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DETERMINING DYNAMIC PROPERTIES FOR EMBANKMENT DAMS FROM LABORATORY TESTING

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by

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December 1984

Geotechnical Branch
Division of Research and Laboratory Services
Engineering and Research Center
Denver, Colorado

SI METRIC

UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. Administration.

The research covered by this report was funded under the Bureau of Reclamation "Program Related Engineering and Scientific Studies" allocation, *Soil Performance During Seismic Activity* (E-6), Dam Safety Research Program.

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LETTER SYMBOLS and QUANTITIES

CD	Consolidated-drained triaxial test
CU	Consolidated-undrained triaxial test
C_r	Stress correction factor for cyclic triaxial tests
D	Damping ratio
D_{50}	The diameter at which 50% of the soil is finer
E	Modulus of elasticity
G	Shear modulus
G_{max}	Shear modulus at zero shear strain amplitude
Hz	Hertz
K_c'	Ratio of effective major principal stress to effective minor principal stress
K_o	Coefficient of earth pressure at rest
N	Standard penetration resistance (blows per foot)
N_1	Standard penetration resistance corrected to an overburden pressure equal to 1 ton per square foot
SPT	Standard penetration resistance test
t	Time
γ	Shear strain
ε_a	Axial strain
σ	Stress
$\bar{\sigma}_d$	Deviator stress
$\pm\sigma_{dp}$	Cyclic deviator stress
$\bar{\sigma}_{fc}$	Consolidation stress normal to failure surface due to isotropic consolidation
$\bar{\sigma}_o$	Effective mean principal stress
$\bar{\sigma}_v$	Effective vertical stress
$\bar{\sigma}_1$	Effective major principal stress
$\bar{\sigma}_3$	Effective minor principal stress
ν	Poisson's ratio
τ	Shear stress
$\pm\tau_{cy}$	Cyclic shear stress
τ_{fc}	Shear stress on the failure plane
ϕ	Angle of internal friction
$\frac{\pm\sigma_{dp}}{2\bar{\sigma}_3}$	Cyclic stress ratio

INTRODUCTION

The laboratory phase of a dynamic analysis involves an extensive amount of testing. It includes not only tests normally conducted as part of a major geotechnical investigation, such as the index properties, static strength, consolidation, and deformation tests, but also the tests performed specifically to evaluate the behavior of the soil under dynamic loading — namely, the dynamic properties and cyclic strength tests. This report deals mainly with the dynamic tests currently conducted by the Bureau (Bureau of Reclamation). The test equipment, procedures, and methods of analyses discussed are the result of work done by many researchers and practitioners in the field of soil dynamics.

Prior to laboratory testing, a field exploration and sampling program must be conducted. The scope and techniques employed to determine the in situ conditions and to obtain the samples needed in the laboratory will depend on the type of structure involved (e.g., rolled earth or hydraulic fill, existing or proposed), on the type and condition of soil encountered, and on existing data. In addition to these factors, laboratory and analyses requirements must be considered in planning and performing the field investigations.

If suitable facilities are available, radiography can be used to aid in selecting representative soil samples to be used in laboratory testing. Radiographs of undisturbed samples are used to obtain information such as sample disturbance, stratigraphy, density variations, and presence of gravel size particles [1].*

A discussion of these and other applications of radiography is contained in *Radiography in the Earth Sciences and Soil Mechanics* [2].

EFFECTS OF SOIL SAMPLING ON LABORATORY TEST RESULTS

In obtaining "undisturbed" soil samples, it is inevitable that some degree of disturbance will be introduced: (1) during drilling of boreholes when samplers are forced into the soil, (2) when samples are brought to the surface causing the reduction of in situ stresses and pore water pressure, and (3) during transport, storage, and preparation of samples of testing [3]. This may result in changes of the in situ density of the sample, in structural disturbance of soil fabric, and in changed stress history [4]. Routine soil sampling methods provide adequate undisturbed samples of cohesive materials. Reliable undisturbed samples of cohesionless soils from below the ground-water table or phreatic surface are not obtainable by these methods. The variety of methods for obtaining undisturbed soil samples is discussed elsewhere [5, 6, and 7]. Alternative sampling techniques such as in situ freezing and coring, or dewatering and block sampling followed by laboratory freezing and coring are considered to provide the highest quality undisturbed samples of cohesionless soils. To ensure quality samples, the samples must be drained before freezing. If a sample contains a large percentage of fines, insufficient drainage may cause freezing which has deleterious effects on the quality of the sample [8].

Where appropriate, considerations for relating laboratory test results with in situ soil properties will be discussed in the following sections for the specific dynamic tests involved.

INDEX PROPERTIES TESTS

Index properties tests are necessary for classification and determination of the basic properties of the soils obtained in the field investigations. They include: natural moisture and density, Atterberg limits, gradation, specific gravity, Proctor compaction, and relative density. Since these tests are common to most geotechnical investigations, the procedures for performing them are fairly well standardized and can be found in numerous publications [9 and 10].

The fact that these tests are not discussed in more detail should not be construed as an indication of their lack of importance in a dynamic analysis. To the contrary, the results obtained provide the basis for the selection of samples and the specimen placement conditions for the entire laboratory testing program. Since the dynamic behavior of many soils is strongly influenced by relatively small changes in soil and specimen properties, it is imperative that close attention be given to procedures and techniques in conducting the index properties tests.

STATIC TRIAXIAL TESTS

An important part of a dynamic analysis is the determination of the preearthquake static stresses existing within the embankment and its foundation. The method by which these stresses are calculated incorporates nonlinear stress-strain parameters obtained from the static triaxial test [11, 12, and 13]. Since these parameters are dependent upon the type of triaxial test conducted, the loading conditions applied in the laboratory must be representative of those occurring in the field.

* Numbers in brackets refer to entries in Bibliography.

For the long-term or steady-state condition, excess pore pressure induced during construction and subsequent reservoir filling will have dissipated, allowing the strains to develop under drained conditions. To simulate this condition, the consolidated-drained (CD) triaxial test is used. Other methods of analysis may require additional static triaxial testing. In the equivalent nodal point force method of analysis, for instance, effects of the earthquake are represented by a series of equivalent static forces. In this case, nonlinear stress-strain behavior of soil during the earthquake is determined from consolidated-undrained (CU) triaxial tests.

Specimens tested may be remolded or undisturbed depending on whether the dam is proposed or already constructed. The specimens may be partially saturated or saturated according to the seepage conditions which have developed or are expected to develop. Equipment used for conducting the static triaxial test is shown on figure 1. Detailed discussions of equipment design and test procedures can be found in the "Measurement of Soil Properties in the Triaxial Test" [14], "Earth Manual" [10], and "Laboratory Soils Testing" [15].

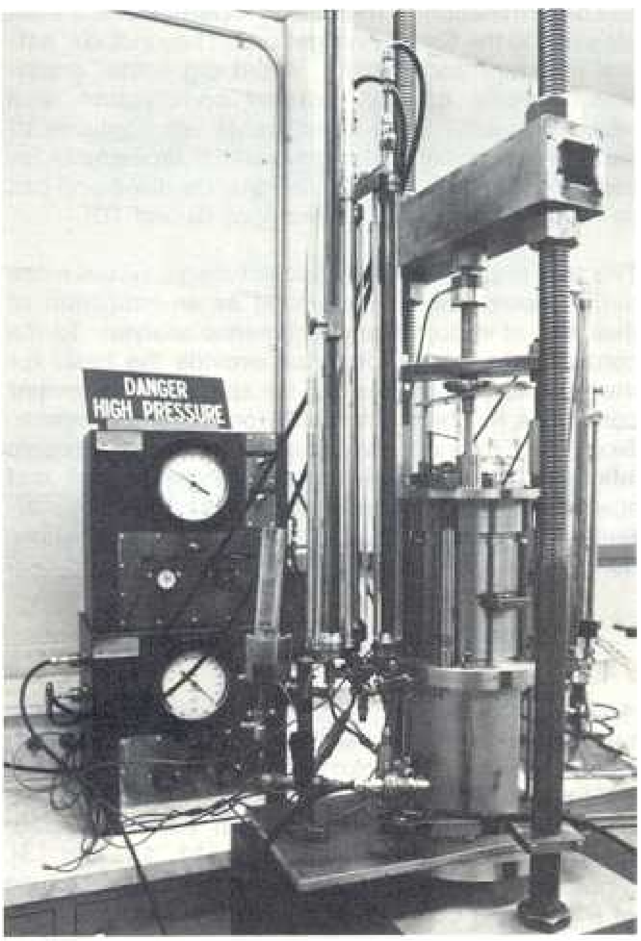


Figure 1 Triaxial shear test equipment. Photo P801-D-80837.

Although the main purpose for conducting static triaxial tests is to determine stress-deformation parameters, soil strength properties are also obtained. Thus, it is possible to perform static stability analysis which may be an important consideration for complete evaluation of dams built before current testing and analysis techniques were available as well as for the design of new dams.

STATIC STRENGTH FOLLOWING CYCLIC LOADING

Static strength following cyclic loading (postcyclic strength testing) is usually used to determine the strength of cohesive soils which is mobilized in the field after cyclic loading conditions.

The procedure used applies a series of repeated cyclic deviator stresses to a soil specimen in an undrained condition. Following application of the desired number of loading cycles, the specimen is statically loaded to failure. Drainage is not permitted any time during the test. Determination of the cyclic loading parameters may be on the basis of a desired number of cycles at a specified stress ratio [16] or the stress level required to reach a failure criteria in a specified number of cycles [17 and 18].

Postcyclic static strengths are then compared to the conventional strength static envelope (as determined by conventional undrained static shear tests on similiar specimens) to determine the loss of strength, if any, due to cyclic effects. The postcyclic strengths also can be compared with the induced cyclic stresses to indicate the capacity of soils to resist earthquake loading.

Prager and Lee [19] found that, for cohesive soils, even when the pore pressure that developed during cyclic loading equaled the cell pressure (100 percent excess pore pressure), specimens were still able to resist static loads immediately following cyclic loading. This behavior differs from that of a cohesionless soil that has liquefied (100 percent excess pore pressure). For the cohesionless soil, appreciable deformation must occur before any resistance to static load is measured.

Postcyclic strength can be related to the peak cyclic strain induced. Tests performed on undisturbed and remolded silty clays resulted in the conclusion that about 80 percent of the static strength remained following cyclic loading provided that the peak cyclic strain induced was less than one-half the failure strain [20]. A reduction in static strength following cycling was apparent also from tests on alluvial clay and loam [18]. However, when an initial static stress of approximately one-half the static strength was applied to the specimen prior to cyclic loading a 20-percent increase in postcyclic strength resulted.

Tests performed by the Bureau upon undisturbed cohesive soils resulted in postcyclic strengths ranging from about 97 to 117 percent of the strength exhibited by specimens that were statically loaded only. In postcyclic tests, specimens were usually subjected to about 15 cycles of loading at a cyclic stress ratio of about 0.35 prior to static loading. Excess pore pressure was not allowed to relieve any time during the test, but specimens were allowed to come to equilibrium for about 30 to 60 minutes following cycling and prior to static loading.

A limited number of tests performed on undisturbed cohesionless soil (ML) resulted in corresponding strength losses from 10 to 50 percent of the static strength of specimens that were not subjected to cyclic loading. The strength loss seemed to be related to the amount of excess pore pressure developed during cyclic loading.

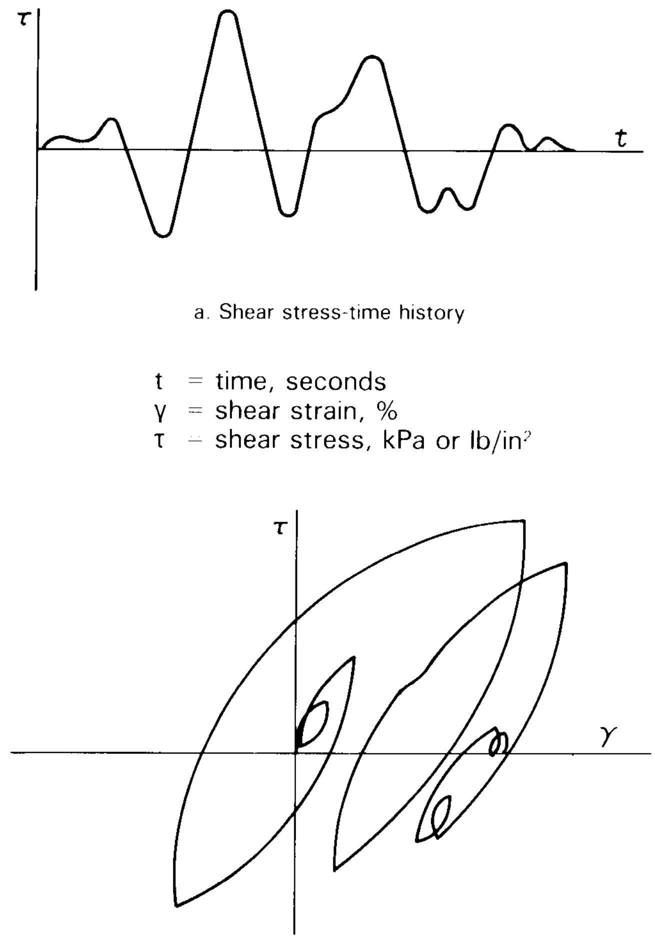
DYNAMIC PROPERTIES TESTS

Introduction

Dynamic properties tests are usually the first group of laboratory tests conducted specifically for the dynamic analysis of an embankment dam. These tests provide soil properties from which the response of the embankment to a given earthquake time history can be calculated.

In a dynamic analysis, usually it is assumed that motion within an embankment and its foundation is produced by horizontally oriented shear waves propagating vertically. These waves produce shear stress reversals of varying magnitude and frequency as shown on figure 2a. Corresponding to this shear-time history would be the stress-strain response of soil as idealized on figure 2b. Each complete cycle of loading and unloading — assuming zero initial shear stress — can be represented by a stress-strain curve having the form of a closed hysteresis loop on figure 3. Each hysteresis loop is fully described by the two dynamic response properties: shear modulus and damping.

The shear modulus G is defined as the equivalent linear secant modulus corresponding to the slope of the line drawn through the end points of the hysteresis loop. By measuring the area outlined by the loop (energy dissipated during the cycle) and the area beneath the "skeleton" curve (potential energy stored during the cycle) shown cross-hatched, the equivalent linear damping ratio D can be calculated using the expression [21 and 22]:



a. Shear stress-time history

t = time, seconds
γ = shear strain, %
τ = shear stress, kPa or lb/in²

b. Idealized shear stress-strain response

Figure 2. — Shear stress-time history and idealized shear stress-shear strain response.

