27 Apr 1981, 10:30 am - 1:00 pm

# Load, Deformation and Strength Behavior of Soils under Dynamic Loadings

Marshall L. Silver

*University of Illinois, Chicago, Illinois*

Proceedings: First International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, April 26 - May 3, 1981, St. Louis, Missouri

# Load, Deformation and Strength Behavior of Soils under Dynamic Loadings

**Marshall L. Silver**
**University of Illinois, Chicago, Illinois**

SYNOPSIS The state of the art is summarized for the evaluation of the stress, strain and strength properties of soils in terms of appropriate test equipment, test procedures and the presentation of test results in both the laboratory and the field. Different testing requirements for measuring soil properties for 1) design and analysis problems and 2) for constitutive property modeling are compared and recommendations on minimum test result reporting requirements are given. In addition, methods for overcoming equipment and test procedure limitations are presented.

The importance of combining field and laboratory test results is stressed and ways to make more extensive use of geophysical test measurements to obtain insitu soil properties are summarized. On a site specific basis, it appears that geophysical test results may correlate well with many soil index properties and measures of insitu soil dynamic properties. Thus, much useful site information may be obtained by combining a limited geophysical test program and a more extensive traditional site investigation program.

## INTRODUCTION

It is a great pleasure and honor to be asked to prepare a state of the art report on load, deformation and strength behavior of soils under dynamic loading. In the beginning of this effort it quickly became apparent that three methods could be used to prepare this report:

1. Summarize dynamic soil behavior by cataloging and tabulating test results reported in the literature for various types of soils.

2. Summarize what is known about the dynamic behavior of soils and propose a constitutive relationship or model to analytically describe this behavior.

3. Present guidance in the selection of test procedures and appropriate data from the laboratory, the field or from the literature for analysis, design and for the development of constitutive relationships.

A tabulation of available data is very useful to the profession. Excellent and useful summaries have been prepared by Seed and Idriss (1970), Hardin and Drnevich (1972a and b), Ferrito, et al (1979) and others.

A good summary of available data allows practitioners involved in analysis and design to select appropriate dynamic soil property values from the published literature for use in their particular problem. A good summary of available data also gives the theoritician and researcher insight into soil behavior and experimental values useful for the development of constitutive models.

Unfortunately, most of the published data in the literature is unusable for both purposes. Published literature describing the results of cyclic or dynamic laboratory and field tests is almost always lacking in sufficient information on 1) index properties values, 2) test procedures, 3) specimen preparation methods, 4) the effect of the number of loading cycles on dynamic behavior and pore pressure response and 5) the experimental state of stress(particularly for field tests). Without this information, the practitioner is unable to make a meaningful comparison between measured dynamic soil properties reported in the literature and the estimated dynamic soil properties of his project soils. Similarly, without this information the researcher has incomplete data on which to base or to test his constitutive model.

Well qualified investigators have used the second method for preparing a state of the art report and have summarized what is known about the dynamic behavior of soils (Yoshimi, et al, 1978). Further, a number of investigators have proposed constitutive models to describe dynamic soil behavior as will be described subsequently. It would seem of little useful purpose to add a description of the author's favorite constitutive relationship here in this state of the art.

Because of the incomplete nature of the published data on dynamic soil behavior, because of the excellent summaries of dynamic soil behavior already published and because of the number of constitutive relationships already proposed, the third approach which provides guidance in selecting data from the literature seems a better method for preparing a state of the art report on the subject of dynamic soil behavior.

To be useful, such a state of the art should describe the advantages and disadvantages of the various types of laboratory and field test procedures, evaluate the state of stress which is imposed in each type of test, explain experimental problems that can influence the reported test results, and make suggestions for minimum data requirements in order to make published data useful in analysis and design problems. Further, it must be remembered that both laboratory and field techniques may be used to provide data on the dynamic load, deformation and strength behavior of soils. Thus, this state of the art report on the subject will include an evaluation of both field and laboratory dynamic test methods and test results.

With these goals in mind, the following pages describe recent advances in the development of experimental dynamic test methods, present requirements for the reporting of dynamic soil test results and critically describe how to evaluate the usefulness of published literature describing dynamic stress-strain and strength properties of soils. This discussion is intended to provide the practicing engineer with guidance in the selection of data from the literature useful in preliminary evaluation of soil-structure interaction problems and soil stability problems. This discussion is also intended to help the researcher in selecting data for constitutive relationships. Further, it is hoped that the criteria described in this paper will help to improve the quality of experimental data published in the literature so that the data will be more complete and thus more helpful to the profession.

## BACKGROUND

### Existing State of the Art Reports

A number of excellent state of the art reports has been prepared in the last few years that may be used to help evaluate load, deformation strength behavior of soils under dynamic loads. An annotated list of many of these state of the art reports is presented in Table1.

Over 1400 references are included in the state of the art papers described in Table 1. It is the goal of this report to draw upon the information and conclusions provided in these papers to provide guidance in ways to evaluate and measure dynamic load, deformation and strength behavior of soils.

### Laboratory Testing Versus Field Testing

As described previously, both laboratory and field tests are available for measuring load deformation and strength behavior of soils under dynamic loads. A number of the elements involved in field testing are common to laboratory testing. Thus, it is desirable to discuss the features of both field and laboratory tests together before preparing a detailed description of the state of the art for both types of testing.

The relative advantages and disadvantages of laboratory testing and field testing are well known (Woods, 1978). Advantages of field testing are that a large mass of soil is studied and in some cases sample disturbance can be minimized. Disadvantages of field testing are difficulty in controlling the boundary conditions of the test and the small strain levels that can generally be developed. Advantages of laboratory testing are the ease with which test parameters can be varied and the ability to define boundary conditions of the test. Disadvantage of laboratory testing include disturbance caused by sampling required to obtain representative field samples for laboratory testing.

Thus, it is clear that the advantages and disadvantages of field testing are strongly balanced by the disadvantages and advantages of laboratory testing. Therefore by combining laboratory and field testing in the same experimental program more information can be obtained than if only laboratory or only field testing is used.

### Non-Linear Cycle Dependent Stress-Strain Behavior.

Both laboratory testing and field testing must model the non-linear, hysteretic, stress-strain behavior of soils. Moreover, these hysteretic properties also change with increasing numbers of loading cycles. A number of simplifications have been used to represent this complicated soil stress-strain behavior as shown in Figure 1. Once an appropriate stress-strain representation has been chosen (Figure 1c), it is necessary to model the effect of strain level on properties (Figure 1a). At low strain values, modulus values are high and damping values (proportional to the size of the hysteresis loop) are low. On the other hand, for high strain values, modulus values decrease and damping values increase.

The effect of number of loading cycles on stress-strain behavior is shown in Figure (1b) where it may be seen that for dry sand, modulus values increase and damping values decrease with increasing numbers of cycles (Silver and Seed, 1971). On the other hand for saturated sands and clays, modulus values decrease and damping values increase with increasing numbers of cycles (Silver and Park, 1976). In the worst case, with increasing number of cycles, the pore pressure can rise to values equivalent to the confining pressure and the soil can loose all strength. This is commonly called liquefaction and can result in the development of large strains.

### Soil Behavior Testing

Historically, there have always been two classes of results from static tests or dynamic tests performed either in the laboratory or in the field. The first class of test results gives the engineer basic information for analysis and design. Such test results may not be an exact representation of insitu soil behavior. However, these tests do provide material property values which, when combined with experience, give design values useful in the analysis and design of soil structures and foundation systems. For example, the direct shear test provided one of the earliest methods of determining soil behavior and soil strength. In the early years of geotechnical engineering practice, engineers confidently used the results of direct shear tests for the analysis of many soil problems. With time, the profession started to learn more about the limitations of the direct shear test and the test lost favor. Recent work, however, has brought the direct shear test back into repute and today it is a popular item in the soils laboratory where it is being used to study the ultimate or residual strength of soils.

The goal of a test like the direct shear test should be to obtain design information. The test does not (and often can not) exactly match field conditions. Rather, test results should be reproducible between operators and laboratories. By combining reproducible test results and the results of field case history studies meaningful design procedures can be developed. The experimental value of the results obtained from these soil property tests is not as important as the ability to reproduce test results given the same input parameters. We should be concerned with the goal of obtaining reproducible test results in both the field and in the laboratory which are useful as index values of soil behavior.

Thus, the need in soil behavior testing in both laboratory and field studies is to prepare meaningful and adequate test procedures that can be followed to help ensure that test results are reproducible between various laboratories and operators. This has been done for the cyclic triaxial strength test (Silver, 1978), and for the resonant column test (Drnevich, 1978). Additional standardization of insitu tests has been described by the Corps of Engineers (1980). More effort in this area for other tests is required.

### Constitutive Behavior Testing

A second class of experiments performed both in the laboratory and in the field are to obtain experimental soil property values useful for the development of constitutive relationships. Such tests are best developed using guidance from the constitutive relationships to aid in the design of the test technique. Nonetheless, tests useful in providing constitutive relationships for soils must be much better than tests which provide design values. It is meaningless to use poorly measured soil properties in a sophisticated constitutive relationship and it is embarrassing to find that a constitutive relationship accurately predicts experimental soil behavior which subsequent evaluation shows to be incorrect.

Thus the goal of constitutive behavior testing is to understand the test. Therefore the following minimum criteria is required to obtain adequate values of soil behavior for the development of constitutive relationships:

1. The boundary conditions of the experiment must be understood.

2. The limitations of the test equipment must be understood. This requires an evaluation and measurement of equipment friction and compliance.

3. The limitations of the test procedures must be understood. This includes an understanding of the effect of specimen preparation techniques, saturation methods and consolidation procedures on measured soil behavior.

4. The entire stress-strain behavior of soil must be measured and adequately reported as a function of 1) time, both during the static phase of the test and during the application of cyclic load, 2) strain level, and 3) stress level. Without such information little use can be made of the data in the development of constitutive relationships.

A review of the published literature shows that these four criteria are seldom if ever met at the present time.

## USE OF LABORATORY TEST METHODS TO DETERMINE THE DYNAMIC STRESS-STRAIN AND STRENGTH PROPERTIES OF SOILS

A comprehensive state of the art paper describing various types of laboratory test equipment that can be used to study dynamic stress-strain and strength properties of soils was presented by Woods (1978). Figure 2 shows the shear strain amplitude capabilities of various classes of laboratory test equipment and Table 2 describes the various dynamic properties that can be measured with the various classes of equipment.

Common to all classes of laboratory test equipment is the measurement of parameters needed to define the static or dynamic laboratory stress-strain and strength properties These parameters are only load, deformation and pore water pressure. A description of how these parameters are measured for each class of laboratory test is shown in Table 3. It may be seen that load measurements are routinely made and few problems are encountered with the measurement. Further, axial and shear strains are also routinely measured with little problem . More difficult , however, is the measurement of lateral deformation during shear. Such deformation measurements are often critical to the evaluation of test results yet these measurements are often not made or not properly made. The result is that important data is not available for test evaluation and test result interpretation.

Further, it may be seen in the table that pore pressure values are normally measured at the boundary of the specimen. This is probably acceptable for dynamic tests on cohesionless materials where pore pressure equalization is almost instantaneous. On the other hand, for clay specimens where pore pressure equalization may take a significant amount of time, pore pressure measurements at the boundary may not represent the average pore pressure through out the specimen (Sangrey,et al, 1978).

Important Considerations Common to the Evaluation of all Classes of Laboratory Test Results

No matter what test is performed and how the test parameters are measured,there are certain important considerations common to the evaluation of all classes of laboratory test results. These considerations include:

1. Specimen preparation
2. Effect of time
3. Equipment friction
4. Membrane penetration
5. Field sampling effects
6. Specimen boundary conditions and the internal state of stress

Incomplete understanding of the effect of each one of these parameters or errors in interpreting their effect can influence dynamic soil test results. Therefore, it is meaningful to discuss the influence of each of these parameters in more detail.

