9.0 LABORATORY TESTING OF SOIL AND ROCK

The purpose of laboratory testing is to provide the basic data with which to classify soils and to quantitatively assess their engineering properties. Laboratory tests should be carefully performed following the proper testing procedures for the soil involved and the information desired. A thorough understanding of the engineering properties of soils is essential not only to the use of current methods in the design of foundations and earth structures, but also as the key to further progress in geotechnical engineering.

Laboratory tests of soils may be grouped broadly into two general classes:

- *Classification tests:* may be performed on either disturbed or "undisturbed" samples.
- *Quantitative tests:* for hydraulic conductivity (permeability), compressibility and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have an unstable soil structure. In these cases, tests may be performed on specimens prepared in the laboratory.

Test results are no better than the samples on which they are performed, or the care used in performing them.

Procedures for most soil tests are given generally by AASHTO and by ASTM. Appropriate test procedures are referenced in Appendix D for the soil tests discussed in the following sections. Techniques for dynamic testing are in a state of development. Consequently, they are changing rapidly and standardized test procedures do not exist. Before undertaking dynamic tests, recent literature should be reviewed and the assistance of an expert in the field sought.

9.1 REQUIREMENTS OF THE LABORATORY

9.1.1 Equipment

In general, a laboratory should be located on a ground floor or basement with a solid floor free of traffic or machine vibrations. The laboratory should be fully equipped with modern soil testing equipment suitable for performing the required classification and property tests. Ideally, separate areas should be designated for dust producing activities, such as sieve analysis and sample processing.

In general, equipment is arranged in areas according to the class or type of testing to provide the most efficient use of personnel and space. If possible, the temperature of the entire laboratory should be controlled. However, if temperature controlled space is limited, this space should be used for consolidation, triaxial and permeability testing. A humid room large enough for storage of undisturbed samples and preparation of test specimens should be available.

Regular inspection and calibration of testing equipment should be performed to maintain accuracy. Malfunctioning equipment should be removed from service until repair, replacement and recalibration have been completed.

9.1.2 Personnel

All laboratory testing should be performed and supervised by personnel who are qualified by training and experience to undertake the assigned testing. They must be thoroughly familiar with the equipment, test procedures and good laboratory techniques in general. Personnel must appreciate the purpose of each test they perform.

Insofar as possible, there should be programs for the indoctrination and training of personnel. New personnel or personnel seeking qualification in additional procedures should receive extensive on-the-job training.

9.1.3 Quality Assurance (Control)

In general, quality assurance (control) should provide control over activities affecting the quality of laboratory testing to an extent consistent with their importance to the results. Quality control should provide for the review and assessment of the following activities as a minimum:

- Handling and storage of soils.
- Specimen preparation.
- Adherence to proper testing procedures.
- Accuracy in measurements.
- Equipment maintenance.
- Review and checking of test data.
- Presentation of test data.

Personnel who are involved in soil testing must constantly be aware of the importance of accuracy in measurements. Inaccurate measurements will produce test results which are useless and misleading. The general philosophy in the laboratory should be that one good test is not only far better than many poor tests, but it is less expensive and less likely to permit a misjudgement in design.

9.2 PLANNING PROJECT-RELATED TEST PROGRAMS

The amount of laboratory testing required for foundation design will vary with each project depending on whether the foundation soils within a given geographic area have been adequately defined by previous explorations, the character of the soils and the requirements of the project. The decision regarding the type and number of laboratory tests to be performed for a project should be based on the complexity of the subsurface conditions, the magnitude and distribution of foundation loads, importance of differential settlement, and local experience.

Laboratory tests should be selected to give the desired and necessary data as economically as possible. Complicated and expensive tests are justified only if the data will reduce costs or risk of a costly failure. In general, relatively few carefully conducted tests on specimens selected to cover the range of soil properties with the results correlated by classification or index tests will give good usable data.

The primary tests of importance to geotechnical engineers, in approximate order of increasing cost, are:

- Visual examination
- Natural moisture content
- Liquid and plastic limit
- Grain-size analysis (mechanical)
- Laboratory vane shear
- Unconfined compression
- Moisture-density or relative density
- California Bearing Ratio
- Permeability
- Direct shear
- Triaxial compression
- Consolidation

9.3 SAMPLE HANDLING

9.3.1 Storage and Preparation

Samples should be identified and logged in when received at the laboratory. Each sample, as well as boxes of samples, should be properly labeled as to name and number of project, boring and sample number, date of sampling, borehole location and depth of recovery. Any field notes relative to deviations from standard drilling and sampling procedures, state of disturbance or unusual characteristics should be recorded for later use.

