CHAPTER 6

MATERIAL DESCRIPTION, CLASSIFICATION, AND LOGGING

GEOTECHNICAL DESIGN MANUAL

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Table of Contents

List of Tables

List of Figures

CHAPTER 6

MATERIAL DESCRIPTION, CLASSIFICATION, AND LOGGING

6.1 INTRODUCTION

Geomaterials (soil and rock) are naturally occurring materials used in highway construction by SCDOT. Understanding soil and rock behavior is critical to the design and construction of any project. Soil and rock classification is an essential element of understanding the behavior of geomaterials. Field explorations in South Carolina encounter 3 types of geomaterials (i.e., soil, IGM and rock).

Soil and rock are either unconsolidated or consolidated solid particles, respectively, while IGM is a material with both soil and rock characteristics and properties. Soil is the result of the weathering of rock and may be transported to another location or may be left in-place (i.e., residual soil). Consolidated soils typically have some degree of cementation while unconsolidated soils typically have no cementation. Rock is normally a durable, hard naturally occurring material. IGM is used only in the design of drilled shafts (see Chapter 16 for discussion on how IGM is applied to design). O'Neill, Townsend, Hassan, Buller and Chan (1996) defined IGM more specifically as:

- argillaceous geomaterials – heavily overconsolidated clays, clay shales, and saprolites that are prone to smearing when drilled
- calcareous rocks – limestone and limerock and argillaceous materials that are not prone to smearing when drilled
- very dense granular geomaterials – residual and completely decomposed rock with an SPT N-value between 50 and 100 blows per foot

The first 2 IGM types indicated above are considered Cohesive IGM, while the 3rd is considered Cohesionless IGM. The argillaceous IGMs composed of transported materials containing between 12 and 40 percent clay fraction (CF) while the saprolites are the result of in-situ chemical weathering of the parent rock material that contains between 12 and 40 percent CF. If design dictates that the type of IGM needs to be determined, then the percent CF shall be determined using ASTM D7928 (hydrometer analysis). The unconfined compressive strength, q_u, ranges from 5 tons per square foot (tsf) to 50 tsf; therefore, for a soil to be considered Cohesive IGM, both conditions (i.e., the CF and q_u) must be met for the argillaceous geomaterials. For calcareous rocks only q_u must be met (i.e., q_u ranges from 5 to 50 tsf) for the geomaterials to be considered cohesive IGM. The q_u shall be determined by laboratory shear strength testing on undisturbed samples. The use of field methods to determine shear strength shall be allowed only when approved in writing by the OES/GDS prior to the field testing. The Cohesionless IGM is treated as very dense sand in the design of drilled shafts (see Chapter 16).

As required in Chapter 4 and indicated in Chapter 5 soils are typically drilled using either hollow stem augers (HSA) or rotary wash (RW) methods (see Chapter 5 for drilling method to be used where). The problem in the field is when rock coring is required as opposed to other drilling methods. Coring shall begin at drilling refusal. An SPT shall be performed at drilling refusal. Drilling refusal is defined as the inability to advance the auger in areas where HSA are allowed. In borings using RW methods, drilling refusal is defined as the inability to advance a roller cone (tricone) bit.

As indicated in Chapter 5, there are numerous field and laboratory testing procedures used by SCDOT to explore project sites. Included in this Chapter is a discussion of the presentation of only some of these methods, specifically soil test borings (including SPT and rock coring results), CPT and DMT test results as well as results of field geophysical testing. For convenience, the classification of soil will be discussed first for the soil borings, CPT and DMT with the classification of rock following. In addition, figures indicating the presentation of the field data are included.

Details of the subsurface conditions encountered, including basic material descriptions and details of the drilling and sampling methods shall be recorded. See ASTM D5434 - *Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock*. During field exploration, specifically soil borings, a field log shall be kept of the materials encountered. In addition, the field log shall also include driller notes concerning the advancement of the test method (i.e., were hard layers encountered between SPT samples, etc.). The field personnel keeping the field logs shall have a minimum of 2 years of soil classification experience using ASTM D2488 – *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).* The exception to this is for rock coring. All rock coring shall be observed and all rock cores shall be logged by either a registered engineer or registered geologist with a minimum of 4 years of rock coring observation and logging experience. Daily, copies of driller field logs shall be scanned and forwarded to the GEOR for review. The GEOR, at his/her discretion, may make changes to the field operations based on observations from the field logs.

Upon delivery of the samples to the laboratory, a registered engineer or registered geologist shall verify and modify as necessary the material descriptions and classifications based on the results of a more detailed visual-manual inspection of samples. Draft logs shall only be submitted to the RPG/GDS after verification of the classifications in the laboratory. The RPG/GDS shall use the draft logs to assign laboratory testing as required for those projects conducted by the RPG/GDS. Classifications shall be further modified based on the results of the laboratory testing and final logs shall be prepared based on the revised classifications.

Material descriptions, classifications, and other information obtained during the subsurface explorations are heavily relied upon throughout the remainder of the investigation program and during the design and construction phases of a project. It is therefore necessary that the method of reporting this data be standardized. Records of subsurface explorations should follow as closely as possible the standardized formats presented in this Chapter.

This Chapter is divided into two primary sections, the first is associated with the description and classification of soil and the second section will discuss the description and classification of rock. The soil description and classification section will discuss the two soil classification systems used by SCDOT (i.e., the USCS and AASHTO).

6.2 SOIL DESCRIPTION AND CLASSIFICIATION

6.2.1 Soil Test Borings

A detailed description for each material stratum encountered should be included on the Soil Test Log (see Figures 6-14, 6-19 and 6-20) and on the Manual Auger Log (see Figures 6-18 and 6-21). The extent of detail will be somewhat dependent upon the material itself and on the purpose of the project. However, the descriptions should be sufficiently detailed to provide the GEOR with an understanding of the material present at the site. The descriptions should be sufficiently detailed to permit grouping of similar materials and aid in the selection of representative samples for testing.

Soils should be described with regard to soil type, color, relative density/consistency, etc. The description shall match the requirements of the Unified Soil Classification System (USCS) and the AASHTO soil classification system. A detailed soil description shall include the following items and shall match the descriptive terms discussed in the following sections, in order:

1. Relative Density/Consistency
2. Moisture Condition
3. Soil Color
4. Particle Angularity and Shape (for coarse-grained soils)
5. Hydrochloric (HCl) Reaction
6. Cementation
7. Gradation
    a. Coarse-Grained Soils
    b. Fine-Grained Soils
8. Unified Soil Classification System (USCS)
9. AASHTO Soil Classification System (AASHTO)
10. Other pertinent information

6.2.1.1 Relative Density/Consistency

Relative density refers to the degree of compactness of a coarse-grained soil. Consistency refers to the stiffness of a fine-grained soil. When evaluating subsurface soil conditions using correlations based on SPT N-values, the N-values shall be corrected (see Chapter 7 for corrections). However, only actual field recorded (uncorrected) SPT N-values (N_{meas}) shall be included on the Soil Test Boring Log and shall be used to determine the relative density and/or consistency.

Standard Penetration Test N-values (blows per foot) are usually used to define the relative density and consistency as follows:

Table 6-1, SPT Relative Density / Consistency Terms

Relative Density[1,2]			Consistency[1,3]		
Descriptive Term	**Relative Density**	**SPT Blow Count (bpf)[4]**	**Descriptive Term**	**Unconfined Compression Strength (q_u) (tsf)**	**SPT Blow Count (bpf)[4]**
Very Loose	0 to 15%	≤ 4	Very Soft	≤0.25	≤2
Loose	16 to 35%	5 to 10	Soft	0.26 to 0.50	3 to 4
Medium Dense	36 to 65%	11 to 30	Firm	0.51 to 1.00	5 to 8
Dense	66 to 85%	31 to 50	Stiff	1.01 to 2.00	9 to 15
Very Dense	86 to 100%	≥51	Very Stiff	2.01 to 4.00	16 to 30
			Hard	≥4.01	≥ 31
[1]For Classification only, not for design					
[2]Applies to coarse-grained soils (major portion retained on No. 200 sieve)					
[3]Applies to fine-grained soils (major portion passing No. 200 sieve)					
[4]bpf – blows per foot of penetration at 60 percent ER (see Chapter 7 for ER determination)					

6.2.1.2 Moisture Condition

The in-situ moisture condition shall be determined using the visual-manual procedure. The term "saturated" shall not be used, unless the degree of saturation is actually determined. The moisture condition is defined using the following terms:

Table 6-2, Moisture Condition Terms

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

6.2.1.3 Soil Color

The color of the soil shall be determined using the Munsell color chart and shall be described while the soil is still at or near the in-situ moisture condition. The Munsell color designation shall be provided at the end of the soils description.

