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CHAPTER 7.0

LABORATORY TESTING FOR SOILS

7.1 GENERAL

Laboratory testing of soils is a fundamental element of geotechnical engineering. The complexity of testing required for a particular project may range from a simple moisture content determination to specialized strength and stiffness testing. Since testing can be expensive and time consuming, the geotechnical engineer should recognize the project's issues ahead of time so as to optimize the testing program, particularly strength and consolidation testing.

Before describing the various soil test methods, soil behavioral under load will be examined and common soil mechanics terms introduced. The following discussion includes only basic concepts of soil behavior. However, the engineer must grasp these concepts in order to select the appropriate tests to model the in-situ conditions. The terms and symbols shown will be used in all the remaining modules of the course. Basic soil mechanics textbooks should be consulted for further explanation of these and other terms.

7.1.1 Weight-Volume Concepts

A sample of soil is usually composed of soil grains, water and air. The soil grains are irregularly shaped solids which are in contact with other adjacent soil grains. The weight and volume of a soil sample depends on the specific gravity of the soil grains (solids), the size of the space between soil grains (voids and pores) and the amount of void space filled with water. Common terms associated with weight-volume relationships are shown in Table 7-1. Of particular note is the void ratio (e) which is a general indicator of the relative strength and compressibility of a soil sample, i.e., low void ratios generally indicate strong soils of low compressibility, while high void ratios are often indicative of weak & highly compressible soils. Selected weight-volume (unit weight) relations are presented in Table 7-2.

TABLE 7-1.
TERMS IN WEIGHT-VOLUME RELATIONS (After Cheney and Chassie, 1993)

Property	Symbol	Units[1]	How obtained (AASHTO/ASTM)	Direct Applications
Moisture Content	w	D	By measurement (T 265/ D 4959)	Classification and in weight-volume relations
Specific Gravity	G_s	D	By measurement (T 100/D 854)	Volume computations
Unit weight	(	FL^{-3}	By measurement or from weight-volume relations	Classification and for pressure computations
Porosity	n	D	From weight-volume relations	Defines relative volume of solids to total volume of soil
Void Ratio	e	D	From weight-volume relations	Defines relative volume of voids to volume of solids

[1] F = Force or weight; L = Length; D = Dimensionless. Although by definition, moisture content is a dimensionless fraction (ratio of weight of water to weight of solids), it is commonly reported in percent by multiplying the fraction by 100.

TABLE 7-2.

UNIT WEIGHT-VOLUME RELATIONSHIPS

Case	*Relationship*	*Applicable Geomaterials*
Soil Identities:	1. $G_s\, w = S\, e$ 2. Total Unit Weight: $$\gamma_T = \frac{(1+w)}{(1+e)} G_s \gamma_w$$	All types of soils & rocks
Limiting Unit Weight	Solid phase only: w = e = 0: $\gamma_{rock} = G_s\, \gamma_w$	Maximum expected value for solid silica is 27 kN/m^3
Dry Unit Weight	For w = 0 (all air in void space): $\gamma_d = G_s\, \gamma_w/(1+e)$	Use for clean sands and dry soils above groundwater table
Moist Unit Weight (Total Unit Weight)	Variable amounts of air & water: $\gamma_t = G_s\, \gamma_w\, (1+w)/(1+e)$ with $e = G_s\, w/S$	Partially-saturated soils above water table; depends on degree of saturation (S, as decimal).
Saturated Unit Weight	Set S = 1 (all voids with water): $\gamma_{sat} = \gamma_w\, (G_s+e)/(1+e)$	All soils below water table; Saturated clays & silts above water table with full capillarity.
Hierarchy:	γ_d # γ_t # γ_{sat} $< \gamma_{rock}$	Check on relative values

Note: $\gamma_w = 9.8$ kN/m^3 (62.4 pcf) for fresh water

7.1.2 Load-Deformation Process in Soils

When a load is applied to a soil sample, the deformation which occurs will depend on the grain-to-grain contact (intergranular) forces and the amount of water in the voids. If no porewater exists, the sample deformation will be due to sliding between soil grains and deformation of the individual soil grains. The rearrangement of soil grains due to sliding accounts for most of the deformation. Adequate deformation is required to increase the grain contact areas to take the applied load. As the amount of pore water in the void increases, the pressure it exerts on soil grains will increase and reduce the intergranular contact forces. In fact, tiny clay particles may be forced completely apart by water in the pore space.

Deformation of a saturated soil is more complicated than that of dry soil as water molecules, which fill the voids, must be squeezed out of the sample before readjustment of soil grains can occur. The more permeable a soil is, the faster the deformation under load will occur. However, when the load on a saturated soil is quickly increased, the increase is carried entirely by the pore water until drainage begins. Then more and more load is gradually transferred to the soil grains until the excess pore pressure has dissipated and the soil grains readjust to a denser configuration. This process is called *consolidation* and results in a higher unit weight and a decreased void ratio.

7.1.3 Principle of Effective Stress

The consolidation process demonstrates the very important principle of effective stress, which will be used in all the remaining modules of this course. Under an applied load, the total stress in a saturated soil sample is composed of the intergranular stress and porewater pressure (neutral stress). As the porewater has zero shear strength and is considered incompressible, only the intergranular stress is effective in resisting shear or limiting compression of the soil sample. Therefore, the intergranular contact stress is called the *effective stress*. Simply stated, this fundamental principle states that *the effective stress (σ') on any plane within a soil mass is the net difference between the total stress (σ) and porewater pressure (*u*).*

When pore water drains from soil during consolidation, the area of contact between soil grains increases, which increases the level of effective stress and therefore the soil's shear strength. In practice, staged construction of embankments is used to permit increase of effective stress in the foundation soil before subsequent fill load is added. In such operations the effective stress increase is frequently monitored with piezometers to ensure the next stage of embankment can be safely placed.

Soil deposits below the water table will be considered saturated and the ambient pore pressure at any depth may be computed by multiplying the unit weight of water (γ_w) by the height of water above that depth. For partially saturated soil, the effective stress will be influenced by the soil structure and degree of saturation (Bishop, *et. al.*, 1960). In many cases involving silts & clays, the continuous void spaces that exist in the soil behave as capillary tubes of variable cross-section. Due to capillarity, water may rise above the static groundwater table (*phreatic surface*) as a negative porewater pressure and the soils may be nearly or fully saturated.

7.1.4 Overburden Stress

The purpose of laboratory testing is to simulate in-situ soil loading under controlled boundary conditions. Soils existing at a depth below the ground surface are affected by the weight of the soil above that depth. The influence of this weight, known generally as the *overburden stress*, causes a state of stress to exist which is unique at that depth for that soil. When a soil sample is removed from the ground, that state of stress is relieved as all confinement of the sample has been removed. In testing, it is important to reestablish the in-situ stress conditions and to study changes in soil properties when additional stresses representing the expected design loading are applied. In this regard, the effective stress (grain-to-grain contact) is the controlling factor in shear, state of stress, consolidation, stiffness, and flow. Therefore, the designer should try to re-establish the effective stress condition during most testing.

The test confining stresses are estimated from the total, hydrostatic, and effective overburden stresses. The engineer's first task is determining these stress and pressure variations with depth. This involves determining the total unit weights (density) for each soil layer in the subsurface profile, and determining the depth of the water table. Unit weight may be accurately determined from density tests on undisturbed samples or estimated from in-situ test measurements. The water table is routinely recorded on the boring logs, or can be measured in open standpipes, piezometers, and dissipation tests during CPTs and DMTs .
The total vertical (overburden) stress (σ_{vo}) at any depth (z) may be found as the accumulation of total unit weights (γ_t) of the soil strata above that depth:

$$\sigma_{vo} = \int \gamma_t \, dz \approx \Sigma \gamma_t \, z \qquad (7\text{-}1)$$

For soils above the phreatic surface, the applicable value of total unit weight may be dry, moist, or saturated depending upon the soil type and degree of capillarity (see Table 7-2). For soil elements situated below the groundwater table, the saturated unit weight is normally adopted.

The hydrostatic pressure depends upon the degree of saturation and level of the phreatic surface and is determined as follow:

Soil elements above water table: $u_o = 0$ (Completely dry) (7-2a)

$u_o = \gamma_w(z - z_w)$ (Full capillarity) (7-2b)

Soil elements below water table: $u_o = \gamma_w(z - z_w)$ (7-2c)

where z = depth of soil element, z_w = depth to groundwater table. Another case involves partial saturation with intermediate values between (7-2a and 7-2b) which literally vary daily with the weather and can be obtained via tensiometer measurements in the field. Usual practical calculations adopt (7-2a) for many soils, yet the negative capillary values from (7-2b) often apply to saturated clay & silt deposits.

The effective vertical stress is obtained as the difference between (7-1) and (7-2):

$$\sigma_{vo}' = \sigma_{vo} - u_o \qquad (7\text{-}3)$$

A plot of effective overburden profile with depth is called a σ_v diagram and is extensively used in all aspects of foundation testing and analysis (see Holtz & Kovacs, 1981; Lambe & Whitman, 1979).

7.1.5 Selection and Assignment of Tests

Certain considerations regarding laboratory testing, such as when, how much, and what type, can only be decided by an experienced geotechnical engineer. The following minimal criteria should be considered while determining the scope of the laboratory testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc.

The selection of tests should be considered preliminary until the geotechnical engineer is satisfied that the test results are sufficient to develop reliable soil profiles and provide the soil parameters needed for design.

Following this subsection are brief discussions of frequently used soil properties and tests. These discussions assume that the reader will have access to the latest volumes of AASHTO and ASTM standards containing details of test procedures and will study them in connection with this presentation. Table 7-3 presents a summary list of AASHTO and ASTM tests frequently used for laboratory testing of soils.

TABLE 7-3.

AASHTO AND ASTM STANDARDS FOR FREQUENTLY-USED LABORATORY TESTING OF SOILS

Test Category	Name of Test	Test Designation	
		AASHTO	ASTM
Visual Identification	Practice for Description and Identification of Soils (Visual-Manual Procedure)	-	D 2488
	Practice for Description of Frozen Soils (Visual-Manual Procedure)	-	D 4083
Index Properties	Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method	T 265	D 4959
	Test Method for Specific Gravity of Soils	T 100	D 854
	Method for Particle-Size Analysis of Soils	T 88	D 422
	Test Method for Amount of Material in Soils Finer than the No. 200 (75-: m) Sieve		D 1140
	Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	T 89 T 90	D 4318
	Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (600 kN-m/m^3)	T 99	D 698
	Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (2,700 kN-m/m^3)	T 180	D 1557
Corrosivity	Test Method for pH of Peat Materials	-	D 2976
	Test Method for pH of Soils	-	D 4972
	Test Method for pH of Soil for Use in Corrosion Testing	T 289	G 51
	Test Method for Sulfate Content	T 290	D 4230
	Test Method For Resistivity	T 288	D 1125 G 57
	Test Method for Chloride Content	T 291	D 512
	Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	T 194	D 2974
	Test Method for Classification of Soils for Engineering Purposes	M 145	D 2487 D 3282

TABLE 7-3 (Continued)

AASHTO AND ASTM STANDARDS FOR FREQUENTLY USED LABORATORY TESTING OF SOILS

Test Category	Name of Test	Test Designation	
		AASHTO	ASTM
Strength Properties	Unconfined Compressive Strength of Cohesive Soil	T 208	D 2166
	Unconsolidated, Undrained Compressive Strength of Clay and Silt Soils in Triaxial Compression	T 296	D 2850
	Consolidated-Undrained Triaxial Compression Test on Cohesive Soils	T 297	D 4767
	Direct Shear Test of Soils For Consolidated Drained Conditions	T 236	D 3080
	Modulus and Damping of Soils by the Resonant-Column Method (Small-Strain Properties)	-	D 4015
	Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	-	D 4648
	Test Method for Bearing Ratio of Soils in Place	-	D 4429
	Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils	-	D 1883
	Test method For Resilient Modulus of Soils	T 294	-
	Method for Resistance R-Value and Expansion Pressure of Compacted Soils	T 190	D 2844
Permeability	Test Method for Permeability of Granular Soils (Constant Head)	T 215	D 2434
	Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	-	D 5084
Compression Properties	Method for One-Dimensional Consolidation Properties of Soils (Oedometer Test)	T 216	D 2435
	Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	T 258	D 4546
	Test Method for Measurement of Collapse Potential of Soils	-	D 5333

7.1.6 Visual Identification of Soils

Guidelines for visual identification of soils can be used in field as well as laboratory investigations.

Visual Identification of Soils	
AASHTO ASTM	- D 2488, D 4083
Purpose	1. Verify the field description of soil color and soil type. 2. Select representative specimens for various tests. 3. Select specimens for special tests (i.e., slickensided soils for triaxial testing) to determine the effects of the soil macro structure on the overall properties. 4. Locate and identify changes, intrusions, and disturbances within a sample. 5. Verify or revise the soil description to be included in the boring logs or in soil profile presentations.
Procedure	The visual-manual examination should be done expeditiously to ascertain the percent fines, relative percentages of gravel, sand, silt, & clay, as well as constituents & composition.
Commentary	Prior to assigning laboratory tests, all soil samples submitted to a laboratory should be subjected to visual examination and identification. It is advisable for the geotechnical engineer to be present during the opening of samples for visual inspection. He should remain in contact with the laboratory, as he can offer valuable assistance in assessing soil properties. **Disturbed Samples** As discussed earlier, disturbed samples are normally bulk samples of various sizes. Visual examinations of these samples are limited to the color, contents (i.e., gravel, concretions, sand, etc.) and consistency, as determined by handling a small, representative piece of the sample. The color of the soil should be determined by examining the samples in a jar or sealed can, where the moisture content is preserved near or at its natural condition. If more than one sample is obtained from the same deposit, the uniformity of the sample or lack of it is determined at this stage. This determination is used to decide on the proper mixing and quartering of disturbed samples to obtain representative specimens. **Undisturbed Samples** Undisturbed samples should be opened for examination one sample at a time. Prior to opening, the sample number, depth and other identifying marks placed on the sample tube or wrapping should be checked against field logs. Samples should be laid on their side on a clean table top. If samples are soft, they should be supported in a sample cradle of appropriate size; they should not be examined on a flat table top. Samples should be examined in a humid room where possible, or in rooms where the temperature is neither excessively warm nor cold. Once the samples are unwrapped, the technician, engineer or geologist examining the sample identifies its color, soil type, variations and discontinuities identifiable from surface features such as silt and sand seams, trace of organics, fissures, shells, mica, other minerals, and important features. The apparent relative strength, as determined by a hand-held penetrometer, is often noted during this process. Samples should be handled very gently to avoid disturbing the material. The examination should be done quickly before changes in the natural moisture content occur.

7.1.7 Index Properties

Index properties are used to characterize soils and determine their basic properties such as moisture content, specific gravity, particle size distribution, consistency and moisture-density relationships.

Moisture Content	
AASHTO ASTM	T 265 D 4959
Purpose	To determine the amount of water present in a quantity of soil in terms of its dry weight and to provide general correlations with strength, settlement, workability and other properties.
Procedure	Oven-dry the soil at a temperature of 110±5ºC to a constant weight (evaporate free water); this is usually achieved in 12 to 18 hours.
Commentary	Determination of the moisture content of soils is the most commonly used laboratory procedure. The moisture content of soils, when combined with data obtained from other tests, produces significant information about the characteristics of the soil. For example, when the in situ moisture content of a sample retrieved from below the phreatic surface approaches its liquid limit, it is an indication that the soil in its natural state is susceptible to larger consolidation settlement. Serious errors may be introduced if the soil contains other components, such as petroleum products or easily ignitable solids. When the soils contain fibrous organic matter, absorbed water may be present in the organic fibers as well as in the soil voids. The test procedure does not differentiate between pore water and absorbed water in organic fibers (although the procedure does suggest evaluating organic soils at a lower temperature of 60ºC to reduce decomposition of highly organic soils). Thus the moisture content measured will be the total moisture lost rather than free moisture lost (from void spaces). As discussed later, this may introduce serious errors in the determination of Atterberg limits.

Specific Gravity	
AASHTO ASTM	T 100 D 854
Purpose	To determine the specific gravity of the soil grains.
Procedure	The specific gravity is determined as the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights being taken in air.
Commentary	Some qualifying words like *true, absolute, apparent, bulk or mass,* etc. are sometimes added to "specific gravity". These qualifying words modify the sense of specific gravity as to whether it refers to soil grains or to soil mass. The soil grains have permeable and impermeable voids inside them. If all the internal voids of soil grains are excluded for determining the true volume of grains, the specific gravity obtained is called *absolute* or *true* specific gravity. Complete de-airing of the soil-water mix during the test is imperative while determining the *true* or *absolute* value of specific gravity. A value of specific gravity is necessary to compute the void ratio of a soil, it is used in the hydrometer analysis, and it is useful to predict the unit weight of a soil (see Table 7-2). Occasionally, the specific gravity may be useful in soil mineral classifications; e.g., iron minerals have a larger value of specific gravity than silica.

Unit Weight

The measurement of unit weight for undisturbed soil samples in the laboratory is simply determined by weighing a portion of a soil sample and dividing by its volume. This is convenient with thin-walled tube (Shelby) samples, as well as piston, Sherbrooke, Laval, and NGI samplers, as well. The water content should be obtained at the same time to allow conversion from total to dry unit weights, as needed.