Apparatus Friction. Methods for reducing the friction are well known and have been documented (Silver, 1976). Mechanical means for reducing friction includes O-rings, quad-rings, rolling diagrams, rotating bushings and air bearings. However, in some cases it is not possible to minimize the effect of friction satisfactorily in the test apparatus. When this occurs, it is often possible to put the transducers directly within the test chamber to measure test parameters. Nonetheless, no matter which method is used to minimize friction, it is important that the values are measured and the measurement methods are documented so that the effect of friction can be considered when evaluating the quality of the test results.

Platten Design Requirements. The requirements for successful platten design are 1) to minimize weight, 2) to provide sufficient friction to hold the sample without slippage, or 3) to provide a frictionless end condition. Methods such as epoxying the test material to the platten (particularly effective with cohesionless materials), fins, pins and adhesive are proven methods for holding the sample to the platten (Drnevich, 1978). In some cases just the opposite effect is required and frictionless end plattens have been used (Lee, 1975). In general, a comparison of dynamic test results with and without frictionless end plattens shows little difference. This is probably due to the fact that commonly used frictionless end platten techniques are not completely effective at common cyclic loading rates of 1 Hz. It is probably necessary to reduce the testing frequency to much less than 0.1 Hz to see the effect of frictionless end plattens. For this reason, frictionless end plattens are generally not used in cyclic tests.

Membrane Penetration. Membrane penetration can cause errors in measuring the pore pressure response of cohesionless soils. This is summarized in Table 4 which shows results of measurements of membrane penetration performed by various researchers. In general, the effect of membrane penetration is to underestimate pore pressure values in contractive soils and to overestimate pore pressure values in dilative soils. However, there is some evidence to suggest that the effect of membrane penetration may decrease for large particle sizes and for large samples.

It is clear that more research must be conducted to assess the effect of membrane penetration on dynamic stress-strain and strength properties of cohesionless materials. However, the effect of membrane penetration may turn out to be unimportant for tests used in design and analysis problems. On the other hand, an understanding of membrane penetration effects clearly influences our ability to develop constitutive relationships for soils. Thus, a comprehehsive state of the art report on this subject with suggestions on how to evaluate the effect of membrane penetration for various types of dynamic laboratory tests needs to be prepared and new research should be undertaken to complete our understanding of this important consideration.

Sampling and Disturbance Effects on Cohesive Soils. Sampling disturbance has a large effect in cohesive soils 1) on residual pore pressure remaining after loading, 2) on changes in pore water pressure during loading and 3) on internal migration of pore water and changes in water content throughout the sample. However, sampling effects can be evaluated by making X-radiographs of the core, by measuring pore water pressure after sampling, by evaluating volume change during consolidation, and by evaluating axial strain during shear. Experience may be used to relate these measurements to an evaluation of the amount of the disturbance in the sample (Broms, 1980).

When the amount of disturbance is unacceptable, disturbance effects can be reduced by using better samples or by taking block samples (Horn, 1979). A systematic representation of the influence of sample disturbance on shear strength is shown in Figure 3. It may be seen that block samples give higher test results than 5 inch and 3 inch tube samples whereas 2 inch tubes give much lower test results that may significantly underestimate shear strength values. Anisotropic consolidation or consolidation past the insitu pressure may also be used to reduce the effect of sample disturbance in cohesive soils (Ladd and Foote, 1975).

Sampling and Disturbance Effects on Cohesionless Soils

Sampling disturbance probably has a larger effect on cohesionless soils than on cohesive soils. For example, sampling disturbance affects both soil density and the arrangement of soil particles (which is the fabric of the soil).

Sampling effects can be evaluated however, by making X-radiographs of sample tubes (Krinitzsky, 1970). X-raying of tubes should be a routine technique in any important project where laboratory tests are to be performed on cohesionless materials.

Marcuson and Franklin (1979) have summarized methods for taking better undisturbed samples of cohesionless soils for laboratory testing. Recent experience has shown that careful field work can obtain high quality undisturbed samples of many sands using a fixed piston sampler with drilling mud. However, dense sands tend to loosen and loose sands densify. Further, the use of radiographs adequate and reliable non-destructive method for determining layering and degree of disturbance of the sample. On the other hand, the only reliable method of recovering undisturbed samples with gravel is by hand carving block samples in test pits. Further, in place freezing and coring may provide a better method for obtaining undisturbed samples.

Even with careful sampling there is still controversy over the ratio of undisturbed to remolded strength of cohesionless materials. This is shown on Table 5 which plots the ratio of undisturbed to remolded strength reported by various investigators (Seed, et al, 1975). Horn (1979) describes how such comparisons are difficult to make and interpret. For example, Figure 4 shows typical results of cyclic triaxial strength tests performed on intact and on reconstituted specimens of the same material. It may be seen that the relationship between strain build up and the number of cycles is different for reconstituted and undisturbed specimen. Thus, for low numbers of cycles and low values of cyclic strain, it would appear that undisturbed test specimens are stronger than reconstituted test specimens. On the other hand for high numbers of cycles and larger values of strain, it would appear that reconstituted test specimens are stronger than undisturbed test specimens. Thus, the selection of failure criteria affects the ratio of undisturbed to remolded strength. On a site specific basis where a given failure strain is selected, this strength cross over may not be important. However, when test results from various projects and from different sites are compared together, this type of cycle dependent behavior would give inconsistent comparisons. Thus, the reader is cautioned in evaluating the difference between test results obtained from tests on undisturbed and remolded specimens reported in the literature.

<u>Specimen Boundary Conditions and Internal State of Stress</u>

There is little question that laboratory tests do not exactly model insitu soil behavior. Thus, we must be able to assess the relative effect of 1) sample disturbance, including density changes and fabric changes, 2) the state of stress on boundary of the element and 3) the state of the stress throughout the element. Even for the simplest and best understood test, boundary effects and the internal state of stress can significantly influence test results. This is shown in Figure 5 which plots stress distribution in loaded soil samples in the triaxial test (Gerard and Wardle, 1971). A much more complicated state of stress exists in other types of laboratory equipment such as the simple shear test and the torsional shear test (Saada, et al., 1980).

However, it must be remembered that a laboratory test does not have to exactly model insitu conditions to give useful test values for design and analysis. If the test measures essential physical factors that underlie and dictates the pattern of insitu behavior, useful information can be expected from the test. On the other hand for the development of constitutive relationships, much better understanding of equipment boundary conditions and the internal state of stress is required in order to properly use experimental test results.

<u>Time Effects</u>

Time effects influence results from all classes of laboratory tests and these effects can be very significant.

Time effects must be considered both for consolidation and for testing. For example, Anderson, Stokoe and their coworkers have shown for resonant column tests that the time for consolidation of specimens will influence low amplitude modulus values. This effect is shown in Figure 6 which plots modulus as a function of shear strain for specimens consolidated 1 day, 1 week and 1 month. Also shown on the plot is estimated field performance obtained from field insitu geophysical tests. Clearly, an estimate of field consolidation time must be made before it is possible to use the results of laboratory tests to predict field performance. Methods for making these estimates are described by Anderson and Stokoe (1978).

Consolidation time also influences cyclic triaxial strength results and by inference, consolidation time probably influences cyclic triaxial properties test results as well. This is shown in Figure 7 which shows the cyclic strength of soil specimens consolidated for various lengths of time. It may be seen that the aging effect can significantly increase the cyclic strength of soils (Seed, 1979). Thus, it may be expected that aging effects will also influence modulus values obtained from cyclic triaxial tests.

## SPECIALIZED PROBLEMS AND SOLUTIONS FOR COMMON CLASSES OF CYCLIC AND DYNAMIC LABORATORY TEST EQUIPMENT

Previous pages have described problems common to all classes of laboratory test equipment. However, each specific class of laboratory equipment has particular problems associated with testing and test interpretation. Therefore, it is instructive to discuss each of these classes of test equipment individually and to describe methods for improving the testing procedure and test interpretation.

<u>Resonant Column Test</u>

The resonant column test is the most popular low strain amplitude properties test presently in use. Testing procedures have been documented by Drnevich,et al (1978) and a new ASTM Standard for the procedure should appear in the ASTM Book of Standards in 1982.

Test details required to ensure that meaningful test results are obtained have been described by Drnevich (1978) who summarized the important problems as 1) estimating the maximum strain and amplitude capabilities of the apparatus, 2) coupling between plattens and specimens, 3) limiting specimen stiffness and 4) controlling air migration through the membrane. Drnevich (1978) describes methods for minimizing these detrimental effects.

High Strain Amplitude Cyclic Propreties Tests

Cyclic triaxial, cyclic simple shear and cyclic torsional shear tests are all used to obtain values of stress-strain and strength properties of soils at strain amplitudes higher than can be achieved in the resonant column test. Unfortunately there are no published test procedures for these tests. Further it has been clearly shown that test details can significantly influence test results. These important test details include 1) equipment design, 2) deformation monitoring techniques, 3) pore water pressure measurements, 4) specimen preparation, 5) specimen density, 6) length of the testing period, and 7) the definition of data evaluation terms. Each of these factors will be discussed in detail below.

Equipment Design. All too often laboratory test equipment is not adequate to meet the quality of test results required for both analysis and design and for constitutive relationships. Very often the apparatus stiffness is not sufficient to provide accurate rigidity for the parameters being measured. Further, piston friction is often excessive, alignment between the top and bottom plattens is not correct and platten design is often not acceptable. Techniques for minimizing the effects of equipment design on test results are summarized by Silver (1976).

Pore Water Pressure Monitoring. It is unfortunate but true that most pore water pressure measurement systems are unacceptable. Therefore, in many cases cyclic pore water pressure measurement values are often incorrect. This is particularly true in clays where cyclic pore water pressure measurements are probably not possible to make except at low testing rates (several cycles per day) because of the need for pore pressure equalization (Sangrey Pollard & Egan, 1978). For sands, the need for pore pressure equalization is not as important and generally it is felt that pore pressure measurements can be made at common testing frequencies of 1 Hz.

Certain minimum requirements for pore pressure measurements have been suggested by Silver (1976). These include:

1. Short, small diameter, stiff pressure tubing must be used.

2. Stiff low volumes change transducers must be used.

3. The transducer volume change should not exceed 2.5 x $10^{-6}cm^3/kN/m^2$.

4. The entire pore pressure measurement system should have volume change characteristics less than 2.5 x $10^{-4}cm^3/kN/m^2$.

In most laboratories throughout the world, these criteria are not met with the result that pore pressure measurements are often suspect.

Effect of Specimen Density. Control of density for reconstituted specimens is critical if reproducible test results are to be achieved. It has been shown that densities of reconstituted specimens must be $\pm$ 8 $kg/m^3$ ( 0.5 $lb/ft^3$) to reproduce test results between the various operators in different laboratories (Silver et al, 1976). Further, specimen measurements must be carefully made. A circumference tape must be used to measure the diameter of the specimen and a dial indicator should be used to measure the height of the specimen. Calipers that contact the side of the specimen should not be used because it has been shown that such measurements give incorrect values of specimen diameter.

Definition of Data Evaluation Terms. No matter what testing procedure is used it is important that the data evaluation terms used to calculate the test parameters be clearly defined. In all too many cases failure criteria, load values, deformation values and pore pressure values are not clearly defined with the result that the data cannot be properly used in design and analysis and for the development of constitutive relationship. Figure 8 shows a typical definition of parameters measured in the cyclic triaxial properties test. No matter what terms or definitions are used, such plots should be included in all papers and reports to clearly tell the reviewer and reader how the test parameters are defined, how they were measured and how the test results were calculated.