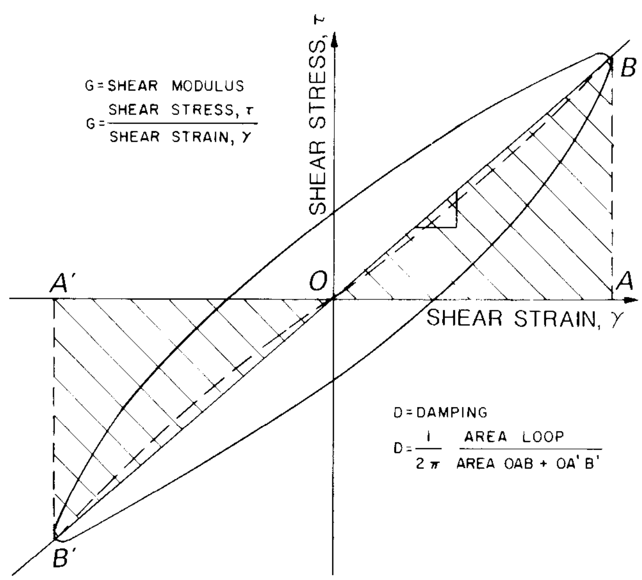


Figure 3. — Hysteretic stress-strain relation [69].

$$D = \frac{1}{2\pi}\left(\frac{\text{area of loop}}{\text{area of } OAB + OA'B'}\right)$$

For computational purposes, the shear modulus and damping ratio are treated as equivalent linear properties, although soil is actually a nonlinear material, exhibiting a decrease in modulus and an increase in damping as the magnitude of the imposed strain increases as noted on figure 4. Therefore, to provide representative values for calculating the in situ behavior, laboratory testing must be conducted over the range of strains anticipated to occur during the earthquakes selected for the dynamic analysis. Five types of laboratory tests [24] used to obtain the dynamic response properties are:

1. Resonant column
2. Cyclic simple shear
3. Cyclic triaxial
4. Cyclic torsional shear
5. Shake table

A sixth method, pulse velocity [25 and 26], can be used to measure the shear wave velocity from which the shear modulus can be calculated. The approximate strain range of each of these tests is shown on figure 5. Of these tests, the Bureau routinely performs the resonant column and cyclic triaxial tests. A brief description of these two tests, and apparatus employed, and an example of the results obtained are presented in the following paragraphs. The cyclic simple shear test also is discussed as it is expected that the Bureau soon will begin routinely performing this test.

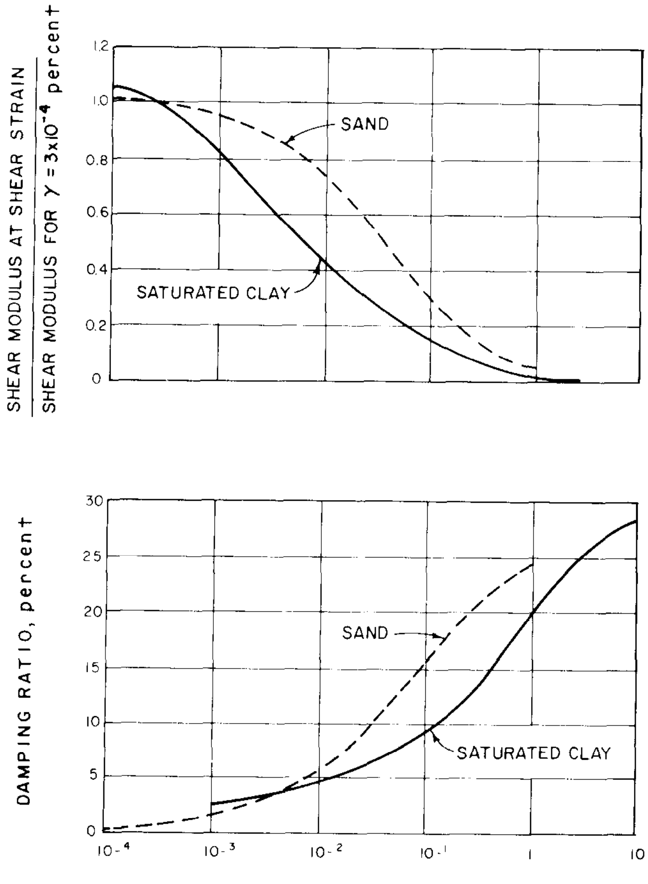


Figure 4. — Typical variation of shear moduli and damping ratio with shear strain (Seed and Idriss [23]).

Resonant Column Tests

In the resonant column test [29], one end of a cylindrical specimen, called the active end, is forced to vibrate in either the torsional or longitudinal mode. The other end, the passive end, may be rigidly fixed or free to vibrate according to the design of the particular apparatus being used. Transducers measure the amplitude of vibration at the active end and at the passive end if it is not rigidly fixed. The system, consisting of specimen, attached platens, and vibration excitation device, is brought to resonance by varying the frequency of excitation. The shear or compression moduli can be calculated by incorporating the resonant frequency and the physical properties of the specimen along with the system calibration factors in the appropriate relationship [30].

A measure of the damping ratio can be obtained by either of two methods: (1) amplitude decay (sometimes called free vibration) or (2) steady-state vibration (sometimes called magnification factor). In the amplitude decay method, the system is forced to vibrate at the resonant frequency and, after the driving power is turned off, the decay in vibration amplitude is recorded as a function of time. This decay, known as the *logarithmic decrement,* and hence the *damping ratio,* can be computed by measuring the amplitude prior to and at different numbers of cycles

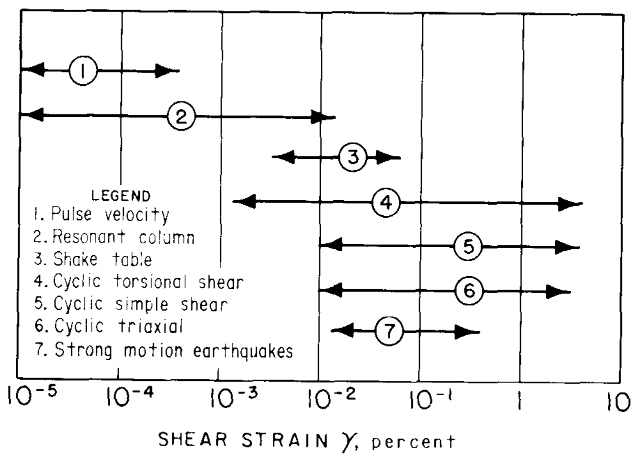


Figure 5. — Approximate strain range of laboratory tests used to obtain dynamic response parameters (Silver and Seed [27]; Woods [28]).

(less than 10) following the interruption of power. The steady-state vibration method involves the measurement of the vibration amplitude of the specimen and the current applied to the vibration excitation device at the system resonant frequency. This latter method is preferred because it is the simpler of the two and provides a measure of damping as a function of strain amplitude. The amplitude decay method is used generally for system calibration and spot checking.

Varying the amplitude of vibration allows the effect of strain on the modulus and damping ratio to be investigated. The strain range obtainable with a given apparatus depends, in part, on the size and stiffness of the specimen and is on the order of 1×10^{-5} to 0.1 percent. The maximum strain capability of a resonant column device can be estimated using the method given by Drnevich [30].

Various types of resonant column equipment have been designed. The three used by the Bureau are the Hardin oscillator, the Drnevich apparatus, and the SDI (Soil Dynamics Instruments) device and are shown on figure 6. Using the Hardin device [31], the specimen is subjected to torsional vibrations under either isotropic or anisotropic static stress conditions. For the Drnevich device, only isotropic static stresses can be simulated; however, both torsional and longitudinal vibrations can be applied, thereby furnishing a measure of Poisson's ratio. The SDI device also simulates only isotropic static stresses but it is capable of accommodating a specimen 150 mm in diameter by 300 mm long, (6- by 12-in), thereby allowing the inclusion of some gravel-size particles in the test specimen.

A more detailed discussion of the resonant column test, including apparatus description, calibration and testing procedures, and aids for data reduction, can be found in Drnevich, et al. [32] and ASTM Designation: D 4015-81 [33]. An example of the results obtained in the resonant column test is shown on figure 7. The average shear strain is reported because of nonlinear torsional strain distribution in the specimens.

The principal advantage of the resonant column test is that the low strain levels correspond to those produced in geophysical tests. This makes it possible to investigate effects of sample disturbance, remolding, removal of large size particles, etc. It has been found that values of shear modulus obtained at low strain levels in laboratory resonant column tests have compared well with values obtained from in situ geophysical tests, especially for young deposits such as shallow deltaic and fluvial sands [34]. For other soils, however, resonant column data may under estimate in situ shear modulus values by about 5 to 50 percent [35, 36 and 37]. This difference may be due to the effects of disturbance from sampling on laboratory test specimens [38], the limited number of test specimens and the limitation on grain sizes that could be tested [39], or on duration of application of the confining pressure to the test specimen [40].

Novak and Kim [41] developed a modified resonant column apparatus to alleviate the difficulties caused by air penetration into the confining medium and sample during long-term tests on cohesive soils. Test results on these soils are presented in a companion paper [42].

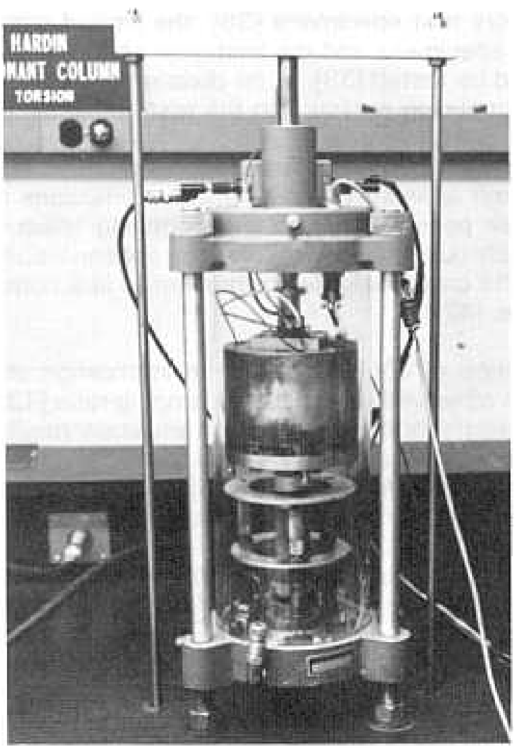
Duration of confining pressure application also has been observed to affect the damping ratio [43]. Suggested methods to estimate in situ shear moduli from resonant column tests may be found in Richart, et al. [40], Stokoe and Lodde [44], Drnevich and Massarsch [45], and Larkin and Taylor [46].

The strains produced by a strong motion earthquake are somewhat greater than those applied in the resonant column apparatus. Therefore, the data must be extended to higher strains. This can be done using "typical" modulus-damping curves (fig. 4) or by additional testing using devices producing higher levels of strain. Since the dynamic material properties, particularly the shear modulus, have a significant affect on the results of a dynamic analysis additional testing to provide data at higher strain levels has been recommended [39].

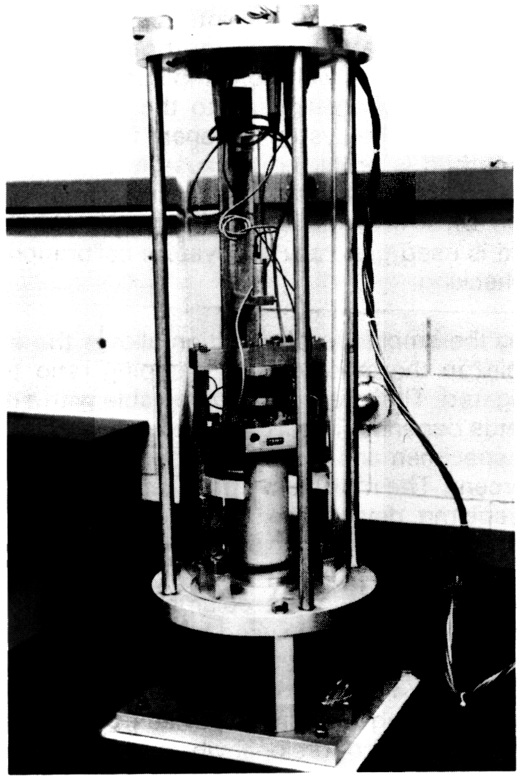
Cyclic Simple Shear Test

Of the available dynamic test procedures, the cyclic simple shear [17] most nearly duplicates the loading conditions thought to occur during an earthquake. In this test, shear strains are applied to the specimen and the shear modulus is calculated from the ratio of the shear stress to the shear strain. Two types of specimen confinement are used: one, an articulated rigid rectangular shear box [47] and the other, a cylindrical rubber membrane which is restricted from deforming laterally by wire reinforcement [48]. In both, K_o (coefficient of earth pressure at rest) static loading conditions are developed.

The device used by the Bureau (fig. 8) was developed by M. L. Silver and uses a wire-wound membrane. Either stress- or strain-controlled loading can be applied. Normally, the test is conducted at a frequency of 0.5 Hz with continuous monitoring of the load and deformation. On the tenth cycle the hysteresis loop (like that shown on fig. 3) is recorded. Using this loop and the corresponding shear stress and strain, the shear modulus and damping ratio are calculated and plotted as shown on figure 9. The response parameters are dependent on the number of stress cycles applied. The greatest change in shear stress occurs



a. Hardin oscillator resonant column



b. Drnevich resonant column

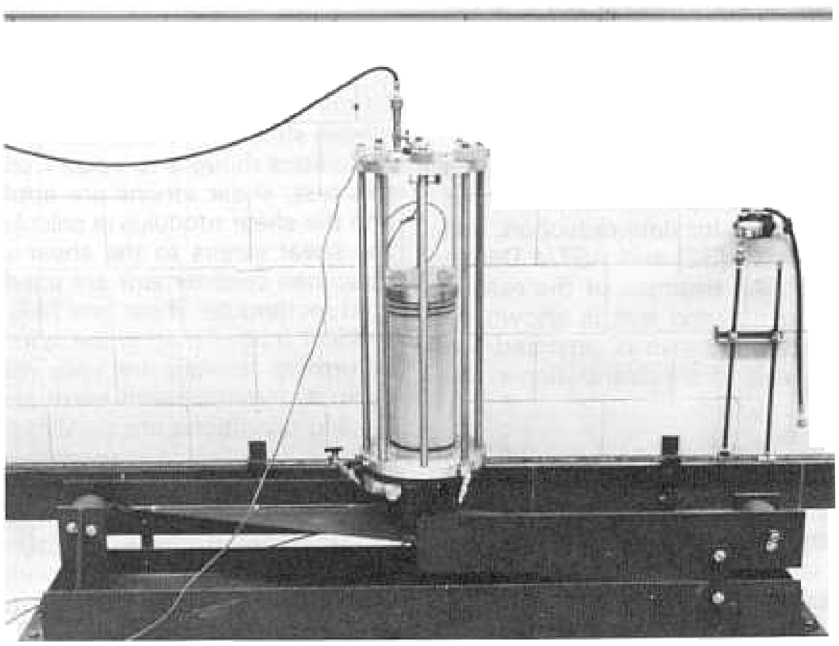


Figure 6. — Resonant column test equipment. Photos a. P801-D-80838, b. P801-D-81839 and c. P801-D-80840.