6.2.1.4 Particle Angularity and Shape

Coarse-grained soils are described as angular, subangular, subrounded, or rounded. Gravel and cobbles can be described as flat, elongated, or flat and elongated. Descriptions of fine-grained soils will not include a particle angularity or shape.

Table 6-3, Particle Angularity and Shape

Descriptive Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges
Flat	Particles with a width to thickness ratio greater than 3
Elongated	Particles with a length to width ratio greater than 3
Flat and Elongated	Particles meeting the criteria for both Flat and Elongated

6.2.1.5 HCl Reaction

The terms presented below describe the reaction of soil with HCl (hydrochloric acid). Since calcium carbonate is a common cementing agent, a report of its presence on the basis of the reaction with dilute hydrochloric acid is important.

Table 6-4, HCl Reaction

Descriptive Term	Criteria
None	No visible reaction
Weakly	Some reaction, with bubbles forming slowly
Strongly	Violent reaction, with bubbles forming immediately

6.2.1.6 Cementation

The terms presented below describe the cementation of intact coarse-grained soils.

Table 6-5, Cementation

Descriptive Term	Criteria
Weakly Cemented	Crumbles or breaks with handling or little finger pressure
Moderately Cemented	Crumbles or breaks with considerable finger pressure
Strongly Cemented	Will not crumble or break with finger pressure

6.2.1.7 Gradation

The classification of soil is divided into 2 general categories based on gradation, coarse-grained and fine-grained soils. Coarse-grained soils (gravels and sands) have more than or equal to 50 percent (by weight) of the material retained on or above the No. 200 sieve, while fine-grained soils (silts and clays) have more than 50 percent of the material passing the No. 200 sieve. Gravels and sands are typically described in relation to the particle size of the grains. Silts and clays are typically described in relation to plasticity. The primary constituents are identified considering grain-size distribution. In addition to the primary constituent, other constituents which may affect the engineering properties of the soil should be identified. Secondary constituents are generally indicated as modifiers to the principal constituent (e.g., sandy clay or silty gravel, etc.). Other constituents can be included in the description using the terminology of ASTM D2488 through the use of terms such as trace (<5%), few (5-10%), little (15-25%), some (30-45%), and mostly (50-100%).

6.2.1.7.1 Coarse-Grained Soils

Coarse-grained soils are those soils with more than or equal to 50 percent by weight retained on or above the No. 200 sieve. Coarse-grained soils divided into 2 categories, well- and poorly-graded with the difference between well- and poorly-graded depending upon the Coefficient of Curvature (C_c) and the Coefficient of Uniformity (C_u). Coarse-grained soils with a C_c between 1 and 3 ($1 \leq C_c \leq 3$) and a C_u greater than or equal to 4 ($C_u \geq 4$) are considered to be well-graded. C_c and C_u are determined using the following equations.

$$\boldsymbol{C_c = \frac{(D_{30})^2}{[(D_{10})(D_{60})]}}$$ **Equation 6-1**

$$\boldsymbol{C_u = \frac{(D_{60})}{(D_{10})}}$$ **Equation 6-2**

Where,

D_{10} = Diameter of particle at 10% finer material, millimeters (mm)
D_{30} = Diameter of particle at 30% finer material, mm
D_{50} = Diameter of particle at 50% finer material, mm
D_{60} = Diameter of particle at 60% finer material, mm
D_{85} = Diameter of particle at 85% finer material, mm
% Fines = Percent passing the No. 200 Sieve

The D_{50} is the mean grain size and is used in scour analysis and is provided to the HEOR. The D_{10} is also termed the effective size of the soil. The D_{85} is used in the design of geosynthetic filtration requirements. The percent pass the No. 200 sieve is termed the fines content. The D_{10}, D_{30}, D_{50}, D_{60}, D_{85} and percent fines shall be graphically determined, if the data is present. If no data is present then the diameter at a specific percent finer shall be reported as unknown (UNK).

The particle size for gravels and sands are provided in Table 6-6 and the adjectives used for describing the possible combinations of particle size are provided in Table 6-7.

Table 6-6, Coarse-Grained Soil Constituents

Soil Component	Grain-size
Gravel	
Coarse	3” to ¾”
Fine	¾” to No. 4 sieve
Sand	
Coarse (c)	No. 4 to No. 10 sieve
Medium (m)	No. 10 to No. 40 sieve
Fine (f)	No. 40 to No. 200 sieve

Table 6-7, Adjectives For Describing Size Distribution

Particle-Size Adjective	Abbreviation	Size Requirements
Coarse	c	< 30% m/f Sand or < 12% f Gravel
Coarse to medium	c/m	< 12% f Sand
Medium to fine	m/f	< 12% c Sand and > 30% m Sand
Fine	f	< 30% m Sand or < 12% c Gravel
Coarse to fine	c/f	> 12% of each size

6.2.1.7.2 Fine-Grained Soils

Fine-grained soils are those soils with more than 50 percent passing the No. 200 sieve. Silt size particles range from the No. 200 Sieve (0.074 mm) to 0.002 mm ($0.002 \leq D \leq 0.074$). Clays have particle sizes less than 0.002 mm. These materials are defined using moisture-plasticity relationships that were developed in the early 1900's by the Swedish soil scientist A. Atterberg. Atterberg developed 5 moisture-plasticity relationships, of which 3 are used in engineering practice and are known as the Atterberg Limits. These limits are the shrinkage limit (SL), the plastic limit (PL) and the liquid limit (LL). The SL is defined as the moisture content at which there is no additional volume change in soil sample with further reduction in moisture content and is the moisture content when a soil behaves as a solid. The PL is defined as the moisture content at which a 1/8-inch diameter thread can be rolled out and at which the thread just begins to crumble and is the moisture content when soil begins behaving plastically. The LL is the moisture content at which a soil will flow when dropped a specified distance and a specified number of times and is the moisture content when a soil begins behave as fluid-like material and begins to flow. In addition, the plasticity index (PI) is the range between the liquid limit and the plastic limit (LL-PL). Figure 6-1 provides a chart indicating the relationship between increasing moisture content (X-axis) and increasing volume (Y-axis). The Plasticity Chart, Figure 6-2, is used to determine low and high plasticity and whether a soil will be Silt or Clay. If the results of the LL and PI plot above or to the left of the "U" Line, the testing procedure and results should be checked. Table 6-8 provides the adjectives used to describe plasticity and the applicable plasticity range.

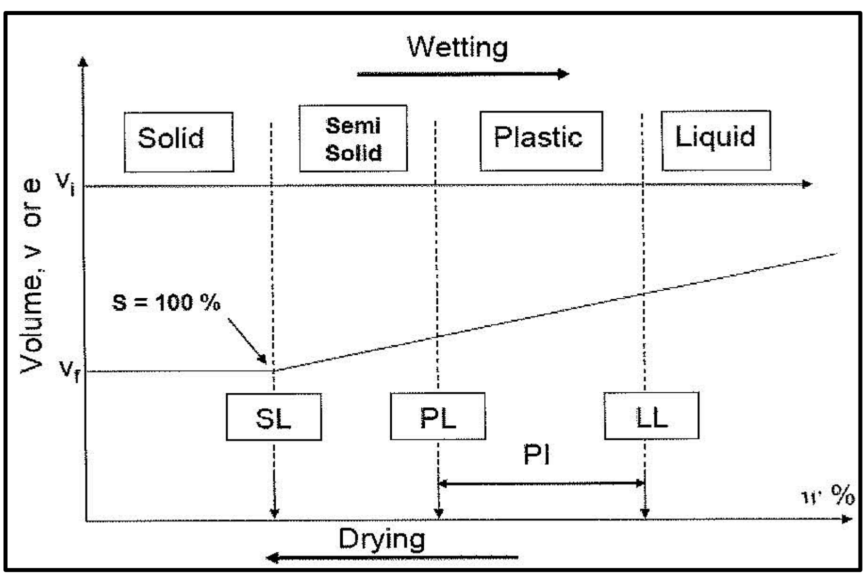


Figure 6-1, Moisture Content versus Volume Change

Because of the extremely hazardous nature of determining the SL (i.e., mercury is used), SL testing will typically not be performed. If SL testing is required, contact the OES/GDS for concurrence on the proposed testing method and provide an explanation as to how the results of the testing will be used or benefit the project.