Cyclic Strength Tests

Cyclic strength tests using triaxial equipment, simple shear equipment and torsional equipment are routinely performed. Test procedures for cyclic triaxial tests are described by Silver (1976) and by the Corps of Engineers (1980). The same procedures can be applied to simple shear tests and to torsional tests. As described previously for resonant column tests and for cyclic properties tests, test details are important if reproducible test results are to be obtained from cyclic strength tests. In particular, the following test details, many of which were described previously, are important:

1. Equipment design
2. Pore pressure measurement
3. Specimen density
4. Length of testing period
5. Specimen preparation
6. Definition of data evaluation terms

In equipment design, the shape of the loading trace has been found to be extremely important (Silver, 1978). For example, Figure 9 shows acceptable and unacceptable loading trace forms. Similarly load fall off, where the load trace cannot keep up with the sample deformation, can affect the test results and load reduction must not be excessive. Criteria for selecting appropriate traces and for evaluating test results are described in detail in Silver (1976).

## MINIMUM REQUIREMENTS FOR THE PRESENTATION OF DYNAMIC SOIL TEST RESULTS

If laboratory test results are not properly presented and material index properties are not adequately described, data both in published papers or in consulting reports cannot meet the needs of the engineer. To minimize this problem, Table 6 presents minimum requirements for the presentation of dynamic soil test results. It may be seen that complete information is required on 1) the material tested, 2) the specimen preparation procedure, 3) equipment characteristics, 4) test procedures, 5) specimen characteristics and 6) test results as a function of time.

In almost all cases, published work has incomplete information on the physical characteristics of the materials tested. Similarly, in some cases specimen preparation procedures are described but more information is usually needed. On the other hand, few papers ever describe the characteristics of the test equipment particularly with regard to piston friction and the characteristics of the pore water pressure measurement system. Further, test procedures describing saturation, consolidation and the time for shear are often lacking.

Other important test details often unreported are the initial, consolidated and final characteristics of specimens in terms of density, unit weight, axial strain, volumetric strain, lateral strain, and water content. Only with such data can a reviewer or designer evaluate the quality of the test results.

Further, very little can be done with test results unless the data is presented as a function of time or of the number of cycles. All too often data is reported for some given number of cycles which provides no information on strain build up, pore pressure values or load characteristics as a function of increasing numbers of cycles. Such incomplete data does not serve the needs of the designer who must select an appropriate number of loading cycles, or the researcher developing constitutive relationships where time effects must be modeled.

Better test result reporting can significantly improve the state of the art in geotechnical dynamic testing. In most cases the required data is collected but not presented. More forethought and care in the presentation of complicated data can do much to improve the state of the art in dynamic geotechnical stress-strain and strength testing.

## USE OF GEOPHYSICAL TESTING METHODS TO DETERMINE THE DYNAMIC STRESS-STRAIN AND STRENGTH PROPERTIES OF SOILS

Geophysical testing methods are well known techniques for obtaining lithology and stratigraphy of soils. Further, geophysical test methods may be used to obtain measures of insitu shear wave and compressive wave velocity in underlying soil layers from which modulus values and Poisson's ratio values can be evaluated.

However, it appears that even more information on insitu dynamic soil properties may be obtained from commonly used geophysical test methods. This can be achieved with an improved understanding of the physical nature of the tests and a more thorough understanding of the relationship between geophysical test methods and dynamic soil properties. Therefore, the following pages will briefly describe acceptable techniques for making geophysical measures in the field and discuss ways for obtaining dynamic stress-strain and strength properties of soils from these measurements.

### Evaluation Requirements and Geophysical Investigation Procedures Required for Dynamic Analysis.

When evaluating dynamic response and stability, a number of soil property characteristics are required including gradation and soil classification, degree of saturation, density and relative density, dynamic modulus, damping, and strength values. Each of these soil properties can be obtained from exploration, geophysical testing, or insitu testing depending on the particular soil property required. This concept is summarized in Table 7 which shows the classes of dynamic properties required for a dynamic analysis and the exploration, geophysical, and insitu test best suited to obtain these properties. In many cases the three test methods should be combined to give a complete picture of the required soil properties.

Exploratory study methods are well known and consist of traditional laboratory and field index tests. On the other hand, geophysical test methods and insitu test methods are less well known and are not always routinely used for determining dynamic soil stress-strain and strength properties. Therefore, it is reasonable to discuss briefly the types of geophysical and insitu tests available and their potential for use in obtaining dynamic stress-strain and strength properties.

### Geophysical Testing Procedures and Purposes

An excellent description of the available geophysical test methods was presented by Woods (1978). Figure 10 shows the strain range generated by the various insitu dynamic testing procedures. It may be seen that geophysical testing generates low shear strain values while cyclic insitu tests (CIST) generate strains over a wide strain range.

A comprehensive description of available test procedures for geophysical testing was presented by the Corps of Engineers, (1980). This reference describes in detail test methods for determining location and correlation of stratigraphy, lithology, discontinuities, depth of over burden, depth to weathered rock and the quality of rock. Further, it discusses how to obtain values of insitu shear wave velocity from which modulus values can be calculated. The following paragraphs describe how these shear wave velocity measurement values and the resulting modulus values may be related to other important geophysical properties.

## Laboratory Geophysical Testing

Laboratory geophysical testing provides an opportunity to measure, under controlled laboratory conditions, the influence of soil properties on geophysical values of shear wave velocity, compression wave velocity and damping. The advantage of laboratory testing is complete control over boundary conditions and test parameters. The disadvantage of laboratory testing is that only a small volume of material is tested and that the material is influenced by sample disturbance.

The most common test procedure used in the laboratory for determining geophysical properties is the resonant column test. Test results are presented in terms of shear wave velocity versus void ratio and shear wave velocity versus shear strain. Also commonly presented are damping values and empirical relationships relating the test parameters together. The basic relationship relating laboratory geophysical measurements to dynamic soil properties is given by the equation:

$$G_{max} = \frac{\gamma_t}{g} v_s^2$$

where $G_{max}$ is the shear modulus at low shear strain values (on the order of $10^{-4}$% shear strain), $\gamma_t$ is the total unit weight, $v_s$ is the shear wave velocity and g is the acceleration of gravity.

Hardin, Drnevich and their coworkers have expressed the relationship between the maximum shear modulus, $G_{max}$, and material properties using the expression

$$G_{max} = 1230 \; OCR^K \frac{(2.973 - e)^2}{1 + e} \bar{\sigma}_m^{0.5}$$

where OCR is the over consolidation ratio, e is the void ratio, $\bar{\sigma}_m$ is the mean effective stress equal to $(\bar{\sigma}_1 + \bar{\sigma}_2 + \bar{\sigma}_3)/3$ and K is a constant depending on the plasticity index (Hardin, 1978):

| PI | K |
|---|---|
| 0 | 0 |
| 20 | 0.18 |
| 40 | 0.30 |
| 60 | 0.41 |
| 80 | 0.48 |
| 100 | 0.50 |

In this equation G and $\bar{\sigma}_m$ are in psi.

For many soils and for routine studies this relationship is often used to define the low strain modulus of soils as measured in the laboratory. However, for some soils and for special studies the modulus values obtained from the equation are checked with laboratory testing.

Seed has developed a similar relationship which relates modulus values of sand and the confining pressure using the following relationship

$$G_{max} = 1000 \; K_{max} \; \bar{\sigma}_m^{0.5}$$

where $K_{max}$ is a constant, and $\bar{\sigma}_m$ is the mean stress as defined above. Seed and Idriss (1970) give the following values for $K_{max}$ for a uniform sand at various relative densities

| $K_{max}$ | Sand Relative Density |
|---|---|
| 62 | 80% |
| 52 | 60% |
| 42 | 45% |

Typical laboratory geophysical test measurements obtained from resonant column tests plotting shear wave velocity versus the void ratio e is shown in Figure 11 (Hardin and Richart, 1963). This plot shows how confining pressure influence the shear wave velocity. Figure 12 shows the same curve for two different soils showing that there is some influence of grain shape on dynamic material behavior. Such plots are valuable as they show the influence of material properties on geophysical measured dynamic soil behavior such as shear wave velocity and compression wave velocity. A number of such plots and summaries exist in the literature (Seed and Idriss, 1970; Richart, et al., 1970).

## Insitu Geophysical Testing

Values of shear wave velocity and the compression wave velocity can also be determined from insitu geophysical testing. The advantages of such testing is that a relatively large soil mass is sampled with minimum disturbance. Disadvantages of field testing include borehole disturbance and a limited understanding of the boundary conditions of the tests.

Test results most commonly and economically obtained in the field include shear wave velocity values and compression wave velocity values. Test results are generally presented in terms of shear wave velocity versus depth, and compression wave velocity versus depth. Measurements of the shear wave velocity and the compression wave velocity make it possible to calculate Poisson's ratio, $\mu$, from the relationship

$$\mu = \frac{v_r^2 - 2}{2(v_r^2 - 1)}$$

where $v_r = v_p/v_s$. In addition, the shear wave velocity, $v_s$, and the compression wave velocity $v_p$, can be related to $G_{max}$ and $E_{max}$ respectively from the relationship

$$G_{max} = \frac{\gamma_t}{g} v_s^2$$

$$E_{max} = \frac{\gamma_t}{g} v_p^2$$

Young's modulus and the shear modulus can be related together in terms of Poisson's ratio with the expression taken from the theory of elasticity

$$E_{max} = 2(1 + \mu) G_{max}$$

Field Testing Procedures for Dynamic Design and Analysis Properties

The most common field testing procedures for dynamic design and analysis problems are 1) seismic refraction tests, 2) cross hole tests, 3) uphole tests, 4) downhole tests and 5) cyclic insitu tests. The characteristics, advantages and disadvantages of each of these techniques is described in Woods (1978).

In the United States the crosshole test is the most commonly used method for measuring values of insitu compression wave and shear wave velocity. Figure 13 shows a schematic drawing of the test for both the two hole and multiple hole test method. It is recommended that the multiple hole technique be used whenever possible since it avoids the problem of having an accurate electronic trigger required to define the time of generation of the crosshole pulse.

No matter what technique is used, it is important that bore hole logging take place to actually measure the horizontal distance between the boreholes. It is well known that even with good drilling, exploration holes can deviate significantly from the vertical. Therefore a bore hole inclinometer should be used in any hole greater than 10m in depth to accurately define the distance between the test holes for accurate calculation of shear wave velocity values.

Uphole tests and downhole tests, schematically represented in Figure 14, are more commonly used overseas. This test, with only one borehole, is much more economical to perform than the crosshole test. On the other hand interpretation of the test results becomes more involved and difficult for the uphole and the downhole test.

Use of Insitu Geophysical Test Results

Often, other insitu geotechnical properties are measured from samples taken from geophysical test boreholes such as void ratio and insitu density. From these measurements, values of insitu shear modulus and Young's modulus can be calculated from the shear wave and compression wave velocity values as described above. Further, it is possible to evaluate a value of Poisson's ratio if both shear wave and compression wave velocity measurements are taken.

A typical plot of shear wave velocity versus void ratio for data obtained from 3 investigators is shown in Figure 15. It may be seen that the data for a single site agree well together and that a straight line can be drawn to relate void ratio and shear wave velocity values. The results of the three investigations are plotted together in Figure 16 showing what might be considered as reasonable plots of void ratio versus shear wave velocity.

However, it is instructive to compare the results obtained from laboratory geophysical tests with data obtained from field geophysical tests. This is shown in Figure 17 for the data from Stokoe and Abdel-razzak (1975) by plotting the experimental data from the field with values obtained using Hardin's equation. It may be seen that this comparison gives an entirely different picture of the data. For example, for the dike site, it appears that field values and Hardin equation values agree well together. This is reasonable since the Hardin equation predicts soil behavior in the laboratory for short consolidation times. The dike site in this case was only 60 days old and it should be expected that the results would agree well together. On the other hand, for the much older field site, it can be seen that Hardin's equation would predict much lower values of shear wave velocity than measured in the field. This is to be expected. On the other hand, it may be seen that the slope of the data predicted by Hardin's equation is completely differen than a reasonable straight line drawn through the data. This shows clearly the site specifi nature of insitu geophysical measurements and the false picture that can be obtained by trying to plot data from different sites together on the same plot without knowledge of the theo retical or experimental relationship between wave velocity and physical soil parameters.

