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SOIL BEHAVIOR UNDER EARTHQUAKE LOADING CONDITIONS

INTERIM REPORT NO. 2

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

February 1972

SW-AJA

A Joint Venture of

SHANNON & WILSON, INC. AND AGBABIAN-JACOBSEN ASSOCIATES

Seattle • Los Angeles

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FOREWARD

This report presents the results of two separate studies undertaken as part of the continuing second year effort on the subcontract to evaluate free field soil behavior under earthquake loading. The two areas of study correspond with Tasks Nos. 7 and 8 as given in Supplemental Agreement No. 2 to Subcontract No. 3354. The subjects covered by the tasks include 1) laboratory testing studies, large simple shear device and shake table: feasibility, design guidelines, and suggested approach for future studies, and 2) analytical studies directed towards the modeling of gravelly and silty soils. 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.

The first subject is covered in the first three chapters of the report. Chapter 4 deals with the findings of the analytical studies. Details on the instrumentation required for the proposed laboratory investigations are presented in the Appendix. The standard nomenclature, terminology, definitions and symbols presented in the Appendices of the state-of-the-art report (SW-AJA, 1972) have been followed in the preparation of the text.

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

A number of contributors from both members of the joint venture assisted in the preparation of this report. For SW, the primary contributors were Dr. I. Arango and Messrs. F.R. Brown and M.S. Aggour; Messrs. S.D. Wilson and R.J. Dietrich reviewed this report in detail. Dr. R.P. Miller was the Project Engineer for SW and is also the Project Manager for the joint venture. For AJA, the primary contributors were Drs. J.W. Workman, H.S.

Ts'ao and Y.C. Eric Lee. Dr. H. Bolton Seed, Professor Civil Engineering, University of California, who is special consultant to the joint venture on this project, provided valuable guidance in these studies and in the report preparation.

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CHAPTER 1

STUDY OF THE EFFECTS OF SEISMIC LOADING ON GRANULAR SOIL DEPOSITS

1.1 INTRODUCTION

The application of cyclic loads or vibrations to either natural or man-made soil deposits has created unstable soil conditions (primarily liquefaction or subsidence in granular soils) and in some cases excessive distress or damage to buildings founded on or within these deposits.

Ground vibration may induce compaction of cohesionless sand deposits which in turn produces settlement of the ground surface. Excessive settlement may cause damage to structures supported by this material. Engineers concerned with seismic disturbances would therefore like to be able to predict the volume change potential that might be expected to develop from ground motions of known form and characteristics. Some progress has been made in studying the settlement of dry sand deposits under cyclic loading. Silver and Seed, 1969, developed an analytical tool that utilizes existing computer techniques and material properties obtained from simple shear tests to determine ground surface settlements in horizontal deposits of dry sands caused by ground shaking. To check the accuracy of their method, calculated settlements were compared with settlements observed in shaking table tests. The overall degree of agreement found was good. However, there has been no opportunity to compare computed results with field behavior and similar data for saturated sands are not available to date.

During seismic shaking, a soil element is subjected to a series of random motions. The resulting cyclic shear strains deform the element, causing rearrangement of soil particles in granular deposits, resulting in volume change of the soil. If the soil is saturated and drainage cannot occur, these volume change tendencies will lead to changes in the pore-water pressure. Thus, if the cyclic motion causes densification, a rise

in pore pressure will occur and, if large enough, may reduce the shearing strength of the sand to the point where the soil may either undergo excessive strain or lose its supporting capacity. If the pore-water pressure becomes equal to the overburden pressure, the effective stress becomes zero, and liquefaction occurs. The term "liquefaction" has been applied to the process by which a saturated mass of soil is caused by external forces to suddenly lose its shearing strength and undergo significant deformations. Much of the damage which occurred in the Chilean earthquake in May, 1960, and in Niigata, Japan, and Alaska during major earthquakes in 1964, was caused by the liquefaction and resulting subsidence of foundation soils or by liquefaction-induced landslides. This damage has focused attention on research to determine the significant parameters affecting the liquefaction potential of saturated sands.

At the present time, the liquefaction potential of a site is evaluated by: 1) using empirical rules based on observed field performance, 2) comparing the local soil conditions of the site and design earthquake characteristics with a similar site that has liquefied (usually Niigata), or 3) using the general or simplified procedure presented by Seed and Idriss, 1970a. To check the applicability of the above, and of potential new methods to improve our understanding of liquefaction, two avenues of approach show promise for future investigations, namely:

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

The following sections describe the merits of each approach and the recommendations of the SW-AJA joint venture towards advancing the state of knowledge in these areas.

1.2 METHODS OF APPROACH

1.2.1 Field Investigations

Probably the most logical area for meaningful improvement in the evaluation of soil liquefaction potential lies in increased knowledge obtained from the actual performance of real sites. This additional information comes slowly, especially in the strong motion earthquake range. Actual sites must be instrumented, the distribution and engineering properties of the soils determined by field explorations, and the site subjected to significant earthquakes. The San Francisco-Oakland area offers a unique opportunity in this respect. Sites of known liquefaction potential have been studied in detail. Meaningful information could be obtained from instrumentation installed within the deposits since strong ground motions are expected to affect the sites in the future. Until more reliable information of this type is obtained to fix more clearly the differences between soil deposits that have or have not liquefied, the results can only be used to empirically assess the liquefaction potential of any site. The collection of these data does, however, provide a basis or rough guide for assessing the results obtained in laboratory studies.

The largest effort to date in compiling data of this nature is reported by Seed and Peacock, 1970. Their investigation reports data from 35 sites in which liquefaction either did or did not occur during earthquake shaking. The authors note that considerable judgement was involved in evaluating 27 of the sites, since data were lacking and had to be estimated. It is likely that those researchers involved in liquefaction studies will continue to update and refine the future data as more earthquakes occur and field cases become evident.

It is important to realize, however, that the analysis of actual field performance of real sites faces very serious difficulties. A review of the various factors that may contribute to the liquefaction of a given sand deposit under a given ground motion shows that any attempt to separate the influence of any one of the variables at any given site represents a formidable task. Thus, even if it is assumed that the soil engineering properties of the deposit are known accurately, it would also be necessary to gain information on the actual ground shaking at the site, which in most cases can only be estimated. On the other hand, one could never be sure whether the given soil deposit would have liquefied or not under a lower or higher number of stress reversals than those to which it was subjected, a factor which complicates the interpretation of the actual field data. Since it is not possible at this time to predict when and where an earthquake will occur, researchers are forced to direct their efforts toward earthquake simulation to predict the effects of these vibrations isolating the separate effects of the different variables on soil deposits, earth structures and buildings. One approach to this problem is to study the behavior of a given system by performing tests in the laboratory.

1.2.2 Laboratory Investigations

Since the smaller scale cyclic triaxial and simple shear tests presently used to evaluate liquefaction characteristics have a number of limitations which are known but cannot be readily resolved because of boundary condition restraints, the testing of larger size specimens appears very desirable. Here, boundary effects can be minimized and other variables studied more easily.

As defined in the "Revised Program Definition" report (SW-AJA, 1970), this study is oriented toward the feasibility of utilizing large testing devices as a means for studying soil liquefaction. By testing large scale specimens of soil, it is hoped that many of the deficiencies in the smaller scale cyclic

simple shear and triaxial tests can be eliminated. From the more recent work by others and our current studies, it is apparent that the use of large scale methods for studying liquefaction is likely to produce meaningful results.

1.3 CYCLIC LOADING IN THE FIELD

During an earthquake, a soil element in the ground is subjected to a complex system of deformations resulting from the erratic sequence of ground motions induced by the earthquake. For many deposits, a major part of the soil deformations may be attributed to the upward propagation of shear waves from underlying layers so that a soil element may be considered to be subjected to a series of cyclic shear strains or stresses that reverse many times. If the ground surface is approximately horizontal, then before the earthquake, there is no shear stress on the horizontal plane. During the earthquake, cyclic shear stresses are induced on this plane for the duration of the ground shaking; this loading pattern takes place under the following general conditions:

- 1) The soil element is initially consolidated under an effective overburden pressure; which, on account of no lateral deformation, constitutes a K_0 -type consolidation.
- 2) During the earthquake, the soil element experiences a cyclic reorientation of the principal stress directions. The major principal stress is initially vertical and rotates through some angle to the right and left of its initial position.
- 3) Deformations take place under plane strain and undrained conditions.
- 4) Shear loading of the deposit takes place along three axes (2 horizontal and 1 vertical).

The basic philosophy of laboratory testing must consist of subjecting soil samples, representative of the soil elements in the field, to the same loading condition that they would experience in the field. The closer a particular testing device is

able to reproduce the field loading conditions, the more accurately it will be able to predict the field behavior. Therefore, the success in the application of laboratory test results to a particular field problem depends on the similarity between the laboratory and field conditions.

1.4 LIMITATIONS OF LABORATORY TESTING EQUIPMENT

The validity of a series of test results obtained in any laboratory testing device may be assessed by comparing such results with data obtained in other laboratory devices which more closely simulate the field loading conditions. The ultimate validity check, however, is provided by the careful comparison between any set of test results intended to reproduce the field conditions with the actual field behavior. The application of the laboratory test results to field conditions must be based on a careful evaluation of the degree to which the laboratory test procedures provide a meaningful representation of the field loading conditions.