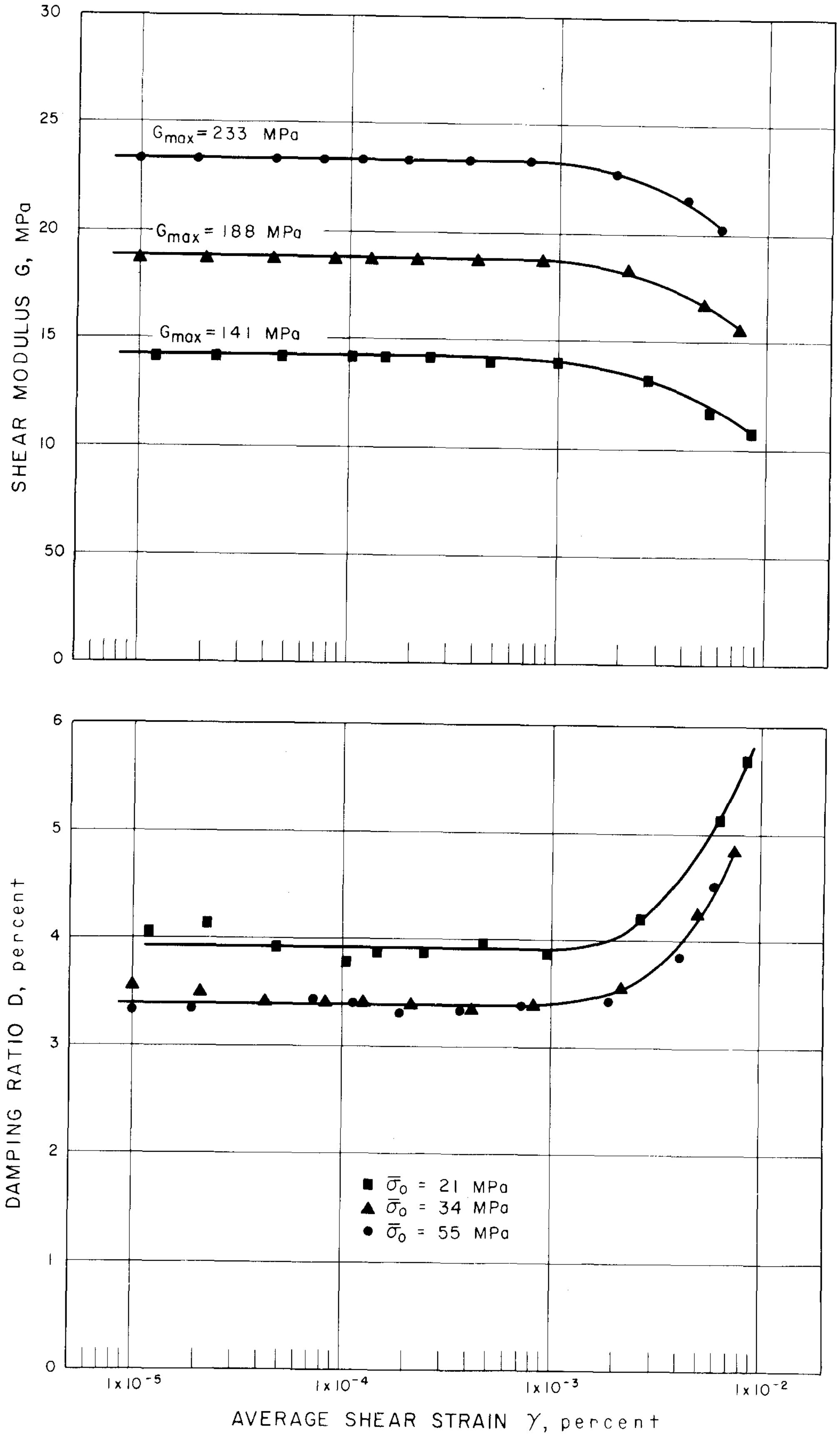


Figure 7. — Example of resonant column test results.

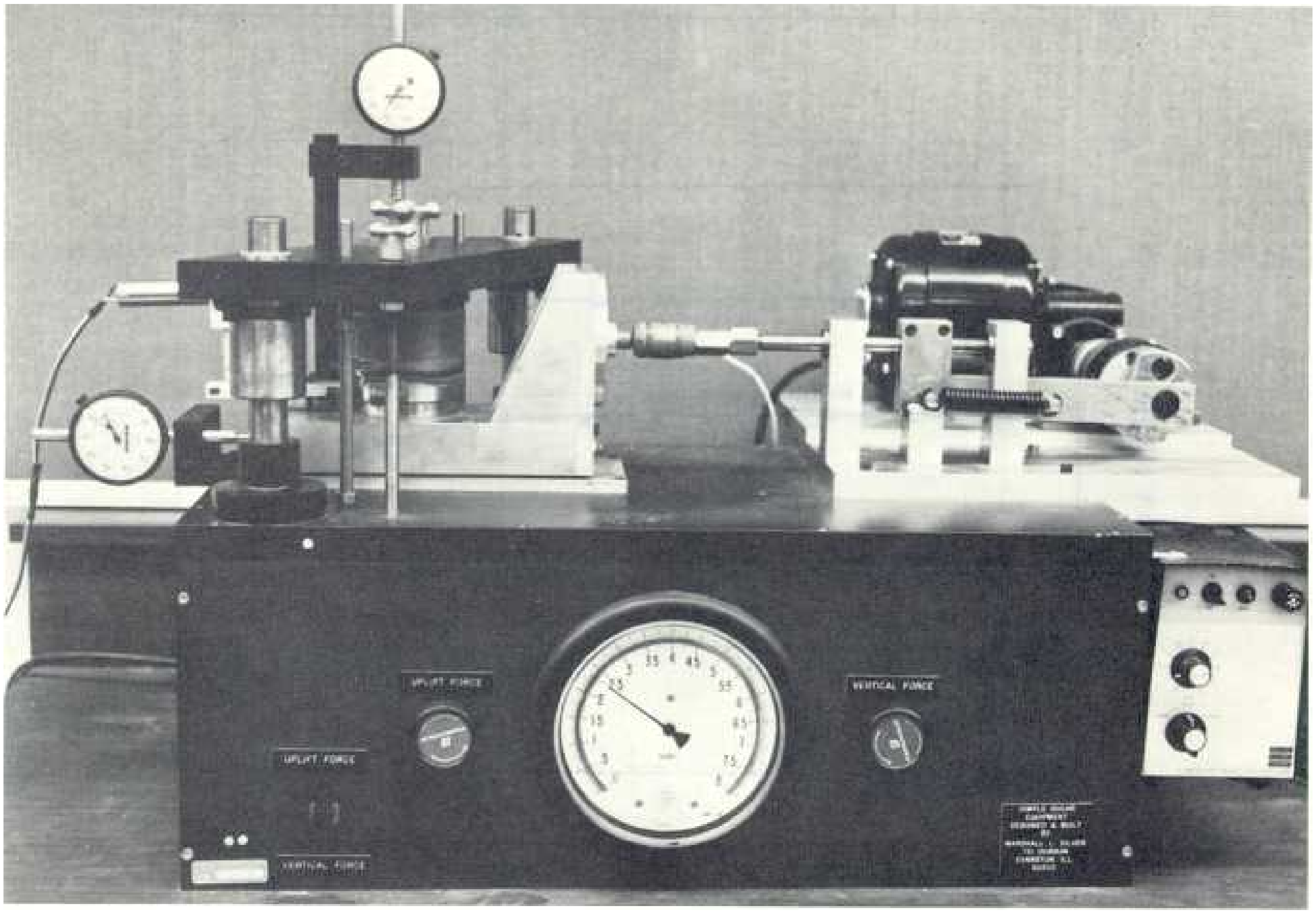


Figure 8. Cyclic simple shear apparatus. Photo P801-D-80841

during the first few cycles. The tenth cycle has been chosen as representative of the number of equivalent significant cycles occurring during a strong motion earthquake [49].

Objections to the cyclic simple shear test have been raised from studies showing that the boundary conditions in the test develop shear stress and strain distributions that are far from uniform and may result in significant under estimations of shear modulus [50, 51, and 52].

System compliance also leads to large errors [53]. However, a close correlation between in situ and laboratory shear modulus has been obtained using the cyclic simple shear test [54], and a close correlation has been found between cyclic simple shear data and data obtained from large-scale shaking table tests [55].

Pyke [56] found good agreement between shear modulus results obtained from tests on Monterey No. 0[1] sand performed with cyclic triaxial and cyclic simple shear apparatus.

From shake table results, Seed [57] suggests the test errors due to stress concentrations may not be as large as previously thought or they are counterbalanced by some other test feature. A more detailed account of available cyclic simple shear equipment and the advantages and disadvantages of the test can be found in Woods [28], and Yoshimi, et al. [58].

Cyclic Triaxial Test For Dynamic Properties

Of the test methods used to evaluate the dynamic shear modulus and damping ratio for earthquake analyses, the cyclic triaxial has had the most widespread usage [59, 60, 61, 62 and 63]. This is due largely to the availability of and familiarity with triaxial equipment. In the past, the cyclic triaxial apparatus was not often used to evaluate the dynamic shear modulus and damping characteristics of soils because reliable stress and strain measurements were difficult to achieve because of the nature of the ap-

[1] Monterey No. 0 sand is obtained from Monterey, Calif. and commonly is used for calibration tests and comparison testing in laboratories.

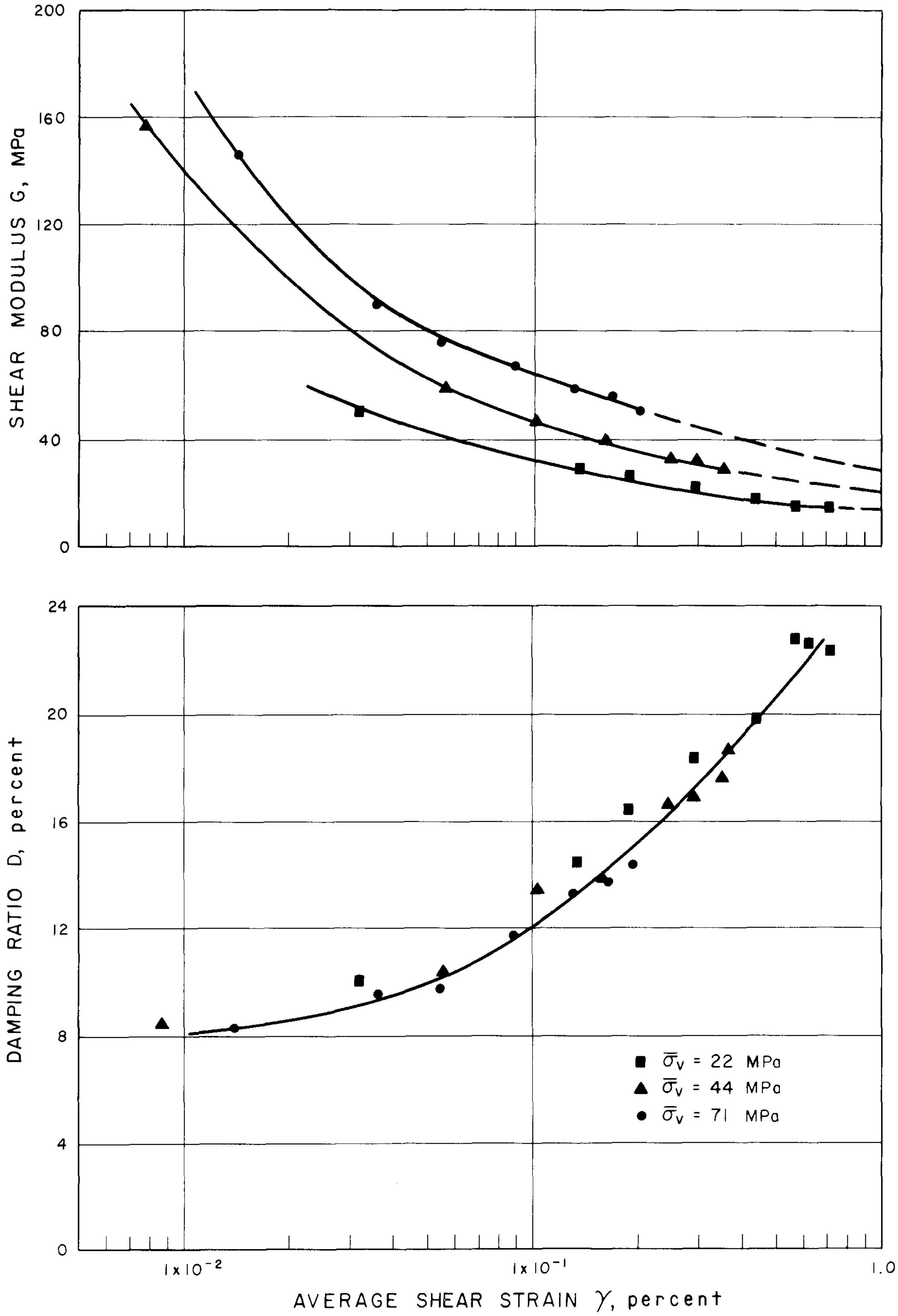


Figure 9. — Example of cyclic simple shear test results.

paratus. Improved apparatus have been developed to address this problem [64, 65, and 66]. The Bureau performed some dynamic properties testing using conventional cyclic triaxial chambers with limited success. Measurements of shear modulus and damping often resulted in a fair amount of scatter. This was assumed to be the result of friction between the O-ring seals and loading piston and the fact that the load cell was located outside the chamber. In the test, a cylindrical specimen is subjected to a series of repetitive axial compression and extension loads while the vertical deformation is monitored. A measure of the compression (Young's) modulus E and axial strain ε_v is thereby obtained. Conversion of these values to shear modulus G and shear strain γ can be made by using the expressions:

$$G = \frac{E}{2\,(1 + \nu)}$$

$$\gamma = \varepsilon_v(1 + \nu)$$

where ν is Poisson's ratio. Depending on soil type and moisture condition, Poisson's ratio is assumed to range from 0.33 for dry clean sands, to 0.5 for soft saturated clays.

The axial load can be either strain or stress controlled. The apparatus shown on figure 10 is a closed-loop electrohydraulic system. It can function in either the strain- or stress-controlled modes and — through the use of electronic feedback signals — automatically maintains the desired level of stress or strain. In addition, any type of wave form can be imposed on the specimen. The pneumatically actuated apparatus shown on figure 11 is a simpler device but not as versatile, generating only a stress-controlled wave. This type of apparatus was introduced in the late 1950's [67]. As originally developed, it provided a rectangular-shaped wave (some modification of the shape could be obtained by valving); however, a recent modification [68] makes it possible to approximate a sine wave.

The test is conducted in a similar manner as the cyclic simple shear. Following isotropic consolidation, the specimen is subjected to a 0.5-Hz sine wave. The load and deformation are monitored continuously and, on the tenth cycle, the hysteresis loop is recorded. Using the above expressions, the shear modulus and shear strain are calculated and presented along with the damping as shown on figure 9.

Synthesis of Test Results

To obtain the variation in shear modulus and damping ratio over the range in shear strain required for the dynamic analysis, the resonant column and the cyclic simple shear or cyclic triaxial results must be com-

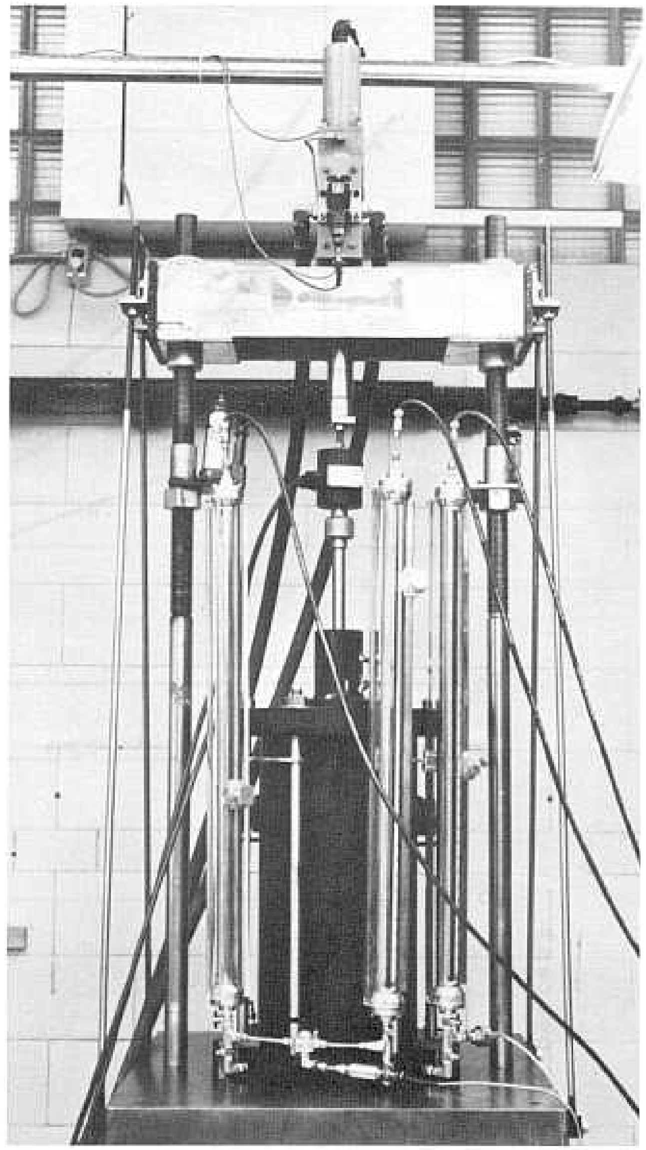


Figure 10. — Closed-loop electrohydraulic cyclic triaxial test apparatus. Photo P801-D-80842.

bined. It has become common practice to relate the shear modulus to the effective mean principal stress $\bar{\sigma}_o$ [23 and 69]. Under the isotropic stress conditions applied in the resonant column and cyclic triaxial tests, the value of $\bar{\sigma}_o$ is equal to the effective confining or lateral pressure. However, in the cyclic simple shear test, the specimen is subjected to anisotropic stresses, and the applied vertical stress, $\bar{\sigma}_v$, must be expressed in terms of $\bar{\sigma}_o$ if the results of the different tests are to be related [62].