Another type of plot relating insitu shea wave velocity to confining pressure is shown i Figure 18 for data obtained by Anderson, et al (1978). In order to obtain such a plot, it was necessary to know the state of stress both in the horizontal and vertical directions. For their investigation a measure of the horizontal stress was not obtained; therefore, it was assumed that $K_o$ was 0.5. Similar data obtained by Cunny and Fry (1973) is plotted in Figure 19. In their investigation both shear wave velocity and compression wave velocity values were measured which made it possible to calculate the coefficient of earth pressure at rest, $K_o$, using the expression $K_o = \mu/(1 - \mu)$.

An extensive evaluation of these and similar data has shown that accurate representation of insitu confining pressure in terms of the vertical stress and horizontal stress is necessary to accurately use the results of insitu geophysical tests to evaluate insitu geophysical properties. However, in most published literature and in most consulting reports the insitu state of stress is either not measured or is not reported. For this reason it is recommended that measurement of the insitu state of stress be made a part of all geophysical investigations to better determine dynamic stress-strain and strength properties of soils.

Methods for Obtaining Insitu State of Stress

Huck, et al (1974) have made a comprehensive study of the advantages, disadvantages and relative accuracy of methods available for measuring the insitu state of stress. They studied a number of techniques for obtaining the insitu state of stress including geophysical testing, the bore hole pressure meter, the bore hole stress probe, hydraulic fracturing and anisotropic vane shear. The relative accuracy of each of these devices is summarized in Table 8.

Geophysical testing to obtain values of the insitu state of stress is relatively inaccurate. The value of Poisson's ratio is obtained by dividing numbers of the same relative magnitude. Because of this, small test problems can yield large errors in the value of Poisson's ratio. Therefore, full reliance on geophysical test measurements to obtain values of the insitu state of stress should not be made. Geophysical test measurements should be combined with other measurements to determine the insitu state of stress.

The borehole pressuremeter represented in Figure 20 is routinely used to measure the compressibility of soils. However, few researchers suggest that it gives accurate values of the insitu state of stress because of borehole disturbance involved with the insertion of the device into the ground. Borehole disturbance is minimized with a self boring pressuremeter (Fig. 21). However, again few people working with the device claim that the device can give accurate values of the insitu state of stress.

On the other hand, the boreholes stress probe seems to be a reasonably accurate technique for measuring the insitu state of stress. Marchetti (1980) has shown how the borehole stress probe can be used to measure the horizontal state of stress in various classes of soils. The use of such a probe would add little to the cost of a comprehensive geophysical field exploration program and would provide valuable information useful for increasing the value of the program.

Hydraulic fracturing is another technique for measuring the insitu state of stress. It is favored by some practitioners and disfavored by others. Similarly, the anisotropic vane shear test has been used to evaluate the insitu state of stress in soft clay. However, it has shown few favorable results.

The applicability of various field methods for measuring $K_o$ is summarized in Table 9 . As a first approximation it may serve as a guide for selecting a technique for measuring the insitu state of stress.

## Degree of Saturation

The degree of saturation appears to be an important parameter useful in evaluating the potential for liquefaction of a site. Laboratory tests have clearly shown that soils which are not saturated show great resistance to liquefaction (Chaney, 1978). Allen, et al (1980) have clearly shown the relationship between compression wave velocity and degree of saturation. Their data, summarized in Figure 22 shows that compression wave velocity decreases significantly as the degree of saturation decreases from 100% to 99%. Various researchers have shown that at 99% degree of saturation, liquefaction is difficult to obtain in the laboratory. Thus, it appears that field geophysical tests measuring the compression wave velocity may be a powerful tool for evaluating the degree of saturation of a deposit and thus, the potential for liquefaction.

## Minimum Requirements for the Presentation of Insitu Geophysical Test Results

In reviewing geophysical testing results, it quickly becomes apparent that insufficient information is generally presented both in the published literature and in consulting reports be able to make important comparisons between geophysical measurements and cyclic stress-strain and strength properties. Therefore, as presented previously for laboratory test results, a list of minimum requirements for the presentation of geophysical test data is presented in Table 10. As a minimum, it is important that information on the soil profile, material properties, wave velocities as a function of depth, and insitu confining pressure be presented in any summary of insitu geophysical test results. With such data a much more comprehensive picture of the characteristics of a deposit can be prepared and information useful in understainding the relationship between geophysical test results and insitu soil behavior will be available.

## SUMMARY AND CONCLUSIONS

1. This state of the art paper describes better techniques for the use of both laboratory and field test methods to predict the cyclic stress-strain and strength properties of soils. A significant amount of information is obtained both in laboratory and field investigations. However, in only a few cases is this data described in the literature or in consulting reports with sufficient accuracy and scope to make the data useful for design and analysis.

2. There are two classes of tests used to measure the dynamic stress-strain and strength properties of soils. These include 1) soil behavior for design and analysis problems and 2) soil properties for the development of constitutive relationships. The requirements for each class of investigation are quite different and require knowledge of the use to which the data is to be made.

3. For design and analysis problems the following is a relative ranking of the most useful laboratory test procedures based on equipment availability and ease of testing:

- 1a. Resonant column test (Small strain)
- 1b. Cyclic triaxial test (Large strain)
- 2. Cyclic simple shear test
- 3. Torsional shear test

However, the following test details must be closely scrutinized to insure that test results are meaningful:

1. Specimen preparation (reconstituted specimen)
2. Sample disturbance (undisturbed sample)

3. Specimen dimensions and density
4. Equipment friction
5. Pore pressure measurements
6. Shape of the force or deformation loading trace
7. Time effects

4. For constitutive relationships, the following is the relative ranking of the most common laboratory test procedures based on the potential for obtaining the maximum amount of information on soil behavior and the ease of testing:

1. Triaxial shear test
2. Torsional shear test (hollow samples)
3. Simple shear test
4. Cubical shear test

Other useful tests, but with more limited access, include the centrifuge test and the shaking table test .

In addition of all the important test details described in 3 above, the following features of the test must be understood to be able to use the data in developing meaningful constitutive relationships:

1. Boundary conditions
2. State of stress within the specimen
3. Equipment compliance
4. Membrane penetration effects

5. In all types of testing, more complete documentation of the test must be presented in both published papers and in consulting reports. Minimum requirements include a detailed description of:

1. The material tested
2. Specimen preparation procedures
3. Test equipment characteristics
4. Test procedures
5. Specimen characteristics (before consolidation, after consolidation and after testing)
6. Test results as a function of time

6. All test data is generally lacking in information on material deformation and the behavior of the specimen as a function of the number of cycles of loading. These defficiencies can be easily overcome by additional instrumentation and by more complete plotting of the measured test data.

7. More extensive use of field test procedures should be made to obtain dynamic stress-strain and strength properties of soils. At present, insitu testing is probably the most useful technique for obtaining soil properties for design and analysis problems even though there is lack of control over test variables and test boundary conditions.

8. The following is the relative ranking of the most common field testing procedures based on equipment availability, ease of testing, and the state of the art in test interpretation:

1. Standard penetration test
2. Cone penetration test
3. Crosshole test
4. Uphole test
5. Downhole test
6. Refraction survey
7. Cyclic insitu test

9. Additional useful information can be obtained from existing field geophysical test methods. On a site specific basis shear wave and compression wave velocity data may help to extend the amount of dynamic insitu soil behavior data obtained from a routine geophysical testing program.

10. In all types of geophysical testing, more complete documentation of the tests must be presented in both published papers and in consulting reports. Minimum requirements include a detailed description of

1. The soil profile
2. Material index properties
3. Wave velocities as a function of depth
4. Insitu confining pressure (both vertical and horizontal)

Acknowledgements:

The help of Mr. Andrew Blystra, graduate student in the Department of Materials Engineering, is gratefully acknowledged for his assistance in investigating the relationship between geophysical and geotechnical soil properties. The paper was typed by Ms. Ariani Brenner and drafting assistance was given by Mr. Robert Sanchez and Ms. Mescatt Assian.

REFERENCES

Allen, N.F., Richart, F.E., Jr., Woods, R.D., (1980), "Fluid Wave Propagation in Saturated and nearly Saturated Sands," Journal of the Geotechnical Engineering Division, ASCE, March.

Anderson, D.G., Espana, C., and McLamore, V.R., (1978), "Estimating in Situ Shear Moduli at Competent Sites," Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference, Earthquake Engineering and Soil Dynamics, Pasadena, California, Vol. 1, June 19-21, pp. 181-197.

Anderson, D.G., and Stokoe, K.H., II, (1978), "Shear Modulus: A Time-Dependent Soil Property," Dynamic Geotechnical Testing, ASTM, STP 654, American Society for Testing and Materials.

Anderson, D.G., and Woods, R.D., (1975), "Comparison of Field and Laboratory Shear Moduli," Proceedings of the In Situ Measurement of Soil Properties, ASCE, June 1-4, Raleigh N.C., Vol. 1, pp. 69-92.

Benerjee, N.G., Seed, H.B., and Chan, C.K., 1979), "Cyclic Behavior of Dense Coarse Grained Materials in Relation to The Seismic Stability of Dams," Report No. UCB/EERC 79/13, Earthquake Engineering Research Institute, University of California, Berkeley, June, 252 pp.

Broms, B.B., (1980) "Soil Sampling in Europe: State-of-the-Art," Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT1, January, pp. 65-98.

Chaney, R.C., (1978), "Saturation Effects on the Cyclic Strength of Sands," Proceedings of the Specialty Conference on Earthquake Engineering and Soil Dynamics, ASCE, Vol. 1, Pasadena, California, pp. 342-358.

Conlon, R.J., and Isaacs, R.M.F., (1970), "Effect of Sampling and Testing Techniques on the Shear Strength of a Glacial-Lacustrine Clay from Welland, Ontario," ASTM Symposium on Sampling of Soil and Rock, STP 483, American Society for Testing and Materials, Toronto, Canada, June 26, pp. 10-29.

Corps of Engineers, (1979),"Geophysical Exploration,"EM 1110-1-802, Engineer Manual, Department of the Army, Corps of Engineers, Office of the Chief of Engineers, May.

Corps of Engineers, (1980), "Laboratory Soils Testing," Engineer Manual EM 1110-2-1906, Reprint Volume with Change I, Headquarters, Department of the Army, Office of the Chief of Engineers, May 1.

Dobry, R., and Athanasiou-Grivas, (1978), "Stress-Strain Relation for Soils Under Earthquake Loading," Report No. 78-2, Department of Civil Engineering, Rensselear Polytechnic Institute, Troy, N.Y., July, 77 pp.

Drnevich, V.P., (1978), "Resonant Column Testing Problems and Solutions," Dynamic Geotechnical Testing, ASTM, STP 654, American Society for Testing and Materials, pp. 384-398.

Drnevich, V.P., Hardin, B.O., and Shippy, D.J., (1978), "Modulus and Damping of Soils by the Resonant Column Method," Dynamic Geotechnical Testing, ASTM, STP 654, American Society for Testing and Materials, pp. 66-90.

Ferrito, J.M., Forrest, J.B., and Wu, G.,(1979), "A Compilation of Cyclic Triaxial Liquefaction Test Data," Geotechnical Testing Journal Vol. 2., No. 2, June, pp. 106-113.

Frydman, S., Zertlin, I.G., and Alpan, I.,(1973), "The Membrane Effect in Triaxial Testing of Glanular Soils," Journal of Testing and Evaluation, Vol. I., No. 1, pp. 37-41.

Gerrard, C.M., and Wardle, L., (1971), "The Predicted Effect of Soil Sampling Disturbance on the Stress-Strains Developed During Triaxial Testing," Proceedings specialty Session on Quality in Soil Sampling, 4th Asian Conference on Soil Mechanics and Foundation Engineering, pp. 40-48.

