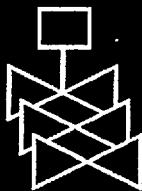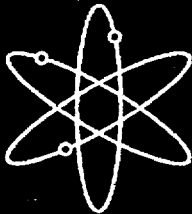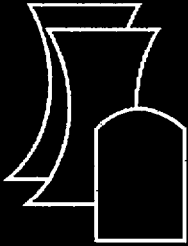# Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Facilities

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Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Facilities

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ABSTRACT

This document provides technical bases for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.138, Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants, reflecting current and state-of-the-art techniques related to laboratory testing of soils. The report summarizes the processes required in a laboratory testing program. Topics range from storage, selection, and handling of test specimens, to static and dynamic testing methods and equipment. Specific requirements for liquefaction analysis and field site investigations are not addressed in this document but are covered in companion technical bases documents.

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PREFACE

The study covered by this report was performed by the U.S. Army Engineer Waterways Experiment Station (WES) for the U.S. Nuclear Regulatory Commission (NRC) under Inter-Agency Agreement RES-95-006 during the period June 1995 to January 1999. The study was directed by Mr. Robert Kornasiewicz, Office of Nuclear Regulatory Research, NRC.

The report was prepared by Ms. Tina Holmes of the Soil and Rock Mechanics Division (S&RMD) and Dr. Joseph P. Koester of the Earthquake Engineering and Geosciences Division (EEGD), Geotechnical Laboratory, WES. External review was provided by the following individuals, whose insightful and thorough critique was essential and very much appreciated: Dr. Gonzalo Castro, GEI Consultants, Inc., Professor Charles Ladd, Massachusetts Institute of Technology (MIT), Prof. Tom Brandon, Virginia Polytechnic Institute and State University, and Professor Robert Whitman, MIT (retired).

At the time of publication of this report, Commander and Acting Director of WES was COL Robin R. Cababa, EN.

1 INTRODUCTION

This document provides a technical basis for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Facilities." It was prepared to provide information on laboratory testing procedures and issues that have developed and been put into practice since the current Regulatory Guide. A companion technical bases document details on-site investigations for foundations of nuclear facilities and refers to this report with regard to soils testing practice (Torres, 1998).

This report is organized in the manner of the 1978 Regulatory Guide, except where new testing methods applicable to the determination of engineering properties for nuclear facility foundations were not reported therein. The report will serve as an index of references that should be consulted for procedural details; it is not a manual that describes all steps in the various laboratory soils tests. Reviews and discussions of specific test procedures or equipment in this report are not intended as official endorsements.

Where existing guidance referenced in the previous Regulatory Guide is sufficient and represents the current state of geotechnical engineering practice, no update is indicated. Where improvements or new techniques have evolved, descriptions are provided. The following team of technical specialists was formed to provide oversight and counsel during the development of this report, for which the authors are very grateful:

> Dr. John Christian, Consultant
> Professor Carl Costantino, City College of New York, retired
> Professor W. D. Liam Finn, University of British Columbia
> Professor I. M. Idriss, University of California, Davis
> Professor Robert V. Whitman, Massachusetts Institute of Technology, retired

The state of laboratory soils testing practice has remained largely stable since 1978, and conservatism implicit in the measurement and reporting of engineering properties is still appropriate in most cases. Many of the technological advances made in engineering practice during the previous 20 years have resulted in more prevalent automation of procedures and increased speed of data acquisition and processing. Faster turn-around of laboratory testing and delivery of results is not universally a consequence of these advances, however, owing to the time-dependency of many soils phenomena. For the most part, soils laboratory testing is less tedious and labor-intensive than in 1978.

2 LABORATORY FACILITY

The overall design requirements for a laboratory facility have not changed during the past ten years. The basic requirements are still adequate test space, temperature controlled areas, adequate ventilation and air flow, etc. Every facility should be equipped with the proper equipment, (from calipers and sieves to triaxial testing devices) necessary to perform the type of test for which the facility was designed.

3 LABORATORY EQUIPMENT

Accuracy in measurements is of utmost importance to those engaged in testing of soils. Inaccurate measurements will produce test results which are valueless and misleading. Corps of Engineers Laboratory Soils Testing Manual EM 1110-2-1906 (Department of the Army, 1986) describes the more common possible errors associated with the procedures described therein. Use and care of laboratory equipment are also discussed in detail in EM 1110-2-1906, as well as in Das (1992), and Head (1992). Specifications for balances and scales are described in ASTM D 4753.

3.1 Apparatus

In soil testing, as in all laboratory work, it is necessary to take and record measurements of different kinds. Instruments such as gages, calipers, dials, balances, sieves, etc. are basic measuring devices used in laboratory testing. Tests that require special measuring instruments will require procedural guidance specific to the instruments.

3.1.1 Scholey, et al. (1995)

The authors present a review of instruments used for measuring small strain. Their paper discusses the sources of potential errors when strains are derived from conventional deformation measurements for analyzing soil stiffness, the requirements of instruments suitable for small strain measurement during triaxial tests, and the mode of operation, benefits, and limitations of the instruments. Scholey, et al. (1995) also summarize the characteristics of an ideal system for small strain measurements.

3.1.2 Germaine and Ladd (1988)

The problems associated with triaxial testing of saturated cohesive soils are discussed in this report. Their conclusions include the errors in testing that may be caused by the equipment or procedures used, such as membrane and filter drain resistance, piston friction, and leakage of water and gas.

3.2 Calibration

It should never be assumed that even the simplest equipment works as intended. Weights can vary from their claimed value, or ovens may not maintain a constant temperature (Clayton, et al.,1995). Instrument calibrations may be performed either by an outside organization or in-house using the laboratory's own standards of reference. EM 1110-2-1909 (Department of the Army, 1986) provides procedures recommended by the Corps of Engineers' soils laboratories for the calibration of testing equipment. ASTM D 3740 also provides information on equipment calibration.

Sallfors (1989) demonstrates the importance of equipment calibration. In his paper, equipment was calibrated by performing parallel tests by eight different laboratories on identical samples. It was found that two of the laboratories had oedometer rings a few tenths of a millimeter too wide. Figure 1 compares data from a suitable ring to that of an oversized ring. The preconsolidation pressure of ring B differed consistently by more than 10 percent for those performed on ring A. Frequent checking of equipment against laboratory standards can guard against most inaccuracies.

3.3 Reagents and Water

Chemical testing in a soil laboratory is usually limited to routine tests. These tests determine such constituents as organic matter, chlorides, pH value, and sulfates. Table 1 is excerpted from Head (1992) and provides information on the most widely used chemical tests for soils and groundwater.

4 HANDLING AND STORAGE OF SAMPLES

The identification markings of all samples should be verified immediately upon their arrival at the laboratory, and an inventory of the samples received should be maintained. Further information on handling and storage of soil samples can be found in ASTM D 4220. Samples requiring special treatment should be detailed in the specific testing procedures.

4.1 Disturbed Specimens

Samples should be examined and tested as soon as possible after arrival; however, in the case of large testing programs, storage of samples may be required for several days or weeks.

4.2 Undisturbed Specimens

Undisturbed specimens must be protected from damage from vibration, shock, or freezing and from changes in water content. They should be stored in humid rooms and may require rewaxing and relabeling before being stored. Even the most careful sealing and storing of undisturbed samples cannot prevent physical and chemical changes. Therefore, the samples should not be retained for long periods, particularly if in contact with unprotected steel sampling tubes. Storage for long periods of time may discredit any subsequent determinations of their engineering properties.

In the case of clay specimens, the delay between sampling and testing and the control kept over their volumes during storage are known to affect the strengths and compressibilities measured in the laboratory. These measured properties will also be affected by the reconsolidation procedures. Graham, et al. (1990) describes tests on reconstituted specimens of illite that were stored for fixed periods of up to one week, reconsolidated using three different procedures, and then sheared undrained. When this was done the undrained strength and porewater pressure results varied by about ±5 to 6 percent, with larger differences expected in the relative stiffness.

Brown and Chow (1988) describe a simple form of a sample compressor, designed to prevent deterioration of clay samples during storage. In laboratory tests, reconstituted kaolin was used to determine the effects on the modulus and strength of two different methods of sample storage after unloading, it was found that the use of the sample compressor to restore the in situ vertical effective stress was successful in reducing the deterioration of samples between loading and testing.

Figure 2 shows a simpler version of the compressor. This version does not enable measurement of sample compression, but may nearly eliminate deterioration, provided the spring is sufficiently compressible. The use of the sample compressor is believed to be based on sounder principle than any method that relies on sealing the sample against ingress of air or water while negative pore pressures exist inside the sample.

4.3 Rocks

Rock samples should be transported as a fragile material and protected from excessive changes in humidity and temperature. Like soil samples, rock samples should be examined and tested as soon as possible. They may however, be stored for several days or weeks for a large testing program. Every effort must be made to protect stored samples against damage.

5 CONCLUSIONS

The initial laboratory description of a sample should include but not be limited to what is seen, felt, and smelled.

5.1 ASTM D 2488

Practice for Description and Identification of Soils (Visual-Manual Procedure). This practice was originally published in 1966. The latest approval version, in 1993, incorporated the addition of X5 as one of the abbreviated soil classification symbols. It also describes the procedures necessary for the description and identification of a soil sample. It is based primarily on visual identification and manual tests. This standard practice also allows identification of soils based on the classification system characterized in ASTM D 2487.

5.2 ASTM D 4452

X-Ray radiography of soil samples. This standard was published in 1985, it can be used to determine the quality of a sample before testing. It is especially useful for undisturbed samples. X-raying the sample before testing allows the detection of inherent abnormalities and disturbances. Such information allows comparison with the sample after testing to determine the effects of the test on the sample.

5.3 TM 3-357 (1960)

The unified soil classification system (USCS). This technical manual (TM) describes the various soil groups in detail and discusses the method of identification in order that a uniform classification procedure may be followed by those that use the system. Table 2 details how to identify and describe a soil according to the USCS.

5.4 RTH 102-93

(Rock Testing Handbook, 1993) Recommended practice for petrographic examination of rock cores. This practice was published in 1990 and modified in 1993, it describes the procedures used in the petrographic examination of rock core samples. Petrographic examination could use procedures such as light microscopy, X-ray diffraction analysis, differential thermal analysis, or infrared spectroscopy. Petrographic examinations are made to determine the physical and chemical properties of a material, to describe and classify a sample, and to determine amount of specific materials that may affect the specimen's intended use.

6 SELECTION AND PREPARATION OF TEST SPECIMENS

During preparation, contact with surfaces contaminated by oils or other substances may alter the properties of the sample. This altering of properties could give misleading test results. Therefore, care should be taken to assure that all surfaces are free from contaminants. When cutting fluids are required during preparations, water is the preferred liquid unless testing procedures state otherwise for the type of test performed.

6.1 Undisturbed Samples

The preparation of undisturbed test samples should be conducted to preserve the natural structure and water content of the material. Careless handling of the sample during preparation could result in erroneous test data. The sample should always be prepared in a humid room. Trimming instruments should be sharp and clean and the sample adequately supported.

6.2 Reconstituted or Remolded Samples

Laboratory personnel should record a complete detailed description of the specimen. The description should include but not be limited to identification of the material, color an consistency, brittleness of the material, and indication of disturbance of boring samples. Disturbed samples should not be used for any test other than classification, specific gravity, or water content according to EM 1110-2-1906. The difficulty inherent in sampling many soils and maintaining their in situ structure makes this prohibition impractical, however; laboratory test series on reconstituted specimens may provide useful strength behavior information for parametric studies. Cyclic strength measurement is particularly sensitive to effects of sample disturbance and is treated with more detail in the companion report on liquefaction potential evaluation for nuclear power facilities.

6.3 Scalping of Large Particles

Standard-sized laboratory testing equipment will not readily accommodate gravel and larger particles. Such materials are typically scalped, or removed from the total sample and the finer fraction tested. A research study on earth-rock mixtures was conducted at the U.S. Army Engineer Waterways Experiment Station to determine particle size and gradation effects on engineering behavior of these materials. Compaction control measures are specifically reported by Torrey and Donaghe (1991, available from NTIS, NO. A241703), which provides recommendations for compaction testing equipment, procedures, and data interpretation where gravelly soils are of concern. Fractional analysis of density to account for scalped gradation is described in that report for situations where larger-scale laboratory devices are not available. Evans and Zhou (1995) report the effects on cyclic strength of inclusion of gravel size particles in various gradations of granular soils. The authors contend that scalping may result in underestimation of cyclic strength when the finer remaining fraction is tested at the same relative density as the in situ material. A method for estimating equivalent fraction density is proposed to account for the increased cyclic strength resulting from inclusion of the gravel particles.

7 LABORATORY TESTING PROGRAM

The study of soil and rock mechanics covers the investigation, description, classification, testing and analysis of soil and rock to determine their interaction with structures built in or upon them, or built with them. The physical properties of soils and rocks are often determined by carrying out tests on samples of soil in a laboratory. These tests can be divided into two main categories: classification tests and engineering properties tests. Classification tests indicate the general type of soil and the engineering category to which it belongs. Engineering properties are determined by specific tests that require careful consideration of field conditions, various design loading conditions, material properties, and possible problems at the site.

The focus of laboratory investigations will depend on the design requirements and the nature of problems encountered or suspected at the site. Specific testing requirements and details of testing procedures will depend on the nature of the soils and rocks. It is common practice to modify the testing procedures to meet specific requirements of an investigation. A laboratory testing program should be designed to supplement and refine the information obtained from the subsurface investigation and field tests.

The goal of this report is to give a general update on the state of soil and rock laboratory testing. References are provided for specific guidance on individual procedures; details are included in this document only where they are otherwise inaccessible to the general practice.