This can be done using the relationship:

$$\bar{\sigma}_o = \frac{\bar{\sigma}_v}{3}(1 + 2K_o)$$

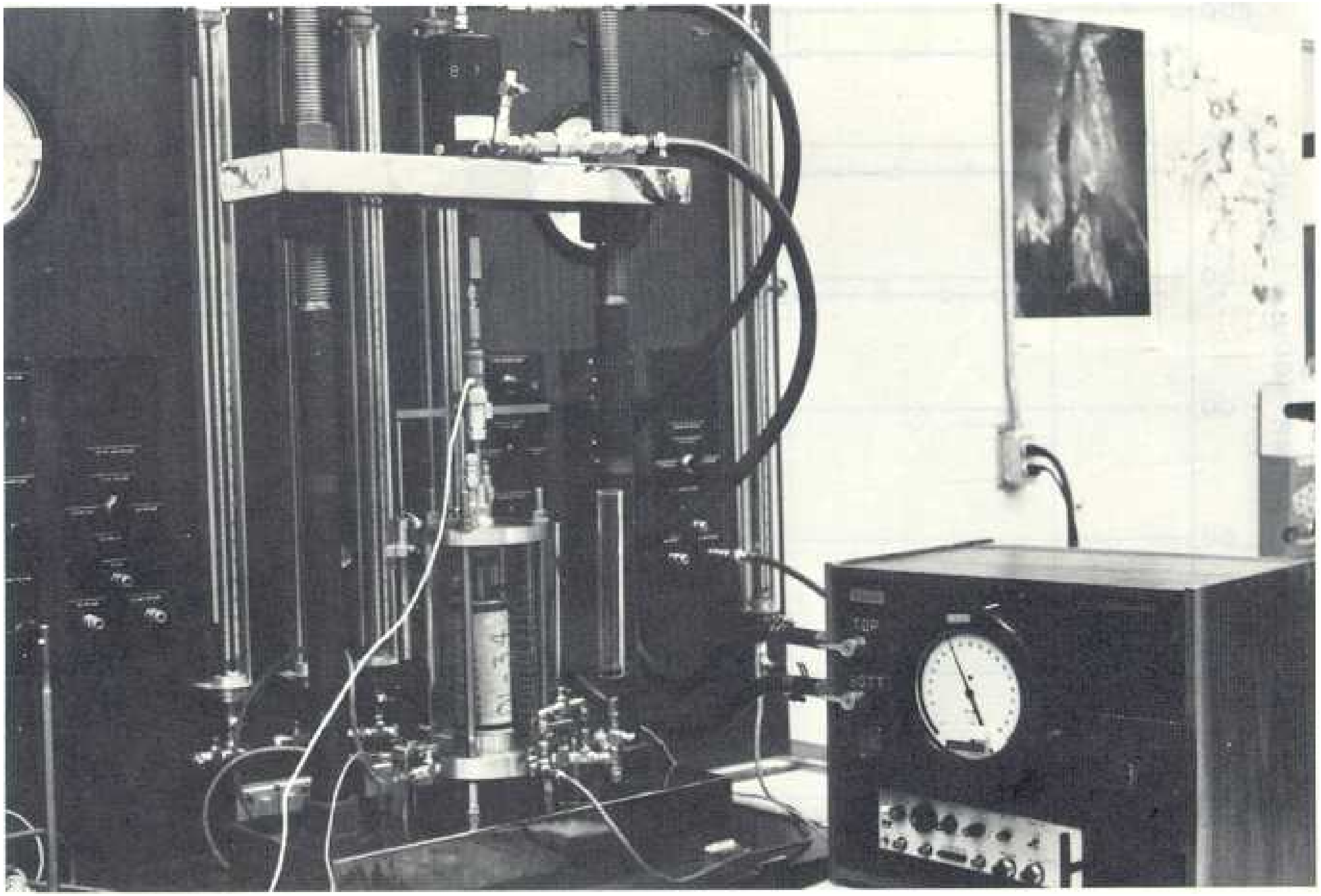


Figure 11 Pneumatic cyclic triaxial test apparatus. Photo P801-D-80843.

Where K_o can be approximated by $1 - \sin \varnothing$ (the empirical formula given by Jaky [70] for normally consolidated soils) where $\varnothing$ is the angle of internal friction obtained from static triaxial tests. It has been found to be more expedient to evaluate the above relationship in terms of values of $\bar{\sigma}_v$ which correspond to $\bar{\sigma}_o$ applied in the resonant column test and apply those stresses in the simple shear test.

After reducing the data to a function of the mean principal stress, the results of the individual tests are combined on a single plot of shear modulus and damping ratio versus single amplitude shear strain shown on figure. 12. The shear modulus is influenced strongly by the effective confining pressure. If the shear modulus is expressed as the ratio of the shear modulus at any strain to the maximum shear modulus G_{max} a single curve indicating the relationship between shear modulus and shear strain is obtained on figure. 13.

The damping ratio also is influenced by the effective confining pressure. However, for sands in particular, the influence of shear strain becomes of more importance as the pressure increases [23]. Such a trend can be observed in the results of resonant column tests (fig. 7). Of the two response properties, shear modulus and damping, damping is the most difficult to measure accurately because of its sensitivity to variations in test equipment and techniques. For these reasons, a curve of average damping ratio versus shear strain generally is used (fig. 12).

Other Methods for Obtaining Dynamic Properties

Other test methods for determining dynamic properties include shaking table tests and various configurations of torsional shear tests. Cyclic torsional simple shear devices, accommodating hollow cylindrical specimens tapered at the top or bottom surface [71, 72, and 73], have been designed with the intent of maintaining more nearly uniform distributions of shear strains throughout the test specimens. Because of factors such as cost and availability of equipment, cost of conducting a test, configuration or size of test specimen, and strain range of the test, these methods have not had as widespread a usage as the tests described previously. Discussions of these methods along with pertinent references can be found in "Soil Behavior Under Earthquake Loading

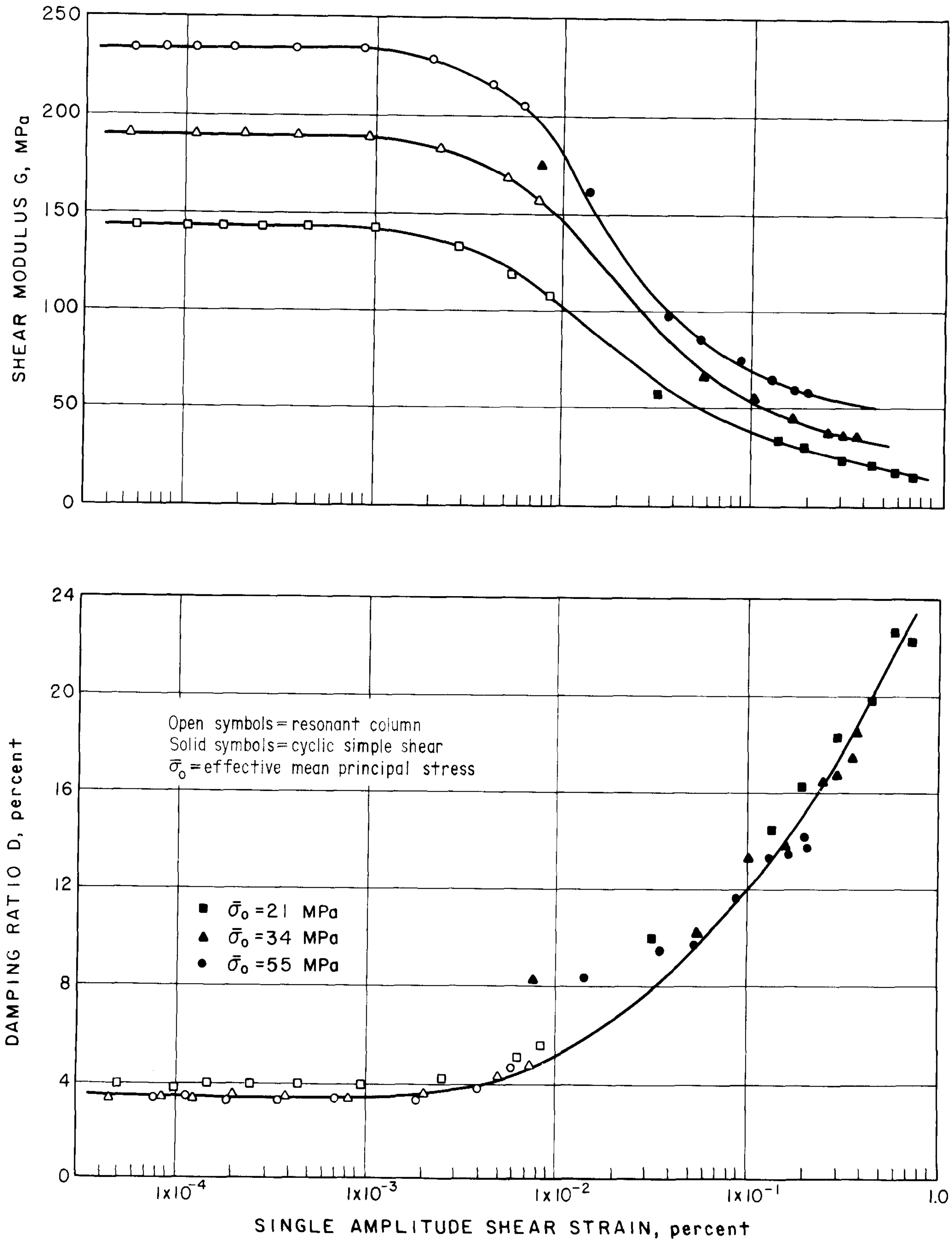


Figure 12. — Shear modulus and damping versus single amplitude shear strain.

Conditions" [24], Woods [28], and Yoshimi, et al. [58]. Tatsuika et al. [74] reported on a comprehensive series of cyclic torsional shear tests on hollow cylindrical specimens of a clean sand. Relationships of the effect of stress ratio and initial shear stress on shear modulus and damping are discussed. Dynamic properties can also be obtained by the use of relationships such as those presented by Hardin and Drnevich [69, 75], Seed and Idriss [23], Edil and Luh [76], Iwasaki, et al. [77], and Sherif and Ishibashi [78]. These relationships are empirical in nature and are based on a large number of laboratory tests. They are useful in conducting a preliminary analysis, planning a testing program, cross-checking or extrapolating test results, and providing data that may not otherwise be obtainable; however, they should be applied judiciously and not considered to be a substitute for a good field and laboratory testing program.

An estimate of the shear modulus has been obtained from field SPT (standard penetration tests). Relationships between the shear modulus at small strains and the N-value obtained from the SPT have been developed in Japan [79] and some reasonable data have been obtained in the United States [80].

CYCLIC STRENGTH TESTS

Introduction

The cyclic strength tests are the second group of dynamic tests intrinsic to the dynamic analysis of an embankment dam. Whereas the dynamic properties tests provide parameters for determining the shear stresses induced in the embankment, the cyclic strength test results are used for evaluating the soil's ability to resist these shear stresses. The evaluation can be on the basis of excess pore pressure development (full or 100 percent pore pressure ratio) or deformation.

There are two reasons for conducting a second series of dynamic tests. First, the dynamic properties are measured at low strains, generally less than 1 percent; whereas for evaluating the soil strength, strains up to 20 percent may be of interest. (A method has been proposed by Silver and Park [81], for predicting the cyclic strength of loose to medium dense sands from cyclic triaxial properties tests.) Second, the cyclic strength is affected significantly by the static stress conditions. The stresses applied in the laboratory must simulate those existing in or beneath the embankment. This involves a wider range in conditions than those employed for determining the response properties which can be related to a mean principal stress.

Laboratory test methods which have been used to determine the dynamic strength include:

Shake table [55, 82, 83, 84, and 85]
Cyclic simple shear [27, 53, 86, 87, and 88]
Cyclic torsional shear [89 and 74, 90, and 91]
Cyclic triaxial [1, 92, 93, 94, 95, 96, 97, 98, and 99]

In addition to these laboratory methods, some studies hae been performed to predict seismic soil strength from SPT [100, 101, and 102].

Because of the wide range of stress conditions which exist in an embankment [103] and the need to test undisturbed as well as remolded specimens, the cyclic triaxial test has had the most widespread usage in the dynamic analysis of embankments dams.

Cyclic Triaxial Test For Liquefaction Potential

The preearthquake static stresses in the embankment dam are simulated in the cyclic triaxial test by consolidating specimens under isotropic $K_c' = 1.0$., and anisotropic $K_c' > 1.0$ stresses. The effective principal stress ratio K_c' is defined as the ratio of the major $\bar{\sigma}_1$ to the minor $\bar{\sigma}_3$ effective principal stress. In the Bureau's current dynamic analysis procedure, only the horizontal shear stresses induced by the earthquake are evaluated. The maximum dynamic shear stresses are generated along horizontal planes and these planes are considered to be the potential failure planes.

In soil elements located near the centerline of an embankment dam or beneath level ground surfaces, static shear stresses are not developed on the horizontal planes — a condition simulated in the triaxial chamber by isotropically consolidating the laboratory specimens. However, in elements located below sloping surfaces, initial shear stresses are induced on the horizontal planes, and simulation of this condition requires anistotropic consolidation at representative K_c' values. Estimation of the appropriate K_c' values can be obtained from the finite element analysis or by the method suggested by Lee and Idriss [104].

Normally, tests are conducted at three different confining pressures with the highest pressure approximating the maximum effective overburden pressure that exists or will exist in the embankment or foundation. Three K_c' values are selected, having values of 1.0, 1.5, and 2.0 being common. A series of tests are conducted for each combination of confining pressure and K_c' value. Since each series may require 3 or 4 specimens, the total number of specimens (27 to 36 per soil type) involved in a single testing program can be significant, especially when more than one type of soil or placement condition must be investigated.