Where undisturbed samples are not available, the unit weight is evaluated from weight-volume relations between the water content and/or void ratio, as well as the assumed or measured degree of saturation (see Table 7-2).   Additional methods using in-situ test data are discussed in Chapter 9.

Figure 7-1.   Laboratory Sieves for Mechanical Analysis for Grain Size Distributions.
Shown (right to left) are Sieve Nos. 3/8-in. (9.5-mm), No. 10 (2.0-mm), No. 40 (250-: m)
and No. 200 (750-: m) and example soil particle sizes including (right to left):
medium gravel, fine gravel, medium-coarse sand, silt, and dry clay (kaolin).

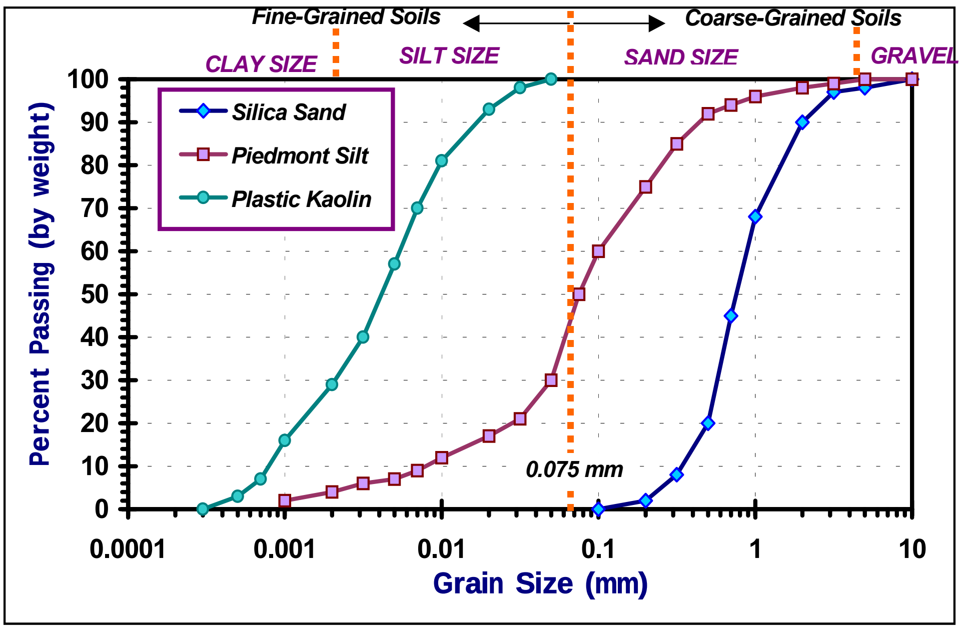
<table>
<tr><th colspan="2">Sieve Analysis</th></tr>
<tr><td>AASHTO
ASTM</td><td>T 88
D 422, D 1140</td></tr>
<tr><td>Purpose</td><td>To determine the percentage of various grain sizes. The grain size distribution is used to determine the textural classification of soils (i.e., gravel, sand, silty clay, etc.) which in turn is useful in evaluating the engineering characteristics such as permeability, strength, swelling potential, and susceptibility to frost action.</td></tr>
<tr><td>Procedure</td><td>Wash a prepared representative sample through a series of sieves (screens). Figure 7-1 shows a selection of sieves and soil particle sizes. The amount retained on each sieve is collected dried and weighed to determine the percentage of material passing that sieve size. Figure 7-2 shows several grain size distributions obtained from sieving and hydrometer methods including natural clays, silts, and various sands.


Figure 7-2: Representative Grain Size Curves for Several Soil Types.</td></tr>
<tr><td>Commentary</td><td>Obtaining a representative specimen is an important aspect of this test. When samples are dried for testing or “washing,” it may be necessary to break up the soil clods. Care should be made to avoid crushing of soft carbonate or sand particles. If the soil contains a substantial amount of fibrous organic materials, these may tend to plug the sieve openings during washing. The material settling over the sieve during washing should be constantly stirred to avoid plugging.

Openings of fine (< No. 200) mesh or fabric are easily distorted as a result of normal handling and use. They should be replaced often. A simple way to determine whether sieves should be replaced is the periodic examination of the stretch of the sieve fabric on its frame. The fabric should remain taut; if it sags, it has been distorted and should be replaced. A common cause of serious errors is the use of “dirty” sieves. Some soil particles, because of their shape, size or adhesion characteristics, have a tendency to be lodged in the sieve openings.</td></tr>
</table>

Hydrometer Analysis	
AASHTO ASTM	T 88 D 1140
Purpose	To determine distribution (percentage) of particle sizes smaller than No. 200 sieve (< 0.075 mm) and identify the silt, clay, and colloids percentages in the soil.
Procedure	Soil passing the No. 200 sieve is mixed with a dispersant and distilled water and placed in a special graduated cylinder in a state of liquid suspension. The specific gravity of the mixture is periodically measured using a calibrated hydrometer to determine the rate of settlement of soil particles. The relative size and percentage of fine particles are determined based on Stoke's law for settlement of idealized spherical particles.
Commentary	The principal value of the hydrometer analysis is in obtaining the clay fraction (percent finer than 0.002 mm). This is because the soil behavior for a cohesive soil depends principally on the type and percent of clay minerals, the geologic history of the deposit, and its water content rather than on the distribution of particle sizes. Replicable results can be obtained when soils are largely composed of common mineral ingredients. Results can be distorted and erroneous when the composition of the soil is not taken into account to make corrections for the specific gravity of the specimen. Particle size of highly organic soils cannot be determined by the use of this method.

Atterberg Limits	
AASHTO ASTM	T 89, T 90 D 4318
Purpose	To describe the consistency and plasticity of fine-grained soils with varying degrees of moisture.
Procedure	For the portion of the soil passing the No. 40 sieve, the moisture content is varied to identify three stages of soil behavior in terms of consistency. These stages are known as the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL) of soils. ‘ The *liquid limit* (LL) is defined as the water content at which 25 blows of the liquid limit machine (Figure 7-3a) closes a standard groove cut in the soil pat for a distance of 12.7 cm. An alternate procedure in Europe and Canada uses a fall cone device to obtain better repeatability (Figure 7-3b). ‘ The *plastic limit* (PL) is as the water content at which a thread of soil, when rolled down to a diameter of 3 mm, will crumble. ‘ The *shrinkage limit* (SL) is defined as that water content below which no further soil volume change occurs with further drying.
Commentary	The Atterberg limits provide general indices of moisture content relative to the consistency and behavior of soils. The LL defines a liquid/semi-solid change, while the PL is a solids boundary. The difference is termed the *plasticity index* (PI = LL - PL). The *liquidity index* is LI = (w-PL)/PI is an indicator of stress history; LI . 1 for normally consolidated (NC) soils and LI . 0 for over-consolidated (OC) soils. By and large, these are approximate and empirical values. They were originally developed for agronomic purposes. Their widespread use by engineers has resulted in the development of a large number of rough empirical relationships for characterizing soils. Considering the abstract and manual nature of the test procedure, Atterberg limits should only be performed by experienced technicians. Lack of experience, and lack of care will introduce serious errors in the test results.

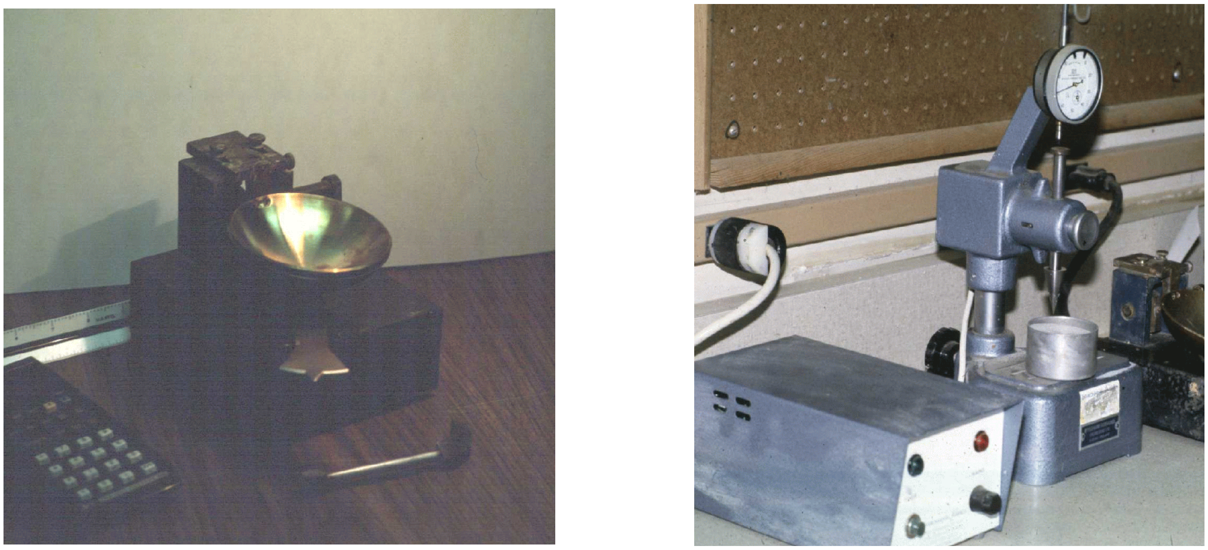


Figure 7-3. Liquid Limit Test by (a) Manual Casagrande Cup Device; (b) Electric Fall Cone.

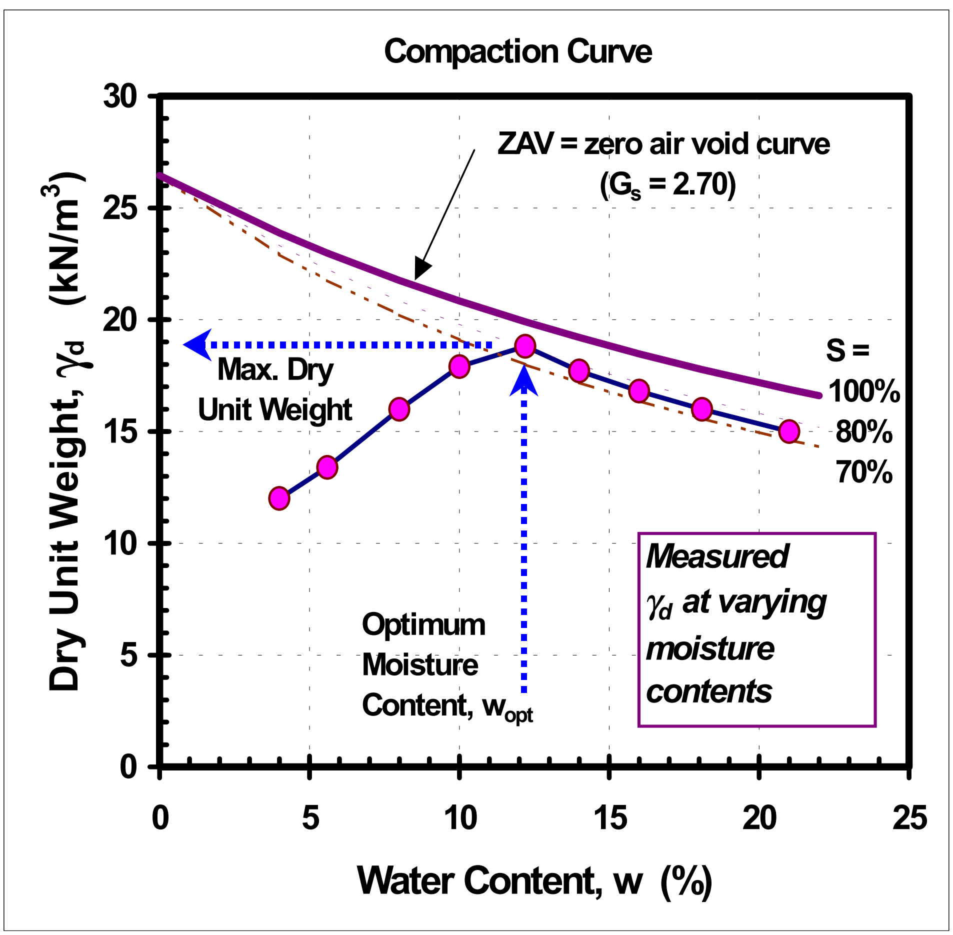


Figure 7-4. A Representative Moisture-Density Relationship from a Standard Compaction Test.

Moisture-Density (Compaction) Relationship	
AASHTO ASTM	T 99 (Standard Proctor), T 180 (Modified Proctor) D 698 (Standard Proctor), D 1557 (Modified Proctor)
Purpose	To determine the maximum dry density attainable under a specified nominal compaction energy for a given soil and the (optimum) moisture content corresponding to this density.
Procedure	Compaction tests are performed using disturbed, prepared soils with or without additives. Normally, soil passing the No. 4 sieve is mixed with water to form samples at various moisture contents ranging from the dry state to wet state. These samples are compacted in layers in a mold by a hammer in accordance with a specified nominal compaction energy. Dry density is determined based on the moisture content and the unit weight of compacted soil. A curve of dry density versus moisture content is plotted in Figure 7-4 and the maximum ordinate on this curve is referred to as the maximum dry density (γ_{dmax}). The water content at which this dry density occurs is termed as the optimum moisture content (OMC).
Commentary	In the construction of highway embankments, earth dams, retaining walls, structure foundations and many other facilities, loose soils must be compacted to increase their densities. Compaction increases the strength and stiffness characteristics of soils. Compaction also decreases the amount of undesirable settlement of structures and increases the stability of slopes and embankments. The density of soils is measured as the unit dry weight, γ_d, (weight of dry soil divided by the bulk volume of the soil). It is a measure of the amount of solid materials present in a unit volume. The higher the amount of solid materials, the stronger and more stable the soil will be. To provide a "relative" measure of compaction, the concept of relative compaction is used. Relative compaction is the ratio (expressed as a percentage) of the density of compacted or natural in-situ soils to the maximum density obtainable in a compaction test. Often it is necessary to specify the achieving of a certain level of relative compaction (e.g. 95%) in the construction or preparation of foundations, embankments, pavement sub-bases and bases, and for deep-seated deposits such as loose sands. The design and selection of a placement method to improve the strength, dynamic resistance and consolidation characteristics of deposits depend heavily on relative compaction measurements. During the compaction of several specimens, the total unit weight of each compacted specimen is measured at each water content and the two soil identities used to obtain the needed parameters: *(1)* $G_s w = S e$,and *(2)* $\gamma_t = G_s \gamma_w (1+w)/(1+e)$. The dry unit weight is obtained as: $\gamma_d = \gamma_t/(1+w)$. It is also convenient to plot the zero air voids (ZAV) curve on the moisture-density graph, corresponding to 100 percent saturation (see Figure 7-4). The measured compaction curve response should not fall on or above this ZAV line. The maximum dry unit weight ("density") found as the peak value often corresponds to saturation levels of between 70 to 85 percent. Where a variety of soils are to be used for construction, a moisture-density relationship for each major type of soil present at the site should be established.

Moisture-Density (Compaction) Relationship	
AASHTO ASTM	T 99 (Standard Proctor), T 180 (Modified Proctor) D 698 (Standard Proctor), D 1557 (Modified Proctor)
	When additives such as Portland cement, lime, or fly ash are used to determine the maximum density of mixed compacted soils in the laboratory, care should be taken to duplicate the expected delay period between mixing and compaction in the field. It should be kept in mind that these chemical additives start reacting as soon as they are added to the wet soil. They cause substantial changes in soil properties, including densities achievable by compaction. If in the field the period between mixing and compaction is expected to be three hours, for example, then in the laboratory the compaction of the soil should also be delayed three hours after mixing the stabilizing additives. Relative density (D_R) (ASTM D 4253) is often a useful parameter in assessing the engineering characteristics of granular soils and is defined as: $D_R = 100 (e_{max} - e)/(e_{max} - e_{min})$ (7-4) that can also be expressed in terms of dry unit weights. A greater discussion of D_R is given later in Chapter 9.

Classification of Soils	
AASHTO ASTM	M 145 D 2487, D 3282
Purpose	To provide in a very concise manner information on the type and fundamental characteristics of soils, their utility as construction or foundation materials, their constituents, etc.
Procedure	See Section 4.6
Commentary	See Section 4.6

Corrosivity of Soils	
AASHTO ASTM	T 288, T 289, T 290, T 291 G 51, D 512, D 1125, D 2976. D 4230 , D 4972
Purpose	To determine the aggressiveness and corrosivity of soils, pH, sulfate and chloride content of soils.
Procedure	Usually the pH of a soil material is determined electrometically by a pH meter which is a potentiometer equipped with a glass-calomel electrode system calibrated with buffers of known pH. Measurements are commonly performed on a suspension of soil, water and/or alkaline (usually calcium chloride) solutions.
Commentary	Because of their environment or composition soils may have varying degrees of acidity or alkalinity, as measured by the pH test. Measurements of pH are particularly important for determining corrosion potential where metal piles, culverts, anchors, metal strips, or pipes are to be used. pH is also an important parameter for evaluating the durability of geosynthetics.