7.1 Testing Procedures of Determining Static Soil Properties

Classification tests and determination of engineering properties should be performed according to an accepted and published method. Procedures for some of the most common tests along with other related references are shown in Appendix B. The following information elaborates, where necessary, on the description of the standard or procedure and states the origination and/or revised date of the procedure listed in Appendix B.

7.1.1 Water (Moisture) Content

The water content (or moisture content) of soil is defined as the ratio of the weight of water in a specimen to the weight of solids in the specimen, expressed as a percentage.

7.1.1.1 ASTM D 425. *Centrifuge Moisture Equivalent of Soils.* The original version of this procedure was published in 1979; the standard was revised in 1988 and reapproved in 1994. This test determines the moisture equivalent. It is limited to coarse-grained soil having fines of low plasticity and to soil passing the 2.00-mm sieve.

7.1.1.2 ASTM D 1558. *Moisture Content Penetration Resistance Relationships of Fine-Grained Soils.* Originally published in 1958 and revised in 1994, this standard procedure determines the moisture-penetration resistance relationship of fine-grained soils as determined using the soil penetrometer.

7.1.1.3 ASTM D 2216. *Laboratory Determination of Water (Moisture) Content of Soil and Rock.* This standard was originally published in 1963 and revised in 1992, and covers determination of the water (moisture) content of soil, rock, and similar materials by mass.

7.1.1.4 ASTM D 2974. *Moisture, Ash, and Organic Matter of Peat and Other Organic Soils.* This standard was originally published in 1971, and was revised in 1987 and reapproved in 1995. It determines the moisture content, ash content, and organic matter in peats and other organic soils.

7.1.1.5 ASTM D 4643. *Determination of Water (Moisture) Content of Soil by the Microwave Oven Method.* Originally published in 1987 and revised in 1993, this method determines the water (moisture) content of soils by incrementally drying soil in a microwave oven. It is applicable to most soil types.

7.1.1.6 TR GL-88-21. *Computer-Controlled Microwave Oven System for Rapid Water Content Determination.* A microwave oven system (equipment and technique) developed for rapid, accurate, and reliable determination of the water content of inorganic soils is described in Gilbert (1988). This system dries soils specimens and gives accurate water contents within a time frame of 10 to 15 minutes. This saves much time over the conventional oven method.

7.1.1.7 Lui and Evett, (1990). The microwave oven method is much quicker than the conventional oven method; it has been shown to give reliable results for most soil types. The procedures used by this method may not give reliable results for (1) soils containing significant amounts of mica, gypsum, halloysite, montmorillonite, or other hydrated materials; (2) highly organic soils; or (3) soils in which the pore water contains dissolved solids.

The microwave oven method generally gives results comparable to those obtained using conventional ovens, but if there are questions of accuracy between the two methods, the conventional oven method takes precedence. The microwave oven method is not a replacement for the conventional oven method, but is to be used as a supplement when rapid results are needed to expedite other phases of testing.

7.1.1.8 TR GL-90-26. *Computer-Controlled Microwave Drying of Potentially Difficult Organic and Inorganic Soils.* Gilbert (1990) extended the use of the microwave oven method for water content determination to include difficult organic and inorganic soils. The difficult soils investigated are gravels and earth-rock mixture, dredged materials, fly ash, gypsum rich soils, calcite rich soils, peat, and tropical residual soils. The microwave oven system was evaluated and found to be adequate for use in the field.

7.1.1.9 ASTM D 4959. *Determination of Water (Moisture) Content of Soil by the Direct Heating Method.* Originally published in 1989, this procedure is used to determine the water (moisture) content of soils by drying with direct heat, such as using a hotplate, stove, blowtorch, etc. This method is not a replacement for D 2216 but is to be used as a substitute when more rapid and less accurate results are acceptable or desired to expedite other phases of testing.

7.1.1.10 EM 1110-2-1906. *Appendix I, Water Content - General.* This manual describes the procedures for determining the water content of soil according to the U.S. Army Corps of Engineers using the conventional oven drying method. It was originally published in 1970 and reprinted with corrections in 1986.

7.1.2 Atterberg Limits

Atterberg limits are water contents which define the limits of the various stages of consistency of a soil. The liquid limit (LL) and the plastic limit (PL) define the upper and lower limits, respectively, of the plastic range of a soil. The numerical difference between these two limits represents the plasticity of a soil and is termed the plasticity index (PI). The shrinkage limit (SL) defines the lower limit of the semi-solid range of a soil.

7.1.2.1 ASTM D 427. *Shrinkage Factors of Soils by the Mercury Method.* Originally published in 1935 and revised in 1993, this method determines the data to calculate the shrinkage limit and shrinkage ratio.

7.1.2.2 ASTM D 4318. *Liquid Limit, Plastic Limit, and Plasticity Index of Soils.* This standard was originally published in 1983 and revised in 1995. It replaces standards D 423 and D 424. This standard provides both the wet and dry methods allowed for preparation of test specimens.

7.1.2.3 ASTM D 4943. *Shrinkage Factors of Soils by the Wax Method.* Originally published in 1989 and revised in 1995, this standard specifies procedures for determining the shrinkage limit of soils. The data obtained may be used to calculate the shrinkage ratio, volumetric shrinkage, and linear shrinkage. The test is only applicable for cohesive soils that pass the No. 40 (425 μm) sieve.

7.1.2.4 EM 1110-2-1906. *Appendix IIIB, Shrinkage Limit Test.* This engineering manual describes the apparatus, procedures, and formulas for the calculation of the shrinkage limit (SL), shrinkage ratio (R), and linear shrinkage L_s.

7.1.2.5 EM 1110-2-1906. *Appendix III, Liquid and Plastic Limits.* Detailed procedures for determining the liquid and plastic limits for use in classifying soils are given along with possible errors.

7.1.2.6 Koester (1992a). Koester examined laboratory procedures used to determine Atterberg limits of cohesive soils observed to have liquefied in the People's Republic of China (PRC) during the 1976 Tangshan earthquake. The paper summarizes earlier comparisons of liquid limit values measured using a variety of procedures and also presents results of a limited laboratory test series comparing liquid and plastic limits measured using United States (U.S.) and PRC methods (the latter employing a laboratory fal-sone penetrometer) for artificial mixtures and natural low-to-medium plasticity soils. The paper reports that liquid limits may vary slightly when determined using U.S. (Casagrande cup) and other fall-cone devices; the variations may be significant to the determination of liquefaction potential in low-plasticity silty soils.

Liquid limit data obtained for silts by means of the Casagrande device exhibit considerable scatter; however, the Casagrande method is the currently accepted U.S. standard.

7.1.2.7 Bobrowski, et al.(1992). Bobrowski, et al. describes the design and use of a new device used to determine the plastic limit (Figure 3). This device tries to eliminate the factors associated with the human hand that may affect the plastic limit results. Factors such as the size and shape of an individual's hand can determine when the thin thread of soil crumbles while being rolled. The results of correlation testing are shown in Tables 3 and 4 .

7.1.3 Specific Gravity

The term specific gravity is defined as the ratio of the weight of a given volume of material to the weight of an equal volume of water.

7.1.3.1 ASTM D 854. *Specific Gravity of Soils.* Originally published in 1945 and revised in 1992, this procedure determines the specific gravity of the soil that passes the No. 4 sieve by way of a pycnometer.

7.1.3.2 ASTM D 5550. *Specific Gravity of Soil Solids by Gas Pycnometer.* This method was published in 1994 and determines the specific gravity of soil solids by means of a gas pycnometer. Particle size is limited by the dimensions of the specimen container of the particular pycnometer being used.

7.1.3.3 EM 1110-2-1906. *Appendix IV, Specific Gravity.* The specific gravity of soil is expressed in three different forms in Corps of Engineer guidance:

- the specific gravity of solids, G_s
- the apparent specific gravity, G_a
- the bulk specific gravity, G_m.

The specific gravity is applied to soils finer than the No. 4 sieve. The apparent specific gravity and the bulk specific gravity are applied to soils retained on the No.4 sieve.

7.1.3.3.1 Specific Gravity of Solids. The specific gravity of solids of a soil is the ratio of the weight in air of a given volume of soil particles at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. The specific gravity of solids is not applied to coarse particles because they normally contain voids from which air cannot be displaced unless the particles are ground into finer particles to eliminate the voids.

7.1.3.3.2 Apparent Specific Gravity. The apparent specific gravity of a soil is the ratio of the weight in air of a given volume of the impermeable portion of a permeable material (that is, the solid matter including its impermeable pores or voids) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. When dealing with coarser particles it is more convenient to work with the apparent specific gravity of the particle mass.

7.1.3.3.3 Bulk Specific Gravity. The bulk specific gravity is the ratio of the weight in air of a given volume of a permeable material (including both permeable and impermeable voids normal to the material) at a stated temperature to the weight in air of an equal volume of distilled water at a stated temperature. The bulk specific gravity is used in special calculations, such as correction of density and water content for soils containing gravel sizes.

7.1.4 Gradation Analysis

Gradation (grain-size) analysis is a process in which the proportion of material of each grain size present in a given soil (grain-size distribution) is determined. The grain-size distribution of coarse-grained soils is determined directly by sieve analysis, while that of fine-grained soils is determined indirectly by hydrometer analysis. The grain-size distribution of mixed soils is determined by combined sieve and hydrometer analyses.

7.1.4.1 ASTM D 421. *Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants.* Originally published in 1935, revised in 1985, and reapproved in 1993, this practice describes the procedure for dry preparation of soil samples for particle size analysis and determination of the soil constants.

7.1.4.2 ASTM D 422. *Particle-Size Analysis of Soils.* Originally published in 1935, approved in 1963, and reapproved in 1990, this method is an alternate means to determine the particle size distribution.

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7.1.4.3 ASTM D 2217. *Wet Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants.* This method was originally published in 1963 and revised in 1985. It describes procedures for wet preparation of soil samples as received from the field for particle-size analysis and the determination of soil constants.

7.1.4.4 EM 1110-2-1906. *Appendix V, Grain-Size Analysis.* The testing manual describes to methods of grain-size analysis; sieve analysis for coarse-grained soils and hydrometer analysis for fine-grained soils.

7.1.4.4.1 Sieve Analysis. The determination of the grain-size distribution of coarse-grained soils by sieve analysis consists of passing a sample through a set of sieves and weighing the amount of material retained on each sieve. Sieves are constructed of wire screens with square openings of standard sizes. The sieve analysis is performed on material retained on a U.S. Standard No. 200 sieve. The sieve analysis, in itself, is applicable to soils containing small amounts of material passing the No. 200 sieve, provided the grain-size distribution of that portion of the sample passing the No. 200 sieve is not of primary interest.

7.1.4.4.2 Hydrometer Analysis. The hydrometer method of analysis is based on Stokes' law, which related the terminal velocity of a sphere falling freely through a fluid to its diameter. It is assumed that Stokes' law can be applied to a mass of dispersed soil particles of various shapes and sizes. The hydrometer is used to determine the percentage of dispersed soil particles remaining in suspension at a given time. The maximum grain size equivalent to a spherical particle is computed for each hydrometer reading based on Stokes' law. The hydrometer analysis is applicable to soils passing the No. 10 sieve for routine classification purposes; when greater accuracy is required (such as in the study of frost-susceptible soils), the hydrometer analysis should be performed on only the fraction passing the No. 200 sieve.

7.1.4.5 Howard and Horz (1988). Howard and Horz present recommended minimum test specimen masses for gradation analysis of soil based on the maximum particle size present and also documents the rationale for the recommendations. Table 5 lists the minimum specimen sizes recommended in various standards (ASTM, AASHTO, and so forth) that are currently being used worldwide. As a comparison, Table 6 lists the minimum specimen masses recommended, by the authors, for gradation analysis testing when the results are reported to ± 1 percentage point.

7.1.5 Erodibility Tests

Erodibility tests are used to identify dispersive clays, clays that are susceptible to eroding. Dispersive clays cannot be identified by conventional index tests such as particle size distribution, Atterberg limits, and compaction characteristics.

7.1.5.1 Pinhole Test. The pinhole erosion test is the most reliable test for determining the erodibility of a soil. Figure 4 shows a schematic representation of the pinhole test.

7.1.5.1.1 ASTM D 4647. *Identification and Classification of Dispersive Clay Soils by the Pinhole Test.* Originally published in 1987 and revised in 1993, this test directly measures, qualitatively, the dispersibility and consequent colloidal erodibility of clay soils by causing water to flow through a small hole punched in a specimen. This method is complemented by D 4221.

7.1.5.1.2 EM 1110-2-1906. *Appendix XIII, Pinhole Erosion Test for Identification of Dispersive Clays.* Detailed procedures for all four erosion tests (Pinhole test, Crumb tests, Soil Conservation Service (SCS), and Cylinder dispersion test) dispersion test are given in this testing manual. Some of the limitations of the pinhole test:

- undisturbed samples of high sensitivity may test as dispersive, while in nature they may be resistant to erosion,
- soils with high sodium (> 80 percent) and low total dissolved solids (< 0.4 meq/*l*) in the soil pore water may show nondispersive behavior in the test, while the soil may exhibit dispersive performance in the field.

7.1.5.2 Crumb Test. The crumb test is a simple means of identifying dispersive clays without requiring special equipment. It can be used in the laboratory or the field. The crumb test gives an indication only of dispersive behavior. If the crumb test indicates dispersion, then the soil is probably dispersive, however many dispersive soils (kaolinitic soils) do not react to the crumb test.

7.1.5.3 Soil Conservation Service (SCS) Dispersion Test. This test is sometimes referred to as the double hydrometer test (Head, 1992). It estimates the propensity for dispersion in the field based on the degree of dispersion of clay particles achieved during the pre-treatment state of a hydrometer sedimentation test.

