits to known horizontal earthquake vibrations at the soil-rock contact. The results of the computations give the surface acceleration-time history from which response spectra and other surface response data can be readily computed. The results also give the computed stress history at all points within the soil deposit. The three methods are: an approximate solution using discrete lumped masses with soil having linear elastic properties; a bilinear solution; and an equivalent linear solution for materials with bilinear properties. The accuracy of each method is evaluated by comparison with the rigorous closed form solution for a simple linear elastic case. The computer programs for these solutions were not given, but are available in reports listed in the references.

IDRISS, I.M., and SEED, H.B. (1970)

"Seismic Response of Soil Deposits," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM2, Mar., pp. 631-638.

An analytical procedure for the evaluation of the seismic response of horizontal soil layers incorporating equivalent linear moduli and damping ratios varying with depth is presented. The use of this procedure is illustrated and its results compared to recorded values and to computed values using a constant damping solution. These comparisons indicate that the variable damping solution often provides response values that are in better agreement with recorded values than those obtained from a constant damping solution. The difference does not appear to be very great when the deposit consists of soils that are essentially of similar characteristics. Much greater differences are obtained when the deposit consists of sublayers having greatly varying characteristics.

IMAMURA, A. (1929)

"On the Earth-Vibrations Induced in Some Localities at the Arrival of Seismic Waves," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 7, pp. 489-494.

Seismograms may involve, besides seismic waves propagated from the focus to recording stations, earth-vibrations induced locally by the direct seismic waves. Differences between earth movements measured at the base and surface of a thin soil layer are examined. Attempts are made to relate short-period contributions to elastic wave reflections in the soil layer. For the deep focus earthquake considered, the P-phase is much stronger than the S-phase.

IRELAND, H.O., MORETTO, O., and VARGAS, M. (1970)

"The Dynamic Penetration Test: A Standard That Is Not Standardized," Geotechnique, Vol. 20, No. 2, June, pp. 185-192.

Presents a description and a discussion of the technique of making the so-called standard penetration test as used in North and South America. Dynamic penetration tests such as the SPT are used widely in an effort to get results that can be interpreted by engineers on a universal basis. However, it has become evident that the results of the tests are often a compromise because of details in the test procedure which are generally not appreciated. It is common that those engaged in doing these tests change the procedure of the test or the dimensions of the tools and continue to call it standard, apparently unconscious that changes result in variations in the penetration resistance number N. To describe the test and consider the various implications of the method properly the following items are considered independently: a procedure for boring, sounding and sampling which seems to be indicated for the test is described; the SPT in its original form is described in detail; as examples of variations, two modified and improved samplers and test procedures used in South America are mentioned; finally the applicability and the scope of the test are discussed.

ISENBERG, J. (1970)

"Interaction Between Soil and Nuclear Reactor Foundations During Earthquakes," Report No. 6915-1200, Agabian-Jacobsen Associates, June.

Treats the dynamic response of a nuclear reactor foundation subjected to earthquake motion. The structure is embedded below the ground surface. The structure and soil are represented by an assembly of two-dimensional finite elements. Both elastic and inelastic soil properties are considered. The computer program used to perform the analyses and the version of plasticity theory used to represent the inelastic soil properties are described.

ISHIHARA, K. (1967)

"Vibration Behavior of a Sand Layer Resting Upon Inclined Firm Underground Boundaries," Soil and Foundation (Japan), Vol. 7, No. 1, Jan., pp. 15-30.

Sand was placed in a box 5 x 1 x 1/2 m. One end and the base of the box were fixed. Vibrations were applied at the other free end, and vibration accelerations were measured within the sand at many positions along the box. The box was intended to simulate a natural rock layer, and the fixed end could be inclined to simulate a sloping bedrock layer. Interpretation of the experimental data was difficult because of the discontinuity effects of the vibrating boundary. However, based on certain simplifying assumptions, the data was interpreted to show an increase in the measured response approaching the fixed boundary. The slope of the fixed boundary also appeared to have some effect. The data was briefly compared to observed data from real earthquakes. Of research interest.

JACOBSEN, L.S. (1930)

"Steady Forced Vibration as Influenced by Damping," Transactions, ASME, Vol. 52, Pt. 1, pp. 169-181.

Method of obtaining approximate solutions of steady forced vibration in a damped system, of one degree of freedom, for case of sinusoidally varying disturbing forces; approximation consists in expressing all damping terms of original differential equation by single equivalent damping term, proportional to first power of velocity of motion; disc. of experimental results at Stanford Univ.; comparison of Den Hartog's exact theory and approximate theory.

JACOBSEN, L.S. (1960)

"Damping in Composite Structures," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 2, pp. 1029-1044.

Damping in composite structures occurs mainly at their various joints. A discussion of the adequacy of the equivalent viscous friction concept is followed by an attempt to discern simple geometrical design principles for an idealized relationship between load and deflection in a hysteretic but non-deteriorating joint. A comparison of performances of idealized joints with those of actual joints is made.

JENNINGS, P.C., HOUSNER, G.W., and TSAI, N.C. (1969)

"Simulated Earthquake Motions for Design Purposes," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 1, No. A1, pp. 145-160.

Simulated earthquake motions were developed for the epicentral zone of four different sizes of earthquakes ranging from magnitude 8.0 to 8.5 down to $M = 4.5$ to 5.5. Two different motions for each earthquake were worked out to simulate two horizontal components. An accelerogram, ground velocity and ground displacement and velocity spectra diagrams are presented for each of the eight motions. The maximum intensities of motion for these earthquakes were obtained by interpolating from existing recorded strong motion data and are intended to represent average ground conditions. The wave distributions were obtained from random motion, with a rapid increase to maximum acceleration and a long tailing-off effect. Copies of listings of the acceleration computer card decks are available from the authors at the California Institute of Technology, Pasadena, California, U.S.A.

JENSEN, J.W. (1958)

"Damping Capacity--Its Measurement and Significance," Report of Investigation No. 5441, U.S. Bureau of Mines, Rolla, Mo.

This report was prepared as a background reference for future publications concerning the damping capacity of metals and alloys. It contains a broad description of the theory and methods of measuring damping capacity and correlates the developments of the past decade in terminology, units, and techniques. Damping-capacity instrumentation at the Mississippi Valley Experiment Station, Rolla, Mo., is described. The bibliography contains over 150 references to recent articles, each tabulated as to material investigated and test variables, such as frequency, stress, mode of vibration, and temperature.

JONES, R. (1958)

"In-Situ Measurement of the Dynamic Properties of Soil by Vibration Methods," Geotechnique, Vol. 8, No. 1, Mar., pp. 1-21.

The dynamic shear modulus and its variation with depth can be determined from a) the phase velocity of induced surface vibrations, and b) the resonant frequency of a mass vibrating normal to the surface.

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.

The simplest case in which a distortional wave of a purely plane type propagated vertically upwards in an elastic semi-infinite medium is partly transmitted and partly reflected at the bottom boundary of a superficial visco-elastic layer as well as reflected at the surface boundary of the same layer is examined.

KANAI, K. (1961)

"An Empirical Formula for the Spectrum of Strong Earthquake Motions," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 39.

Develop an improved empirical formula for predicting response spectrum for strong earthquake motion. Application is made of the procedure to twenty different earthquakes.

KANAI, K., and TANAKA, T. (1961)

"On Microtremors, VIII," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 39, Pt. 1, pp. 97-114.

The results of systematic measurements of microtremors at thousands of places in Japan show that these disturbances, chiefly of artificial origin, have a period distribution curve at any given place that reflects the local nature of the ground and are similar to earthquake motions at that place. Therefore, microtremor measurement is useful not only for anticipating the features of destructive earthquake motions but also for determining the seismic force coefficient to be considered in earthquake-proof construction design.

KANAI, K., TANAKA, T., and SUZUKI, T. (1968)

"A Short Note on Abnormal Earthquake Damage and Microtremors," (in Japanese with English abs.), Bull., Tokyo Univ. Earthquake Research Inst., Vol. 46, Pt. 3, May, pp. 783-790.

The relation is considered between the abnormally severe damage to Japanese-style wooden houses in an area about 400 km from the epicenter (Taisha Town) of the Nankai earthquake, $M=8.3$, on December 21, 1944 and the period distribution curves of microtremors observed in the same area. The shapes of period distribution curves of microtremors in the severely damaged area are extremely flat. Comparisons are made with data from previous earthquakes in the same area.

KANAI, K., and YOSHIZAWA, S. (1956)

"Relations Between the Amplitude of Earthquake Motions and the Nature of Surface Layers, IV," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 34, pp. 167-183.

The paper is a continuation of research work on the problem of earthquake movement of the superficial stratum and deals with the cases where the primary seismic waves are incident upwards normally to doubly stratified layers residing on the surface of a semi-infinite body.

Two cases are considered:

The case where an infinite train of harmonic plane waves is transmitted to the surface layer consisting of two layers, taking the solid viscosity into consideration.

The case where a finite train of harmonic plane waves is transmitted toward the surface layer consisting of two layers.

KEIGHTLEY, W.O. (1966)

"Vibrational Characteristics of an Earth Dam," Bull., Seismol. Soc. of Am., Vol. 56, No. 6, Dec., pp. 1207-1226.

An earth dam was excited into vibrations, in the upstream-downstream direction, by four rotating eccentric-mass vibration generators which were operated on the crest. Natural frequencies, mode shapes, and equivalent viscous modal damping constants of the dam were revealed by the forced vibrations. A theoretical analysis of the dam, based on consideration of shearing deformations only, shows moderately good agreement with the behavior which was observed at the lower frequencies.

KHANNA, J. (1969)

"Elastic Soil-Structure Interaction," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A6, pp. 143-152.

The finite element method is used to study the seismic response of a hypothetical 3-story frame building on a soil foundation. Various different soil depths were used, and in some studies the presence of piles was also simulated in the computations. The finite element system included only 10 elements, and the results were intended to indicate qualitative trends. The effects of 10 seconds of two different strong motion earthquake accelerograms were studied separately: El Centro, 1940 and Mexico City, 1962. Each record was scaled to give a maximum acceleration of 0.32 g. The results indicate that the response of the building can be considerably affected by the soil foundation conditions, and by the nature of the input earthquake. Significant bending moments in the piles were also computed in some cases.

KIEFER, F.W., JR. (1968)

"Influence of Soil Conditions on Ground Response during Earthquakes," Ph.D. thesis, Univ. of Calif. at Berkeley.

The usefulness of various combined methods of response analysis has been demonstrated by the analysis of 9 sites in Japan and the El Centro site in the U.S. The computed maximum surface acceleration and response spectra compared well with the measured values. Analysis of 8 typical soil sites with four base motions represent response conditions for moderate to severe earthquake motions and serve as a useful guide in estimating ground response at sites which resemble the typical sites closely enough to permit comparison.

KIEFER, F.W., SEED, H.B., and IDRIS, I.M. (1970)

"Analyses of Earthquake Ground Motions at Japanese Sites," Bull., Seismol. Soc. Vol. 60, No. 6, Dec., pp. 2057-2070.

Ground motions at several Japanese sites are evaluated using recently-developed which involve assessing the base-rock motion at the site due to an earthquake, the properties of the overlying soil layers, and computing the response at the face using a lumped-mass-analysis procedure. The maximum acceleration and the response spectra for ground motions recorded at the several sites agree reasonably with the computed values. It is concluded that where the soil conditions can be reasonably estimated, the analytical procedures offer good promise as a means of considering the effect of local soil conditions in the prediction of ground response.

KISHIDA, H. (1969a)

"Characteristics of Liquefied Sands During Mino-Owari, Tonankai and Fukui Earthquakes," Soils and Foundations (Japan), Vol. 9, No. 1, Mar., pp. 75-92.

The soil conditions were investigated at the 3 sites listed in the title, in which liquefaction had been observed during past earthquakes. The earthquakes occurred in 1944 and 1948, and in all cases there were severe examples of soil liquefaction and often sand and water volcanoes. All sites were characterized by deep sand and a high water table. Some investigations were also carried out at site general areas where liquefaction did not occur. The results were studied on a basis in conjunction with the laboratory cyclic loading data published recently by Peacock. From all these studies it was concluded that the following criterion for liquefaction for this intensity earthquakes, (Richter magnitude about 7): C_u (sand) soil with D_{50} less than 2 mm and C_u less than 10, effective overburden less than 2.0 kg/cm², relative density less than 75%, and no fine grained soil (silt) above the saturated sand.

KISHIDA, H. (1969b)

"A Note on Liquefaction of Hydraulic Fill During the Tokachi-Oki Earthquake," Proc. 1st to 2nd Seminar on Soil Behavior and Ground Response During Earthquakes, Univ. of California, Berkeley, Aug.

Brief report on the liquefaction of hydraulic fill at Nanae beach, Hokkaido is: the Tokachi-Oki earthquake. It was observed that liquefaction began about 5 sec after the first shock and continued for about an hour resulting in about 4 inch settlement. Results of subsurface investigations conducted before and after the quake are presented along with general conclusions as to what soils were most susceptible to liquefaction.

KISHIDA, H. (1970)

"Characteristics of Liquefaction of Level Sandy Ground During the Tokachi-Oki Earthquake," Soils and Foundations (Japan), Vol. 10, No. 2, June, pp. 103-111.

Field investigations to clarify the characteristics of liquefaction of level sand were carried out at Nanaehama beach near Hakodate in Hokkaido after the Tokachi-Oki earthquake of 1968. Three borings were carried out about three weeks after the earthquake and one of these borings was compared with those before the earthquake. The soil strata presumed to have liquefied was identified by the proposed criteria and compared with test results and photomicrographs. The hydraulic fill at Nanaehama beach under liquefaction during the Tokachi-Oki earthquake of 1968.

KISHIDA, H., and TAKANO, A. (1970)

"The Damping in Dry Sand," Proc., 3rd Japan Earthquake Eng. Symposium, Tokyo,

Free vibration tests, in which samples of dry sand under triaxial compression were subjected to sudden applications of axial stress, were used to determine the internal damping of dry sand. Tests were conducted under different confining pressures and different initial principal stress ratios.

KISHINOUE, F. (1960)

"Microseisms and Subsoil Conditions," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 3, pp. 1649-1653.

Briefly reviews literature to conclude that there exists a positive correlation between earthquake damage and amplitude of microseisms.

KJELLMAN, W. (1951)

"Testing the Shear Strength of Clay in Sweden," Geotechnique, Vol. 2, No. 3, June, pp. 225-235.

Brief review of Swedish shear test methods including direct shear, triaxial and cone testing.

KLIRISHIKOV, V.D. (1958)

"On Plasticity Law for Work-Hardening Materials," PMM, Vol. 22, No. 1, pp. 129-160.

This study is an attempt to trace the development of methods used in solving the problem of stress-strain relations in work-hardening plastic materials.

It attempts to describe the logical path of the evolution of plasticity theory, which in a reasonably short period of time led researchers from the simple Hencky-Nadai theory to the contemporary, rather broad concepts. This review does not deal with problems of large deformations, thermodynamics of deformations, or questions related to time-dependent properties of materials. Neither does it discuss methods of solving boundary-value problems on the basis of this or that plasticity law.

KO, H.Y. (1966)

"Static Stress-Deformation Characteristics of Sand," Ph.D. thesis, Calif. Inst. of Technol.

A soil test box, capable of applying any combination of principal stresses to a cubical soil sample, was developed for the experimental investigation of the behavior of granular soils under static loading. A stress control device was also developed, enabling a continuous and proportionate change to be made in the stresses along a stress path and considerably simplifying the calculation of the stress state in the sample. The apparatus was used to investigate (a) the hydrostatic compression of an Ottawa sand, and (b) the behavior of the same soil under various deviatoric stress paths in both loading and unloading conditions.

A theoretical "holey" model was postulated for sand under hydrostatic stress and the results of the analysis of this model were found to correlate closely with the experimental data.

The qualitative behavior of sand under shear stresses was examined from a particulate point of view. Specially designed tests were performed on the Ottawa sand with loading and unloading along stress paths which involved different combinations of hydrostatic and deviatoric stresses, with the purpose of examining the proportions of recoverable and irrecoverable deformations. A failure envelope was obtained for a medium dense and a medium loose sand by monotonically increasing τ_{OCT} while keeping σ_{OCT} constant under various conditions of stress distribution, and it was found that the value of equivalent Coulomb ϕ increased from 42° in triaxial compression to 48° in triaxial extension for the medium dense sand and from 36° in triaxial compression to 44° in triaxial extension for the medium loose sand.