Resistivity	
AASHTO ASTM	T 288 G 57
Purpose	To determine the corrosion potential of soils.
Procedure	The laboratory test for measuring the resistivity of soils is performed using dried prepared soil passing the No. 8 screen. The soil is placed in a box approximately 10.2 cm x 15.2 cm x 4.5 cm with electrical terminals attached to the sides of the box such that they remain in contact with the soil. The terminals in turn are connected to an ohmmeter. A reading of the current passing through the dry soil is taken as the baseline reference resistance. The soil material is then removed and 50 ml to 100 ml of distilled water is added and thoroughly mixed, and placed back in the box. Another reading is taken. The conductivity (conductivity is the reverse of resistivity) of the soil as read by the ohmmeter increases as water is added. The procedure is repeated until the conductivity begins dropping. The highest conductivity, or the lowest resistivity, is used to compute the resistivity of the soil. The method is very sensitive to the distribution of water in the soils placed in the box. The resistivity may also vary significantly with the presence of soluble salts in soils.
Commentary	Where construction materials susceptible to corrosion are to be used in subgrades it is necessary to determine the corrosion potential of soils. This test is routinely performed for structures where metallic reinforcements, soil anchors, nails, culverts, pipes, or piles are included.

Organic Content of Soils	
AASHTO ASTM	T 194 D 2974
Purpose	To help classify the soil and identify its engineering characteristics.
Procedure	Oven-dried (at 110±5ºC) samples <u>after</u> determination of moisture content are further gradually heated to 440ºC which is maintained until the specimen is completely ashed (no change in mass occurs after a further period of heating). The organic content is then calculated from the weight of the ash generated.
Commentary	Organic materials affect the behavior of soils in varying degrees. The behavior of soils with low organic contents (<20% by weight) generally are controlled by the mineral components of the soil. When the organic content of soils approaches 20%, the behavior changes to that of organic, or peaty soils. The consolidation characteristics, permeability, strength and stabilization of these soils are largely governed by the properties of organic materials. Thus it is important to determine the organic content of soils. It is not sufficient to simply label a soil as "organic" without showing the organic content. Organic soils are those formed throughout the ages at low-lying sediment-starved areas by the accumulation of dead vegetation and sediment. Top soils are very recently formed mixtures of soil and vegetation that form part of the food chain. Top soils are not suitable for use in construction and therefore its organic content is not usually determined.

7.1.8 Strength Tests

The design and analysis of shallow and deep foundations, excavations, earth retention structures, and fills and slopes require a thorough understanding of soil strength parameters. The selection of strength parameters needed and the corresponding types of tests to be performed vary depending on the type of construction, the foundation design, the intensity, type and duration of loads to be imposed, and soil materials existing at the site.

The shear strength should be determined by a combination of both field and laboratory tests. Lab tests provide reference strengths under controlled boundaries and loading. However, limited quality samples are obtained from the field, particularly for sandy materials. The interpretation of strength from in-situ tests in sands and clays is important and discussed in Chapter 9.

For clays, commonly used laboratory tests include the unconfined compression (UC) and unconsolidated undrained tests (UU), however, these do not attempt to replicate the ambient stress regime in the ground prior to loading and therefore can only be considered as index strengths. Preferably, the consolidated triaxial shear and direct shear box tests can be used in conjunction with consolidation/oedometer tests in a normalized stress history approach (Ladd & Foott, 1974; Jamiolkowski, et al. 1985).

Both undisturbed and remolded or compacted samples are used for strength tests. Where soils are to be disturbed and remolded, compacted or stabilized specimens are tested for strength determination at specified moisture contents and densities. These may be chosen on the basis of design requirements or the in-situ density and moisture content of soils. Where obtaining undisturbed samples is not practical (i.e., sandy and gravelly soils), specimens remolded close to their natural moisture content and density are prepared for testing.

Total and Effective Stress Analysis

Soils are controlled by the effective stress strength envelope (c' and ϕ') and therefore the proper determination of these parameters is paramount. The strength envelope is best determined by either a series of (1) consolidated undrained triaxial shear tests with porewater pressure measurements ($\overline{CU}$); (2) consolidated drained triaxial tests at slow strain rates (CD); or (3) drained direct shear tests (DDS). For long-term analyses, the drained parameters are equal to effective cohesion intercept c' and effective friction angle ϕ' from the effective stress Mohr-Coulomb envelope (see Figure 7-5). The shear strength (τ_{max}) is given by:

$$\tau_{max} = c' + \sigma_N' \tan \phi' \tag{7-5}$$

Usually, $c' \approx 0$ is adopted because lab tests are affected by rate & duration effects and c' is a bond that weathers with time (e.g., Mesri & Abdel-Ghaffar, 1993). Effective strength parameters apply to all soil types, including gravels, sands, silts, and clays.

The stress dependency of soil can be characterized by the stress path method. A stress path gives a numerical and graphical representation of the past, present and future state of stress on a representative soil element. It captures the geologic stress history of the element, the current stresses acting on the element, and the anticipated future changes in stress on the element. The stress path method determines what these stresses are, subjects representative elements of soil to these stress paths, and measures the resulting mechanical behavior of the soil. The measurements are used to determine strength, compressibility and permeability for specific stress paths. These stress path dependent mechanical properties are then used in analysis and design to predict the future performance of a constructed facility.

The 66 triaxial test results can be used to develop the "stress path" of the soil under the test conditions by plotting the effective strength for each load increment from the start to finish of the test. Using the stress path method, the test results can then be analyzed with respect to the approximate field stress and strain conditions before, during, and after construction (Lambe, 1967 and Lambe and Marr, 1979).

For short-term loading of clays & silts, total stress analysis uses the undrained shear strength (designated s_u or c_u)[1] that is a soil behavioral response that reflects the combination of the effective stress frictional envelope (cr and Nr) plus excess porewater pressures that depend on stress history. From this regard, perhaps the simple shear is the most appropriate test for stability & bearing capacity analyses, however, the device is not in widespread use in the U.S. Other modes of s_u include triaxial compression & extension, plane strain active & passive, true triaxial, hollow cylinder, and directional shear, all of which provide different values of s_u depending upon the boundary conditions, direction of loading, strain rate, and initial stress state. As this is a complex issue, the best value is calculated from the normalized value (Jamiolkowski, et al., 1985):

$$s_u / F_{vo}r = 0.5 \sin Nr \, OCR^{0.8} \qquad (7\text{-}6)$$

For extensively fissured clays and tills, the macrofabric of discontinuities reduces the overall strength and (7-6) should be reduced by a factor of 2. In the case of fissured geomaterials, it is also common that these exhibit past problems with landsliding and slope instability, therefore the drained strength parameters may be more appropriately assigned to the residual values (c_rr and N_rr). Residual strengths can be determined by ring shear tests or series of repeated drained direct shear box tests (Lupini, et al. 1981).

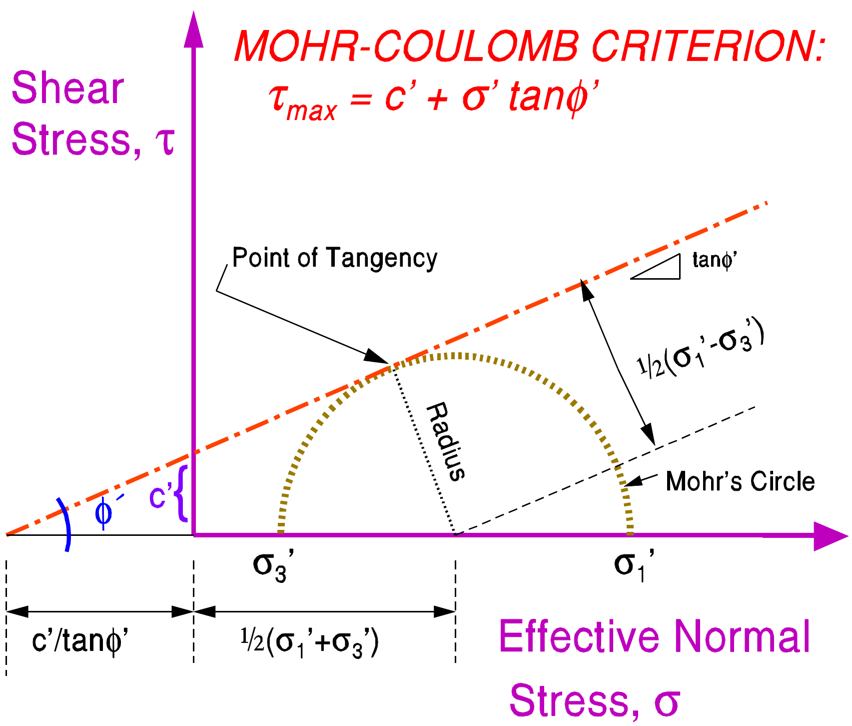


Figure 7-4. Definitions of Effective Stress Parameters For Mohr-Coulomb Failure Criterion.

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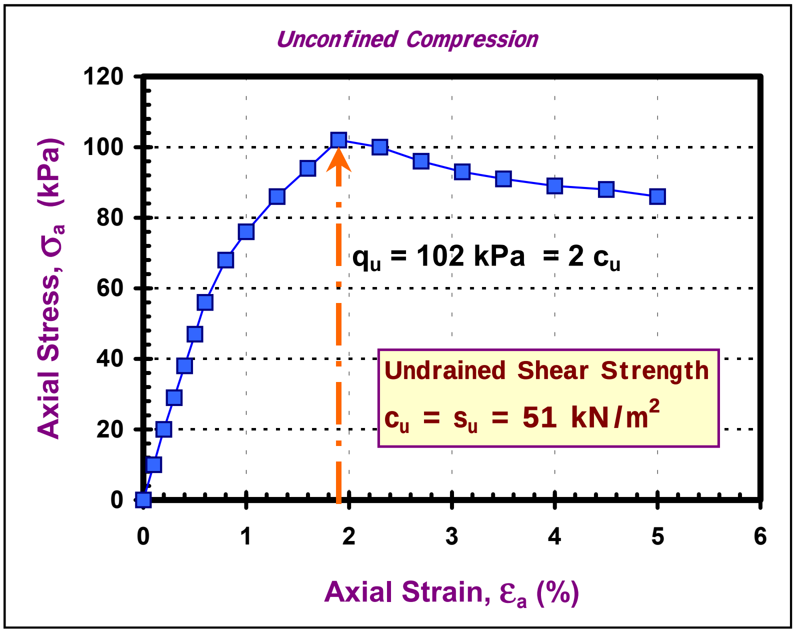
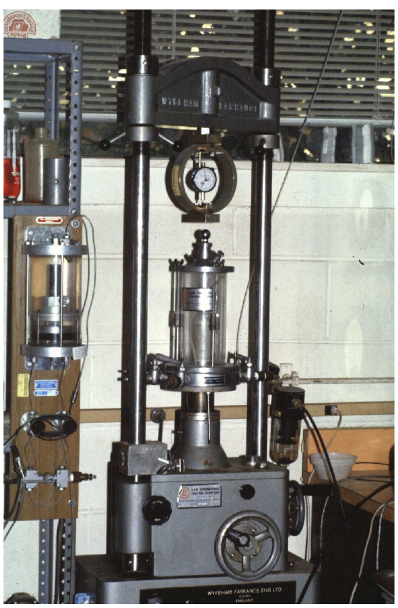
[1] Note: The old archaic term "cohesion" designated "c" has been replaced with undrained shear strength.

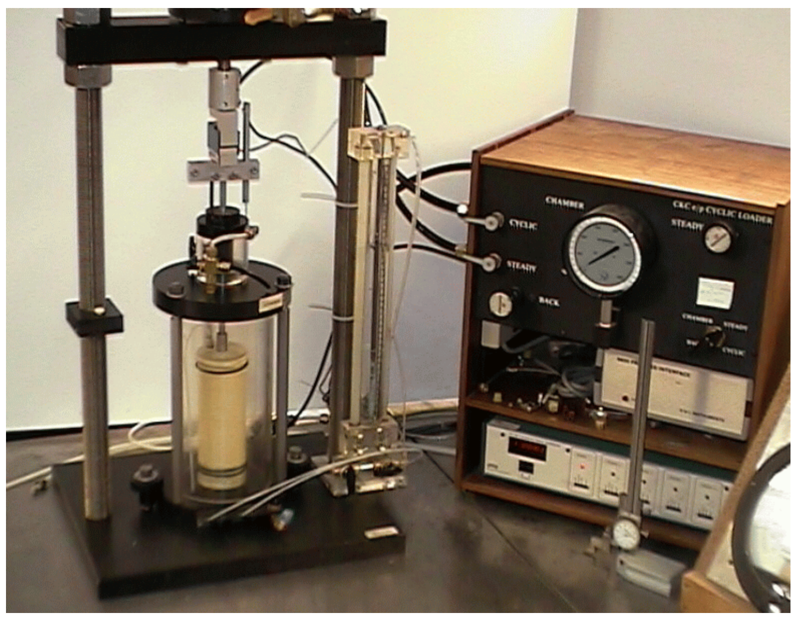
Unconfined Compressive Strength of Soils

AASHTO ASTM	T 208 D 2166
Purpose	To determine the undrained shear strength (c_u) of clay and silty clay soils.
Procedure	The soil specimens are tested without any confinement or lateral support (F_3=0). Axial load is rapidly applied to the sample to cause failure. At failure the total minor principal stress is zero (F_3 = 0) and the total major principal stress is F_1 (see Figure 7-6). The maximum measured force over the sample area is q_u and referred to as the unconfined compression strength. Since the undrained strength is independent of the confining pressure, $c_u = q_u/2$.  **Figure 7-6. Measured Stress-Strain for Unconfined Compressive Test.**
Commentary	The determination of unconfined compressive strength of undisturbed, remolded or compacted soils is limited to cohesive or naturally or artificially cemented soils. Application of this test to non-cohesive soils may result in underestimation of the shear strength. The test is inexpensive and requires a relatively short period of time to complete. However, due to the absence of lateral pressures and lack of control over pore pressures, it has major inaccuracies. The stress-strain curves and failure modes observed during testing provide an index value of the soil properties in addition to strength. For example, an ill-defined failure or yielding of the sample signifies a relatively soft, fat clay, while a sudden brittle failure indicates that of a desiccated clay or cemented material. The stress-strain curves developed from these tests should be used with caution when determining soil modulus for input to numerical analyses, such as finite element analysis, which are very sensitive to minor variations of the modulus. Soils with inclined fissures, sand & silt lenses and slickensides have a tendency to fail prematurely along these weaker planes in unconfined compression tests. It is essential that such failure modes be reported to the geotechnical engineer, who may request further more sophisticated testing such as triaxial tests to obtain more realistic determination of the in situ strength.

Triaxial Strength	
AASHTO ASTM	T 296, T 297 D 2850, D 4767
Purpose	To determine strength characteristics of soils including detailed information on the effects of lateral confinement, porewater pressure, drainage and consolidation. Triaxial tests provide a reliable means to determine the friction angle of natural clays & silts, as well as reconstituted sands. The stiffness (modulus) at intermediate to large strains can also be evaluated.
Procedure	The triaxial test set-up is shown in Figure 7-7. Test samples are typically 35 to 75 mm in diameter and have a height to length ratio between 2 and 2.5. The sample is encased by a thin rubber membrane and placed inside a plastic cylindrical chamber that is usually filled with water or glycerine. The sample is subjected to a total confining pressure (σ_3) by compression of the fluid in the chamber acting on the membrane. A backpressure (u_o) is applied directly to the specimen through a port in the bottom pedestal. Thus, the sample is initially consolidated with an effective confining stress: $\sigma_3' = (\sigma_3 - u_o)$. (Note that air should not be used as a compression medium). To cause shear failure in the sample, axial stress is applied through a vertical loading ram (commonly called *deviator stress* = $\sigma_1 - \sigma_3$). Axial stress may be applied at a constant rate (strain controlled) or by means of a hydraulic press or dead weight increments or hydraulic pressure (stress controlled) until the sample fails. The axial load applied by the loading ram corresponding to a given axial deformation is measured by a proving ring or electronic load cell attached to the ram. Connections to measure drainage into or out of the sample, or for porewater pressure are also provided. Deflections are monitored by either dial indicators, LVDTs, or DCDTs.
Commentary	In general, there are five types of triaxial tests: • Undrained Unconsolidated (UU test) • Consolidated Undrained (CU test) • Consolidated Drained (CD test) • Consolidated Undrained with pore pressure measurement (66) • Cyclic Triaxial Loading Tests (CTX) In a UU test, the samples are not allowed to drain or consolidate prior to or during the testing. The results of undrained tests depend on the degree of saturation (S) of the specimens. Where S=100%, the test results will provide a value of undrained shear strength (s_u), however, the test is affected by sample disturbance and rate effect (Ladd, 1991). This test is not applicable for granular (S=100%) soils. The (66) test with porewater pressure measurements is the most useful as it provides a direct measure of the undrained shear strength (s_u), for triaxial compressive mode, as well as the important effective stress parameters (c' and ϕ'). The CD tests also provide the parameters c' and ϕ'. Cyclic triaxial tests are used for projects with repeated and/or cyclic loading, resilient modulus determinations, and/or liquefaction analysis of soils. In each of these tests, the specimen is initially consolidated to the effective vertical overburden stress (σ_{vo}') prior to shear. If additional specimens from the same tube are tested, these may be tested at confining stress levels of 0.5 (σ_{vo}') to 1.5 (σ_{vo}'), in order to provide a range of operating values. The results can be presented in terms of Mohr Circles of stress to obtain the strength parameters (Figure 7-8). If more than two or three tests are conducted, the results are more conveniently plotted on q-p space, where $q = ½(\sigma_1 - \sigma_3)$ and $p' = ½(\sigma_1' + \sigma_3')$, as illustrated in Figure 7-9. In addition, the entire stress path from start to finish can be followed.