The purpose of this section of this chapter is to assess the applicability of the small triaxial and simple shear tests in evaluating the liquefaction of saturated cohesionless soils under idealized cyclic stress applications, Seed and Peacock, 1970. The need for larger sized test specimens will be emphasized.

1.4.1 Triaxial Compression Tests

Because of its more widespread availability and the greater simplicity in testing procedures, cyclic loading triaxial compression tests are often used to evaluate the liquefaction characteristics of saturated sands for design purposes. In this type of test, a sample is initially consolidated under an ambient pressure, σ_c . The sample is then subjected to an increase in axial stress of σ_{dc} and a simultaneous reduction in lateral stress of equal amount. The normal stress on a 45 degree plane in the sample is then unchanged and the shear stress developed on the plane is equal to $\sigma_{dc}/2$. The stress changes are then reversed

so that the direction of shear stress application on the 45 degree plane is reversed; thus 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 degree plane are thus similar to those on the horizontal plane for the field loading condition. For convenience, in dealing with saturated samples, the same effective stress conditions can be produced more easily by keeping the lateral stress constant and cycling the axial stress by $\pm \sigma_{dc}$.

It should be noted that in this type of test, it is only possible to produce the desired stress changes by consolidating the sample initially under an ambient stress condition corresponding to $K_0 = 1$. If any other initial stress condition is used, there is no plane in the sample which receives the desired symmetrical changes in shear stress, corresponding to those on the horizontal plane in the field.

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_c which causes liquefaction to occur for any given sample and test condition; that is, in terms of the ratio $\sigma_{dc}/2\sigma_c$ or τ_{max}/σ_c .

It is important to note that the shear stress involved in this ratio is the maximum shear stress on the sample, whereas in the field condition and simple shear tests, the shear stress determined is considerably less than the maximum stress. In view of this, together with the fact that the triaxial test requires the adoption of a value of K equal to unity rather than the normal field condition of about 0.4, it is apparent that for these reasons alone, the stress conditions causing failure in the triaxial test will be substantially different from those causing failure in the field or the simple shear test.

However, quite apart from this difference in stress conditions, the accuracy of cyclic loading triaxial tests will also be affected by the following factors:

- 1) Under field loading conditions, the direction of the major principal stress will vary from about 0

degrees to 40 degrees 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 degrees 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 of the test specimen with the cap and the base can cause stress concentrations which can lead to premature failure of the specimen.
- 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 larger 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 take place.
- 5) Penetration of the membrane into the voids between grains may cause a change in volume to occur as the pore pressure changes.

The effects of the first four 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 $K_O = 1$ condition in the test procedure, together with a failure criterion expressed in terms of the ratio $\sigma_{dc}/2\sigma_c$, 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 are higher than the corresponding stresses in the field and will normally need to be reduced before they are used to evaluate liquefaction under field conditions.

1.4.2 Simple Shear Tests

It is readily apparent that the field loading conditions as previously described are correctly simulated in principle in the laboratory simple shear test. Both for soil elements in the field and the laboratory, the soil is first consolidated under an effective overburden pressure σ_o , and, since the sample is restrained from lateral deformation, a K-type consolidation is developed. When, during an earthquake or as a result of stress application in the laboratory, the soil element is subjected to a horizontal (and vertical) shear stress application τ_{hv} , the stress condition changes. It may be noted that due to the initial stress conditions, the maximum shear stress in the sample is not the applied shear stress τ_{hv} , but τ_{max} , a somewhat larger value. Also when the shear stress is acting, there is a reorientation of principal stress directions; the direction of the major principal stress rotates progressively through some small angle (usually less than 40 degrees) on each side of the vertical.

While, in principle, it should be a relatively simple matter to reproduce these stress conditions on representative samples in the laboratory, accurate simulation of field conditions is limited by the following factors:

- 1) The development of simple shear test equipment suitable for testing samples under conditions where pore pressures develop is no simple task, since for undrained test conditions the sample must be enclosed in a constant volume system; the simple shear device developed by Roscoe, 1953, or a torsional shear device with lateral confinement appear to date to be the only equipment available for developing these test conditions.
- 2) Unless the sample is specially constrained, it is difficult to ensure the development of a uniform shear strain throughout the height of the sample. If the shear stress is applied at the top and base of the sample, deformations may tend to concentrate in a zone of weakness within the sample, especially under the larger shear strains which develop as liquefaction is approached, leading to non-uniform stress and strain conditions within the sample. The simple shear device developed by Roscoe ensures uniformity of strains by confining the sample within rigid, but moving walls; but this type of equipment is not widely available.
- 3) Difficulties are encountered in ensuring a uniform application of shear stress across the width of the sample, the uniformity depending to a considerable extent on the stress transfer mechanism between the cap and base and the sample. Finn, Pickering and Bransby, 1969, have stated that although the rigid boundaries of the sample space within the simple shear machine displace in a mode compatible with simple shear deformation of the soil sample, this does not imply that the sample deforms in simple shear. To achieve the uniform deformation of the sample, the shear load

applied to the apparatus must be uniformly transferred to the soil sample and joints on the boundaries of the sample, so that there is no relative motion between the sample and the upper and lower plates. These requirements can be met only if adequate friction can be developed between the sand sample and the upper and lower plates. To ensure the development of adequate friction, sufficiently rough plates must be used and they must be properly seated on the same sample to develop full frictional resistance.

- 4) Difficulties are encountered in ensuring the development of complementary shear stresses along the vertical faces of a test specimen; the absence of these stresses leads to non-uniformities in stress and strain conditions near the edges of the sample and to some rocking of the cap and base under cyclic shear stress applications.
- 5) Because of the difficulties in enclosing test specimens to maintain constant volume and/or uniform strain, there is considerable difficulty in preparing test specimens in a uniform condition representative of field conditions.

The net effect of these limitations of the apparatus and test procedures and the consequent non-uniformities in stress conditions is to cause samples to fail under lower applied stress conditions than would be required in the field. The amount of the discrepancy between the stresses causing liquefaction under laboratory and field conditions will necessarily depend on the particular equipment and procedures being used. However, it may well be substantial.

Since the smaller scale cyclic simple shear tests presently used to evaluate liquefaction and ground subsidence have a great number of limitations which are known but cannot be readily resolved, as discussed above, the testing of larger

specimens appears very desirable. Here, boundary effects can be minimized and other variables studied more easily (for example, the pore pressure distribution in a large mass of saturated sand during liquefaction in a free field environment). The development of a large scale simple shear device, involves, however, a number of serious difficulties and limitations:

- 1) High initial cost. Preliminary cost estimates of the simple shear device design, which is presented in the next chapter, indicate that to develop a reasonably large device would require a sizeable investment not only in the making of the testing device itself, but also in the acquisition of the supporting equipment.
- 2) A method of preparing soil samples of a uniform density throughout is required. The preparation method should be capable of providing a wide range of density values. Several researchers have developed different techniques in sample preparation either for the small shear device or for the shake table. Serious difficulties have been found in both cases. Since no large simple shear device has ever been built, one can only speculate as to what the best placement method could be. The review of previous methods does indicate, however, that a pluvial compaction method, similar to the one used by Silver and Seed in connection with shake table studies, appears to be the most promising. If this is the case, an additional expenditure to build the spreader box would be required.
- 3) Although the limitations listed in the case of the small size simple shear device would be diminished in a larger device, it may be impossible to eliminate them completely; furthermore, other limitations may develop.

The difficulties and limitations inherent in a large scale simple shear device, as explained above, indicate that other tests, perhaps the shake table, are at this time more appropriate to continue the research on liquefaction and subsidence of sand under seismic loadings.

1.4.3 Shake Table Tests

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

Many studies employing vibrating platforms or shaking tables have been made in the past to obtain a qualitative idea regarding the response of soil masses to dynamic loads. As technology developed, more sophisticated shaking tables and instrumentation became available and investigators obtained quantitative data on the response of soil masses. An extensive review of the historical development and progress in shake table testing was presented in Chapter 4, SW-AJA, 1971, and will not be repeated here.

1.5 INFORMATION GENERATED FROM SHAKE TABLE TESTS

Shake tables offer the possibility of being useful in

soil dynamics research, especially for generating information concerning soil behavior under simulated earthquake loading conditions. Generally, three major areas would benefit, namely: 1) soil properties (shear moduli and damping ratios); 2) response studies (for natural soil deposits, banks, earth structures, and soil-structure interaction); and 3) soil behavior characteristics (settlement and liquefaction). This section deals only with the third major area of study, namely, settlement and liquefaction of granular deposits; since it covers the major effects of seismic loading on granular soil deposits, which is the purpose of the present study.