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.

The deformational behavior of a granular soil under hydrostatic compressive stress is studied. A theoretical model consisting of regular arrays of uniform spheres, but with a probabilistic distribution of holes in the arrays, is postulated. When Hertzian behavior at the points of contact between the spheres is assumed and the holes are made to close under increasing pressure, a theoretical hydrostatic stress-strain relationship is derived. The theory is used to predict the compression of an Ottawa sand at various densities, and is found to give good agreement with the measurements obtained by testing the soil in a new apparatus. The behavior is found to be nonlinear but almost completely elastic.

KO, H.Y., and SCOTT, R.F. (1967b)

"Deformation of Sand in Shear," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM5, Pt. 1, Sept., pp. 283-310.

This is a further paper in a series by these authors describing tests on a 4 in. cube of soil in a new apparatus in which the normal stress on each of the three principal planes can be controlled independently. Because of flexible boundaries, it is claimed that the apparatus can produce a more nearly uniform state of stress than other types of presently available equipment. Simulated triaxial tests on dry uniform Ottawa sand were made to develop a more fundamental understanding of the constitutive equations for sand and to study the elastic and plastic components of deformation. It was found that deformations were essentially elastic and recoverable under hydrostatic loading before and after applying a shearing stress. Large plastic components of deformation developed under shearing stress applications.

KO, H.Y., and SCOTT, R.F. (1967c)

"A New Soil Testing Apparatus," Geotechnique, Vol. 17, No. 1, Mar., pp. 40-57.

A new apparatus for determining the stress-strain properties of soil is described in some detail. The apparatus tests a 4 x 4 x 4 in. cube of soil under compressive stresses which can be varied at will to follow any desired stress path. The sample is enclosed in a rubber membrane, which is itself located in a cubic metal box. The space between the box and the membrane is divided into six separate chambers - one for each face of the specimen, and controlled hydraulic pressure acts separately in each of these six separate chambers. A special control device was also developed to simplify the stress application along any predetermined stress path. These new pieces of equipment are described in some detail. Sample results of a drained test on Ottawa sand made to simulate a conventional triaxial compression test show that much greater volumetric strain is developed with the new device.

KO, H.Y., and SCOTT, R.F. (1968)

"Deformation of Sand at Failure" Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM4, July, pp. 883-898.

This is a continuation of a series of papers by the same authors dealing with the load-deformation characteristics of sand as determined in a special three dimensional compression apparatus. In the tests described herein the soil (Ottawa sand) was loaded to an all round confining pressure of 20 psi and then sheared with constant σ_{oct} using various combinations of σ_1 , σ_2 and σ_3 . Failure was defined by the stress at which large shear and volumetric strains began to develop. It was found that the strength was significantly dependent on σ_2 . In simulated triaxial extension tests on loose sand ϕ was 7° higher than for simulated triaxial compression tests. For dense sand the difference was about 5°. Intermediate differences were observed for intermediate values of σ_2 .

KOIZUMI, Y., and ITO, K. (1969?)

"On the Mechanism of Liquefaction of Saturated Sands During Earthquakes," mimeographed paper, publication unknown.

Liquefaction of saturated sands during earthquakes may be classified into general and local liquefaction.

General liquefaction is such a condition that the ground liquefies to a certain depth from the ground surface, while local liquefaction is such a condition that it occurs within a limited range of depth.

Whichever type of liquefaction is caused by either or both of unfavorable deformational characteristics against repeated loading and upward seepage flow during an earthquake.

Criteria for liquefaction and damage due to it are different in each type.

KONDNER, R.L., and KRIZEK, R.J. (1965)

"Dynamic Response of Cohesive Soils for Earthquake Considerations," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 1, pp. 1:96-106.

Energy storage and dissipation characteristics of cohesive soil are important in earthquake phenomena. These are studied in steady state vibratory uniaxial compression. The soil response is represented in terms of viscoelastic parameters and shows definite nonlinear (underlinear) behavior, even at small values of dynamic strain. Phase angles (dissipation measure) between applied strains and resulting stresses are small and require special instrumentation. Nonlinear response gives calculated compression moduli and propagation velocities which decrease with dynamic stress or strain level. For the limited range considered, applied static stress level has little effect on the dynamic response.

KOVACS, W.D. (1968)

"An Experimental Study of the Response of Clay Embankments to Base Excitation," Ph.D. thesis, Univ. of Calif., Berkeley.

This thesis consists of three parts: (1) the determination of the dynamic shear moduli and damping characteristics of a clay using both cyclic simple shear tests and free vibration tests; (2) the fabrication and dynamic testing on a shake table of clay embankments with surface accelerations being measured; (3) the comparison of experimental response with the predicted response using the laboratory determined dynamic soil properties in the finite element method of analysis. The author reported good agreement between the two procedures for the three different embankment configurations studied. The author therefore concluded that the response of full scale soil embankment can be predicted with reasonable accuracy by the finite element method if appropriate soil parameters are included in the computations.

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, Jan., pp. 59-75.

Cyclic, simple shear, and free vibration tests were performed on 12 in. x 12 in. x 6 in. block samples of a remolded 3:1 kaolinite-bentonite clay mixture of various water contents. The simple shear tests were performed by using predetermined sinusoidal displacements at frequencies ranging from 1 cps to 10 cps. A free-vibration damping factor was obtained from the logarithmic decrement of the free-vibration decay curve. By using the theory of the transverse vibration of shear beams, it was possible to obtain the shear modulus from a known period of free vibration. The shear moduli, established by simple shear and free-vibration tests, were in good agreement. The modulus for a given strain decreased as the shear strain increased and as the number of cycles increased. The damping value, which is also strain-dependent, increased with increasing strain. The value of free-vibration damping was always higher than the value of hysteretic damping for a given strain value.

KOVACS, W.D., SEED, H.B., and IDRIS, I.M. (1970?)

"Studies of the Seismic Response of Clay Banks," paper presented for publication to the ASCE.

Surface acceleration measurements on small scale clay banks show good agreement with predicted response using the finite element method of analysis incorporating laboratory determined dynamic soil properties. The model test embankments (i.e., a soil mass consisting of a slope with an extensive horizontal surface behind the crest and beyond the toe of the slope) were constructed of a soft kaolinite/montmorillonite mixture on the roughened surface of a horizontal shaking table. The embankments were 3 to 6 inches thick and 15 inches wide and extended to about 10 feet in length, and included a 4:1 slope. The response of the embankments studied was found to be independent of the width. Three different embankment configurations were studied. The embankments were subjected to sinusoidal base acceleration at various frequencies. Based on the success with the finite element method of analysis using laboratory determined moduli and damping characteristics, it seems reasonable to conclude that the response of full scale soil layers could be predicted with reasonable accuracy by the finite element method if appropriate soil parameters are included in the computations.

KRISHNA, J., PRAKASH, S., and THAKKAR, S.K. (1969)

"A Study of Earth Dam Models under Shock Loading," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A5, pp. 89-100.

A brief resume is presented of some model shaking table tests conducted in connection with the seismic analysis of a 122 meter high rockfill dam with a clay core. Models to 1/200 and 1/100 scale were tested using modeling techniques described in previous publications by University of California researchers. Application of several shocks of the design acceleration of 0.12 g produced no noticeable effect. Some small plastic deformations occurred when several 0.5 g shocks were applied, but significant cracking did not develop until the applied accelerations exceeded 5.0 g.

KRIZEK, R.J., and FRANKLIN, A.G. (1967)

"Energy Dissipation in a Soft Clay," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., pp. 797-807.

A constant strain series of laboratory tests performed on soft Kaolin clay utilizing cone-plate viscometer techniques show that the energy dissipation is relatively independent of

frequency from approximately 0.1 to 30 Hz. Clay specimens are subjected to harmonically applied torsional shear strain with measurement of the resulting stress. The energy dissipation is found from the calculated phase angles and measured areas of hysteresis loops. The values obtained from the phase angle are generally higher than those obtained by measuring areas of hysteresis loops. The stress strain behavior of the clay tested is nonlinear, even at low values of strain.

KRIZEK, R.J., and FRANKLIN, A.G. (1968a)

"Nonlinear Dynamic Response of Soft Clay," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 96-114.

Remolded kaolin clay at various water contents below and well above the liquid limit tested in a Weissenberg rheogoniometer shows the clay to be strain dependent with respect to the complex modulus and phase angle. The small clay samples are subjected to torsional oscillations at controlled shear strain amplitude and frequency. The testing device permitted application of uniform stress and strain over the sample. The response is found to be non-linear with the complex modulus decreasing to the -1.3 power with increasing strain amplitude. In addition the phase angle (the measure between zero-axis crossings of stress and strain) is found to vary with the 0.4 power of the strain amplitude. Hysteresis loops also provided a direct measurement of the specific loss of the soil tested. The value of the complex modulus varies with the water content with a significant difference below and above the liquid limit.

KRIZEK, R.J. and FRANKLIN, A.G. (1968b)

"Torsional Shear Testing Technique for Dynamic Properties of Clay," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 115-137.

A description of a Weissenberg rheogoniometer and the measurement of the dynamic stress-strain-time behavior of remolded cohesive soils. The soft clay sample is held by two circular platens consisting of a flat circular plate and a coaxial cone. The remolded soil is placed between the two plates. Torsional shear is imposed by rotation of the lower flat plate. The apparatus can test samples at desired frequency and shear strain. Frequency variation is from 0.0000379 to 30 Hz. Displacement and force are measured by transducers attached to the mechanism. Basic material response parameters are discussed. The device allows reasonably homogeneous stresses and strains throughout the sample.

KUBO, K. (1969)

"Vibration Test of a Structure Supported by Pile Foundation," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A6, pp. 1-12.

A huge shaking table was constructed, 10 x 2 meters in plan with a 4 m deep box. This box was filled with a cinder and oil mixture to model soft ground. The first tests were performed to model the seismic response of a 21,400 ton tank for liquid gas founded on piles driven through a 15 m deep layer of soft clay. The length scale was 1:5 and the weight scale was 1:123. Preliminary results show a definite soil-structure interaction. The stresses and deformations of the piles were considerably affected by the dynamic action of the soil pushing against them. The dynamic response was quite different from the response of a lateral static load applied to the tank.

KUMMENEJE, O., and EIDE, O. (1961)

"Investigations of Loose Sand Deposits by Blasting," Proc., 5th Internat. Conf. on Soil Mech. and Found. Eng., Paris, Vol. 1, pp. 491-497.

Blasting used to assess the probability of flow slides occurring and to compact loose sand.

LADD, C. (1964)

"Stress-Strain Modulus of Clay in Undrained Shear," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 90, No. SM5, Sept., pp. 103-132.

A comprehensive review of methods of obtaining stress-strain data (E) from undrained triaxial shear tests on saturated clays. The application of such data is illustrated by the calculation of the immediate settlement (shear at constant volume) of a clay deposit. The value of E is influenced by applied shear stress, degree of overconsolidation, time, and type of stress system. CU-tests are considered less influenced by sample disturbance than UU-tests, and are recommended for determining E-values.

LAMBE, T.W., and WHITMAN, R.V. (1969)
Soil Mechanics, John Wiley & Sons, Inc., New York.

Presents the basic concepts and principles used in the solution of most soil engineering problems. This text, written primarily for use in University courses, provides valuable background reference material for practicing engineers.

LAREW, H.G. (1960)

"Strength and Deformation Characteristics of Compacted Soils Under the Action of Repeated Axial Loads," Ph.D. thesis, Purdue Univ.

A failure criterion (based on strength rather than strain) for compacted fine-grained soils acted upon by repeated loads has been established by conventional and repeated triaxial tests on as-compacted and soaked micaceous silt, limestone residual silty clay, and a sand-clay. After the first few load applications, a critical level of repeated deviator stress is reached at which the slope of the deformation vs. repetitions curve is constant. Curves of critical void ratio vs. dry weight vs. $w\%$ vs. compactive effort. Elastic rebound observations.

LAREW, H.G., and LEONARDS, G.A. (1962)

"A Strength Criterion for Repeated Loads," Proc., Natl. Research Council, Highway Research Board, Vol. 41, pp. 529-556.

Triaxial tests on compacted fine-grained soils, utilizing repeated applications of loads of constant magnitude, has shown that a critical level of repeated deviator stress exists at which the slope of the deformation versus number of repetitions curve is constant.

To evaluate the effect of repeated loads on the strength of fine-grained soils used for highway subbases, a triaxial test device has been developed. Details of the equipment and testing technique are presented. 20-22 cycles per minute were applied to 1.4 in dia. samples. Triaxial testing with repeated load applications indicate that the stress level for pavement subbases often exceeds the elastic conditions, thus rendering useless the standard design charts for flexible pavements. The failure strength due to repeated loads on highly plastic clays was found to decrease with increasing compactive effort.

LEE, K.L. (1970)

"Comparison of Plane Strain and Triaxial Tests on Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM3, May, pp. 901-923.

Conventional laboratory practice consistently uses triaxial test procedures for soil strength investigations although many field cases in soil mechanics approximate plane strain conditions. Data from a wide variety of sources indicate that for drained tests on sand, plane strain gives the greatest strength. Very little is known about the comparative strength and deformation properties in undrained tests. This study describes a series of drained and undrained tests on saturated sand under a wide range of density and confining pressure conditions using both triaxial and plane strain loading. It was found that, for undrained conditions, at low pressures triaxial tests gave the greatest strengths, but at high pressures the greatest strengths were determined from plane strain loading. Several arguments were offered to explain this and other observations in the behavior of plane strain versus triaxial tests on saturated sands.

LEE, K.L., and FITTON, J.A. (1968)

"Factors Affecting the Cyclic Loading Strength of Soil," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 71-95.

A laboratory testing program employing cyclic triaxial tests on a range of saturated soils from silty clays to medium gravels indicates that the strength of soils increases with increasing grain size. The relative effects of grain size, grain shape and grain size distribution on the dynamic strength was studied on remolded, isotropically consolidated soils. Of the 18 different size soils tested, all but four had uniformity coefficients of 1.2 to 2.2. The test results show that very fine sands and silty sands are the weakest when tested under cyclic testing. The strength also increases up to three times the strength of fine sands for the compacted clay at equal density. Undisturbed soils were not discussed.

LEE, K. L., and SEED, H. B. (1966)

"Strength of Anisotropically Consolidated Samples of Saturated Sand Under Pulsating Loading Conditions," Report No. TE-66-3, Univ. of Calif., Soil Mechanics and Bituminous Materials Research Lab., Berkeley, Calif., July.

This article presents the results of cyclic triaxial tests performed to investigate the behavior of saturated sand under pulsating loading conditions. Special emphasis of the investigation was directed toward the determination of the strength characteristics which might be used in seismic stability analyses of earth slopes.

LEE, K.L., and SEED, H.B. (1967a)

"Cyclic Stress Conditions Causing Liquefaction of Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM1, Jan., pp. 47-70.

The susceptibility of a field deposit to liquefaction during an earthquake has been investigated by cyclic loading triaxial testing of saturated sand at various relative densities and confining pressures. Isotropically consolidated-undrained tests with cyclic deviator stress applications indicated that liquefaction should be divided into complete, partial, and failure. An almost linear relationship exists between the stress required to cause initial liquefaction in a given number of stress cycles and relative density as well as confining pressure. The cyclic stresses required to induce liquefaction or failure are much less than the failure stresses in static tests at the same confining pressure. Data of this type may be used for determining the depth of the zone in which liquefaction will first occur because of ground vibrations.

LEE, K.L., and SEED, H.B. (1967b)

"Drained Strength Characteristics of Sands," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM6, Nov., pp. 117-141.

A laboratory study of certain fundamental aspects of the strength of sand as determined by drained triaxial tests at confining pressures ranging up to 140 kg per sq cm. It was proposed that the measured strength was made up of three components: (1) sliding friction between particles which was almost independent of confining pressure; (2) dilatancy, which decreased progressively with increased confining pressures; and (3) particle crushing and rearranging, which increased with confining pressure. The conditions for shear at constant volume were discussed, and Mohr envelopes for constant volume conditions were found to compare favorably with the envelopes obtained by Rowe's or Bishop's dilatancy formulas. The so called "critical void ratio" at which sand shears at constant volume was shown to be a function of confining pressure, and a term, "critical confining pressure," was introduced to define the confining pressure required for a soil at any given density to shear at constant volume.