(a) (b)





(c) (d)

Figure 7-7. Triaxial Test Apparatuses and Equipment:
(a) Specimen Being Consolidated in Triaxial Cell Prior to Shear: (b) Automated Cyclic Triaxial Equipment (Geocomp Corp); (c) Mechanical Gear-Driven Load Frame and Triaxial System (Wykeham Farrance Ltd.); (d) Controlled Triaxial System for Isotropic and/or K_o -Consolidated Triaxial Compression and Extension Testing (CKC System).

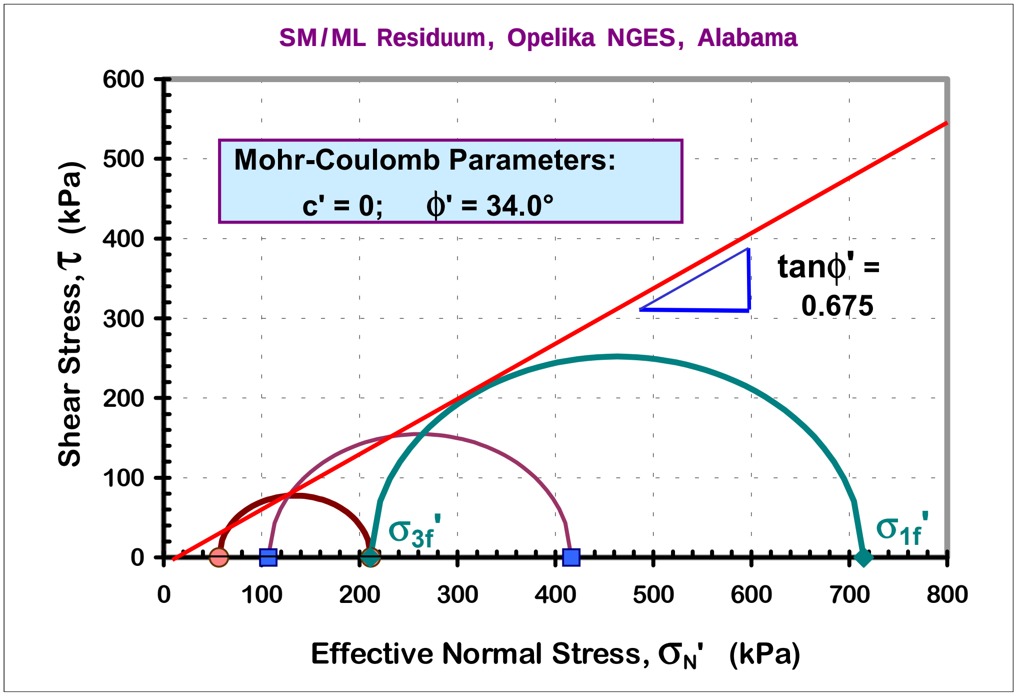


Figure 7-8. Effective Stress Mohr Circles for Consolidated Undrained Triaxial Tests.

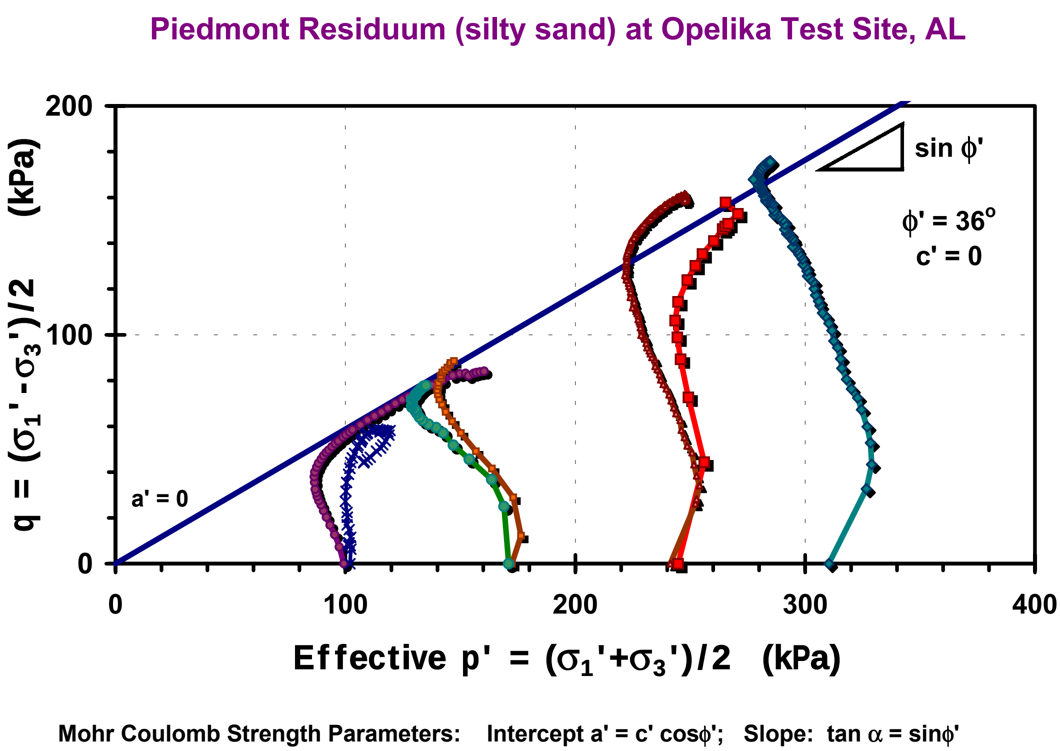
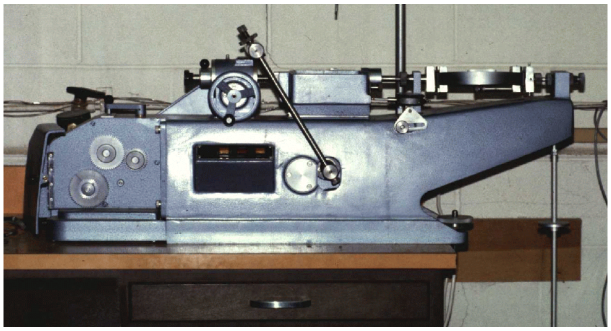
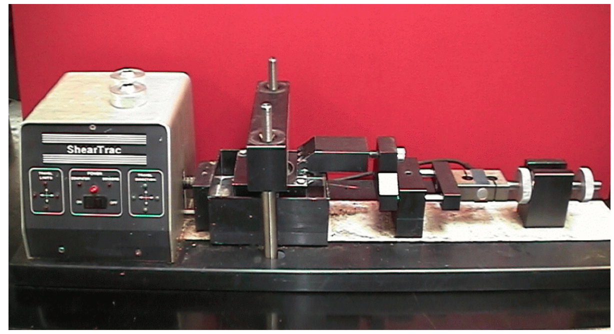


Figure 7-9. Effective q-p' Strength Envelopes for Consolidated Undrained Triaxial Tests.

Direct Shear	
AASHTO ASTM	T 236 D 3080
Purpose	To determine the shear strength of soils along a pre-defined (horizontal) planar surface
Procedure	The direct shear (DS) test is performed by placing a specimen into a cylindrical or square-shaped shear box which is split in the horizontal plane. DS devices are shown in Figure 7-10. A vertical (normal) load is applied over the specimen that is allowed to consolidate. While either the upper or lower part of the box is held stationary, a horizontal load is exerted on the other part of the box in an effort to shear the specimen on a predefined horizontal plane. The test is repeated at least three times using different normal stresses (F_Nr) The results are plotted in the form shear stress (J) vs. horizontal displacement (*), and corresponding J vs. F_Nr. The effective cohesion intercept and angle of internal friction values can be determined from this latter plot.
Commentary	**Direct Shear (Box) Test** The DS test is the oldest and simplest form of shear test arrangement. It has several inherent shortcomings due to the forced plane of shearing: C The failure plane is predefined and horizontal; this plane may not be the weakest. C As compared to the triaxial test, there is little control over the drainage of the soil. C The stress conditions across the soil sample are very complex. The distribution of normal stresses and shearing stresses over the sliding surface is not uniform; typically the edges experience more stress than the center. Due to this, there is progressive failure of the specimen, i.e., the entire strength of the soil is not mobilized simultaneously. In spite of the above shortcomings, the direct shear test is commonly used as it is simple and easy to perform. The device uses much less soil than a standard triaxial device, therefore consolidation times are shorter. The DS provides reasonably reliable values for the effective strength parameters, cr and Nr, provided that slow rates of testing are utilized (see Figure 7-11). Repeated cycles of shearing along the same direction provide an evaluation of the residual strength parameters (c_rr and N_rr). The direct shear test is particularly applicable to those foundation design problems where it is necessary to determine the angle of friction between the soil and the material of which the foundation is constructed, e.g., the friction between the base of a concrete footing and underneath soil. In such cases, the lower box is filled with soil and the upper box contains the foundation material. **Direct Simple Shear (DSS) Test** The DSS test was developed in an attempt to refine the direct shear test by providing shear strain distortion, rather than horizontal displacement. Earlier DSS test devices used a cylindrical specimen confined in rubber membrane reinforced with a series of evenly spaced rigid rings. Later versions developed by the Norwegian Geotechnical Institute (NGI) used square specimens with hinged end plates that could tilt to maintain fixed specimen length during shearing. The NGI version is used by a number of European geotechnical agencies. Some of the studies performed show that this device provides a means of studying plane strain (i.e., embankment loads). Studies at MIT, NGI, Swedish Geotechnical Institute, and Politecnico di Torino have concluded that the DSS provides the most representative mode for the mobilized undrained strength in stability analyses involving embankments, footings, and excavations in soft ground.



(a)



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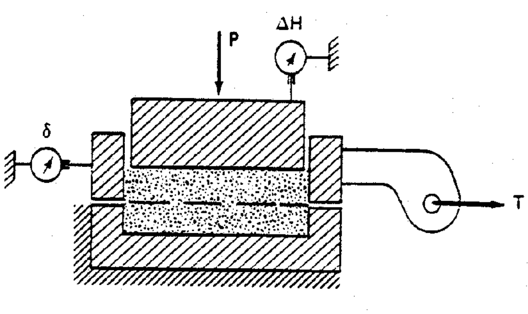
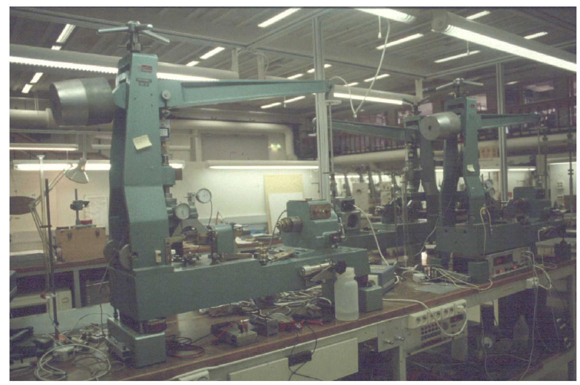


(c)



(d)

Figure 7-10. Direct Shear Test Devices: (a) Mechanical Wykeham Farrance Device; (b) Electro-Mechanical ShearTrac (GeoComp Corp) ; (c) Shear Box Cross-Section; (d) NGI Direct Simple Shear.

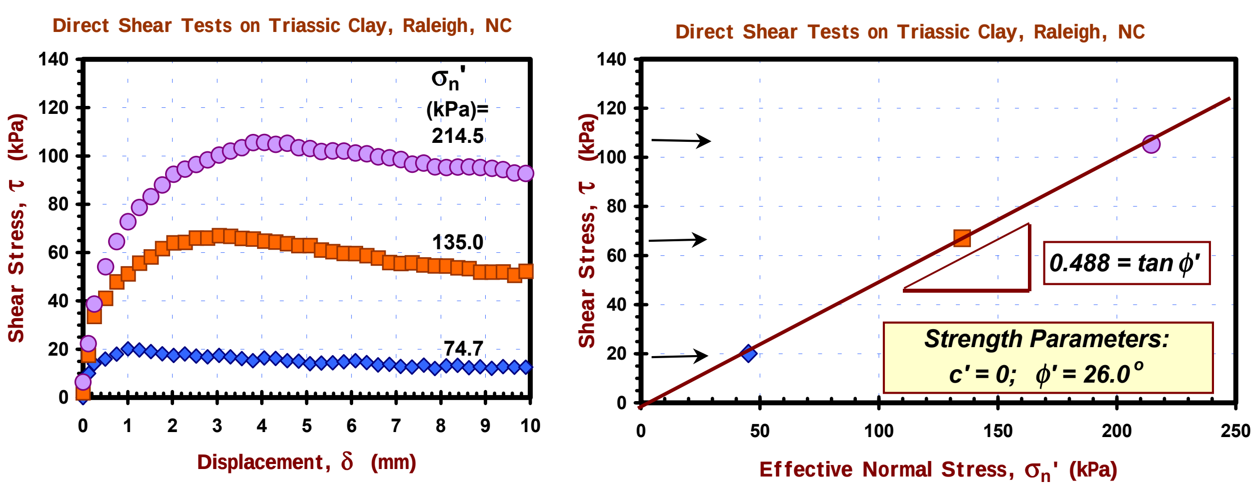


Figure 7-11. Illustrative Results from DS Tests on Clay Involved in Route 1 Slope Stability Study, Raleigh, NC.

Resonant Column	
ASTM	D 4015
Purpose	To determine the shear modulus (G_{max} or G_0) and damping (D) characteristics of soils at small strains for cases where dynamic forces are involved, particularly seismic ground amplification and machinery foundations. Recent research has shown the results are also applicable to static loading at very small strains (< 10^{-6} percent); for example (Burland, 1989).
Procedure	Prepared cylindrical specimens are placed in an special triaxial chamber and consolidated to ambient overburden stresses (Figure 7-12). Very low amplitude torsional vibrations are applied to one end of the specimen by use of a special loading cap with electromagnetics. The resonant frequency, damping, and strain amplitudes are measured by the use of motion transducers (Woods, 1994).
Commentary	The resonant column test (RCT) requires a high-caliber laboratory setup with special care in calibration and maintenance of frequency-domain electronics (e.g., spectrum analyzer). The fundamental measurement of shear wave velocity (V_s) provides the small-strain shear modulus: $$G_{max} = D_T (V_s)^2 \qquad (7\text{-}7)$$ where D_T = $(_T/g$ = total soil mass density and g = 9.8 m/s^2 = gravitational acceleration constant. Although field methods such as the crosshole, downhole, surface wave, and suspension logging techniques provide direct in-situ measurements of V_s, the RCT is advantageous in that it can evaluate the variation (decrease) of G_{max} with increasing shear strain ($(_s$), as well as the increase of damping (D) with $(_s$, as illustrated in Figure 7-13. There are however significant time (soil aging) effects, which can lead to lower values than obtained in the field. Generally, the RCT is considered a nondestructive test and the material properties are essentially unchanged during the small-strain torsional loading. Therefore, it is common that the same specimen can be subjected to several levels of effective confining stress. Over three decades experience with the RCT on soils indicates that G_{max} is a function of void ratio (e) and mean effective confining stress, F_o' =⅓(F_{vo}r+2 F_{ho}r), as well as cementation, aging, saturation, and other factors. A well-known expression is: $$G_{max} = (625/e^{1.3})(F_{ATM}\, F_o')0.5\ OCR^5 \qquad (7\text{-}8)$$ where 5 . ($PI^{0.72}$)/50 and F_{ATM} = atmospheric pressure (1 bar . 100 kPa . 1 tsf).

Figure 7-12. Resonant Column Test (RCT) Equipment for Determining G_{max} and D in Soils.