Samples should be tested as soon as possible after their arrival. Samples which are not scheduled for immediate testing should be resealed, if necessary, to minimize loss of moisture and maintain samples at natural moisture content prior to testing. Although immediate testing is desirable, sample storage may be necessary.

Undisturbed samples of cohesive soils should be sealed with a nonshrinking flexible micro-crystalline wax. The wax should be installed in several layers to minimize shrinkage and cracking. Samples should be stored in a humid room with relative humidity near 100 percent, if possible. The temperature of the humid room should be approximately at mean ground temperature in order to minimize bacterial action in the organics that may be in the soil samples. Tubes should be stored vertically to minimize the formation of air channels by plastic flow of the wax. Samples should not be stored one on top of the other since higher stress increases plastic flow.

Samples which are generally unaffected by changes in moisture content may be stored in glass jars, canvas or heavy bags, cans or bins. Each container should have a label or tag giving the necessary sample data.

The handling and exposure to atmosphere of undisturbed samples should be kept to a minimum. If possible, all preparation of undisturbed test specimens should be done in a humid room. Sealing wax should be cut into strips with a sharp knife or saw during removal to minimize disturbance. Extrusion of a sample should be in the same direction as sampling to minimize disturbance. Excess portions of tubes should be removed prior to extrusion. In order to minimize disturbance due to side friction, the sample tube may be cut into predetermined lengths for tests prior to extruding. However, if the sampled soil is layered, it may be necessary to extrude the sample from the full-length tube and assign tests appropriate to portions of the sample as it is extruded. Any burrs should be removed from the inside of the tube prior to extrusion.

The sample should be extruded using a device that applies a steady force. Hand screw and hydraulic devices are the most commonly used equipment. A record of tube sample should be maintained indicating the sample depth, the sample length, soil description, location of test specimens, results of classification tests and any apparent disturbance or unusual characteristics. Test specimens should not be handled with bare hands nor should moisture be wiped off the specimen. A sheet of wax paper can be used to minimize moisture loss during handling and specimen preparation. The specimen should be supported over its entire length when transporting from one area to another.

Proper care in trimming test specimens helps to minimize disturbance. Trimming of test specimens is normally performed using a piano wire trimmer and soil lathe. Test specimens for consolidation and direct shear testing are often trimmed into collars using specialized lathes or cutting shoes to minimize handling.

Classification tests such as liquid and plastic limits, grain size analysis and specific gravity do not require undisturbed samples. However, care should be given not to mix soils from different layers prior to testing. It is also important that laboratory tests be performed on samples representative of the soil encountered in the field. It is also important that the technician identify and record the nature and description of the test specimen.

9.3.2 Disturbance

Disturbances to soil samples may be classified in five basic types (Hvorslev, 1949), proceeding from relatively slight to more severe:

- Change in stress conditions
- Change in water content and void ratio
- Disturbance of the soil structure
- Chemical Changes
- Mixing and segregation of soil constituents

The influence of disturbance on laboratory test results depends on the type and degree of disturbance and on the nature of the soil and the type of testing.

9.3.2.1 Change in Stress Conditions. The stress changes which occur during boring and sampling can be minimized by the use of proper methods and equipment. However, a total stress reduction to atmospheric pressure cannot be avoided when the sample is removed from the tube or liner and during preparation of the test specimens. Hvorslev (1949) presents a discussion of the consequences of such stress reduction for various soil types.

9.3.2.2 Change in Water Content and Void Ratio. In a non-gaseous, fully saturated soil, a change in void ratio (volume) is accompanied by a corresponding change in water content. However, the void ratio of gaseous soils can be changed without a change in water content and the water content of partially saturated soils with interconnecting voids may be changed with only minor changes in void ratio.

Changes in volume may occur before, during and after sampling. Volume changes resulting from expansion and displacement of soil during drilling and sampling usually affect only the upper part of the sample taken from a borehole. Volume changes associated with extrusion from sampling tubes can be minimized by cutting the tube into appropriate lengths for testing prior to extruding the soil.

9.3.2.3 Disturbance of the Soil Structure. Disturbance of soil structure can occur before, during and after drilling and sampling. For sampling in boreholes, the disturbance before sampling is usually limited to the upper part of the sample. By the use of proper equipment and methods, disturbance can be minimized, especially for the central portion of the sample. However, the lower part may be disturbed when separating the sample from the subsoil. Disturbance after sampling can be minimized by proper care in sealing, shipment and handling of the sample.

9.3.2.4 Chemical Changes. Disturbance associated with chemical changes is usually caused by infiltration of wash water or drilling fluid in the sample, oxidation after sampling and during specimen preparation, contact with the sample containers and electrical charge. The greatest danger of chemical changes is associated with samples stored in untreated steel containers for long periods of time. Containers should be coated with lacquer and the sealing caps should be an inert material or of the same type as the container.