1.5.1 Settlement Characteristics

Ground vibration induces compaction of cohesionless sand deposits which in turn produces settlement of the ground surface. Excessive settlement may cause damage to structures supported by this material. Engineers concerned with seismic disturbances should therefore be in a position to predict the volume change potential that might be expected to develop from ground motions of known form and characteristics. Of the various methods available, shaking table tests are probably the most convenient means for actually measuring the settlement of sand layers under dynamic loading. Advances in this area have been rather limited in the past because of technical difficulties in preparing large soil samples and controlling testing conditions. These difficulties, however, appear to be largely resolved and shake table techniques seem to provide promising tools for research. Extensive research on settlements of cohesionless soils, subjected to seismic loading conditions is being carried out at various universities at the present time, utilizing available small scale simple shear devices.

1.5.2 Liquefaction

Several researchers who have recently reported the results of liquefaction studies on large scale samples in shake

table tests include Nunnally, 1966; Yoshimi, 1967, Whitman, 1970; and Finn, Emery and Gupta, 1970. These efforts have shown that the inability to control three major factors have limited the usefulness of data obtained in shake table tests. These limiting factors are:

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

The results of previous research are questionable in view of the measurement procedures and equipment used. In the tests by Nunnally and Yoshimi, the dimensions of the specimen, boundary conditions, and the location of pressure measuring devices at the edge of the samples were something less than desirable. The tests performed in Chile (Whitman, 1970) likewise had the same undesirable features as well as very small surcharge loads and no provision for preventing water from draining to the top of the specimen, thus relieving any tendency for the pore pressure to build up.

The work reported by Finn, Emery, and Gupta, 1970, indicates progress in the elimination of two out of the three major factors which restrict the usefulness of the shake table tests. The drainage limitations were avoided by placing a thin rubber membrane over the sample, ensuring that there was no air entrapped. The surcharge limitation was overcome by designing a rigid box, strong enough to allow air pressures up to 50 psi to be applied to the sealing rubber membrane. Only the boundary conditions and, consequently, the limited magnitude of the cyclic shear stress available for testing the samples remained unsolved. This difficulty, however, may be overcome by designing a box with adequate dimensions so that a "free-field" state of stress may be developed within a zone of the test sample. Although the difficulties in developing a shake table large enough to be successfully used in liquefaction studies must not be minimized, a comparison with the problems in connection with a large simple shear device, as discussed previously, will show the advantages

inherent to the shake table testing approach:

- 1) Low initial cost. Existing facilities, such as those at the University of California (Richmond Field Station) may be utilized, so that the only new piece of equipment required would be a sand sample container, which can be developed at a reasonably low cost. The remaining supporting equipment (hydraulic and electronic) is already available. The cost aspects will be described in detail in the next chapter.
- 2) There is a good back log of experience and know-how in the preparation of large samples for shake table tests. For instance, the spreader box utilized in the previous experiments by Silver and Seed would also be available for liquefaction research.
- 3) A properly designed large sand container would provide conditions free from boundary and other effects present in the large simple shear device. It is felt that field conditions could be faithfully reproduced in the laboratory, at least within the instrumented zone of the large shake table box.

Table 1.1 of this chapter has been prepared for the purpose of comparing the conditions during cyclic loading prevailing in a soil element within a field deposit of granular soil, and in a similar soil element tested in a large simple shear device, and in a large shake table. The relative advantages of the large shake table option over the large simple shear device are readily seen.

The following chapter describes both the simple shear and the shake table equipment which have been considered during this feasibility study. A cost comparison is then made between the two devices.

TABLE 1.1 CYCLIC LOADING OF SAND ELEMENTS - FIELD AND LABORATORY CONDITIONS

<u>ITEM</u>	<u>FIELD CONDITIONS</u>	<u>LABORATORY CONDITIONS</u>	
		<u>LARGE SIMPLE SHEAR DEVICE</u>	<u>LARGE SHAKE TABLE</u>
1	K ₀ -type initial consolidation	Same	Same
2	Deformations take place under plane strain conditions	Same	Same
3	Deformations take place under undrained conditions	Same	Same
4	Simultaneous loading along two horizontal directions and one vertical direction	Loading along one horizontal axis only	Loading along one horizontal and/or one vertical axis with existing equipment; along three mutually perpendicular axis when 3 degrees of freedom shake table becomes available
5	Stress strain response of soil element in a "free field condition"	Rigid boundary effects, stress concentrations and side friction effects unavoidable in devices of feasible, economical, and practical size. Required complementary shear stress difficult to develop in small distance into box	Field conditions achieved within the "free field zone" of the shake table test specimen

CHAPTER 2

DESIGN OF LARGE SCALE CYCLIC SIMPLE SHEAR APPARATUS AND SHAKE TABLE FACILITIES

2.1 SIMPLE SHEAR APPARATUS

2.1.1 Introduction

The first part of this chapter covers the development of the laboratory equipment and the test procedures to conduct cyclic loading simple shear tests on large saturated soil samples. The primary purpose of the large tests is to study soil behavior during cyclic loading, minimizing the effects of boundary stress conditions which develop in the smaller scale simple shear tests. The boundary effects will be minimized by testing reasonably long and wide test specimens in relation to their height. The sample dimensions were chosen as large as economically feasible, since previous theoretical and practical studies using smaller shear boxes have not provided conclusive evidence as to the size which would ensure a minimum development of undesirable side effects.

The equipment under consideration consists of: 1) a simple shear apparatus, 2) a control system for applying a cyclic load or displacement to the soil, and 3) a supporting frame. The simple shear apparatus is a plane strain device which ideally constrains a sample to deform in a uniform simple shear mode. The large simple shear box suggested in this investigation was designed after consideration of the shear box of Peacock and Seed, 1968, who had in turn patterned their box after an original design by K.H. Roscoe, 1953.

2.1.2 Simple Shear Apparatus

The apparatus consists of three main components: an upper base, lower base, and the hinged end plates. The sample dimensions are 24 in. x 24 in. x 3 in. high. The following

paragraphs discuss each item separately and how they function with respect to each other. Reference is made to Fig. 2.1, where the basic dimensions and arrangements of the various parts are shown.

a. Lower Base

The lower base is supported by four linear motion bearings, mounted on the bottom side of the base and running on 1 1/2-inch diameter steel rods. The base is steel, composed of plate and I-beam sections welded together. A function of the lower base is to carry the whole system out from under the loading frame so that the soil sample can be conveniently prepared, then slid again under the frame and secured with bolts to the floor during testing. Two vertical plates are welded to the lower base to provide a rigid support for the upper part of the hinged end plates.

b. Upper Base

The upper base is supported by four linear motion bearings. These are mounted at the sides of the base and run on 1 1/2-inch diameter steel rods. The two steel rods are fixed to the sides of the vertical plates that are welded to the lower base. The base would be constructed of steel plates and channel sections welded together. This base is free to move horizontally in the direction of the shear stress application, and would be loaded horizontally by means of an MTS system electro-hydraulic closed-loop actuator. Welded to the upper base are two vertical steel plates, which form the sides of the shear box and provide the vertical slots for the hinged end plates.

c. Hinged End Plates

Two aluminum hinged end plates are located at the ends of the shearing chamber. They are pin-connected at the top to the vertical plates welded to the lower base and provided at the lower end with ball bearing pins which ride along the

FIG. 2.1 GENERAL DESIGN GUIDE LINES FOR
A LARGE SIMPLE SHEAR APPARATUS

vertical slots in the upper base side plates.

2.1.3 Control System

From the discussion in the feasibility study of the various control systems, (Chapter 4, SW-AJA, 1971), the closed-loop electro-hydraulic technique was considered to be the most suitable means of providing the cyclic loads for the large laboratory samples, because the system can be programmed to follow steady-state, prescribed earthquake type, or random motion with a great accuracy, and is capable of producing large force out-puts at both large and small displacements.

The basic philosophy of such a system is the comparison of the desired condition of a controlled parameter with the actual performance of the parameter, and the generation of correction signals which cause the actual performance to equal the desired condition. The basic components of the system are described below.

a. Hydraulic Power Supply

As the name implies, the object is to provide fluid under pressure.

b. Servo-Valve

This is an electrical-hydraulic power transducer, to regulate the flow of fluid into a hydraulic actuator in proportion to a programmed electric signal. This flow causes the movement of the actuator piston and a change in the value of the control parameter.

c. Feedback Transducer

This element sends an electrical response signal to the servo-controller to be compared with the original input.

d. Servo-Controller

This is the module which detects the difference between

the desired value of a controlled parameter and the actual value of the parameter. If any difference is detected, a correction signal is amplified and sent to the servo valve which adjusts the flow of hydraulic fluid into the actuator to eliminate the difference.

e. Control

The control parameter is displacement, but it is possible to program velocity or acceleration by integrating the program signal once or twice, respectively, to give a displacement command.

f. Actuator Mounting

The actuator mounting bracket must be stiff to provide a fixed reference for displacement measurement within the closed-loop system. Any tilting of the actuator mounting, relative to its base, would lead to serious misalignment of the actuator.

2.1.4 The Loading Frame

The simple shear box is surrounded by a loading frame which provides a vertical reaction during testing. The loading frame is fixed to the floor. A frictionless rolling piston containing four sets of vertically aligned roller bearings is placed within the loading frame. During operation, the piston is connected to the top cap which rests on the surface of the sample. The vertical load can be applied to the soil sample by means of a single-acting air piston, or by means of an MTS closed-loop actuator.