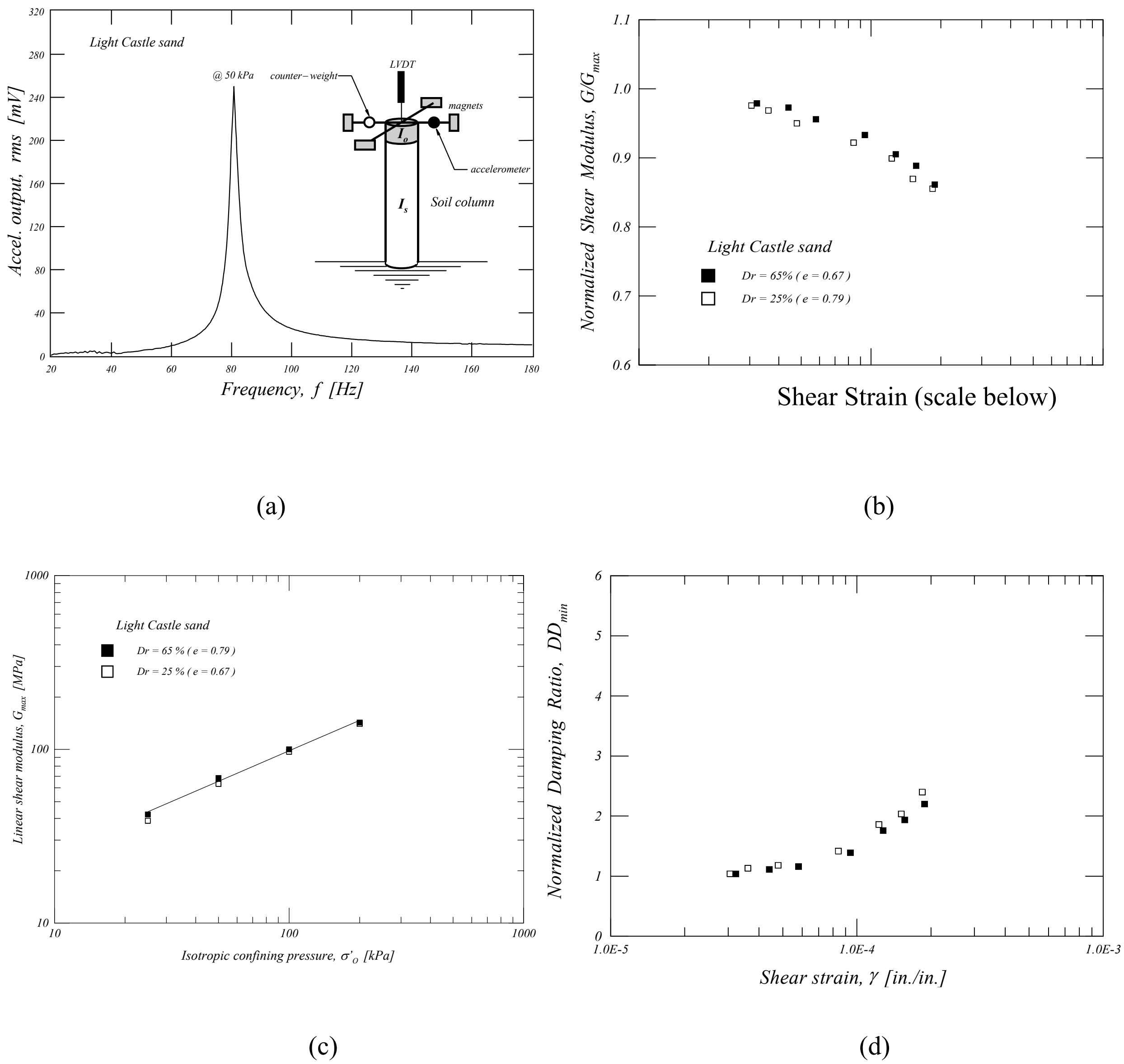


Figure 7-13. Results from Resonant Column Testing of Light Castle Sand:
(a) Measured Resonance at a Given Effective Confining Stress and Shear Strain;
(b) Normalized Modulus Reduction (G/G_{max}) with Shear Strain; (c) Variation of Small-Strain Shear Modulus (G_{max}) with Effective Confining Stress Level; and (d) Damping Ratio (D) increase with Shear Strain.

Miniature Vane	
AASHTO ASTM	- D 4648
Purpose	To determine the undrained shear strength (s_u) and sensitivity (S_t) of saturated clays and silts
Procedure	The test is performed by inserting a four-bladed vane into the soil and applying rotation to shear a cylindrical surface. The undrained shear strength is computed from the measured torque (see Chapter 5). The miniature vane is similar to the field vane shear device, except that it is smaller (blade diameter 12.7 mm, blade height 25.4 mm).
Commentary	The test assumes that the stresses applied are limited to the cylindrical surface represented by the diameter and the height of the vane. This is hardly the case in reality. Depending on the strength and stiffness, the soils in an area radiating outward from the surface of the idealized cylindrical zone are also disturbed by the shearing action of the vane. A portion of the torque therefore is used to mobilize this zone. Thus the assumption that the only sheared zone is the one defined by the outline of the vane blades introduces varying degrees of error. The analysis of the test assumes that strength of the soil being tested is isotropic, which is not true for all deposits. The test, however, can be a useful tool for measuring anisotropy and remolded strength of saturated clays and silts. The ratio of peak to remolded undrained strengths is the *sensitivity* (S_t). The laboratory vane shear test should be used as an index test.

California Bearing Ratio (CBR)	
AASHTO ASTM	T 193 D 4429 (for field); D 1883 (for laboratory)
Purpose	To determine the bearing capacity of a compacted soil under controlled moisture and density conditions.
Procedure	The test results are expressed in terms of a bearing ratio which is commonly known as the California Bearing Ratio (CBR). The CBR is obtained as the ratio of the unit load required to cause a certain depth of penetration of a piston into a compacted specimen of soil at some water content and density, to the *standard unit load* required to obtain the same depth of penetration on a standard sample of crushed stone (usually limestone). Typically soaked conditions should be used to simulate anticipated long-term conditions in the field. The CBR test is run on three identically compacted samples. Each series of the CBR test is run for a given relative density and moisture content. The geotechnical engineer must specify the conditions (dry, at optimum moisture, after soaking, 95% relative density, etc.) under which each test should be performed.
Commentary	CBR is a practical bearing capacity test, yet provides only discrete point test data for evaluation. Most CBR testing is laboratory-based, thus the results will be highly dependent on the representativeness of the samples tested. The test results are used for highway, airport, parking lot and other pavement designs using empirical local or agency-specific methods (i.e., FHWA, FAA, AASHTO). More often than not, pavement failures are due to poor drainage, overloaded truck traffic, increased overall road traffic, and wear.

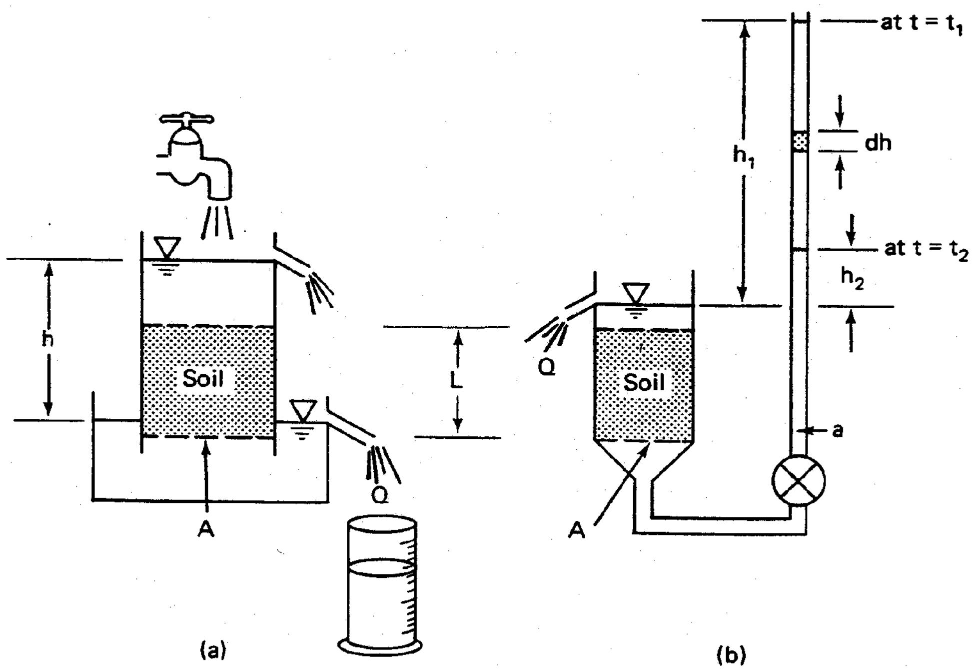
R-Value Test	
AASHTO ASTM	T 190 D 2844
Purpose	To determine the ability of a soil to resist lateral deformation when a vertical load acts upon it. The resistance is indicated by the R-value.
Procedure	Measuring the R-value of a soil is done with a stabilometer. A stabilometer is similar to a triaxial device consisting of a metal cylinder in which there is a rubber membrane; the annular space between the two is filled with oil that transmits lateral pressure to the specimen. Compacted, unstabilized or stabilized soils and aggregates, can be used in these tests. Samples are compacted using a special kneading compaction device. When the specimen is vertically loaded, a lateral pressure is transmitted to the soil, which can be measured on a pressure gage. From the displacement measured for a specified lateral pressure, the R-value is determined.
Commentary	The R-Value test was developed by the California Division of Highways for use in the empirical design method developed by them. Later it was widely adopted for use in pavement design. The kneading compactor used to prepare the test samples is considered to more closely model the compaction mode of field equipment by its kneading action. Specimens fabricated by this method develop internal structures more representative of actual field compacted materials where soil particles are kneaded together rather than densified by impact force. The R-Value is used either directly or translated into more common factors (i.e., CBR) through correlation charts to be used with other more common design methods (i.e., AASHTO). This test method indirectly measures the strength of pavement materials by measuring the resistance to deformation under lateral and normal stresses. The test also allows the measurement of swell pressure of expansive soils. The strength data is used in the design of pavements to determine the thickness of various components of pavement structures. The swell pressure or expansion pressure data is used in determining the suitability of expansive soils for use under pavements and the intensity of stress needed, in the form of overburden, to control the expansion of these soils.

Resilient Modulus	
AASHTO ASTM	T 294 -
Purpose	To determine the approximate relationships between applied stress and deformation loading of pavement component materials.
Procedure	A compacted or undisturbed cylindrical specimen is placed in an oversized triaxial chamber. An axial deviator stress of constant magnitude and duration and frequency is applied at the same time that a lateral stress is maintained in the triaxial chamber. The recoverable or resilient axial strain of the specimen is measured for varying increments of axial stresses.
Commentary	The test is time-consuming and requires special test and laboratory setup. One specimen can be used for a variety of axial loads. Both undisturbed and disturbed specimens representing the pavement materials can be used. Sample preparation of remolded specimens requires a thorough appreciation of the existing or expected field conditions. Values obtained can be used to determine the linear or non-linear elastic response of pavement component materials.

7.1.9 Permeability

The hydraulic conductivity or permeability is an important flow property of soils.

Permeability of Soils	
AASHTO ASTM	T 215 D 2434 (Granular Soils), D 5084 (All Soils)
Purpose	To determine the potential of flow of fluids through soils.
Procedure	The ease with which a fluid passes through a porous medium is expressed in terms of coefficient of *permeability* (k), also known as *hydraulic conductivity*. There are two basic standard types of test procedures to directly determine permeability: (1) constant-head; and (2) falling-head procedures (see Figure 7-14). In both procedures, undisturbed, remolded, or compacted samples can be used. The permeability of coarse materials is determined by constant head tests. The permeability of clays is normally determined by the use of a falling head permeameter. The difference between the two tests is that in the former, the hydraulic gradient of the specimen is kept constant, while in the latter, the head is allowed to decrease as the water permeates the specimen. Evaluations of soil permeability are obtained from time readings required for a measured volume of water to pass through the soil as shown in Figure 7-14.
Commentary	Permeability is one of the major parameters used in selecting soils for various types of construction. In some cases it may be desirable to place a high-permeability material immediately under a pavement surface to facilitate the removal of water seeping into the base or sub-base courses. In other cases, such as retention pond dikes, it may be detrimental to use high-permeability materials. Permeability also significantly influences the choice of backfill materials. Both test procedures determine permeability of soils under specified conditions. The geotechnical engineer must establish which test conditions are representative of the problem under consideration. As with all other laboratory tests, the geotechnical engineer has to be aware of the limitations of this test. The process is sensitive to the presence of air or gases in the voids and in the permeant or water. Prior to the test, distilled, de-aired water should be run through the specimen to remove as much of the air and gas as practical. It is a good practice to use de-aired or distilled water at temperatures slightly higher than the temperature of the specimen. As the water permeates through the voids and cools, it will have a tendency to dissolve the air and some of the gases, thus removing them during this process. The result will be a more representative, albeit idealized, permeability value. The type of permeameter, (i.e., flexible wall - ASTM D 5084 -versus rigid - ASTM D 2434 and AASHTO T215) may also affect the final results. For testing of fine-grained, low-permeability soils, the use of flexible-wall permeameters is recommended which are essentially very similar to the triaxial test apparatus (see Figure 7-15). When rigid wall units are used, the permeant may find a route at the sample-permeameter interface, thus it may drain through that interface rather than travel through the specimen. This will produce erroneous results. It should be emphasized that permeability is sensitive to viscosity. In computing permeability, the correction factors for viscosity and temperatures should be applied. During testing, the temperature of the permeant and the laboratory should be kept constant. Laboratory permeability tests produce reliable results under ideal conditions. Permeability of fine-grained soils can also be computed from one-dimensional consolidation test results, although these results are not as accurate as direct k measurements (e.g., Tavenas, et al. 1983).



Computation of Coefficient of Permeability, k

For *Constant Head Test* (Figure a):

$$k = \frac{QL}{hAt} \tag{7-9}$$

where Q = total discharge volume, m^3, in time, t (seconds), and
A = cross-sectional area of soil sample, m^2

For *Falling Head Test* (Figure b)

$$k = 2.3\frac{aL}{A\Delta t}\log_{10}\frac{h_1}{h_2} \tag{7-10}$$

where a = area of standpipe,
A,L = soil sample area and length,
Δt = time for standpipe head to decrease from h_1 to h_2.

Figure 7-14. Permeability Test Schematics: (a) Constant Head Device; (b) Falling Head Test.

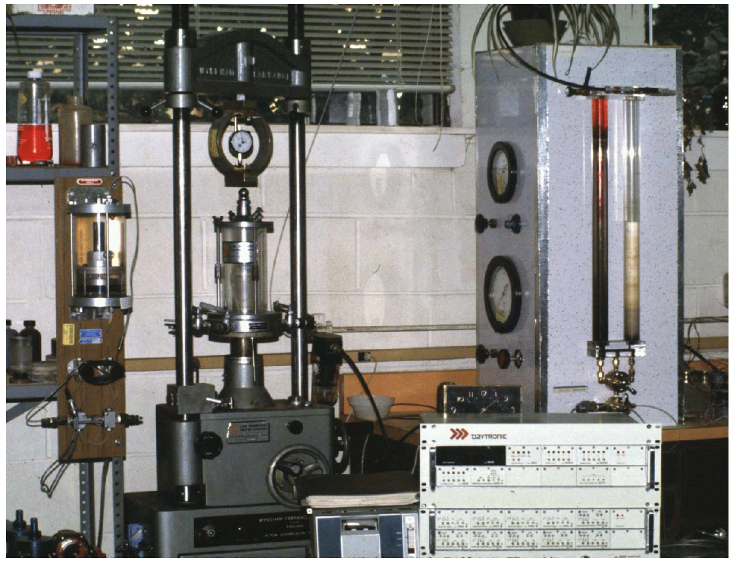
 

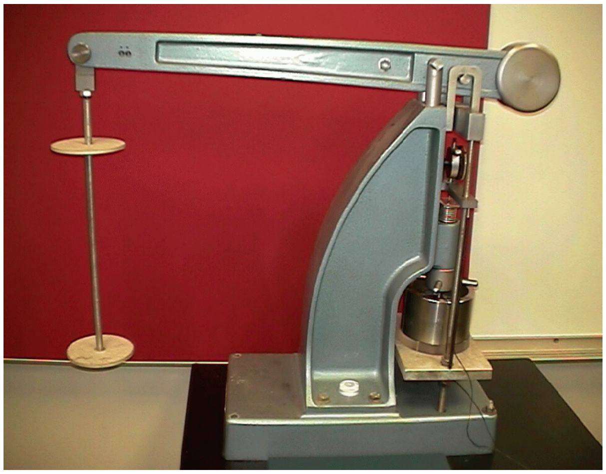
Figure 7-15. Permeameter Equipment: (a) Flexible-Walled Permeameter Cell; (b) Permeability Station with Automatic Volume Change Device (left) and Backpressure Panel Board (right side).

7.1.10 Consolidation

The one-dimensional *consolidation* test (or *oedometer* test) provides one of the most useful and reliable laboratory measurements for soil behavior. The test determines the compressibility parameters (C_c, C_s, C_r), stiffness in terms of constrained modulus ($D' = 1/m_v$), preconsolidation stress (σ_p'), rate of consolidation (c_v), creep rate (C_α), and approximate value of permeability (k).

One-Dimensional Consolidation	
AASHTO ASTM	T 216 D 2435
Purpose	Determination of preconsolidation stress, compression characteristics, creep, stiffness, and flow rate properties of soils under loading.
Procedure	The test is performed using a small 50-mm to 75-mm diameter thin specimen (25 mm thick) taken from an undisturbed sample. Selection of representative samples for testing is critical. Prepared samples are placed in a rigid-walled loading device called a consolidometer or oedometer (see Figure 7-16). All loads and recorded deformations are in the vertical direction. The specimen is subjected to incremental loads, which are doubled after each equilibrium phase is reached (after t_p corresponding to the end of primary consolidation). Tradition would use a 24-hour increment per load, although this is conservative. Alternatively, specimens can be loaded continuously with monitoring by load cells and porewater pressure transducers. Generally, it is desirable to perform an unload-reload cycle during the test, with the unloading initiated at a loading increment along the virgin portion of the consolidation curve. The unload-reload cycle provides a more reliable estimate of the recompression characteristics of the soil.

