Chapter 5

FIELD AND LABORATORY TESTING PROCEDURES

Final

SCDOT GEOTECHNICAL DESIGN MANUAL

August 2008

Table of Contents

Section **Page**

List of Tables

CHAPTER 5

FIELD AND LABORATORY TESTING PROCEDURES

5.1 INTRODUCTION

This Chapter discusses items related to field and laboratory testing procedures. The first item is sampling procedures and will discuss the different methods of retrieving soil and rock samples. The second item is the drilling procedure and discusses what types of equipment are typically available. The third item is the soil/rock laboratory testing and will discuss the different types of testing procedures. Tests shall be performed in accordance with ASTM and/or AASHTO.

5.2 SAMPLING PROCEDURE

5.2.1 Soil Sampling

ASTM and AASHTO have procedures that must be followed for the collection of field samples. All samples must be properly obtained, preserved, and transported to a laboratory facility in accordance with these procedures in order to preserve the samples as best as possible. There are several procedures that can be used for the collection of samples as described below. See ASTM D4220 - *Standard Practices for Preserving and Transporting Soil*.

5.2.1.1 Bulk Samples

Bulk samples are highly disturbed samples obtained from auger cuttings or test pits. The quantity of the sample depends on the type of testing to be performed, but can range up to 50 lb (25 kg) or more. Typical testing performed on bulk samples include moisture-density relationship, moisture-plasticity relationship, grain-size distribution, natural moisture content, and triaxial compression on remodeled specimens.

5.2.1.2 Split-Barrel Sampling

The most commonly used sampling method is the split-barrel sampler, also known as standard split-spoon. This method is used in conjunction with the Standard Penetration Test. The sampler is driven into soil by means of hammer blows. The number of blows required for driving the sampler through three 6-inch intervals is recorded. The last two 6-inch intervals is added to make up the standard penetration number, N_{meas}. After driving is completed the sampler is retrieved and the soil sample is removed and placed into air tight containers. Each standard penetration number and collection of samples is to be done at 5-foot intervals, except in the upper 10 feet where samples will be collected every 2 feet. This type of sampling is adequate for moisture content, grain-size distribution, Atterberg Limits tests, and visual identification. See ASTM D1586 - *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils* (AASHTO T206 - *Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soil*s).

5.2.1.3 Shelby Tube

The Shelby tube is a thin-walled steel tube pushed into the soil to be sampled by hydraulic pressure and spun to shear off the base. Afterwards the sampler is pulled out and immediately sealed and taken to the laboratory facility. This process allows the sample to be undisturbed as much as possible and is suitable for fine-grained soils that require strength and consolidation tests. See ASTM D1587 - *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes* (AASHTO T207 - *Standard Method of Test for Thin-Walled Tube Sampling of Soils*). There are a variety of methods that may be used to collect a Shelby tube samples. Listed in the following sections are the types of sampling methods commonly used. It is not the intention of this Manual that this list be comprehensive. If another sampling procedure/method is to be used, contact the PCS/GDS for review prior to acceptance.

5.2.1.3.1 Fixed Piston Sampler

This sampler has the same standard dimensions as the Shelby tube, above. A piston is positioned at the bottom of the thin-wall tube while the sampler is lowered to the bottom of the hole, thus preventing disturbed materials from entering the tube. The piston is locked in place on top of the soil to be sampled. A sample is obtained by pressing the tube into the soil with a continuous, steady thrust. The stationary piston is held fixed on top of the soil while the sampling tube is advanced. This creates suction while the sampling tube is retrieved thus aiding in retention of the sample. This sampler is suitable for soft to firm clays and silts. Samples are generally less disturbed and have a better recovery ratio than those from the Shelby tube method.

5.2.1.3.2 Floating Piston Sampler

This sampler is similar to the fixed method above, except that the piston is not fixed in position but is free to ride on the top of the sample. The soils being sampled must have adequate strength to cause the piston to remain at a fixed depth as the sampling tube is pushed downward. If the soil is too weak, the piston will tend to move downward with the tube and a sample will not be obtained. This method should therefore be limited to stiff or hard cohesive materials.

5.2.1.3.3 Retractable Piston Sampler

This sampler is similar to the fixed piston sampler; however, after lowering the sampler into position the piston is retracted and locked in place at the top of the sampling tube. A sample is then obtained by pushing the entire assembly downward. This sampler is used for loose or soft soils.

5.2.1.3.4 Hydraulic (Osterberg) Piston Sampler

The hydraulic piston sampler is made similar to the Shelby tube. Instead of a rod pushing the sampler into the soil and then spun to shear off, the thin walled tube is pushed into the soil and a piston closes the end of the thin walled tube. After the tube closes, pressure is released thus preventing distortion by neither letting the soil squeeze into the sampler tube very fast nor admitting excess soil. This technique is especially useful for soil samples that require the most undisturbed sample in soft clays and silts.

5.2.2 Rock Core Sampling

The most common method for obtaining rock samples is diamond core drilling. There are three basic types of core barrels: Single tube, double tube, and triple tube. See ASTM D2113 - *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation* (AASHTO T225 - *Standard Method of Test for Diamond Core Drilling for Site Investigation*).

5.3 FIELD TESTING PROCEDURES

Assuming access and utility clearances have been obtained and a survey base line has been established in the field, field explorations are begun based on the subsurface exploration request prepared by the GDS for in-house or by the Geotechnical Engineering Consultant for all other projects. Many methods of field exploration exist; some of the more common are described below. These methods are often augmented by in-situ testing. The testing described in this Chapter provides the Geotechnical Engineer with soil and rock parameters determined in-situ. This is important on all projects, especially those involving soft clays, loose sands, and/or sands below the water table, due to the difficulty of obtaining representative samples suitable for laboratory testing. For each test included, a brief description of the equipment, the test method, and the use of the data is presented.

5.3.1 Test Pits

These are the simplest methods of inspecting subsurface soils. Test pits consist of excavations performed by hand, backhoe, or dozer. Hand excavations are often performed with posthole diggers. Test pits offer the advantages of speed and ready access for sampling; however, test pits are severely hampered by limitations of depth and by the fact that advancement through soft or loose soils or below the water table can be extremely difficult. Test pits are used to examine large volumes of near surface soils and can be used to obtain bulk samples for additional testing.