7.1.5.4 ASTM D 4221. *Dispersive Characteristics of Clay Soil by Double Hydrometer.* The original guidance was published in 1983 and revised in 1990. It provides an indication of the natural dispersive characteristics of clay soils when used in conjunction with a test performed by method D 422 on a duplicate soil sample. This test is applicable only to soils with a plasticity index greater than 4 as determined by method D 4318 and more than 12 percent of the soil fraction finer than 5μm as determined by method D 422.

7.1.5.5 Cylinder Dispersion Test. This test was developed as an extension to the crumb test. Its main purpose was to examine soil behavior in the "fully softened" state when it was submerged in water, i.e. under zero effective stress. This test is performed on a cylindrical specimen of remolded soil which has been consolidated from a slurry.

7.1.6 Compaction

7.1.6.1 ASTM D 698. *Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft³ (600 kN-m/m³)).* This method was approved as a standard in 1991. It determines the relationship between water content and dry unit weight of soils compacted in a 4- or 6-in. (101.6- or 152.4-mm) diameter mold with a 5.5-lbf (24.4-N) rammer.

7.1.6.2 ASTM D 1557. *Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft³ (2,700 kN-m/m³)).* Originally published in 1958 and revised in 1991, this method determines the relationship between water content and dry unit weight of soils compacted in a 4- or 6-in. (101.6 or 152.4-mm) diameter mold with a 10-lbf (44.5-N) rammer.

7.1.6.3 ASTM D 4253. *Maximum Index Density and Unit Weight of Soils Using a Vibratory Table.* This standard procedure was published in 1983, edited in 1991, and republished in 1993. It covers four procedures for determining the maximum index density/unit weight of cohesionless, free-draining soils using a vertically vibrating table.

7.1.6.4 ASTM D 4254. *Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density.* Originally published in 1983 and republished in 1991 this standard details the test methods used to determine the minimum index density/unit weight of cohesionless, free-draining soils.

7.1.6.5 ASTM D 5080. *Rapid Determination of Percent Compaction.* Published in 1990 and revised in 1993, this specification describes the procedure for rapidly determining the percent compaction and the variation from optimum moisture content of an in-place soil. The test is normally performed for soils containing more than 15 percent fines (minus No. 200 sieve size).

7.1.6.6 EM 1110-2-1906. *Appendix VI, Compaction Tests.* Details three procedures for compacting soils: standard compaction test, modified compaction test, and the 15-blow compaction test. The standard compaction test is used to simulate field compaction for routine foundation and embankment design. The modified test uses a greater compactive effort which results in higher maximum densities and lower optimum water contents and is typically specified for structural fills. This method is appropriate for determining the maximum density for large material, however the specimen must be checked for particle breakage. The 15-blow compaction test differs from the standard test in that a lesser compactive effort is used resulting in lower maximum densities and higher optimum water contents. The apparatus and procedures used in both the modified and 15-blow compaction tests vary slightly from that of the standard compaction test.

7.1.7 Permeability

7.1.7.1 ASTM D 2434. *Permeability of Granular Soils (Constant Head).* Originally published in 1965, approved in 1968, and reapproved 1994, this standard determines the coefficient of permeability by a constant-head method for the laminar flow for water through granular soils. It is limited to disturbed granular soils containing not more than 10 percent by weight passing the No. 200 sieve.

7.1.7.2 ASTM D 5084. *Measurement of Hydraulic Conductivity of Saturated Porous Material Using a Flexible Wall Permeameter.* D 5084 was last published in 1990, and determines hydraulic conductivity of fine-grained soil specimens that may be prepared from either undisturbed or compacted samples. This standard procedure is applicable to soils having hydraulic conductivity less than or equal to 1×10^{-5} m/s. More permeable soils should be tested according to ASTM D 2434.

7.1.7.3 EM 1110-2-1906. *Appendix VII, Permeability Tests.* Describes the several testing methods and equipment: constant-head test with permeameter cylinder, falling-head test with permeameter cylinder, permeability test with sampling tubes, permeability test with pressure chamber, permeability tests with back pressure, and permeability test with consolidometer.

7.1.8 Consolidation

Consolidation is the process of time-dependent volume change of saturated clayey soil when subjected to a changed loading.

7.1.8.1 ASTM D 2435. *One-Dimensional Consolidation Properties of Soils.* This standard procedure was published in 1965 and revised in 1990, and determines the magnitude and rate of consolidation of soil when it is retained laterally and drained axially while subjected to incrementally applied controlled stress loads.

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7.1.8.2 ASTM D 4186. *One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading.* Originally published in 1982 and revised in 1989, standard D 4186 determines the rate and magnitude of consolidation of soil when it is restrained laterally and drained axially and subjected to controlled-strain loading. The results may be used to analyze or estimate one-dimensional consolidation settlements and rates.

7.1.8.3 EM 1110-2-1906. *Appendix VIII, Consolidation Test.* In the laboratory consolidation is usually determined by the one-dimensional consolidation test. This test, laterally confines the soil subjected to successively increased vertical pressure, allowing free drainage from the top and bottom surfaces.

7.1.9 Shear Strength Tests

As part of their summary of drained versus undrained testing Bishop and Bjerrum (1960) set forth guidelines regarding strength parameters for triaxial testing:

(1)   Effective stress analysis is a generally valid method for analyzing any stability problem and is particularly valuable in revealing trends in stability which would not be apparent from total stress method. Its application in practice is limited to cases where the pore pressures are measured or can get estimated with reasonable accuracy, such as long-term stability where the pore pressure is controlled either by the static water table or by a steady-state flow pattern.

(2)   Where a saturated clay is loaded or unloaded at such a rate that there is no significant dissipation of the excess pore pressures set up, the stability can be determined by the $\phi_u = 0$ (S_u = constant) analysis, using the undrained strength obtained in the laboratory from undrained or unconfined triaxial tests or from in situ vane tests. This is essentially an end of construction method, and in the majority of foundation problems, where the factor of safety increases with time, it provides a sufficient check on stability. For cuts, on the other hand, where the factor of safety generally decreases with time, the long term stability must be checked by the effective stress method..

(3)   For saturated soils, the values of c' and ϕ' are obtained from drained tests or consolidated-undrained tests with pore pressure measurements carried out on undisturbed samples. The range in stresses at failure should be chosen to correspond to those in the field. Values measured in the laboratory appear to be in satisfactory agreement with field records with two exceptions. In stiff fissure clays, the field values of c' is lower than the value given by standard laboratory tests - in some very sensitive clays, the field value of ϕ' is lower than the laboratory value.

7.1.9.1 ASTM D 4648. *Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil.* The standard was originally published in 1987 and the current edition was approved and published in 1994. This method covers the miniature vane test in very soft to stiff saturated fine-grained clayey soils ($\phi = 0$). The vane shear test provides a way to analyze strength anisotropy in the vertical and horizontal directions.

7.1.9.2 Unconsolidated-Undrained Test (Q (UU) Test)

7.1.9.2.1 ASTM D 2850. *Unconsolidated Undrained Compressive Strength of Cohesive Soils in Triaxial Compression.* Published in 1970 and revised 1995, standard D 2850 determines the strength and stress-strain relationships of a cylindrical specimen. This procedure provides data for determining the undrained strength properties and stress-strain relation for the soil if there is no change in the water content of the soil during construction.

7.1.9.2.2 EM 1110-2-1906. *Appendix X, Triaxial Compression Tests.* Water content of the test specimen is not permitted to change during the application of the confining or during the loading of the specimen to failure by increasing the deviator stress. The Q test is usually applicable only to soils which are not free-draining, i.e. to soils having a permeability less than 10^{-4} cm/sec.

7.1.9.3 Consolidated-Undrained Test (R (CU) Test).

7.1.9.3.1 ASTM D 4767. *Consolidated-Undrained Triaxial Compression Test on Cohesive Soils.* Published in 1988 this standard determines the strength and stress-strain relationships for a cylindrical specimen of either an undisturbed or remolded saturated cohesive soil when it is isotropically consolidated and sheared undrained in compression at a constant rate of axial deformation (strain controlled).

7.1.9.3.2 EM 1110-2-1906. *Appendix X, Triaxial Compression Tests.* The test specimen is completely consolidated under the confining pressure, then with the water content held constant, the specimen is loaded to failure by increasing the deviator stress. Specimens must as a general rule be completely saturated before application of the deviator stress.

7.1.9.4 Consolidated-Drained

7.1.9.4.1 ASTM D 3080. *Direct Shear Test of Soils Under Consolidated Drained Conditions.* This test method was originally published as a standard in 1972 and revised in 1990. It covers the determination of the consolidated drained shear strength properties of a soil in direct shear loading.

7.1.9.4.2 EM 1110-2-1906. *Appendix X, Triaxial Compression Tests.* In the S (CD) test, complete consolidation of the test specimen is permitted under the confining pressure and during the loading of the specimen failure by increasing the deviator stress. Consequently, no excess pore pressure exist at the time of failure.

7.1.9.5 ASTM D 2166. *Unconfined Compressive Strength of Cohesive Soil.* The last edition of this standard was approved in 1985, the current version was approved and published in 1991. ASTM D 2166 allows the determination of the unconfined compressive strength of cohesive soil in the undisturbed, remolded, or compacted condition, by using strain-controlled application of the axial load. This method gives an approximate value of the strength of cohesive soils in terms of total stresses.

7.2 Testing Procedures of Determining Dynamic Soil Properties

7.2.1 Cyclic Triaxial

Historically, the most common cyclic loading technique for investigating liquefaction resistance involves the performance of the cyclic triaxial test, as a consequence of such factors as availability of equipment and relative ease of preparing undisturbed specimens. This is in spite of wide recognition of the inability of the test to accurately represent field earthquake stresses and boundary conditions (Seed and Idriss, 1982). Figures 5 and 6 are a schematic drawing of the cyclic triaxial test apparatus and a sample recording of load, deformation, and pore pressure response, respectively. Cyclic strength curves such as are typically generated from cyclic triaxial data are shown in Figure 7. Instructions for performance of cyclic triaxial tests may be found in EM 1110-2-1906.

Previous studies have demonstrated that cyclic triaxial strengths (in fact, strengths determined from any unidirectional loading test) are higher than those expected to produce equivalent effects in the field (Seed, 1976). Reduction factors were developed to adjust laboratory cyclic test strengths to estimate field liquefaction resistance (e.g., multiplication of cyclic triaxial strengths by factors ranging from 0.57 to 1 for soils where the lateral earth pressure coefficient, K_0, ranges from 0.4 to 1, respectively, described by Seed and Idriss, 1982). Estimation of field cyclic strengths from laboratory cyclic test results may not be possible by universal application of simple factors; recent research has shown that the comparison between cyclic triaxial and cyclic simple shear strengths depend on gradation, density, and soil type (Koester, 1992b). A companion report on liquefaction potential evaluation for nuclear power facilities details the state-of-the-art applied to cyclic strength evaluation (Koester and Sharp, 1999).

The constant volume condition implicitly assumed to accompany undrained testing behavior required for cyclic triaxial tests may be compromised in coarse-grained soil specimens by membrane compliance. The compliance imparts compressibility that is a relic of the test boundary conditions and therefore not representative of true, in situ behavior. The elastic membrane used to encapsulate soil specimens in, for example, triaxial tests, may intrude into the voids at the circumferential surface of the cylindrical specimen. Research by Evans (1987), Hynes (1988), and others have shown that, in cyclic tests, pore water pressures developed within the soil specimen may be partially relieved by these surficial voids, since the confining fluid will not resist the tendency for the membrane to deform. Membrane compliance may result in artificially high undrained cyclic strengths (resistance to liquefaction) of individual specimens, leading to unconservative evaluation of the liquefaction resistance of a deposit in situ. Membrane compliance is a function of gradation, and the correction that must be applied to adjust cyclic strengths is a function of specimen size, confining pressure, and other factors. The magnitude of the membrane compliance effect on cyclic strength is exemplified by the data represented in Figure 8, from Hynes (1988), which may serve as a correction chart.

7.2.1.1 Tatsuoka, et al. (1994). This report covers the measuring of local strains in cyclic triaxial tests was reported in Tatsuoka, et al. (1994). The deformation properties of sand and gravel were evaluated by cyclic loading triaxial tests and monotonic loading triaxial compression tests using small and large triaxial apparatuses. The results were compared with those from other testing methods. Two types of local gages set on the lateral surface of the specimen measured local axial and radial strains. The authors concluded that:

(1)   Local measurements of axial strain are imperative for accurate evaluation of stiffness of granular materials (sands and gravels) in both monotonic and cyclic loading triaxial tests. This is also the case for measuring damping characteristics in cyclic loading triaxial tests.

(2)   In triaxial tests at a constant confining pressure, accurate radial strains can be obtained by measuring the change in the specimen diameter.

(3)   Stiffness and damping during cyclic loading of granular materials for a strain range from about 0.0001 percent to about 1.0 percent can be obtained from static cyclic loading tests (i.e., very low frequency tests). The initial elastic stiffness can be estimated also by relevant monotonic loading triaxial compression tests, since the behavior is nearly elastic at strains less than about 0.001 percent.

(4)   It is suggested that field values of stiffness at small strains (say, up to 0.1 percent) under monotonic and cyclic loading conditions can be estimated from field shear wave velocities while taking into account that its dependency on pressure level and strain level. The strain level-

dependency is obviously different between monotonic and cyclic loading tests and between torsional and triaxial tests.