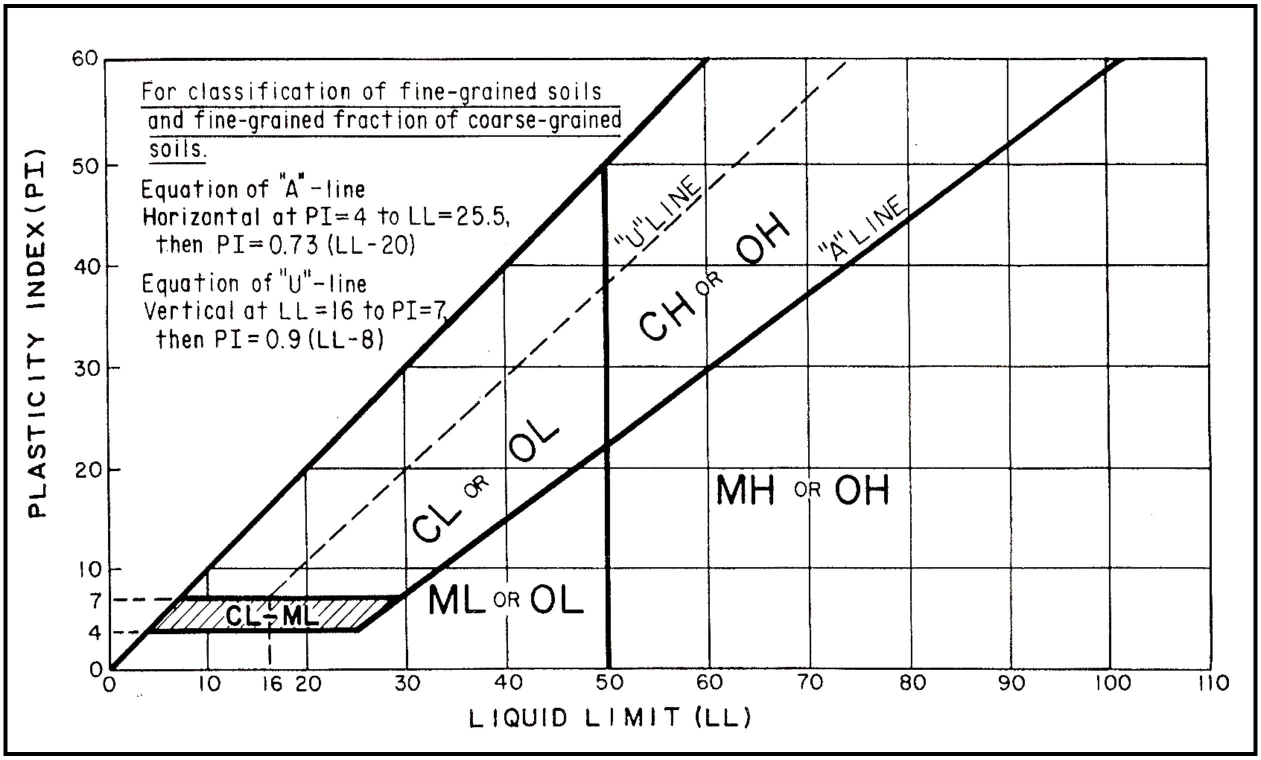


Figure 6-2, Plasticity Chart

Table 6-8, Soil Plasticity Descriptions

PI Range	Adjective	Dry Strength
0	non-plastic	none – crumbles into powder with mere pressure
1 – 10	low plasticity	low – crumbles into powder with some finger pressure
11 – 20	medium plasticity	medium – breaks into pieces or crumbles with considerable finger pressure
21 – 40	high plasticity	high – cannot be broken with finger pressure
> 41	very plastic	very high – cannot be broken between thumb and a hard surface

6.2.1.8 Unified Soil Classification System (USCS)

Dr. A. Casagrande developed the USCS for the classification of soils used to support Army Air Corps bomber bases. This system incorporates textural (grain-size) characteristics into the engineering classification. The system has 15 different potential soil classifications with each classification having a 2-letter designation. The basic letter designations are listed in Table 6-9.

Table 6-9, Letter Designations

Letter Designation	Meaning	Letter Designation	Meaning
G	Gravel	O	Organic
S	Sand	W	Well-graded
M	Non-plastic or low plasticity fines (Silt)	P	Poorly-graded
C	Plastic fines (Clay)	L	Low liquid limit
Pt	Peat	H	High liquid limit

The classification of soil is divided into 2 general categories, coarse-grained and fine-grained soils. Coarse-grained soils (gravels and sands) have more than or equal to 50 percent (by weight) of the material retained on the No. 200 sieve, while fine-grained soils (silts and clays) have more than 50 percent of the material passing the No. 200 sieve. Gravels and sands are typically described in relation to the particle size of the grains (See Section 6.2.1.7.1). Silts and clays are typically described in relation to plasticity (see Section 6.2.1.7.2).

In many soils, 2 or more soil types are present. When the percentage of the minor soil type is equal to or greater than 30 percent and less than 50 percent of the total sample (by weight), the minor soil type is indicated by adding a "y" to its name; i.e., Sandy SILT, Silty SAND, Silty CLAY, etc.

Figures 6-3, 6-4, 6-5, 6-6, and 6-7 provide the flow charts for the classification of coarse- and fine-grained soils using the USCS. See ASTM D2487 – *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).*

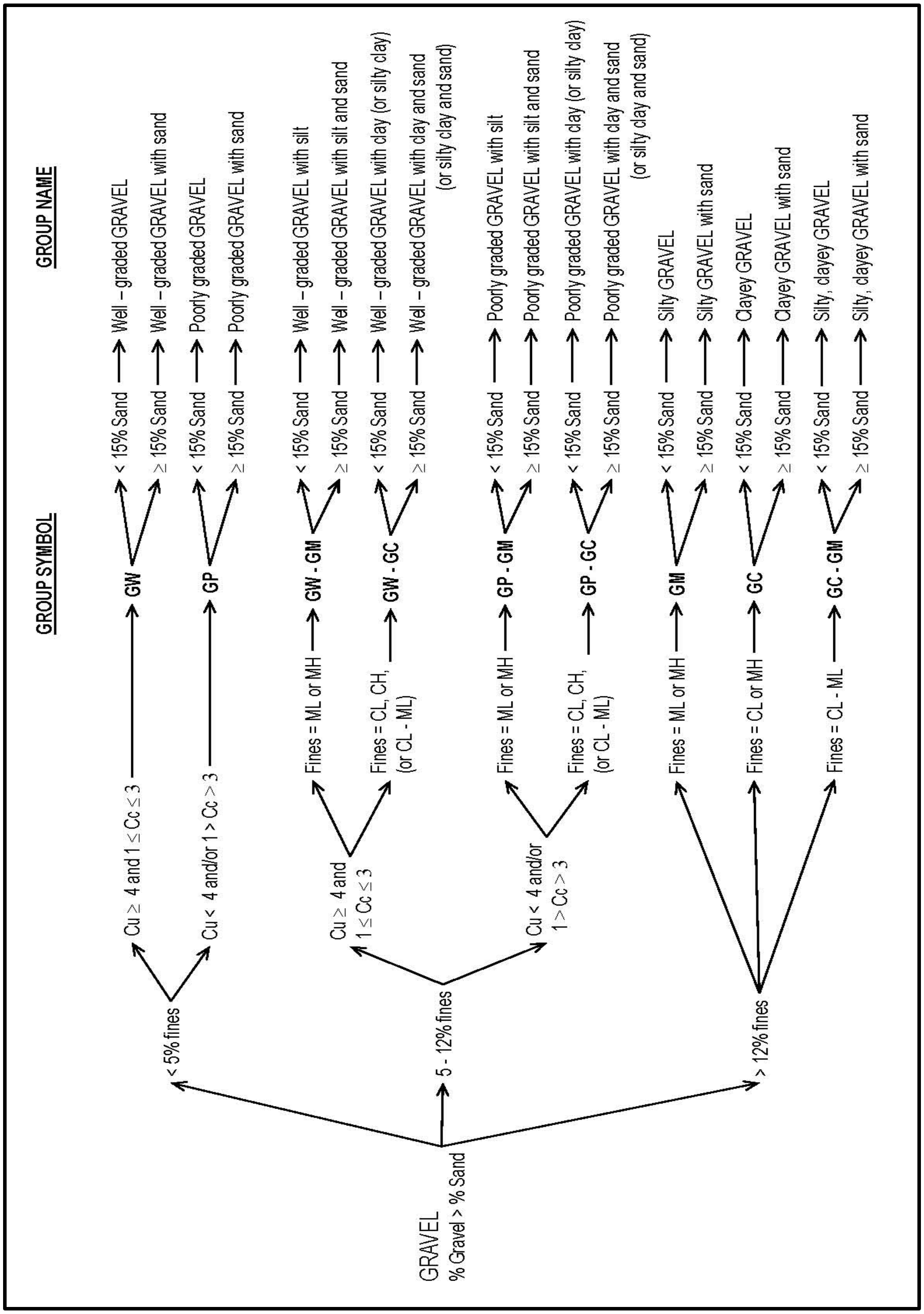


Figure 6-3, Group Symbol and Group Name Coarse-Grained Soils (Gravel) (Mayne, Christopher and DeJong (2002))