LEE, K.L., and SEED, H.B. (1967c)

"Dynamic Strength of Anisotropically Consolidated Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM5, Pt. 1, Sept., pp. 169-190.

Pulsating loading tests were performed on anisotropically consolidated triaxial samples of saturated sands and a silt to simulate the effect of earthquake induced stresses in soil elements behind slopes. The strength of soil under pulsating loading conditions increased with the increasing density, increasing confining pressure, and increasing ratio of major to minor principal stress during consolidation. The pulsating loading strengths were compared to strengths obtained from static loading tests, but no relations could be found to correlate the strengths from the two types of loading. However, the pulsating test methods gave results which could be used directly in calculating the stability of slopes for earthquake loading.

LEISTNER, H.G., JONES, R.E., and WALKER, W.C. (1969)

"Structure-Medium Interaction and Design Procedures Study," SAMSO TR 69-313, The Boeing Company, Vol. 1, Oct.

This report contains a detailed description of an analysis method developed for structure-medium interaction. The material properties representation and the structure modeling concepts used in the analysis models are presented. Results of analyses performed for verification of the method and compilations of typical earth media and construction material physical properties are included.

A summary section presents a description of capabilities of the FEAT computer codes developed for the analysis of structure-medium interaction. It also includes discussion of the application of the analysis method to the development of design information for advanced, hardened missile facilities. Recommended uses for the method are presented, together with suggested extensions for the enhancement of its capabilities. The computer codes can also be used to examine free field soil response.

LOMENICK, T. F., et al. (1970)

"Earthquakes and Nuclear Power Plant Design," ORNL-NSIC-28, Oak Ridge National Laboratory, Nuclear Safety Information Center, Oak Ridge, Tenn., July.

This report discusses earthquakes and nuclear plant design principally on the basis of what the earth science and engineering professions were doing about earthquakes prior to 1968. The earthquake-related phenomena of shaking, fault movement, and tsunamis are discussed from two points of view: 1) incidence and quantitative characterization and 2) engineering practices and capabilities with respect to each phenomenon. The appendices present a glossary of terms and background information, including data on U.S. earthquakes and site evaluations.

LOUW, J.M. (1971)

"Estimating Settlements of Footings on Sand," The Civil Engineer in South Africa, Transactions, South African Inst. of Civil Eng., Vol. 13, No. 5, May, pp. 155-165.

The CBR test is considered as a model plate loading test simulating a prototype footing of which the settlement is desired. CBR values can be determined rapidly by a calibrated portable dynamic CBR-penetrometer (DCP). Scale factors are determined by analysing, with the aid of the elastic theory, a number of case histories in which both settlements and Standard Penetration (SPT) blows were recorded and published by others. Predicted settlements are compared with observed settlements from the cases referred to. The method is shown to be direct and to predict satisfactorily settlements, in the linear range, on sands whether clean, gravelly or slightly clayey, above or below the water table. More realistic extrapolations from other model tests like screwplates and standard plate loadings are made possible. It also appears possible to estimate the dry density from simultaneous penetration and screwplate tests. Furthermore, the modulus of compressibility, though not required to be computed in this direct approach to the prediction of settlements, can be determined from the penetration values and is shown to agree well with field measurements. Finally, design curves are proposed for estimating the allowable bearing pressure on shallow footings.

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.

Provides results of a study of the effect of the deformability of rock underlying nine different soil deposits on the acceleration amplitude, and frequency characteristics of the motions developed in the rock and at the surface of the soil deposit using Kanai's wave propagation method.

MALCEV, A.T. (1964)

"Interpretation of Standard Spoon Penetration Testing," Proc., Symposium on the Economic Use of Soil Testing in Site Investigation, Univ. of Birmingham, pp. 3:11-18, disc., pp. 3:27-51.

A presentation of techniques of penetration resistance interpretation by means of a series of graphs showing relationships between the essential geotechnical soil properties and the penetration blow-count (as applied at Yugoslavia's Danube Hydrosystem Project). The development of the Standard split-barrel sample percussion penetration test (SSP) is described. The design graphs presented for the purpose of interpreting the SSP blow-count correspond to the Unified Soil Classification System and cover the following properties: relative density of sand; overburden pressure vs. relative density; relative density vs. bearing capacity of cohesionless soils; consistency and unconfined compressive strength of cohesive soils; in-place modulus of compressibility; angle of internal friction; and effect of relative density on the friction factor for granular soils.

MARACHI, N.D., et al. (1969)

"Strength and Deformation Characteristics of Rockfill Materials," Report No. TE-69-5, Univ. of Calif., Inst. of Transportation and Traffic Eng., Berkeley, Calif., Sept.

The authors investigated experimentally in the laboratory the factors affecting the behavior of granular materials under plane strain testing conditions and the effects of modeling the grain size distribution on the strength and deformation characteristics of rockfill materials. To investigate these factors a large scale testing apparatus was used. Results of each test series is discussed along with conclusions about each factor considered.

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.

This report contains the results of a study made of the dynamic response characteristics of the Bon Tempe Dam in California. The validity of the finite element theory was checked by means of forced vibration tests and compared with "shear slice" theories. The dynamic properties and response of the earth fills were also studied by large scale in situ testing and laboratory tests on undisturbed samples.

MASLOV, N. N. (1957)

"Questions of Seismic Stability of Submerged Sandy Foundations and Structures," Proc., 4th Intl. Conf. on Soil Mech. and Found. Eng., London, Vol. 1, pp. 368-372.

The condition of liquefaction is investigated and reported. Both laboratory and field tests are performed to verify proposed formulae. Field investigations involved using special vibration installations with a sample mass amounting to 25 tons of saturated sand. The authors conclude that the theory derived is of interest for the evaluation of stability, of submerged earth slopes, foundations for machines with a dynamic regime, and constructions on sandy foundations erected in seismic districts.

MASLOV, N.N. (1958)

"Problem of High Earth Dam Density in Conditions of Seismic Activity," Trans., 6th Internat. Cong. on Large Dams, Vol. 3, pp. 349-372.

Describes vibratory laboratory tests on sands and clays for purpose of assessing their stability under earthquakes. Argues that hydraulic fill dams are not well enough compacted to be used in high seismic zones.

MATSUSHITA, K. (1969)

"Factors to be considered in Calculating the Input Earthquake Force to Buildings," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. B5, pp. 29-40.

This general review paper discusses the importance of soil-structure interaction and points out the need of considering (among other things) the soil foundation conditions in defining the earthquake response of buildings. It is pointed out that most building codes do not take adequate account of the soil conditions. The non-linear deformation properties of soil require that consideration be given not only to the treatment of soil as an elastic material but also to consider that the seismic response will be different depending on the weight of the building and the stress level of the seismic forces. Other comments are also given strictly concerning the structural aspects: resonance, strain energy wave velocities in the structure and desirability of referring the computations to an absolute set of axes. The various points discussed are illustrated by sample diagrams showing the results of theoretical and experimental data.

MATSUSHITA, K., KISHIDA, H. and KYO, K. (1967)

"Experiments on Damping of Sands," Trans., Architectural Inst. of Japan, Summaries of Technical Papers (Annual Meeting of AIJ, 1967), p. 166.

Damping ratios for dry sands were determined by means of triaxial compression tests conducted under cyclic loading conditions; values of damping ratio ranging from 24 to 28 percent were determined for shear strains ranging from 0.2 to 2 percent.

MATTHIESEN, R.B., et al. (1964)

"Site Characteristics of Southern California Strong-Motion Earthquake Stations, Part II," Report No. 64-15, Univ. of Calif., Dept. of Eng., Los Angeles, Feb.

As an indication of the amplification of earthquake waves by layered soils, a continuous sinusoidal plane wave is considered to pass vertically from bedrock into a stratified soil system. The soil system is considered to consist of horizontal layers of defined thickness, density, and velocity of propagation. From the solutions of the equations of motion, a ratio of surface motion of the soil system with layering to that of a system without layering is obtained as a function of the frequency or the period of the incoming wave.

Most of the in-situ soil properties used in the computation of the amplification spectra in this report were presented in Part I. However, only compressional wave velocities were reported in Part I, whereas shear wave velocities were desired in the computation of the spectra. The present report summarizes the results of a study made of the relationship between shear and compressional wave velocities. In addition, some minor refinement of the previous data has been made.

MAXWELL, A.A., and FRY, Z.B. (1967)

"A Procedure for Determining Elastic Moduli of Insitu Soils by Dynamic Technique," Proc., Internat. Symposium on Wave Propagation & Dynamic Properties of Earth Materials, N. Mex., Aug., pp. 913-919.

A procedure is presented to evaluate insitu elastic moduli. The compressional and surface wave velocities are determined. It is assumed that the shear wave velocity can be considered the same as the Rayleigh wave velocity for all practical purposes. Using elastic theory Poisson's ratio, the shear modulus, G, and Young's modulus, E, are computed for the soil layers. For soil deposits the best correlation between G and E occur at a depth equal to one half the length of the surface wave. Vibrators are placed on the ground and operated at various constant frequencies. A seismic pickup and tape are used to measure the wave length on the ground surface. A small high frequency electro magnetic vibrator is used to determine elastic moduli of near surface soils while a heavier mechanical vibrator (below 30 cps) is used for deep wave penetration. Of the 40 sites investigated no more than 10% resulted in widely scattered moduli vs depth graphs. Method requires sophisticated electronic equipment.

MEUDAV, T. (1967)

"New Methods in the Application of Seismic Techniques to the Determination of Rock and Soil Properties," Proc., 3d Asian Regional Conf. on Soil Mech. and Found. Eng., Haifa, Vol. 1, pp. 342-344.

The theoretical and empirical relationships between compressional and shear wave velocities and the dynamic moduli of elasticity are outlined, and empirical relationships between the elastic moduli and the compressive strengths of sandstone and limestone rocks are given. A brief description of the instrumentation, field techniques and an analytical approach for separating and identifying the slower shear waves from the faster compressional waves is included.

MELZER, K.J. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

MOHAN, D., AGGARWAL, V.S., and TOLIA, D.S. (1970)

"The Correlation of Cone Size in the Dynamic Cone Penetration Test With the Standard Penetration Test," Geotechnique, Vol. 20, No. 3, Sept., pp. 315-319.

Dynamic cone penetration tests have been conducted in the field with varying cone sizes at different sites, with a view to finding a suitable size of cone for future adoption in the field investigations. The study has shown the possibility of relating the ratio of dynamic cone and the standard penetration test values made with the standard 51 mm o.d. split spoon sampler. On the basis of the exhaustive study made to standardize the size of the cone for dynamic penetration tests, a cone of 62.5 mm is recommended.

MROZ, Z. (1963)

"Non-Associated Flow Laws in Plasticity," J. de Mechanique, Vol. 2, No. 1, Mar., pp. 21-42.

Starting from the idealized model of a deforming element and assuming inhomogeneity of plastic deformation on a microscale, it is shown that the vector of plastic strain increment can rotate for a smooth yield surface. Next, a linear, non-associated flow law is examined from the point of view of the uniqueness criterion.

MURAYAMA, S., and SHIBATA, T. (1960)

"On the Dynamic Properties of Clay," Proc., 2nd World Con'. on Earthquake Eng., Tokyo and Kyoto, Vol. 1, pp. 297-310.

Results of slow repetitional and vibrational loading and shearing tests on clay. Strengths are plotted versus frequency of vibration. Some equipment details.

NAIR, K. (1969)

"Dynamic and Earthquake Forces on Deep Foundations," Special Technical Pub. No. 444, Symposium on Performance of Deep Foundations, ASTM, pp. 229-263.

A state of the art review of the definition of loads and the methods of analysis of deep foundations for dynamic and earthquake loads. Machine loads and pile driving loads are briefly dealt with. Water wave and earthquake forces are treated more thoroughly. The effects of dynamic loads on soil properties, such as liquefaction, are discussed. Discrete element techniques are suggested for dynamic analysis of piles and pile groups. General methods presently used in practice do not take full advantage of progress in structural dynamics. Progress in the field of soil-structure interaction will depend upon the interaction between the disciplines of soil mechanics and structural mechanics.

NATIONAL ACADEMY OF ENGINEERING, COMMITTEE ON EARTHQUAKE ENGINEERING RESEARCH (1969)

"Earthquake Engineering Research; A Report to the National Science Foundation," National Academy of Sciences, Washington, D.C.

This report addresses itself to the practical problems posed by earthquakes and discusses the research needed to solve these problems. The purpose is threefold:

1. To describe briefly the nature of the earthquake problem and the present state of knowledge in the field.
2. To indicate to research workers the area where knowledge is lacking and where further research is needed.
3. To bring the earthquake problem to the attention of government agencies and other organizations that may initiate, direct, or fund research and to provide them with information helpful in planning.

NEWMARK, N.M. (1965)

"Effects of Earthquakes on Dams and Embankments," Geotechnique, Vol. 15, No. 1, June, pp. 137-160.

Types of possible earthquake damage to embankments are described, and it is pointed out that the magnitude of movement occurring during the earthquake may cause slumping but not necessarily catastrophic failure. Theoretical expressions are derived for the stability analyses of different geometries of sliding, based on that shear force which is required to just keep the body sliding. The method is checked by test data on models of dams built of dry sand reported elsewhere in the literature.

NEWMARK, N. M. (1967a)

"Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards," Urbana, Ill., mimeographed paper, May 25.

The author presents a step by step procedure along with basic assumptions to arrive at seismic design criteria for nuclear power plants. The procedure starts with a general discussion of earthquake characteristics and follows with ground motion characteristics, soil amplification, soil structure interaction, and energy absorption characteristics of the structure itself. Also a simplified procedure for obtaining a design response spectra for various intensity earthquakes and various soil conditions is presented.

NEWMARK, N.M. (1967b)

"Problems in Wave Propagation in Soil and Rock," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., pp. 7-26.

A general review of some problems still confronting the engineer in the area of wave propaga-

tion in earth materials. The author discusses the types of waves generated by earthquakes, various types of blasts, and machinery vibrations and their effect on man made structures. Both elastic and conditions corresponding to failure are considered. Dimensional analysis in the form of dimensionless displacement, velocity, acceleration, and a variable component present an attempt to obtain reasonable correlations with the existing wide range of data. Although present computers can solve one and two dimensional numerical calculations, the results obtained for large distances and/or layered systems, unusual boundary conditions, etc., may contain large errors.

NEWMARK, N.M., and HALL W.J. (1969)

"Seismic Design Criteria for Nuclear Reactor Facilities," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 2, No. B4, pp. 37-50.

The entire problem of seismic design of nuclear reactors is discussed and ranges of values for various key design factors are given. The design assumes 3 classes of structures depending on the relative possibilities of a nuclear incident. For each class of structure a discussion and some design criteria are presented concerning the selection of a design earthquake, ground motions, soil amplification, soil structure interaction, damping in the soil and structure and methods of analysis. The paper is valuable in that it discusses so many aspects of the problem. Unfortunately as the authors state, there is too little available information on many aspects such as seismic pressures against buried structures and the response of pile foundations to earthquakes.

NOWATZKI, E.A. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

NUNNALLY, S.W. (1966)

"Development of a Liquefaction Index for Cohesionless Soils," Ph.D. thesis, Northwestern Univ.

The magnitude of excess pore water pressure developed in a saturated sand due to dynamic impulse of short duration was measured in a special liquefaction test device. A multiple regression analysis of the results was performed by a digital computer, and the data then were related to the variables of void ratio, peak acceleration of the soil, depth and surcharge load. A Liquefaction Index was defined which permits prediction of the relative sensitivity of cohesionless soils to liquefaction due to shock. The index was shown to be compatible with 28 cases of liquefaction reported in the literature.

OHSAKI, Y. (1969)

"Effects of Sand Compaction on Liquefaction During the Tokachi-Oki Earthquake," presented at the 2nd Seminar on Soil Behavior and Ground Response During Earthquakes under the auspices of U.S.-Japan Cooperative Science Program, Aug.