7.2.1.2 Vucetic and Dobry (1991). Vucetic and Dobry studied the influence of plastic index on the cyclic stress-strain parameters of saturated soils. Figure 9 shows the effect of PI on the location of the modulus reduction curve G/G_{max} (G - shear modulus; G_{max} - shear modulus at small strains) versus cyclic shear strain γ_c, and on the material damping ratio λ versus γ_c curve. These charts are based on experimental data from 16 publications on testing of normally consolidated and overconsolidated clays (OCR = 1 - 15), and sands. From the charts, in the paper it is shown that the PI may strongly control G/G_{max} and λ for a wide variety of soils. If for a given γ_c the PI increases, G/G_{max} rises and λ is reduced. Also presented was the influence of the PI on the rate of modulus degradation with the number of cycles in normally consolidated clays. Vucetic and Dobry (1991) concluded that soils with higher plasticity tend to have a more linear cyclic stress-strain response at small strains and to degrade less at larger γ_c than soils having lower PI. Possible reasons for this behavior are discussed in their paper. The charts in Figure 9 are recommended for use in preliminary seismic site-response evaluations and microzonation. Because of the scatter of the data points used to produce the charts, they should be applied with caution.

7.2.2 Resonant Column

7.2.2.1 ASTM D 4015. *Modulus and Damping of Soils by the Resonant-Column Method.* Published in 1981 and revised 1992, this standard determines the shear modulus, shear damping, rod modulus (Young's modulus), and rod damping for solid cylindrical specimens of soil in undisturbed and remolded conditions by vibration using the resonant column.

7.2.2.2 Andréasson (1981). Andréasson (1981) studied the shear modulus of a soft, high-plastic clay under dynamic loading conditions. Two resonant column devices, designed and built at Chalmers University of Technology in Sweden, testing both solid and hollow soil samples, were used in the test. One of the two resonant column devices is shown in Figure 10. Andréasson concluded that the initial shear modulus determined by shear wave velocity measurements in-situ and resonant column tests in the laboratory were in good correlation, provided that the laboratory test results were extrapolated to a time period corresponding to the age of the soil deposit.

The reduction of shear modulus with increasing shear strain amplitude was studied in the dynamic loading screw-plate tests in-situ and in the high amplitude resonant column tests in the laboratory. Good correlation between the field and laboratory test results was achieved, if the shear strain within the influenced soil in the screw-plate tests was calculated as 0.75 times the relative deformation of the screw plate.

7.2.3 Resonant Column and Torsional Shear

7.2.3.1 Zavoral, et al. (1994). Zavoral, et al. (1994) evaluated the effect of frequency on the dynamic response of clay. Details of the modified apparatus are described and test results over a wide range of frequencies at different strain levels were presented. The results show that the dynamic shear modulus of samples tested by the resonant column in axial vibration was always greater than the dynamic shear modulus of the same samples tested at lower frequencies in torsional shear. Samples were tested in torsional shear at various frequencies to confirm that the increase in shear modulus was due to frequency (strain rate). Results indicated that the shear modulus always increased with frequency, though the effect was fairly minor. No observable effect of frequency on damping ratio was measured for samples tested in

either resonant column or torsional shear modes. There also was no significant variation in damping ratio for samples tested in torsional shear under various frequencies.

7.2.3.2 Kim, et al. (1991). Kim, et al. investigates the assumption that material damping in shear of soil measured by hysteresis loops is zero at small strains, strains less than about 0.001 percent. A single piece of equipment is used for both the torsional shear (TS) and resonant column (RC) tests (Figure 11). Material damping in shear for a dry sand and a compacted clay were obtained by varying the strain level, frequency of the excitation and number of loading cycles. Their results show that for the dry sand the damping ratios obtained from the RC and TS shear test match at shearing strains below about 0.002 percent (Figure 12). At higher strains, the damping ratios are sensitive to number of loading cycles (Figure 13). Damping values obtained from the first TS cycle are much larger than those computed from later cycles or from the RC test. Damping ratios from TS and RC test are essentially equal at the same strain level and the same number of cycles. The effect of frequency on material damping is negligible for this dry sand.

For the compacted clay, the damping ratios obtained from the TS and RC tests are different over the complete strain range (from 0.0004 to 0.05 percent). This difference results from the difference in frequencies used in the two methods of testing. However, the effect of frequency in the TS test does not begin to increase material damping until the frequency exceeds about 5 Hz. The effect of the number of loading cycles is negligible in the compacted clay. The device was modified so that damping measurements could be made over strains ranging from about 10^{-4} to 10^{-1} percent.

7.3 Testing Procedures of Determining Engineering Properties of Rock

7.3.1 Porosity

ASTM D 4404. *Determination of Pore Volume and Pore Volume Distribution of Soil and Rock by Mercury Intrusion Porosimetry.* The current edition of ASTM D 4404 was last reapproved in 1992. This test method describes the determination of pore characteristics of soil and rock for the range of apparent pore entrance diameters of about 100 μm and 2.5 nm (0.0025 μm). The process uses mercury intrusion under varying pressures and will only measure volume of pores that are open to the outside of a soil or rock fragment. The size and volume of pores affects the integrity and behavior of soil or rock. The standard describes special porosimeter equipment and testing limitations.

7.3.2 Permeability

ASTM D 4525. *Permeability of Rocks by Flowing Air.* The current edition of this testing method was published in 1990. It determines the coefficient of specific permeability for the flow of air through rocks. Designed to measure the permeability to air of a small sample of rock, by extrapolation, this method can also be used to determine an equivalent of the liquid permeability.

7.3.3 Seismic Velocity

ASTM D 2845. *Laboratory Determination of Pulse Velocities and Ultrasonic Elastic Constants of Rock.* The original version of this standard was published in 1969; the last previous edition was in 1983 and reapproved in 1990. This method describes equipment and procedures for measurement of the pulse velocities of compression waves and shear waves in rock in the laboratory. It also determines the ultrasonic elastic constants of an isotropic rock.

7.3.4 Direct Tensile Strength

ASTM 2936. *Direct Tensile Strength of Intact Rock Core Specimens.* Originally published in 1971 and reapproved in 1995, this standard determines the tensile strength of intact cylindrical rock. To determine the failure condition of a structure the tensile strength is used as the failure strength for the structure.

7.3.5 Unconfined Compression

7.3.5.1 ASTM D 2938. *Unconfined Compressive Strength of Intact Rock Core Specimens.* This standard method was originally published in 1971, reapproved and published again in 1995. The apparatus, instrumentation, and procedures for the determination of the unconfined compressive strength of intact rock core specimens are discussed in the method. The unconfined compressive strength is used in design formulas and sometimes as an index property for selecting the appropriate excavation technique.

7.3.5.2 ASTM D 3148. *Elastic Moduli of Intact Rock Core Specimens in Uniaxial Compression.* The currents edition of this method was published in 1993, it describes the procedures used to determine the elastic moduli of intact rock core specimens in uniaxial compression. The stress-axial strain and the stress-lateral strain curves, along with the Young's modulus and Poisson's ratio can be determined by this standard.

7.3.5.3 ASTM D 3967. *Splitting Tensile Strength of Intact Rock Core Specimens.* The current edition of the standard test method was published in 1995. It determines the splitting tensile strength of rock by diametral line compression of a disk. This test is an alternative to the direct uniaxial tensile test which is difficult and expensive for routine testing.

7.3.6 Triaxial Compression

7.3.6.1 ASTM D 5407. *Elastic Moduli of Undrained Intact Rock Core Specimens in Triaxial Compression Without Pore Pressure Measurements.* Approved and published in 1993, ASTM D 5407 determines the elastic moduli of intact rock core specimens in undrained triaxial compression. The stress-axial strain and the stress-lateral strain curves along with Young's modulus and Poisson's ration can be determined using the procedures detained in the standard method.

7.3.6.2 ASTM D 2664. *Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements.* This method was originally published in 1967. The current edition was approved and published in 1986. It covers the determination of the strength of cylindrical rock core specimens in an undrained state under triaxial compression loading. The data provided by this method is useful in determining the strength and elastic properties of rock without pore pressure measurements.

7.3.7 Slake Durability

Slaking tests are valuable when the project is to be found on or within moisture-sensitive clays and clay shales, and foundation design requirements indicate that the foundation and cut slope areas will be exposed temporarily to wetting and drying conditions.

ASTM D 4644. *Slake Durability of Shales and Similar Weak Rocks.* This test method was published in 1987, and describes the procedures used to determine the slake durability index of shale and rock after two drying and wetting cycles with abrasion.

8 CONCLUSION

The main purpose of this document was to provide basic information on laboratory testing methods that have been developed and put into practice since the publication of the current Regulatory Guide 1.138 (1972). It was not the intent of the authors to describe in detail the testing methods discussed (i.e., detailed equipment requirements, detailed procedures, etc.), only to give a brief description of current research and accepted methods available. The inclusion of any particular testing method or equipment should not be perceived as an endorsement by the authors. This decision is left solely up to the Nuclear Regulatory Commission.

Although it would be impossible to describe all of the research conducted in laboratory testing since 1972, this document has covered some of the most accepted methods and recommends that as extended research, the users should consider the references listed by a particular method.

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APPENDIX A
ABBREVIATIONS AND SYMBOLS

λ	Damping Ratio
γ_c	Cyclic Shear Strain
ASTM	American Society for Testing and Materials
c_v	Coefficient of Consolidation
D	Diameter
EM	Engineering Manual
G	Shear Modulus
G_a	Apparent Specific Gravity
G_m	Bulk Specific Gravity
G_{max}	Shear Modulus at Small Strains
G_s	Specific Gravity of Solids
K_o	Lateral Earth Pressure Coefficient
LL	Liquid Limit
L_s	Linear Shrinkage
OCR	Overconsolidation Ratio
PI	Plasticity Index
PL	Plastic Limit
PRC	Peoples Republic of China
R	Shrinkage Ratio
RC	Resonant Column
RTH	Rock Testing Handbook
SL	Shrinkage Limit
TS	Torsional Shear

APPENDIX B
LABORATORY TESTING METHODS FOR SOIL AND ROCK

Name of Test	STANDARD OR PREFERRED METHOD	OTHER REFERENCES	PROPERTIES OR PARAMETERS DETERMINED	REMARKS/SPECIAL EQUIPMENT REQUIREMENTS
		SOILS — INDEX AND CLASSIFICATION TESTS		
Gradation Analysis	ASTM D 421 D 422 D 2217 D 4221	Ref. 12, 13, 29, 34, 38	Particle size distribution	Methods are applicable to some rocks, after disaggregation.
Percent fines	ASTM D 1140	Ref. 12, 34, 38	Percent of weight of material finer than No. 200 sieve.	
Atterberg Limits	ASTM D 427 D 4318 D 4943 Ref. 12	Refs. 7, 11, 13, 31, 34, 35, 57	Liquid and plastic limit, plasticity index, shrinkage factor (limit)	
Specific Gravity	ASTM D 854 D 5550 Ref. 12	Ref. 13, 38	Specific gravity, apparent specific gravity, bulk unit weight sufficiently fine to eliminate internal voids in the intact rock.	Boiling should not be used for de-airing. Method can be used for rock, after grinding
Radiography	ASTM D 4452			
Description of Soil and Rock	ASTM D 2488 D 4452 RTH 102-93 TM 3-357		Description of soil from visual-manual examination	
		SOILS — MOISTURE-DENSITY RELATIONS		
Bulk Unit Weight	Ref. 12		Bulk unit weight (bulk density)	Methods are applicable to some rocks, with some Obvious modifications
Water (Moisture) Content	ASTM D 425 D 1558 D 2216 D 2974 D 4643 D 4959 Ref. 12	Refs. 13, 20, 21, 24, 35, 37	Water content as a percent of dry weight	Method is applicable to rock.
Relative Density	Ref. 12		Maximum and minimum density of cohesionless soils	Requires vibration table. In vibration table testing, both amplitude and frequency should be adjusted to values that yield greatest density. However, treatment that produces breakage of grains should be avoided and mechanical analyses should be performed as a check on grain breakage.

Appendix B Laboratory Testing Methods for Soil and Rock

Name of Test	STANDARD OR PREFERRED METHOD	OTHER REFERENCES	PROPERTIES OR PARAMETERS DETERMINED	REMARKS/SPECIAL EQUIPMENT REQUIREMENTS
Compaction	ASTM D 698 D 1557 D 4253 D 4254 D 5080 Ref. 12	Ref. 13, 40	Maximum dry unit weight of soil	Method for earth-rock mixtures is given in Ref 6.
SOILS — CONSOLIDATION AND PERMEABILITY				
Consolidation	ASTM D 2435 D 4186 Ref. 12	Ref. 13, 26, 34, 38, 40	One-dimensional compressibility, permeability of cohesive soil	
Permeability	ASTM D 2434 D 5084 Ref. 12	Ref. 13, 28, 29	Permeability	Suitable for remolded or compacted soils. For natural, In situ soils, field test should be used.
SOILS — PHYSICAL AND CHEMICAL PROPERTIES				
Mineralogy	Ref. 59	Refs. 1, 2, 17, 63	Identification of minerals	Applicable to rock. Requires X-ray diffraction apparatus Differential thermal analysis apparatus may also be used.
Organic Content	Ref. 1	ASTM D 2974 Ref. 44	Organic and inorganic carbon content as percent of dry weight.	Dry combustion methods (ASTM D 2974) are acceptable, but where organic matter content is critical, data so obtained should be verified by wet combustion tests.
Soluble Salts	ASTM D 4542	Ref. 52, 55		Concentration of soluble Salts in soil pore water
Erodibility Tests Pinhole Test	ASTM D 4221 D 4647 Ref. 12	Refs. 46, 47, 41		Significant in evaluation of potential erosion or piping.
Crumb Test	Ref. 12			
SCS Test	Ref. 12	Ref. 24, 27		
Cylinder Dispersion	Ref. 12			
SOILS — SHEAR STRENGTH AND DEFORMABILITY				
Unconfined Compression	ASTM D 2166	Ref. 12	Strength of cohesive soil in uniaxial compression.	
Direct Shear, Consolidated-drained	ASTM D 3080 Ref 12		Cohesion and angle of internal friction under drained conditions	
Triaxial Compression, Unconsolidated-Undrained	ASTM D 2850 Ref. 12	Ref. 6, 13	Shear strength parameters; Cohesion and angle of internal friction for soils of low permeability.	
Triaxial Compression, Consolidated-Drained	Ref. 12	Refs. 6, 13	Shear strength parameters; Cohesion and angle of internal friction. For long-term loading conditions.	Circumferential drains, if used, should be slit to avoid stiffening test specimen.