5.3.2 Soil Borings

Soil borings are probably the most common method of exploration. Soil borings can be advanced using a number of methods. In addition, several different in-situ tests can be performed in the open borehole. The methods for advancing the boreholes will be discussed first followed by the methods of in-situ testing.

5.3.2.1 Manual Auger Borings

Manual auger borings are advanced using hand held equipment. Typically, these borings are conducted in areas where access for standard drilling equipment is severely restricted. Manual auger borings are limited in depth by the presence of ground water or collapsible soils that cause caving in the borehole. The Dynamic Cone Penetrometer test is usually conducted in conjunction with this boring method.

5.3.2.2 Hollow Stem Auger Borings

A hollow-stem auger (HSA) consists of a continuous flight auger surrounding a hollow drill stem. The hollow-stem auger is advanced similar to other augers; however, removal of the hollow stem auger is not necessary for sampling. SPT and undisturbed samples are obtained through the hollow drill stem, which acts like a casing to hold the hole open. This increases usage of hollow-stem augers in soft and loose soils. See ASTM D6151 - *Standard Practice for Using Hollow-Stem Augers for Geotechnical Exploration and Soil Sampling* (AASHTO T306 - *Standard Method of Test for Progressing Auger Borings for Geotechnical Explorations*). This drilling method is limited to areas where the ground water is not anticipated effecting the Standard Penetration Test (SPT).

5.3.2.3 Wash Rotary Borings

In this method, the boring is advanced by a combination of the chopping action of a light bit and the jetting action of water flowing through the bit. A downward pressure applied during rapid rotation advances the hollow drill rods with a cutting bit attached to the bottom. The drill bit cuts the material and drilling fluid washes the cuttings from the borehole. This is, in most cases, the fastest method of advancing the borehole and can be used in any type of soil except those containing considerable amounts of large gravel or boulders. Drilling mud or casing can be used to keep the borehole open in soft or loose soils, although the former makes identifying strata change by examining the cuttings difficult. SPT and undisturbed samples are obtained through the drilling fluid, which holds the borehole open. This method of drilling is required in the Lowcountry and the Pee Dee Regions and in Aiken, Allendale, Bamberg, Barnwell, Calhoun, Lexington, Orangeburg and Richland Counties of the Midlands Region (see Chapter 1).

5.3.2.4 Coring

A core barrel is advanced through rock by the application of downward pressure during rotation. Circulating water removes ground-up material from the hole while also cooling the bit. The rate of advance is controlled so as to obtain the maximum possible core recovery. See ASTM D2113 – *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation*. A professional geologist or geotechnical engineer shall be on-site during coring operations to perform measurements in the core hole to allow for determination of the Rock Mass Rating (RMR) (see Chapter 6).

5.3.3 Standard Penetration Test

This test is probably the most widely used field test in the United States. It has the advantages of simplicity, the availability of a wide variety of correlations for its data, and the fact that a sample is obtainable with each test. A standard split-barrel sampler (discussed previously) is advanced into the soil by dropping a 140-pound safety or automatic hammer attached to the drill rod from a height of 30 inches. **[Note: Use of a donut hammer is not permitted]**. The sampler is advanced a total of 18 inches. The number of blows required to advance the sampler for each of three 6-inch increments is recorded. The sum of the number of blows for the second and third increments is called the Standard Penetration Value, or more commonly, N-value (N_{meas}) (blows per foot). Tests shall be performed in accordance with ASTM D1586 - *Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils* (AASHTO T206 - *Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soils*). The

Standard Penetration Test shall be performed every 2 feet in the upper 10 feet (5 N_{meas}) and every 5 feet thereafter. The exception is beneath embankments, the Standard Penetration Test will also be performed every 2 feet in the first 10 feet of original ground surface. The depth to the original ground surface may be estimated based on the height of the existing embankment.

When Standard Penetration Tests (SPT) are performed in soil layers containing shell or similar materials, the sampler may become plugged. A plugged sampler will cause the SPT N-value to be much larger than for an unplugged sampler and, therefore, not a representative index of the soil layer properties. In this circumstance, a realistic design requires reducing the N-value used for design to the trend of the N-values which do not appear distorted. However, the actual N-values should be presented on the Soil Test Boring Logs (see Chapter 6). A note shall be placed on the Soil Test Boring Logs indicating that the sampler was plugged.

The SPT values should not be used indiscriminately. They are sensitive to the fluctuations in individual drilling practices and equipment. Studies have also indicated that the results are more reliable in sands than clays. Although extensive use of this test in subsurface exploration is recommended, it should always be augmented by other field and laboratory tests, particularly when dealing with clays. The type of hammer (safety or automatic) shall be noted on the boring logs, since this will affect the actual input driving energy. N_{meas} require correction prior to being used in engineering analysis (see Chapter 7).

The amount of driving energy shall be measured using ASTM D4633 - *Standard Test Method for Energy Measurement for Dynamic Penetrometers*. Since there is a wide variability of performance in SPT hammers, this method is used to evaluate an individual hammer's performance. The energy of a hammer can be effected by the mechanical state of the hammer system (i.e. maintained or not), the condition of the rope, the experience of the driller, the time of day, and the weather. For SPTs performed under the Geotechnical On-Call Contract, a QA/QC plan is required. For SPTs performed under the General Services On-Call Contract, a QA/QC plan for measuring hammer energy is also required. The QA/QC plans under either contract shall be submitted to the Department for acceptance, prior to being used in the field.

The SPT installation procedure is similar to pile driving because it is governed by stress wave propagation. As a result, if force and velocity measurements are obtained during a test, the energy transmitted can be determined.

5.3.4 Dynamic Cone Penetrometer

The Dynamic Cone Penetrometer is a dynamic penetration test usually performed in conjunction with manual auger borings. Dynamic Cone Penetrometer testing shall be conducted using the procedure presented by Sowers and Hedges (1966). The Dynamic Cone Penetrometer resistance values shall be correlated to N_{meas}, by performing an SPT adjacent to the Dynamic Cone Penetrometer test.