A case history in which actual liquefaction occurred is presented. Following the Niigata earthquake, a potentially liquefiable site is treated by vibroflotation and a paper manufacturing plant constructed. Following the Tokachi-Oki earthquake, the behavior of the plant founded on both treated and untreated areas is reported in considerable detail. Details of vibroflotation are presented as well as soil penetration data both before and after treatment.

OKAMOTO, S. (1956)

"Bearing Capacity of Sandy Soil and Lateral Earth Pressure During Earthquakes," Proc., 1st World Conf. on Earthquake Eng., Berkeley, Calif., pp. 27:1-16.

By means of model shaking tests and theoretical calculations, design criteria are obtained for the lateral pressure against walls and the bearing capacity of footings on sand. Graphs and design tables are given.

OLSON, R.E. (1960)

"The Migration of Pore Fluid During Shear in Consolidated-Undrained (R) Triaxial Tests," Proc., ASCE Research Conf. on Shear Strength Of Cohesive Soils, Univ. of Colo., pp. 1043-1049.

Data showing that it is possible to measure pore pressures (in the failure zone) through the base pedestal only if the shear stresses are applied slowly enough to allow pore water migration.

ONDA, I. (1966)

"Plane Pulse Propagation Through a Heterogeneous Medium with a Periodic Structure," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 44, pp. 1-12.

A plane pulse passing through a heterogeneous medium consists of two parts: an undisturbed and a disturbed pulse. The undisturbed pulse transmits without changing its form. On the other hand, the envelope of the disturbed pulse is of the form $\sin x/x$, its maximum appears to be delayed after the undisturbed pulse, and the period of its carrier wave is $\Pi(\text{average velocity})/(\text{wave length of the structure})$. This disturbance is interpreted as scattering by the heterogeneity of the medium.

PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4TH, SAN JUAN, PUERTO RICO (1971)
Proceedings, Vol. 1-3, American Society of Civil Engineers, New York. (Vol. 3 not yet pub.)

Arce, C.M., Torres, F.L. and Vercelli, H.J. "Experiencias Comparadas con el E.N.P. (Compared Experiences with the S.P.T.)."

This report correlates results of the Standard Penetration Test with those of other dynamic and static penetration methods. It also analyses their capacity for sampling, and the possibility of getting reliable laboratory data. It deals with silt and clays of different sedimentary origin, occasionally loessic, that cover a vast important area of the Argentina. These soils are frequently infiltrated with a calcareous binder. Deducted correlations are given, as well as an analysis of their value for shallow and deep foundation design and some comments on the reliability of these sounding and sampling methods in the exploration of the encountered soils.

De Mello, V.F.B. "The Standard Penetration Test"

From intuitions, through uses and abuses, into a state-of-the-art report, the SPT is analyzed, establishing that it comprises a well-defined penetration phenomenon involving impact energy transmission problems and energy consumed in penetration against point resistance and skin friction forces. The theoretically indicated trends should orient bona-fide statistical regressions in substitution for the existing correlations at random which are confusing. Since in every practical foundation problem many unknowns are at play, and it is quite hopeless to solve a single equation for multiple unknowns, it is proposed that the future rational development of the intuitively correct idea of penetration testing-sampling, should be based on the judicious simultaneous use of two or multiple samplers, with characteristics appropriately developed from the "casually invented" ones, for establishment of the necessary number of convenient equations. The reasons for much of the discussion around SPT values may be traced to random variations of significant parameters. Thus, rather than standardizing a single fortuitous equipment and procedure, it is concluded that the aim should be at simultaneous judicious standardization of a number of analogous, mutually complementary, techniques. The confidence limits of various present uses of SPT values are discussed.

Dobry, R. "Cono Dinamico y Prueba Standard en Arenas Limosas (Dynamic Cone and Standard Penetration Test in Silty Sands)."

The 2 inch diameter dynamic cone penetration test uses the same energy as the standard test, and it is extensively used in some countries to reduce exploration costs. A literature review was done of the cases in which both tests have been performed in silty sands. In general $N_c \geq N_s$ (c = cone; s = standard), due to lateral friction between soil and rods; the friction usually develops below the water table. Theoretical considerations, supported by the available field evidence, suggest that $\Delta N = N_c - N_s$ is a better correction index than N_c/N_s . The variation of ΔN with depth can be established by local studies. For the cases in which N_s is proportional to the depth h (which is approximately equivalent to a constant Relative Density, $\Delta N = B(h - h_0)^2 + C$). Conservative values are suggested for B , C and h_0 , useful for preliminary calculations.

Ferraz, A.D. "Sobre a Medida de Resistencia a Penetracao em Sondagens (On the Standard Penetration Test in Test Borings)."

This paper discusses the results of the work done by the I.S.S.M.F.E.'s committee on penetration resistance, specially from the American (both North - and South -) point of view, and supports the idea of the SPT standardization on universal bases for universal use. But it is suggested that such a standardization is to be accomplished in connection with rules of good practice in making reconnaissance borings, so that these ones be considered as real tests, and not just as operations. As a consequence, it is claimed that even being more expensive, the good test boring is always more economical than the poor boring. Dealing with the SPT, a distinction

based on the main function is proposed between penetrometer and sampler, even when both are represented by a single object - the split barrel. Then, retaking of the SPT universal standardization is proposed together with the idea of doing it also for the cone static penetrometer, in an attempt to take advantage of the accumulated experience in the Americas and in Europe. In conclusion, it is emphasized that if this attempt succeeded it could represent the beginning of an universal field geotechnology.

Gibbs, H.J. "Standard Penetration Test for Sand Denseness."

The purpose of this paper is to review the criteria for evaluating the standard penetration-resistance test in uncemented sand and observe the test results where sufficient additional data are available for some correlation. It is emphasized that the penetration-resistance test is only considered to be an approximate method of interpreting relative density and should not be expected to fully substitute the need of undisturbed sampling. However, it is this type of test that is extremely useful to broaden the range of foundation investigations and allow much greater coverage of data that would otherwise be limited by the expense and time-consuming care required for undisturbed sampling.

Hedges, S. "Standard Test Boring With Drilling Mud."

Improper drilling fluid (mud) technique can cause erroneous penetration test results, contaminated soil samples or damaged undisturbed samples. A drilling mud made of a premium-grade bentonite, a bentonite with a plasticity index of at least 300, is recommended for test boring when the groundwater has no electrolytes. The use of an attapulgite, with a plasticity index of at least 100, or an attapulgite-bentonite mixture when an electrolyte is present in the groundwater. The viscosity of drilling mud should be between 45 and 60 seconds as determined with a Marsh funnel. The sand content, incurred from the formation cuttings, should be limited to less than 5 percent as determined by a sand content test kit.

Langfelder, J. and Johnston, D. W. "Settlement of Two Tanks on Loose Cohesionless Soil."

A field measurement program was undertaken to evaluate the validity of settlement estimates using N-value procedures and confined compression test results. The N-value procedure greatly overestimated the actual settlement. The confined compression test results also overestimated the actual settlements but by a much smaller amount.

Melzer, K.J. "Standard Penetration Test and Relative Density."

Since groundwater greatly influences penetration resistance of soil, an empirical relation was established between the number of blows applied in the standard penetration test to sand below ground-water level and the corresponding number applied to air-dry sand at the same relative density. Also, since the number of blows was found to depend not only on the relative density, but also on the compactibility and the grain size of the penetrated sand, an empirical relation was developed between the number of blows and the relative density, with compactibility and mean grain diameter taken into account. This relation was verified by results from laboratory tests conducted with a small static penetrometer.

Nowatski, E.A. "A Theoretical Assessment of the SPT."

Static and dynamic penetration tests are widely used in the field and laboratory to obtain comparative measures of soil strength. Although the inherent inadequacies of the Standard Penetration Test (SPT) have been recognized from past experience, no theoretically rigorous method of analysis of the test exists. A theoretically correct three dimensional static analysis of the SPT, using plasticity theory and the Coulomb failure criterion, is presented. The differential equations of plastic equilibrium are solved numerically for an ideal homogeneous dry sand to show the variation of slip line field geometry and ultimate load with changes in a number of pertinent variables. The results are assessed with reference to their bearing on the factors that have been observed to influence blow count data in the field.

Reginatto, R. "Standard Penetration Tests in Collapsible Soils."

The limits of application and reliability of Standard Penetration Tests (SPT) for collapsible soil deposits are evaluated. Main characteristics and properties of these soils are given. Comparison and correlation of N values (number of blows/ft)

from SPT with the Collapsibility Coefficient (which defines the susceptibility to collapse), shear strength, and moisture content are shown. It is concluded that the SPT is of very limited value when applied to these particular soils, since the danger of collapse of the soil structure in the event of saturation is not disclosed by the SPT.

PARMALEE, R. A., et al. (1964)

"Seismic Effects on Structures Supported on Piles Extending through Deep Sensitive Clays," Research Report No. SESM 64-2, Univ. of Calif., Dept. of Civil Eng., Berkeley, Aug.

Analytical procedures for evaluating the seismic response of a structure on piles extending through soils were developed utilizing a bilinear load-deformation relationship for the soils. The results of free vibration tests (longitudinal vibrations) on samples of Elkhorn Slough silty clay were reported.

PEACOCK, W.H., and SEED, H.B. (1968)

"Sand Liquefaction Under Cyclic Loading Simple Shear Conditions," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM3, May, pp. 689-708.

A series of undrained simple shear tests were performed on saturated samples of clean medium grained sand under cyclic loading conditions intended to simulate, in the laboratory, the strength of elements of soil below level surfaces of ground during earthquake loading conditions. Qualitatively, the behavior in simple shear was very similar to the behavior in the triaxial test. For each relative density and confining pressure the cyclic shear stress required to cause liquefaction followed a well defined pattern. However, when the results of the two types of tests were compared the strengths from the simple shear tests were consistently only about 35% of the strengths from the triaxial tests. Applying certain corrections to account for the effect of the rigid boundaries of the simple shear apparatus, but realizing that this test most closely approximates the field E.Q. loading condition, it was suggested that for E.Q. liquefaction problems the triaxial test data could be employed but the strengths should be reduced by 50 percent in the analysis.

PECK, R. B., and BAZARAA, A. R. S. (1969)

Discussion of "Settlement of Spread Footings on Sand" (by D. J. D'Appolonia, E. D'Appolonia, and R. F. Brissette, Jour. of the Soil Mech. and Found. Div., ASCE, May, 1968), Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM3, May, pp. 905-909.

The authors point out inconsistencies in the Gibbs and Holtz correlation curves for estimating relative density from the penetration resistance. Also several suggested modifications are presented for improving the accuracy of settlement predictions in sand.

PENZIEN, J., et al. (1964)

"Seismic Analysis of Bridges on Long Piles," Jour. of the Eng. Mech. Div., ASCE, Vol. 90, No. EM3, June, pp. 223-254.

A method of analysis for investigating the seismic effects on bridges which are supported on long piles extending through deep sensitive clays is presented. The analysis is separated into (1) the determination of the dynamic response of the clay medium alone when excited through its lower boundary by a prescribed horizontal seismic motion, and (2) the determination of the interaction of the entire structural system, including the piles, with the moving clay medium. Application of the method to a specific bridge structure being designed by the California State Division of Highways is outlined and some preliminary results of this investigation are presented in graphical form.

REGINATTO, A.R. (1971) see PANAMERICAN CONF. ON SOIL MECH. AND FOUND. ENG., 4th, 1971.

RECHART, F.E., JR. (1960)

"Foundation Vibrations," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 86, No. SM4, Aug., pp. 1-34.

A comprehensive theoretical analysis of vibrations of foundations resting on soil. Design curves for vertical vibrations. Typical values of and methods of determining shear modulus and Poisson's ratio for soils. Examples. Table showing influence of point-bearing piles on the natural frequency.

RICHART, F.E., JR. (1965)

"Dynamically Loaded Foundations," Proc., Symposium on Bearing Capacity and Settlement of Foundations, Duke Univ., pp. 69-81, disc. pp. 135-147.

A review of the problem of dynamically loaded foundations. Data is presented on the effects of strain rate on soil properties; dynamic modulus of soils; theory of response of footings; field and laboratory tests for footings undergoing small motions; a summary of dynamic bearing capacity determinations for footings on sands and on clays; and a suggested method for estimating dynamic response of footings. It is suggested that the use of elastic theory is satisfactory for small amplitude vertical steady-state vibrations of shallow foundations, whereas a phase-plane solution should be used for small amplitude vertical dynamic response of shallow footings subjected to impulsive loads. Dynamic loading tests indicate that failure occurs by "punching" rather than by a general shear failure and that a "strain rate factor" may be applied to the static load-settlement curve for a given footing to establish a dynamic load-settlement curve.

RICHART, F.E., JR. (1968)

"Vibrations of Foundations," Soil Mechanics Lecture Series, Chicago, pp. 53-89.

A clear and concise description is presented almost in textbook style outlining the presently available methods for designing foundations for vibrating equipment and for protecting adjacent foundations against the effects of nearby vibrations. Considerably more information is given concerning the former than the latter. At the outset, both problems require criteria to be established for allowable deformations. Having established these requirements, fairly straightforward methods exist for designing a satisfactory foundation for given vibratory equipment. The method is a trial and error procedure assuming certain values and checking the computed deformations against the design requirements. Guide lines and suggestions are given for selecting the required soil data. Modulus, damping and equivalent mass. Some recent experimental results are briefly reviewed.

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.

Forced vibration tests (both longitudinal and torsional vibrations) were conducted to determine the velocities of propagation of compression and shear waves in granular soils. The vibration decay method was used to evaluate damping in these soils.

RICHART, F. E., JR., HALL, J. R., JR., and WOODS, R. D. (1970)

Vibrations of Soils and Foundations, Prentice-Hall, Inc., Englewood Cliffs, N.J.

This book presents an up-to-date thorough presentation of the theoretical techniques, field and laboratory methods and field measurements related to vibrations in soils and foundations. Coverage includes vibration of elementary systems; wave propagation in an elastic, homogeneous, isotropic medium; elastic waves in layered systems; wave propagation in saturated media; behavior of dynamically loaded soils; theories for vibrations of foundations on elastic media; isolation of foundations; instrumentation for field and laboratory measurements; and design procedures for dynamically loaded foundations.

RICHART, F.E., JR., and WHITMAN, R.V. (1967)

"Comparison of Footing Vibration Tests with Theory," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM6, Nov., pp. 143-168.

The theory of the single degree of freedom damped oscillator is briefly reviewed. The theoretical response of a vibrating mass on a semi-infinite elastic half space is presented in some detail. The results of large scale vibration tests by the U.S. Corps of Engineers are compared with the elastic half space theory, and good agreement was observed after establishing values of shear wave velocity from lab. tests and reasonable assumptions for Poisson's ratio. Reference was then made to the lumped parameter or single-degree of freedom system and a paper

by Lysmer and Richart (J. SM & F. Div., ASCE, V. 92, No. SM1) showing that with proper values for mass, spring constant, and damping correct response can also be predicted. It is suggested that the elastic half space theory is most useful for obtaining values of these lumped parameters, and the lumped parameter system is more useful for design.

ROCKER, K., JR. (1968)

"The Liquefaction Behavior of Sands Subjected to Cyclic Loading," Progress Report No. 3, Repeated Load and Vibration Tests Upon Sand, Research Report R68-36, Soils Pub. No. 221, Mass. Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., June.

Examines the liquefaction behavior of two medium to fine sands subjected to cyclic reversing deviator stresses in the isotropically consolidated triaxial state. Sand samples were tested over a range of deviator stress and density, and at two effective confining stress levels. Changes in pore pressure, axial strain, and axial stress were precisely measured by electrical recorders.

Equipment for this experimental study was constructed or adapted and proved satisfactory for these liquefaction tests. This equipment included: a modified triaxial cell; a load cell attached above the sample top cap; a pore pressure transducer; a linearsyn differential transducer (strain measurements), an air-pressure operated stress applicator; a cyclic air-pressure application system; and the necessary recording instrumentation.

These tests concluded: a) that there were significant differences in the liquefaction behavior of the two sands tested; b) there are characteristic patterns of effective stress, pore pressure, and strain behavior that may be identified; c) pre-cycling or a small percentage of air-filled voids significantly increased deviator stress required to cause liquefaction; d) the two tested sands required considerably lower deviator stress to cause failure at the same number of cycles as did previous tests on similar sands under slightly different test conditions.