Name of Test	STANDARD OR PREFERRED METHOD	OTHER REFERENCES	PROPERTIES OR PARAMETERS DETERMINED	REMARKS/SPECIAL EQUIPMENT REQUIREMENTS
Triaxial Compression, Consolidated-Undrained	ASTM D 4767 Ref. 12	Refs. 6, 13	Shear strength parameters; Cohesion and angle of internal friction for consolidated soil. With pressure measurements, cohesion and friction may be obtained.	Circumferential drains, if used, should be slit to avoid stiffening of test specimen.
Cyclic Triaxial	ASTM D 3999 D 5311 Ref. 49	Refs. 14, 32, 35, 45, 46, 47, 48, 50, 54, 57, 62	Local strain, modulus and damping	
Cyclic Simple Shear		Refs. 134 40, 51, 55	Shear modulus and damping values and cyclic-strength of cohesive and cohesionless soils	Tests may be run with either stress control or strain control. Two different types of apparatus, NGI and Roscoe devices, are described in Refs. 16 and 58, respectively.
Resonant Column	ASTM D 4015	Refs. 1, 4, 23, 30, 33, 60, 63	Shear modulus and damping in cohesive and cohesionless soils. Some devices can be used with deformations in longitudinal mode to determine Young's modulus. Some devices can be used to determine cyclic strength.	Requires resonant column device.

ROCKS ---- ENGINEERING PROPERTIES

Name of Test	STANDARD OR PREFERRED METHOD	OTHER REFERENCES	PROPERTIES OR PARAMETERS DETERMINED	REMARKS/SPECIAL EQUIPMENT REQUIREMENTS
Porosity	ASTM D 4612	Refs. 9, 15	Bulk unit weight, specific gravity, and total porosity (Melcher Method) or effective porosity (Simmons or Washburn-Bunting Method)	Soil testing methods generally applicable with minor modification.
Permeability	ASTM D 4525	Refs. 9, 15	Permeability of intact rock	Laboratory permeability values are not normally representative of in situ permeability of shallow jointed rock masses.
Seismic Velocity	ASTM D 2845	Refs. 21, 35	Compressional and shear wave velocities in intact rock	Requires signal generator, transducers, oscilloscope.
Direct Tensile Strength	ASTM D 2936	Ref. 35	Uniaxial tensile strength of intact rock	
"Brazilian Test"	ASTM D 3967	Ref. 35	Indirect measure of tensile strength of intact rock	
Modulus of Rupture		Ref. 35	Indirect measure of tensile strength of intact rock	
Unconfined Compression	ASTM D 2938	Ref. 35	Young's moduli and unconfined compression strength of intact rock	
Uniaxial Compression	ASTM D 3148			
Triaxial Compression Undrained	ASTM D 2664	Ref. 35	Young's moduli, cohesion friction parameters of failure envelope	

Appendix B   Laboratory Testing Methods for Soil and Rock

Name of Test	STANDARD OR PREFERRED METHOD	OTHER REFERENCES	PROPERTIES OR PARAMETERS DETERMINED	REMARKS/SPECIAL EQUIPMENT REQUIREMENTS
Triaxial Compression Without Pore Pressure Measurements	ASTM D 5407			
Triaxial Compression With Pore Pressure Measurements		Ref. 24	Young's moduli, cohesion friction parameters of effective stress conditions.	
Slake Durability	ASTM D 4644	Ref. 16	Index of resistance to slaking	
Direct Shear	ASTM D 5607			

APPENDIX C
FIGURES .
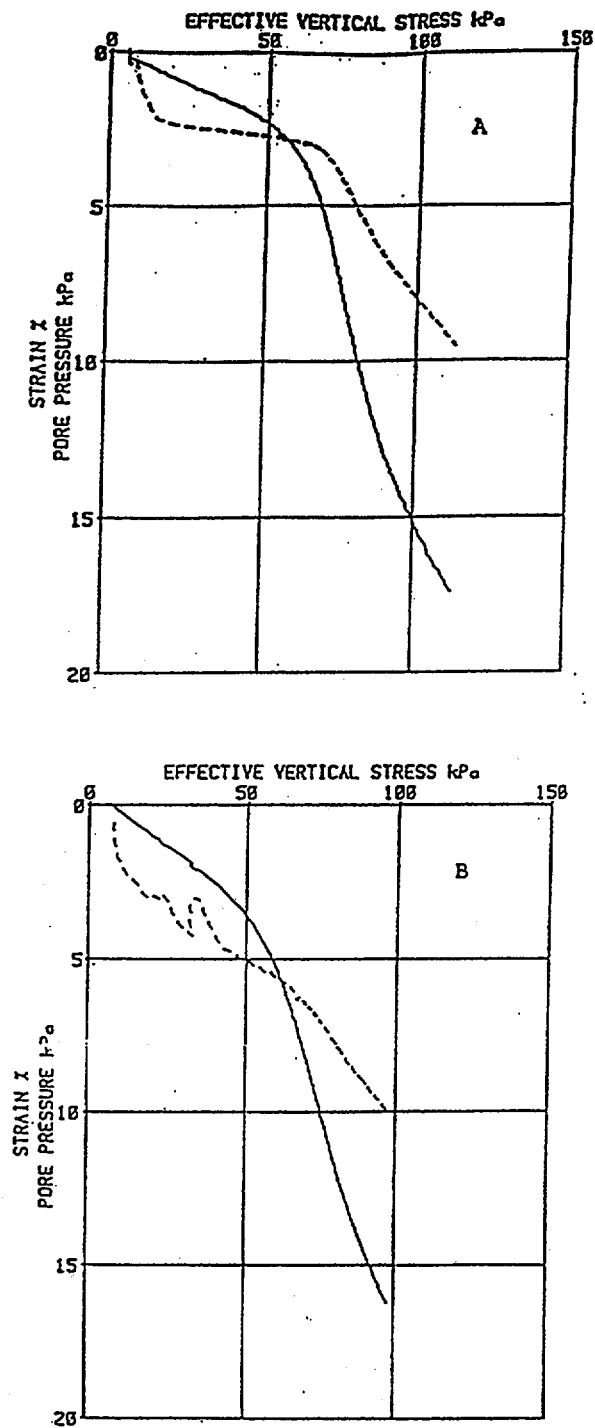
Figure 1.   Consolidation test data on soft clay: a) high quality equipment; b) oedometer ring 0.2 mm too large (Sallfors, 1989, reprinted with permission from A. A. Balkema Publishers, Ltd.)
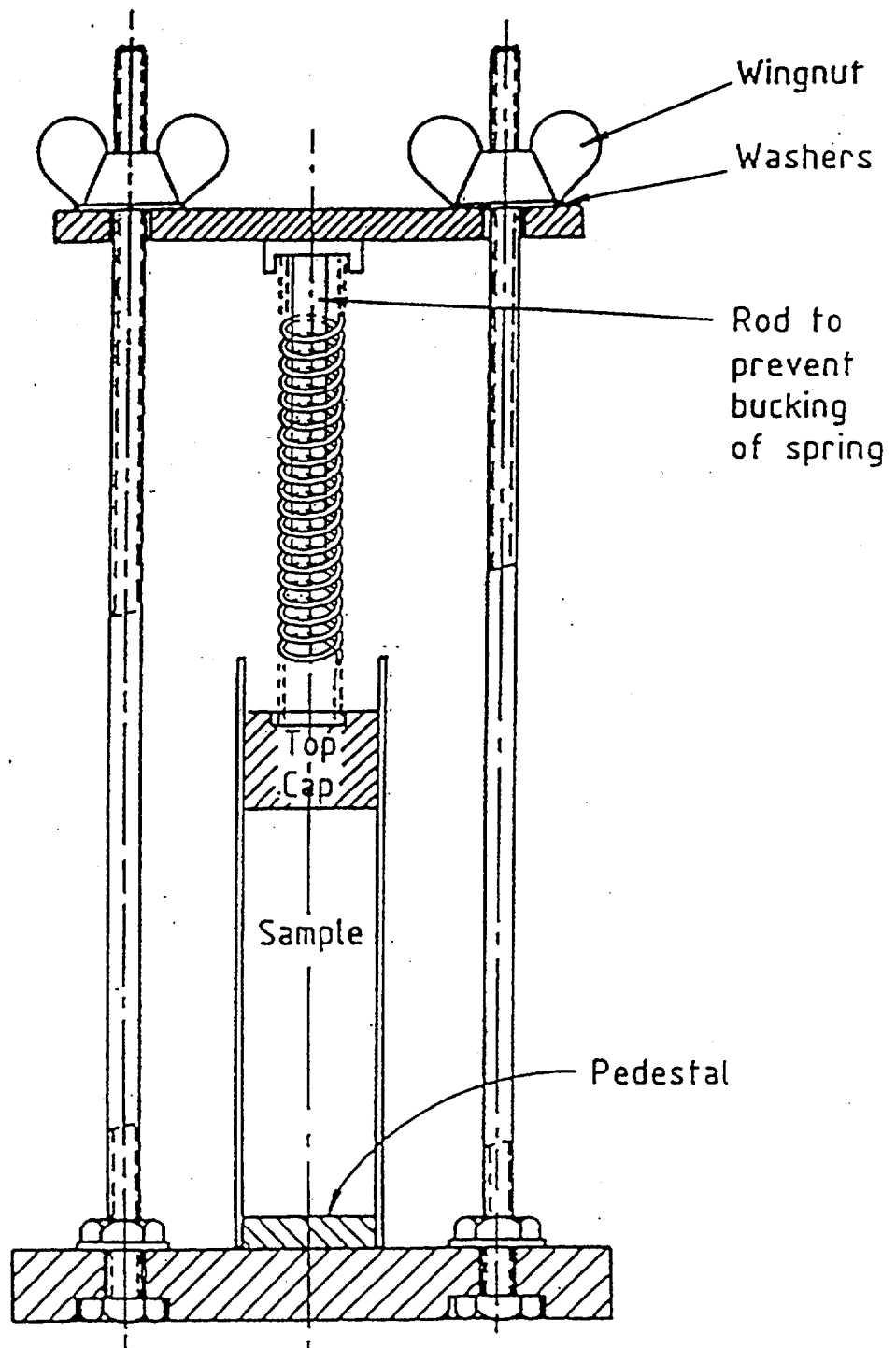
Figure 2.   Simpler version of the sample compressor (Brown and Chow, 1988, reprinted with permission from ASTM)

6.4mm(0.25in)

3.2± 0.5mm
(0.125± 0.02in.)

114.3mm
(4.5in.)

215.9mm
(8.5in.)

215.9mm
(8.5in.)

101.6mm
(4.0in.)

Figure 3.   Plastic limit device (Bobrowski and Griekspoor, 1992, reprinted with permission from
ASTM)

Figure 4. Apparatus for pinhole test: (a) general arrangement, (b) details of nipple (Head, 1994, reprinted with permission from John Wiley & Sons, Ltd.)

A. HYDROSTATIC SEAL AIR SUPPLY PORT
B. VENT
C. TOP DRAINAGE PORT
D. CHAMBER FLUID DRAIN
E. TRANSDUCER CABLE CONNECTOR
F. BOTTOM DRAINAGE PORT