2.1.5 Operation of the System

As explained, the upper base would be loaded horizontally by means of an MTS electro-hydraulic closed-loop actuator.

Alternating shear loads or displacements of controlled magnitudes and frequencies would be applied with no drainage

allowed to take place in the test sample. Measurements of the normal and shear loads transmitted to the apparatus, the horizontal and vertical movements of the sample, and the pore pressure would be recorded at all times.

The shear strain is defined as the horizontal displacement of the upper base divided by the height of the sample. For the designed equipment, the horizontal travel of the upper base has the limits of plus or minus 2 inches, equivalent to plus or minus 66 percent shear strain for a sample height of 3 inches. The maximum normal stress (confining pressure) which can be applied to the sample is of the order of 40 psi.

2.1.6 Cost Estimate

A breakdown of the total estimated cost to design, build, and set up the installation for the large cyclic simple shear device follows. The cost estimate does not include the value of the various measuring instruments (pressure transducers, LVDT's).

SIMPLE SHEAR DEVICE - COST BREAKDOWN

Item 1 - Design and construction of a simple shear apparatus and loading frame according to the general guide lines contained in Fig. 2.1	\$38,000
Item 2 - Design and construction of a spreader box of type similar to that in operation at the University of California, Richmond	\$ 5,000
Item 3 - Control System (MTS) consisting of:	
Hydraulic Power Supply and Manifold;	\$12,000
Hydraulic Actuator and Servo-Valve;	\$12,000
Master Control;	\$12,000
System Engineer;	\$ 5,000
Item 4 - Recording System (6 channel)	<u>\$10,000</u>
Total Estimated Cost	\$94,000
Estimated completion time for fabrication	6 months

NOTE: Budget does not include the cost of the instrumentation.

2.2 SHAKE TABLE EQUIPMENT

2.2.1 General

As in the case of the simple shear device, the shake table facilities involve three separate pieces of equipment: 1) the box proper to contain the sample to be tested, 2) a control system for applying a cyclic load to the box, and 3) a supporting frame. Of these components, the control system and the supporting frame need not be specifically designed for this program if use is made of the existing facilities at the Richmond Field Station of the University of California. Only the first item, namely, the box or sample container will need to be developed as a "new" piece of equipment. The following paragraphs describe separately the individual components of the complete set up.

2.2.2 Sample Container

Three main considerations must be kept in mind in designing the sample container:

- (1) The size of the container must be large enough so that rigid boundary effects are eliminated within a large enough portion of the sample so that meaningful data may be obtained from the tests.
- (2) The container must be rigid enough so that pressures of the required magnitude may be applied to the sample in order to study the liquefaction phenomenon under relatively high effective pressures and shear stresses.
- (3) The box design must provide and ensure undrained sample conditions during cyclic loading.

Previous theoretical investigations by Idriss, 1968, using finite element analysis, and experimental studies by Arango, 1971, have shown that both the rigid walls of the sample container and the geometry of the sample influence the dynamic response of the mass of soil being tested. The same studies indicate that an 8-inch high layer of sand with 1.0 vertical (V) on 3.0 horizontal (H) slopes at both ends, and a total length of 18 feet in the direction of the ground motion, would provide a 3- to 6-foot long central zone within which no end boundary effects are present; hence, providing a response equivalent to the free field conditions. Experimental studies with clays by Arango, 1971, also showed that in the direction transverse to the ground motion, the effects of the rigid container walls extend for a distance of about twice the height of the soil bank. A 5-foot wide shake box, in which an 8-inch high sand sample is placed, would provide a central zone of about 2.5 feet width, where again boundary effects are absent.

An 18-foot by 5-foot container would provide a relatively large central zone where free field response would be developed during dynamic testing. Fig. 2.2 shows the construction details of such a box. The container is intended to be used with pressures up to 50 psi. It would be provided with 6 roundway bearings attached to the bottom plate. These bearings slide on hardened steel round ways providing an almost frictionless system. The sand container would be fabricated of aluminum and would have inside dimensions of 210 by 56 inches. Sloping edge plates to support the thin rubber membrane are provided at each end of the box. The box would be actuated by a hydraulic ram mounted on a fixed base to the floor. The box accelerations would be measured with a Kistler servo-accelerometer which would directly feed the electrical signal back to the MTS control console.

CROSS-SECTIONAL DETAIL SHOWING SAMPLE AND RUBBER MEMBRANE INSTALLED

DETAIL ①: DUAL ROUNDWAY BEARING AND V-MOUNTING BLOCK

DETAIL ②: SINGLE ROUNDWAY BEARING AND MOUNTING BLOCK

Scale Inches: 0, 1", 2", 3"

FIG. 2.2 GENERAL DESIGN GUIDE LINES FOR THE PROPOSED 18' x 5' SHAKE TABLE CONTAINER

2.2.3 Control System

a. General

As in the case of the simple shear device, the closed-loop electro-hydraulic technique was considered to be the most suitable means of providing the cyclic loads for the shake box. The basis of the system and the description of the several components were presented at the beginning of this chapter in connection with the large simple shear device.

The main features of the existing available facilities at the University of California, Richmond Field Station, will be presented here in order to provide the required background for the discussion of their testing capability in connection with the recommended shake table experiments.

b. System 1

The main features of the system consist of a hydraulic power supply capable of delivering 40 gpm at 3000 psi. The 10-kip hydraulic actuator has an effective area of 5.4 sq. in., a $\pm$ 3-inch stroke, and is activated by a 60-gpm servo-valve. The control console is provided with all the required command modules as described in the text.

This system would be able to produce accelerations up to 1.5g in the shake box container and sample described above, at frequencies of 2.5 to 10 cps. The structural floor at the rock-fill testing facilities of the Experimental Field Station is adequate to provide the necessary reaction or support to the actuator.

All of the above components are available at the Field Station and all that is needed is to set up the installation and connect the various elements.

c. System 2

System 1 can be easily expanded to provide larger accelerations, if needed. A new 25-kip actuator with effective area

equal to 12.5 sq. in., and a $\pm$ 6-inch stroke would provide accelerations of the order of 1.0g at a frequency of 3.5 cps, or up to 3.0g at a higher frequency of 10 to 20 cps. The cost of this actuator and other accessory equipment is estimated to be of the order of \$1,000.

d. System 3

The set up described as System No. 2 may also be fed with a hydraulic power supply delivering 200 gpm at 3000 psi. Such a power supply is available in the Structural Dynamics Laboratory at the Richmond Field Station. By changing the actuator servo-valve to one having 170 gpm flow capacity, accelerations of the order of 3.8g, at a frequency of 4.5 cps or 0.8g at 1 cps may be obtained. The cost of the servo-valve and other required auxiliary equipment is estimated to be of the order of \$10,000.

2.2.4 Operation of the System

The pluvial compaction method mentioned in Chapter 1 offers the most convenient means of preparing large uniform sand layers at any required relative density.

The spreader box suggested for this investigation consists of the 1/3 cu. yd. capacity device used by Silver and Seed, 1969, available at the University of California. A detailed discussion on the design, construction, and calibration of the spreader has been presented by the same authors, 1969.

The sand would be placed inside the shake container in a dry condition. The required instrumentation, namely, accelerometers, pressure and displacement transducers would be placed within the sample at their proper locations as the sand is being placed. When placement has been completed, the excess material can be screeded off, using a metal blade which can slide along the inside angles welded to the sides of the container. A thin rubber membrane can then be placed on the sample and clamped to the side angles of the box, making a

water-tight connection. After ensuring a water-tight membrane fit, the container lid would be placed on the box and securely bolted to it.

The test specimen may then be saturated. It is expected that pre-saturation in a carbon dioxide atmosphere will facilitate this operation. Carbon dioxide gas is highly soluble in water, and therefore, if entrapped between soil grains during saturation, easily dissolves when slightly pressurized. Trapped air can likewise be dissolved as routinely accomplished in conventional triaxial apparatus, however, considerably higher pressures are required. When working with larger test specimens, it is more advantageous to achieve saturation at a minimum back pressure. This allows the sample to be tested over a higher range of effective confining pressures for the planned design capacity (50 psi) of the container. Following saturation and the application of an effective confining pressure, the sample is then ready for testing.

Motion of the container can be induced either by direct coupling of the box to the hydraulic actuator (for horizontal motion), or else by simply mounting it on the 20 by 20-foot shake table (for horizontal and vertical motions simultaneously) available at the Richmond Field Station. In either case, the acceleration within the free field zone of the test specimen would be measured, enabling the shear stresses to be computed. Shear strains would also be computed by observing the extensometers placed at various heights within the sample.

The capability of Control System 1 is such that the earthquake free field response of sand deposits equivalent to a depth up to 10 feet can be studied in the laboratory. Control System 2 would expand such capability to study the conditions in soil deposits up to 20 feet deep. Control System 3 will further increase the range to soil deposits equivalent to about 25 feet in depth.

2.2.5 Cost Estimate

The total estimated cost to design, build, and set up the installations for the shake table box, utilizing the existing facilities of the University of California at the Richmond Field Stations follows. It is considered likely that the experimental station has, at the present time, the required pressure transducers, accelerometers, LVDT's, amplifiers, and recording equipment which would be needed for such research.