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.

Following a brief review of some of the field evidence concerning amplification, this report treats in detail the theory for one-dimensional amplification employing Fourier transform and transfer function techniques. The approximate solution based upon modal analysis of a discrete model is also discussed, and an expression is developed whereby the effect of energy loss back into the underlying rock may be introduced. The final chapter discusses the relationship between the amplification spectrum and the ratio of response spectra.

ROSCOE, K.H. (1953)

"An Apparatus for the Application of Simple Shear to Soil Samples," Proc., 3rd Internat. Conf. on Soil Mech. and Found. Eng., Switzerland, Vol. 1, pp. 186-191.

Special direct shear apparatus for rectangular samples. Theory of interpretation.

ROSENBLUETH, E. (1960)

"The Earthquake of 28 July 1957 in Mexico City," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 1, pp. 359-379.

Provides a discussion and analysis of damage to structures in Mexico City from earthquake of 28 July 1957. Damage was divided into three zones and was based largely on foundation conditions.

SW-AJA (1971)

"Soil Behavior Under Earthquake Loading Conditions: Interim Report No. 1," A Joint Venture of Shannon & Wilson, Inc., and Agbabian-Jacobsen Associates, Dec.

The results of four separate studies undertaken as part of the continuing first year work effort to evaluate free field soil behavior under earthquake loading conditions are presented. 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. Where appropriate, deficient soil study areas are identified and recommendations provided for future consideration. The final chapter describes the technical approach of the joint venture to be followed during the coming year.

of subcontract work. 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.

SAMEH, A.H. M., and ANG, A.H.-S. (1968)

"Numerical Analysis of Axisymmetric Waves Propagation in Elastic-Plastic Layered Media," Civ. Eng. Studies, Structural Research Series No. 335, Univ. of Ill., Urbana, Ill., May.

A numerical procedure for the analysis of axisymmetric wave propagation in elastic and elastic-perfectly plastic layered media is developed. Simple finite difference analogs of the differential equations governing the motion are developed and related to a discrete element (lumped mass, spring) representation. Elastic and elastic-perfectly plastic media may be included.

SANGREY, D.A. (1968)

"The Behavior of Soils Subjected to Repeated Loading," Ph.D. thesis, Cornell Univ.

Consolidated undrained triaxial compression tests at controlled rates of strain and with pore water measurements were made on undisturbed samples of a saturated, inactive, silty clay employing a variety of consolidation histories in order to determine the response of soils to repeated loading. A cycle of loading to a particular stress difference (not necessarily failure) and subsequent unloading to the original stress difference produced a permanent or residual strain and a residual pore water pressure which in most cases was greater than that prior to loading. This response pattern was cumulative. Repeated loading below a so-called "critical level of repeated stress" resulted in a nonfailure elastic equilibrium showing no further increase in residual strain or pore water pressure with further cycles of loading. Repeated loading above this level ultimately achieved complete failure.

SANGREY, D.A., HENKEL, D.J., and ESRIG, M.I. (1969)

"The Effective Stress Response of a Saturated Clay Soil to Repeated Loading," Canadian Geotechnical Jour., Vol. 6, No. 3, Aug., pp. 241-252.

Triaxial specimens of an undisturbed silty clay were subjected to slowly cycled repeated loads. The rate of loading was slow (10 hours per cycle) so that accurate pore water pressures could be measured at all times during the test. The samples were tested after isotropic consolidation and loaded such that there was never any reversal in the directions of the principal stresses. For this soil it was found that, if the axial load intensity was less than 2/3 of the normal failure load, the sample reached a state of equilibrium and cycled indefinitely in a closed hysteresis loop. However, if the cyclic load intensity exceeded this value, then further axial strains continued to develop with each additional stress cycle. The explanation for this was found in the effective stress behavior. At the lower stress levels the effective principal stress ratio did not reach a failure condition, whereas the higher cyclic stress levels lead to an effective stress failure condition. A few tests on anisotropically consolidated and overconsolidated samples indicated a similar behavior pattern.

SCHMERTMANN, J.H. (1967)

"Static Cone Penetrometers for Soil Exploration," Civil Eng., (ASCE), Vol. 37, No. 6, June, pp. 71-73.

Research at the University of Florida with Swedish and Dutch static cone penetrometers has indicated that this test is a useful method for soil identification as well as for measurement of soil friction and cone bearing capacity. Details of the cones and the testing techniques (which have been refined greatly in Europe in recent years) are discussed and illustrated. The Florida experience indicates that the soil type penetrated can be predicted with reasonable accuracy and that the design value of the friction-cone penetration test is impressive, in particular for the design of pile foundations. Present U.S. contract cost of such explorations would probably be about \$3.50 to \$4.00 per foot. The Swedish equipment has a thrust capability of 2 tons, versus 12 tons for the Dutch equipment, and it is readily portable and has advantage for investigations near the ground surface where a production of about 300 ft. per day with two men and a contract cost of about \$1.00 per foot can be attained.

SCHMERTMANN, J.H. (1970)

"Static Cone to Compute Static Settlement Over Sand," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 96, No. SM3, May, pp. 1011-1043.

At least two methods for using static cone data to predict settlement exist and have received extensive use - those of Terzaghi-Peck-Meyerhof and Buisman-DeBeer. Another method is presented in this paper. All three methods are then applied to 16 test cases involving either

actual foundations, or plate load tests, with measured settlements. The proposed method simplifies calculations without sacrificing conservatism, yet appears most accurate over the full scope of the available test cases. These involve foundation widths of 2 ft to 184 ft. The key feature of the new method is that a single distribution of a strain influence factor is assumed for all cases. This method eliminates the need to compute the intermediate parameter of change in vertical stress with depth below a shallow footing. Also presented is a new correlation between static cone bearing capacity and sand compressibility. Compressibility was measured in-situ by a special screw-plate form of plate bearing load test. New data are included for the correlation between the static cone bearing capacity and SPT blow count values in sands.

SCHROEDER, W.L. (1967)

"Liquefaction of Saturated Sands," Ph.D. thesis, Univ. of Colo.

A literature review and a theoretical and experimental investigation of phenomena associated with the response of saturated sands to seismic loading. Elastic wave propagation theory is employed to describe the effects of the passage of an earthquake through a natural sand deposit. A test apparatus is developed with which simulated earthquake disturbances can be applied to small saturated sand samples, and the test results are compared with those of other investigators and with observations of naturally occurring phenomena. The results show that the effects of an earthquake can be described by a set of periodically varying normal stresses. It is concluded that liquefaction, strictly speaking, may not always occur during strong earthquakes, but that sufficient strains may develop in the presence of increased pore water pressures to cause failure of the foundations of structures.

SCHROEDER, W.L., and SCHUSTER, R.L. (1968a)

"Laboratory Simulation of Seismic Activity in Saturated Sands," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 57-70.

Laboratory dynamic triaxial test equipment and test procedures are briefly discussed wherein a mechanism for sinusoidally varying the confining pressure simulates the effects of compression waves during earthquakes. The authors conclude that in many cases the initial destructive effects of ground shaking result from compression wave effects alone. Ottawa sand and Platte River sand were tested. Variables include relative density principle stress ratio, confining pressure, initial pore pressure, and magnitude of cyclic confining stress. Qualitatively, the tests run simulate the effects of earthquake loading. It is also concluded that strains can develop corresponding to failure without partial or complete liquefaction taking place.

SCHROEDER, W.L., and SCHUSTER, R.L. (1968b)

"Liquefaction Phenomena in Saturated Sands," Proc., 6th Annual Eng. Geology and Soils Eng. Symposium, Boise, Idaho, pp. 1-22.

Laboratory experiments, designed to investigate the stress effects of an earthquake on granular soil deposits, have resulted in the following general conclusions: (1) pore water pressure increases are greater for lower initial principal effective stress ratios; (2) pore water pressure increases become smaller with increasing initial effective confining pressure; (3) greater disturbances produce greater pore-pressure increases; and (4) increasing the ratio of initial pore water pressure to initial intergranular pressure produces a more liquefaction susceptible condition. The variables investigated were those pertaining to foundation design.

SCHULTZE, E., and MELZER, K.J. (1965)

"The Determination of the Density and the Modulus of Compressibility of Non-Cohesive Soils by Soundings," Proc., 6th Internat. Conf. on Soil Mech. and Found. Eng., Montreal. Vol. 1, pp. 354-358.

Data from penetration tests performed in a 5.5 meter deep tank filled with sand and data from a large number of field penetration tests in sands were statistically evaluated to show correlation between relative density and modulus of compressibility, with static and dynamic penetration resistance.

SCOTT, R. F. (1963a)

Discussion of "Penetrometer Study of the In Situ Strength of Clays," (by R. L. Kondner, Materials Research and Standards, Vol. 2, No. 3, pp. 193), Materials Research and Standards, Vol. 3, No. 9, Sept., pp. 736-737.

Discussion of the effect of the coefficient of consolidation and of creep (e.g. adding the

time factor to the dimensionless analysis) upon penetrometer test results. Creep test results suggest that short-term determinations could cause significant deviations in the test results.

SCOTT, R. F. (1963b)

Principles of Soil Mechanics, Addison-Wesley Pub. Co., Reading, Mass.

This book presents the basic principles of soil mechanics developed for use in college classrooms and as a reference for practicing engineers. Noteworthy features of this work are its emphasis on the influence of the surface chemistry on the structure of cohesive soils, its continual attention to the relationship of soil void ratio and structure to applied stresses, and its detailed treatment of deformation, yield criterion, and stress paths. Also it includes practical techniques of approximate numerical analysis in the solution of steady-state and plastic equilibrium.

SEED, H.B. (1960)

"Soil Strength During Earthquakes," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 1, pp. 183-195.

Briefly reviews literature on past tests using vibratory and transient loads. Describes laboratory tests on compacted clay and San Francisco Bay mud using repeated loads. Compacted clay gives up to 60% strength increase depending on the number of pulses.

SEED, H.B. (1965)

"Soil and Foundation Behavior During Earthquakes," Proc., Symposium on Earthquake Eng., Vancouver, B.C., pp. 7:1-16.

The kinds of soil problems that can develop during earthquakes include compaction (with associated settlement) of cohesionless soils due to ground vibration; submergence of coastal areas; damage of structural components due to differential settlements; the formation of mud spouts or sand boils due to upward flow of water resulting from compaction of saturated deposits; landslides; or the failure of retaining walls and bridge abutments to withstand the extra soil pressure exerted on them during earthquakes. These failure modes are illustrated by photographs of recent earthquakes. The simulation of earthquakes in the laboratory and the behavior of various soils under simulated earthquake conditions are reviewed and illustrated by eight plates with typical test results for sands and clays.

SEED, H.B. (1967a)

"Slope Stability During Earthquakes," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM4, July, pp. 299-323.

A state-of-the-art review of the significant developments during the past 50 years on the subject of slope stability during earthquakes. A very few examples of landslides during recent earthquakes are discussed to illustrate the scope and importance of the problem. Reference is then made to the recent progress in analytical techniques for analyzing slope stability during earthquakes. Topics discussed include: seismic coefficient; new methods of analysis; vertical as well as horizontal E.Q. motion; effect of the duration of shaking; soil testing for dynamic strength; and the concept of how much deformation constitutes failure. None of these topics are treated exhaustively, but the critical review of the 52 references makes a useful contribution and forms a good starting place for further work in this area.

SEED, H.B. (1967b)

"Soil Stability Problems Caused by Earthquakes," Univ. of Calif., Dept. of Civil Eng., Berkeley, Calif.

The various types of soil instability problems which may develop during an earthquake are classified and illustrated by examples from recent major earthquakes in Alaska, Japan (Niigata), and Chile. Instabilities examined include settlement of cohesionless soils, liquefaction of saturated sands, flow slides due to liquefaction of cohesionless soils, bulkhead failures due to backfill liquefaction, slides caused by liquefaction of thin sand layers, failures of fills on weak foundations, and lateral movement of bridge abutments.

SEED, H.B. (1968b)

"Landslides During Earthquakes Due to Soil Liquefaction," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM5, Sept., pp. 1057-1122.

Sixty case histories of landslides which occurred as a result of soil liquefaction during earthquakes from as early as 373 BC to the present are reviewed, some in considerable detail. From this review it was concluded that this type of landslide was likely to occur where loose saturated sands or silty sands were subjected to an earthquake of magnitude 5 or greater. In many cases the problem soils represented minor geological details that could easily be missed in routine explorations. The paper continues with a description of laboratory and analytical methods of analysis to assess the stability of a given slope relative to the possibility of failure during an earthquake. Excerpts from a referenced study on the failure of the Sheffield dam were used to illustrate this method of analysis.

SEED, H.B. (1969)

"The Influence of Local Soil Conditions on Earthquake Damage," Specialty Session No. 2, Soil Dynamics, 7th Internat. Conf. on Soil Mech. and Found. Eng., Mexico City.

The damage resulting from earthquakes may be influenced in a number of ways by the characteristics of the soils in the affected area. A brief review is presented of the current state-of-the-art concerning the engineering evaluation of the influence of local soil conditions on: (1) ground response and shaking intensity, (2) soil settlement, (3) soil liquefaction, and (4) slope instability during earthquakes. The review attempts to emphasize techniques for developing a better understanding of the relationships between soil conditions and damage during earthquakes. A brief review is presented of the various types of slides caused by earthquakes, and methods of analyzing slope stability during earthquakes.

SEED, H.B., and CHAN, C.K. (1961)

"Effect of Duration of Stress Application on Soil Deformation under Repeated Loading," Proc., 5th Internat. Conf. on Soil Mech. and Found. Eng., Paris, Vol. 1, pp. 341-344.

The effect of duration of stress application on soil deformation varies widely both for sand and clay soils subjected to repeated loading, depending on the interval between stress applications.

SEED, H.B., and CHAN, C.K. (1966)

"Clay Strength under Earthquake Loading Conditions," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 92, No. SM2, Mar., pp. 53-78.

A study of the effects of various combinations of sustained and pulsating stresses on the strength and deformation characteristics of clays. The strength developed under sustained plus pulsating stress conditions normally will be considerably lower than the transient strength, but often it may be reasonably near that determined by undrained tests at normal rates of loading. A test procedure for determining the combinations of sustained stress and pulsating stress that will cause failure is outlined and illustrated by tests on three soil types. The strains resulting from different stress combinations also can be determined. In sensitive clay a series of vibrations can increase the pore water pressure and initiate creep movement that will lead to failure some time after the vibrations have ceased.

SEED, H. B., and FEAD, J. W. N. (1959)

"Apparatus for Repeated Load Tests on Soils," Special Technical Pub. No. 254, Papers on Soils, 1959 Meetings, ASTM, pp. 78-87.

Describes the details of the cyclic triaxial loading test which was developed for the purpose of determining for pavement design the permanent deformation of the soil and the elastic or resilient deformation of a soil subjected to a large number of load applications.

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.

A comprehensive theoretical and model test study of the effect of ground accelerations on the stability of slopes of cohesionless materials. The yield acceleration at which slippage occurs was determined by plain strain model tests. For dense soils a small shear strength intercept, s_i , was found at zero confining pressure, causing sliding to occur at shallow depth and increasing the yield acceleration. Useful charts of s_i vs. initial factor of safety of slope; s_i vs. grain size of sand; yield accel. vs. slide surface geometry; initial depth of sliding vs. slope length, etc.

SEED, H.B., and IDRISS, 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.

An analysis of the cause of liquefaction which destroyed much of Niigata, Japan, during the earthquake of June 16, 1964. The foundation soil was loose fine sand with very high water table. Many structures settled more than 3 ft. into the soil. Many tilted severely. Light hollow structures were buoyed up by the liquefied sand. A theoretical response analysis was performed to determine the probable dynamic stresses which were produced within the foundation soil during the earthquake. The results of pulsating loading tests on samples of saturated sand were used to estimate the dynamic strength of this soil. A comparison of the computed stresses and the laboratory determined strengths at various depths showed that a zone of failure, or liquefaction, probably developed between 15 and 25 ft. below the surface, and the liquefaction worked its way to the top. Field studies of the soil after the earthquake tend to confirm this hypothesis.

SEED, H.B., and IDRISS, I.M. (1969)

"Influence of Soil Conditions on Ground Motions During Earthquakes," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM1, Jan., pp. 99-137.