Hardin, B.O., (1978), "The Nature of Stress-Strain Behavior for Soils," Proceedings, Conference on Earthquake Engineering and Soil Dynamics, ASCE, Pasadena, California, Vol. 1, pp. 3-90.

Hardin, B.O., and Drnevich, V.P., (1972), "Shear Modulus and Damping in Soil: Measurement and Parameter Effects," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 98, No. SM6, June, pp. 603-624.

Hardin, B.O., and Drnevich, V.P., (1972), "Shear Modulus and Damping in Soil: Design Equations and Curves," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98, No. SM7, July, pp. 667-692.

Hardin, B.O., and Richart, F.F., Jr., (1963), "Elastic Wave Velocities in Granular Soils," Journal of the Soil Mechanics and Foundations Engineering Division, ASCE, Vol. 89, No. SM1, February, pp. 33-65.

Horn, H.M., (1979), "North American Experience in Sampling and Laboratory Dynamic Testing," Geotechnical Testing Journal, Vol. 2, No. 2, June, pp. 84-97.

Huck, P.J., Pincus, H.J., Singh, M.M., and Chugh, Y.P., (1974), "Determination of the In-Situ State of Stress in Soil Masses," Report No. FHWA-RD-74-68, Federal Highway Administration, Washington, D.C., September, 316 pp.

Kiekbusch, M., and Schuppener, B., (1977), "Membrane Penetration and Its Effects on Pore Pressures," Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT 11, pp. 1267-1279.

Krinitzsky, E.L., (1970), Radiography in the Earth Sciences, and Soil Mechanics, Plenum Press, New York, 163 pp.

Ladd, C., and Foott, R., (1974), "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT7, July, pp. 763-786.

Lee, K.L., (1975), "Influence of End Constraint on Undrained Triaxial Strength of Sand," Waterways Experiment Station, Vicksburg

Lysmer, J., (1978), "Analytical Procedures in Soil Dynamics," Proceedings, Conference on Earthquake Engineering and Soil Dynamics, ASCE, Pasadena, Vol. III, pp. 1267-1316.

Marchetti, S., (1980), "In Situ Tests by Flat Dilatometer," Journal of the Geotechnical Engineering Division, ASCE, Vol. 106, No. GT3, March, pp. 299-321.

Marcuson, W.F., and Franklin, A.G., (1979), "State of the Art of Undisturbed Sampling of Cohesionless Soils," Proceeding of the International Symposium of Soil Sampling, Singapore, pp. 57-72.

Martin, R.M., Finn, W.D.L., and Seed, H.B., (1978), "Effects of System Compliance in Liquefaction Tests," Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT4, April, pp. 463-479.

Mori, H., (1978), "Japanese Experience in Soil Sampling and Its Influence on Dynamic Laboratory Testing," Preprint No. 3440, ASCE Convention, Chicago, pp. 87-112.

Newland, P.L., and Allely, B.H., (1959), "Volume Changes During Undrained Triaxial Tests on Saturated Diletant Granular Materials," Geotechnique, Vol. 9, pp. 174-182.

Raju, V.S., and Sadaswan, S.K., (1974), "Membrane Penetration in Triaxial Tests on Sands," Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT 10, pp. 482-489.

Raju, V.S., and Venkataramana, (1980), "Undrained Triaxial Tests to Assess Liquefaction Potential of Sands - Effect of Membrane Penetration," Proceedings, International Symposium on Soils Under Cyclic and Transient Loading, Swansea, pp. 483-494.

Richart, F.E., Jr., Hall, J.R., Jr., and Woods, R.D., (1970), Vibrations of Soils and Foundations, Prentice-Hall, Englewood Cliffs, N.J., 414 pp.

Roscoe, K.H., Schofield, A.N., and Thurairajah, A., (1963), "An Evaluation of Test Data for Selecting a Yield Criterion for Soils," Laboratory Shear Testing of Soils, ASTM, STP 361, pp. 111-128.

Saada, A.S., Townsend, F.H., and Gilbert, P.A., (1980), "Strength Laboratory Testing of Soils - A State of the Art," ASTM Symposium on the Shear Strength of Soils, Chicago, June.

Sangrey, D.A., Pollard, W.S., and Egan, J.A., (1978), "Errors Associated with Rate of Undrained Cyclic Testing of Clay Soils," Dynamic Geotechnical Testing, ASTM, STP 654, American Society for Testing and Materials, pp. 280-294.

Seed, H.B., (1979), "Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT 2, February, pp. 201-255.

Seed, H.B., Arango, I., and Chan, C.K., (1975), "Evaluation of Soil Liquefaction Potential During Earthquakes," Report No. EERC 75-28, Earthquake Engineering Research Center, University of California, Berkeley, May.

Seed, H.B., and Idriss, I.M., (1970), "Soil Moduli and Damping Factors for Dynamic Response Analysis," Report No. EERC 70-10, Earthquake Engineering Research Center, University of California, Berkeley.

Silver, M.L., (1976), "Laboratory Triaxial Testing Procedures to Determine the Cyclic Strength of Soils," Report NUREG-31, U.S. Nuclear Regulatory Commission, Washington, D.C.

Silver, M.L., and Park, T.K., (1976), "Liquefaction Potential Evaluated from Cyclic Strain Controlled Properties Tests on Sands," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 16, No. 3, September, pp. 51-65.

Silver, M.L., and Seed, H.B., (1971), "Deformation Characteristics of Sand Under Cyclic Loading," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM8, August, pp. 1081-1098.

Silver, M.L., and others, (1976), "Cyclic Triaxial Strength of Standard Test Sand," Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT5, May, pp. 511-523.

Stokoe, K.H., II, and Abdel-razzak, K.G., (1975), "Shear Moduli of Two Compacted Fills," Proceedings of the Conference on In Situ Measurement of Soil Properties, ASCE, June 1-4, Raleigh, N.C., Vol. 1, pp. 422-449.

Woods, R.D., (1978), "Measurement of Dynamic Soil Properties," Proceedings, Conference on Earthquake Engineering and Soil Dynamics, Pasadena, ASCE, Vol. I, pp. 91-178.

Yoshimi, Y., Richart, F.E., Jr., Prakash, S., Barkan, D.D., and Ilyicher, V.A., (1977), "Soil Dynamics and Its Application to Foundation Engineering," State-of-the-Art Report, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp. 605-650.

TABLE 1

Recent State of The Art Reports Relating to The Dynamic Behavior of Soil

| Subject | Content and Conclusions | No. of Citations | References |
|---|---|---|---|
| Soil Dynamics | Discussion of dynamic stress-strain relationships; liquefaction; seismic response of soil deposits, dams and structures; dynamics of bases and foundations; and soil structure interaction. | 278 | Yoshimi et al (1977) |
| Dynamic Field and Laboratory Testing Procedures | Summary of dynamic field and laboratory test methods. Discussion of test procedures. | 171 | Woods (1978) |
| Analytical Procedures in Soil Dynamics | Summary of soil dynamic analysis for foundation vibrations, pile vibrations, seismic site response problems and soil structures interaction. | 162 | Lysmer (1978) |
| Stress-Strain Behavior of Soils | Discussion of elastic and plastic strains in soils under dynamic loading | 79 | Hardin (1978) |
| Stress-Strain Behavior of Soils Under Dynamic Loading | Summary of analytical models developed for earthquake response analysis, stress-strain behavior and non-linear models for earthquake loading. | 169 | Dobry and Athanasiou-Grivas (1978) |
| Effect of Sampling on Dynamic Soil Behavior | Discussion of field sampling methods (block samples, large diameter samples) sample disturbance and other factors on measured laboratory cyclic strength values. Experience from Europe, Japan and the United States. | 21<br>175<br>81 | Mori (1978)<br>Broms (1980)<br>Horn (1979) |
| Undisturbed Sampling of Cohesionless Soils | High quality undisturbed samples can be obtained using a fixed piston sampler and drilling mud. However, the sampling process loosens dense sands and densifies loose sands. | 34 | Marcuson and Franklin (1979) |
| Geophysical Exploration | Provides guidance and information concerning the use of exploration geophysical methods and equipment in geological and foundation investigations. | 121 | Corps of Engineers (1979) |
| Static Laboratory Testing Procedures | Summary of test devices and an evaluation of state of stress imposed on specimens. Discussion of factors (end platten roughness, membrane penetration, etc.) that influence test results. | 159 | Saada, Townsand and Gilbert, (1980) |

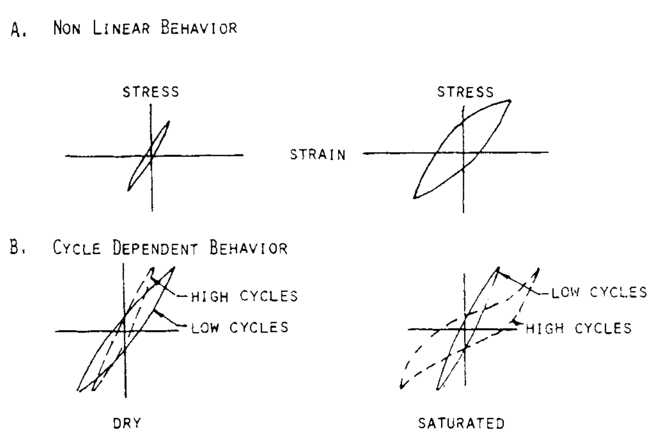


Fig. 1 Dynamic Non Linear, Hysteretic, Strain Dependent Soil Behavior.

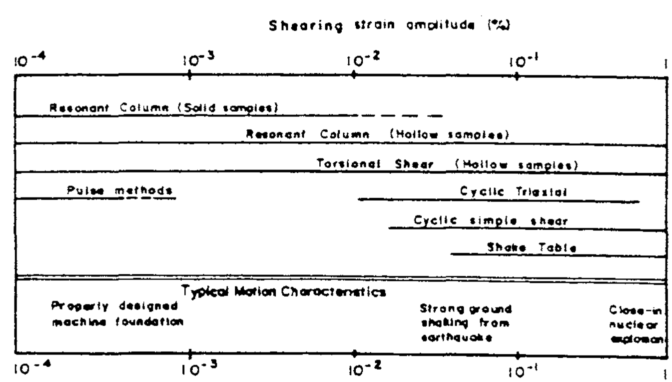


Fig. 2 Common Laboratory Testing Procedures Used to Evaluate The Dynamic Properties of Soils.