SHAKE TABLE FACILITY-COST BREAKDOWN

- Item 1 - Design and construction of a container box for liquefaction studies, according to the general guidelines as presented in Fig. 2.2.
- Item 2 - Modification of the sand spreader box to be used in connection with the above container.
- Item 3 - Conditioning of the MTS control system to provide the range of performance as stated under Control System 1 in the text.
- Item 4 - Conditioning of the instrumentation and recording equipment needed for the research. Initial calibration tests to check the performance of the equipment.

Total estimated cost	\$45,000
Estimated completion time	6 months

NOTE: The Richmond Field Station is equipped with the necessary electronic amplifiers and recorders which are normally required in connection with the instrumentation needed for shake table tests. However, during the course of the research, the need for special measuring devices may arise. The above cost estimate provides for a contingency allowance should this need arise during the investigation.

CHAPTER 3

SUGGESTED APPROACH FOR FUTURE STUDIES

3.1 DISCUSSION AND RECOMMENDATIONS

Since reliable field data on soil liquefaction under seismic loading can only be obtained when strong motion earthquakes occur in well-instrumented areas, further concentrated study of this nature cannot be widely undertaken until these earthquakes take place. Therefore, further advances in our understanding of the liquefaction phenomenon, in the meantime, may need to continue to rely on laboratory methods of investigation.

For such investigations, two large scale devices, the shake table and a large scale simple shear device, have been studied as possible methods for evaluating liquefaction. It appears that both can develop valuable information for advancing the state of knowledge on liquefaction. Initial considerations earlier in the subcontract indicated that use of the simple shear device appeared to have certain advantages over the use of shake tables. However, as a result of recent work and our research on the subject, we believe that the major limitations of shake table tests have been largely resolved. It now appears that the shake table offers a better method for studying liquefaction than the large scale cyclic simple shear device for the following reasons:

- 1) Duplication of field conditions - Table 1.1, Chapter 1 was prepared with the purpose of comparing the conditions that would prevail under cyclic loading in a soil element within a field deposit of granular soil, and in a similar soil element tested in a large simple shear device or in a shake table. The relative advantages of the shake table option are apparent. The shake table test would closely duplicate field

conditions within the free-field area of the test specimens, a condition not achieved in the simple shear device. As stated in Chapter 1, the limitations inherent in the simple shear device would be minimized in the large-sized container. The extent of the improvement is difficult to evaluate. Basically, the shake table offers a testing method closer to the ideal conditions, namely duplication of field conditions.

- 2) Design of equipment - It is much more difficult to design and fabricate a test device with moving parts, especially side panels as in the suggested cyclic simple shear test. On the other hand, the design of the shake table container has been envisioned as a box which would either be hooked up to a hydraulic actuator and function as a shake table per se, or else it could be mounted directly on available shake tables whenever more than one degree of freedom motion is to be studied.
- 3) Cost of fabrication - As discussed in Chapter 2, the expense incurred in fabricating the simple shear device, and acquiring all the necessary electronic and hydraulic auxiliary equipment, is more than double the cost required to build the shake box container and adapt existing facilities, such as those at the University of California in Richmond, for testing purposes.
- 4) Flexibility of testing - Shake table tests can be performed under one directional loading as well as multi-directional loading. The simple shear device is limited to one directional loading only. When conducting research studies, flexibility of the testing facilities is highly desirable.

- 5) Experience with testing - Experience gained from previous shake table tests has been reported and thus can serve as a guide in sample preparation and testing. A new device, such as the proposed simple shear machine, would require initial experimentation to resolve unknown questions which may have already arisen and been answered in shake table studies, for example: sample preparation and instrumentation. It is again noted that equipment necessary to prepare shake table samples is known to be available at the University of California.

For the above reasons, the SW-AJA joint venture recommends that further research in the area of subsidence and liquefaction of granular soils under seismic loading be continued in the laboratory, following a comprehensive research program utilizing a shake table box container, as described in the previous paragraphs, and the existing testing facilities at the Richmond Field Station of the University of California.

A suggested scope of work and schedule of operations is discussed in the following sections of this chapter.

3.2 RECOMMENDED SCHEDULE OF OPERATIONS

The work schedule for the shake table studies may be subdivided into four different phases:

Phase 1 - Construction of the box container, according to the general guidelines explained in Chapter 2, and in Fig. 2.2. Adaptation of the existing laboratory facilities at the University of California for use in connection with the shake table testing program. The testing facilities are those identified as System 1 in Chapter 2, capable of reproducing free field conditions representing field

deposits up to a thickness equivalent to 10 feet.

- Phase 2 - Study of the basic variables known to influence the liquefaction and subsidence phenomena in granular deposits. These factors are discussed in the following section dealing with the required instrumentation.
- Phase 3 - Extension of the previous investigations to the observations of the behavior of sand specimens representative of deposits equivalent to thicknesses up to 20 feet. This phase would involve the adaptation of the shake table box container to the control system identified in Chapter 2 as System 2.
- Phase 4 - Scope of investigation as described under Phase 3, but at a lower frequency range. This phase, which may or may not be needed, would involve the adaptation of the shake table box container to the control system identified in Chapter 2 as System 3.
- Phase 5 - Study of the effects of two simultaneous (vertical and horizontal degree of freedom) motions upon liquefaction and subsidence development. This phase would be achieved by simply mounting the box container to the now operational 20 x 20-foot, two-degree-of-freedom table at the Richmond Field Station.

Simultaneous research programs with the small simple shear devices now available at several places may be in order during the above phases, in order to develop possible correlation factors between the results of the shake table tests and those in the simple shear tests. This research program would be defined as work advances.

3.3 SCOPE OF INVESTIGATIONS - INSTRUMENTATION

3.3.1 General

While the schedule of operations may be defined with a fair degree of certainty, the scope of work must be sufficiently flexible to adapt the investigation to the newer findings that will develop as the investigation advances. A general scope of work is, however, discussed in some detail in the following paragraphs.

Both laboratory investigations and observations of field performances have shown that the liquefaction potential of a soil deposit to earthquake motions depends on the characteristics of the soil, the initial stresses acting on the soil, and the characteristics of the earthquake involved. In addition to saturation, the significant factors include soil types, relative density or void ratio, initial confining pressure, intensity of ground shaking, and duration of ground shaking.

Some of the above factors have been the object of much attention in dealing with liquefaction, and controversies in explaining the phenomenon still remain. The effects of the initial stresses acting on the soil, the magnitude of the cyclic shear stresses, the duration of motion, and the effect of the relative density of the soil deposit upon the liquefaction potential, are undoubtedly some of the factors that must be considered first in the proposed research.

Subsequent stages of the investigation may be involved with other factors, such as the influence of soil type (gradation, amount of fines, angularity of sand grains); characteristics of the ground motion (amplitude of shear strains, effects of ground velocity and rate of shear on the seismic strength of sand); effects of the simultaneous loading in two directions (one horizontal and one vertical) on the development of the pore-water pressure and on the rate and propagation of liquefaction in a given soil deposit, etc.

3.3.2 Instrumentation

A complete laboratory study of the liquefaction and soil subsidence phenomena of granular soils, associated with cyclic loading, must be capable of measuring either directly or indirectly the above factors. The following paragraphs discuss the instrumentation required for an adequate evaluation of these variables.

a. Relative Density

Relative densities are important in that they should be consistently similar throughout each large scale specimen within a given test for each test series. Consistent placement densities may be achieved by employing a calibrated standard technique for each relative density required. A volume change device may be used to measure the volume of water expelled from the soil during the consolidation stage of the test so that the placement density may be properly corrected.

b. Shear Strain Measurements

The shear strain γ_{xy} is calculated as the horizontal displacement X of a point at the surface of the sand deposit divided by the height of the deposit H , that is, $\gamma_{xy} = X/H$. The horizontal displacement in the shake table test would be measured with specially designed LVDT's (Linearly Variable Differential Transformers) capable of measuring the displacement of a point on the surface of the sand layer, relative to other points within the sand mass or the box itself.

c. Shear Stresses

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 in the shake box container, with a height H , behaved as a rigid body and the maximum surface acceleration were A_{max} , the maximum shear stress τ_{max} on the soil element would be

determined by the expression:

$$\tau_{\max} = \gamma H \frac{A_{\max}}{g} \quad (3.1)$$

where γ is the total unit weight of the sand, H is the height of the layer, and $A_{\max}/g$ is the acceleration at level H in g 's.

By installing a series of accelerometers at various heights within the sand box, equation (3.1) could be evaluated and the resulting patterns of shear stresses within the whole deposit estimated.