9.3.2.5 Mixing and Segregation of Soil Constituents. Mixing and segregation of constituents is generally associated with sampling operations and can be minimized by using proper drilling and sampling procedures. When only soil layers in close proximity have been mixed, the sample as a whole may be representative of the average condition and acceptable for identification and determination of the suitability for construction purposes.

9.3.3 Undisturbed Soil Samples

Due to the reduction of total stresses during sampling and specimen preparation, a truly undisturbed sample cannot be obtained.

However, a sample may be suitable for laboratory testing and for practical purposes considered undisturbed if the following requirements are met:

- No disturbance of the soil structure
- No change in water content or void ratio
- No change in constituents or chemical composition

Because it is very difficult to evaluate whether these requirements are satisfied, Hvorslev (1949) proposed that the strict requirements for undisturbed sampling be replaced by the following practical or modified requirements.

- The specific recovery ratio shall not be greater than 1.00 nor smaller than (1−2Ci), where Ci is the inside clearance ratio of the cutting edge. If entrance of excess soil is prevented, it is generally sufficient that the total recovery ratio (ratio of sample length to push length) be equal to or slightly smaller than 1.00.
- On the surface or in sliced sections of the sample, there must be no visible distortions, planes of failure, or pitting attributed to the sampling operation or handling of the samples.
- The net length and weight of sample and the results of other control tests must not change during shipment, storage and handling of the sample.

9.4 LABORATORY ASPECTS OF SOIL CLASSIFICATION

The Unified Soil Classification System (USCS) (U.S. Army, 1953) discussed in Appendix E, is based on the identification of soils according to the type and predominance of the constituents considering the following:

- Grain size
- Gradation (shape of grain size distribution curve)
- Plasticity and compressibility

The system divides soils into three major divisions:

- Coarse grained (more than 50 percent retained on the No. 200 sieve)
- Fine grained (more than 50 percent passing the No. 200 sieve)
- Highly organic (peaty) soils

Coarse grained soils are classified as to their particle size and shape of the grain size distribution curve. Fine grained soils are classified as to their position on the plasticity chart.

9.4.1 Grain Size Analysis

The grain size distribution of a coarse soil can be very useful for both classification and evaluation of specification criteria. To some extent, the grain size curve for sands can be related to engineering behavior such as soil permeability, frost susceptibility, angle of internal friction, bearing capacity and liquefaction potential. The behavior of fine grained soils (silty clays and clays) are more a function of the degree of plasticity, type of mineral and geologic history.

The grain size distribution of a soil is expressed as a plot of percent finer by weight versus diameter in millimeters. The grain size distribution of a coarse-grained soil is determined by sieve analysis while a hydrometer test is used for fine-grained soils. As a general note, if nearly all (approximately 80 percent) of the particles of a soil are greater than a No. 200 sieve (openings of 0.074 mm), the sieve analysis is used. For soils which are nearly all finer than a No. 200 sieve, the hydrometer test is used. Soils which have portions of their particles both larger and smaller than a No. 200 sieve require a combined analysis.

A sieve analysis consists of passing a sample through a set of standard sieves and weighing the amount retained on each sieve. The recommended test procedure for grain size analysis is included in Appendix C. The results are plotted on a grain size distribution curve in the form of percent fines by weight versus particle size to a log scale. The shape of the grain size curve is indicative of the grading. A "uniformly" graded soil has a grain size curve that is nearly vertical and a "well-graded" soil has a more flat curve that extends across several log cycles of particle size.

The uniformity of a soil may be represented by the uniformity coefficient, Cu, defined as D60/D10, where D60 is the particle size for which 60 percent of the specimen weight is finer and D10 is the particle size for which 10 percent of the specimen weight is finer. The coefficient of curvature (Cc), defined as $(D30)^2/(D60 \times D10)$, is also used to describe particle size characteristics. In accordance with the Unified Soil Classification System:

Soil	*Range*
Poorly (uniform) graded	Cu < 4
Well-graded gravel	Cu > 4, 1 < Cc < 3
Well-graded sand	Cu > 6, 1 < Cc < 3

The hydrometer (sedimentation) analysis is based on Stokes law, which relates the velocity at which a spherical particle falls through a fluid medium to the diameter and specific gravity of the particle and the viscosity of the fluid. The particle size is obtained by measuring the density of the soil-water suspension using a hydrometer. The hydrometer test is generally performed on soil passing the No. 10 sieve.

For soils with both coarse and fine constituents, a combined analysis should be performed. The sieve analysis is performed on soil retained on the No. 200 sieve and the hydrometer analysis is performed on soil passing the No. 10 sieve.