5.3.5 Cone Penetrometer Test

The Cone Penetrometer Test is a quasi-static penetration test in which a cylindrical rod with a conical point is advanced through the soil at a constant rate and the resistance to penetration is measured. A series of tests performed at varying depths at one location is commonly called a sounding.

Several types of penetrometer are in use, including mechanical (Dutch) cone, mechanical friction-cone, electric cone, electric friction-cone, and electro-piezocone. Cone penetrometers measure the resistance to penetration at the tip of the penetrometer or end-bearing component of resistance. Friction-cone penetrometers are equipped with a friction sleeve, which provides the added capability of measuring the side friction component of resistance. Mechanical penetrometers have telescoping tips allowing measurements to be taken incrementally, generally at intervals of 8 inches (200 mm) or less. Electronic penetrometers use electronic force transducers to obtain continuous measurements with depth. Electro-piezocones are also capable of measuring pore water pressures during penetration. Electro-piezocones or some variation (i.e. seismic electro-piezocones) are the only allowed cone penetrometed device.

For all types of penetrometers, cone dimensions of a 60-degree tip angle and a 10 cm^2 (1.55 in^2) projected end area are standard. Friction sleeve outside diameter is the same as the base of the cone. Penetration rates should be between 10 to 20 mm/sec. Tests shall be performed in accordance with ASTM D5778 - *Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils* (electro-piezocones).

The penetrometer data is plotted showing the tip stress, the friction resistance and the friction ratio (friction resistance divided by tip stress) vs. depth. Pore pressures, can also be plotted with depth. The results should also be presented in tabular form indicating the interpreted results of the raw data. See Chapter 6 – Materials Description, Classification and Logging for presentation of CPT data.

The friction ratio plot can be analyzed to determine soil type. Many correlations of the cone test results to other soil parameters have been made, and design methods are available for spread footings and piles. The penetrometer can be used in sands or clays, but not in rock or other extremely dense soils. Generally, soil samples are not obtained with soundings, so penetrometer exploration should always be augmented by SPT borings or other borings with soil samples taken. Since soil samples are not obtained, the CPT should be correlated to the in-situ soils by performing a boring adjacent to the sounding.

The electro-piezocones can also be used to measure the dissipation rate of the excessive pore water pressure. This type of test is useful for subsoils, such as fibrous peat, muck, or soft clays that are very sensitive to sampling techniques. The cone should be equipped with a pressure transducer that is capable of measuring the induced water pressure. To perform this test, the cone will be advanced into the subsoil at a standard rate of 20 mm/sec. Pore water pressures will be measured immediately and at several time intervals thereafter. Use the recorded data to plot pore pressure dissipation versus log-time graph. Using this graph, direct calculation of the pore water pressure dissipation rate or rate of settlement of the soil can be performed.

Electro-piezocones can be fitted with other instrumentation above the friction sleeve. The additional instrumentation can include geophones that may be used to measure shear wave velocities. Another instrument that may be included is an inclinometer to determine if the instrument is getting off plumb. Other instruments include microphones and nuclear density equipment.

5.3.6 Dilatometer Test

The dilatometer is a 3.75-inch wide and 0.55-inch thick stainless steel blade with a thin 2.4-inch diameter expandable metal membrane on one side. While the membrane is flush with the blade surface, the blade is either pushed or driven into the soil using a drilling rig. Rods carry pneumatic and electrical lines from the membrane to the surface. At depth intervals of 12 inches, pressurized gas is used to expand the membrane, both the pressure required to begin membrane movement and that required to expand the membrane into the soil 0.04 inches (1.1 mm) are measured. Additionally, upon venting the pressure corresponding to the return of the membrane to its original position may be recorded. Through developed correlations, information can be deduced concerning material type, pore water pressure, in-situ horizontal and vertical stresses, void ratio or relative density, modulus, shear strength parameters, and consolidation parameters. Compared to the pressuremeter, the flat dilatometer has the advantage of reduced soil disturbance during penetration. Tests shall be performed in accordance with ASTM D6635 - *Standard Test Method for Performing the Flat Plate Dilatometer*.

5.3.7 Pressuremeter Test

This test is performed with a cylindrical probe placed at the desired depth in a borehole. The Menard type pressuremeter requires pre-drilling of the borehole; the self-boring type pressuremeter advances the hole itself, thus reducing soil disturbance. The PENCEL pressuremeter can be set in place by pressing it to the test depth or by direct driving from ground surface or from within a predrilled borehole. The hollow center PENCEL probe can be used in series with the static cone penetrometer. The Menard probe contains three flexible rubber membranes. The middle membrane provides measurements, while the outer two are “guard cells” to reduce the influence of end effects on the measurements. When in place, the guard cell membranes are inflated by pressurized gas while the middle membrane is inflated with water by means of pressurized gas. The pressure in all the cells is incremented and decremented by the same amount. The measured volume change of the middle membrane is plotted against applied pressure. Tests shall be performed in accordance with ASTM D4719 - *Standard Test Method for Prebored Pressuremeter Testing in Soils.*

Studies have shown that the “guard cells” can be eliminated without sacrificing the accuracy of the test data provided the probe is sufficiently long. Furthermore, pumped air can be substituted for the pressurized gas used to inflate the membrane with water. The TEXAM® pressuremeter is an example of this type.

Results are interpreted based on semi-empirical correlations from past tests and observation. In-situ horizontal stresses, shear strength, bearing capacities, and settlement can be estimated using these correlations. The pressuremeter test results can be used to obtain load transfer curves (p-y curves) for lateral load analyses. The pressuremeter test is very sensitive to borehole disturbance and the data may be difficult to interpret for some soils.

5.3.8 Field Vane Test

This test consists of advancing a four-bladed vane into cohesive soil to the desired depth and applying a measured torque at a constant rate until the soil fails in shear along a cylindrical surface. The torque measured at failure provides the undrained shear strength of the soil. A second test run immediately after remolding at the same depth provides the remolded strength of the soil and thus information on soil sensitivity. Tests shall be performed in accordance with ASTM D2573 - *Standard Test Method for Field Vane Shear Test in Cohesive Soil* (AASHTO T223 - *Standard Method of Test for Field Vane Shear Test in Cohesive Soil*).