General test procedure. — The initial procedures of the cyclic triaxial test are similar to those followed in conducting a consolidated-undrained static triaxial test. Normally, a cylindrical specimen is placed in a triaxial chamber and a seating pressure of about 35 kPa (5 lb/in^2) is applied to the chamber. The specimen is then back pressured to saturation. The desired isotropic or anisotropic consolidation pressure is applied next. In some cases, depending upon the specific testing program, the specimen is first consolidated and then saturated. Prior to application of cyclic loading, the chamber is partially drained to create an air pocket at the top of the chamber so that the cyclic movements of the loading piston do not cause fluctuation of the chamber pressure. The chamber then is placed in the loading apparatus, and the pore pressure lines are closed to prohibit drainage. A series of uniform, load-controlled axial compressional and extensional stresses are applied and the load, axial deformation, and pore pressure are monitored. The cyclic load can be applied by either an electrohydraulic (fig. 10) or a pneumatic (fig. 11) apparatus. The Bureau uses a pneumatic loader manufactured by (CKC) Soil Engineering Equipment Company for cyclic testing performed on 50-mm diameter by 125-mm high (2-by-5-in) specimens. This loader supplies both a cyclic and constant air pressure to a double acting piston mounted above the triaxial chamber. An Exact waveform generator is used to produce an electrical signal which is converted to a pneumatic signal in this apparatus. Excellent results have been obtained with this apparatus. For larger specimens [150-mm diameter by 375-mm high (6-by 15 in)], an electrohydraulic system is currently in the developmental stage.

The electrohydraulic system uses a Pegasus control unit to control the hydraulic actuator which is mounted above the triaxial chamber, and is capable of producing a force up to ±900 kg (2,000 lb) at a frequency at 1 Hz. A more detailed discussion, including cyclic triaxial equipment standards and testing methodologies, can be found in Silver [105 and 106], and "Laboratory Soils Testing" [15].

The cyclic triaxial testing techniques employed may have considerable effect on the results of the test. While the magnitude of the effects of the loading wave form and frequency are dependent on the soil type, a sine wave loading form generally has been found to produce higher strengths than triangular or rectangular wave forms [107, 108, and 109]. The cyclic frequency, within a range of about 0.02 to 28 Hz, has shown to have little effect on sands [110] but will significantly affect the behavior of clays [88, 111, and 112]. Since clays tend to creep under sustained load, Lee and Vernese [110] conclude that the use of square wave forms and lower frequencies increase the duration of sustained load per cycle leading to quicker strength deterioration. The Bureau laboratory, like many others in the United States [113], has adopted a 1-Hz sine wave cyclic loading.

There has been considerable study on the effects of the various methods of specimen reconstitution on the measured cyclic strength of soils, as well as to the effects of testing reconstituted specimens as opposed to undisturbed specimens. While the magnitude of the difference depends on the soil type, it has been shown that different methods of reconstituting specimens to the same density may result in significantly different cyclic strengths [114, 115, and 116].

The Bureau uses various methods of preparing compacted soil specimens. For 50-mm (2-in) diameter specimens, cohesive soils usually are compacted in 10 layers using a split mold and a uniform compaction effort. For larger diameter specimens, the number of layers depends upon the maximum particle size. Coarse-grained cohesionless soils are placed using a cylinder with a 2.00-mm sieve (U.S.A. standard series No. 10 screen) mounted in its base. The soil is poured into the cylinder and then flows through the screen into a split mold. The sides of the split mold are physically tapped during pouring while the screened cylinder is slowly raised. Alternatively, the Bureau uses an under compaction technique as described by Ladd [117].

Studies have been made to determine the effect of the degree of saturation on the cyclic response of compacted sand specimens. Results show that soils must be at least 99 percent saturated to achieve liquefaction in less than 1000 cycles of loading for any stress ratio studied [118].

It also has been shown that undisturbed specimens will almost always be stronger than reconstituted specimens [116, 119, 120 and 121]. This difference in strength may be attributed to, among other factors, the soil grain fabric stress history, in situ K_o conditions, and the age of the soil deposit [122, 123, and 124]. For these reasons, it is advised that the best possible undisturbed samples be obtained for cyclic testing [125]. A comprehensive review of factors affecting cyclic triaxial test results can be found in Townsend [109].

There are certain limitations inherent in the cyclic triaxial test configuration for simulating the stress and strain conditions of a soil element in situ during an earthquake. For example, stress concentrations exist at the cap and base of the laboratory specimen, a 90° rotation of the direction of the major principal stress occurs during the two halves of the loading

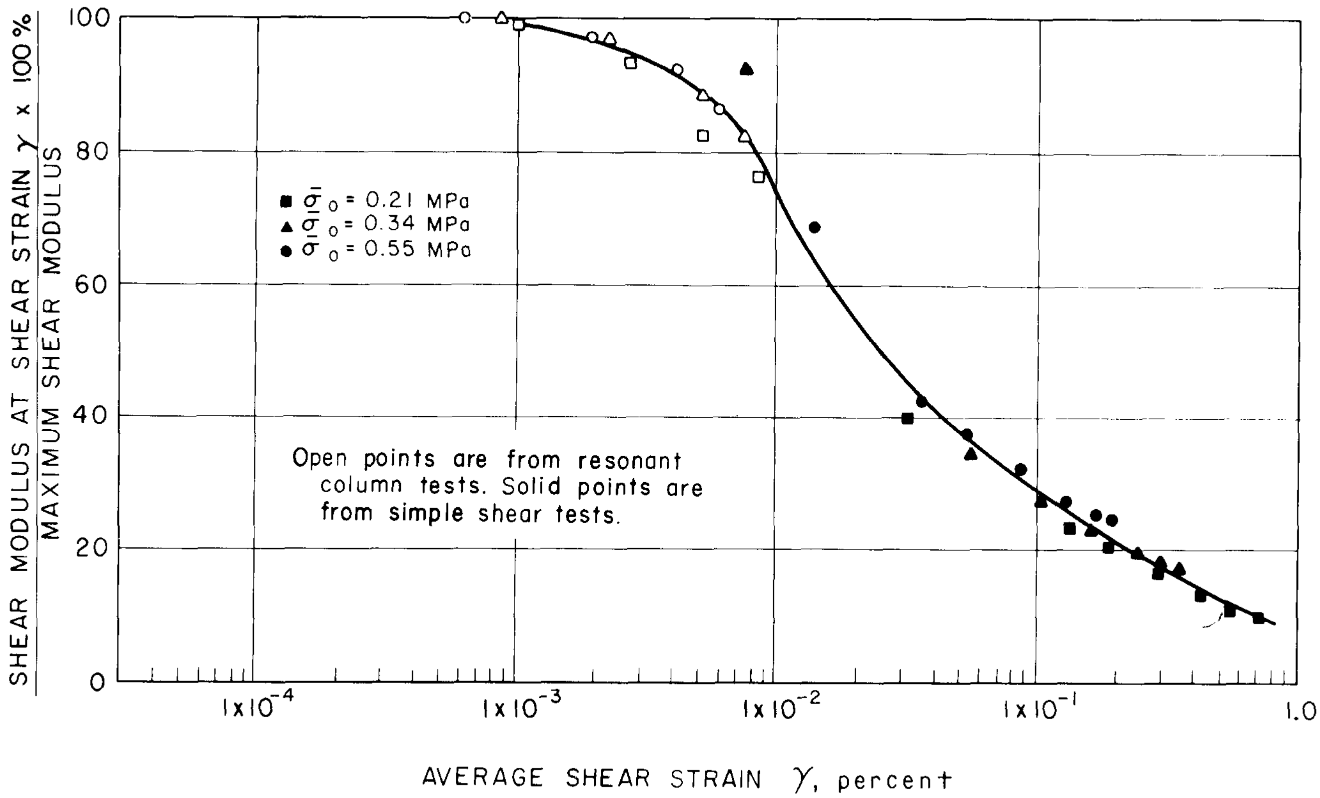


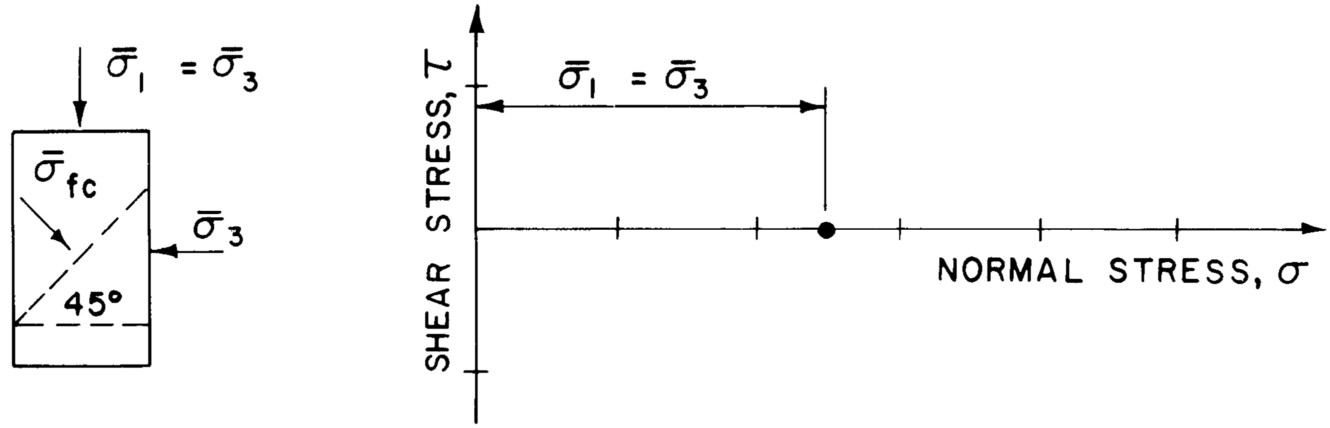
Figure 13. — Reduction of shear modulus with single amplitude shear strain.

cycle, and necking may develop in the specimen and invalidate data beyond that point [58 and 125]. Despite these limitations, and consideration for the factors affecting test results, carefully conducted cyclic triaxial tests can provide data on the cyclic behavior of soils with a degree of accuracy adequate for meaningful evaluations of cyclic strength [125]. A more valid representation of field conditions is simulated in the anistropic cyclic triaxial test for soil samples where large initial shear stresses exist on horizontal planes [123, 126, and 127].

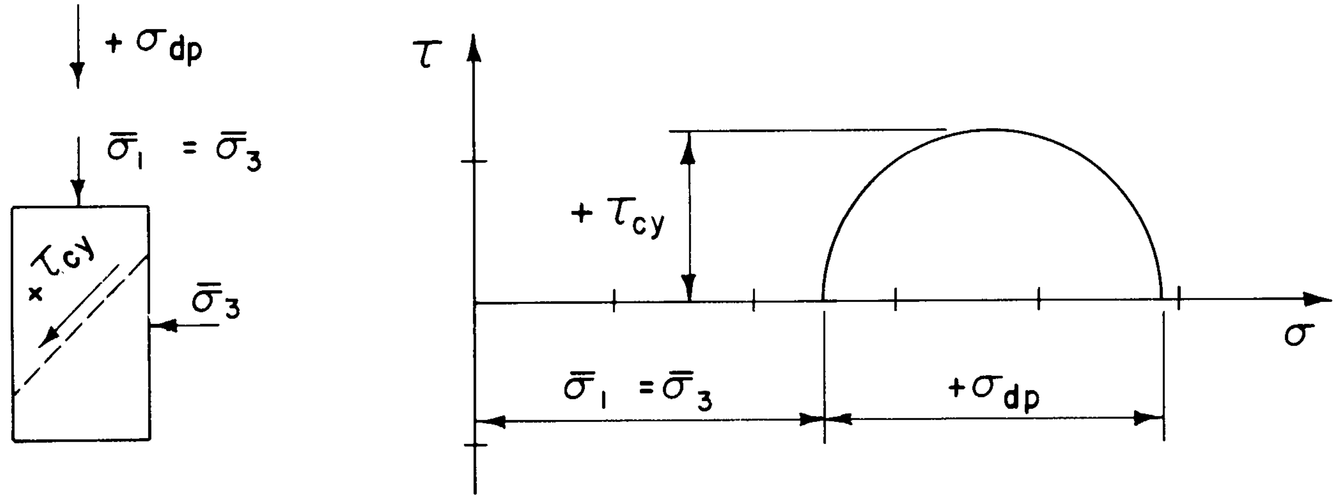
Many of the procedures followed in conducting isotropic and anisotropic tests are similar. However, significant differences exist between the two types of tests, particularly in analyzing the results, and they are discussed separately.

Isotropic (K_c' = 1.0) tests. — The isotropic tests are used to simulate the behavior of elements of soil that have zero initial shear stress on horizontal planes. In figure 14, under isotropic consolidation the major $\bar{\sigma}_1$ and minor $\bar{\sigma}_3$ effective principal stresses are equal and shear stresses are not developed in the specimen. The consolidation stress that develops normal to the failure plane is shown as $\bar{\sigma}_{fc}$. As the compressive portion of the cyclic load $+\sigma_{dp}$ is applied, the maximum cyclic shear stress $+\tau_{cy}$ developed on a 45° plane in the specimen is equal to $\sigma_{dp}/2$. During application of the extension portion of the cyclic load $-\sigma_{dp}$, the principal stresses are reversed and $-\tau_{cy}$ is developed. To prevent the top platen from being lifted from the specimen, the maximum uplift stress that can be applied must not be allowed to exceed the consolidation pressure $\bar{\sigma}_3$. The relevance of these loading conditions to those produced by an earthquake was discussed by Lee and Seed [128].

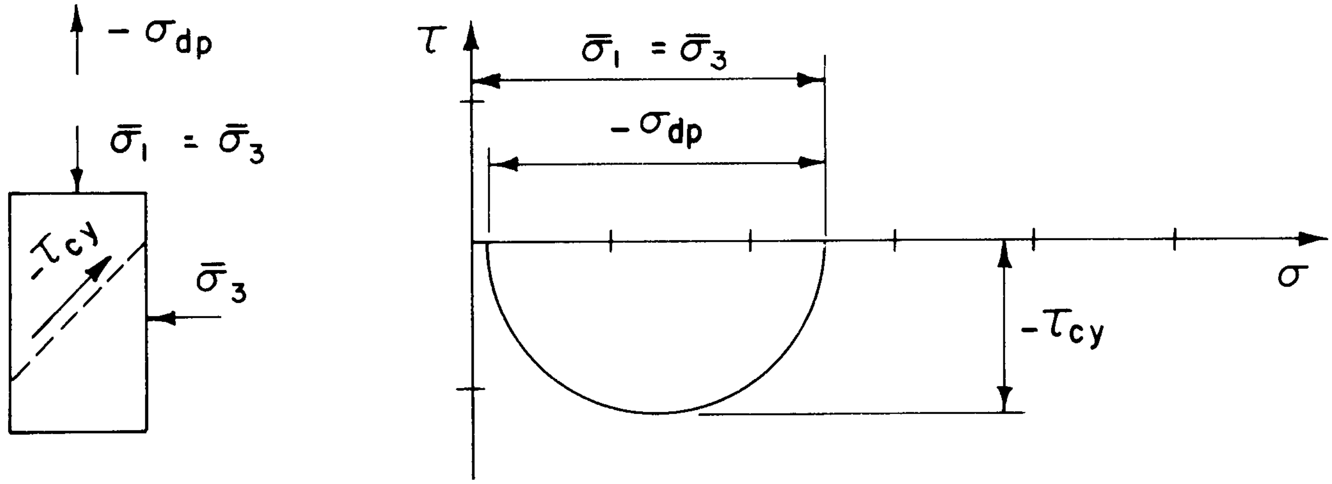
An example of the type of record obtained during a test is shown on figure 15. As can be seen, the pore pressure increases as the number of cycles increases until it equals the applied confining pressure. At this point, termed full or 100 percent pore pressure ratio, the effective stress is reduced to zero, and the strain, in low to medium dense sands, rapidly increases. While full or 100 percent pore pressure ratio (sometimes referred to as "initial liquefaction") has been used successfully as a failure criteria for saturated, low to medium dense sands [126 and 127], its applicability for dense sands or fine-grained soils is questionable as difficulties may be encountered in obtaining accurate pore pressure measurements. For dense sands or fine-grained soils, a failure criterion based on the developed axial strain has been adopted. The number of cycles required to obtain a given magnitude of strain can be determined by two different methods. For example, a single amplitude strain of



a. Isotropic consolidation conditions



b. Application of compressive cyclic stress



c. Application of extensional cyclic stress

Figure 14. — Isotropically consolidated cyclic triaxial strength test.