9.4.2 Liquid and Plastic Limits

Liquid and plastic (Atterberg) limits are empirical boundaries which separate the states of fine grained soil. For example, a soil at a very high water content is in a liquid state. As the water content decreases, the soil passes the liquid limit and changes to a plastic state. As the water content decreases further, the soil passes the plastic limit and changes to a semi-solid state.

The liquid limit (LL) is defined as the water content at which a standard groove closes after 25 blows in a liquid limit device. The plastic limit (PL) is the water content at which the soil begins to crumble when rolled into 3.2 mm (0.125 in.) diameter threads. The thread should break into numerous pieces between 3.2 mm (0.125 in.) and 9.5 mm (0.375 in.) long. Refer to Appendix C for recommended test procedures for liquid and plastic limits. The purpose of the limits is to aid in the classification of fine-grained soils (silts and clays) to evaluate the uniformity of a deposit and to provide some general correlations with engineering properties.

In accordance with the Unified Soil Classification System, a fine-grained soil is classified as to its position on the plasticity chart, Figure E-4. The uniformity of a fine grained soil deposit can be evaluated by plotting the test results of natural water content and Atterberg limits versus depth or elevation.

The liquid and plastic limits are not well correlated with engineering properties that are a function of soil structure or its undisturbed state. However, some general empirical correlations for fine-grained soils have been developed based on index properties, natural water content and Atterberg limits.

9.4.2.1 *Correlation of Various Properties*

- *Rebound or Swelling:* According to U.S. Navy, 1971.[1]
- *Consolidation Stress versus Liquidity Index:* According to U.S. Navy, 1971.[1]
- *Coefficient of Consolidation versus Liquid Limit:* According to U.S. Navy, 1971.[1]
- *Angle of Shearing Resistance versus Plasticity Index (PI):* According to U.S. Navy, 1971.[1]

9.4.2.2 Other Controls Over Atterberg Limits. Experience has shown that the liquid and plastic limits of some fine-grained soils are sensitive to the pore fluid (salt concentration for marine illitic clays) and the pretreatment (air or oven dried or natural water content) before running the tests. It has been shown that soils sensitive to oven drying generally contain one of the following:

- organic matter
- high montmorillonite content
- hydrated halloysite
- hydrous oxides

It is recommended that limits be determined on fine grained soil starting with the soil at or near the natural water content (Lambe, 1951). Soil that has been dried (air or oven) should be thoroughly mixed with water and allowed to equilibrate for several days before testing. Soils with organic content should not be dried prior to testing.

9.4.3 Specific Gravity

The specific gravity of a soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of four degrees Celsius. The specific gravity of a soil is used in computations for most laboratory tests. In addition, the specific gravity is often used to relate the weight of a soil to its volume of solids for use in phase relationships, such as unit weight, void ratio, moisture content, and degree of saturation.

The specific gravity is of only limited value for identification or classification of most soils because the specific gravities of most soils fall within a narrow range.

9.5 SHEAR STRENGTH

The shear strength of a soil is determined by the resistance to sliding between particles that are trying to move laterally past each other. The laboratory tests most commonly used to determine shear strength are direct shear, unconfined compression and triaxial compression.

Shear resistance of soil is due to both cohesion and friction. The shear strength of a soil is expressed by the Mohr-Coulomb failure criteria, shown graphically in Figure 9-1:

$$s = c + \bar{\sigma} \tan \phi$$

where s = shear strength
c = cohesion
$\bar{\sigma}$ = effective stress normal to the shear plane
ϕ = angle of internal friction of the soil.

Coarse-grained soils generally exhibit little or no cohesion (i.e., cohesionless) and therefore the shear strength depends primarily on the frictional resistance. An estimate of the shear strength of the cohesionless soil *in situ* can be difficult to determine in the laboratory because the strength can vary with density or critical void ratio, composition of the soil (particle size, gradation and angularity of soil particles), nonhomogeneity of the deposit and the loading conditions. Therefore, the soil should be tested in the laboratory under conditions which simulate the most critical condition in the field.