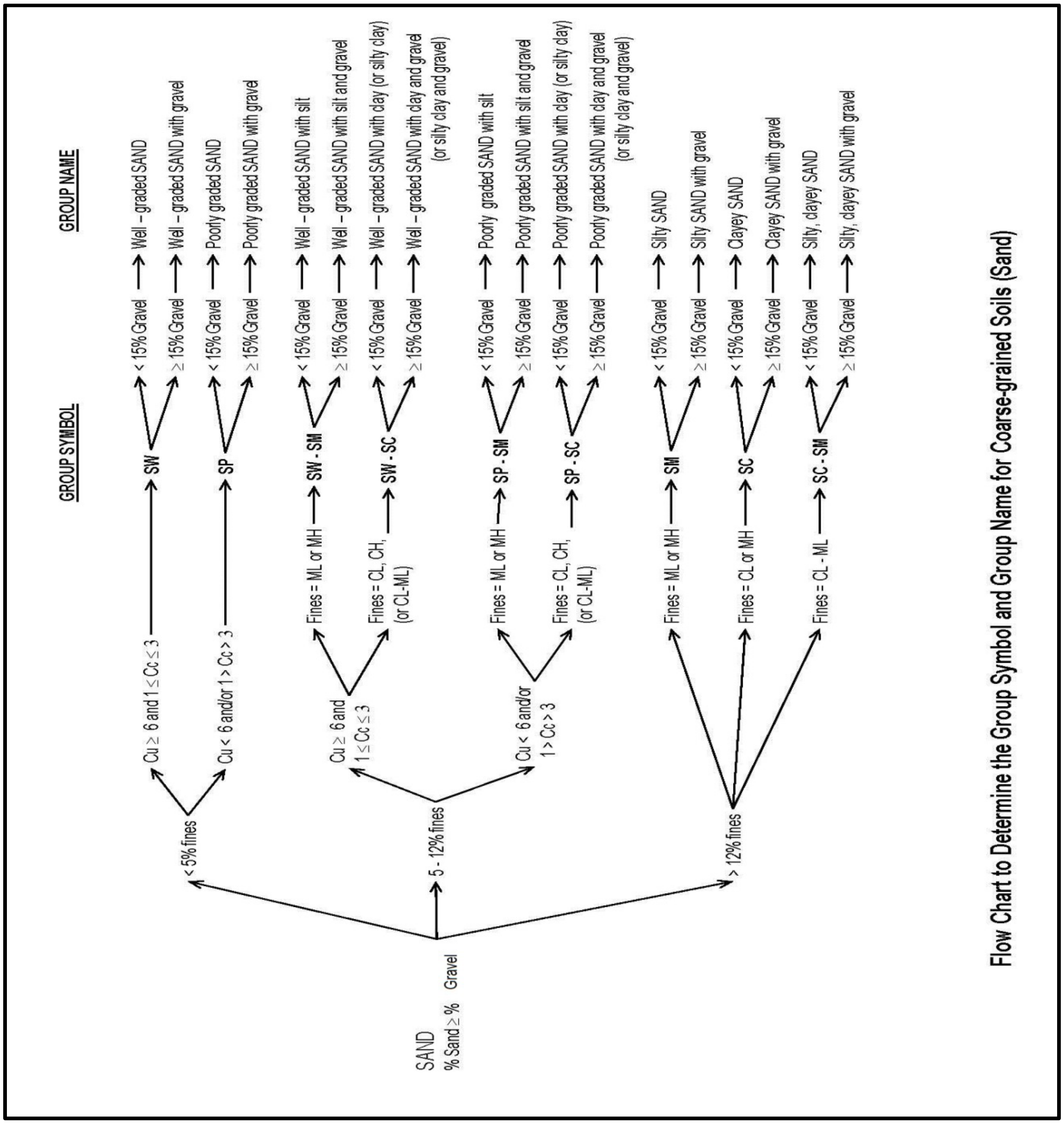


Figure 6-4, Group Symbol and Group Name for Coarse-Grained Soils (Sand) (modified Mayne, et al. (2002))

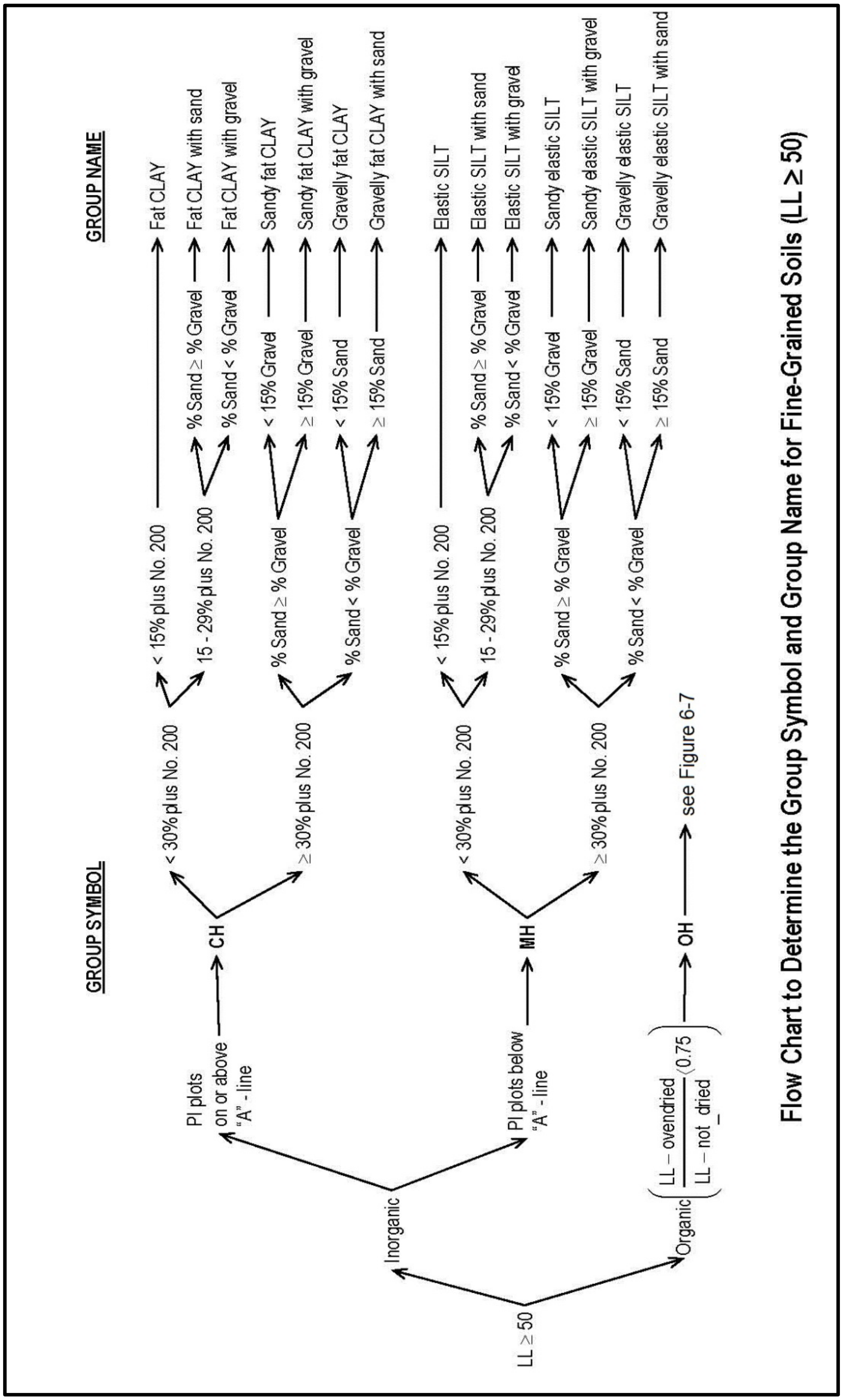
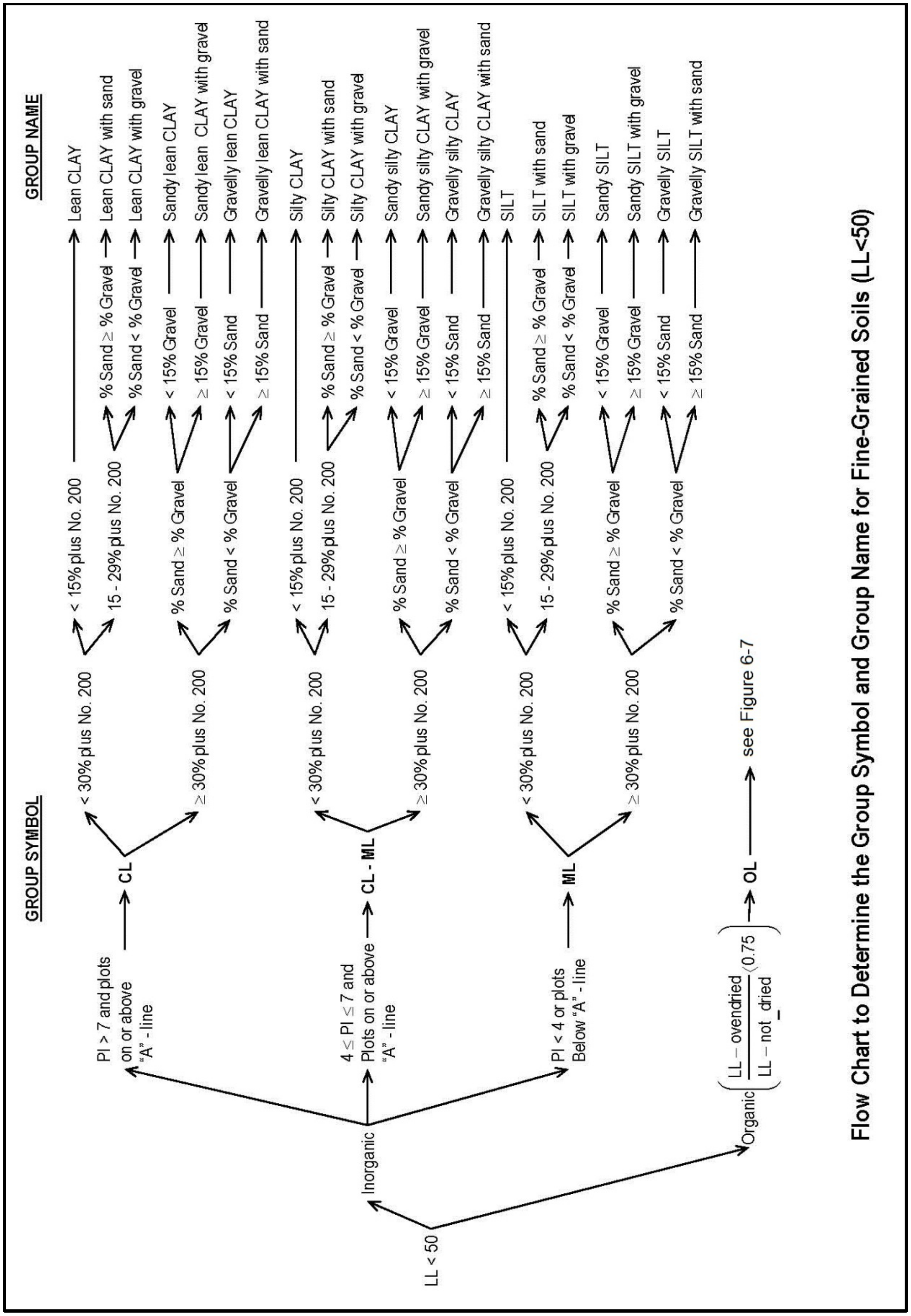