This method is commonly used for measuring shear strength in soft clays and organic deposits. It should not be used in stiff and hard clays. Results can be affected by the presence of gravel, shells, roots, or sand layers. Shear strength may be overestimated in highly plastic clays and a correction factor should be applied.

5.3.9 Geophysical Testing Methods

Geophysical testing methods are non-destructive testing procedures which can provide general information on the general subsurface profile, depth to bedrock or water, location of granular borrow areas, peat deposits or subsurface anomalies and provide an indication of certain material properties (i.e. compression wave (V_P) and shear wave velocity (V_S)). Geophysical testing methods are not limited to subsurface conditions, but can also be used to evaluate existing bridge decks, foundations and pavements. The reader should see Application of Geophysical Methods to Highway Related Problems, FHWA-IF-04-021, for additional information on the application of geophysical test methods to other areas other than subsurface conditions.

5.3.9.1 Surface Wave Methods

Surface wave methods consist of Spectral Analysis of Surface Waves (SASW) or Multi-channel Analysis of Surface Waves (MASW). The SASW and MASW are used to measure layer thickness, depth and the shear wave velocity (V_S) of the layer. The shear wave velocity is more of bulk (general) velocity than a discrete velocity of a layer. Discrete shear wave velocity may be determined by crosshole or downhole methods. While the SASW will typically have 2 geophones, the MASW will have additional geophones spread over a larger area. Typically SASW and the MASW profiles are limited to a depth of approximately 130 feet using man portable equipment. Additional depth can be obtained but heavier motorized equipment is required.

5.3.9.2 Downhole Shear Wave Velocity Methods

Downhole methods for determining shear wave velocity differ from surface methods in that equipment is placed in the ground. In downhole methods, either a casing is placed in the ground and geophone is lowered in the casing or a seismic cone penetrometer (SCPT) is pushed into the ground. The SCPT has a geophone typically mounted above the friction sleeve on the cone. With either method, a shear wave is induced at the ground surface and the time for arrival is determined. If casing is used, care must be taken during construction. One of the major limitations of the SCPT is refusal to advance in dense soils.

5.3.9.3 Crosshole Shear Wave Velocity Methods

In crosshole shear wave velocity testing, shear wave velocities are determined between a series of casings. A downhole hammer and geophone are lowered to the same depth, but in different holes. The hammer is tripped and time for the shear wave to travel to the geophone is recorded. The major limitation to the crosshole method is the expense of the installation of the required casings. In addition, the care that must be taken during the construction of the casings to assure that the casings are plumb and in the same horizontal plane.

5.3.9.4 Seismic Refraction

Seismic refraction is used to determine the depth to bedrock. This method works well for depths less than 100 feet. A seismic energy source is required for producing seismic waves. A sledge hammer is typically used for depths less than 50 feet and either a drop weight or a black powder charge is used for depths between 50 and 100 feet. The seismic compression waves penetrate the overburden material and refract along the bedrock surface. This method can be used for up to 4 soil on rock layers; however, each layer must have a higher shear wave velocity than the overlying layer.

5.3.9.5 Seismic Reflection

Seismic reflection uses a surface seismic wave source to create seismic waves that can penetrate the subsurface. The waves are reflected at interfaces that have either a change in shear wave velocity and/or a change in density. Changes in velocity or density are termed impedance contrasts. At impedance contrasts, a portion of the seismic wave is reflected back to the ground surface and a portion continues into the subsurface where it is reflected at the next impedance contrast. Seismic reflection techniques can obtain information in excess of 100 feet.

5.3.9.6 Resistivity

Resistivity is used to find the depth to bedrock since soil and rock typically have different electrical resistances. The depth of the resistivity survey is typically 1/3 of the electrode spacing. For example, to reach a depth of 50 feet an electrode spacing of 150 feet is required. Resistivity surveys can reach depths of 160 feet. Resistivity testing is affected by the moisture content of the soil and the presence or lack of metals, salts and clay particles. In addition, resistivity surveys may be used to model ground water flow through the subsurface. Further, resistivity surveys may also be used to determine the potential for corrosion of foundation materials for the in-situ subsurface materials.

5.4 SOIL/ROCK LABORATORY TESTING

5.4.1 Grain-Size Analysis

There are two types of tests: Grain-Size with wash No. 200 and Hydrometer test. Grain size with wash No. 200, also known as Sieve Analysis, is for coarse-grained soils (sand, gravels). The hydrometer analysis is used for fine-grained soils (clays, silts).

The results of the analyses are presented in a semilogarithmic plot known as particle-size distribution curves. In the semilogarithmic scale, the particle sizes are plotted on the log scale. The percent finer is plotted in arithmetic scale. Therefore, the graph is easy to read the percentages of gravel, sand, silt, and clay-size particles in a sample of soil.

The grain-size analysis can also be used for obtaining three basic soil parameters from the curves. These parameters are: effective size (D10), Coefficient of Uniformity (Cu), and Coefficient of Curvature (Cc). The Hydraulic Engineering Group requires these parameters for scour analysis. Those soil test-boring logs at the Interior Bents of a bridge over a water environment must have a Hydrometer test performed at depths from 0-5 ft. See ASTM D422 - *Standard Test Method for Particle-Size Analysis of Soils* (AASHTO T88 - *Standard Method of Test for Particle Size Analysis of Soils*).

5.4.1.1 Sieve Analysis

The sieve analysis is a method used to determine the grain size distribution of soils. The soil is passed through a series of woven wires with square openings of decreasing sizes. The test gives a soil classification based on the percentage retained on the sieve. See ASTM C136 - *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* (AASHTO T311 - *Standard Method of Test for Grain-Size Analysis of Granular Soil Materials*).

5.4.1.2 Hydrometer

The Hydrometer analysis is used to determine the particle size distribution in a soil that is finer than a No. 200 sieve size (0.075 mm), which is the smallest standard size opening in the sieve analysis. The procedure is based on the sedimentation of soil grains in water. It is expressed by Stokes Law, which says the velocity of the soil sedimentation is based on the soil particles shape, size, weight, and viscosity of the water. Thus, the hydrometer analysis measures the change in specific gravity of a soil-water suspension as soil particles settle out over time. See ASTM D422 - *Standard Test Method for Particle-Size Analysis of Soils* (AASHTO T88 - *Standard Method of Test for Particle Size Analysis of Soils*).