The shear strength of fine grained (cohesive) soils is a complex subject. In terms of total stress, the shear strength may be expressed as

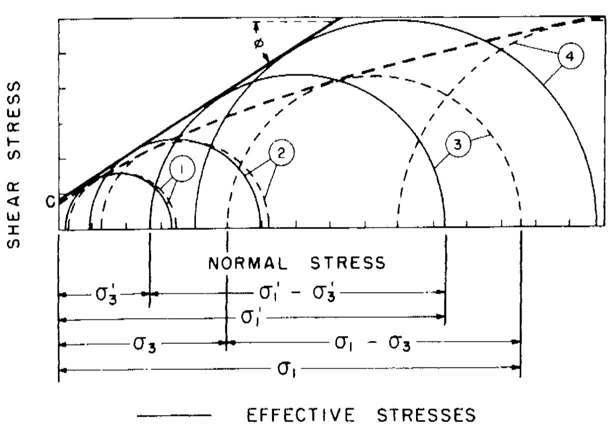


Figure 9-1. Mohr-Coulomb failure criteria. (Haley & Aldrich, Inc.)

$$s = c + (\sigma - u_f) \tan \phi$$
where u_f = pore pressure at failure

For some foundation problems, the pore pressure at failure is unknown or cannot be readily evaluated. For such problems, it is appropriate to use undrained strength (S_u, "total-stress" strength parameter) in analyses to determine the factor of safety or lateral loading, rather than "effective-stress" strength parameters, $\bar{c}$ and $\bar{\phi}$.

Experience has shown that undrained strength is independent of changes in the total stress, unless a change in water content occurs. Because undrained strength is determined by the initial conditions prior to loading, it is not necessary to determine the effective stresses that would exist at failure. The undrained shear strength of cohesive soils, as determined by laboratory tests, can be difficult to determine. Estimating strength from the results of laboratory tests ideally calls for performance of tests that will duplicate *in situ* conditions. It is very difficult to achieve this situation for many reasons; such as, effects of sample disturbance, lack of knowledge of the *in situ* stresses and equipment and testing limitations that impose non-uniform stresses or the wrong stress system.

The appropriate strength parameters for given field conditions are discussed in Section 9.5.4.

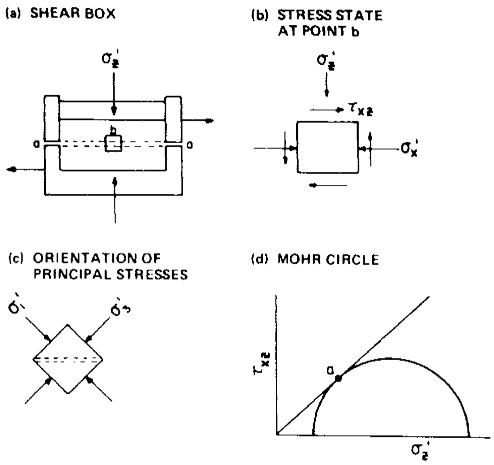
9.5.1 Loading Devices

Loading devices used to test laboratory specimens of soil can be classified as either strain-controlled or stress-controlled. Strain-controlled loading devices apply strain to the specimen at a predetermined, controlled, constant rate of strain. A stress-controlled loading device applies a constant load or stress to the specimen, generally in increments and at predetermined time intervals, by using dead weights, applied either directly or by a lever system or by using air or hydraulic pressure controlled by very precise pressure regulators.

Measurement of the load applied to a laboratory soil specimen is usually accomplished using a proving ring or an electronic load cell. Load cells and proving rings should be calibrated periodically to maintain accurate measurements.

9.5.2 Direct Shear

In a direct shear test, the soil is placed in a split shear box and stressed to failure by moving one part of the container relative to the other (Figure 9-2). The specimen is subjected to a normal force and a horizontal shear force. The normal force is kept constant throughout the test and the shear force is increased



Simple shear test.

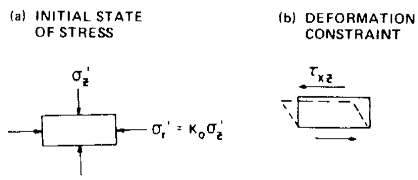


Figure 9-2. Laboratory shear tests. Above, is shown the direct shear test in which shear failure is induced along a specified plane, and its relationship to a Mohr concept for cohesionless material. Below is illustrated the simple shear test that is performed in a triaxial test cell on undisturbed samples (From Wu and Sangrey, 1978).

usually at a constant rate of strain to cause the specimen to shear along a predetermined horizontal plane.

The use of the direct shear test to determine the shear strength of soil has been questioned. In the direct shear test, only the normal and shear stresses on a single, predetermined plane are known. Hence, it is not possible to draw the Mohr Circle giving the state of stress. However, if it is assumed that the horizontal plane is equivalent to the failure plane for the soil, then the friction angle can be calculated from the results of a series of tests performed at various normal stresses. Lambe and Whitman (1969) report that comparisons between the value of ϕ, from triaxial and direct shear tests, after averaging out experimental errors in the determination of the values, yield results that differ generally by no more than two degrees.

The direct shear test offers the easiest way to measure the friction angle of a sand or other dry soil. It is not useful for testing soils containing water unless they are free draining and have a very high permeability, because it is difficult to control the drainage and thus volume changes during testing. For this reason, the direct shear tests should be used with caution in determining the undrained shear strength of cohesive soils.