The need for considering the foundation soil properties (thickness, modulus and damping) in assessing the effect of earthquakes on the vibratory motion at the surface is explored in considerable detail. The response of a number of sites of different soil stiffness is computed for one earthquake motion. Measured and computed values of predominant period and magnification of base rock motion for two sites are presented. The variation of soil modulus and damping with strain is shown to be significantly large so that direct extrapolations from micro-tremors to large earthquakes are not possible. Finally computed and measured response motions from 4 sites are compared. The computed motions were obtained by methods described in detail in the listed references.

SEED, H.B., and IDRISS, 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.

Records of accelerations at three different depths within the soil profile at Union Bay were obtained during a mild nearby earthquake and three distant nuclear blast events. These recordings were made by instruments which had been placed in peat 10 feet below the ground surface, in clay 61 feet below the ground surface and in glacial till 105 feet below the ground surface. Accelerations in the EW, NS and vertical directions were recorded simultaneously at these three depths. Using an equivalent linear variable damping lumped-mass solution, incorporating strain-dependent material properties for the peat and clay layers, the motions throughout the deposits were computed. The recorded accelerations in the glacial till were used as input base motion. Comparisons of the time histories of accelerations, maximum accelerations and spectral values of the motions recorded during the nearby earthquake with the corresponding computed motions indicated a high degree of agreement. The maximum accelerations recorded during the distant nuclear blast events also compared well with the computed accelerations.

SEED, H.B., and IDRISS, 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.

The authors identify some of the significant factors affecting the liquefaction potential of sands during earthquakes, present a simplified procedure for evaluating liquefaction potential taking these factors into consideration, assemble available field data concerning the liquefaction or non-liquefaction behavior of sands during earthquakes and compare these data with evaluations of performance made by the simplified procedure.

SEED, H.B., IDRISS, I.M., and DEZFULIAN, H. (1970)

"Relationship Between Soil Conditions and Building Damage in the Caracas Earthquake of July 29, 1967," Report No. EERC 70-2, Univ. of Calif., Earthquake Eng. Research Center, Berkeley, Calif., Feb.

A report to the Venezuelan Presidential Commission for Investigation of damage resulting from 1967 earthquake. Objective, to determine if the damage was influenced by the characteristics of the local soil conditions. Study utilized finite element analyses to obtain soil response as well as a statistical analysis between building damage and depths of soils underlying buildings.

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.

Rock motions during earthquakes are characterized in terms of the maximum amplitude of accelerations, the predominant period of the motions, and the duration of the motions. It is shown that these characteristics depend mainly on the magnitude of the earthquake and distance of the site from the zone of energy release. Available data relating the rock motion characteristics to the magnitude of the earthquake and the distance to the causative fault are presented and used to establish a general procedure for assessing the maximum acceleration, the predominant period, the duration of shaking and the time history of accelerations developed during any given earthquake. As an example, the rock motions at the El Centro site (for the 1940 El Centro earthquake) are estimated using the proposed procedure and incorporated in analyses of the response of the site. The computed surface motions are shown to be in good agreement with those recorded during the 1940 El Centro earthquake.

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.

Pulsating triaxial tests on samples of saturated sands were used to study the factors governing liquefaction of these types of materials during earthquakes. It was found that virtually all saturated sands would liquefy under an appropriate pulsating stress. The looser the sample, the larger the stress pulse, and the lower the confining pressure, the easier it was to liquefy the samples. Typical test results are given and found to correlate well with qualitative data from large shaking table tests. The test method is readily applicable to problems involving seismic design of soil foundations.

SEED, H.B., and LEE, K.L. (1967)

"Undrained Strength Characteristics of Cohesionless Soils," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM6, Nov. pp. 333-360.

A theory is presented to explain the nature of the undrained shear strength of saturated cohesionless soils and to predict the undrained strength from the results of drained tests. The key factor is a critical confining pressure, which is physically the confining pressure required to force a sample to shear so that the volume change at failure is zero. The critical confining pressure was found to be a significant soil property which depended only on void ratio. It tends to be higher for soils which are strongly dilatant, such as sands with sound rounded grains. For many dense sands and gravels the value will range between 10 and 30 kg per sq cm. The significance of the critical pore water pressure and pore pressure cavitation was also emphasized. The theoretical hypotheses were checked by numerous triaxial test results.

SEED, H.B., and LEE, K.L. (1969)

"Pore-Water Pressures in Earth Slopes under Seismic Loading Conditions," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A5, pp. 1-11.

The results of undrained pulsating loading tests on saturated samples of sand with pore pressure measurements are discussed. It is shown that the pore pressures fluctuate greatly during the test. There is an overall increase in pore pressure with increasing number of cycles. In addition, there is a very large fluctuation of pore pressure with each change of load during every individual cycle. Because of these very large changes it was felt that an effective stress slope stability analysis was too difficult to attempt with present technology. Therefore, a total stress method of slope stability analysis was briefly outlined. This makes use of the results of pulsating loading tests on soil samples, but on a total rather than an effective stress basis. Therefore, it is not necessary to include these highly fluctuating pore pressures in the analysis.

SEED, H.B., LEE, K.L., and IDRIS, I.M. (1969)

"Analysis of Sheffield Dam Failure," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM6, Nov., pp. 1453-1490.

A study of the factors which led to the failure of this 25-ft high dam built of poorly compacted silty material in 1917 and a stability analysis using recent advances in analytical and laboratory techniques. It failed during a moderate earthquake (M 6.3) by sliding out on its base as if the lower sections of soil had suddenly lost all strength. Soil samples from the site were subjected to cyclic loading triaxial and simple shear tests, and a seismic response analysis was made using the finite element method. The theoretical minimum factor of safety before the earthquake was 1.68. Various types of stability analyses, using trial curved surfaces, indicated a factor of safety during the earthquake of about unity. However, none of these methods predicted the correct mode of failure. Finally, a detailed study was made of the time history of the stresses at various elements within the dam. This showed that failure by liquefaction began at the base below the crest of the dam and spread horizontally toward the toes, finally weakening the base sufficiently so that the reservoir water could push it downstream, as was actually observed.

SEED, H.B., and MITCHELL, J.K. (1963)

"General Report on Test Interpretation and Errors," Special Technical Pub. No. 361, Laboratory Shear Testing of Soils, ASTM, pp. 401-422.

This general report of the fourth session of the symposium reviews the contributions (indexed separately) towards solution of the problems regarding influence of sampling procedure; stress conditions in triaxial compression testing; methods of stress application; measurements of pore water pressures. Subsequent discussions add further data on these matters.

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.

The authors attempt to evaluate the applicability of the laboratory simple shear and triaxial compression tests for evaluating the liquefaction of saturated cohesionless soils under the idealized cyclic stress applications, which are the conceptual simplification of field loading conditions during earthquakes. A summary of actual stress data from the simple shear and the cyclic triaxial test is presented along with a discussion of the limitations of each test. This data is then compared with actual field stress data for known cases of liquefaction and non-liquefaction to establish a basis for suggesting the magnitude of the correction factor to be applied to the laboratory tests to obtain the simple shear stress in the field under earthquake loading conditions.

SEED, H.B., and WILSON, S.D. (1967)

"The Turnagain Heights Landslide, Anchorage, Alaska," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM4, July, pp. 325-353.

A thorough discussion is given of the nature and subsequent investigation of this landslide which occurred during the 1964 Alaska earthquake. About 130 acres of ground along a 70 ft. high bluff moved seaward as much as 2000 ft. The soil was extremely variable, but consisted essentially of horizontally bedded loose sands, silts and sensitive clays. Pulsating dynamic triaxial tests were performed in the laboratory on samples of these soils which showed that the sand and silt would probably fail by liquefaction. A model test conducted on a shaking table demonstrated the nature of the failure and helped explain the final observed positions of the debris.

SEZAWA, K. (1930)

"Possibility of the Free-Oscillations of the Surface-Layer Excited by the Seismic Waves," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 8, pp. 1-11.

An investigation of the excitation of the natural frequencies of a surface layer by seismic disturbances propagating upward through the ground is presented. The simplest case in which a plane dilatational pulse propagates vertically upward in an elastic medium and is partly transmitted and partly reflected at the bottom boundary of a superficial layer as well as reflected at the surface boundary is examined in detail, using Fourier's integral theorem. In the resulting expression a series, each term of which represents a wave resulting from a separate reflection, appears naturally. The excitation of harmonic oscillations of the surface layer is shown to be strongly dependent on the characteristic time of the input pulse.

SEZAWA, K., and KANAI, K. (1935)

"The Rate of Damping Seismic Vibration of a Surface Layer of Varying Density or Elasticity," Bull., Tokyo Univ. Earthquake Research Inst., Vol. 13, pp. 484-495.

It is established that the damping of vibrations of a surface layer, whether free or forced, depends on the ratio of both density and elastic constants of the layer to those of the subjacent medium. The question may arise as to how the vibrations should be damped in the case of a layer with varying density or elastic constants. Although the damping in such a case naturally differs greatly according to the distribution of densities or elastic constants in both the layer and in the subjacent medium, it is nevertheless possible to conclude that the damping is due mainly to the propagation of vibrational energy into the subjacent medium and very little to other causes of damping, such as internal friction in the ground, etc..

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.

The behavior of dry sand subjected to seismic loading conditions is investigated and used to check the applicability of a simple method for predicting the vertical settlement due to compaction. Measurements of shear moduli, damping and volume changes are recorded and compared using the simple shear tests. Also the effect of various factors such as strain, confining pressure and relative density are investigated to determine their influence on these measured soil properties.

A method for calculating the settlements of sand layers subjected to horizontal accelerations is suggested together with details of a typical analysis. Calculated settlements were compared with measured settlements of one foot high layers of sand subjected to horizontal shaking in shake table tests. Good agreement of compared settlements was obtained.

SOWERS, G.F. (1963)

"Strength Testing of Soils," Special Technical Pub. No. 361, Laboratory Shear Testing of Soils, ASTM, pp. 3-31.

This paper has three objectives. First, to define the requirements of strength testing and the test conditions that must be employed to meet them; second, to describe the technical features of the tests that have been developed; and third, to evaluate these tests in terms of the required test conditions and objectives.

SPARROW, R.W., and TORY, A.C. (1966)

"Behavior of a Soil Mass Under Dynamic Loading," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 92, No. SM3, May, pp. 59-83.

For the initial stages of a major research project concerned with the distribution of stress and strain in multi-layered pavement systems subjected to dynamic surface loading the work described was limited to direct loading of a single layer of silty clay contained in a pit 8 ft. sq. by 5 ft. deep, in which were buried instruments capable of measuring stress, strain and deflection under dynamic conditions. The basis for design of the instruments used in this work is outlined and a novel strain cell is described. To carry out this work under dynamic conditions it was necessary to design a loading system that was capable of delivering a single pulse of up to 5 tons with a variable duration; details are given of the pneumatic system that was developed. The initial layout of instruments to enable principal stresses and strains to be obtained is described. Tests were designed to determine stress and strain patterns set up for a variety of uniform surface contact pressures and rates of loading and the results obtained are compared with existing theories for stress distribution and surface deflection in a linear isotropic elastic half space. In addition, the relationship between apparent in-situ soil modulus and applied stress is compared with results obtained from cyclical laboratory compression tests.

STEVENS, H.W. (1966)

"Measurement of the Complex Moduli and Damping of Soils Under Dynamic Loads," Technical Report 173, U.S. Army Cold Regions Research and Eng. Lab., Hanover, N.H., Apr.

A laboratory method is described which is currently being used to investigate the required soil properties necessary in solving foundation design problems for facilities that incorporate dynamic or vibration loading within the structure and have extremely high stability requirements. Emphasis is placed on the test apparatus, nomenclature, and the analysis and computation of the test results. It has been determined that the moduli and damping vary with soil physical conditions (density, water content, degree of saturation, etc.), static load conditions (vertical and lateral pressures), and dynamic loading conditions (frequency, drive force, and drive amplitude). The test provides a measure of these relationships.

SWAIN, R.J. (1962)

"Recent Techniques for Determination of 'In-Situ' Elastic Properties and Measurement of Motion Amplification in Layered Media," Geophysics, Vol. 27, No. 2, Apr., pp. 237-241.

Determination of the elastic properties of earth materials by laboratory means is subject to sizable errors. These errors may be serious in spite of the large safety factors normally used in engineering design, particularly if the structure must withstand severe dynamic loads. Means of obtaining elastic constants "in-situ" are described. Further, the phenomenon of displacement multiplication between the underlying competent rock and overlying alluvium or fill is reviewed and the first instrumentation system designed specifically to measure this relationship for engineering purposes is described.

TAHARA, Y., TAKATA, T., and FUKUOKA, M. (1960)

"Some Experimental Studies on the Earthquake Proof Design of the Foundation of a Bridge Pier in Soft Ground," Proc., 2nd World Conf. on Earthquake Eng., Tokyo and Kyoto, Vol. 1, pp. 197-209.

Progress report of research to determine static and dynamic k value (lateral force coeff.) from tests within drill holes. Data given.

TANIMOTO, K. (1967)

"Liquefaction of Sand Layer Subjected to Shock and Vibratory Loads," Proc., 3d Asian Regional Conf. on Soil Mech. and Found. Eng., Haifa, Vol. 1, pp. 362-365.

Submerged fine-grained sand samples were subjected to a single pulse of impact, and excess water pressure and vibratory acceleration in the layer, surface settlement and average void ratio were measured. The tests were conducted to study the liquefaction of sand which occurs during earthquakes. Impact loading of the sample resulted in settlement due to the impact itself (collapse of the granular structure) as well as due to pore pressure dissipation. The surface settlement of the sample due to consolidation causes dissipation of excess water pressure. The dissipating process approximately coincides with that governed by the general theory of consolidation. The coefficient of consolidation increases as the average void ratio of the sample decreases.

TAYLOR, P.W., and BACCHUS, D.R. (1969)

"Dynamic Cyclic Strain Tests on a Clay," Proc., 7th Internat. Conf. on Soil Mech. and Found. Eng., Mexico City, Vol. 1, pp. 401-409.

Strain-controlled dynamic triaxial tests on clay show the dependence of pore pressure and, thus, mean effective stress on the applied strain amplitude. Artificial samples of halloysite were tested in triaxial cells under one hundred sinusoidal strain-controlled cycles with a complete reversal of the applied strain (and shear stress) in each cycle. On normally consolidated samples the pore pressure buildup was such that the mean effective principal stress showed negligible cyclic variation, while a continuous decrease at a reducing rate was noted with increasing number of cycles. For overconsolidated samples the effective mean principal stress increased during the first cycle and then decreased. The initial increase was found to increase with the over-consolidation ratio. After the dynamic test, the samples were subjected to a conventional compression test and a loss in strength was noted.

TAYLOR, P.W., and HUGHES, J.M.O. (1965)

"Dynamic Properties of Foundation Subsoils as Determined from Laboratory Tests," Proc., 3d World Conf. on Earthquake Eng., New Zealand, Vol. 1, pp. 196-214.

Elastic and energy-dissipation properties of clays are required in the dynamic analysis and design of earthquake resistant structures founded on (or, in the case of dams, constructed of) such materials. This paper describes a series of dynamic tests on soil samples in which the effects of amplitude and number of repetitions of loading are particularly investigated. It is found that the elastic modulus decreases as the number of cycles of loading increases. It is also found that the measured value of elastic modulus is markedly dependent on strain amplitude, being greatest at small amplitudes. The implications of these results as affecting the dynamic response of structures are discussed.

TAYLOR, P.W., and MENZIES, B.K. (1963)

"The Damping Characteristics of Dynamically-Stressed Clay," Proc., 4th Australia-New Zealand Conf. on Soil Mech. and Found. Eng., Univ. of Adelaide, So. Australia, pp. 57-60.

Saturated remolded clay samples were tested triaxially under oscillatory sinusoidal loads of constant amplitude. Stresses and strains were simultaneously displayed on an oscilloscope,

the resulting hysteresis loop representing damping characteristic of soil. Damping was found to increase with frequency and amplitude of deformation. Results are limited to frequencies between 8 and 42 cycles/sec.

TERZAGHI, K. (1943)

Theoretical Soil Mechanics, John Wiley & Sons, New York.