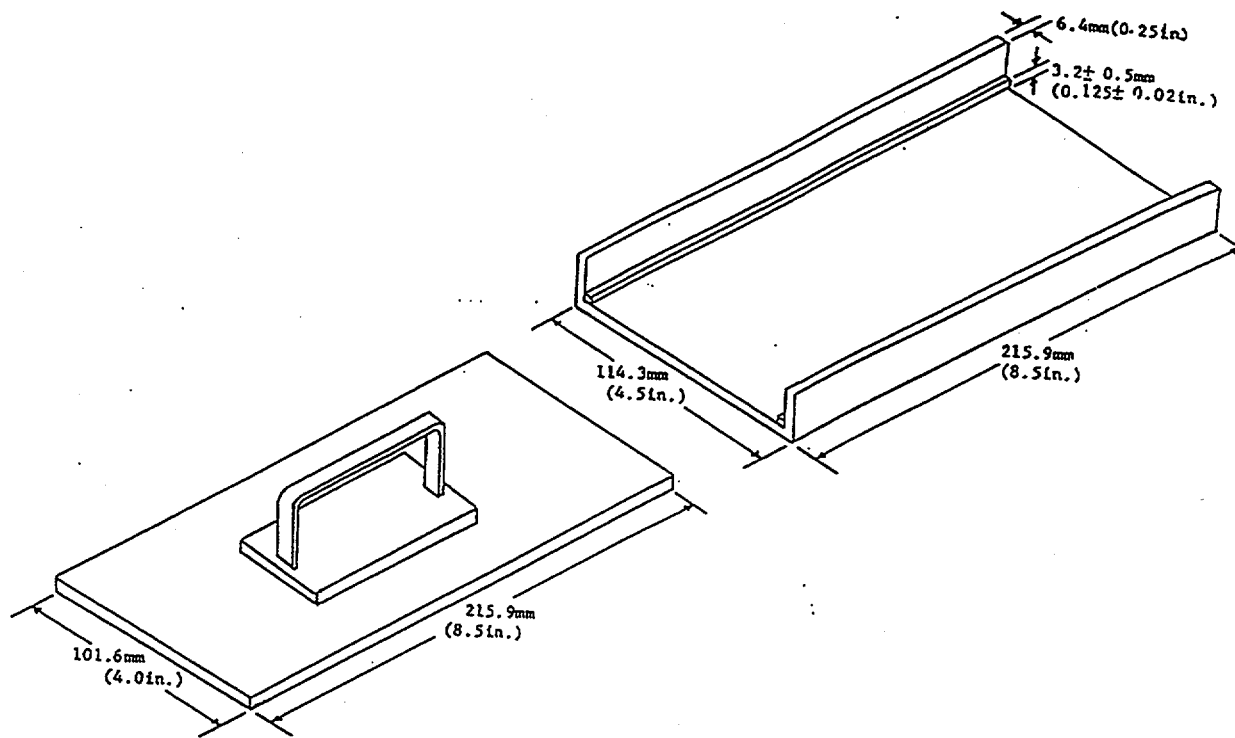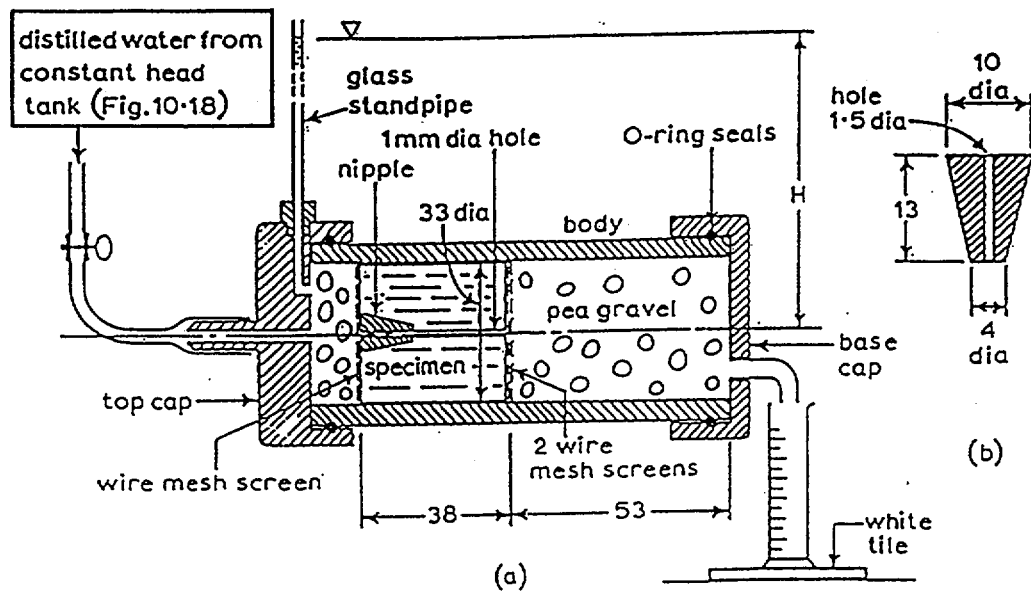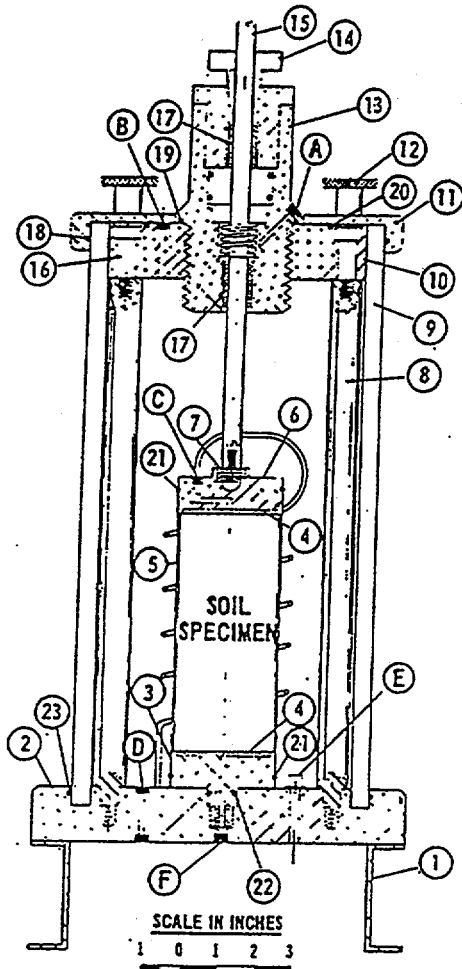| PART NO.	NAME	MATERIAL	QUANTITY
1	CHAMBER FOOT	ALUMINUM CHANNEL	2
2	CHAMBER BASE	ALUMINUM PLATE	1
3	SPECIMEN BASE	ALUMINUM	1
4	POROUS PLATE	STAINLESS STEEL	2
5	SPECIMEN MEMBRANE	RUBBER	1
6	SPECIMEN CAP	ALUMINUM	1
7	SPECIMEN CAP CONNECTOR	BRASS	1
8	BEARING SUPPORT ROD	STAINLESS STEEL	2
9	CYLINDER	CLEAR ACRYLIC	1
10	CHAMBER CAP	ALUMINUM	1
11	CYLINDER SEALING RING	ALUMINUM	1
12	LOCKING KNOBS	STEEL	3
13	AIR BEARING ASSEMBLY	ALUMINUM	1
14	PISTON LOCK	BRASS	1
15	LOADING PISTON	STAINLESS STEEL	1
16	SHOULDER SCREW	STEEL	2
17	BALL BUSHING	STAINLESS STEEL	2
18	O-RING	RUBBER	1
19	O-RING	RUBBER	1
20	O-RING	RUBBER	1
21	O-RING	RUBBER	2
22	O-RING	RUBBER	1
23	O-RING	RUBBER	1