5.4.2 Moisture Content

The moisture content (*w*) is defined as the ratio of the weight of water in a sample to the weight of solids. The weight of the solids must be oven dried and is considered as weight of dry soil. Organic soils can have the water content determined, but must be dried at a lower temperature for the weight of dry soil to prevent degradation of the organic matter. See ASTM D2216 - *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass* (AASHTO T265 - *Standard Method of Test for Laboratory Determination of Moisture Content of Soils*).

5.4.3 Atterberg Limits

The Atterberg Limits are different descriptions of the moisture content of fine-grained soils as it transitions between a solid to a liquid-state. For classification purposes the two primary Atterberg Limits used are the plastic limit (PL) and the liquid limit (LL). The plastic index (PI) is also calculated for soil classification.

5.4.3.1 Plastic Limit

The plastic limit (PL) is the moisture content at which a soil transitions from being in a semisolid state to a plastic state. Tests shall be performed in accordance with ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T90 - *Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils*).

5.4.3.2 Liquid Limit

The liquid limit (LL) is defined as the moisture content at which a soil transitions from a plastic state to a liquid state. Tests shall be performed in accordance with ASTM D4318 - *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* (AASHTO T89 - *Standard Method of Test for Determining the Liquid Limit of Soils*).

5.4.3.3 Plasticity Index

The plasticity index (PI) is defined as the difference between the liquid limit and the plastic limit of a soil. The PI represents the range of moisture contents within which the soil behaves as a plastic solid.

5.4.4 Specific Gravity of Soils

The specific gravity of soil, G_s, is defined as the ratio of the unit weight of a given material to the unit weight of water. The procedure is applicable only for soils composed of particles smaller than the No. 4 sieve (4.75 mm). See ASTM D854 - *Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer* (AASHTO T100 - *Standard Method of Test for Specific Gravity of Soils*). If the soil contains particles larger than the No. 4 sieve (4.75 mm), use ASTM C127- *Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate*.

5.4.5 Strength Tests

The shear strength is the internal resistance per unit area that the soil can handle before failure and is expressed as a stress. There are two components of shear strength, cohesive element (expressed as the cohesion, c, in units of force/unit area) and frictional element (expressed as the angle of internal friction, ϕ). These parameters are expressed in the form of total stress (c, ϕ) or effective stress (c', ϕ'). The total stress on any subsurface element is produced by the overburden pressure plus any applied loads. The effective stress equals the total stress minus the pore water pressure. The common methods of ascertaining these parameters in the laboratory are discussed below. All of these tests are normally performed on undisturbed samples, but may also be performed on remolded samples.

5.4.5.1 Unconfined Compression Tests

The unconfined compression test is a quick method of determining the value of undrained cohesion (c_u) for clay soils. The test involves a clay specimen with no confining pressure and an axial load being applied to observe the axial strains corresponding to various stress levels. The stress at failure is referred to as the unconfined compression strength. The c_u is taken as one-half the unconfined compressive strength, q_u. See ASTM D2166 - *Standard Test Method for Unconfined Compressive Strength of Cohesive Soil* (AASHTO T208 - *Standard Method of Test for Unconfined Compressive Strength of Cohesive Soil*).

5.4.5.2 Triaxial Compression Tests

The triaxial compression test is a more sophisticated testing procedure for determining the shear strength of a soil. The test involves a soil specimen subjected to an axial load until failure while also being subjected to confining pressure that approximates the in-situ stress conditions. There are three types of triaxial tests which are described below.

5.4.5.2.1 Unconsolidated-Undrained (UU), or Q Test

In unconsolidated-undrained tests, the specimen is not permitted to change its initial water content before or during shear. The results are total stress parameters. This test is used primarily in the calculation of immediate embankment stability during quick-loading conditions. Refer to ASTM D2850 - *Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils* (AASHTO T296 - *Standard Method of Test for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression*).

5.4.5.2.2 Consolidated-Undrained (CU), or R Test

The consolidated-undrained test is the most common type of triaxial test. This test allows the soil specimen to be consolidated under a confining pressure prior to shear. After the pore water pressure is dissipated, the drainage line will be closed and the specimen will be subjected to shear. Several tests on similar specimens with varying confining pressures may have to be made to determine the shear strength parameters. See ASTM D4767 - *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils* (AASHTO T297 - *Standard Method of Test for Consolidated, Undrained Triaxial Compression Test on Cohesive Soils*).

5.4.5.2.3 Consolidated-Drained (CD), or S Test

The consolidated-drained test is similar to the consolidated-undrained test except that drainage is permitted during shear and the rate of shear is very slow. Thus, the buildup of excess pore pressure is prevented. Again, several tests on similar specimens will be conducted to determine the shear strength parameters. This test is used to determine parameters for calculating long-term stability of embankments. Refer to ASTM WK3821 - *New Test Method for Consolidated Drained Triaxial Compression Test for Soils*.

5.4.5.3 Direct Shear

The direct shear test is the oldest and simplest form of shear test. A soil sample is placed in a metal shear box and undergoes a horizontal force. The soil fails by shearing along a plane when the force is applied. The test can be performed either in stress-controlled or strain-controlled. In addition the test is typically performed as consolidated-drained test on cohesionless soils. See ASTM D3080 - *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions* (AASHTO T236 - *Standard Method of Test for Direct Shear Test of Soils Under Consolidated Drained Conditions*).

5.4.5.4 Miniature Vane Shear (Torvane) and Pocket Penetrometer

The miniature vane shear and the pocket penetrometer tests are performed to obtain undrained shear for plastic cohesive soils. Both of these tests consist of hand-held devices that are pushed into the sample and either a torque resistance (Torvane) or a tip resistance (pocket penetrometer) is measured. They can be performed in the lab or in the field. See ASTM D4648 - *Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil* for the miniature vane shear test only.