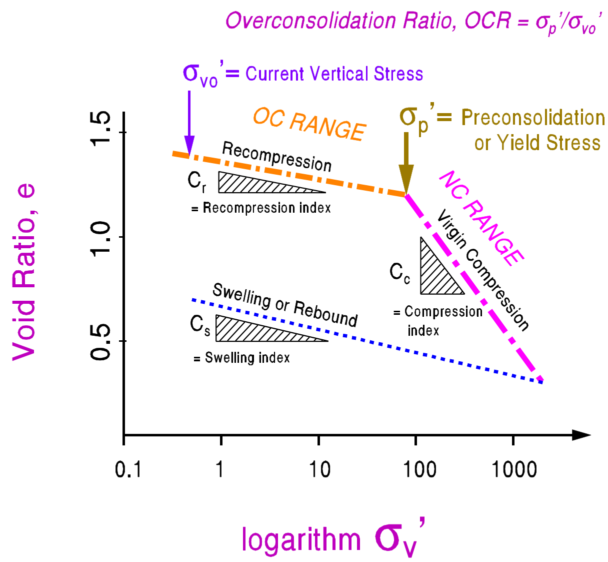
One-Dimensional Consolidation	
AASHTO ASTM	T 216 D 2435
Commentary	When saturated soil masses are subjected to incremental loads, they undergo various degrees of dimensional change. Initially, the incremental load is resisted and carried by the liquid phase of the soil, which develops excess porewater pressures (Δu) in the soil voids. Depending on the permeability and the availability of drainage layer(s) in contact with the soil, the liquids in the voids begin draining and continue to do so until the Δu is dissipated. As the hydrostatic pressure decreases, a proportional amount of the incremental load is transferred to the solid portion of the soil. When the excess hydrostatic pressure reaches zero, all of the new load is carried by the soil's solids. This process is called primary consolidation. In granular, high-permeability soils, this transfer takes place very quickly (since water can drain fast). In clays and low-permeability soils, primary consolidation takes a longer time, which can affect the long-term performance of structures supported by these soils. Time rate is expressed by the coefficient of consolidation (c_v). The one-dimensional consolidation test is most commonly used for the determination of consolidation properties of soils. This test method assumes that dimensional change due to consolidation will take place in the vertical direction. This assumption is generally acceptable for stiff or medium, confined cohesive soils, but it is not true for soft soils or for soils that are not confined (i.e., bridge approaches). The data and the analysis produced from this test have proved to be reasonably reliable. Results of one-dimensional consolidation tests can be presented in a variety of ways, the two most common include: (1) e-log σ_v' graphs whereby the compression indices (C_r, C_c, C_s) are determined as the slopes of Δe vs. Δlog σ_v' for the recompression, virgin compression, and swelling lines, respectively; or (2) $\Delta\sigma_v'$ vs. $\Delta\varepsilon_v$ graphs where the slope is equal to the constrained modulus (D'). Most importantly, the consolidation test provides the magnitude of the preconsolidation stress ($\sigma_{vmax}' = \sigma_p' = P_c'$) of the natural deposit, as shown in Figure 7-16c. The effective preconsolidation represents the recorded past stress history of the soil that may have undergone erosion, desiccation, seismic events, groundwater fluctuations, and other mechanisms of overconsolidation, as discussed further in Chapter 9. In many clays, the primary consolidation is typically followed by secondary compression or long-term creep and represented by the parameter C_α. In thick clay deposits, the magnitude of secondary compression may be substantial. For soils known for their tendency to have significant secondary compression particularly under heavy incremental loads, it may be necessary to predict the long-term effects of secondary compression. In that case, each incremental of the test load is left in place until such time that the time-settlement curve plotted for that load becomes asymptotic to a horizontal line. Heavy organic clays also require longer loading periods. The time-settlement curves produced by heavy organic soils may not clearly show the end of the primary consolidation. In those cases, it may be necessary to monitor the pore pressures of the soil to determine the end of the primary stage. It should be noted that the magnitude of secondary, long term, compression of highly (20% or more) organic soils may be as large or larger than the primary consolidation. Secondary compression in these soils takes place as a result of the continuing compression of organic fibers. The substantial dissipation of the excess hydrostatic pressures during the test does not signal the end of significant compression; expulsion of absorbed water with associated compression from the body of the fiber itself may continue for a long period of time.



(a)

(b)

(c)



(d)

Figure 7-16. One Dimensional Consolidation Devices and Results:
(a) Wykeham Farrance oedometer with moment loading arm; (b) Pneumatic consolidometers (Anteus); (c) Rowe cell using hydraulic loading system (GeoComp Corp); and (d) Idealized graphs of e-log $F_v r$ for obtaining parameters.

Swell Potential of Clays	
AASHTO ASTM Test	T 256 D 4546
Purpose	To estimate the swell potential of (expansive) soils
Procedure	The swell test is typically performed in a consolidation apparatus. The swell potential is determined by observing the swell of a laterally-confined specimen when it is surcharged and flooded. Alternatively, after the specimen is inundated, the height of the specimen is kept constant by adding loads. The vertical stress necessary to maintain zero volume change is the swelling pressure.
Commentary	Swelling is a characteristic reaction of some clays to saturation. The potential for swell depends on the mineralogical composition. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has none to moderate swell characteristics, and kaolinite exhibits almost none. The percentage of volumetric swell of a soil depends on the amount of clay, its relative density, the compaction moisture and density, permeability, location of the water table, presence of vegetation and trees, and overburden stress. Swelling of foundation, embankment, or pavement soils result in serious and costly damage to structures above them. It is therefore important to estimate the swell potential of these soils. The one dimensional swell potential test is used to estimate the percent swell and swelling pressures developed by the swelling soils. This test can be performed on undisturbed, remolded, or compacted specimens. If the soil structure is not confined (i.e. bridge abutment) such that swelling may occur laterally and vertically, triaxial tests can be used to determine three dimensional swell characteristics.

Collapse Potential of Soils	
AASHTO ASTM	- D 5333
Purpose	To estimate the collapse potential of soils
Procedure	The collapse potential of suspected soils is determined by placing an undisturbed, compacted or remolded specimen in the consolidometer ring and in a loading device at their natural moisture content. A load is applied and the soil is saturated to measure the magnitude of the vertical displacement.
Commentary	Loess or loess type soils is predominantly composed of silts, and contain 3% to 5% clay. Loess deposits are wind blown formations. Loess type deposits have similar composition and they are formed as a result of the removal of organics by decomposition or the leaching of certain minerals (calcium carbonate). In both cases disturbed samples obtained from these deposits will be classified as silt. When dry or at low moisture content the in situ material gives the appearance of a stable silt deposit. At high moisture contents these soils collapse and undergo sudden changes in volume. Loess, unlike other non-cohesive soils, will stand on almost a vertical slope until saturated. It has a low relative density, a low unit weight and a high void ratio. Structures founded on such soils, upon saturation, may be seriously damaged from the collapse of the foundation soils. The collapse during wetting occurs due to the destruction of clay binding which provide the original strength of these soils. It is conceivable that remolding and compacting may also destroy the original structure.

7.2 QUALITY ASSURANCE FOR LABORATORY TESTING

The ability to maintain the quality of samples is largely dependent on the quality assurance program followed by the field and laboratory staff. Significant changes in the material properties may take place as a result of improper storage, transportation and handling of samples resulting in misleading test, and therefore design, results.

7.2.1 Storage

Undisturbed soil samples should be transported and stored such that their structure and their moisture content are maintained as close to their natural conditions as practicable (AASHTO T 207, ASTM D 4220 and D 5079). Specimens stored in special containers should not be placed, even temporarily, in direct sunlight. Undisturbed soil samples should be stored in an upright position with the top side up.

Long term storage of soil samples should be in temperature-controlled environments. The temperature control requirements may vary from subfreezing to ambient and above, depending on the environment of the parent formation. The relative humidity for soil storage normally should be maintained at 90 percent or higher.

Storage of soil samples long term in sampling tubes is not recommended. During long term storage, the sample tubes may experience corrosion. This accompanied by the adhesion of the soil to the tube may develop such resistance to extrusion that some soils may experience internal failures during the extrusion. Often these failures can not be seen by the naked eye; only x-ray radiography (ASTM D 4452) will reveal the presence of such conditions. If these samples are tested as undisturbed specimens the results may be misleading.

Long term storage of samples, even under the best conditions, may cause changes in the characteristics of the of samples. Research has shown that soil samples stored more than fifteen or more days undergo substantial changes in strength characteristics. Soil samples stored for long periods of time provide poor quality specimens, and often unreliable results. Stress relaxation, temperature changes and prolonged exposure to the environment in these cases may have serious impacts on the sample characteristics.

7.2.2 Sample Handling

Careless handling of undisturbed soil samples may cause major disturbances with serious design and construction consequences. Samples should always be handled by experienced personnel in a manner that, during preparation, the sample maintains its structural integrity and its moisture condition. Saws and knives used to trim soils should be clean and sharp. Preparation time should be kept to a minimum, especially where the maintenance of the moisture content is critical. During preparation, specimens should not be exposed to direct sun or precipitation. If samples are dropped, in or out of containers, it is reasonable to expect that they will be disturbed. They should not be used for critical tests (i.e. elastic moduli, triaxial) requiring undisturbed specimens.

7.2.3 Specimen Selection

The selection of representative specimens for testing is one of the most important aspects of sampling and testing procedures. Selected specimens must be representative of the formation being investigated. Seldom one finds a uniform homogeneous deposit or formation.

The senior laboratory technician, the geologist and/or the geotechnical engineer need to study the drilling logs, understand the geology of the site, and visually examine the samples before selecting the test specimens. Samples should be selected on the basis of their color, physical appearance, and structural features. Specimens should be selected to represent all types of materials present at the site, not just the worst or the best. Samples with discontinuities and intrusions may cause premature failures in the laboratory. They, however, would not cause such failures in situ. Such failures should be noted but not selected as representative of the deposit of the formation.

There is no single set of rules that can be applied to all specimen selection. In selecting the proper specimens, the geotechnical engineer, the geologist, and senior laboratory technician must apply their knowledge and experience with the geologic setting, materials, and project requirements.

7.2.4 Equipment Calibration

All laboratory equipment should be periodically checked to verify that they meet the tolerances as established by the AASHTO and ASTM test procedures. Sieves, ovens, compaction molds, triaxial and permeability cells should be periodically examined to assure that they meet the opening size, temperature and volumetric tolerances. Compression or tension testing equipment, including proving rings and transducers should be checked quarterly and calibrated at least once a year using U.S. Bureau of Standards certified equipment. Scales, particularly electronic or reflecting mirror types, should be checked at least once every day to assure that they are leveled and in proper adjustment. Electronic equipment and software should also be checked periodically (i.e. quarterly) to assure that all is well.

7.2.5 Pitfalls

Sampling and testing of soils are the most important and fundamental steps in the design and construction of all types of structures. Omissions or errors introduced in these steps, if gone undetected, will be carried through the process of design and construction resulting often in costly or possibly unsafe facilities. Table 7-4 lists topics that should be considered for proper handling of samples, preparation, and laboratory test procedures. Table 7-4 should in no way be construed as being a complete list of possible important items in the handling or testing of soil specimens; there are many more. These are just some of the more common ones.

TABLE 7-4.

COMMON SENSE GUIDELINES FOR LABORATORY TESTING OF SOILS

1. Protect samples to prevent moisture loss and structural disturbance.
2. Carefully handle samples during extrusion of samples; samples must be extruded properly and supported upon their exit from the tube.
3. Avoid long term storage of soil samples in Shelby tubes.
4. Properly number and identify samples.
5. Store samples in properly controlled environments.
6. Visually examine and identify soil samples after removal of smear from the sample surface.
7. Use pocket penetrometer or miniature vane only for an indication of strength.
8. Carefully select "representative" specimens for testing.
9. Have a sufficient number of samples to select from.
10 Always consult the field logs for proper selection of specimens.
11. Recognize disturbances caused by sampling, the presence of cuttings, drilling mud or other foreign matter and avoid during selection of specimens.
12. Do not depend solely on the visual identification of soils for classification.
13. Always perform organic content tests when classifying soils as peat or organic. Visual classifications of organic soils may be very misleading.
14. Do not dry soils in overheated or underheated ovens.
15. Discard old worn-out equipment; old screens for example, particularly fine (<No. 40) mesh ones need to be inspected and replaced often, worn compaction mold or compaction hammers (an error in the volume of a compaction mold is amplified 30x when translated to unit volume) should be checked and replaced if needed.
16. Performance of Atterberg Limits requires carefully adjusted drop height of the Liquid Limit machine and proper rolling of Plastic Limit specimens.
17. Do not use of tap water for tests where distilled water is specified.
18. Properly cure stabilization test specimens.
19. Never assume that all samples are saturated as received.
20. Saturation must be performed using properly staged back pressures.
21. Use properly fitted o-rings, membranes etc. in triaxial or permeability tests.
22. Evenly trim the ends and sides of undisturbed samples.
23. Be careful to identify slickensides and natural fissures. Report slickensides and natural fissures.
24. Also do not mistakenly identify failures due to slickensides as shear failures.
25. Do not use unconfined compression test results (stress-strain curves) to determine elastic moduli.
26. Incremental loading of consolidation tests should only be performed after the completion of each primary stage.
27. Use proper loading rate for strength tests.
28. Do not guesstimate e-log p curves from accelerated, incomplete consolidation tests.
29. Avoid "Reconstructing" soil specimens, disturbed by sampling or handling, for undisturbed testing.
30. Correctly label laboratory test specimens.
31. Do not take shortcuts: using non-standard equipment or non-standard test procedures.
32. Periodically calibrate all testing equipment and maintain calibration records.
33. Always test a sufficient number of samples to obtain representative results in variable material.

7.3 SELECTION AND ASSIGNMENT OF TESTS

Certain considerations regarding laboratory testing, such as when, how much, and what type, can only be decided by an experienced geotechnical engineer. The following minimal criteria should be considered while determining the scope of the laboratory testing program:

- Project type (bridge, embankment, rehabilitation, buildings, etc.)
- Size of the project
- Loads to be imposed on the foundation soils
- Types of loads (i.e., static, dynamic, etc.)
- Critical tolerances for the project (e.g., settlement limitations)
- Vertical and horizontal variations in the soil profile as determined from boring logs and visual identification of soil types in the laboratory
- Known or suspected peculiarities of soils at the project location (i.e., swelling soils, collapsible soils, organics, etc.)
- Presence of visually observed intrusions, slickensides, fissures, concretions, etc.

The selection of tests should be considered preliminary until the geotechnical engineer is satisfied that the test results are sufficient to develop reliable soil profiles and provide the soil parameters needed for design. Laboratory visual identification of all soil samples extracted from the borings should be performed. The soil groups with similar engineering properties should be classified using the Unified Soil Classification System (ASTM D2487) [preferred for geotechnical practice] or the AASHTO system (M145) with classification tests performed on selected samples as requested by the engineer. Moisture content analysis should be performed on all cohesive samples and, if possible, on all samples. The geotechnical engineer should then determine the appropriate tests required to obtain the design parameters or validate design parameters obtained from field tests for each soil layer. A summary of information needs and testing considerations for a range of applications is provided in Table 7-5 (from GEC 5). Additional guidance on the selection of soil and rock properties is contained in the FHWA "Soil and Foundations Workshop" reference manual.

Table 7-5. Summary of information needs and testing considerations for a range of highway applications.
(from FHWA Geotechnical Engineering Circular 5 – Soil and Rock Properties, 2002)

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Shallow Foundations	• bearing capacity • settlement (magnitude & rate) • shrink/swell of foundation soils (natural soils or embankment fill) • chemical compatibility of soil and concrete • frost heave • scour (for water crossings) • extreme loading	• subsurface profile (soil, groundwater, rock) • shear strength parameters • compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) • frost depth • stress history (present and past vertical effective stresses) • chemical composition of soil • depth of seasonal moisture change • unit weights • geologic mapping including orientation and characteristics of rock discontinuities	• vane shear test • SPT (granular soils) • CPT • dilatometer • rock coring (RQD) • nuclear density • plate load testing • geophysical testing	• 1-D oedometer tests • direct shear tests • triaxial tests • grain size distribution • Atterberg Limits • pH, resistivity tests • moisture content • unit weight • organic content • collapse/swell potential tests • rock uniaxial compression test and intact rock modulus • point load strength test
Driven Pile Foundations	• pile end-bearing • pile skin friction • settlement • down-drag on pile • lateral earth pressures • chemical compatibility of soil and pile • driveability • presence of boulders/ very hard layers • scour (for water crossings) • vibration/heave damage to nearby structures • extreme loading	• subsurface profile (soil, ground water, rock) • shear strength parameters • horizontal earth pressure coefficients • interface friction parameters (soil and pile) • compressibility parameters • chemical composition of soil/rock • unit weights • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • pile load test • CPT • vane shear test • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• triaxial tests • interface friction tests • grain size distribution • 1-D oedometer tests • pH, resistivity tests • Atterberg Limits • organic content • moisture content • unit weight • collapse/swell potential tests • slake durability • rock uniaxial compression test and intact rock modulus • point load strength test

Table 7-5. Summary of information needs and testing considerations for a range of highway applications (continued).