SOIL SPECIMEN

SCALE IN INCHES
1 0 1 2 3

TRIAXIAL COMPRESSION CHAMBER

Figure 5.   Diagram of cyclic triaxial test equipment (EM 1110-2-1906, Department of the Army, 1986)
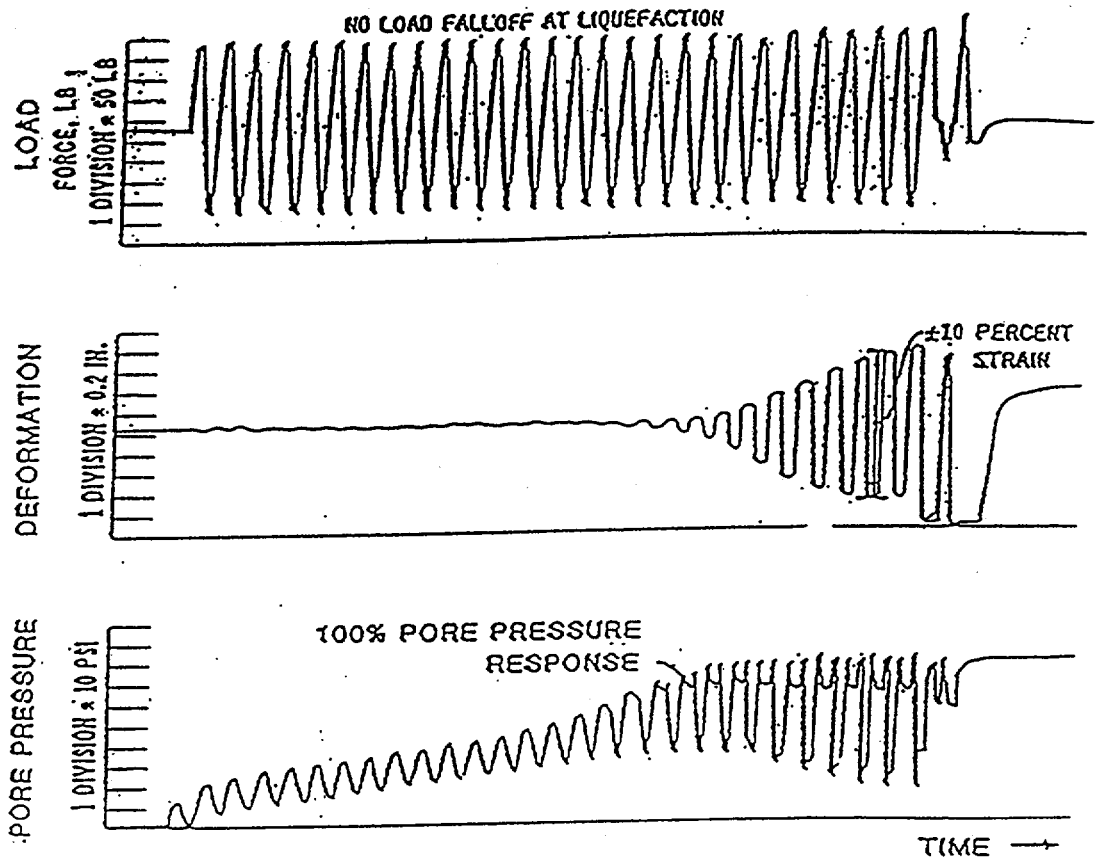
Figure 6.   Typical analog recordings of load, deformation, and pore pressures during a cyclic triaxial test (EM 1110-2-1906, Department of the Army, 1986)
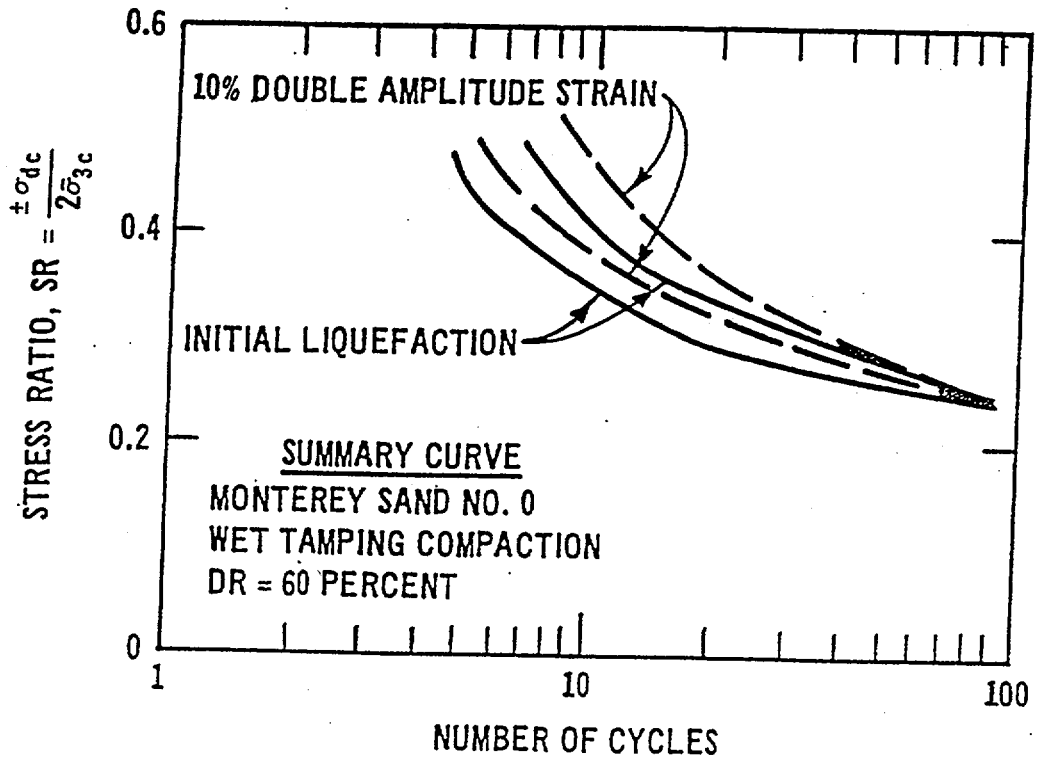
Figure 7.   Cyclic triaxial strength curves for Monterey No. 0 sand (EM 1110-2-1906, Department of the Army, 1986)
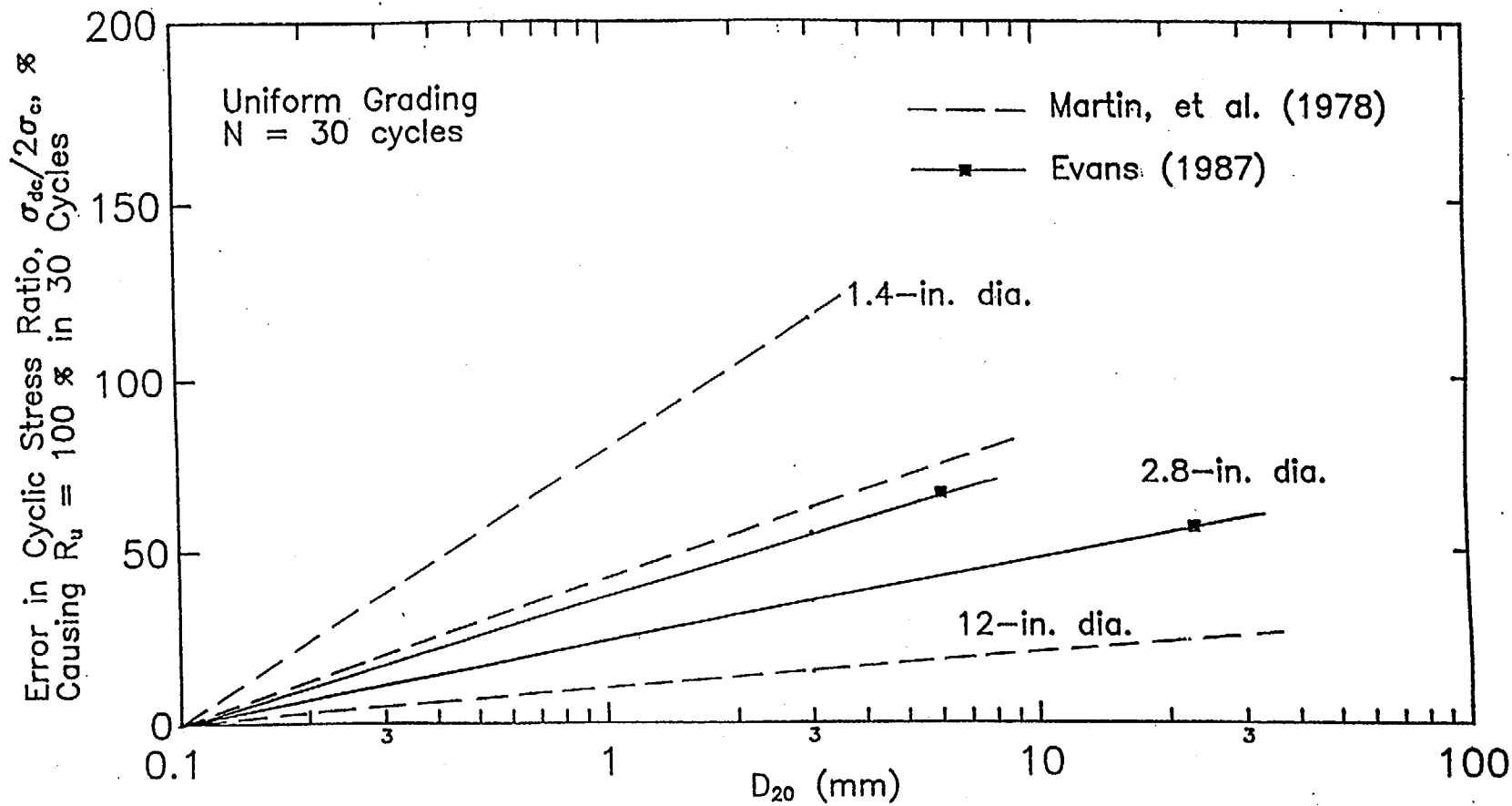
Figure 8.   Error in cyclic stress ratio due to membrane compliance versus D_{20} for various specimen diameters (Hynes 1988, after Evans 1987)
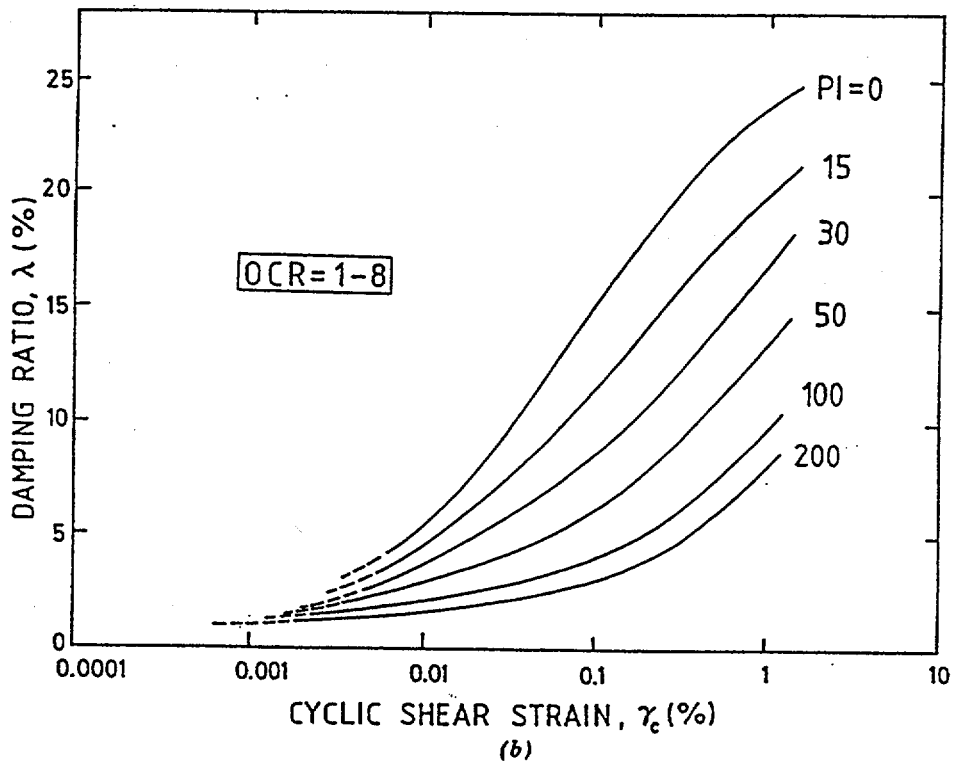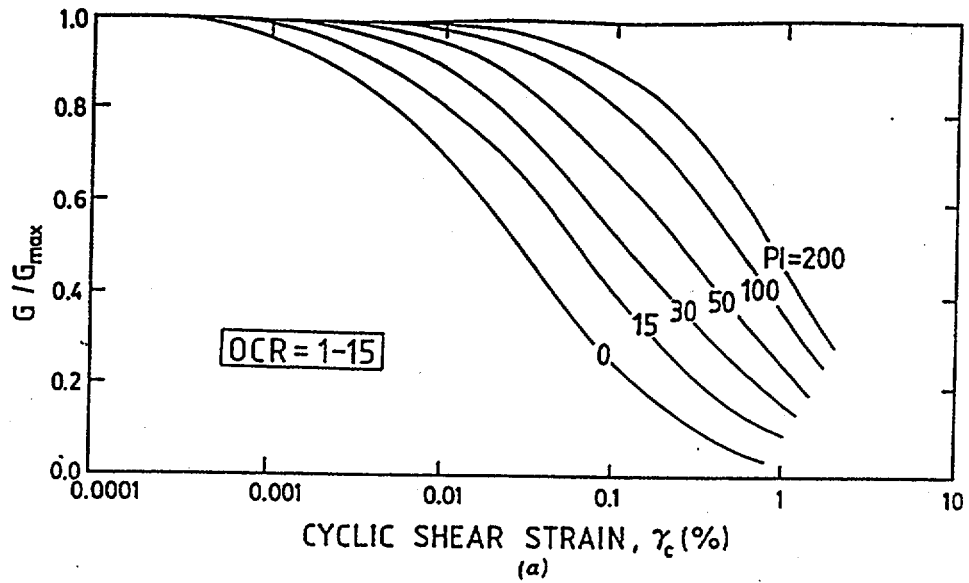
Figure 9. Relations between (a) G/G_{max} versus γ_c and (b) λ versus γ_c curves and soil plasticity for normally and overconsolidated soils (Vucetic and Dobry, 1991, reprinted with permission from ASCE)
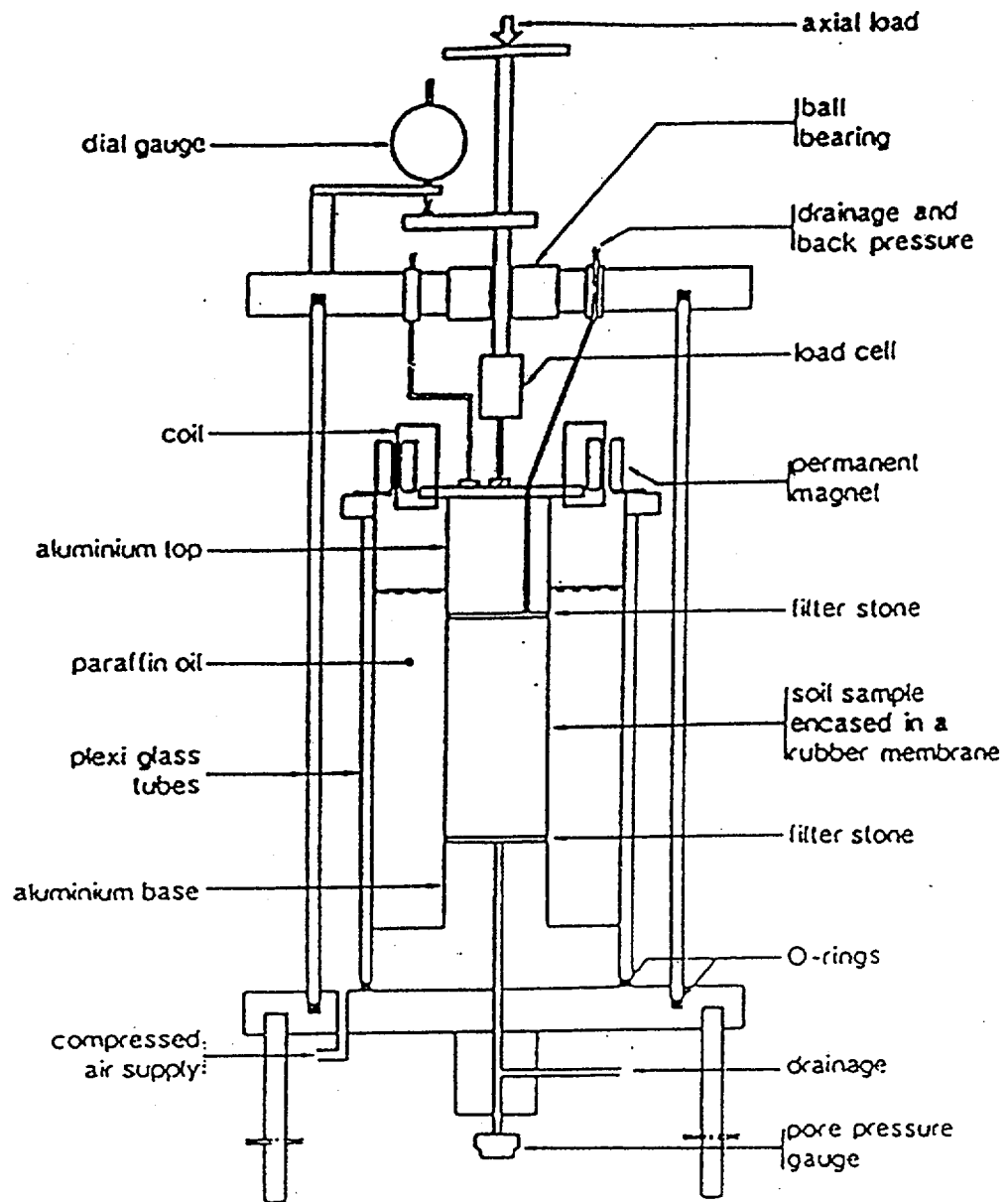
Figure 10.   Resonant column device (Andréasson, 1981)
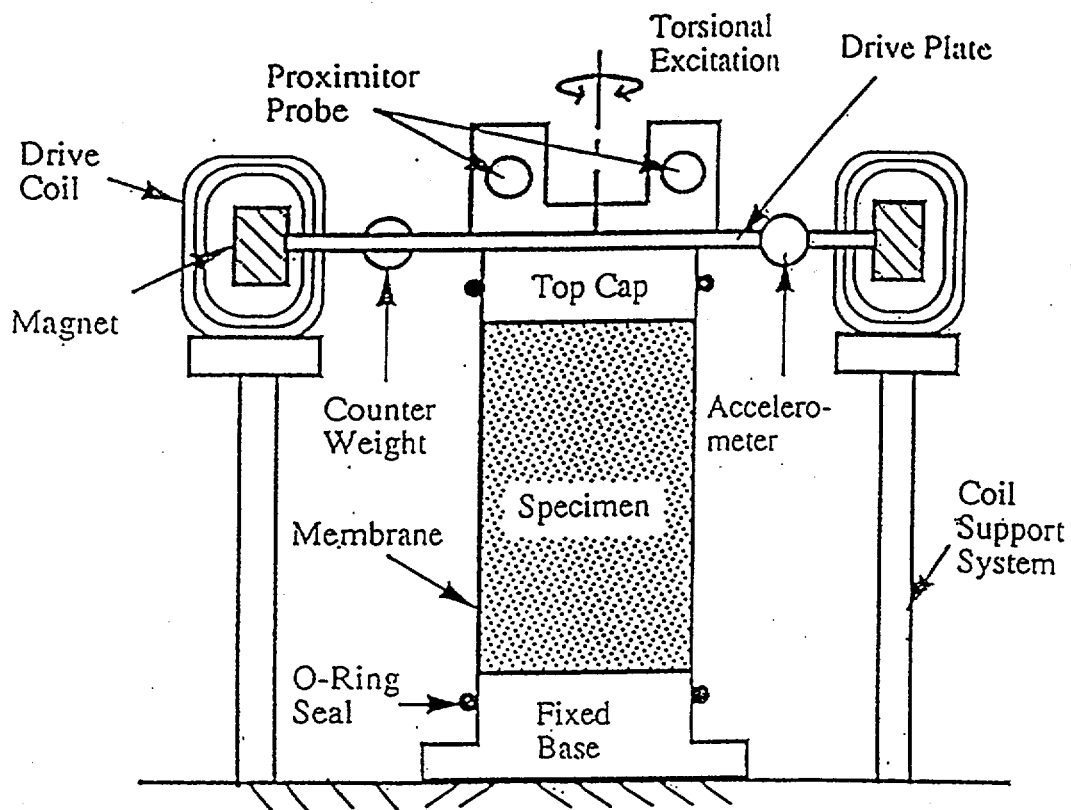
Figure 11. Configuration of simplified resonant column/ torsional shear (RCTS test equipment (without outer confinement chamber)) (Kim, et al. 1991, reprinted with permission; copyright 1991, Computational Mechanics Publications, U.K.)
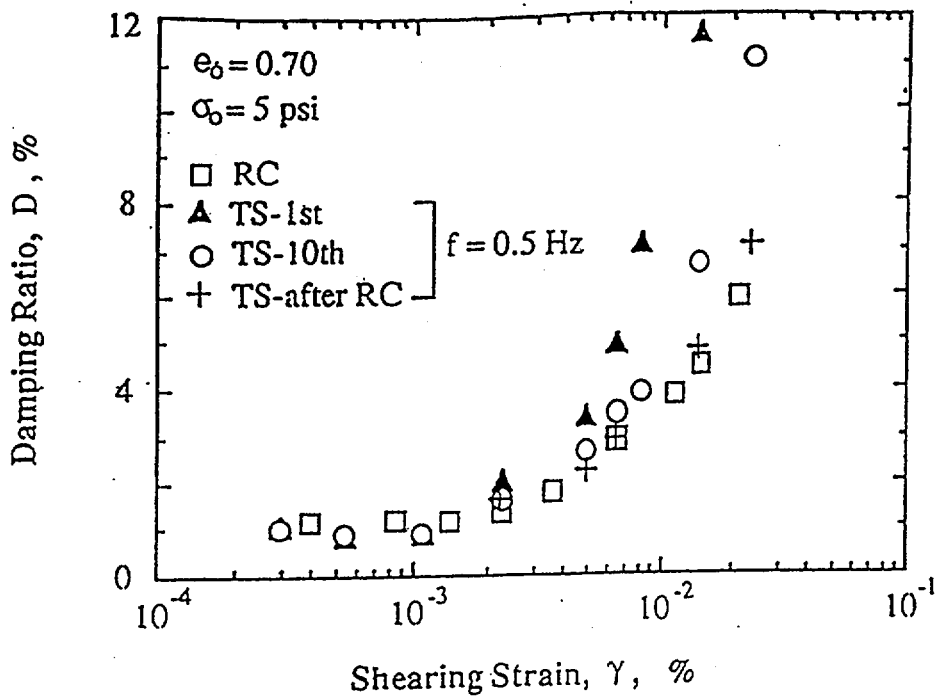
Figure 12.   Variation in damping ratio with strain at different loading cycles (for dry sand)
(Kim, et al., 1991, reprinted with permission, copyright 1991, Computational Mechanics
Publications, U.K.)