9.5.3 Unconfined Compression Test

The unconfined compression test measures the compressive strength of a cylinder of cohesive soil which has no lateral confinement (unconfined). The undrained shear strength is normally taken as approximately equal to one-half the compressive strength.

The test is generally performed on an undisturbed specimen of cohesive soil at its natural water content. Cohesionless soils, such as sands and non-plastic silts and fissured or layer materials, should not be tested unconfined because the shear strength of these types of soils is a function of the *in situ* confining stress.

Because no lateral confinement is used in the unconfined compression test, it has several features:

- It is the simplest, quickest and least expensive laboratory test to measure the undrained shear strength of a cohesive soil.
- Unconfined compression tests may be performed in the field using portable equipment for rapid measurement of undrained shear strength.

Refer to Appendix D for recommended test procedures for this test.

9.5.4 Triaxial Compression Test

The triaxial test is the most common and versatile test available to determine the stress-strain properties of soil. In the triaxial compression test, a cylindrical specimen is sealed in a rubber membrane and placed in a cell and subjected to fluid pressure. A typical triaxial cell is shown in Figure 9-3. A load is applied axially to the specimen increasing the axial stress until the specimen fails. Under these conditions, the axial stress is the major principal stress, σ_1 , and the intermediate and minor principal stresses, σ_2 and σ_3 respectively, are equal to the cell pressure. The increment of axial stress, $\sigma_1 - \sigma_3$, is referred to as the deviator stress or principal stress difference.

Drainage of water from the specimen is controlled by connections to the bottom cap as shown in Figure 9-3. Alternatively, pore water pressures may be measured if no drainage is allowed. Triaxial tests are generally classified as to the condition of drainage during application of the cell pressure and loading, respectively, as follows:

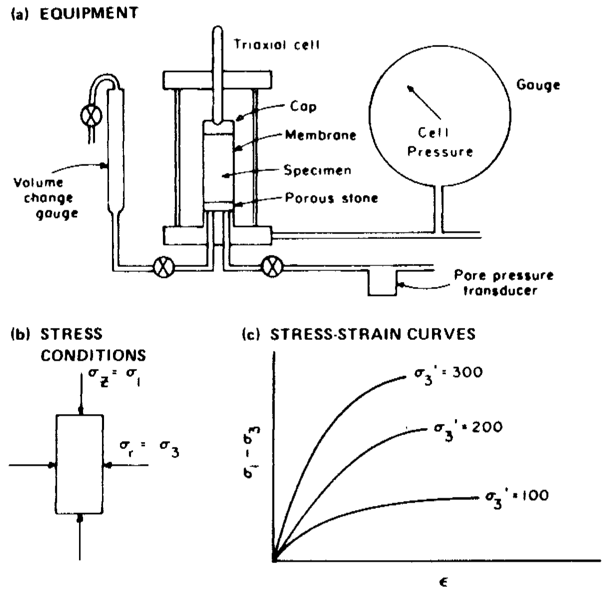


Figure 9-3. The triaxial test employed on undisturbed and remolded soil samples. A variety of *in situ* stresses and stresses related to expected structural loading conditions can be modeled into the test, showing that the soil shear strength parameters vary dramatically under different conditions of pore pressure accumulation and stress and strain levels as well as strain rates. As shown in this illustration, the deviator stress (sigma one minus three) varies considerably with the cell pressure (sigma three) utilized in the test (From Wu and Sangrey, 1978).

- *Unconsolidated-Undrained (UU).* No drainage is allowed during application of the cell pressure or confining stress and no drainage is allowed during application of the deviator stress.
- *Consolidated-Undrained (CU).* Drainage is allowed during application of the confining stress so that the specimen is fully consolidated under this stress. No drainage is permitted during application of the deviator stress.
- *Consolidated-Drained.* Drainage is permitted both during application of the confining stress and the deviator stress, such that the specimen is fully consolidated under the confining stress and no excess pore pressures are developed during testing.

9.5.4.1 Unconsolidated-Undrained (UU) Test. This test is generally performed on undisturbed saturated samples of fine grained soils (clay, silt and peat) to measure the *in situ* undrained shear strength ($\phi = 0$ analysis). For soils which exhibit peak stress-strain characteristics, the failure stress is taken as the maximum deviator stress ($\sigma_1 - \sigma_3$) measured during the test. For soils which exhibit an increasing deviator stress with strain, the failure stress is generally taken as the deviator stress at a strain equal to 20 percent. The undrained shear strength, S_u, is taken as one-half the deviator stress or $S_u = \frac{\sigma_1 - \sigma_3}{2}$.