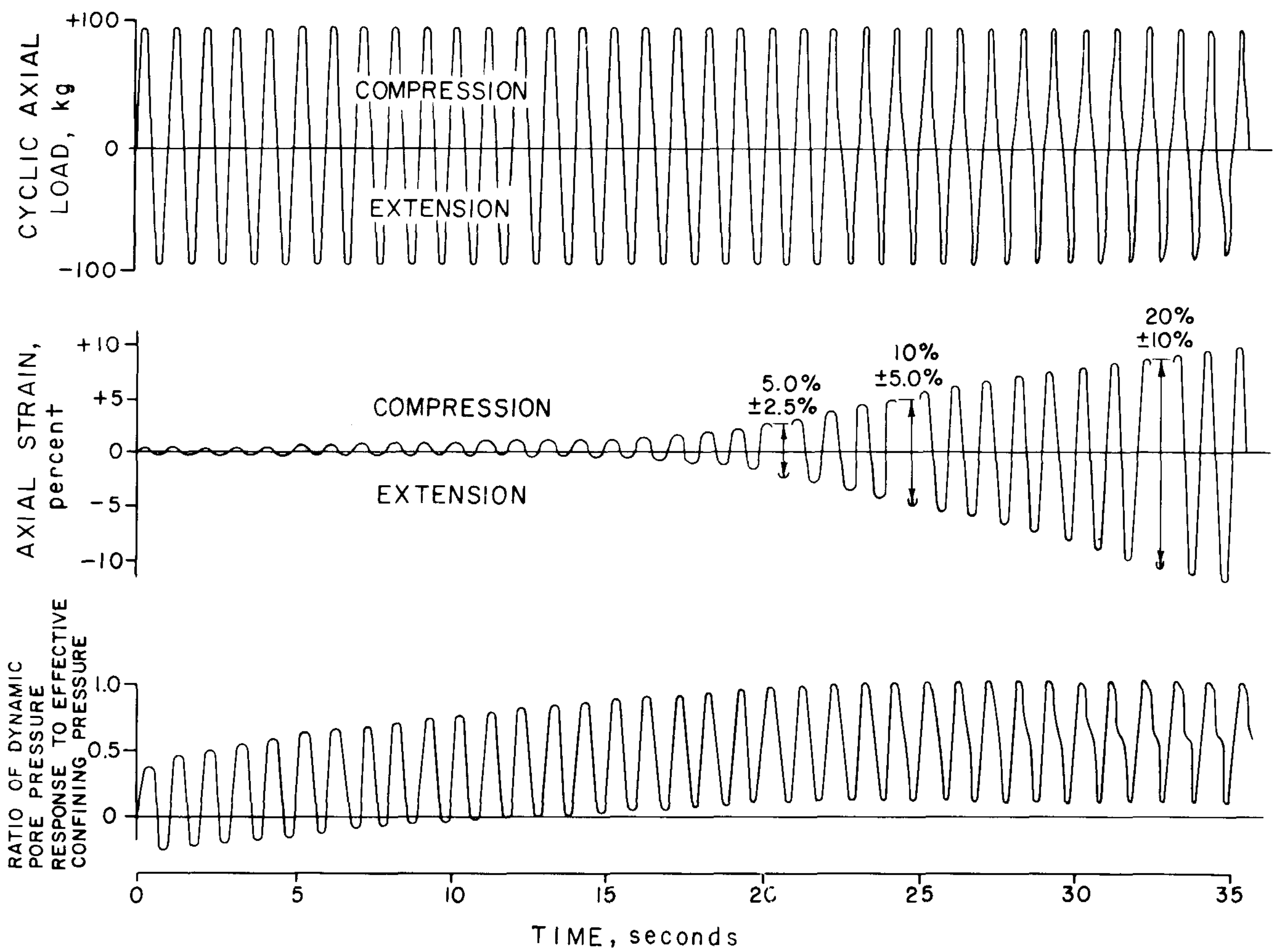


Figure 15. — Example of the type of record obtained from a cyclic triaxial test on an isotropically consolidated specimen.

2.5 percent can be defined as the point where the double amplitude strain (peak-to-peak) equals 5.0 percent, or where the maximum excursion (either in extension or compression) from the static condition equals 2.5 percent [129]. The Bureau has adopted the former definition.

As a first step in analyzing the results, the double amplitude strains and the increase in pore pressure are plotted versus the number of cycles applied. The pore pressure values are expressed as the ratio of the dynamic pore pressure to the effective confining pressure. Specimens subjected to the same effective confining pressure and effective principal stress ratio may be summarized in a single plot as shown on figure 16. These same data area replotted on figure 17 to show the number of cycles to obtain a given level of strain (or 100 percent pore pressure ratio) in terms of the cyclic stress ratio $\pm\ \sigma_{dp}/2\ \bar{\sigma}_3$.

Significant differences exist between the stress conditions developed in the isotopically consolidated laboratory specimens and the insitu soil elements [87]. Some limitations of the cyclic triaxial test in simulating the stress and strain conditions on soil elements in the field during earthquakes were mentioned earlier. In addition, the cyclic stress ratio used to express cyclic triaxial test results, $\pm\sigma_{dp}/2\ \bar{\sigma}_3$, is the ratio of the maximum shear stress developed in the test specimen to the effective confining pressure. A stress ratio representative of field simple shear conditions is the ratio of the shear stress developed on the horizontal plane to the initial effective overburden pressure. Furthermore, conventional cyclic triaxial tests impose only unidirectional shaking, while in situ soil elements are subjected to multidirectional shaking during earthquakes. Several methods have been proposed to account for these differences [55, 57, 86, 130, 131, 132, 133, and 134].

While differing in form Kramer et. al. [95], indicate for clean, normally consolidated sands the laboratory stress ratio should be multiplied by a factor, c_r, ranging from about 0.5 to 0.7 with an average value of 0.57 as recommended by Seed [125] being commonly used. For clayey soils [88] and for anisotrop-

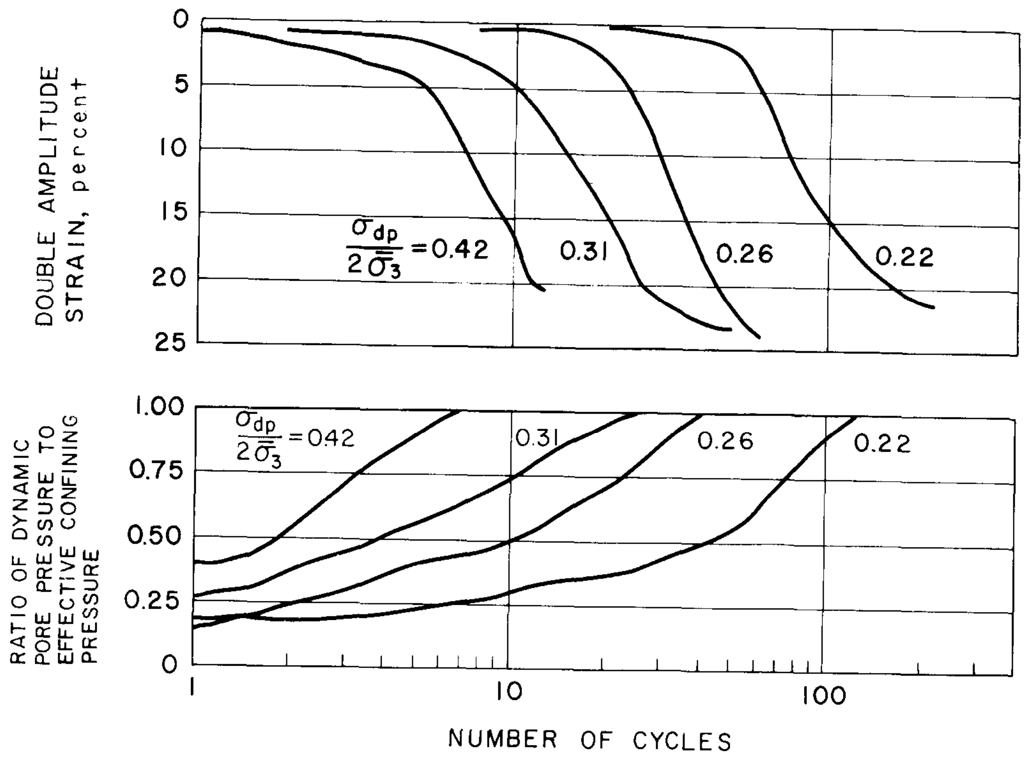


Figure 16. — Results of cyclic triaxial tests on isotropically consolidated specimens.

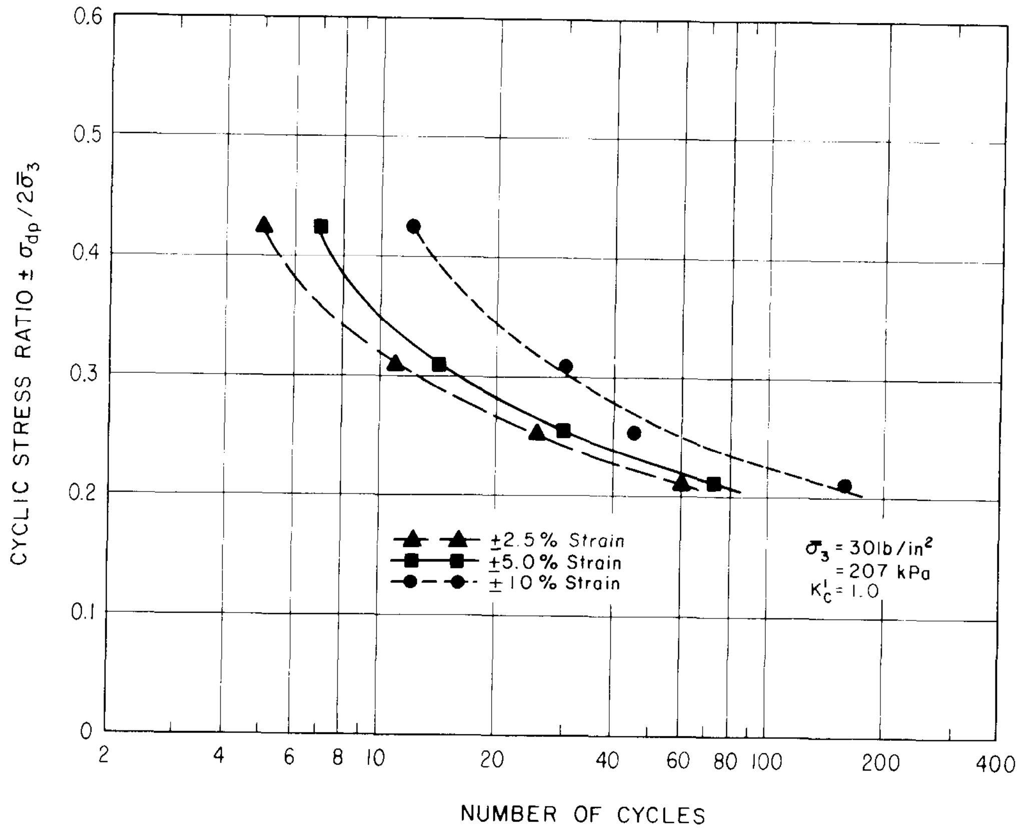


Figure 17. — Example of an isotropically consolidated cyclic triaxial strength test.

ically consolidated soils with $K_c' \geq 1.5$ [125], c_r is approximately equal to 1.0. For granular soils, it may be appropriate to correct the data for membrane penetration effects [135, 136, 137, and 138].

Anistotropic ($K_c' > 1.0$) tests. — The strength of in situ elements of soil acted upon by initial static shear stresses can be evaluated by anisotropically consolidating the triaxial specimens. Application of a confining pressure $\bar{\sigma}_3$ and an axial deviator stress $\bar{\sigma}_d = \bar{\sigma}_1 - \bar{\sigma}_3$ creates a normal stress $\bar{\sigma}_{fc}$ and a shear stress τ_{fc} on the potential failure plane as shown on figure 18. Assuming the failure plane to be inclined at an angle of $45° + \varnothing'/2$ to the horizontal, the stresses can be calculated by the following relationships:

$$\bar{\sigma}_{fc} = \bar{\sigma}_3 + \bar{\sigma}_d\,[\cos 45° + (\varnothing'/]^2$$

$$\tau_{fc} = 1/2\ \bar{\sigma}_d\,(\cos \varnothing')$$

Application of the compressive portion of the cyclic stress $+\sigma_{dp}$ induces an additional shear stress τ_{cy} on the failure plane which is calculated as:

$$\tau_{cy} = 1/2\ \sigma_{dp}\,(\cos \varnothing')$$

Depending on the anisotropic stress ratio and the magnitude of the cyclic stress, there may or may not be a shear stress reversal during the extension portion of the cyclic stress. On figure 18, $-\sigma_{dp}$ is greater than the initial deviator stress σ_{dp} and a shear stress reversal occurs.

An example of the type of test record obtained on an anisotropically consolidated specimen is shown on figure 19. The behavior of this specimen differs considerably from that of an isotropically consolidated specimen because of the initial static shear stress. As a result, the maximum compressional strain is used rather than one-half of the peak-to-peak value. Other investigators have reported the strain as the variation between zero strain and the mean of the peak-to-peak [139]. Test data are plotted to show the number of cycles required to obtain selected levels of strain (and 100 percent pore pressure ratio, if applicable) for various cyclic stress levels applied. Cyclic stress can be expressed as a function of the peak deviator stress during cyclic loading, or as a ratio of the cyclic shear stress τ_{cy} to the normal stress on the failure plane during consolidation $\bar{\sigma}_{fc}$ as shown on figure 20.

Cyclic strength results. — The results of the isotropic and anisotropic tests can be summarized in a variety of ways to show the effect of the combined static and cyclic stresses on the cyclic strength. From plots shown on figure 21, the cyclic stresses required to cause a selected percent strain in a specific number of cycles can be determined. By combining these results with those obtained under different consolidation pressures, the effect of confining pressure can be seen on figure 22.

EVALUATION OF LIQUEFACTION POTENTIAL USING STANDARD PENETRATION RESISTANCE

There are essentially two methods available for evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking. The first is by laboratory testing performed on undisturbed soil samples. Because of the difficulty in obtaining samples that accurately represent in situ conditions, it was desirous to establish a method to correlate liquefaction potential to field observation techniques. Soil index parameters such as standard penetration resistance, cone penetration resistance, electrical properties, and shear wave velocity provide the basis for the second evaluative technique, but of particular interest is standard penetration resistance because of its widespread use and availability of field performance data. The evaluation of research regarding correlative techniques is well described by Seed, et al. [140].