Figure 13.   Variation in damping ratio with number of cycles (for dry sand) (Kim, et al., 1991, reprinted
with permission, copyright 1991, Computational Mechanics Publications, U.K.)

**APPENDIX D
TABLES**

Appendix D Tables

Table 1. Chemical Tests for Soils (Head, 1992)

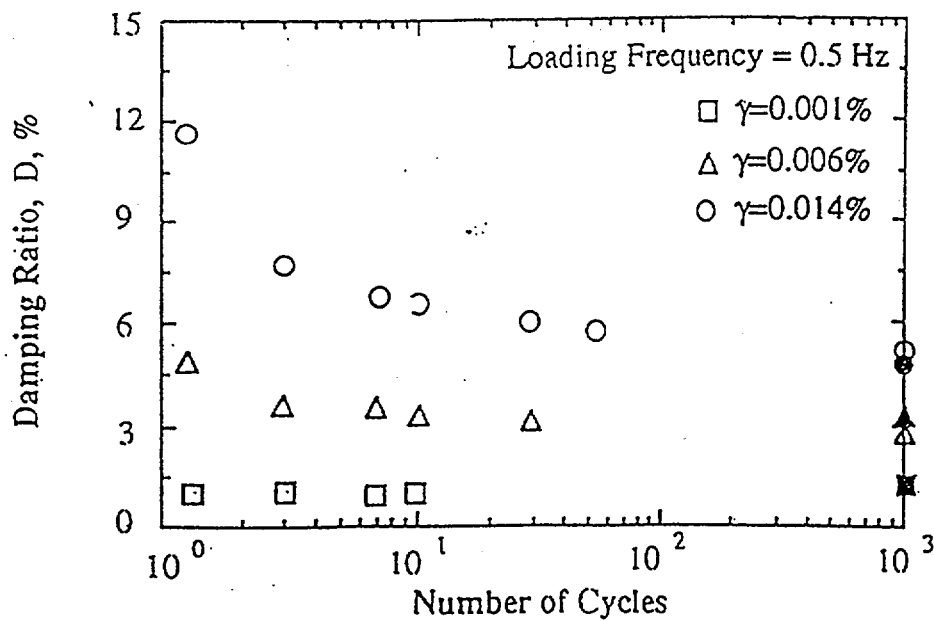| Section reference	Type of test	Procedure	Reference ('BS' implies BS1377:1990)	Uses	Limitations and comments
5.5	pH value	Indicator papers	Supplier's instructions	Simple and quick. Useful for determining approximate pH range for a more sensitive test.	Gives approximate values only.
		Colorimetric (Kuhn's method)		Quick field test for soils. Apparatus available as a kit.	Requires colour comparison with chart printed in British standard. Values given to nearest 0.5.
		Lovibond Comparator	Manufacturer's instructions	Colour comparison with standard coloured discs gives pH to nearest 0.2. Range of indicators available.	
		Electrometric	BS Test Part 3:9	BS 'standard' method. Accurate to 0.1 pH unit or better.	Requires a special electrical apparatus, although low-priced portable battery models are available. Electrodes age slowly, and should be checked periodically with buffer solutions.
5.6	Sulphate content	Total sulphates in soils	BS Part 3:5.2, 5.5 BRE Digest 250	Accurate if performed with care and with proper chemical testing facilities. Gives the total amount of sulphates present, including calcium sulphate, which is insoluble in water.	If the measured sulphate content is greater than 0.5%, the water-soluble sulphates should also be measured.
		Water-soluble sulphates in soils (gravimetric)	BS Part 3:5.3, 5.5	Accuracy as above. Gives the amount of water-soluble sulphates only, which are those most likely to attack concrete.	
		Water-soluble sulphates in soils (ion exchange)	BS Part 3:5.3, 5.6	Quick, easy	Cannot be used if chloride, nitrate or phosphate ions are present. Requires a special ion-exchange resin which needs reactivating frequently.
		Sulphates in groundwater (ion exchange)	BS Part 3:5.4, 5.6	As above.	As above.
		Sulphates in groundwater (gravimetric)	BS Part 3:5.4, 5.5	As for water-soluble sulphates in soils.	
5.7	Organic content	Peroxide oxidation		Eliminates organic matter before sedimentation particle size tests.	Has limited action on undecomposed plant remains (e.g. roots and fibres).
		Dichromate oxidation	BS Part 3:3	Accurate, if proper chemical testing facilities used. Suitable for all soils. Presence of carbon and carbonates does not affect results. Fairly rapid, suitable for small batches.	Presence of chlorides affects results but a correction can be applied if chlorides are measured separately. Their effect can be overcome by adding mercuric sulphate.
5.8	Carbonate content	Rapid titration	BS Part 3:6.3	For carbonate content exceeding 10%	Accuracy no better than 1% carbonates
		Gravimetric	BS Part 3:6.4 BS1881: Part 124	Requires precision weighing and chemical testing facilities.	Method as used for hardened concrete.
		Calcimeter (Collin's modification of Schleiber's apparatus)		Compact, simple, fairly quick. Measures the volume of carbon dioxide evolved.	An approximate method, but accurate enough for most engineering purposes. Atmospheric pressure must be known.
5.9	Chloride content	Reaction with silver nitrate (Volhard's method):		Titration process requiring proper chemical testing facilities. Designed for concrete aggregates.	Several standardised reagents are required
		Water soluble	BS Part 3:7.2		
		Acid soluble	BS Part 3:7.3		
		Mohr's titration method	Bowley (1979)	Simpler than Volhard's method. Designed for concrete aggregates.	Both methods require an analytical balance.
5.10.2	Total dissolved solids	Evaporation	BS Part 3:8	Simple procedure.	Requires very accurate weighing.
5.10.3	Loss on ignition	Ignition	BS Part 3:4	Destroys all organic matter. Suitable for sandy soils containing little or no clay or chalk.	High temperature breaks down certain minerals in clay, and carbonates, and removes water of crystallisation.
5.10.4	Concentration of certain salts	Indicator papers	Manufacturer's instructions	Very simple, quick, inexpensive.	Gives approximate indication only; not for accurate work. Presence of salts other than those being tested might affect readings

Table 2. Soil Classification Chart (Unified Soil Classification System, 1960)

LABORATORY A

Plastic Limit Results

Specimen	PI	Hand Method	Rolling Device
1	4	12	13
2	8	14	12
3	10	16	15
4	16	16	17
5	20	14	12
6	27	14	14
7	35	13	14
8	39	15	14
9	44	19	17
Avg.		15	14

Regression Output:

Constant	3.90
Standard Error of Y Estimated	1.26
R Squared	0.59
No. of Observation	9
Degrees of Freedom	7
X Coefficient(s)	0.699
Standard Error of Coefficient	0.218

LABORATORY B

Plastic Limit Results

Specimen	PI	Hand Method	Rolling Device
1	4	13	13
2	8	16	15
3	10	16	15
4	16	17	17
5	20	13	13
6	27	15	14
7	35	15	15
8	39	19	18
9	44	20	20
Avg.		16	16

Regression Output:

Constant	0.25
Standard Error of Y Estimated	0.55
R Squared	0.95
No. of Observations	9
Degrees of Freedom	7
X Coefficient(s)	0.957
Standard Error of Coefficient	0.081

LABORATORY C

Plastic Limit Results

Specimen	PI	Hand Method	Rolling Device
1	4	13	13
2	8	18	16
3	10	18	15
4	16	18	17
5	20	12	13
6	27	14	12
7	35	14	12
8	39	16	15
9	44	18	18
Avg.		16	15

Regression Output:

Constant	2.59
Standard Error of Y Estimated	1.21
R Squared	0.73
No. of Observation	9
Degrees of Freedom	7
X Coefficient(s)	0.764
Standard Error of Coefficient	0.174

Table 3. Plastic Limit Comparison Testing: Current Hand Method Versus Rolling Device (Bobrowski and Griekspoor, 1992)

CURRENT HAND METHOD RESULTS

Sample Number

	1	2	3	4	5	6	7	8	9
Laboratory A	12	14	16	16	14	14	13	15	19
Laboratory B	13	16	16	17	13	15	15	19	20
Laboratory C	13	18	18	18	12	14	14	16	18
Average	13	16	17	17	13	14	14	17	19
Standard Deviation	0.6	2.0	1.2	1.0	1.0	0.6	1.0	2.1	1.0
Range	1	4	2	2	2	1	2	4	2
Average Standard Deviation	1.15								
Average Range	2.22								

PROPOSED ROLLING DEVICE RESULTS

Sample Number

	1	2	3	4	5	6	7	8	9
Laboratory A	13	12	15	17	12	14	14	14	17
Laboratory B	13	15	15	17	13	14	15	18	20
Laboratory C	13	16	15	17	13	12	12	15	18
Average	13	14	15	17	13	13	14	16	18
Standard Deviation	0.0	2.1	0.0	0.0	0.6	1.2	1.5	2.1	1.5
Range	0	4	0	0	1	2	3	4	3
Average Standard Deviation	0.99								
Average Range	1.89								

Table 4. Plastic Limit Comparison Testing: Current Versus Rolling Device (Bobrowski and Griekspoor, 1992)

Reference	No. 4 Sieve	3/8	1/2	3/4	1	1½	2	3	5	6
	MINIMUM SPECIMEN MASS, KG, BASED ON MAXIMUM PARTICLE SIZE, IN.									
ASTM C 136-84A [1]	0.5	1	2	5	10	15	20	60	300	500
ASTM C 117-84 [2]	0.5	1	...	2.5	...	5	...	...	...	...
ASTM D 422-63 [3]	...	0.5	...	1	2	3	4	5	...	...
ASTM D 1140-54 [4]	0.5	...	...	1.5	2	2.5	...	...	...	...
ASTM D 2487-85 [5]	0.1	0.2	...	1.0	...	8.0	...	60.0	...	...
AASHTO T 27-84 [6]	...	1	2	5	10	15	20	60	300	500
AASHTO 11-85 [7]	0.5	1	...	2.5	...	5	...	...	...	...
AASHTO T88-86 [8]	...	0.5	...	...	2	...	4	5	...	...
Corps of Engineers Appendix V, [9]	0.2	...	1	...	2	...	4	6	...	...
BS 1377: 1975 [10]	...	≈0.5	≈1.0	≈2.0	≈5	15	35	...	...	...
USBR E-6 [11]	0.5	1.0	...	1.5	...	2.5	Larger than 1½ "sufficient amount to make a representative sample"			
Recommended mass	0.1	0.25	...	1.1	...	10	...	70	310	...

Table 5. Comparison of Specimen Size for Gradation Testing (Howard and Horz, 1988)

Table 6.   Recommended Minimum Test Specimen Masses (Howard and Horz, 1988)

Maximum Particle Size	Recommended Minimum Test Specimen Mass	
	kg	lbm
No. 4	0.1	0.2
³/₈ in.	0.25	0.5
³/₄ in.	1.1	2.5
1¹/₂ in.	10	20
3 in.	70	150
5 in.	310	680
8 in.	1300	2800
12 in.	4300	9400

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E. G. Zurflueh, NRC Project Manager

11. ABSTRACT *(200 words or less)*

This document provides technical bases for revision of the U.S. Nuclear Regulatory Commission Regulatory Guide 1.138, Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants, reflecting current and state-of-the-art techniques related to laboratory testing of soils. This report summarizes the processes required in a laboratory testing program. Topics range from storage, selection, and handling of test specimens, to static and dynamic testing methods and equipment. Specific requirements for liquefaction analysis and field site investigations are not addressed in this document but are covered in companion technical bases documents.

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