The *in situ* undrained shear strength is applicable to conditions in which construction occurs rapidly enough so that no drainage and hence, no dissipation of excess pore pressures occur during construction. Examples of typical situations in which the *in situ* undrained shear strength would govern stability include construction of embankments on clay deposits or rapid loading of footings on clay.

Unconsolidated-undrained tests are also performed on samples of partially saturated cohesive soils. The principal application of tests on partially saturated samples is to earth-fill materials which are compacted under specified conditions of water content and density. It also applies to undisturbed samples of partially saturated (i.e. residual soils) and to samples recovered from existing fills. However, because the tests are performed on partially saturated soil, the deviator stress at failure will increase with continuing pressure. Bishop and Henkel (1962) indicate that the failure envelope expressed in items of total stress is non-linear and values of c and ϕ can be reported only for specific ranges of continuing pressures. If pore pressures are measured during the test, the failure envelope can be expressed in terms of effective stress.

9.5.4.2 Consolidated-Undrained (CU) Test. This test is performed on undisturbed samples of cohesive soil, on reconstituted specimens of cohesionless soil and, in some instances, on undisturbed samples of cohesionless soils which have developed some apparent cohesion resulting from partial drainage.

Generally, the specimen is allowed to consolidate under a confining stress of known magnitude and is then failed under undrained conditions by applying an axial load. The volume change that occurs during consolidation should be measured. The results of CU tests, in terms of total stress or undrained shear strength, must be applied with caution because of uncertainties in the effects of stress history and stress system (isotropic consolidation) on the magnitude of strength increase with consolidation.

If the pore pressure is measured during the test, the results can be expressed in terms of effective stress, $\bar{c}$ and $\bar{\phi}$.

The principal application of results of CU tests on cohesive soils is to the situation where additional load is rapidly applied to soil that has been consolidated under previous loading (shear stresses). The principal application to cohesionless soils is to evaluate the stress-strain properties as a function of effective confining stress.

9.5.4.3 Consolidated-Drained (CD) Tests. Consolidated drained tests are performed on all types of soil samples, including undisturbed, compacted and reconstituted samples.

In a standard test, the specimen is allowed to consolidate under a predetermined confining stress and the specimen is then sheared by increasing the axial load at a sufficiently slow rate to prevent development of excess pore pressure. Since the excess pore pressure is zero, the applied stresses are equal to the effective stresses and the strength parameters, $\bar{c}$ and $\bar{\phi}$, are obtained directly from the stresses at failure. The volume changes that occur during consolidation and shear should be measured.

The principal application of the results of CD tests on cohesive soils is for the case where either construction will occur at a sufficiently slow rate that no excess pore pressures will develop or sufficient time will have elapsed that all excess pore pressures will have dissipated.

The principal application to cohesionless soils is to determine the effective friction angle.

9.5.4.4 Young's Modulus. The triaxial test may be used to determine Young's modulus for a soil. The standard triaxial test, with increasing axial stress and constant continuing stress, provides a direct measure of Young's modulus. The secant modulus (drawn from zero deviator stress to 1/2 peak deviator stress on a stress-strain curve) is the modulus value generally quoted for soil.

9.5.5 Laboratory Vane Shear

The laboratory vane shear test uses a system of vanes or blades attached to a shaft that is inserted into the exposed ends of undisturbed tube samples of cohesive soil. The torque required to cause failure of the soil is related to the undrained shear strength.

It is assumed that the soil fails along the edges of the vane. Because the vane imposes a stress system during shear that is unlike any mode of failure encountered in practice, the vane test should be treated as a strength index test. That is, the vane strength must be correlated with the results of other undrained strength tests and used as an index property.

There are numerous devices on the market to perform laboratory vane tests. The most common and inexpensive types are hand operated and can be inserted into undisturbed samples of cohesive soils. The use of the vane should be restricted to homogeneous clays without shells, stones, fibers, sand pockets, and other anomalies.

9.6 CONSOLIDATION

Consolidation may be defined as volume change at "constant" load caused by transfer of total stress from excess pore pressure to effective stress as drainage occurs. When load is applied to a saturated soil mass, the load is carried partly by the mineral skeleton and partly by the pore fluid. With time, the water will be squeezed out of the soil and the soil mass will consolidate.

The permeability or rate at which the water can be squeezed out and thus the rate of consolidation, varies with the soil type. Cohesionless soils are generally quite permeable and the rate of consolidation is very rapid and generally not of a concern to foundation engineers. The permeability of cohesive soils such as clay is quite low and the rate of consolidation is quite slow. The remainder of this discussion will deal with consolidation of saturated cohesive soil, specifically clay.