The control system specified as System 1 in Chapter 2, and recommended as our Phase 1 within Chapter 3, is capable of inducing shear stresses up to 120 psf. Since the shake container is designed for pressures up to 50 psi, the ratio of the cyclic shear stress to the initial effective vertical stress for the test specimens could be chosen to vary between the values 0.10 and 0.30, which is the range of interest for actual earthquakes in the field. This equipment would duplicate field conditions of soil deposits varying between 5 and 10 feet thick. The control system designated as System 2 in Chapter 2 would increase the maximum generated shear stresses to a value of 240 psf, which, for the same range ratio of shear/effective initial vertical stress, would duplicate the soil conditions for soil deposits extending to depths between 10 and 20 feet.

d. Acceleration Measurements

In order to check the stress distribution within the free-field zone and control the shake container displacements, it would be necessary to install accelerometers. During sample preparation, acceleration transducers would be installed at selected depths and at the base of the container. The essential requirements of such accelerometers are that they be very small and have a unit weight close to that of the soil being tested so that the inertial effects of such embedded instruments are negligible. Further instrumentation details are discussed in the Appendix.

e. Confining Pressure Measurements

Regulated air pressure would act above the sealing membrane placed on top of the sand specimen. The container is designed for a maximum pressure of 50 psi. The carbon dioxide technique (See Subsection 2.2.6), which would be used for saturation, would require a relatively small amount of back pressure in order to ensure the full saturation of the sand specimen. It is estimated that 20 psi would be sufficient for this purpose, leaving the remaining 30 psi to be used as effective initial confining pressure. Laboratory air pressure regulators, valves, and gages of usual type and specifications would be used during the present investigation. The laboratory at Richmond is equipped with a 100+ psi air power supply.

f. Pore-Water Pressure Measurements

Pore-water pressure indicators would be carefully installed at the selected depths and locations during sample preparation. It is an essential requirement that the devices be capable of measuring dynamic pressures with zero or near zero volume change. Any instrument which is to be placed within the soil should be as small as possible while still maintaining the desired accuracy. A discussion of the available pore-water pressure transducers is contained in the Appendix.

g. Vertical Deformation

Linearly variable differential transformers may be mounted above the sample so that their cores move vertically with the sand bed providing the necessary information to calculate sample compression after any number of stress or strain applications.

3.4 SUMMARY

This report has presented a study which included the design of a large scale simple shear device and a large shake table container; a discussion of their relative merits and

potentials was also presented. From the study, it was concluded that the shake table approach was feasible and at this time appears more advantageous than embarking into the development of a new large scale simple shear device. The results to be obtained from the proposed research program on the shake table should provide meaningful in-depth answers as to the behavior of both unsaturated and saturated granular soil under cyclic loading, such as occurs in the field under earthquakes. It is expected that the close similitude of conditions in the shake table and in the field will provide useful guides and insight into the causes and mechanism of subsidence and liquefaction phenomena of granular deposits.

CHAPTER 4

MODELING OF GRAVELLY AND SILTY SOILS

The results of investigations of gravelly and silty soils are reported in this chapter. Less work has been done on the development of models to represent these two soil types than has been done previously on models for clay and sand soils (SW-AJA, 1971). Section 4.1 briefly identifies the model developed to represent gravelly soils; and in Section 4.2, the response of a reference site is described as determined using this model. The soil profile of this selected site is shown in Fig. 4.1. and represents the average soil conditions at the State Building in San Francisco. This site has been considered as a typical sandy soil site in previous seismic response calculations. Its response when treated predominantly as a sandy site, Fig. 4.1.a, was determined and is represented as Case 1. Additional calculations reported in Section 4.2 are for the modification of this site, Fig. 4.1.b, identified as Case 2, in which materials 6 through 9 are represented by the model for gravelly soils described in Section 4.1. The finite element arrangement used for these two cases is also represented in this same figure.

The results of analyzing two hypothetical site profiles, intended to represent silty soils, are reported in Section 4.3. Because few data are available for silts, as differentiated from sands and clays, no specific model for silty soils was developed. Silty soils have generally been treated in the past either as clays or sands. Hence, the influence of substituting clays for sands in two hypothetical sites derived from the San Francisco State Building site soil profile was examined to determine if a detailed study of silts would be necessary. These two hypothetical sites, a clay/sand site identified as Case 3, and a sand/clay site identified as Case 4, are represented in Fig. 4.2. The seismic input at the base of the soil column

FIG. 4.1 SOIL PROFILE AND FINITE ELEMENT MODELS FOR SAND AND SAND/GRAVEL SITES

FIG. 4.2 SOIL PROFILE AND FINITE ELEMENT MODELS FOR CLAY/SAND AND SAND/CLAY SITES

for all four cases was 0.8 times the Golden Gate Park rock outcrop record of the S80E component of the 1957 San Francisco earthquake (peak acceleration 0.1 g).

Comparisons between surface motions, calculated for one-dimensional finite element models of hypothetical sand/gravel, clay/sand and sand/clay sites, and those calculated for an all-sand representation of the San Francisco State Building site soil profile, are summarized in Section 4.4.

4.1 MODEL OF GRAVELLY SOILS

Limited preliminary data from a study of gravelly soils (SW-AJA, 1972) suggested that the non-linear hysteretic model developed for sands (SW-AJA, 1971) would also be applicable to the gravelly soils if model parameters were adjusted to reflect the higher values of shear modulus observed. The procedures used to generate the hysteretic properties of a gravelly soil are described below.

To determine the virgin loading path for gravel, the modulus curve of dense sand and sandy gravel (Southern California) in Fig. 4.3 was employed. The secant moduli were obtained for four different strain levels (viz., 1×10^{-5} , 3×10^{-5} , 1×10^{-4} , and 3×10^{-4} in/in) and plotted in stress-strain space, Fig. 4.4. The dotted lines enveloping these curves constitute a piece-wise linear virgin loading curve and define the stiffnesses K_1 , K_2 , K_3 , and K_4 listed in Table 4.1.

The reloading and unloading paths are assumed to be parabolic curves. The hysteresis loops enclosed by two parabolic curves under an unload-reload cycle can be determined once the semi-major axis "a" and semi-minor axis "b" of the parabola, Fig. 4.5, are known. For the symmetric loops considered, the value of "a" is a function of strain level, and value "b" is obtained from the damping properties attributed to the soil. Here it was assumed that the energy dissipation per cycle, at a given strain level for gravel, is the same as for sand, Fig. 4.5. Values for "a" and "b" corresponding to strain levels of 1×10^{-4} , and 3×10^{-4}

FIG. 4.3 MODULI DETERMINATIONS FOR GRAVELLY SOILS
 (FROM SEED AND IDRISS 1970b)

FIG. 4.4 VIRGIN LOADING STIFFNESSES FOR GRAVEL

TABLE 4.1 HYSTERETIC PARAMETERS FOR GRAVELS

Soil Layer No.	Depth, ft	a_1	b_1	γ_1 , IN./IN.	a_2	b_2	γ_2 , IN./IN.	K_1 , PSI $\gamma \leq 1 \times 10^{-5}$	K_2 , PSI $\gamma \leq 3 \times 10^{-5}$	K_3 , PSI $\gamma \leq 10^{-4}$	K_4 , PSI $\gamma \leq 3 \times 10^{-4}$	K_5 , PSI $\gamma > 3 \times 10^{-4}$
6	110	1.27×10^{-4}	0.782×10^{-5}	10^{-4}	3.432×10^{-4}	3.35×10^{-5}	3×10^{-4}	1.111×10^5	0.99×10^5	0.794×10^5	4.938×10^4	1.52×10^4
7	130	1.27×10^{-4}	0.782×10^{-5}	10^{-4}	3.432×10^{-4}	3.35×10^{-5}	3×10^{-4}	1.205×10^5	1.078×10^5	0.859×10^5	5.365×10^4	1.65×10^4
8	150	1.27×10^{-4}	0.783×10^{-5}	10^{-4}	3.432×10^{-4}	3.35×10^{-5}	3×10^{-4}	1.291×10^5	1.205×10^5	0.926×10^5	5.753×10^4	1.78×10^4
9	170	1.27×10^{-4}	0.784×10^{-5}	10^{-4}	3.432×10^{-4}	3.35×10^{-5}	3×10^{-4}	1.375×10^5	1.225×10^5	0.986×10^5	6.11×10^4	1.89×10^4

FIG. 4.5 HYPOTHETICAL SHEAR STRESS-STRAIN CURVES FOR DYNAMIC CYCLIC SHEAR TESTS OF SANDS

in/in are listed in Table 4.1. Calculated hysteresis loops for the four previously specified strain levels are shown in Fig. 4.6.

The percent of critical damping for the gravel model at different strain levels was evaluated on the basis of the ratio of energy dissipated per cycle to total work done. The resulting values are compared with the average data for sand in Fig. 4.7.

4.2 EFFECTS OF GRAVEL LAYERS ON THE RESPONSE OF STATE BUILDING SITE

The hysteretic properties of a gravelly soil, described in Section 4.1, were utilized in calculating the response of a modified model of the State Building soil profile to examine the effect of gravelly layers on ground surface response to bedrock earthquake motions.

The properties of the sand layers are those reported in SW-AJA, 1971. In Case 1 the soil column was composed entirely of sands as shown in Fig. 4.1.a. In the soil column used for Case 2, shown in Fig. 4.1.b, materials 6 through 9 are represented by the model for gravelly soils described in Section 4.1. These gravel layers 6 through 9 correspond to elements 2, 3, 4, and 5 in the finite element representation. The soil in finite elements 1 and 6 through 13, as shown in Fig. 4.1.b, is the same as that used in Case 1.