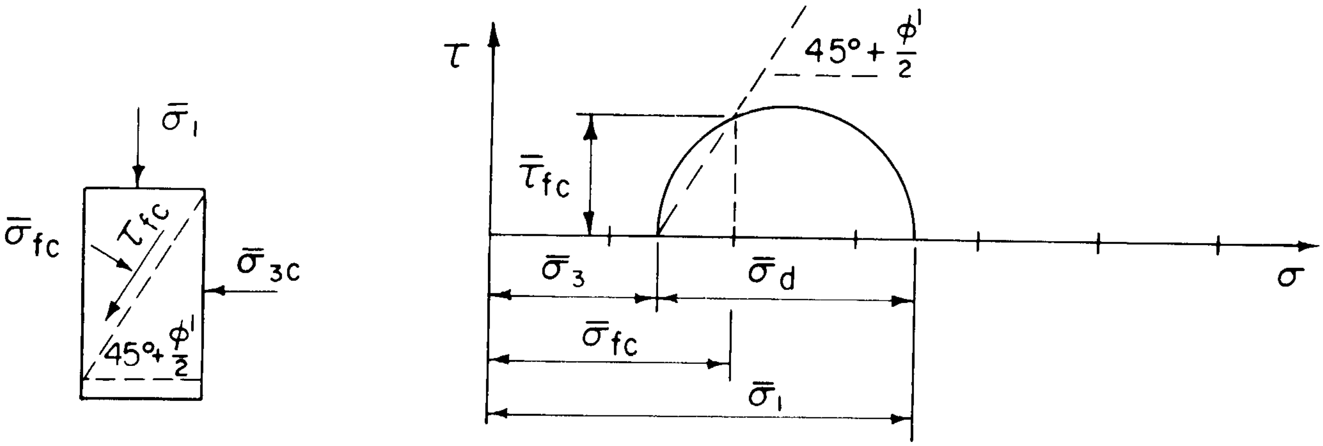
Most techniques were founded on the concept that it was possible to differentiate between liquefiable and nonliquefiable conditions based upon the standard penetration resistance of sand deposits because the factors tending to improve liquefaction resistance also tended to improve SPT resistance [100].

Tatsuoka, et al. [141] developed a correlation that included the effects of grain size upon relative density and *N*-values.

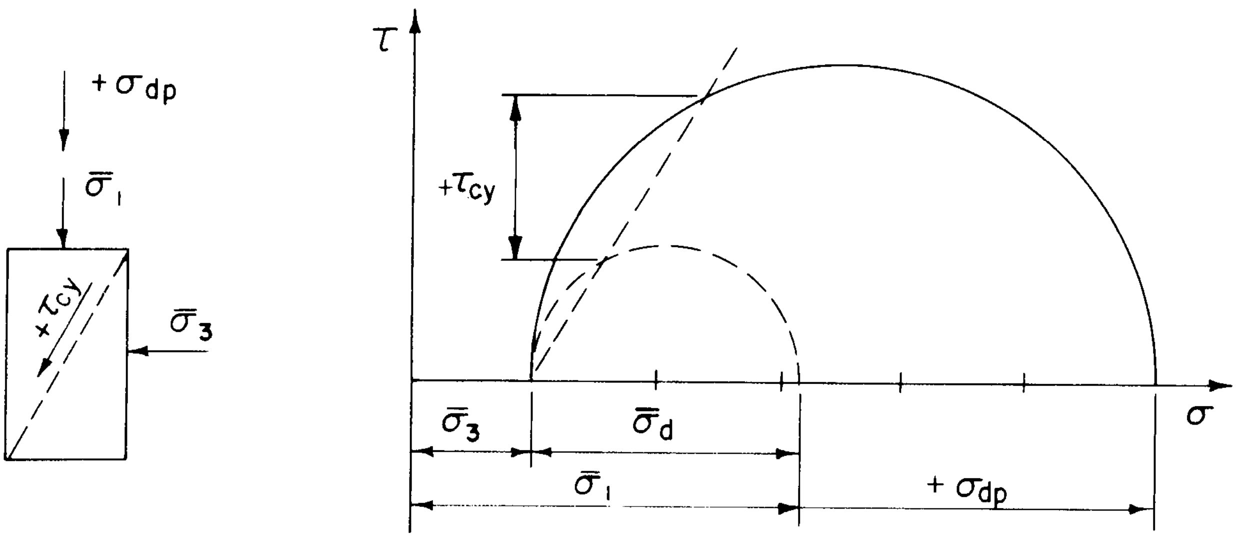
Seed, et al. [142] presented the results of an analysis of available data as shown on figure 23. This figure establishes it is likely that soils falling to the right of the boundary line would not be susceptible to liquefaction.

In 1983, Seed, et al. [140] updated conclusions based upon a growing data base. The data presented on figure 23 were supported by the latest available information. Figure 24 [140] presents the most up-to-date compilation of data available. All the data represent sites subjected to earthquake of magnitude 7.5 on the Richter scale. Data plotted on figure 24 are for clean sands, $D_{50} > 0.25$ mm.

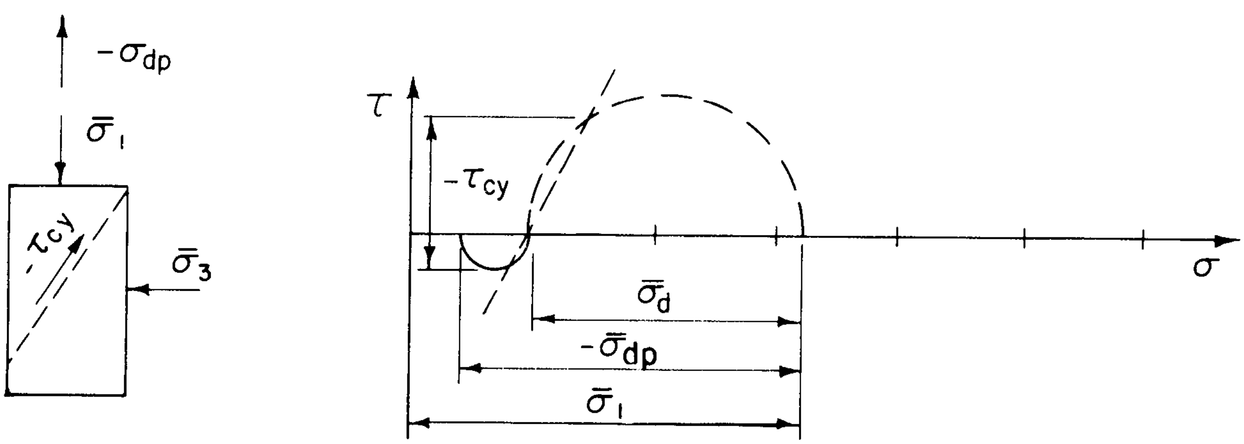
For silty sands ($D_{50} < 0.15$ mm), similar data are plotted on figure 25. It can be seen that the curve for the silty sand is higher than that for clean sands. The two curves are parallel and thus the boundary previously established for sands can be used for silty sands, provided the N_1 value for the silty sand site is increased by 7.5 before using figure 25.



a. Anisotropic consolidation conditions



b. Application of compressive cyclic stress



c. Application of extensional cyclic stress

Figure 18. — Anisotropically consolidated cyclic triaxial strength test.

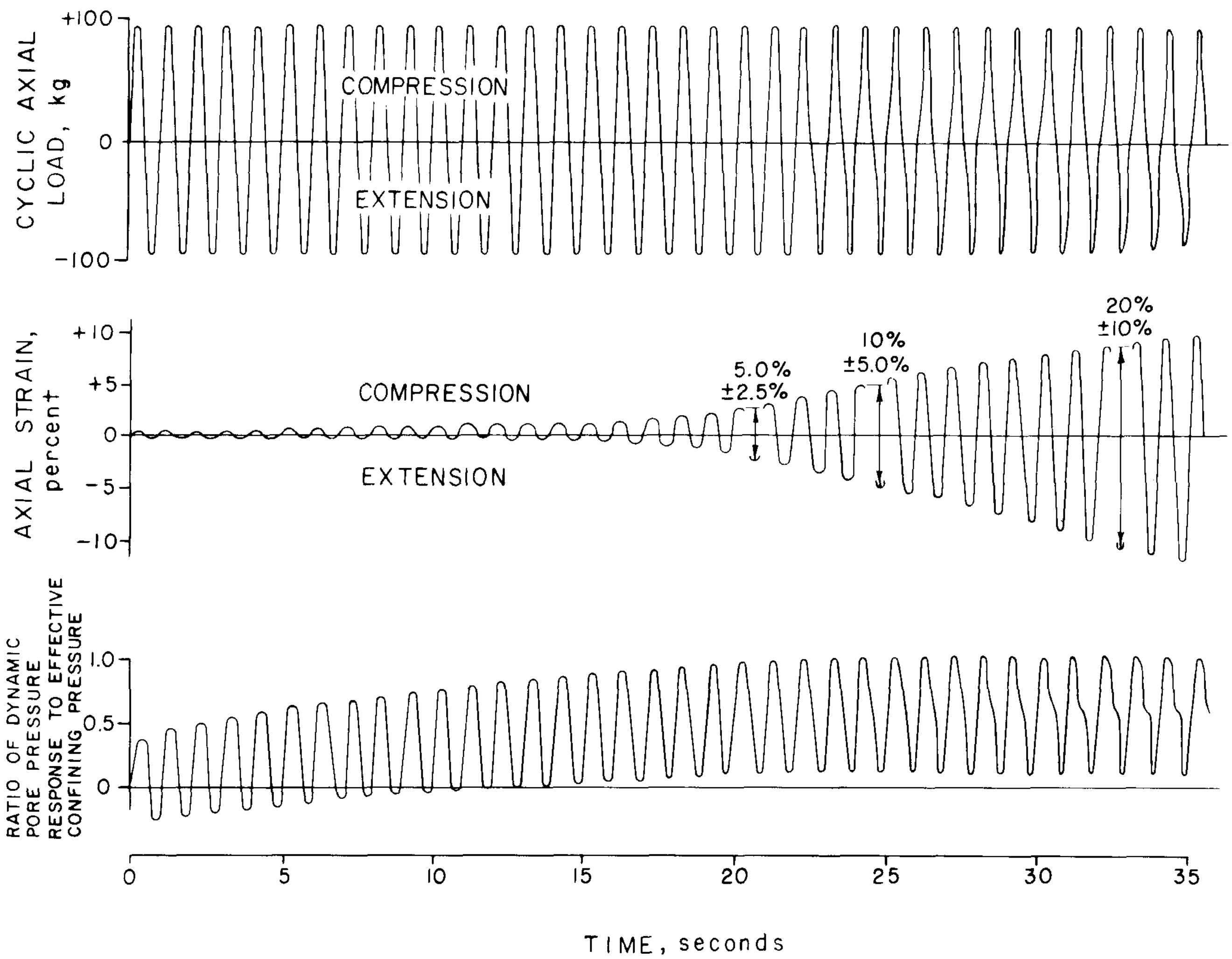


Figure 19. — Example of the type of record obtained from a cyclic triaxial test on an anisotropically consolidated specimen.

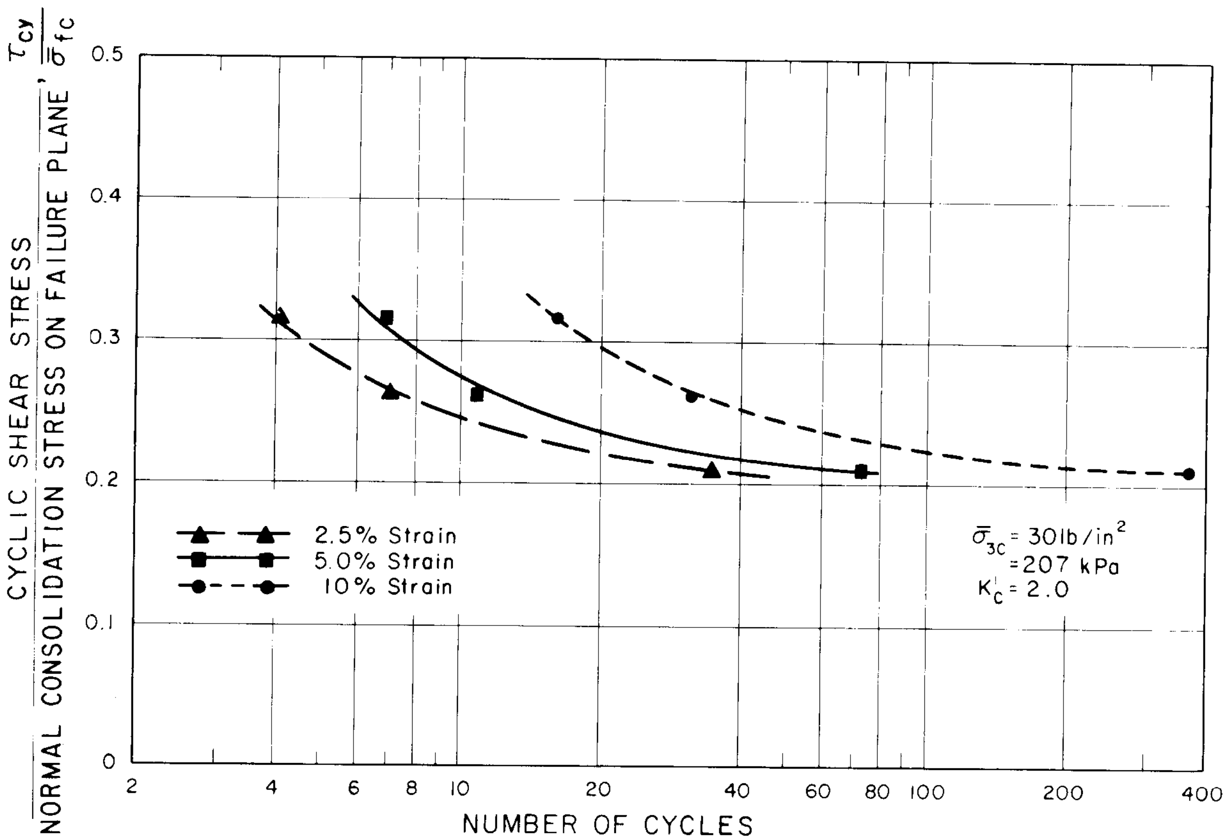


Figure 20. — Example of anistropically consolidated (K_c' = 2.0) cyclic triaxial strength test results.

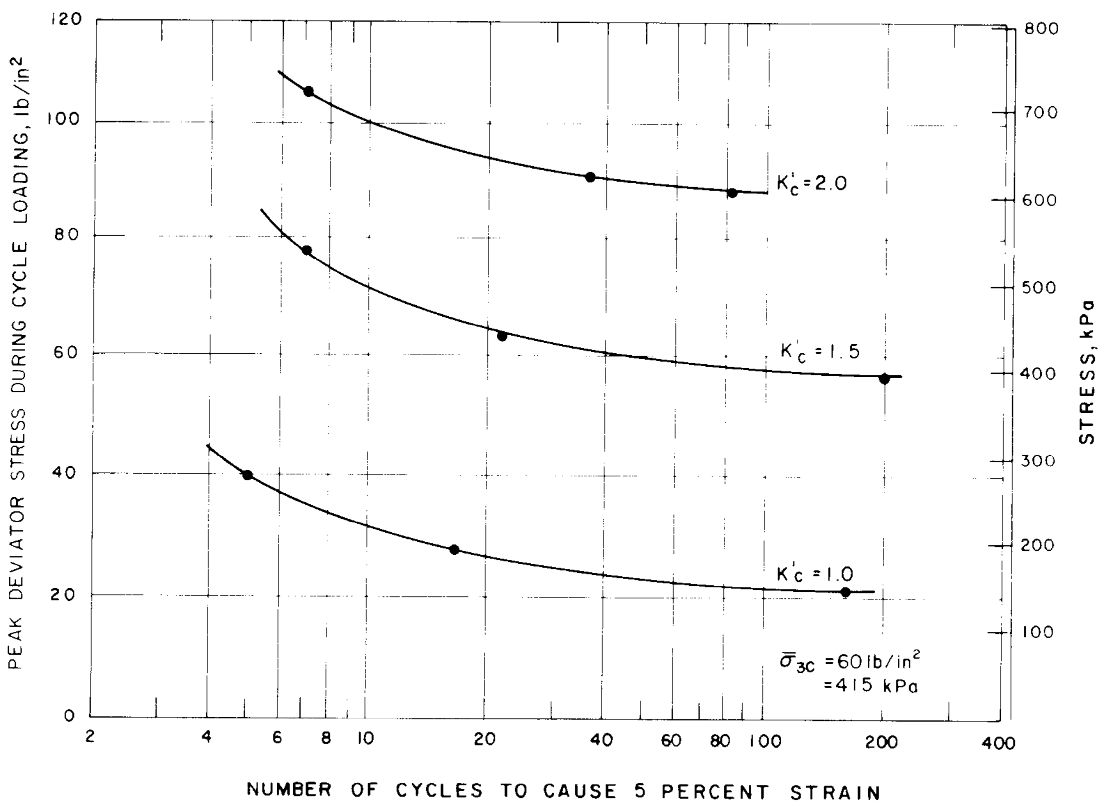


Figure 21. — Peak deviator stress required to cause a 5-percent axial strain.