Figure 6-5, Group Symbol and Group Name for Fine-Grained Soils (LL ≥ 50) (Mayne, et al. (2002))



Flow Chart to Determine the Group Symbol and Group Name for Fine-Grained Soils (LL<50)

Figure 6-6, Group Symbol and Group Name for Fine-Grained Soils (LL < 50) (Mayne, et al. (2002))

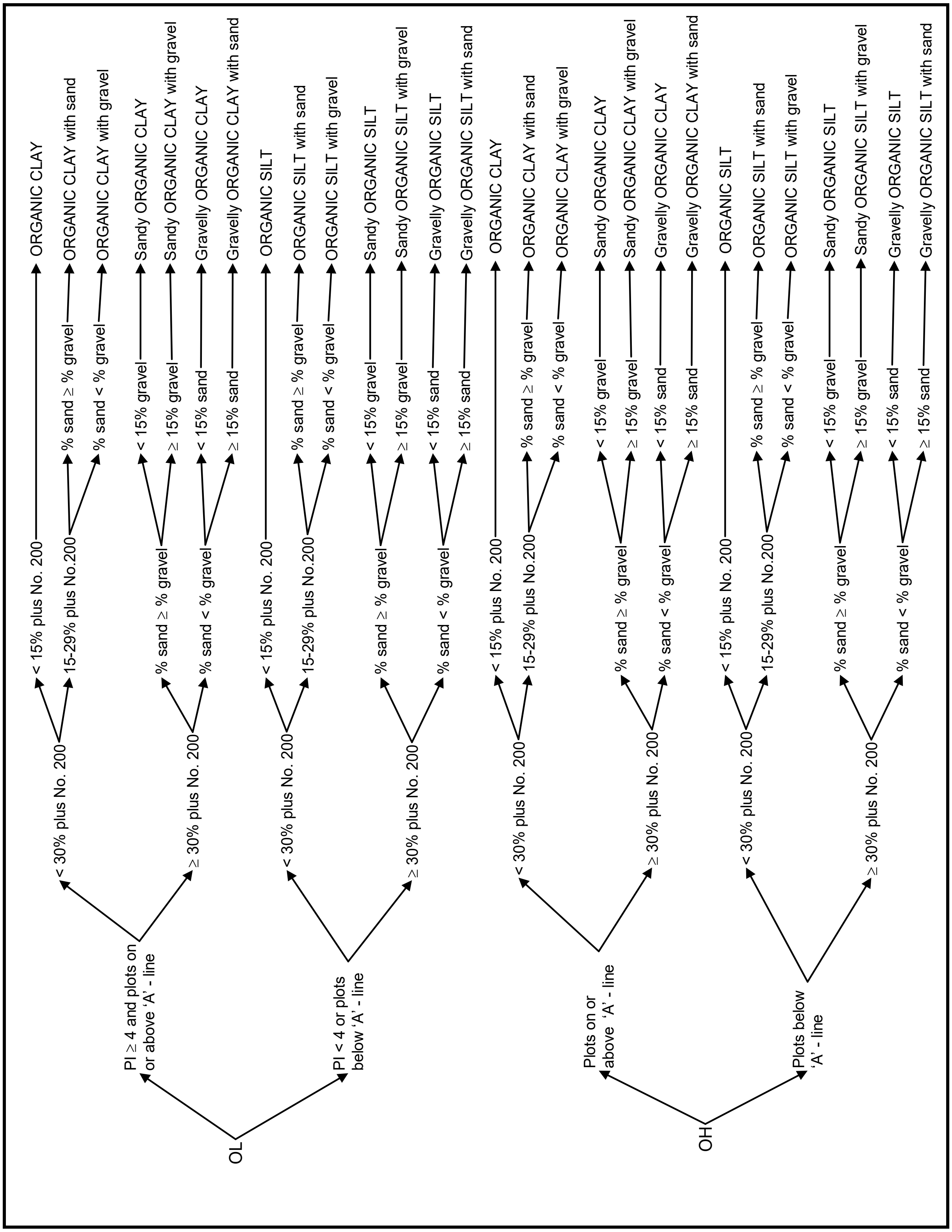


Figure 6-7, Group Symbol and Group Name for Organic Soils (Mayne, et al. (2002))

6.2.1.9 AASHTO Soil Classification System (AASHTO)

Terzaghi and Hogentogler originally developed this classification system for the U.S. Bureau of Public Roads in the late 1920s. This classification system divides all soils into 8 major groups designated A-1 through A-8 (see Figures 6-8 and 6-9). In this classification system, the lower the number the better the soil is for subgrade materials. Coarse-grained soils are defined by groups A-1 through A-3, while groups A-4 through A-7 define the fine-grained soils. Group A-4 and A-5 are predominantly silty soils and group A-6 and A-7 are predominantly clayey soils. Group A-8 refers to peat and muck soils.

Groups A-1 through A-3 have 35 percent or less passing the No. 200 sieve, while groups A-4 through A-7 have more than 35 percent passing the No. 200 sieve. The classification system is presented in Figure 6-9. Table 6-10 indicates the gradation requirements used in the AASHTO classification system. If a full grain-size analysis is not performed then the AASHTO soil classification system cannot be used.