5.4.6 Consolidation Test

The amount of settlement induced by the placement of load bearing elements on the ground surface or the construction of earthen embankments will affect the performance of the structure. The amount of settlement is a function of the increase in pore water pressure caused by the loading and the reduction of this pressure over time. The reduction in pore pressure and the rate of the reduction are a function of the permeability of the in-situ soil. All soils undergo elastic compression, primary and secondary consolidation. Sandy (coarse-grained) soils tend to be relatively permeable and will therefore, undergo settlement much faster. The amount of elastic compression settlement can vary depending on the soil type; however, the time for this settlement to occur is relatively quick and will normally occur during construction. Clayey (fine-grained) soils have a much lower permeability and will, therefore, take longer to settle. Clayey soils undergo elastic compression during the initial stages of loading (i.e. the soil particles rearrange due to the loading). After elastic compression, clayey soils enter primary consolidation. Saturated clayey soils have a lower coefficient of permeability, thus the excess pore water pressure generated by loading will gradually dissipate over a longer period of time. Therefore in saturated clays, the amount and rate of settlement is of great importance in construction. For example, an embankment may settle until a gap exists between an approach and a bridge abutment. The calculation of settlement involves many factors, including the magnitude of the load, the effect of the load at the depth at which compressible soils exist, the water table, and characteristics of the soil itself. Consolidation testing is performed to ascertain the nature of these characteristics.

The most often used method of consolidation testing is the one-dimensional test. The consolidation test unit consists of a consolidometer (oedometer) and a loading device. The soil sample is placed between two porous stones, which permit drainage. Load is applied incrementally and is typically held up to 24 hours. The test measures the height of the specimen after each loading is applied. The results are plotted on a time versus deformation log scale plot. From this curve, two parameters can be derived: coefficient of consolidation (C_v) and coefficient of secondary compression (C_α). These parameters are used to predict the rate of primary settlement and the amount of secondary consolidation.

After the time-deformation plots are obtained, the void ratio and the strain can be calculated. Two more plots can be presented; an e-log p curve, which plots void ratio (e) as a function of the log of pressure (p), or an ε-log p curve where ε equals percent strain. The parameters necessary for settlement calculation can be derived from the e-log p curve and are: compression index (C_c), recompression index (C_r), preconsolidation pressure (P_c), and initial void ratio (e_o). Alternatively, the ε-log p curve provides the compression index ($C_{\varepsilon c}$), the recompression index ($C_{\varepsilon r}$), and the preconsolidation pressure (P_c).

To evaluate the recompression parameters of the sample, an unload/reload cycle can be performed during the loading schedule. To better evaluate the recompression parameters for overconsolidated clays, the unload/reload cycle may be performed after the preconsolidation pressure has been defined. After the maximum loading has been reached, the loading is removed in appropriate decrements. See ASTM D2435 - *Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading* (AASHTO T216 - *Standard Method of Test for One-Dimensional Consolidation Properties of Soils*).

For soils that are high in organic material and highly compressible inorganic soils, secondary consolidation is more important than primary consolidation.

For high organic materials (organic content greater than 50%), research sponsored by the Florida Department of Transportation has shown that the end of primary consolidation occurs quickly in the laboratory and field, and that a major portion of the total settlement is due to secondary consolidation (creep). As a result, differentiating between primary consolidation and creep settlement can be very difficult and generate misleading results. To analyze results from one-dimensional consolidation tests for these types of materials, use the Square Root (Taylor) Method to identify the end of primary consolidation for each load sequence. In addition, each load sequence must be maintained for at least 24 hours to identify a slope for the secondary consolidation portion of the settlement versus time plot.

5.4.7 Organic Content

Organic soils demonstrate very poor engineering characteristics, most notably low strength and high compressibility. In the field these soils can usually be identified by their dark color, musty odor and low unit weight. The most used laboratory test for design purposes is the Ignition Loss test, which measures how much of a sample's mass burns off when placed in a muffle furnace. The results are presented as a percentage of the total sample mass. Tests shall be performed in accordance with ASTM D2974 - *Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils* (AASHTO T267 - *Standard Method of Test for Determination of Organic Content in Soils by Loss on Ignition*).

5.4.8 Shrinkage and Swell

Certain soil types (highly plastic) have a large potential for volumetric change depending on the moisture content of the soil. These soils can shrink with decreasing moisture or swell with increasing moisture. Shrinkage can cause soil to pull away from structure thus reducing the bearing area or causing settlement of the structure beyond that predicted by settlement analysis. Swelling of the soil can cause an extra load to be applied to the structure that was not accounted for in design. Therefore, the potential for shrinkage and swelling should be determined for soils that have high plasticity.

5.4.8.1 Shrinkage

These tests are performed to determine the limits of a soil's tendency to lose volume during decreases in moisture content. The shrinkage limit (SL) is presented as a percentage in moisture content, at which the volume of the soil mass ceases to change. See ASTM D427 - *Test Method for Shrinkage Factors of Soils by the Mercury Method* (AASHTO T92 - *Standard Method of Test for Determining the Shrinkage Factors of Soils*).

5.4.8.2 Swell

There are certain types of soils that can swell, particularly clay in the montmorillonite family. Swelling occurs when the moisture is allowed to increase causing the clay soil to increase in volume. There are a number of reasons for this to occur: the elastic rebound of the soil grains, the attraction of the clay mineral for water, the electrical repulsion of the clay particles and their adsorbed cations from one another, or the expansion of the air trapped in the soil voids. In the montmorillonite family, adsorption and repulsion predominate and this can cause swelling. Testing for swelling is difficult, but can be done. It is recommended that these soils not be used for roadway construction. The swell potential can be estimated from the test methods shown in ASTM D4546 - *Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils* (AASHTO T258 - *Standard Method of Test for Determining Expansive Soils*).

5.4.9 Permeability

Permeability, also known as hydraulic conductivity, has the same units as velocity and is generally expressed in ft/min or m/sec. Coefficient of permeability is dependent on void ratio, grain-size distribution, pore-size distribution, roughness of mineral particles, fluid viscosity, and degree of saturation. There are three standard laboratory test procedures for determining the coefficient of permeability soil, constant and falling head tests and flexible wall tests.