TABLE 2

RELATIVE QUALITY OF LABORATORY TECHNIQUES FOR MEASURING DYNAMIC SOIL PROPERTIES

| | Relative Quality of Test Results | | | | |
|---|---|---|---|---|---|
| | Shear Modulus | Young's Modulus | Material Damping | Effect of No. of Cycles | Atten-uation |
| Resonant Column | Good | Good | Good | Good | ____ |
| with adaptation | ____ | ____ | ____ | ____ | Fair |
| Ultrasonic Pulse | Fair | Fair | ____ | ____ | Poor |
| Cyclic Triaxial | ____ | Good | Good | Good | ____ |
| Cyclic Simple Shear | Good | ____ | Good | Good | ____ |
| Cyclic Torsional Shear | Good | ____ | Good | Good | ____ |
| Shake Table | Fair | ____ | ____ | Good | ____ |

TABLE 3

PARAMETERS MEASURED IN DYNAMIC OR CYCLIC LABORATORY TESTS

| | Resonant Column | Cyclic Triaxial | Cyclic Simple Shear | Torsional Shear |
|---|---|---|---|---|
| 1. Load | Resonant Frequency | Axial Force | Horizontal Force | Torque |
| 2. Deformation | | | | |
| - Axial | Vertical Displacement | Vertical Displacement | Vertical Displacement | Vertical Displacement |
| - Shear | Acceleration | Not Measured | Horizontal Displacement | Rotation |
| - Lateral | Not Usually Measured | Not Usually Measured | Often Controlled | Not Usually Measured |
| - Volumetric | None for undrained tests. Volume of fluid moving into or out of the sample for drained tests. | | | |
| 3. Pore Water Pressure | Not Usually Measured | Measured at Boundary | Measured at Boundary | Measured at Boundary |

## TABLE 4

## EFFECT OF MEMBRANE PENETRATION ON THE CYCLIC STRENGTH OF SAND

| Procedure Used to Assess Effect of Membrane Penetration | Results | Reference |
|---|---|---|
| Under isotropic loading, membrane effect should be the difference between 3 times measured axial and volumetric strain. | Provided quantitative evaluation of effect of membrane penetration. | Newland and Allely (1959) |
| Same as above. Also fabricated specimens with internal rods to obtain effect of membrane penetrations. | A volume charge value without membrane penetration was determined | Roscoe, et al. (1963) |
| Improved the interpretation of the test results presented by Roscoe. | Better evaluation of the effect of membrane penetration. | Raju and Sadasivan (1974) |
| Tests on glass spheres of varying diameter. | Relationship between penetration and $D_{50}$ of the sand. | Frydman, et al (1973) |
| Used thin layer of liquid rubber to reduce membrane penetration. | Confirmed relationship of Frydman et al. Higher pore pressures recorded from static undrained triaxial compression tests using modified membranes. | Kiekbusch and Schuppener (1977) |
| Theoretical analysis of errors arrising from volumetric compliance in cyclic liquefaction tests on saturated sands. | Significant errors in measuring pore pressure are possible. Suggest constant volume simple shear liquefaction tests for accurately assessing effects of membrane compliance. | Martin, Finn and Seed (1978) |
| Study of membrane penetration effects on large (3050 mm) diameter triaxial specimens using special girth gages. | For well graded gravel, membrane compliance effects were not large and resulted in a 10% correction in stress values to reach 100% pore pressure ratio. | Banerjee, Seed and Chan (1979) |
| Used Polyethlene strips and polyurethane coating to reduce membrane penetration. | Membrane penetration causes underestimation of pore pressures in contractive soils and overestimation in dilative soils. | Raju and Venkataramana (1980) |

## TABLE 5

## COMPARISON OF LIQUAFACTION RESISTANCE CHARACTERISTICS OF UNDISTURBED RECONSTITUTED SAMPLES OF COHESIONLESS SOILS (FROM BENERGEE, ET.AL., 1979)

| FIRM | PROJECT | RATIO OF UNDISTURBED TO REMOLDED STRENGTH[1] | SOIL TYPE | METHOD OF RECONSTITUTING |
|---|---|---|---|---|
| Woodward-Clyde (Oakland, Ca.) | South Texas | 1.00 | silty fine sand, $D_{50}$ = 0.07 to 0.27 mm | moist tamping, 3/4" dia. tamping foot |
| Woodward-Clyde (Orange, Ca.) | San Onofre | 1.15 | well-graded coarse to fine sand, 15% - #200 sieve | moist tamping, 3/4" dia. tamping foot |
| U.C. Berkeley | Blue Hills Texas | 1.15 | uniform fine silty sand, $D_{50}$ = 0 4mm, 8% to 15% - #200 sieve | moist tamping, 1.4" dia. tamping foot |
| Dames & Moore (San Fran., Ca ) | Allens Creek (heat sink area) | 1.20 | fine silty, clayey sand, $D_{50}$ = 0.03 to 1 6 mm, 0% to 40% - #200 sieve | moist tamping, 1" dia. tamping foot |
| Dames & Moore (San Fran., Ca.) | Allens Creek (plant area) | 1.27 | fine silty, clayey sand, $D_{50}$ = 0.03 to 1.6 mm, 0% to 40% - #200 sieve | moist tamping, 1" dia. tamping foot |
| Converse-Davis | Perris Dam | 1.45 | clayey sand, LL = 26, PI = 11, 44% - #200 sieve | moist tamping, 1/2" dia tamping foot |
| Law Engineering and Testing | Florida sand | 1.30 | silty sand with shells | dry vertical vibrations, frequency = 120 c.p.s. |
| W. E. S. | Ft. Peck Dam (foundation) | 1.65 to 1.80 | uniform fine silty sand | dry rodding (3/8" dia. foot), followed by static compaction |
| W. E. S. | Ft. Peck Dam (shell) | 1.70 to 2.00 | uniform fine to medium sand | dry rodding (3/8" dia. foot), followed by static compaction |

[1]Ratio of cyclic stress ratios required to cause liquefaction in ten cycles for undisturbed and remolded samples.

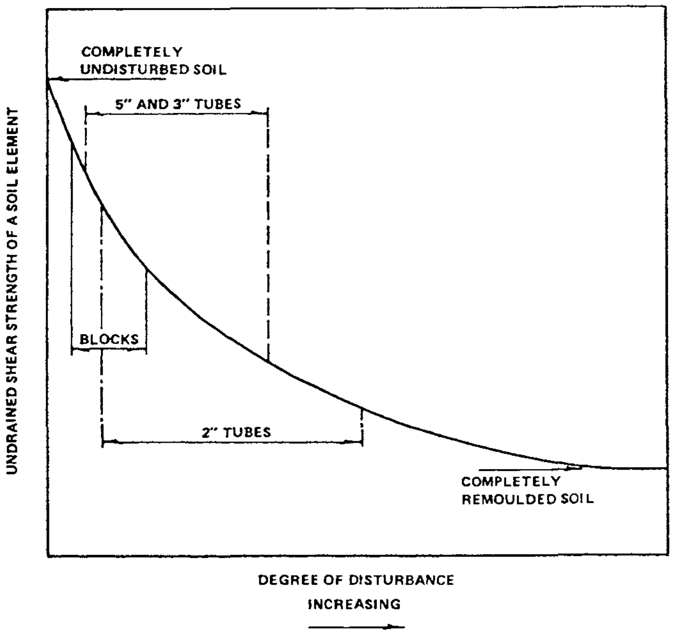


Fig. 3 Influence of Sample Disturbance on The Shear Strength Properties of Soils (from Horn, 1979).

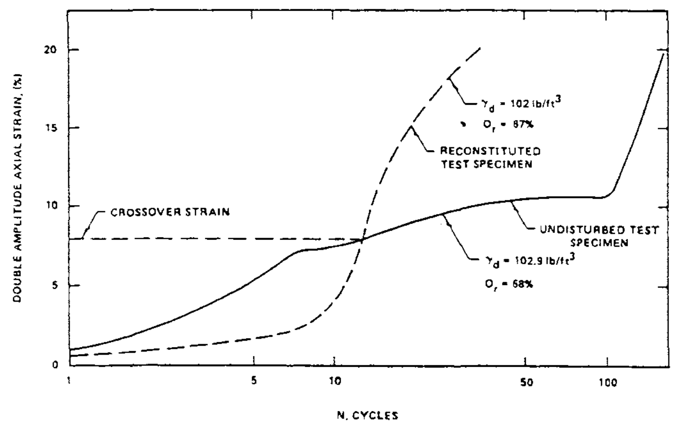


Fig. 4 Effect of Number of Cycles on Specimen Deformation in Cyclic Triaxial Strength Tests (from Horn, 1979).

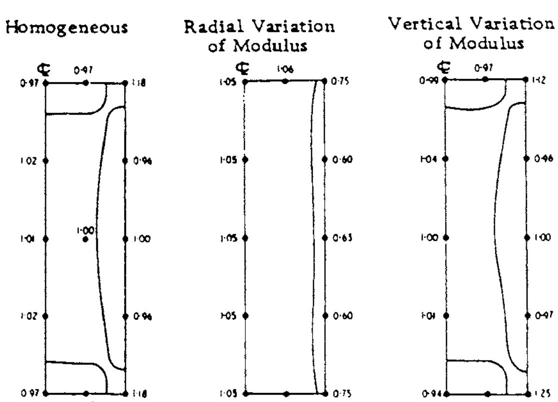


Fig. 5 Stress Distribution in Axially Loaded Soil Samples (from Gerrard Wardle, 1971).

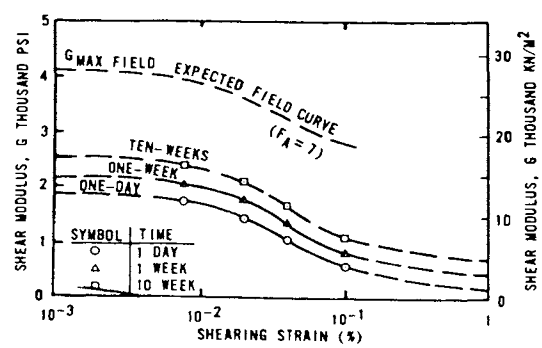


Fig. 6 Effect of Time on Shear Modulus Versus Shear Strain Relationships For Soils (from Anderson and Stokoe, 1978).

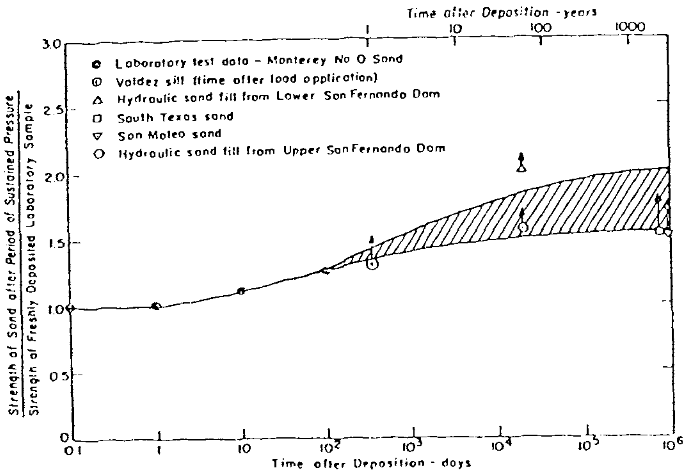

Fig. 7 Influence of Period of Sustained Pressure on Stress Ratio Causing 100% Pore Pressure Response in Cyclic Triaxial Strength Tests (from Seed, 1979).

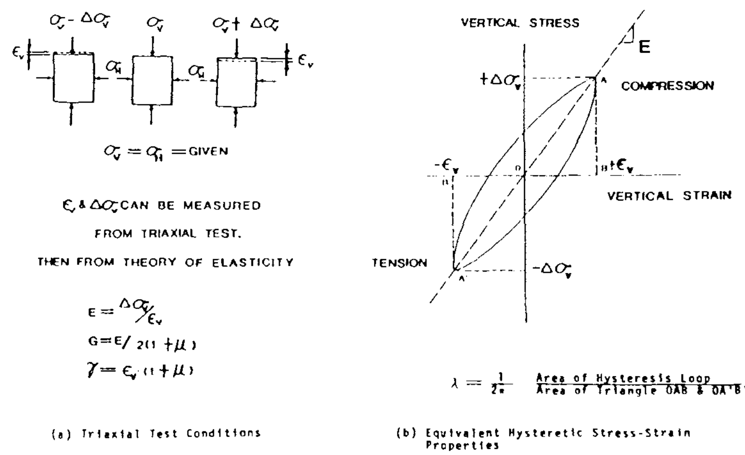

Fig. 8 Definition of Triaxial Test Conditions and Equivalent Linear Hysteretic Stress Strain Properties Calculated From Cyclic Triaxial Properties Tests.

TABLE 6

MINIMUM REQUIREMENTS FOR THE PRESENTATION
OF DYNAMIC SOIL TEST RESULTS

1. Material Tested
   - Classification
   - Grain Size (How was fine fraction measured)
   - Geologic Origin
   - Atterberg Limits (Cohesive soils)
   - Limiting Densities (Cohesionless soils)

2. Specimen Preparation

   Undisturbed
   - Sampling procedure (Borehole or block sample)
   - Sample trimming

   Reconstituted
   - Sample conditioning
   - Specimen preparation procedure
   - Molding water content

3. Equipment Characteristics
   - Piston Friction
   - Membrane Characteristics
   - Pore Pressure Measurement System
   - Platten Characteristics

4. Test Procedures
   - Saturation
   - Consolidation
   - Shear (Time)

5. Specimen Characteristics
   - Initial Dry Weight, Height and Volume
   - Density or Unit Weight*
   - Axial Strain*
   - Volumetric Strain*
   - Lateral Strain (If measured)*
   - Water Content*

   *(Before consolidation, after consolidation and after testing).