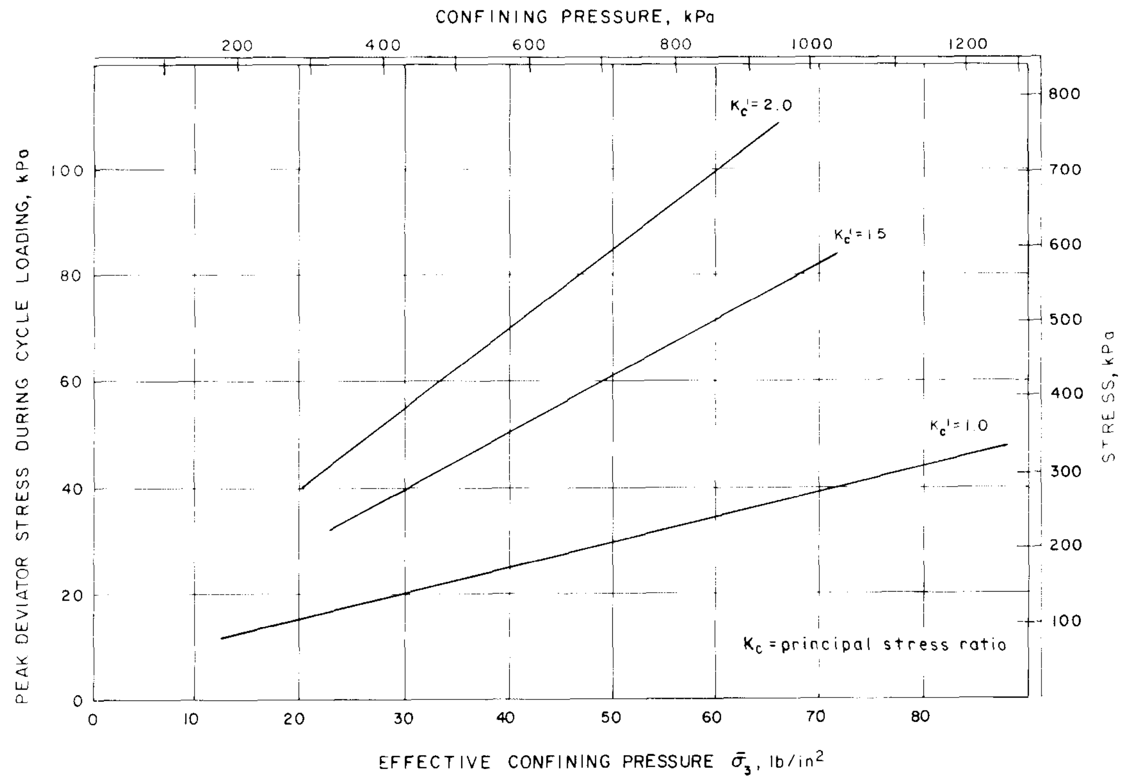


Figure 22. — Peak deviator stress required to cause a 5-percent axial strain in 10 cycles.

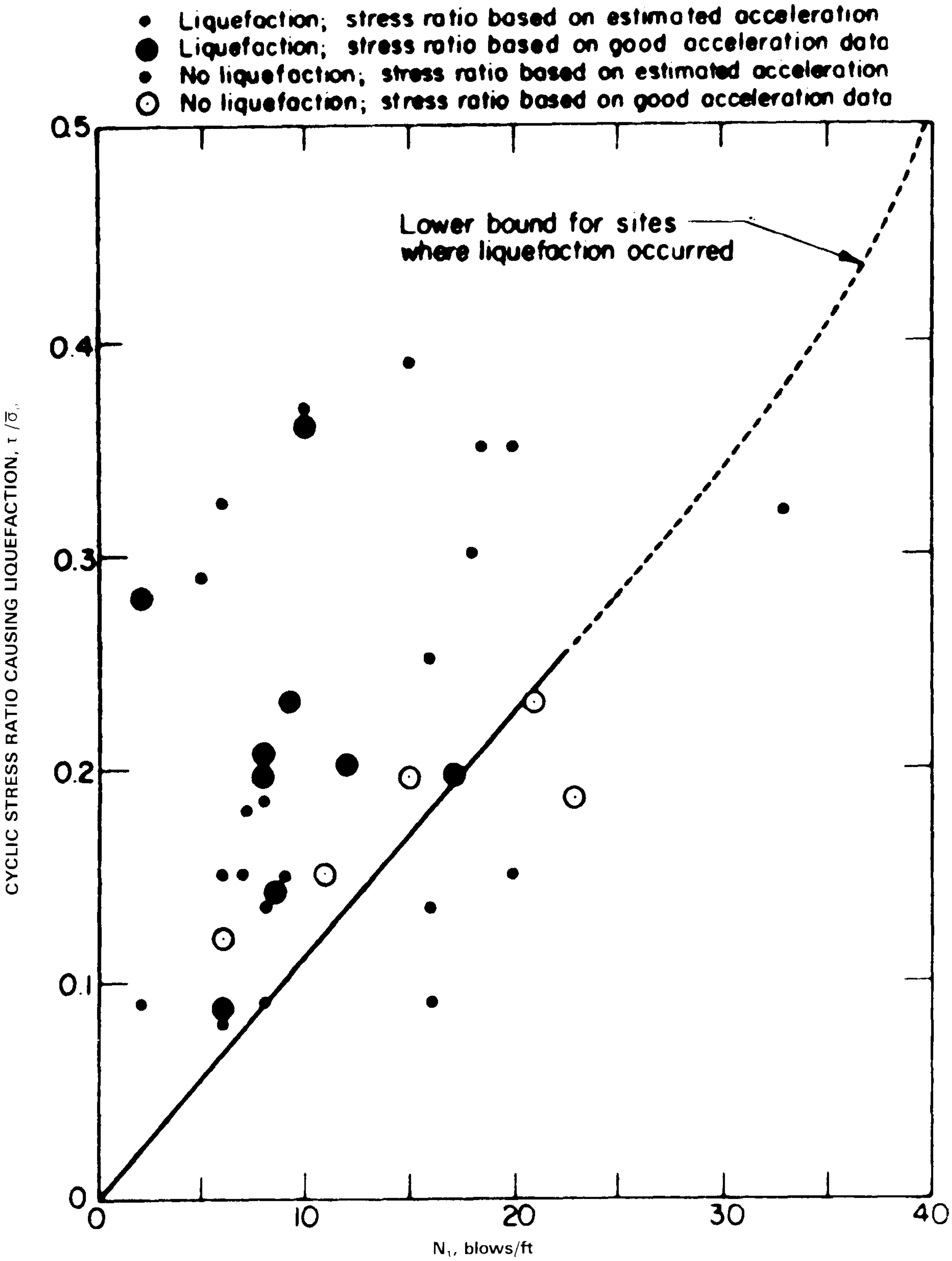


Figure 23. — Correlation between stress ratio cauing liquefaction in the field and penetration resistance of sand (Seed, H.B., I.M. Idriss, and I. Arango [140]).

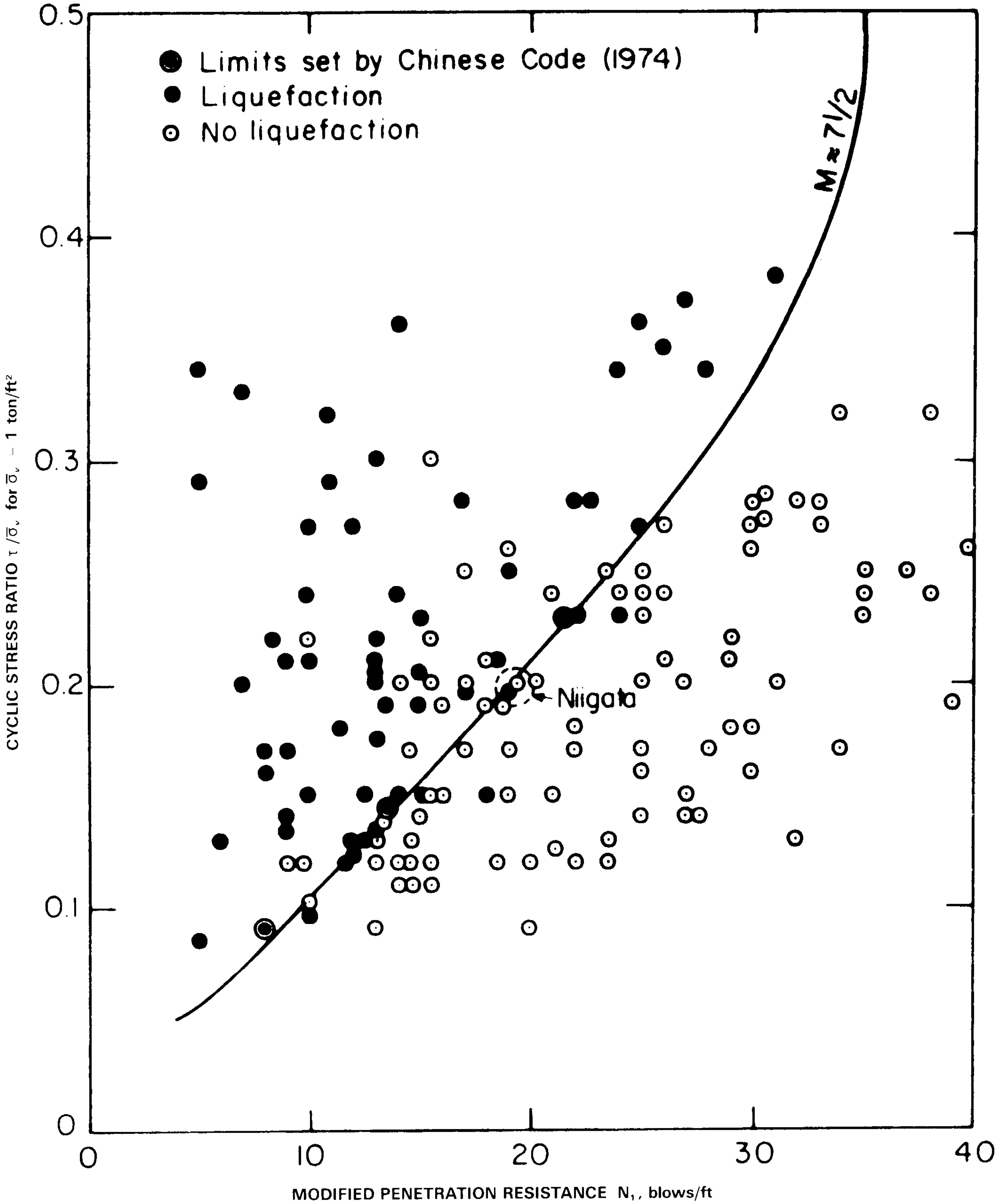


Figure 24. — Correlation between field liquefaction behavior of sands ($D_{50} > 0.25$ mm) under level ground conditions and standard penetration resistance (all data; Seed, H.B., I.M. Idriss, and I. Arango [140]).

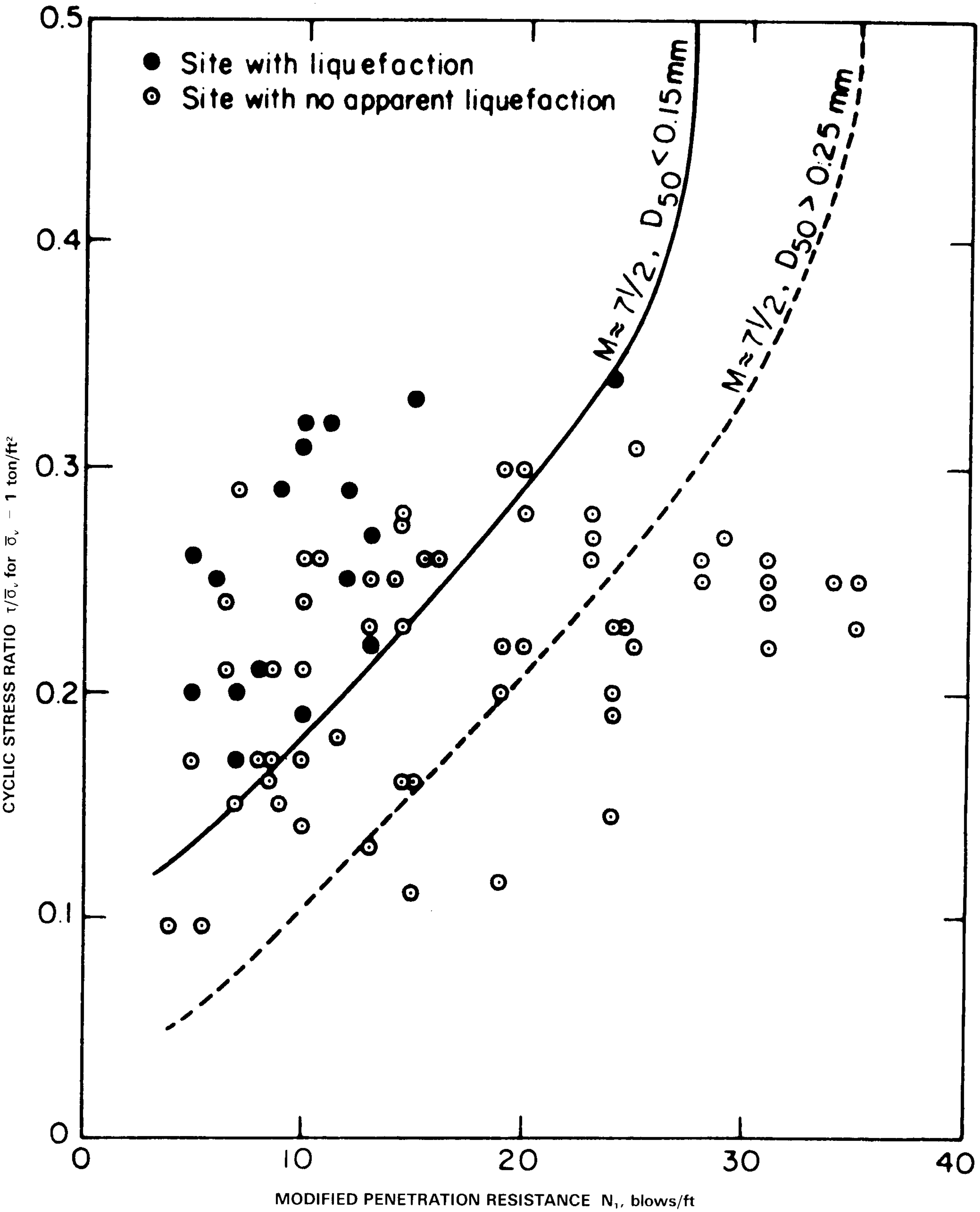


Figure 25. — Correlation between field liquefaction behavior of silty sands (D_{50} < 0.15 mm) under level ground conditions and standard penetration resistance (Seed, H.B., I.M. Idriss, and I. Arango, [140]; and Tokimatsu K. and H. Yoshimi [143].)

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Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.