5.4.9.1 Constant Head Test

In the constant head test, water is poured into a sample of soil, and the difference of head between the inlet and outlet remains constant during the testing. After the flow of water becomes constant, water that is collected in a flask is measured in quantity over a time period. This test is more suitable for coarse-grained soils that have a higher coefficient of permeability. See ASTM D2434 - *Standard Test Method for Permeability of Granular Soils (Constant Head)* (AASHTO T215 - *Standard Method of Test for Permeability of Granular Soils (Constant Head)*).

5.4.9.2 Falling Head Test

The falling head test uses a similar procedure to the constant head test, but the head is not kept constant. The permeability is measured by the decrease in head over a specified time. This test is more appropriate for fine-grained soils. Tests shall be performed in accordance with ASTM D5856 - *Standard Test Method for Measurement of Hydraulic Conductivity of Porous Material Using a Rigid-Wall, Compaction-Mold Permeameter*.

5.4.9.3 Flexible Wall Permeability

For fine-grained soils, tests performed using a triaxial cell are generally preferred. In-situ conditions can be modeled by application of an appropriate confining pressure. The sample can be saturated using back pressuring techniques. Water is then allowed to flow through the sample and measurements are taken until steady-state conditions occur. Tests shall be performed in accordance with ASTM D 5084 - *Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter*.

5.4.10 Compaction Tests

There are two types of tests that can determine the optimum moisture content and maximum dry density of a soil; the Standard Proctor and the Modified Proctor. The results of the tests are used to determine appropriate methods of field compaction and to provide a standard by which to judge the acceptability of field compaction.

The results of the compaction tests are typically plotted as dry density versus moisture content. Tests have shown that moisture content has a great influence on the degree of compaction achieved by a given type of soil. In addition to moisture content, there are other important factors that affect compaction. The soil type has a great influence because of its various classifications, such as grain size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay mineral present. The compaction energy also has an affect because it too has various conditions, such as number of blows, number of layers, weight of hammer, and height of the drop.

5.4.10.1 Standard Proctor

This test method uses a 5-1/2-pound rammer dropped from a height of 12 inches. The sample is compacted in three layers. See ASTM D698 - *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))* (AASHTO T99 - *Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop*).

5.4.10.2 Modified Proctor

This test method uses a 10-pound rammer dropped from a height of 18 inches. The sample is compacted in five layers. See ASTM D1557 - *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3(2,700 kN-m/m^3))* (AASHTO T180 - *Standard Method of Test for Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop*).

5.4.11 Relative Density Tests

The relative density tests are most commonly used for granular or unstructured soils. It is used to indicate the in-situ denseness or looseness of the granular soil. In comparison, Proctor tests often do not produce a well-defined moisture-density curve for cohesionless, free-draining soils. Therefore relative density is expressed in terms of maximum and minimum possible dry unit weights and can be used to measure compaction in the field.

5.4.11.1 Maximum Index Density

In this test, soil is placed in a mold of known volume with a 2-psi surcharge load applied to it. The mold is then vertically vibrated at a specified frequency for a specified time. At the end of the vibrating period, the maximum index density can be calculated using the weight of the sand and the volume of the sand. See ASTM D4253 - *Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table*.

5.4.11.2 Minimum Index Density

The test procedure requires sand being loosely poured into a mold at a designated height. The minimum index density can be calculated using the weight of the sand required to fill the mold and the volume of the mold. See ASTM D4254 - *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*.

5.4.12 Electro-Chemical Tests

Electro-chemical tests provide quantitative information related to the aggressiveness of the subsurface environment, the surface water environment, and the potential for deterioration of foundation materials. Electro-chemical testing includes pH, resistivity, sulfate, and chloride contents. The electro-chemical tests should be performed on soil samples. In addition, surface water should also be tested in coastal regions where the potential intrusion of brackish (salt water) water may occur in tidal streams.

5.4.12.1 pH Testing

pH testing is used to determine the acidity or alkalinity of the subsurface or surface water environments. Acidic or alkaline environments have the potential for being aggressive on structures placed within these environments. Soil samples collected during the normal course of a subsurface exploration should be used for pH testing. Surface water samples shall be obtained in general accordance with standards published by the South Carolina Department of Health and Environmental Control. The pH of soils shall be determined using ASTM D4972 – *Standard Test Method for pH of Soils* (uses an aqueous method); ASTM G51 – *Standard Test Method for Measuring pH of Soils for Use in Corrosion Testing* (uses a nonaqueous method); or AASHTO T289 - *Standard Method of Test for Determining pH of Soil for Use in Corrosion Testing*. Any of these methods may be used; however, the laboratory shall be certified to perform the appropriate test method and shall indicate the method used on the laboratory results report. The surface water samples shall have the pH determined using ASTM D1293 – *Standard Test Methods for pH of Water*.

5.4.12.2 Resistivity Testing

Resistivity testing is used to determine the electric conduction potential of the subsurface environment. The ability of soil to conduct electricity can have a significant impact on the corrosion of steel piling. If a soil has a high potential for conducting electricity, then sacrificial anodes may be required on the structure. This type of testing can be performed in the laboratory or in the field. For the field testing procedure see Section 5.3.7.6 of this Manual. Resistivity shall be determined using ASTM G57 – S*tandard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method* or AASHTO T288 – *Standard Method of Test for Determining Minimum Laboratory Soil Resistivity*. The resistivity of surface water samples can be determined using ASTM D1125 – *Standard Test Method for Electrical Conductivity and Resistivity of Water*. As in pH testing, the surface water sample shall be obtained in accordance with sampling procedures prepared by the South Carolina Department of Health and Environmental Control.

5.4.12.3 Chloride Testing

Subsurface soils and surface water should be tested for chloride if the presence of sea or brackish water is suspected. Chloride testing for soils shall be determined using AASHTO T291 – *Standard Method of Test for Determining Water-Soluble Chloride Ion Content in Soil*. The chloride testing for the surface water shall be performed in accordance with ASTM D512 – *Standard Test Methods for Chloride Ion in Water*.

5.4.12.4 Sulfate Testing

Subsurface soils and surface water should be tested for sulfate. Sulfate testing for soils shall be determined using AASHTO T290 – *Standard Method of Test for Determining Water-Soluble Sulfate Ion Content in Soil*. The sulfate testing for the surface water shall be performed in accordance with ASTM D516 – *Standard Test Methods for Sulfate Ion in Water*.