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Drilled Shaft Foundations	• shaft end bearing • shaft skin friction • constructability • down-drag on shaft • quality of rock socket • lateral earth pressures • settlement (magnitude & rate) • groundwater seepage/ dewatering • presence of boulders/ very hard layers • scour (for water crossings) • extreme loading	• subsurface profile (soil, ground water, rock) • shear strength parameters • interface shear strength friction parameters (soil and shaft) • compressibility parameters • horizontal earth pressure coefficients • chemical composition of soil/rock • unit weights • permeability of water-bearing soils • presence of artesian conditions • presence of shrink/swell soils (limits skin friction) • geologic mapping including orientation and characteristics of rock discontinuities • degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales)	• technique shaft • shaft load test • vane shear test • CPT • SPT (granular soils) • dilatometer • piezometers • rock coring (RQD) • geophysical testing	• 1-D oedometer • triaxial tests • grain size distribution • interface friction tests • pH, resistivity tests • permeability tests • Atterberg Limits • moisture content • unit weight • organic content • collapse/swell potential tests • rock uniaxial compression test and intact rock modulus • point load strength test • slake durability
Embankments and Embankment Foundations	• settlement (magnitude & rate) • bearing capacity • slope stability • lateral pressure • internal stability • borrow source evaluation (available quantity and quality of borrow soil) • required reinforcement	• subsurface profile (soil, ground water, rock) • compressibility parameters • shear strength parameters • unit weights • time-rate consolidation parameters • horizontal earth pressure coefficients • interface friction parameters • pullout resistance • geologic mapping including orientation and characteristics of rock discontinuities • shrink/swell/degradation of soil and rock fill	• nuclear density • plate load test • test fill • CPT • SPT (granular soils) • dilatometer • vane shear • rock coring (RQD) • geophysical testing	• 1-D Oedometer • triaxial tests • direct shear tests • grain size distribution • Atterberg Limits • organic content • moisture-density relationship • hydraulic conductivity • geosynthetic/soil testing • shrink/swell • slake durability • unit weight

Table 7-5. Summary of information needs and testing considerations for a range of highway applications (continued).

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Excavations and Cut Slopes	● slope stability ● bottom heave ● liquefaction ● dewatering ● lateral pressure ● soil softening/progressive failure ● pore pressures	● subsurface profile (soil, ground water, rock) ● shrink/swell properties ● unit weights ● hydraulic conductivity ● time-rate consolidation parameters ● shear strength of soil and rock (including discontinuities) ● geologic mapping including orientation and characteristics of rock discontinuities	● test cut to evaluate stand-up time ● piezometers ● CPT ● SPT (granular soils) ● vane shear ● dilatometer ● rock coring (RQD) ● in situ rock direct shear test ● geophysical testing	● hydraulic conductivity ● grain size distribution ● Atterberg Limits ● triaxial tests ● direct shear tests ● moisture content ● slake durability ● rock uniaxial compression test & intact rock modulus ● point load strength test
Fill Walls/ Reinforced Soil Slopes	● internal stability ● external stability ● settlement ● horizontal deformation ● lateral earth pressures ● bearing capacity ● chemical compatibility with soil and wall materials ● pore pressures behind wall ● borrow source evaluation (available quantity and quality of borrow soil)	● subsurface profile (soil, ground water, rock) ● horizontal earth pressure coefficients ● interface shear strengths ● foundation soil/wall fill shear strengths ● compressibility parameters (including consolidation, shrink/swell potential, and elastic modulus) ● chemical composition of fill/ foundation soils ● hydraulic conductivity of soils behind wall ● time-rate consolidation parameters ● geologic mapping including orientation and characteristics of rock discontinuities	● SPT (granular soils) ● CPT ● dilatometer ● vane shear ● piezometers ● test fill ● nuclear density ● pullout test (MSEW/RSS) ● rock coring (RQD) ● geophysical testing	● 1-D Oedometer ● triaxial tests ● direct shear tests ● grain size distribution ● Atterberg Limits ● pH, resistivity tests ● moisture content ● organic content ● moisture-density relationships ● hydraulic conductivity
Cut Walls	● internal stability ● external stability ● excavation stability ● dewatering ● chemical compatibility of wall/soil ● lateral earth pressure ● down-drag on wall ● pore pressures behind wall ● obstructions in retained soil	● subsurface profile (soil, ground water, rock) ● shear strength of soil ● horizontal earth pressure coefficients ● interface shear strength (soil and reinforcement) ● hydraulic conductivity of soil ● geologic mapping including orientation and characteristics of rock discontinuities	● test cut to evaluate stand-up time ● well pumping tests ● piezometers ● SPT (granular soils) ● CPT ● vane shear ● dilatometer ● pullout tests (anchors, nails) ● geophysical testing	● triaxial tests ● direct shear ● grain size distribution ● Atterberg Limits ● pH, resistivity tests ● organic content ● hydraulic conductivity ● moisture content ● unit weight

CHAPTER 8.0

LABORATORY TESTING FOR ROCKS

8.1 INTRODUCTION

Laboratory rock testing is performed to determine the strength and elastic properties of intact specimens and the potential for degradation and disintegration of the rock material. The derived parameters are used in part for the design of rock fills, cut slopes, shallow and deep foundations, tunnels, and the assessment of shore protection materials (rip-rap). Deformation and strength properties of intact specimens aid in evaluating the larger-scale rock mass that is significantly controlled by joints, fissures, and discontinuity features (spacing, roughness, orientation, infilling), water pressures, and ambient geostatic stress state.

8.2 LABORATORY TESTS

Common laboratory tests for intact rocks include measurements of strength (point load index, compressive strength, Brazilian test, direct shear), stiffness (ultrasonics, elastic modulus), and durability (slaking, abrasion). Table 8-1 gives a summary list of laboratory rock tests and procedures by ASTM. Brief sections discuss the common tests (denoted with an asterisk*) useful for a standard highway project involving construction in rock.

8.2.1 Strength Tests

The laboratory determination of intact rock strength is accomplished by the following tests: point load index, unconfined compression, triaxial compression, Brazilian test, and direct shear. The uniaxial (or unconfined) compression test provides the general reference value, having a respective analogy with standard tests on concrete cylinders. The uniaxial compressive strength ($q_u = F_u$) is obtained by compressing a trimmed cylindrical specimen in the longitudinal direction and taking the maximum measured force divided by the cross-sectional area. The point load index serves as a surrogate for the UCS and is a simpler test in that irregular pieces of rock core can be used. A direct tensile test requires special end preparation that is difficult for most commercial labs, therefore tensile strength is more often evaluated by compression loading of cylindrical specimens across their diameter (known as the Brazilian test). Direct shear tests are used to investigate frictional characteristics along rock discontinuity features.

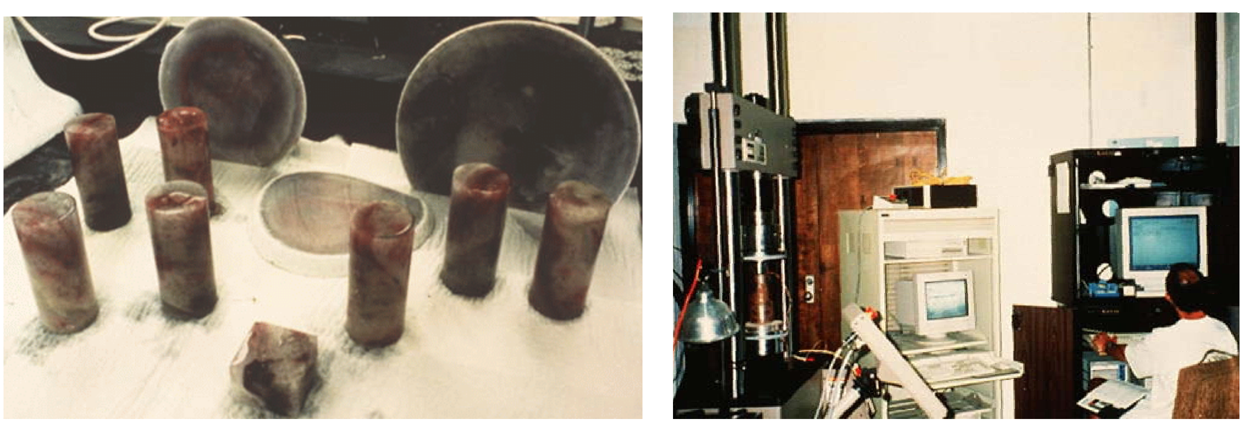


Figure 8-1: (a) Intact Rock Specimens for Laboratory Testing; (b) Compressive Strength Testing.

TABLE 8-1.

STANDARDS & PROCEDURES FOR LABORATORY TESTING OF INTACT ROCK

Test Category	*Name of Test*	*Test Designation*	
		AASHTO	*ASTM*
Point Load Strength	Method for determining point load index (I_s)	-	D 5731*
Compressive Strength	Compressive strength ($q_u = F_u$) of core in unconfined compression (uniaxial compression test)	-	D 2938*
	Triaxial compressive strength without pore pressure	T 226	D 2664
Creep Tests	Creep-cylindrical hard rock core in uniaxial compression	-	D 4341
	Creep-cylindrical soft rock core in uniaxial compression	-	D 4405
	Creep-cylindrical hard rock core, in triaxial compression	-	D 4406
Tensile Strength	Direct tensile strength of intact rock core specimens	-	D 3936
	Splitting tensile strength of intact core (Brazilian test)	-	D 3967*
Direct Shear	Laboratory direct shear strength tests - rock specimens, under constant normal stress	-	D 5607*
Permeability	Permeability of rocks by flowing air	-	D 4525
Durability	Slake durability of shales and similar weak rocks	-	D 4644*
	Rock slab testing for riprap soundness, using sodium/magnesium sulfate	-	D 5240*
	Rock-durability for erosion control under freezing/thawing	-	D 5312*
	Rock-durability for erosion control under wetting/drying	-	D 5313
Deformation and Stiffness	Elastic moduli of intact rock core in uniaxial compression	-	D 3148*
	Elastic moduli of intact rock core in triaxial compression	-	D 5407
	Pulse velocities and ultrasonic elastic constants in rock	-	D 2845*
Specimen Preparation	Rock core specimen preparation	-	D 4543
	Rock slab preparation for durability testing	-	D 5121

Note: *Routine rock test procedure described in this manual

Point Load Index (Strength)	
ASTM	D 5731
Purpose	To determine strength classification of rock materials through an index test.
Procedure	Rock specimens in the form of core (diametral and axial), cut blocks or irregular lumps are broken by application of concentrated load through a pair of spherically truncated, conical platens. The distance between specimen-platen contact points is recorded. The load is steadily increased, and the failure load is recorded. There is little sample preparation. However, specimens should conform to the size and shape requirements as specified by ASTM. In general, for the diametral test, core specimens with a length-to-diameter ratio of 1.0 are adequate while for the axial test core specimens with length-to-diameter ratio of 0.3 to 1.0 are suitable. Specimens for the block and the irregular lump test should have a length of 50±35 mm and a depth/width ratio between 0.3 and 1.0 (preferably close to 1.0). The test specimens are typically tested at their natural water content. Size corrections are applied to obtain the point load strength index, $I_{s(50)}$, of a rock specimen. A strength anisotropy index, $I_{a(50)}$, is determined when $I_{s(50)}$ values are measured perpendicular and parallel to planes of weakness.
Commentary	The test can be performed in the field with portable equipment or in the laboratory (Figure 8-1). The point load index is used to evaluate the uniaxial compressive strength (F_u). On the average, F_u . 25 $I_{s(50)}$. However, the coefficient term can vary from 15 to 50 depending upon the specific rock formation, especially for anisotropic rocks. The test should not be used for weak rocks where F_u < 25 MPa.

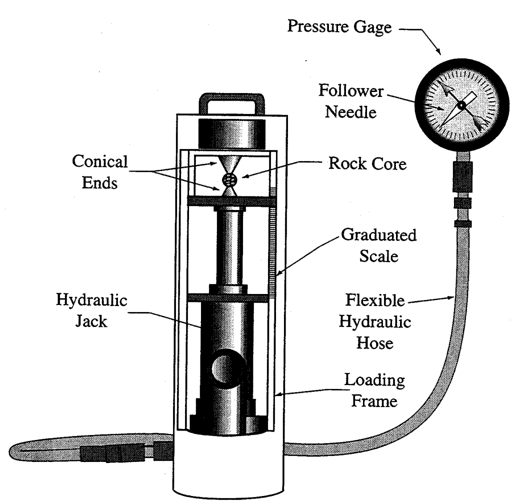
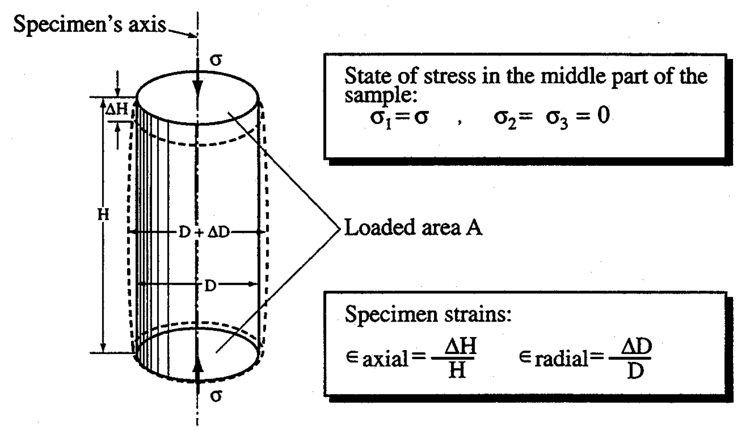


Figure 8-1: Point Load Test Apparatus. (Adopted from Roctest)

<table>
<tr><th colspan="2">Uniaxial Compression Test</th></tr>
<tr><td>AASHTO
ASTM</td><td>-
D 2938</td></tr>
<tr><td>Purpose</td><td>To determine the uniaxial compressive strength of rock ($q_u = F_u = F_C$).</td></tr>
<tr><td>Procedure</td><td>In this test, cylindrical rock specimens are tested in compression without lateral confinement. The test procedure is similar to the unconfined compression test for soils and concrete. The test specimen should be a rock cylinder of length-to-width ratio (H/D) in the range of 2 to 2.5 with flat, smooth, and parallel ends cut perpendicular to the cylinder axis. Originally, specimen diameters of NX size were used (D = 2⅛ in. = 44 mm), yet now the standard size is NQ core (D = 1⅞ in. = 47.6 mm).

(a)


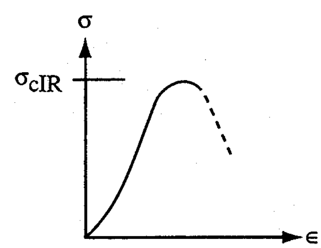
(b)
$\sigma_u =$


Figure 8-2: Uniaxial Compression Test on Rock with (a) Definitions of stress conditions and strains, (b) Derived stress-strain curve with peak stress corresponding to the uniaxial compressive strength ($q_u = F_u$)</td></tr>
<tr><td>Commentary</td><td>The uniaxial compression test is most direct means of determining rock strength. The results are influenced by the moisture content of the specimens, and thus should be noted. The rate of loading and the condition of the two ends of the rock will also affect the final results. Ends should be planar and parallel per ASTM D 4543. The rate of loading should be constant as per the ASTM test procedure. Inclined fissures, intrusions, and other anomalies will often cause premature failures on those planes. These should be noted so that, where appropriate, other tests such as triaxial or direct shear tests can be required.</td></tr>
</table>

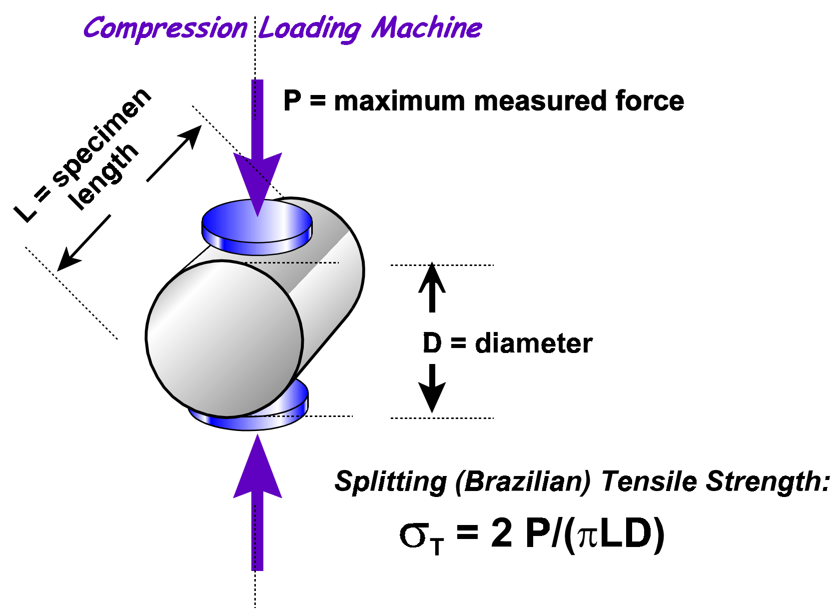
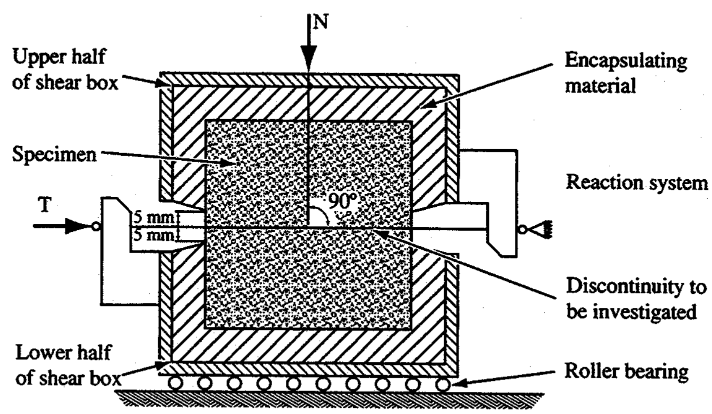
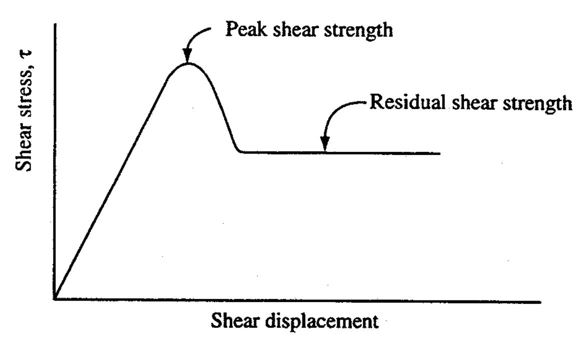
<table>
<tr><th colspan="2">Splitting Tensile (Brazilian) Test for Intact Rocks</th></tr>
<tr><td>AASHTO
ASTM</td><td>None
D 3967</td></tr>
<tr><td>Purpose</td><td>To evaluate the (indirect) tensile shear of intact rock core, F_T.</td></tr>
<tr><td>Procedures</td><td>Core specimens with length-to-diameter ratios (L/D) of between 2 to 2.5 are placed in a compression loading machine with the load platens situated diametrically across the specimen. The maximum load (P) to fracture the specimen is recorded and used to calculate the split tensile strength.