An integration time step of 0.003 seconds was used. The resulting surface motions for the two cases are compared in Figs. 4.8, 4.9, and 4.10. Acceleration and pseudo-velocity response spectra for the two cases are compared in Figs. 4.11 and 4.12.

In this particular example, the presence of the gravel seems to have little effect on the peak values of acceleration (Fig. 4.8); however, there are differences in frequency content and phasing which affect the velocity and displacement. The increase in soil stiffness (modulus) tends to shift the

FIG. 4.6 HYPOTHETICAL HYSTERESIS LOOPS FOR GRAVELS

FIG. 4.7 COMPARISON OF DAMPING FACTOR FOR GRAVEL MODEL WITH AVERAGED DATA FOR SANDS

(a) CASE 1

(b) CASE 2

FIG. 4.8 COMPARISON OF GROUND SURFACE ACCELERATION FOR SAND AND SAND/GRAVEL SITES

(a) CASE 1

(b) CASE 2

FIG. 4.9 COMPARISON OF GROUND SURFACE VELOCITIES FOR SAND AND SAND/GRAVEL SITES

(a) CASE 1

(b) CASE 2

FIG. 4.10 COMPARISON OF GROUND SURFACE DISPLACEMENTS FOR SAND AND SAND/GRAVEL SITES

FIG. 4.11 COMPARISON OF GROUND SURFACE ACCELERATION RESPONSE SPECTRA FOR SAND AND SAND/GRAVEL SITES

FIG. 4.12 COMPARISON OF GROUND SURFACE PSEUDO-VELOCITY RESPONSE SPECTRA FOR SAND AND SAND/GRAVEL SITES

frequency content of the sand/gravel site toward higher values. Although surface velocities (Fig. 4.9) exhibit the same general pattern, the peak value in the sand/gravel site (2.16 in/sec) is 16 percent higher than that in the sand (1.86 in/sec), while the displacements (Fig. 4.10) show the opposite trend, 0.37 inches for sand versus 0.28 inches for sand/gravel. The difference in frequency content of the pseudo-velocity response spectra shown in Fig. 4.12 offers a partial explanation. The stronger content of the sand/gravel spectrum for periods near 0.4 seconds, contrasted with the strong content near 0.9 seconds in the sand site spectrum, would produce the observed trends.

Shear stress time histories at element 3, Fig. 4.13, and element 6, Fig. 4.14, indicate that although frequency contents are altered, as noted previously in the surface motion, there is no significant change in stress magnitude. At element 3 (near the center of the gravel or "sand with some gravel") the peak stress magnitudes are in nearly the same proportion as the peak surface velocities, as noted in the following:

$$\frac{\tau_{\text{sand}}}{\tau_{\text{sand/gravel}}} = 3.0/3.5 = 0.86$$

vs

$$\frac{V_{\text{sand}}}{V_{\text{sand/gravel}}} = 1.86/2.16 = 0.85$$

At element 6, the first element above the gravel elements, the difference in peak shear stresses is considerably less, indicating that the upper sand layers tend to transmit the different motions from the lower layers in the two model sites in nearly the same way, i.e., the transmission properties of the upper 100 feet of sand were not significantly altered by the differences introduced in the bottom 100 feet.

(a) CASE 1

(b) CASE 2

FIG. 4.13 COMPARISON OF SHEAR STRESSES IN SAND AND SAND/GRAVEL SITES: ELEMENT 3

(a) CASE 1

(b) CASE 2

FIG. 4.14 COMPARISON OF SHEAR STRESSES IN SAND AND SAND/GRAVEL SITES: ELEMENT 6

The overall effect of substituting dense (high modulus) gravels for the sands in elements 2 through 5 is probably more easily seen in the comparison of spectra ratios (approximate amplification spectra), shown in Fig. 4.15. Differences in the period ranges, 0.3 to 0.6 seconds and 0.8 to 1.4 seconds, reflect those already seen in comparisons of response spectra and correspond with the two secondary peaks, A and B, of the pseudo-velocity response spectrum obtained from the input motion and shown in Fig. 4.16.

4.3 EFFECTS OF SUBSTITUTING CLAY LAYERS FOR SANDS IN THE STATE BUILDING SOIL PROFILE

Two further modifications of the State Building site were examined to determine whether the sand/silt and silt/clay differentiations are necessary in sites with mixed soils. These modifications are identified as Cases 3 and 4 and are represented in Fig. 4.2. In Case 3, the uppermost 100 feet of medium dense sand at the State Building site sand model (Case 1) were replaced by medium stiff and stiff clays, with ultimate strengths of 1500 psf in the first 50 feet and 3000 psf in the second 50 feet. The soils below 100 feet were defined as sands with the properties used in Case 1. In Case 4, the bottom 100 feet of sand were replaced by stiff to very stiff clays having ultimate strengths of 3000 psf (100 feet to 142 feet) and 4000 psf (142 feet to 200 feet). The upper 100 feet of soil was sand, as defined in Case 1. A family of hysteresis loops for the 3000 psf strength clay is shown in Fig. 4.17.

The finite element mesh used in Cases 3 and 4 is identical to that used in Cases 1 and 2. The integration time step, however, was increased to 0.008 seconds since the shear velocity in the sands and clays is considerably less than that in the gravel. Resolution of signals whose period is greater than 0.1 second (the shortest period considered in preparing response spectra here) is satisfactory in either case; the difference, although capable of producing visible differences in the fine

FIG. 4.15 COMPARISON OF PSEUDO-VELOCITY RESPONSE SPECTRA RATIOS (APPROXIMATE AMPLIFICATION SPECTRA) FOR THE SAND AND SAND/GRAVEL SITES

FIG. 4.16 RESPONSE SPECTRUM DERIVED FROM THE BASE MOTION INPUT RECORD

FIG. 4.17 HYPOTHETICAL HYSTERESIS LOOPS FOR INTERMEDIATE STRENGTH (STIFF) CLAY

details of time histories, should have no effect on response spectra.

The time history plots of surface acceleration, velocity, and displacement obtained for the sand (Case 1), clay/sand (Case 3), and sand/clay (Case 4) site models are compared in Figs. 4.18, 4.19, and 4.20, respectively. The peak accelerations are 8.3, 7.4 and 7.8 percent of gravity. The peak velocities are 1.84, 1.76, and 2.22 in/sec, respectively; and the peak displacements are 0.369, 0.365, and 0.365 inches, suggesting that small changes in the higher frequency details are the source of the differences.

The same observation holds for stress amplitudes in element 3 (see Fig. 4.21) and element 6 (see Fig. 4.22). The differences in frequency content can be observed in the acceleration and pseudo-velocity response spectra in Fig. 4.23 and in the pseudo-velocity response spectra ratios (approximate amplification spectra) in Fig. 4.24. The spectra suggest the importance of the near-surface material in determining the characteristics of the surface motion. The only significant deviations from the sand site (Case 1) spectrum occur for periods between 0.3 and 0.6 seconds for the clay/sand site model (Case 3) response.

The similarity of the motion calculated in Cases 1, 3, and 4 is not entirely unexpected. The sites treated in Cases 3 and 4 have material properties identical to those used in Case 1 over either the upper or lower half of the soil profile. Where clay has been substituted for sand, the properties (shear moduli), except near the surface where confining pressures are low and the modulus of the sand approaches zero, are quite similar. Although the strength characteristics of the sand and clays are quite different (the sand strength arising from confining pressure and internal friction, while the clay strength is due primarily to cohesion), stress levels, almost without exception, are sufficiently low that no permanent (plastic) deformations are expected. Therefore, as previously noted,

(a) CASE 1

(b) CASE 3

(c) CASE 4

FIG. 4.18 COMPARISON OF GROUND SURFACE ACCELERATION FOR SAND CLAY/SAND AND SAND/CLAY SITES

(a) CASE 1

(b) CASE 3

(c) CASE 4

FIG. 4.19 COMPARISON OF GROUND SURFACE VELOCITIES IN SAND, CLAY/SAND AND SAND/CLAY SITES

(a) CASE 1

(b) CASE 3

(c) CASE 4

FIG. 4.20 COMPARISON OF GROUND SURFACE DISPLACEMENTS FOR SAND CLAY/SAND AND SAND/CLAY SITES

(a) CASE 1

(b) CASE 3

(c) CASE 4

FIG. 4.21 COMPARISON OF SHEAR STRESSES IN SAND, CLAY/SAND AND SAND/CLAY SITES: ELEMENT 3

(a) CASE 1

(b) CASE 3

(c) CASE 4

FIG. 4.22 COMPARISON OF SHEAR STRESSES IN SAND, CLAY/SAND AND SAND/CLAY SITES: ELEMENT 6

(a) ACCELERATION

(b) VELOCITY

FIG. 4.23 COMPARISON OF GROUND SURFACE ACCELERATION AND PSEUDO-VELOCITY RESPONSE SPECTRA FOR SAND, CLAY/SAND, AND SAND/CLAY SITES

FIG. 4.24 COMPARISON OF PSEUDO-VELOCITY RESPONSE SPECTRA RATIOS (APPROXIMATE AMPLIFICATION SPECTRA) FOR THE SAND, CLAY/SAND, AND SAND/CLAY SITES

the sand/clay differentiation in soil properties appears to be significant only very near the surface, and the sand/silt and silt/clay differences represent unnecessary refinements in this case.