Table 6-10, AASHTO Gradation Requirements

Soil Component	Grain-size
Gravel	between 3” to No. 10
Sand	between No. 12 to No. 200
Silt and Clay	less than No. 200

For soils in Groups A-2, A-4, A-5, A-6 and A-7 the plasticity of the fines is defined in Table 6-11.

Table 6-11, AASHTO Plasticity Requirements

Soil Component	Plasticity Index
Silty	≤ 10%
Clayey	≥ 11%

To evaluate the quality of a soil as a highway subgrade material, a number called the Group Index (GI) is incorporated with the groups and subgroups of the soil. The GI is written in parenthesis after the group or subgroup designation and is determined by the following equation:

Equation 6-3

$$GI = (F - 35)[0.2 + 0.005(LL - 40)] + 0.01(F - 15)(PI - 10)$$

Where:

F = percent passing No. 200 sieve (in percent)
LL = Liquid Limit
PI = Plasticity Index

Listed below are some rules for determining the GI:

- If the equation yields a negative value for the GI, use zero;
- Round the GI to the nearest whole number, using proper rules of rounding;
- For the upper limit of GI see Figure 6-9;
- Groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3, will always have a GI of zero;

- The GI for groups A-2-6 and A-2-7 is calculated using the following equation:

$$GI = 0.01(F - 15)(PI - 10) \quad \textbf{Equation 6-4}$$

Figure 6-7 provides the range of liquid limit and plasticity index for group A-2 to A-7 soils.

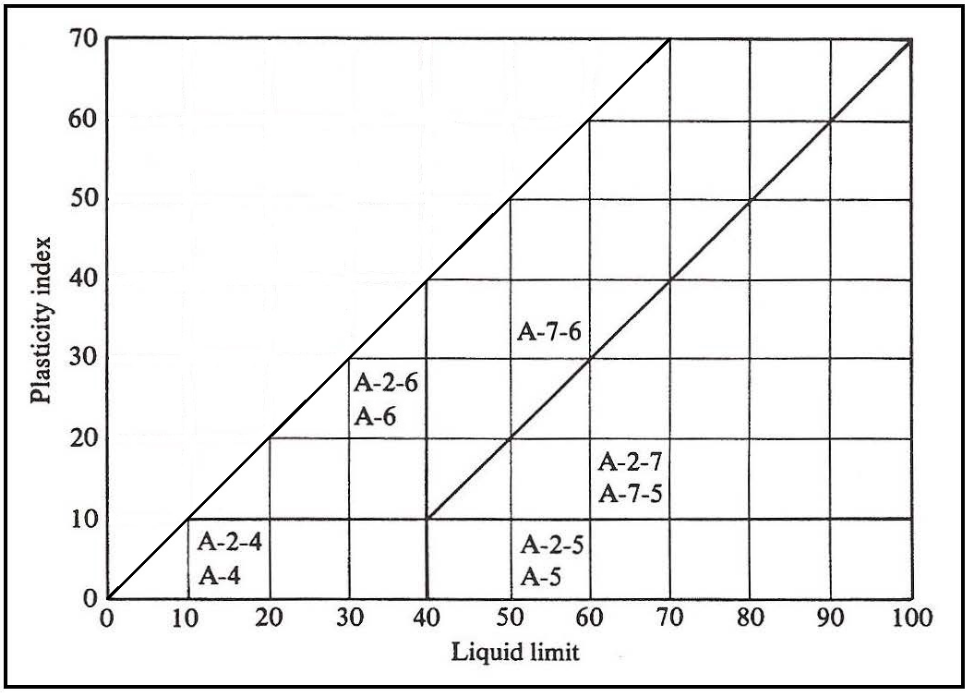


Figure 6-8, Range of LL and PI for Soils in Groups A-2 through A-7 (modified from Mayne, et al. (2002))

GENERAL CLASSIFICATION	GRANULAR MATERIALS (35 percent or less of total sample passing No. 200)							SILT-CLAY MATERIALS (More than 35 percent of total sample passing No. 200)			
GROUP CLASSIFICATION	A-1		A-3	A-2				A-4	A-5	A-6	A-7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A-7-5, A-7-6
Sieve analysis, percent passing:											
2 mm (No. 10)	50 max.										
0.425 mm (No. 40)	30 max.	50 max.	51 min.								
0.075 mm (No. 200)	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.	10 max.	10 max.	11 min.	11 min.*
Usual significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
Group Index**	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.

Classification procedure: With required test data available, proceed from left to right on chart; correct group will be found by process of elimination. The first group from left into which the test data will fit is the correct classification.

*Plasticity Index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity Index of A-7-6 subgroup is greater than LL minus 30 (see Fig 4-9).

**See group index formula (Eq. 4-1) Group index should be shown in parentheses after group symbol as: A-2-6(3), A-4(5), A-6(12), A-7-5(17), etc.

Figure 6-9, AASHTO Soil Classification System (Mayne, et al. (2002))

6.2.1.10 Organic Soil Classifications

Organic soils may be typically identified as having a distinctive odor, color (dark brown or gray to black) and potentially visible organic matter (i.e., small or fine roots, or other small organic matter). In addition, organic soils also have the ability to retain water which results in high water contents, high primary and secondary consolidation settlement, low to minimal shearing capacity and the potential for having an aggressive electro-chemical response. Huang, Patel, Santagata, and Bobet (2009) proposed the classification system indicated in Table 6-12.

Table 6-12, Organic Soil Classification
(Huang, et al. (2009))

Organic Content (%)	Soil Designation
≤ 3	Mineral Soil
3 to ≤ 15	Mineral Soil with Organic Matter
15 to ≤ 30	Organic Soil
> 30	Highly Organic Soil (Peat)

Classify all soils in accordance with both the USCS and AASHTO soil classification systems. In addition to the standard soil classification designations, if the soil has between 3 and 15 percent organics add an “O” to the end of the classification designation (e.g., CL-O (lean CLAY with organics) or A-7-6-O). If the organic content is greater than 15 but less than or equal to 30 percent, add a prefix “O” before the designation (e.g., O-CL (organic lean CLAY) or O-A-7-6). For soils with more than 30 percent organics follow the requirements of the USCS or AASHTO soil classification systems for determining the soil classification designation as well as the naming nomenclature. However, Peat soils will typically have more than 50 percent fiber content and specific gravity less than 1.7 with very high moisture contents (> 500%).

6.2.1.11 Soil Electro-Chemical Classifications

Electro-chemical testing is required for soil and water samples collected from project sites, in accordance with the requirements contained in Chapter 5 so that appropriate materials may be used on the project. Electro-chemical testing consists of pH, resistivity and sulfate and chloride contents. The aggressiveness or non-aggressiveness of a site shall be determined using Table 7-34. In addition, to the electro-chemical tests, the location of the ground water table should also be noted. Fluctuations in the ground water table may lead to aggressive soil environments by allowing increased oxygen content around the foundation. The results of all electro-chemical testing shall be reported to the SEOR and project team for their consideration in the design of the structure.

6.2.1.12 Other Pertinent Information

Additional information that adds to the description of the soil may be included. This information should enhance the soil description. This may include the geologic formation to which the soil belongs. The determination and designation of geologic formations is the responsibility of the GEOR and not the GEC providing the field and laboratory services. The depth to ground water at both the time of boring and approximately 24 hours after drilling are required to be indicated on the Soil Test Boring Log. In some cases the borehole collapses prior to obtaining the ground water reading. The depth of caving shall be indicated on the Soil Test Boring Log. For Sand-Like soils the caved depth may be interpreted as the depth of ground water. In Clay-Like soils the depth to ground water may be interpreted as possibly within 3 or 4 feet above or below the caved depth. The Soil Test Boring Log should also indicate if artesian conditions are encountered and what the estimated artesian head is.