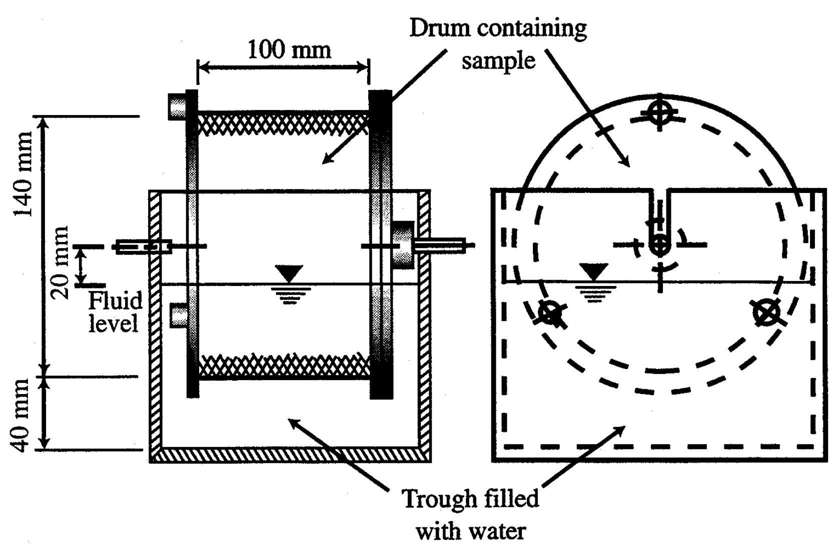
Figure 8-3. Setup for Brazilian Tensile Test in Standard Loading Machine.</td></tr>
<tr><td>Commentary</td><td>The Brazilian or split-tensile strength (F_T) is significantly more convenient and practicable for routine measurements than the direct tensile strength test (T_0). The test gives very similar results to those from direct tension (Jaeger & Cook, 1976). It is a more fundamental strength measurement of the rock material, as this corresponds to a more likely failure mode in many situations than compression. Also, note that the point load index is actually a type of Brazilian tensile strength, that is correlated back to compressive strength. Additional details on tensile strengths of rocks is given in Chapter 10.</td></tr>
</table>

Direct Shear Strength of Rock	
AASHTO ASTM	- D 5607
Purpose	To determine the shear strength characteristics of rock along a plane of weakness.
Procedure	The laboratory test equipment is shown below in Figure 8-4. The specimen is placed in the lower half of the shear box and encapsulated in either synthetic resin or mortar. The specimen must be positioned so that the line of action of the shear force lies in the plane of the discontinuity to be investigated, and the normal force acts perpendicular to this surface. Once the encapsulating material has hardened, the specimen is mounted in the upper half of the shear box in the same manner. A strip approximately 5 mm wide above and below the shear surface must be kept free of encapsulating material. The test is then carried out by applying a horizontal shear force T under a constant normal load, N.  (a)  (b) **Figure 8-4: (a) General Set-up for Direct Shear Strength Testing of Rock** (Wittke, 1990) **(b) Derived Shear Stress vs. Shear Displacement Curve.** (ASTM D 5607, 1995)

	(Direct Shear Testing of Rock - Continued)
Commentary	Determination of shear strength of rock specimens is an important aspect in the design of structures such as rock slopes, foundations and other purposes. Pervasive discontinuities (joints, bedding planes, shear zones, fault zones, schistosity) in a rock mass, and genesis, crystallography, texture, fabric, and other factors can cause the rock mass to behave as an anisotropic and heterogeneous discontinuum. Therefore, the precise prediction of rock mass behavior is difficult. For nonplanar joints or discontinuities, shear strength is derived from a combination base material friction and overriding of asperities (dilatancy), shearing or breaking of the asperities, rotations at or wedging of the asperities (Patton, 1966). Sliding on and shearing of the asperities can occur simultaneously. When the normal force is not sufficient to restrain dilation, the shear mechanism consists of the overriding of the asperities. When the normal load is large enough to completely restrain dilation, the shear mechanism consists of the shearing off of the asperities. Using this test method to determine the shear strength of intact rock may generate overturning moments that induce premature tensile breaking. Thus, the specimen would fail in tension first rather than in shear. Rock shear strength is influenced by the overburden stresses; therefore, the larger the overburden stress, the larger the shear strength. In some cases, it may be desirable to conduct tests in-situ rather than in the laboratory to more accurately determine a representative shear strength of the rock mass, particularly when design is controlled by discontinuities filled with very weak material.

8.2.2 Durability

The evaluation of rock durability becomes an issue when the materials are to be subjected to the natural elements, seasonal weather, and repeated cycles of temperature (e.g., flowing water, wetting and drying, wave action, freeze and thaw, etc.) in its proposed use. Tests to measure durability depend on the type of rock, on its use in construction, and on the elements to which the rock will be subjected. The basis for durability tests are empirical and the results produced are an indication of the rock’s resistance to natural processes; the rock’s behavior in actual use may vary greatly from the test results. These tests, however, provide reasonably reliable tools for quality control. The suitability of various types of rock for different uses should, in addition to these test results, depend on their performance in previous applications. An example of the use of rock durability tests is in the evaluation of shale in rock fill embankments.

Slake Durability	
AASHTO ASTM	- D 4644 (for shales and similar weak rocks)
Purpose	To determine the durability of shale or other weak or soft rocks subjected to cycles of wetting and drying.
Procedure	In this test dried fragments of rock of known weight are placed in a drum fabricated with 2.0 mm square mesh wire cloth. Figure 8-4 shows a schematic of the test apparatus. The drum is rotated in a horizontal position along its longitudinal axis while partially submerged in distilled water to promote wetting of the sample. The specimens and the drum are dried at the end of the rotation cycle (10 minutes at 20 rpm) and weighed. After two cycles of rotating and drying the weight loss and the shape and size of the remaining rock fragments are recorded and the Slake Durability Index (SDI) is calculated. Both the SDI and the description of the shape and size of the remaining particles are used to determine the durability of soft rocks.  **Figure 8-5: Rotating Drum Assembly and Setup of Slake Durability Equipment.** (ASTM D 4644, 1995)
Commentary	This test is typically performed on shales and other weak rocks that may be subject to degradation in the service environment. When some shales are newly exposed to atmospheric conditions, they can degrade rapidly and affect the stability of a rock fill or cut, the subgrade on which a foundation is to be placed, or the base and side walls of drilled shafts prior to placement of concrete.

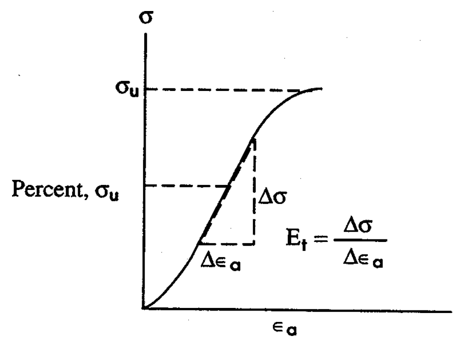
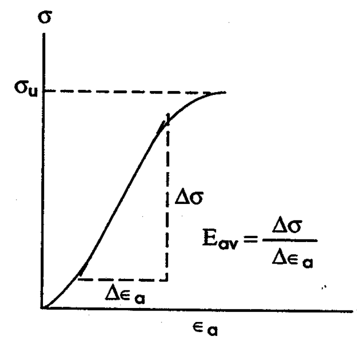
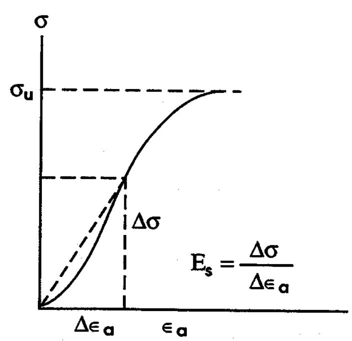
Soundness of Riprap	
AASHTO ASTM	- D 5240
Purpose	To determine the soundness of rock subjected to erosion.
Procedure	The procedure is known as the Rock Slab Soundness Test. Two representative, sawed, rock slab specimens are immersed in a solution of sodium or magnesium sulfate and dried and weighed for five cycles. The percent weight loss as a result of these tests is expressed as percent soundness.
Commentary	One of the most effective means to control erosion along riverbanks and coastal beaches is by covering exposed soil with rip-rap, or a combination of geosynthetics and rip-rap. Rock or stone used in this mode is subject to degradation from weathering effects due to repeated cycles of wetting & drying, as well as repeated exposure to salts used in de-icing of roadways. This test is used to estimate this type of degradation. A similar test for aggregates is available through ASTM C 88.

Durability Under Freezing and Thawing	
AASHTO ASTM	- D 5312
Purpose	To determine the resistance of rock used for erosion control to repeated cycles of freezing and thawing.
Procedure	Slabs of representative rock specimens are subjected to freezing and thawing cycles in the laboratory. The loss of dry weight at the end of five successive cycles of freezing, thawing, and drying is expressed as percent loss due to freeze/thaw.
Commentary	This test is useful in assessing the durability of rock due to weathering effects, in particularly for stone and gravel aggregates used in northern climates where seasonal winters will degrade their use in highway construction. It can also be used to assess the durability of armor stones placed for shore protection or rip-rap placed for shoreline protection or dam embankment protection.

As discussed above, none of these tests provide results which can be used independent of each other or independent of other tests and experience. Often the behavior of rip-rap stone in actual use will vary widely from the laboratory behavior.

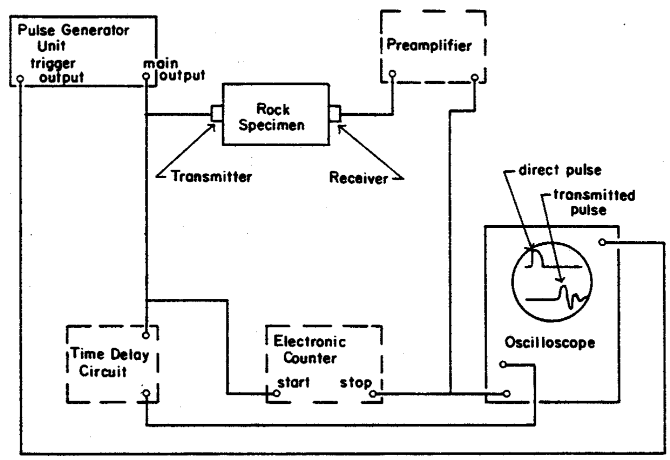
8.2.3. Deformation Characteristics of Intact Rocks

The stiffness of rocks is represented by an equivalent elastic modulus at small- to intermediate-strains.

Elastic Moduli	
AASHTO ASTM	- D 3148
Purpose	To determine the deformation characteristics of intact rock at intermediate strains and permit comparison with other intact rock types.
Procedure	This test is performed by placing an intact rock specimen in a loading device and recording the deformation of the specimen under axial stress. The Young's modulus, either average, secant, or tangent moduli, can be determined by plotting axial stress versus axial strain curves.  (a) Tangent Modulus Measured at a Fixed Percentage of Ultimate Strength  (b) Average Modulus of Linear Portion of Axial Stress-Strain Curve  (c) Secant Modulus Measured up to a Fixed Percentage of Ultimate Strength **Figure 8-6: Definitions for Determining Elastic (Young's) Modulus from Axial Stress-Strain Measurements During Compression Loading ,** including (a) Tangent, (b) Average, and (c) Secant Values. (ASTM D 3148)
Commentary	The results of these tests cannot always be replicated because of localized variations in the each unique rock specimen. They provide reasonably reliable data for engineering applications involving rock classification type, but must be adjusted to take into account rock mass characteristics such as jointing, fissuring, and weathering.

<table>
<tr><th colspan="2">Ultrasonic Testing</th></tr>
<tr><td>AASHTO
ASTM</td><td>-
D 2845</td></tr>
<tr><td>Purpose</td><td>To determine the pulse velocities of compression and shear waves in intact rock and the ultrasonic elastic constants of isotropic rock.</td></tr>
<tr><td>Procedure</td><td>

Ultrasound waves are transmitted through a carefully prepared rock specimen. The ultrasonic elastic constants are calculated from the measured travel time and distance of compression and shear waves in a rock specimen. Figure 8-7 shows a schematic diagram of typical apparatus used for ultrasonic testing.



Note: Components shown by dashed lines are optional, depending on method of travel-time measurement and voltage sensitivity of oscilloscope.

Figure 8-7: Schematic Diagram of the Ultrasonics Apparatus (ASTM D 2845)

</td></tr>
<tr><td>Commentary</td><td>

The primary advantages of ultrasonic testing are that it yields compression (P-wave) and shear (S-wave) velocities, and ultrasonic values for the elastic constants of intact homogeneous isotropic rock specimens. Elastic constants for rocks having pronounced anisotropy may require measurements to be taken across different directions to reflect orthorhombic stiffnesses and moduli, particularly if pronounced foliation, banding, layering, and fabric are evident.

The ultrasonic evaluation of elastic rock properties of intact specimens is useful for rock classification purposes and the evaluation of static and dynamic properties at small strains (shear strains < 10^{-4} %). Older equipment only provides ultrasonic P-waves measurements, while new designs obtain both P- and S-wave velocites. When compared with wave velocities obtained from field geophysical tests, the ultrasonics results provide an index of the degree of fissuring within the rock mass. This test is relatively inexpensive to perform and is nondestructive, thus may be conducted prior to strength testing of intact cores to optimize data collection.

</td></tr>
</table>

8.3 QUALITY ASSURANCE FOR LABORATORY TESTING OF ROCKS

In general, the general quality assurance guidelines presented previously on the laboratory testing of soils (Chapter 7) also apply for laboratory testing of intact rock. Herein, certain precautions applicable to laboratory rock testing are presented.

8.3.1 Cautions

Omissions or errors introduced during laboratory testing, if undetected, will be carried though the process of design and construction, possibly resulting in costly or unsafe facilities. Table 8-2 lists topics that should be considered and given proper attention in order that a reasonable assessment of the rock will be ascertained and an optimization of the geotechnical investigation can be realized in terms of economy, performance, and safety. Guidance in the proper handling and storage of rock cores may be found in ASTM D 5079 (Preserving & Transporting Rock Core Samples).

TABLE 8-2.

COMMON SENSE GUIDELINES FOR LABORATORY TESTING OF ROCKS

1. Provide protection of samples to avoid moisture loss and structural disturbance.
2. Clearly indicate proper numbering and identification of samples.
3. Storage of samples in controlled environments to prevent drying, overheating, & freezing.
4. Take care in the handling & selection of representative specimens for testing.
5. Consult the field logs while selecting test specimens.
6. Recognizing disturbances & fractures caused by coring procedures.
7. Maintain trimming & testing equipment in good operating condition.
8. Use of proper fittings, platens, o-rings, & membranes in triaxial, uniaxial, and shear tests.
9. Careful tolerances in trimming of ends and sides of intact cores.
10. Document frequency, spacing, conditions, & infilling of joints and discontinuities.
11. Maintain calibration of instruments used to measure load, deflections, temperatures, & time.
12. Use of properly-determined loading rate for strength tests.
13. Photo documentation of sample cores, fracture patterns, & test specimens for report.
14. Carefully align & level all specimens in directional loading apparatuses and test frames.
15. Record initial baselines, offsets, and eccentricities prior to testing.
16. Save remnant rock pieces after destructive testing by uniaxial, triaxial, & direct shear.
17. Conduct nondestructive tests (i.e., porosity, unit weight, ultrasonics) prior to destructive strength testing (compression, tensile, shear).