4.4 SUMMARY AND CONCLUSIONS

Surface motion and internal stresses in hypothetical sand, sand/gravel, clay/sand and sand/clay, sites, subjected to identical base motion, were compared to determine the effect of sand/gravel differentiation, and to determine the necessity of sand/silt or silt/clay differentiation. All soils considered were treated as non-linear hysteretic elastic-plastic materials, using the hysteretic model reported previously (Chapter 2, SW-AJA, 1971). The gravel model used was based on extrapolations from low strain level in situ test data, and the observed behavior of sands (SW-AJA, 1972). It represents a nearly three-fold increase in stiffness (modulus) over sand. The clays, whose strength varied from 1500 psf near the surface to 3000 psf at intermediate depths, and reached 4000 psf for depths below 142 feet, had properties comparable to those for sands, except in the near-surface region where a lack of confining pressure produced some differences. For the input levels considered (0.10g at the base of the soil columns), differences between the response observed in the sand, clay/sand and sand/clay models were, with one exception, less than 10 percent. The peak surface velocity in the clay/sand site model exceeded that in the sand site by nearly 20 percent, but peak acceleration, displacements, and shear stresses were within the ± 10 percent band. Differences in response spectra were also relatively small and limited to two narrow frequency bands which coincided with secondary peaks of the response spectrum for the input motion.

On the basis of the comparisons reported here, it appears that sand/silt and silt/clay differentiation may sometimes be unnecessary, even in the near-surface region. The

current practice of treating silty soils either as sands or as clays may be adequate in many cases for the response regime of nearly elastic behavior. At higher levels of excitation, differences in the strength characteristics of cohesive and cohesionless soils might become important, necessitating more refined differentiations between soil types.

The gravel model was selected to represent an extreme case in sand/gravel contrast (a three-fold increase in stiffness (modulus) for the strain levels of interest). The substitution of such a stiff (high modulus) material in the lowermost 100 feet of the soil profile produced considerably less effect on the surface response than was anticipated. Peak surface acceleration differed by only 7 percent, the sand/gravel site having the higher value. Peak surface velocities and displacements showed larger differences, the sand/gravel site velocity exceeding that in the sand by 16 percent; the displacement falling 24 percent below that in the sand.

For the particular site and base motions considered, the calculations reported here suggest a relative insensitivity of the ground surface response to changes in the representation of a soil profile, and further suggest that rather broad limits of material property measurement accuracy may sometimes provide acceptable results. However, to investigate response sensitivity to changes in excitation characteristics, the ability of the modeling and computational procedures to predict the response of soil deposits to known earthquake excitation, and to establish property measurement accuracy requirements, a much more extensive program of calculations supported by field and laboratory measurement programs will be required.

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APPENDIX A

INSTRUMENTATION

APPENDIX A
INSTRUMENTATION

A-1 DISPLACEMENT TRANSDUCERS

The most common type of displacement transducer presently being used in shake table or simple shear studies is the linearly variable differential transformer (LVDT). An LVDT consists essentially of a transformer with one primary and two secondary coils, and a movable iron core. As the core moves from its central position, a voltage differential is produced in the secondary coils, which is directly proportional to displacement. Calibration of the LVDT is accomplished by attaching a micrometer to the extension of the core. LVDT's may be used to measure either vertical or horizontal displacements.

Soil strain gages can also be used to measure relative displacement between two points in a soil mass. The gage system consists of a pair of rugged disk-shaped sensors, embedded in the soil in near parallel and coaxial orientation, separated by a distance over which the strain is to be averaged. The coupling is extremely sensitive to axial distance between the sensors, while the effects of rotational or transverse movements are only second order. Output from the sensors is amplified and recorded by a suitable electronic instrument.

Displacements may also be measured using an optical displacement transducer. This type of instrument is used when it is not possible to use a regular LVDT. The cost of this system is relatively high. Displacements may also be measured by integrating the output from a velocity or acceleration transducer. Integration of this output may be conducted through the use of electronic circuitry; however, the output from the transducer must be relatively accurate as the errors inherent in the basic measurements are magnified during the integration process.

A-2 ACCELEROMETERS

A number of different types of accelerometers are available; each operating on slightly different principles. The following is a brief summary of some of the common types of accelerometers presently available.

a. Piezo-Electric Crystal

Materials such as quartz and Rochelle Salt have the property of developing an electric charge which is directly proportional to the pressure applied to a crystal of the material. Thus a piezo-electric accelerometer consists of a piezo-electric crystal mounted in a base and wired to a cathode follower. The advantages of piezo-electric accelerometers are that they are small, light and rugged, and have a high frequency response (up to about 500 cps). A disadvantage of this type of accelerometer is that the output is non-linear and low-powered. Thus the sensitivity of the instrument is affected by the cable length. Overall accuracy is in the order of 2-5 percent and the cost per instrument is in the order of \$50 to \$500.

b. Strain Gauge

In this instrument, an acceleration-sensitive mass is attached to a wire which changes resistance in proportion to the applied stress. The resistance is measured with a Wheatstone bridge. Strain gage accelerometers are small, light, and rugged and thus considered suitable for use in this study. Disadvantages of the instrument include non-linearity of the output signal, and the requirement of a separate Wheatstone bridge to operate the instrument. Overall accuracy is in the order of 0.5 percent and the cost per instrument is in the range of \$250 to \$400.

c. Servo Accelerometer

There are two types of servo force-balance accelerometers: 1) the pendulum type, having an unbalanced pivoting mass with angular displacement, and 2) the non-pendulum type, having a mass which is displaced linearly. Both types operate on similar principles.

The pendulum type of accelerometer, which is manufactured by the Systron-Donner Corporation and others, operates when an applied acceleration produces a torque on a pivoting mass within the instrument. A position detector then converts the deflection of the mass to a signal that is transmitted to an electromagnetic coil, which produces a restoring torque equal and opposite to the original torque produced by the acceleration. The reaction time of the system is extremely fast so that the actual deflection of the pendulum is minute and thus the instrument is in effect "null" operating. The output of the accelerometer is a voltage developed by the restoring current within the restoring coil circuit. The voltage is directly proportional to the applied acceleration, thus producing a linear relationship throughout the operating range.

The non-pendulum type of accelerometer, as manufactured by Kistler and others, utilizes an electronic restoring system similar to that used in the pendulum accelerometer. The major difference, however, is that the acceleration-sensitive mass moves along a linear axis in the non-pendulum type, rather than along an angular path as in the pendulum type.

An advantage of the servo force-balance accelerometer is that its sensing axis essentially does not pivot or move. Thus, as long as the mounting unit itself does not move off its original axis, accelerations will be recorded along a single axis for all levels of acceleration. Also, the instrument is small and very accurate. As an example, the Kistler 505A/515 servo-accelerometer used by Kovacs, Silver, and Arango at the University of California, has an overall accuracy of about 0.01 percent

including linearity and hysteresis, which represents 0.005g. It also has a flat frequency response up to 500 cps and a full scale range of $\pm 50g$. Other relevant properties of these accelerometers follow: CEC Model 4-274-001, weight 6 gm, unit weight about 450 pcf. Kistler Model 808A, weight 20 gm, unit weight about 430 pcf. Metrix Model 502, weight 6 gm, unit weight about 130 pcf. Wilcoxon Model 127, weight 1.75 gm, unit weight about 185 pcf.

Because of this accuracy, the acceleration information may be integrated electronically to obtain velocities, and integrated a second time to obtain displacements. Cross-axis sensitivity is also very good so that only accelerations in a single plane are measured. Costs for servo force-displacement accelerometers are generally in the range of \$450 to \$1000 although some models, in particular those used in aerospace work, cost considerably more.

A-3 PORE-WATER PRESSURE MEASUREMENTS

Pore-water pressure measurements have been taken in shake table studies by measuring the height of water in standpipes connected to the soil-box boundary, and by measuring the water pressure by use of a pressure cell at the same boundary. Because of the volume change of water necessary to obtain pore-water pressures by means of an open standpipe, this measurement device is generally not feasible for use in recording pore-water pressures, except for perhaps taking initial readings before placing the apparatus in motion.

The measurement of pore-water pressures within the soil mass presents a difficult problem. Any alien material placed within the soil mass may affect the response of the soil mass and may cause liquefaction to occur at a different time or location than would otherwise occur. Also, the presence of the alien object may cause liquefaction to initiate at the object and thus the pore-pressure readings obtained would be

different from those existing in other corresponding locations of the soil mass.

It is therefore necessary that any pore-pressure indicator placed within a soil mass should be as small as possible, and yet have sufficient contact area so that surface tension and other forces do not affect the pressure readings.

Pressure transducers are available in sizes down to about 0.2 inch diameter by 0.1 inch high. Generally, a voltage is applied across the transducer and the output is recorded as a smaller voltage which is proportional to the pressure applied to the face of the transducer. At least three wires are necessary to connect the transducer within the soil mass to the control panel.

The sensitive face of the pressure transducer should be in contact only with water. Therefore, it is necessary to construct a shield, possibly in the form of a porous stone, near the face of a transducer.