6.2.2 Cone Penetrometer Test

The Cone Penetrometer Test shall be conducted in accordance with Chapter 5. The penetrometer data is plotted showing the tip stress (q_t – corrected), the friction resistance (f_s – measured), the friction ratio (R_f) and the pore pressures vs. depth (see Figure 6-24). Typically, the cone penetrometers used in South Carolina have a porous element located just behind the cone tip (shoulder) as depicted in Figure 6-10. Prior to using a cone penetrometer with a different porous element location, approval shall be obtained from the OES/GDS. In addition, to the plotted penetrometer data, the GEC shall provide to the RPG/GDS an electronic file in Excel® format providing the following data in the order shown:

1. Depth, feet
2. q_c – Uncorrected/measured tip resistance, tons per square foot (tsf)
3. f_s – Measured friction resistance, tsf
4. u_2 – Pore pressure behind tip, tsf
5. u_0 – Hydrostatic pore pressure, tsf
6. q_t – Corrected tip resistance (see Equation 6-5), tsf
7. R_f – Friction ratio (see Equation 6-6), percent
8. σ_{vo} – Total overburden stress, tsf
9. σ'_{vo} – Effective overburden stress, tsf
10. B_q – Pore pressure parameter, dimensionless (see Equation 7-15)
11. Q_T – Normalized tip resistance, dimensionless (see Equation 7-13)
12. F_R – Normalized sleeve resistance, dimensionless (see Equation 7-14)
13. I_c – Soil behavior type, dimensionless (see Equation 7-17)
14. Zone # corresponding to I_c, dimensionless (see Figure 6-11 and Table 6-12)
15. N_{60} – Estimated N-value at 60 percent energy, bpf (see Equation 7-21)
16. N_k – Cone factor as known as N_{kt}, dimensionless
17. $(S_u)_{cpt}$ – Undrained shear strength, pounds per square foot (psf) (see Equation 7-33)
18. ϕ' – Effective friction angle, degree (see Equation 7-46)
19. S_t – Sensitivity, dimensionless (see Equation 7-40)
20. V_s – Shear wave velocity, feet per second (fps) (if measured)
21. V_p – Compression wave velocity, feet per second (fps) (if measured)

The Excel® spreadsheet shall also include in the heading the following information:

1. SCDOT Project Number
2. Project Name
3. Station
4. Offset including right or left
5. Latitude
6. Longitude

7. Elevation (NAVD 88)
8. Any other information that identifies the project

Further the GEC shall indicate the equations used for all normalized parameters and correlations and how u_0, σ_{vo} and σ'_{vo} were determined. The correlations shall conform to the requirements of Chapter 7.

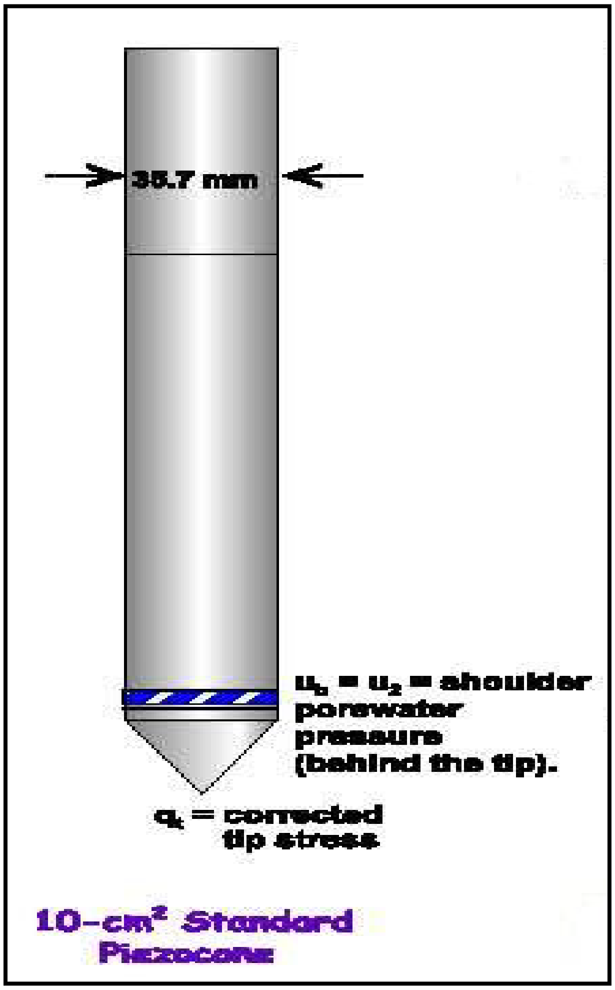


Figure 6-10, Standard Electro-Piezocone (Mayne, et al. (2002))

$$q_t = q_c + (1 - a_n) * u_2 \qquad \textbf{Equation 6-5}$$

$$R_f = \frac{f_s}{q_t} * (100\%) \qquad \textbf{Equation 6-6}$$

Where:

a_n = Net area ratio developed from calibration testing

Provide the a_n value used to compute the corrected tip resistance and the cone factor (N_k) used to compute the undrained shear strength in the Excel® spreadsheet. Similarly to Soil Test Borings, the CPT can be used to classify the soils at a site. However, the classification is based on soil behavior rather than grain-size and plasticity and the various classification systems yield a Soil Behavior Type (SBT or I_c) rather than a USCS soil type. The basic classification is between coarse-grained and fine-grained soils, the differences are indicated below:

1. Coarse-grained
    a. High end resistance, tip stress, (q_c)
    b. Low Friction Ratio, (R_f)
    c. Low pore pressure, (u_2)
2. Fine-grained
    a. Low end resistance, tip stress, (q_c)
    b. High Friction Ratio, (R_f)
    c. High pore pressure, (u_2)

Soil classifications are based on the relationship between normalized Friction Ratio (F_R (F_r in Figure 6-11)) and normalized tip resistance (Q_t (Q_{tn} in Figure 6-11)) as shown in Figure 6-11. Table 6-13 provides the description of the soils by zone as well as the I_c for each zone. Similarly to Soil Test Borings, the relative density and/or consistency can be assigned to a soil layer. The relative density and/or consistency is based on the corrected tip resistance (q_t). Table 6-14 provides the relative density/consistency versus correct tip resistance.

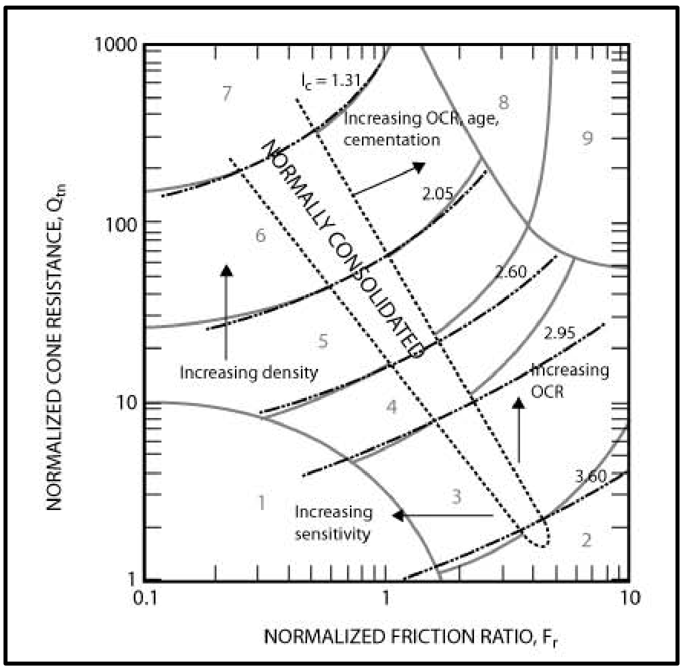


Figure 6-11, Normalized CPT Soil Behavior Chart Using Q_T versus F_R (Robertson and Cabal (2015))

Table 6-13, CPT Soil Behavior Type (Robertson and Cabal (2015))

Soil Behavior Type			
Zone #	Description	I_c	
		Min	Max
1	Sensitive, fine-grained	N/A	
2	Organic soils – peats	≥ 3.6	
3	Clays – Silty Clay to Clay	2.95	3.59
4	Silt mixtures – Clayey Silt to Silty Clay	2.60	2.94
5	Sand mixtures – Silty Sand to Sandy Silt	2.05	2.59
6	Sands – clean Sand to Silty Sand	1.31	2.04
7	Gravelly Sand to dense Sand	≤ 1.30	
8	Very stiff Sand to Clayey Sand (high OCR or cemented)	N/A	
9	Very stiff, fine-grained (high OCR or cemented)	N/A	

Table 6-14, CPT Relative Density / Consistency Terms

Relative Density[1,2]			Consistency[1,3]	
Descriptive Term	Relative Density	q_t[4] (tsf)	Descriptive Term	q_t[4] (tsf)
Very Loose	0 to 15%	≤ 50	Very Soft	≤ 5
Loose	16 to 35%	51 to 100	Soft to Firm	6 to 15
Medium Dense	36 to 65%	101 to 150	Stiff	16 to 30
Dense	66 to 85%	151 to 200	Very Stiff	31 to 60
Very Dense	86 to 100%	≥ 201	Hard	≥ 61
[1]For Classification only, not for design				
[2]Applies to coarse-grained soils (major portion retained on No. 200 sieve)				
[3]Appiles to fine-grained soils (major portion passing No. 200 sieve)				
[4]Corrected Tip Resistance				