6. Test Results as a Function of Time
   - Load
   - Deformation (Including lateral and volumetric deformations)
   - Pore Pressure

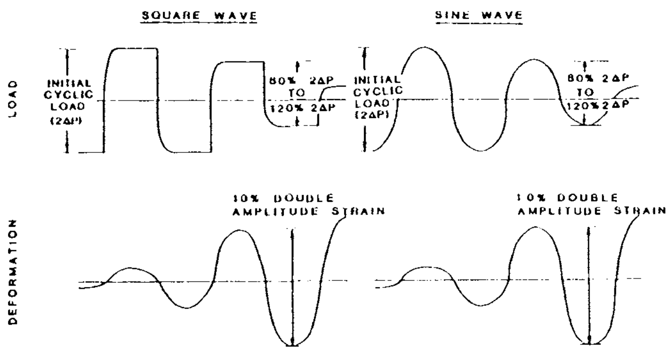


Fig. 9 Characteristics of Acceptable and Unacceptable Wave Forms Generated In The Cyclic Triaxial Strength Test (from Silver, 1976).

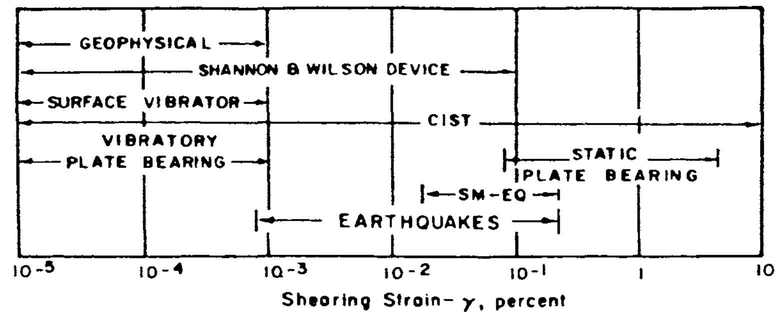


Fig. 10 Common Field Testing Procedures Used to Evaluate The Dynamic Properties of Soils.

TABLE 7

EVALUATION REQUIREMENTS AND INVESTIGATION

PROCEDURES REQUIRED FOR DYNAMIC ANALYSIS

| | |
|---|---|
| Dynamic Stability and Potential for Liquefaction | |
| Gradation and Soil Classification | E. Laboratory Testing<br>G. Crosshole and Uphole/Downhole Surveys; Reflection (3) |
| Degree of Saturation | E. Laboratory Testing<br>G. Lateral Resistivity |
| Density and Relative Density | E. Laboratory Testing<br>G. Crosshole and Uphole/Downhole Surveys; Reflection (3)<br>I. Standard Penetration Test |
| Dynamic Modulus Values | E. Laboratory Testing<br>G. Crosshole and Uphole/Downhole Surveys; Reflection<br>I. Standard Penetration Test |
| Damping Values | E. Laboratory Testing<br>G. Insitu Impulse |
| Dynamic Strength Values | E. Laboratory Testing<br>G. (2)<br>I. Standard Penetration Test |

Notes: (1) The letter "E" represents conventional foundation exploration and laboratory testing procedures. The letter "G" represents geophysical methods. The letter "I" represents conventional insitu procedures.

(2) No procedure available.

(3) Data obtained by these procedures may be based on correlations with such factors as P-Wave velocities, S-Wave velocities, shear modulus, Young's modulus, and Poisson's ratio.

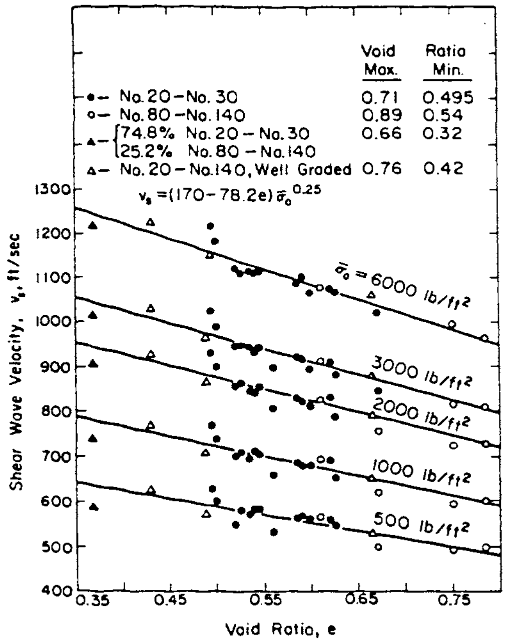


Fig. 11 Variation of Shear Wave Velocity with Void Ratio for Various Confining Pressures, Grain Sizes, and Gradations in Dry Ottawa Sand (from Hardin and Richart, 1963).

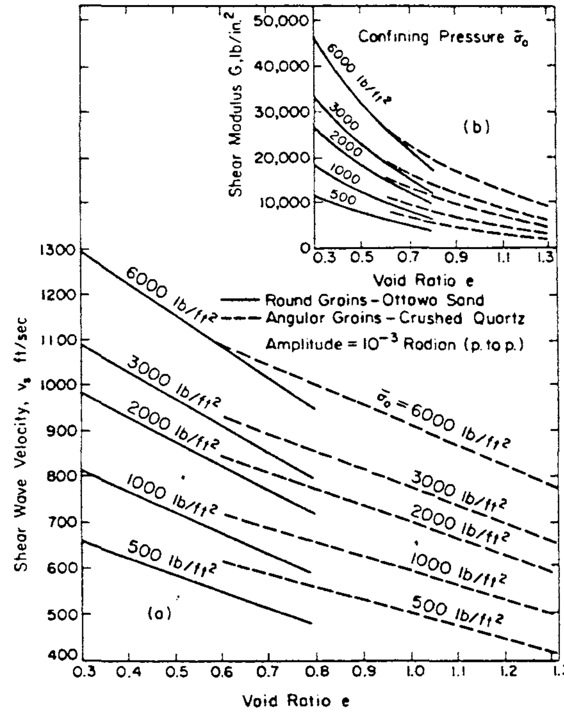


Fig. 12 Variation of Shear Wave Velocity and Shear Modulus with Void Ratio and Confining Pressure for Dry Round and Angular Grained Sands (from Hardin and Richart, 1963).

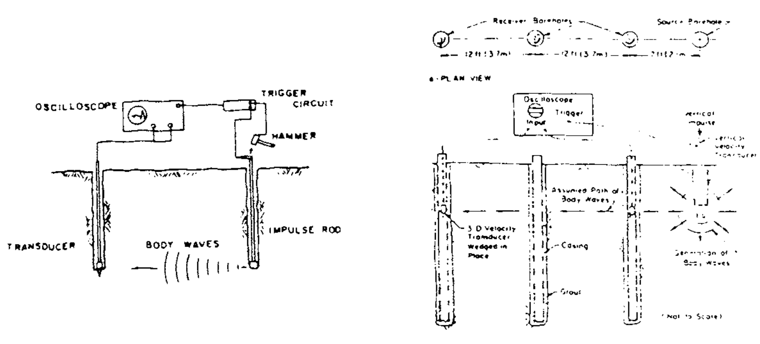


Fig. 13 Seismic Crosshole Survey Techniques Using Two Borehole and Multiple Borehole Methods (from Woods, 1978).

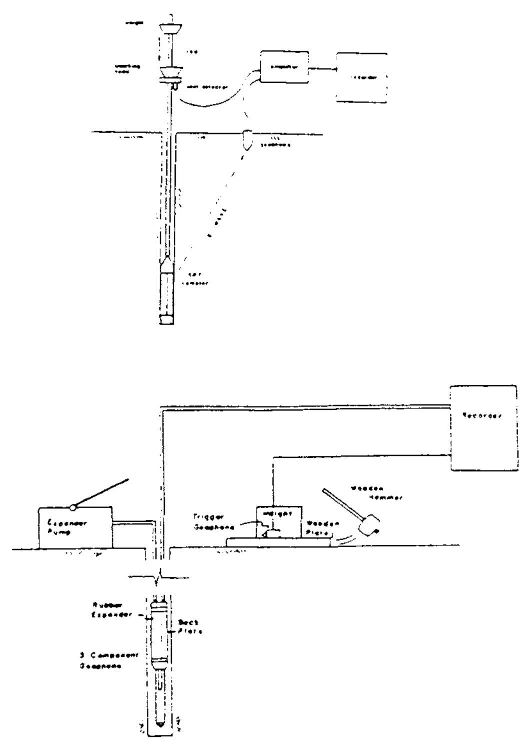

Fig. 14 Seismic Uphole and Seismic Downhole Survey Techniques (from Woods, 1978).

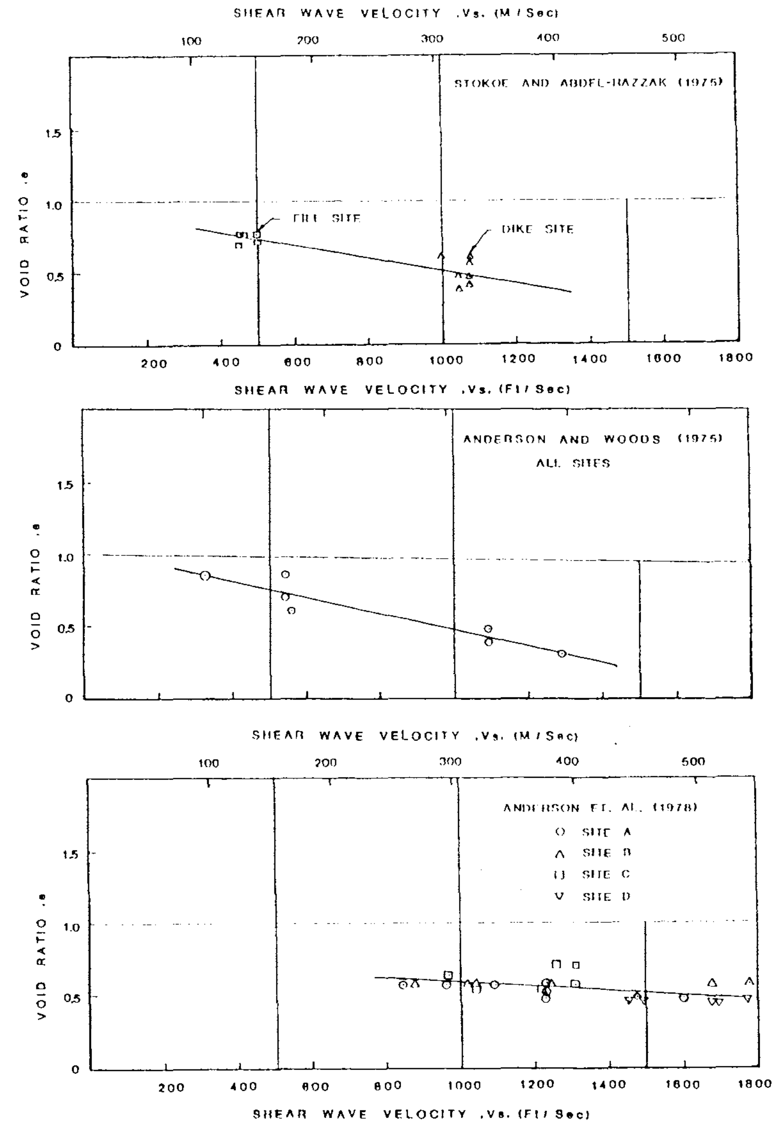


Fig. 15 Void Ratio Versus Shear Wave Velocity Values Measured From Geophysical Crosshole Techniques by Various Investigators.