Chapter XIX: vibration problems reviewed and analyzed. Theories of free harmonic vibrations, forced harmonic vibrations, coefficient of dynamic subgrade reaction. Natural frequency analyses of water tower and engine foundations. Wave transmission theories.

THIERS, G. R. (1965)

"The Behavior of Saturated Clays Under Seismic Loading Conditions," Ph.D. thesis, Univ. of Calif., Berkeley.

Undrained samples of San Francisco Bay Mud were tested under triaxial and simple shear conditions. Variations in strength and the parameters characterizing a bilinear hysteresis stress-strain relationship were determined.

THIERS, G.R., and SEED, H.B. (1968a)

"Cyclic Stress-Strain Characteristics of Clay," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 94, No. SM2, Mar., pp. 555-569.

Cyclic simple shear tests were performed on undisturbed samples of San Francisco Bay mud to determine the shear stress-strain modulus and the strength under cyclic loading. For analysis, the smooth stress-strain curves were simulated with a bilinear parallel stress-strain model defining two values of modulus, G_1 and G_2 . It was shown that the modulus values decreased about 30 percent during the first 50 cycles and were then nearly constant. For samples subjected to strains of less than about 1.5%, the static strength was unaffected by 200 cycles. This was a significant finding since even strong earthquakes were not likely to produce more than 200 cycles of strain amplitude exceeding 1.5%. The simple shear apparatus and its strain-control equipment also are described.

THIERS, G.R., and SEED, H.B. (1968b)

"Strength and Stress-Strain Characteristics of Clays Subjected to Seismic Loading Conditions," Special Technical Pub. No. 450, Symposium on Vibration Effects of Earthquakes on Soils and Foundations, ASTM, pp. 3-56.

Dynamic simple shear tests performed on two clay soils show that loss of soil strength depends on the stress level, the number of cycles and the strain at which the sample fails in static undrained shear. San Francisco Bay mud and Anchorage silty clay (Zone 1) were tested in cyclic simple shear. Failure criteria adopted was when the strain reached 20%. It was found that near identical results were obtained using cyclic triaxial tests provided the triaxial sample was trimmed with its major axis inclined at 45 degrees. A relationship between number of cycles to cause failure and the stress level (in terms of % static strength) multiplied by the failure strain can be used to estimate the available strength under cyclic loading conditions. For the two soils tested it is found that if the peak strain is less than one-half the failure strain then approximately 20% reduction in strength will occur after 200 cycles. A comparison of data obtained in simple shear and triaxial tests is given.

THOMSON, W.T. (1950)

"Transmission of Elastic Waves Through a Stratified Solid Medium" Jour. of Applied Physics, Vol. 21, Pt. 1, Feb., pp. 89-93.

The transmission of a plane elastic wave at oblique incidence through a stratified solid medium consisting of any number of parallel plates of different material and thickness is studied theoretically. The matrix method is used to systematize the analysis and to present the equations in a form suitable for computation.

TIMMERMAN, D.H., and WU, T.H. (1969)

"Behavior of Dry Sands Under Cyclic Loading," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 95, No. SM4, July, pp. 1097-1112.

Cyclic loading tests on triaxial specimens of dry samples of Ottawa standard sand are reported.

Both the axial and the lateral stresses were cycled at frequencies of 2.5, 5, 10 and 25 Hz up to a maximum of 10,000 to 200,000 cycles. Both the axial and the deviatoric components of strain were measured, from which the shear strain could be deduced. Samples of all densities compacted and strained under the repeated loading, but in all cases the shear strains were much greater than the volumetric strains. Most of the significant deformations develop in the first 2,000 cycles. The amount of strains which develop depends on the principal stress ratio developed during the cyclic loading. At low confining pressures (7 psi) significant deformations occur for principal stress ratios about 1/2 of the static failure value, whereas at higher confining pressures (20 psi) 3/4 of the static value is required.

TRIFUNAC, M.D. (1970)

"Response Envelope Spectrum and Interpretation of Strong Earthquake Ground Motion," Report No. EERL 70-06, Calif. Inst. of Technology, Earthquake Engineering Research Laboratory, Pasadena, Calif., Aug.

A multiple filter technique is developed using a single degree of freedom lightly damped oscillator as a narrow band-pass filter. It provides for a physically simple approach to the analysis of accelerograph records from the point of view of structural engineering.

The analysis of several typical accelerograph records indicates that a significant portion of strong earthquake ground motion consists of surface waves. It is concluded that the duration of intense shaking will be determined predominantly by the dispersion properties of the ground.

TROREY, A.W. (1962)

"Theoretical Seismograms with Frequency and Depth Dependent Absorption," Geophysics, Vol. 27, No. 6, Pt. 1, Dec., pp. 766-785.

In the computation of conventional theoretical (or "synthetic") seismograms, the effects of the variation of frequency-dependent absorption with depth are presently ignored. Such absorption can produce significant differences in both the relative amplitudes and frequency spectra of primary and multiple reflections having the same arrival time. This paper describes a feasible way, using a digital computer of the IBM 7090 class, for computing theoretical seismograms which properly take into account the variation of absorption with both frequency and depth, it being assumed that the absorption coefficient varies linearly with frequency. It is pointed out that attempts to solve the problem using Fourier analysis in the frequency domain would lead to significant aliasing errors. Consequently, a method borrowed from the field of network theory utilizing deconvolution is devised whereby solutions are obtained directly in the time domain. Both "primary" and "primary-plus-all-multiple" traces are computed. These calculations demonstrate that absorption can reduce the multiple content of theoretical seismograms.

TRULIO, J.G. (1966)

"Theory and Structure of the AFTON Codes," Report AFWL-TR-66-19, Air Force Weapons Lab., Kirtland AFB, N. Mex., June.

A procedure for writing finite difference analogs of the principles of continuum mechanics is presented. The method leads to analogs of the integral statements of mass and momentum conservation, and the first law of thermodynamics, which are exact under two simple discretization assumptions, and which imply an exactly conservative finite difference equation for the total energy. The method and the equations which follow from it apply to general systems of continuous media, hydrodynamic or otherwise. The finite difference equations form the basis of a set of computer codes for the calculation of motion described by one and two spatial coordinates. The codes permit the use of arbitrary time dependent coordinate systems to solve specific problems.

TSAI, N.C. (1969)

"Influence of Local Geology on Earthquake Ground Motion," Ph.D. thesis, Calif. Inst. of Technol.

As a simplified approach for estimating theoretically the influence of local subsoils upon the ground motion during an earthquake, the problem of an idealized layered system subjected to vertically incident plane body waves was studied. Both the technique of steady-state analysis and the technique of transient analysis were used to analyze the problem. To demonstrate the applicability of the models to actual cases, three examples of actually recorded earthquake events are examined. It is concluded that significant modification of

the incoming seismic waves, as predicted by the theory, is likely to occur in well defined soft subsoils during an earthquake, provided that certain conditions concerning the nature of the incoming seismic waves are satisfied.

TSAI, N.C. (1970)

"A Note on the Steady-State Response of an Elastic Half-Space," Bull., Seismol. Soc. of Am., Vol. 60, No. 3, June, pp. 795-808.

With reference to the influence of local geology on earthquake ground motions, a more complete analytical formulation is made of the well-known problem of a horizontally stratified, linearly-elastic half-space subjected to vertically traveling, sinusoidal, plane waves. A more general interpretation of a result of Kanai is given, and a recursion formula is derived for computing amplification spectra. Some special properties of the system are pointed out and numerical examples are given.

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.

A new method is presented for computing the transient response of a set of horizontally stratified, linearly elastic layers overlying a uniform half-space and excited by vertically incident, transient plane waves. In addition, a simple approximate method of satisfactory accuracy is developed that reduces the computing time required. Calculated responses are compared with motions recorded under Union Bay in Seattle to evaluate the agreement between recorded and calculated motions.

U.S. ATOMIC ENERGY COMMISSION (1971)

"Seismic and Geologic Siting Criteria for Nuclear Power Plants," Atomic Energy Commission Tentative Regulatory Criteria, Feb. (official use only).

A regulatory document which sets forth criteria for the design of nuclear power plant structures, equipment, piping and systems for seismic loads. Gives analysis requirements load combinations and allowable stresses or load factors.

U.S. ATOMIC ENERGY COMMISSION, DIV. OF REACTOR DEVELOPMENT (1963)

"Nuclear Reactors and Earthquakes," TID-7024, Prepared by Lockheed Aircraft Corp. and Holmes & Narver, Inc., Aug.

This report has been prepared to supply pertinent seismological information to engineers in the nuclear reactor field. Included are historical data on occurrence, intensity, and wave shapes. Included also are some techniques for evaluating the response of structures to such events. A brief discussion of certain reactor types and modes of operations is included to provide architectural, structural, and mechanical engineers with an insight into the design requirements of the industry.

WATERWAYS EXPERIMENT STATION (WES) (1956)

"A Review of the Soils Studies," Potamology Investigations Report No. 12-5, Vicksburg, Miss., June.

Presents a review of an extensive investigation conducted to determine the nature and cause of major bank failures along the Mississippi River and to establish criteria for locating potentially unstable deposits. Failures were in part due to liquefaction initiated due to bank erosion and also pore pressure buildup as a result of river level fluctuations. An extensive field and laboratory investigation was conducted along with the development of improved exploration and testing methods. Empirical correlations and criteria for predicting bank failures were developed and improved.

WATERWAYS EXPERIMENT STATION (WES) (1962)

"Rotary Cone Penetrometer Investigations," Potamology Investigations Report 18-1, Vicksburg, Miss., June.

This report describes a series of field and laboratory investigations conducted to develop techniques for use of the rotary cone penetrometer in the field in determining the susceptibility of point bar sand deposits to liquefaction type failure. The first part of the laboratory investigation consisted of a number of cone penetration tests performed on sands having different gradations, placed in a steel tank at different relative densities,

saturated, and subjected to different surcharge pressures. A relation between cone thrust, vertical pressure, and relative density was determined for the different sands tested. The second part of the laboratory investigation included tests to determine the change in density of sand caused by sampling. Corrections for the changes in density during sampling were developed and indicated that the change in the central 18-in. portion of a sample 30 in. long is not significant.

The field investigation consisted of cone penetration borings made in point bar deposits adjacent to undisturbed and general sample borings. Cone criteria for analysis of cone thrust data and prediction of stability with respect to liquefaction-type failures were developed.

Results of the laboratory and field tests indicate that the rotary cone penetrometer measures the combined effects of density and gradation, and can be used to differentiate between stable and unstable sand deposits.

WATERWAYS EXPERIMENT STATION (WES) (1963)

"A Procedure for Determining Elastic Moduli of Soils by Field Vibratory Techniques," Misc. Paper No. 4-577, Vicksburg, Miss., June.

The report describes the equipment, measuring techniques, and method of computing the elastic moduli using the surface vibrator method. The method employs the measurement of the velocity of waves propagated at known frequencies along the exposed surface of the soil. Determining the velocity of wave propagation over a range of frequencies provides a reliable means of deriving the elastic constants of a soil.

WEBB, D.L. (1970)

"Settlement of Structures on Deep Alluvial Sandy Sediments in Durban, South Africa," Proc., Conf., In-Situ Investigations of Soils and Rock, British Geotech. Soc., London, pp. 181-188.

The validity of field correlations between the moduli of compressibility of heterogeneous estuarine sediments obtained from deep helical plate loading tests, and their densities as reflected by standard penetration tests and deepsoundings is checked by means of a large plate loading test. The same correlations are applied to the analysis of the settlement of full scale structures, and the results suggest that it should be possible to make reliable predictions of settlement to an accuracy of within 35 per cent.

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. 45-54.

A triaxial test machine has been modified to determine quantitative values for damping capacity of soil samples subjected to forced load cycling. Equipment and procedure details. Chord modulus E_c , dissipated energy, potential energy and damping capacity for 3 granular samples related to static triaxial stresses, magnitude and frequency of load.

WEN-XI, H. (1961)

"Investigations on Stability of Saturated Sand Foundations and Slopes Against Liquefaction," Proc., 5th Internat. Conf. on Soil Mech. and Found. Eng., Paris, Vol. 2, pp. 629-631.

Brief review of principles for liquefaction of sand masses subjected to vibrations and how to prevent it. Test results of pore pressure versus density versus intensity of dynamic loads versus state of stress in sand mass; useful for design purposes.

WERNER, S.D. (1970)

"A Study of Earthquake Input motions for Seismic Design," Report No. 6914-925, Agbabian-Jacobsen Associates, June.

This report deals with one aspect of the earthquake problem, namely, the estimation of seismic ground motions in the vicinity of the nuclear power plant. It is known that the characteristics of earthquake motions at a nuclear power plant are dependent on the seismicity and fault patterns of the region, the soil properties of the site, and on the interaction effects between the soil and structure. However, due to the scarcity of earthquake ground motion records and the lack of dynamic soil-structure interaction measurements, the basic manner in which these and other parameters combine to affect the earthquake motions at a facility is only partially understood. Therefore, procedures and analysis techniques that consider these parameters in an approximate manner must be used, together with sound engineering judgments, to estimate seismic ground motions

at the site of a nuclear power plant. The purpose of this study is to investigate these procedures, and to provide some guidance regarding the effects of various physical parameters on the earthquake motions at a site.

The specific objectives of the study can be summarized as follows:

To provide a basis for developing guidelines which the engineer can use to select design levels of seismic input at a given site.

To investigate the dynamic response of a layered site and the manner in which it may affect the earthquake ground motions at a particular site.

To provide a discussion of state-of-the-art soil-structure interaction techniques suitable for use in nuclear reactor response analyses.

WHITE, R. W. (1968)

"Preliminary Investigation of Criteria for Earthquake Simulation Tests on Nuclear Power Stations," Research Staff Report WR 68-22, Wyle Laboratories, Research Div., Huntsville Facility, Dec.

This report is directed toward the development of criteria necessary for simulating the effects of earthquakes on the stability and safety of nuclear power reactors by means of model and prototype testing. Major emphasis is on seismicity, although soil amplification effects and excessive settlements and stability of the foundation are considered. It was concluded that scaled models of structure can be designed for studying the response to seismic excitations. Basic scaling criteria and scaling laws for structural models are presented along with a limited discussion of these criteria for soils. A summary of details about 15 existing power plants is also included.

WHITMAN, R.V.

- (1961a) "Compressibility of Silt from Frenchmen's Flat, Nevada," Aug.
- (1961b) "Compressibility of Glacial Till From Above Water Table," Sept.
- (1962a) "Special Compression Studies Upon Ogalla Formation and Shales From the Vicinity of Lubbock, Texas," Feb.
- (1962b) "Special Compression Studies - Idaho Formation," May.
- (1962c) "Special Compression Studies Upon Ogalla Formation From Vicinity of Cheyenne, Wyoming," July.

All above reports prepared for Shannon & Wilson, Seattle, Wash.

Present methods, procedures and test data for evaluating the bulk modulus.

WHITMAN, R. V., ed. (1962d)

"The Behavior of Soils Under Dynamic Loading," Special Summer Program, Mass. Inst. of Technol., Cambridge Mass., Aug

The author compiles a number of papers or notes from various investigators pertinent to this subject. A basic review of the principles of soil mechanics is presented along with wave propagation problems, problems involving shear failure and a discussion of foundation vibrations and pavement deflections.

WHITMAN, R.V. (1966)

"Miscellaneous Studies of the Formation of Wave Fronts in Sand," Report 25, The Response of Soils to Dynamic Loadings, Research Report R66-32, Soils Pub. No. 196, Mass. Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., May.

This report contains:

- (1) Data concerning the stress-strain behavior of dry Monterey sand using initial static stresses ranging from 2 psi to 20 psi:
 - (a) Stress-strain curves for one-dimensional compression with rise-times on the order of 25 milliseconds and stress increments of 100 psi
 - (b) Dilatational wave velocities measured using ultrasonic pulses and pulses generated by a hammer blow;
- (2) Theory for the dispersion of a wave front passing through a system of discrete particles

Another report, DASA 1226-4, utilizes these data and theory in order to study the question: under what conditions do shock fronts (characterized by very short rise times) develop as a stress wave passes through sand.

WHITMAN, R.V. (1968)

"Effect of Local Soil Conditions Upon Seismic Threat to Nuclear Power Plants," Stone & Webster Eng. Corporation, Sept.