5.4.13 Rock Cores

Rock coring is conducted when a soil boring encounters material that has a standard penetration resistance, N, exceeding 100 blows and is termed auger refusal. Typically rock coring is conducted to 10 feet into rock. At each core run, the length of the rock sample obtained and the distance the core run is drilled will give a recovery ratio. The recovery ratio is expressed in percentage with 100% being intact rock and 50% or below as highly fractured rock. Another way to evaluate rock is rock quality designation (RQD) which is also expressed in percentage. The RQD allows the Engineer to determine if compressive strengths can be performed at each core run. It is highly recommended to have rock coring done as close to the proposed shaft or pile as possible. South Carolina geology can have a rock formation that changes in a number of feet along the length or the width of the bridge.

5.4.13.1 Unconfined Compression Test

This test is performed on intact rock core specimens, usually with a rock sample length of 2 times the diameter. The specimen undergoes a confined compression or uniaxial compression. After the test, it provides data determining the strength of the rock, namely the uniaxial strength, shear strengths at varying pressures and varying temperatures, angle of internal friction, (angle of shearing resistance), and cohesion intercept. See ASTM D7012 - *Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures*.

5.5 QUALITY ASSURANCE/QUALITY CONTROL

The Quality Assurance/Quality Control (QA/QC) of the field and laboratory testing procedures/methods can have a significant impact on the results obtained from the testing. Therefore, all field and laboratory testing will require a QA/QC plan to be developed, maintained and implemented. The QA/QC plan shall follow the appropriate national, state or approved industrial standards.

5.5.1 Field Testing QA/QC Plan

All field testing shall be performed in accordance with an approved QA/QC plan. The plan shall at a minimum establish the calibration schedule for the equipment, the method of calibration and provide circumstances when calibration is required differently from the regularly scheduled calibration. The QA/QC plan shall be approved by the PCS/GDS with concurrence by the Office of Materials and Research.

5.5.2 Laboratory Testing QA/QC Plan

All laboratories conducting geotechnical testing shall be AASHTO Materials Reference Laboratory (AMRL) certified. The laboratories shall only conduct those tests for which the laboratory is certified. If the laboratory is not certified to conduct the test, the laboratory may contract to another laboratory that is certified. If no laboratory is certified, then a QA/QC plan for that particular test shall be developed and submitted to the Department for review and approval prior to testing. The QA/QC plan shall indicate which test method is being followed, the most recent calibration of the laboratory equipment to be used and the qualifications of the personnel performing the test. For tests where there is not an established ASTM, AASHTO or State testing standard, then the laboratory may use a testing method established by another Federal or State agency. The use of other agency standards shall be approved in writing by the Department prior to conducting the test. The laboratory requesting the use of another agency standard shall prove proficiency in the standard as well as submitting a QA/QC plan for the test method.

5.6 REFERENCES

Application of Geophysical Methods to Highway Related Problems, FHWA-IF-94-021, August 2004.

ASTM International 2006, 'D7012-04 Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures'.

Das, M. Braja, Principles of Geotechnical Engineering, 3rd edition, PWS Publishing Company, Boston MA, 1994.

Evaluation of Soil and Rock Properties, Geotechnical Engineering Circular No. 5, FHWA-IF-02-034, April 2002.

Sowers, George F., Introductory Soil Mechanics and Foundations: Geotechnical Engineering, 4th edition, Macmillan Publishing Co., Inc., New York, NY, 1970.

Sowers, George F. and Hedges, Charles S, Dynamic Cone for Shallow In-Situ Penetration Testing, Vane Shear and Cone Penetration Resistance Testing of In-Situ Soils, ASTM STP399, 1966

Spangler, Merlin G., and Handy, Richard L., Soil Engineering, 4th edition, Harper & Row, Publishers, New York, NY, 1982.

5.7 SPECIFICATIONS AND STANDARDS

Table 5-1, Specifications and Standards

Subject	ASTM	AASHTO	SCDOT
Limerock Bearing Ratio	-	-	-
Resilient Modulus of Soils and Aggregate Materials	-	T307	-
Absorption and Bulk Specific Gravity of Dimension Stone	C97	-	-
Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate	C127	T85	-
Standard Test Method for Particle-Size Analysis of Soils	D422	T88	-
Test Method for Shrinkage Factors of Soils by the Mercury Method	D427	T92	-
Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))	D698	T99	-
Standard Test Method for Specific Gravity of Soils	D854	T100	-
Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))	D1557	T180	SC-T-140
Standard Test Method for Unconfined Compressive Strength of Cohesive Soil	D2166	T208	-
Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock	D2216	T265	-
Standard Test Method for Permeability of Granular Soils (Constant Head)	D2434	T215	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils	D2435	T216	-
Standard Test Method for Triaxial Compressive Strength of Undrained Rock Core Specimens Without Pore Pressure Measurements	D2664	-	-
Standard Test Method for Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression	D2850	T296	-
Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens	D2938	-	SC-T-39

Table 5-1 (Continued), Specifications and Standards (Continued)

Subject	ASTM	AASHTO	SCDOT
Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils	D2974	T267	-
Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions	D3080	T236	-
Standard Test Method for Splitting Tensile Strength of Intact Rock Core Specimens	D3967	-	-
Standard Test Method for One-Dimensional Consolidation Properties of Soils Using Controlled-Strain Loading	D4186	-	-
Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table	D4253	-	-
Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density	D4254	-	-
Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils	D4318	T89 & T90	-
Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils	D4546	T258	-
Standard Test Method for Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil	D4648	-	-
Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils	D4767	T297	-
Standard Practices for Preserving and Transporting Rock Core Samples	D5079	-	-
Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter	D5084	-	-
Standard Test Method for pH of Soils	D4972	T289	-
Standard Test Method for pH of Soils for use in Corrosion Testing	G51	T289	-
Standard Test Methods for pH of Water	D1293	-	-
Standard Test Method for Determining Soil Resistivity	G57	T288	-
Standard Test Method for Electrical Conductivity and Resistivity of Water	D1125	-	-
Standard Test Method for Determining Chloride	D512	T291	-
Standard Test Method for Determining Sulfate	D516	T290	