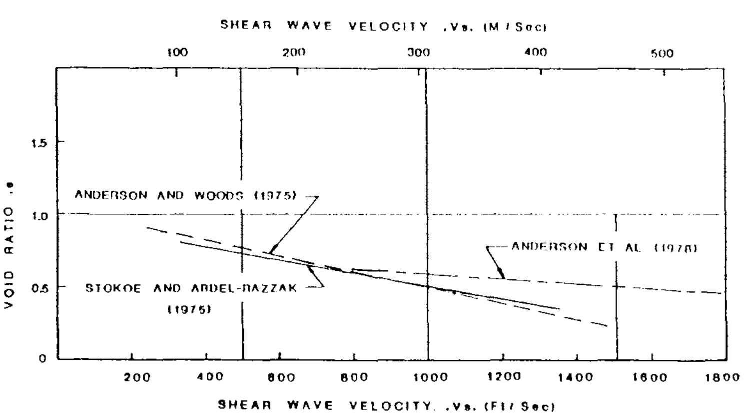


Fig. 16 Summary of Void Ratio Versus Shear Wave Velocity Values Measured From Geophysical Cross-hole Methods.

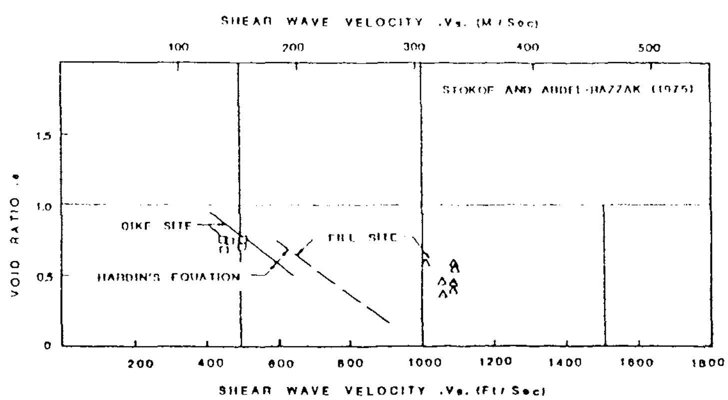


Fig. 17 Comparison of Void Ratio Versus Shear Wave Velocity Values Measured From Geophysical Crosshole Methods and Calculated From Hardin's Equation.

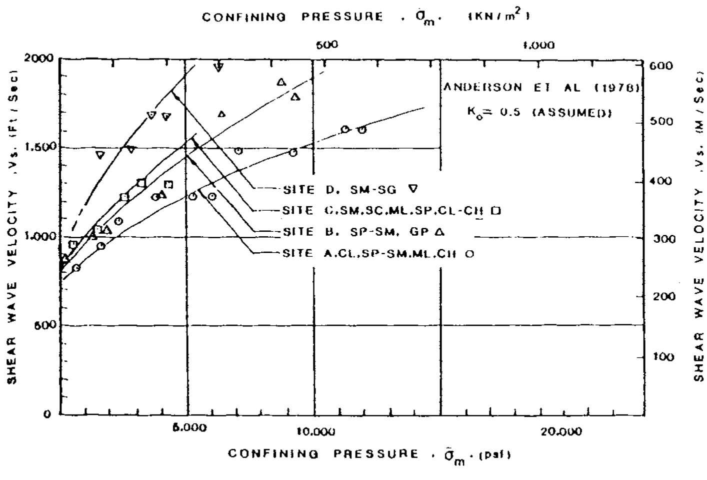


Fig. 18 Relationship Between Shear Wave Velocity and Confining Pressure For Assumed Values of The Insitu State of Stress.

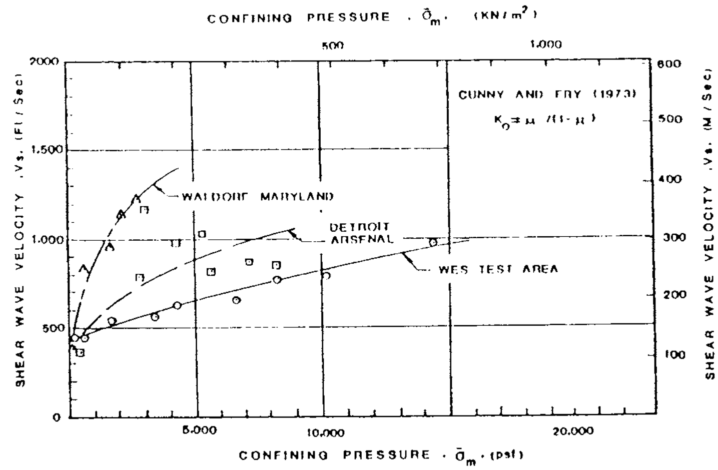


Fig. 19 Relationship Between Shear Wave Velocity and Confining Pressure For Values of The Insitu State of Stress Estimated From Geophysical Test Procedures.

TABLE 8

CHARACTERISTICS OF VARIOUS TECHNIQUES FOR DETERMINING THE INSTITU STATE OF STRESS IN SOILS

| test | most applicable geotechnical conditions | quanity measured |
|---|---|---|
| sonic velocity (2.4.1) | granular or cohesive soils | stress directions by multiple use; estimate mean stress by single use. |
| borehole pressuremeter (self drilling) (2.4.2) | best in cohesive soil | mean lateral stress also soil stiffness. |
| borehole stressprobe (2.4.3) | frictional or cohesive | normal principal stress regardless of orientation. |
| hydraulic fracturing (2.4.4) | fine-grained $K_o \leq 1$ | minimum principal stress regardless of orientation. |
| anisotropic vane shear (2.4.5) | best in frictional soil | stress ratio. |

The choice of technique must be made on the basic circumstances associated both with the site and the experience and facilities available to the engineer. We cannot include all the various capabilities and limitations of each technique and the table is not a substitute for sound engineering judgment.

From Huck, et al (1974)

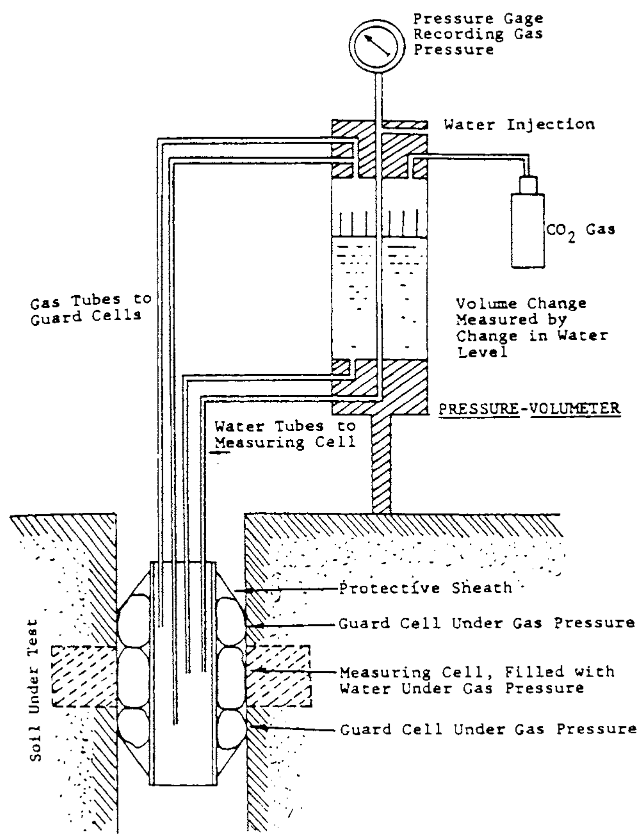


Fig. 20 Schematic Representation of Bore-hole Pressure Meter.

TABLE 9

APPLICABILITY OF FIELD METHODS FOR MEASURING KO UNDER VARIOUS GEOTECHNICAL CONDITIONS

| Soil Type | Fine Grained | | | Course Grained | | | |
|---|---|---|---|---|---|---|---|
| | lateral stresses equal | | | lateral stresses equal | | | |
| STRESS HISTORY | $K_o$ Normally Consolidated | $K_o$ Over-Consolidated | Full $\sigma_1 \neq \sigma_2 \neq \sigma_3$ Triaxial | $K_o$ Normally Consolidated | $K_o$ Over-Consolidated | Full $\sigma_1 \neq \sigma_2 \neq \sigma_3$ Triaxial | Quantity Obtained |
| Acoustic Velocity | X | X | X | X | X | X | $\sigma_n$ |
| Borehole Pressuremeter | X | X | N/A | | | N/A | $\sigma_h$ |
| Borehole Stressprobe | X | X | | X | X | | $\sigma_n$ |
| Hydraulic Fracturing | X | N/A | X | N/A | N/A | N/A | $\sigma_3$ |
| Anisotropic Vane Shear | | | N/A | X | X | N/A | $K_o$ |

STRESS CODE

$K_o = \sigma_h / \sigma_v$

From Huck, et al (1974)

TABLE 10

MINIMUM REQUIREMENTS FOR THE PRESENTATION OF INSITU GEOPHYSICAL TEST RESULTS

1. Soil Profile
   - Water table location
2. Material Properties
   - Classification
   - Geologic origin
   - Grain size (how was fine fraction measured)
   - Limiting densities (cohesionless soils)
   - Atterberg limits (cohesive soils)
   - Insitu unit weight
   - Specific gravity
   - Void ratio
3. Wave Velocity as a Function of Depth
   - Compression wave velocity
   - Shear wave velocity
4. Insitu Confining Pressure
   - Vertical stress
   - Horizontal stress

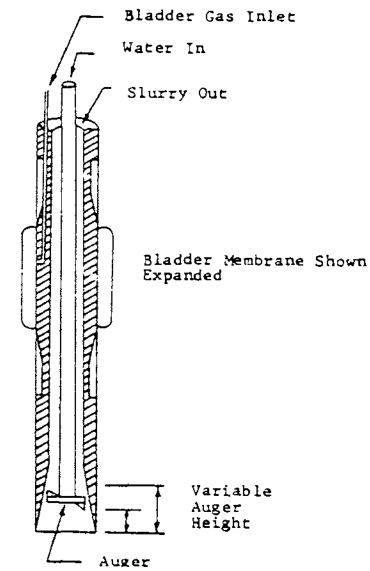


Fig. 21 Schematic Representation of Self Boring Pressure Meter.

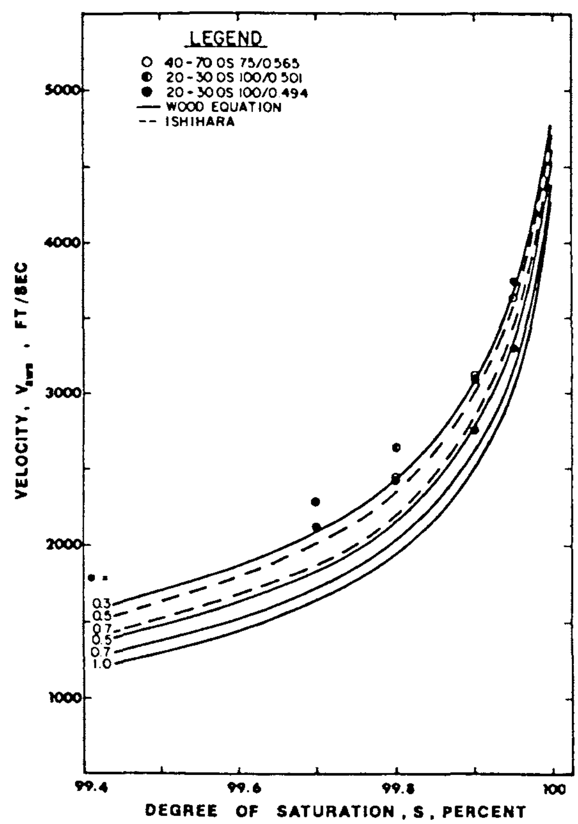


Fig. 22 Fluid Wave Velocity - Degree of Saturation - Void Ratio Relationships for Ottawa Sand in Pulse Chamber (Theoretical Values and Experimental Results) (From Allen et al, 1980).