The different one-dimensional theoretical methods available to determine response spectra which include the amplification effect of the soil were discussed in relation with two basic models of the soil, the continuous and the discrete model. It is further demonstrated that these methods for determining the dynamic behavior of the soil are in fact equivalent, at least theoretically, if appropriate values of the parameters are selected for each one. It was further concluded that the decision on which one to use for a given application depends mainly on its efficiency and its flexibility. The advantages and disadvantages of each model are also included.

WHITMAN, R.V. (1969a)

"Analysis of Liquefaction and Non-Liquefaction during Actual Earthquakes," Stone & Webster Eng. Corporation, Nov.

The author presents a quantitative method of predicting the liquefaction or non-liquefaction of granular soil deposits during earthquakes, based solely upon field observations during actual earthquakes. A plot is presented relating cyclic shear stress and initial effective stress to the relative density where liquefaction did and did not occur. For lack of data, conditions often had to be estimated and therefore the existing relationship should be used with caution until additional data verifies the presented correlation.

WHITMAN, R. V. (1969b)

"The Current Status of Soil Dynamics," Applied Mechanics Reviews, Vol. 22, No. 1, Jan., pp. 1-8.

This report briefly summarizes existing techniques and appropriate strain levels for the three main branches of soil dynamics: machine foundations, protective construction, and earthquake engineering. Currently used techniques for determining the dynamic soil properties are presented along with average curves for estimating both the shear modulus and damping as a function of strain.

WHITMAN, R.V. (1969c)

"Effect of Soil Conditions Upon Damage to Structures--Caracas Earthquake of 29 July 1967," Report to Presidential Commission for Study of the Earthquake, Nov.

A statistical study is made of the relationship between story height, depth-to-rock and degree of damage experienced by buildings in the Valley of Caracas during the earthquake of 29 July 1967. Predictions based upon the theory for the amplification of ground motions by soil are shown to be consistent with the empirical damage relationship. Based upon both the empirical and theoretical results, it is recommended that the seismic coefficient be related to both the fundamental period of the structure and the fundamental period of the ground. A relation taken from the new Chilean code is proposed for this purpose.

WHITMAN R.V (1970a)

"Evaluation of Soil Properties for Site Evaluation and Dynamic Analysis of Nuclear Plants," Seismic Design for Nuclear Power Plants, R.J. Hansen, ed., The M.I.T. Press, Cambridge, Mass., pp. 250-305.

A method for estimating the deformation of soils under loads caused by earthquakes is presented which is based on simple linear stress-strain laws and the selection of parameters needed for a linear viscoelastic analysis. These parameters are modulus or wave velocity, Poisson's ratio, and damping coefficient. The approximation, calculation, and both in-situ and laboratory measurements of these soil parameters are discussed. It is concluded that this analysis method is useful and that, for practical analysis, the accuracy with which soil properties can be evaluated is as good if not better than the accuracy with which the earthquake input can be known.

WHITMAN, R.V. (1970b)

"The Response of Soils to Dynamic Loadings; Report 26, Final Report," Contract Report No. 3-26, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., May.

This report, the 26th in a series of reports prepared under contract by MIT, deals with the role of soil mechanics with regard to weapons effects prediction and protective construction design. It assesses the state-of-the-art as of 1967, and summarizes all the research performed under the contract as well as contributions by other investigators. Results from field

experiments and theoretical analyses are interpreted so as to show the relation between soil properties and such phenomena as crater size and shape, ground motions, and response of buried structures. Chapters deal specifically with the evaluation of dynamic uniaxial strain and dynamic shear strength, and with the relation between seismic wave velocity and soil properties. Appendix A contains abstracts of the 25 earlier reports issued under the contract. Appendix B gives a brief history of the MIT soil dynamics contract.

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 Pub. No. 258, Mass. Inst. of Technol., Dept. of Civil Eng., Cambridge, Mass., May.

This brief report interprets results from shaking table tests performed by the University of Chile, and compares these results from field and other laboratory experiences.

WHITMAN, R.V., and HOEG, K. (1965)

"Two-Dimensional Analysis of Stress and Strain in Soils: Report 2, Development of Plastic Zone Beneath a Footing," Contract Report No. 3-129, U.S. Army Eng. Waterways Experiment Station, Vicksburg, Miss., Dec.

An earlier report by Whitman (1964) analyzed the need for more adequate computational procedures than presently in use for problems in soil engineering. Whitman studied the potential usefulness of plasticity theory for this purpose and suggested the application of a lumped parameter model developed by Ang and Harper (1964) to calculate the distribution of stresses and strains within soil masses. This report presents a continuation of Whitman's studies and has as its main objectives: (1) to examine the performance and accuracy of the computational model; and (2) then proceed to analyze the development of the plastic zone beneath a strip footing resting on an elastic-perfectly plastic foundation material. The report concludes that the use of the simple discrete model is a promising approach, and similar techniques will be applied during a continuation of this research program. The computed results for the flexible and rigid strip footings analyzed in this study, showed the gradual development of the plastic zone, the corresponding displacement fields, and the accumulation of footing settlements as the full shearing resistance of the foundation material was mobilized. The load-settlement curves approached asymptotically the bearing capacity which was in agreement with the ultimate load predicted by plasticity theory.

WHITMAN, R.V., and ORTIGOSA, P. (1969)

"Densification of Sand by Vertical Vibrations," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol. 3, No. A5, pp. 29-37.

A number of experiments were performed using a thin container of sand on a shaking table which produced only vertical vibrations. The tests were performed at a number of peak accelerations ranging up to 2 g, with no surcharge, with a surcharge of dead weights, and with a surcharge of air pressure acting on a rubber membrane covering the sand. In all cases it was found that there was no measurable densification until the vertical accelerations exceeded 1 g. The air pressure surcharge decreased the amount of compaction whereas the dead weight surcharge increased the compaction at any one acceleration. A simple theoretical model is presented to explain these observations. It assumes that before compaction can occur, the particles must break contact and the amount of densification depends on the free fall of the particles after the contact is broken.

WHITMAN, R.V., and RICHART, F.E., JR. (1967)

"Design Procedures for Dynamically Loaded Foundations," Jour. of the Soil Mech. and Found. Div., ASCE, Vol. 93, No. SM6, Nov., pp. 169-193.

Second of a series of two companion papers in this journal. This paper reviews the theory of foundation vibrations based on the single degree of freedom lumped parameter damped system and emphasizes that the principal problem is to insure small amplitude motions. Three physical properties are required in the design: damping, mass, and spring constant. The bulk of the paper is a review and digest of existing methods for determining and existing values already obtained for these three parameters. Damping is only important if the machine is to operate near resonance. In many cases radiation damping is much greater than internal damping. The use of a fictitious "effective soil mass" is not necessary, and good solutions are obtained using a mass equal to the weight of the machine and foundation. The spring constant is the most difficult to obtain. Several methods are discussed and several values tabulated for typical soils.

WHITMAN, R.V., ROBERTS, J.E., and MAO, S.W. (1960)

"One-Dimensional Compression and Wave Velocity Tests," Report 4, The Response of Soils to

Dynamic Loadings, DASA-1199, Mass. Inst. of Technol., Soil Eng. Div., for U.S. Army Eng. Waterways Experiment Station and Defense Atomic Support Agency, Aug.

Included in the report are:

- (a). A general discussion of the stress-strain relationship for particulate masses
- (b). Specifications for the preparation of fat, backswamp clay samples by static compaction
- (c). Data concerning creep, using a load rise-time of 30 milliseconds
- (d). Description of a preliminary attempt to measure pore pressures during compression creep
- (e). Data concerning the sonic dilatational wave velocity in a core of soft clay
- (f). Description of a preliminary attempt to measure the sonic dilatational wave velocity in a soft clay sample during consolidation

WHITMAN, R. V., et al. (1954)

"The Behavior of Soils Under Dynamic Loadings; 3, Final Report on Laboratory Studies," AFSWP-118, Mass. Inst. of Technol., Dept. of Civil and Sanitary Eng., Soil Mech. Lab., Cambridge, Mass., Aug.

The authors summarize the results of laboratory investigations performed to investigate the behavior of soils under dynamic loadings. The studies are discussed in three parts. The first part is concerned with the resistance developed to severe deformation and/or rupture under transient loading conditions. The second part discusses the stresses developed in response to small strains and phenomena associated with wave propagation. The final part deals with the problem of designing foundations for blast-resistant surface buildings.

WIEGEL, R.L. (1970)

Earthquake Engineering, Prentice-Hall, Inc., Englewood Cliffs, N.J.

A compilation of twenty comprehensive articles based on lectures given at a short course at the University of California, Berkeley in September 1965. The lectures were structured to present the state of seismology, geology and engineering as they applied to earthquake engineering.

WIGGINS, J.H., JR. (1961)

"The Effect of Site Conditions on Earthquake Intensity," Ph.D. thesis, Univ. of Ill., Urbana.

A study of site periodicity during strong motion earthquakes, and the magnitude of dynamic amplification factor at soft soil sites compared with rock sites. Over 100 E.Q. records from 7 sites were interpreted. Periodic effects were not found. Theoretical explanations of the observations, and suggestions for incorporating site characteristics into dynamic analysis of structures in high probability earthquake regions.

WILSON, S.D., and DIETRICH, R.J. (1960)

"Effect of Consolidation Pressure on Elastic and Strength Properties of Clay," Proc., ASCE Research Conf. on Shear Strength of Cohesive Soils, Univ. of Colo. pp. 419-435; disc. pp. 1086-1092.

Presentation of vibratory test equipment technique and results, indicating a correlation between strength (and thus consolidation pressure) and elastic modulus for cohesive soils.

WILSON, S.D., and KAARSBERG, E.A. (1966)

"Vibration Apparatus for Determination of Elastic Properties of Soil," Felsmechanik und Ingenieurgeologie, Vol. 4, No. 1, pp. 14-24.

A vibration apparatus has been designed, constructed and used for determining the dynamic elastic properties of soils and slightly consolidated sediments at confining pressures corresponding to those down to a depth about 150 feet in the earth. With little or no modification the apparatus could be used for similar measurements on rock samples.

Rubber membrane enclosed cylindrical specimens are held firmly in contact with a base plate

by air pressure inside a pressure tank. The frequency of either longitudinal or torsional vibrations of the base plate is varied until the quarter wave resonance frequency in the vibrating specimen is reached. This is the frequency at which the ratio of outputs from small electrical pickups attached to the top or free end and the bottom of the specimen becomes a maximum. An oscilloscope, to whose vertical and horizontal plates these outputs from the pickups at the top and bottom are respectively applied, permits this to be visually observed.

The test results for a sand, silt and clay specimen are given, illustrating the effects of varying the pressure, initial density and water content on the elastic properties of these materials.

The test data have proven to be of great value in connection with the design of tracking radar towers and in foundation problems involving vibrations such as those from reciprocating machines.

WU, T.H. (1971)

Soil Dynamics, Allyn and Bacon, Inc., Boston, Mass.

Although problems involving soil vibration have existed for a long time, intensive research into soil dynamics is a recent development. The rapid expansion of knowledge brought not only rational approaches to many soil vibration problems, but also the evolution of soil dynamics as a reasonably unified discipline.

This book attempts to present the fundamentals of soil dynamics. The basic principles of vibration are given due attention but the emphasis is directed toward the adequate representation of soil behavior in analyses. Whenever feasible, experimental and observational data are introduced to assess the reliability of the analytical results. Practical problems of design are treated as an integral part of soil dynamics. Consideration of such problems helps to establish a balanced perspective on material behavior and analysis.

YEN, B.C. (1967)

"Viscosity of Saturated Sand Near Liquefaction," Proc., Internat. Symposium on Wave Propagation and Dynamic Properties of Earth Materials, N. Mex., pp. 877-888.

A laboratory study of the cyclic shear behavior of saturated clean sands shows that it is possible to represent the behavior prior to liquefaction in terms of viscoelastic behavior. Cyclic loading triaxial tests were performed on Elmonte sand. In addition, published data from cyclic triaxial tests on Sacramento River sand and cyclic simple shear tests on Monterey sand were reevaluated in terms of apparent viscosity with similar behavior. Based on this information, the flow curve, a relationship between maximum cyclic shear stress and velocity gradients (shear rate) appears viscoelastic in nature and is independent of the type of sand, confining pressure or type of testing conditions. The viscosity of the sands increased with confining pressure.

YOSHIKAWA, S., SHIMA, M., and IRIKURA, R. (1969)

"Experimental Study of the Vibrational Characteristics of Ground," Proc., 4th World Conf. on Earthquake Eng., Chile, Vol.1, No. A2, pp. 68-80.

A theoretical derivation is presented of the response of a layered system to cyclic loading assuming 1) elastic behavior and 2) visco-elastic behavior. A coordinated experimental study was also carried out at 3 sites where the soil profile consisted of layers of sand, gravel and clay with very different shear wave velocities. A velocity meter was placed at the base of a 10 m bore hole and another at the ground surface. A larger eccentric weight vibrator was located on the ground surface and the dynamic motions on the surface and at the depth were recorded and analyzed. In addition, a record was obtained of a strong earthquake which was also analyzed. The authors conclude that the results of artificial vibrator tests, when used with the theoretical analyses, can be useful in predicting behavior during actual earthquakes.

YOSHIMI, Y. (1967)

"An Experimental Study of Liquefaction of Saturated Sands," Soil and Foundation (Japan), Vol. 7, No. 2, Mar., pp. 20-32.

Loose saturated sand was placed in a rigid box, covered with an impervious membrane, and subjected to horizontal vibration while a surcharge was applied to the membrane. Pore water pressure measurements at different depths enabled the process of liquefaction to be studied in detail. The tests were more qualitative than quantitative. Nevertheless they demonstrated

several aspects of sand liquefaction. During the early stages, the sand appears to try to densify slightly, and the pore pressure steadily increases uniformly throughout the body of sand. Then rather suddenly the pore water pressure increases to the approximate overburden pressure, remains so for a short time, and then slowly decreases as the sand consolidates to a more dense and more stable condition. The length of vibration time prior to liquefaction increased with increasing confining pressure and with decreasing acceleration.

ZEEVAERT, L. (1963)

"The Effect of Earthquakes in Soft Subsoil Conditions," Proc., 30th Annual Convention, Structural Eng. Association of Calif., Oct., pp. 74-82.

Provides a study of the influence of soft subsoil conditions on earthquake ground motion utilizing analytical solution to the equations of motion.

ZEEVAERT, L. (1964)

"Strong Ground Motions Recorded During Earthquakes of May the 11th and 19th, 1962 in Mexico City," Bull., Seismol. Soc. of Am., Vol. 54, No. 1, Feb., pp. 209-231.

The ground accelerations during the earthquakes of May 11 and 19, 1962 were recorded in Mexico City on a soft clay bed that was formerly the floor of an old lake. Records were obtained from two accelerometers, one instrument was located in the basement of a multistory building and the other was located in a nearby park. Response spectra of the ground motions are presented and analysis is made of the local soil conditions with the view to explaining the characteristics of the spectrums.

ZEEVAERT, L. (1967)

"Free Vibration Torsion Tests to Determine the Shear Modulus of Elasticity of Soils," Proc., 3d Panamerican Conf. on Soil Mech. and Found. Eng., Caracas, Vol. 1, pp. 111-129.

The knowledge of the elastic properties of non-consolidated subsoil materials is, at present, an important item of investigation in order to be able to solve seismic problems in the subsoil and earth masses. The determination of the shear modulus of elasticity is basic in the solution of these problems. The author reports in this paper a simple instrument he has used in his private laboratory for several years and by means of which one may obtain satisfactory results from the practical point of view in subsoil engineering. Moreover, results obtained for different soil materials are reported, from which one can obtain typical value of this physical property at different confining hydrostatic pressures.

ZOLKOV, E., and WISEMAN, G. (1965)

"Engineering Properties of Dune and Beach Sands and the Influence of Stress History," Proc., Internat. Conf. on Soil Mech. and Found. Eng., Montreal, Vol. 1, pp. 134-138.

The behavior of beach sand as a foundation material is studied by laboratory tests, field tests, and observations of the behavior of full scale structures founded thereon. Relative density was compared to densities from impact compaction. The N values are strongly influenced by existing and past overburden pressure. The depth of water table and size of footing influence the settlement.