When a load is applied to a saturated deposit of clay, there will be three types of settlement:

- *Initial settlement:* associated with undrained shear deformation of clay.
- *Consolidation settlement:* volume changes associated with the dissipation of excess pore pressure.
- *Secondary Compression* (consolidation): volume changes associated with essentially constant effective stress, after complete dissipation of excess pore pressure.

The relative importance of the three types of settlement depends on such factors as:

- Type and stress history of the soil, i.e., normally consolidated or overconsolidated
- Magnitude of loading
- Rate of loading
- Size of the loaded area in relation to the thickness of the clay deposit

The initial settlement of footings on heavily overconsolidated clay is often a significant portion of the total settlement.

The initial settlement of a clay deposit that is subjected to a load area very large in relation to the clay thickness (one-dimensional consolidation) will be very minor and the consolidation settlement will be of primary importance. In sand drain installations and for one-dimensional compression of organic soils, secondary compression is often of practical significance.

In general the magnitude of consolidation settlement will be of greatest concern for most cases. The laboratory test most commonly used to evaluate consolidation settlement is the oedometer test or one-dimensional consolidation test.

The stress-strain or compressibility characteristics of clays are highly dependent upon their stress history. The stress history of a clay deposit refers to the existing stresses and the degree of overconsolidation. If the vertical consolidation stress $\bar{\sigma}_{vc}$ acting on the clay is the greatest that has ever existed, the clay is called normally consolidated. If the existing stress is less than the maximum value that has ever existed, referred to as the maximum previous stress, $\bar{\sigma}_{vm}$, the clay is called overconsolidated.

If the clay is stressed within the limits of the maximum previous stress, the strain (settlement) will be a function of the recompression ratio (RR) determined from laboratory consolidation tests. If the applied stress exceeds the maximum previous stress, the strain will be proportional to the virgin compression ratio (CR).

9.6.1 Consolidation Tests

In an oedometer (consolidation) test, the soil is placed in an oedometer ring and stress is applied to the soil specimen along the vertical axis. Because strain in the horizontal direction is prevented, the vertical strain is equal to the volumetric strain.

The test is generally performed on a specimen of clay that is 19 or 25 mm (0.75 or 1.0 in.) in thickness and 64 mm (2.50 in.) in diameter. The 64 mm ring is the most common size ring because the specimen can be trimmed from a 76 mm (3-in.) thinwall tube sample.

The load applied to the specimen is generally doubled (Load Increment Ratio equal to unity) and readings of vertical deformation versus time are obtained during each load increment. The information that can be obtained from the test include:

- *Compressibility of the soil* for one-dimensional loading as defined by the compression curve, (vertical strain, ϵ_v, or void ratio, e, plotted versus log consolidation stress, $\bar{\sigma}_{vc}$).
- *Maximum previous stress,* $\bar{\sigma}_{vm}$, as determined by empirical procedures from the compression curve.
- *Coefficient of consolidation,* c_v, using curve fitting techniques, based on the Terzaghi theory of consolidation, applied to the deformation versus time curves.
- *Rate of secondary compression* as defined by the slope of the deformation versus log time plot after primary consolidation is completed.

9.6.2 Presentation of Consolidation Test Data

There are two widely used curve fitting methods that are applied to the deformation versus time curves, the log time and the square root of time method. The square root method places emphasis on the early stages of consolidation whereas the log time method emphasizes the latter stages of consolidation.

The results of consolidation tests are generally presented as a graph of void ratio, e, or vertical strain, ϵ_v , versus consolidation stress, $\bar{\sigma}_{vc}$, plotted to a log scale. This type of plot is used because it exhibits certain characteristic shapes and behavior that have proved useful.

When void ratio is used, compressibility parameters are defined as follows:

C_c = virgin compression index = slope of compression curve in virgin region.
C_r = recompression index = average slope of unloading-reloading cycle.
C_s = swelling index = slope of swelling (rebound curve).

When test results are plotted in terms of strain instead of void ratio, the corresponding parameters when strain is used are:

$CR = C_c/(1 + e_o)$ = virgin compression ratio
$RR = C_r/(1 + e_o)$ = recompression ratio
$SR = C_s/(1 + e_o)$ = swelling ratio

and strain $= \Delta e/(1 + e_o)$

The void ratio versus log stress plot is more commonly used than the strain versus log stress plot. However, the latter has several advantages (Ladd, 1971):

1. Strains are easier to compute than void ratios, which require a knowledge of specific gravity and weight of soil solids.
2. Settlements are directly proportional to strains, whereas, use of Δe data also requires a knowledge of $(1 + e_o)$. Thus, the latter introduces two variables, Δe and $(1 + e_o)$.
3. It is easier to standardize strain plots than void ratio plots.
4. The strain curve can be plotted as the consolidation test is in progress. Any major discrepancies in the test could immediately be noted and corrected, if possible.