6.2.3 Dilatometer Test

The Dilatometer Test (DMT) shall be conducted in accordance with Chapter 5. In addition, to the plotted dilatometer data (see Figure 6-25); the GEC shall provide to the RPG/GDS an electronic file in Excel® format providing the following data in the order shown (1 bar ≈ 1 tsf):

1. Depth, feet
2. A-pressure, bars
3. B-pressure, bars
4. C-pressure, bars
5. ΔA – Corrections from membrane calibration, bars
6. ΔB – Corrections from membrane calibration, bars
7. p_0 – Corrected A-pressure (see Equation 6-7), bars
8. p_1 – Corrected B-pressure (see Equation 6-8), bars
9. p_2 – Corrected C-pressure (see Equation 6-9), bars
10. Z_M – Pressure gauge reading when vented to atmospheric pressure, bars
11. q_d – Corrected thrust required to insert dilatometer, tons

12. σ_{vo} – Total overburden stress, tsf
13. σ'_{vo} – Effective overburden stress, tsf
14. u_0 – Equilibrium pore pressure, tsf
15. I_D – Material index (soil type), dimensionless
16. K_D – Horizontal stress index, dimensionless
17. E_D – Dilatometer Modulus, bars
18. U_D – Pore Pressure Index, dimensionless
19. $(S_u)_{DMT}$ – Undrained shear strength, psf

The Excel® spreadsheet shall also include in the heading the following information:

1. SCDOT Project Number
2. Project Name
3. Station
4. Offset including right or left
5. Latitude
6. Longitude
7. Elevation
8. Any other information that identifies the project

Further the equations for determining the previous correlations shall be indicated. The GEC shall also indicate how σ_{vo} and σ'_{vo} were determined. The correlations shall conform to the requirements of Chapter 7. Through developed correlations (see Chapter 7), 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.

Where:

p_0 – Corrected A-pressure

$$\boldsymbol{p_0 = 1.05 * (A - Z_M + \Delta A) - 0.05 * (B - Z_M - \Delta B)} \qquad \textbf{Equation 6-7}$$

p_1 – Corrected B-pressure

$$\boldsymbol{p_1 = (B - Z_M - \Delta B)} \qquad \textbf{Equation 6-8}$$

p_2 – Corrected C-pressure (u_0 – Equilibrium pore pressure)

$$\boldsymbol{u_0 = p_2 = (C - Z_M + \Delta A)} \qquad \textbf{Equation 6-9}$$

Similarly to CPT, the DMT can be used to classify the soils at a site based on behavior. Soil classifications are based on the material index (I_D) as indicated in Table 6-15.

Table 6-15, DMT Material Index (Marchetti, et al. (2001))

Soil Type	Material Index, (I_D)	
	Min	Max
Clay	0.1	0.6
Silt	0.6	1.8
Sand	$\geq$ 1.8	

Another general indicator of soil type is the pore pressure index (U_D). A U_D of between 0.0 and approximately 0.2 indicates that the soils are "free-draining". "Free-draining" (permeable) soils are typically coarse-grained (i.e., clean sands and gravels) soils. Impermeable soils are typically fine-grained (clays (lean and fat) and elastic silts) soils and have a U_D of 0.7 or greater. Soils with a U_D between 0.2 and 0.7 have an intermediate permeability. A wide range of soils can have an intermediate permeability. U_D provides a general indication of soil type and is not considered exact; therefore, U_D should be used in conjunction with I_D to determine soil type.

6.3 ROCK DESCRIPTION AND CLASSIFICATION

Rock descriptions should use technically correct geologic terms, although accepted local terminology may be used provided the terminology helps to describe distinctive characteristics. Rock cores shall be logged when wet for consistency of color description and greater visibility of rock features. Geologists classify all rocks according to their origin and into 3 distinctive types as indicated in Table 6-16. All 3 rock types are found here in South Carolina: igneous rocks are found in the Piedmont region, metamorphic rocks are found in the Piedmont and Blue Ridge regions, and sedimentary rocks are found in the Coastal Plain. The Department uses both the geological history as well as the engineering properties to describe rock materials.

Table 6-16, Rock Classifications

Rock Type	Definition
Igneous	Derived from molten material
Metamorphic	Derived from preexisting rocks due to heat, fluids, and/or pressure.
Sedimentary	Derived from settling, depositional, or precipitation processes

The geologic conditions of South Carolina have a direct bearing on the activities of SCDOT. This is because the geological history of a rock will determine its mechanical behavior. Therefore, construction costs for a project, especially a new project with substantial foundation construction, are frequently driven by geological, subsurface factors. It is for this reason that much of the initial site investigation for a project requiring foundation work focuses on mechanical behavior of the subsurface materials within the construction limits. A detailed geologic description shall include the following items, in order:

1. Rock Type
2. Rock Color
3. Grain-Size and Shape
4. Texture (stratification/foliation)

5. Mineral Composition
6. Weathering and Alteration
7. Strength
8. Rock Discontinuity
9. Rock Fracture Description
10. Other pertinent information
11. Geologic Strength Index
12. Rock Mass Rating

In addition to the above information being included on the boring record, a photographic log of the cores shall also be provided. The photographic log shall be obtained in the field upon completion of the specific core run. The top and bottom of each individual core run shall be clearly labeled. The label shall include the top and bottom depth of each core run as well as the core run number. A tape measure or ruler shall be placed cross the top or bottom edge of the core box to provide a scale for the photograph. The ruler shall be large enough and provide enough contrast to allow for differentiation between the markings on the ruler. All breaks that occur during coring or are required to fit the core run into the core box shall be indicated to be mechanical breaks.

Rock Quality Designation (RQD) is used to indicate the quality of the rock and is frequently accompanied with descriptive words. It is always expressed as a percent. Percent recovery can be greater than 100 percent if the core from a prior run is recovered during a later run. Figure 6-12 further illustrates the determination of the RQD.

In addition, rock may be classified as soft, weathered or hard based on the shear wave velocity (V_s) for use in seismic design. Provided in Table 6-17 are the rock definitions to be used in seismic designed based on the V_s of the rock. Please note these are approximations and are not to be used to determine shear strength of the rock, but instead are intended as a guide for use in seismic design.

Table 6-17, Rock Classifications for Seismic Design

Definition	Vs (ft/s)
Soft	≤ 2,500 to < 8,200
Weathered	≤ 8,200 to < 11,500
Hard	≤ 11,500

6.3.1 Rock Type

The rock type shall be identified by either a licensed geologist or geotechnical engineer with a minimum of 4 years of experience classifying rock. Rocks are classified according to origin into the 3 major groups: igneous, sedimentary and metamorphic. These groups are subdivided into types based on mineral and chemical composition, texture, and internal structure.

6.3.1.1 Igneous

Intrusive, or plutonic, igneous rocks have coarse-grained (large, intergrown crystals) texture and are believed to have been formed below the earth's surface. Granite and gabbro are examples of intrusive igneous rocks found in South Carolina. Extrusive, or volcanic, igneous rocks have fine-grained (small crystals) texture and have been observed to form at or above the earth's surface. Basalt and tuff are examples of an extrusive igneous rocks found in South Carolina. Pyroclastic igneous rocks are the result of a volcanic eruption and the rapid cooling of lava, examples of this type of rock are pumice and obsidian. Pyroclastic igneous rocks are not native to South Carolina.

6.3.1.2 Metamorphic

Metamorphic rocks result from the addition of heat, fluid, and/or pressure applied to preexisting rocks. This rock is normally classified into 3 types, strongly foliated, weakly foliated, and nonfoliated. Foliation refers to the parallel, layered minerals orientation observed in the rock. Schist is an example of a strongly foliated rock. Gneiss (pronounced "nice") is an example of a weakly foliated rock, while marble is an example of a nonfoliated rock. Schist, gneiss, slate and marble are metamorphic rocks found in South Carolina.

6.3.1.3 Sedimentary

Sedimentary rocks are the most common form of rock and are the result of weathering of other rocks and the deposition of the rock sediment and soil. Sedimentary rocks are classified into 3 groups called clastic, chemical, and organic. Clastic rocks are composed of sediment (from weathering of rock or erosion of soil). Mudstone and sandstone are examples of clastic sedimentary rock found in South Carolina. Chemical sedimentary rocks are formed from materials carried in solution into lakes and seas. Limestone, dolomite, and halite are examples of this type of sedimentary rock. Organic sedimentary rocks are formed from the decay and deposition of organic materials in relatively shallow water bodies. Examples of organic sedimentary rocks are chalk, shale, coal, and coquina. Coquina is found within South Carolina.

6.3.2 <u>Rock Color</u>

The color of the rock shall be determined using the Munsell Color Chart and shall be described while the rock is still at or near the in-situ moisture condition. The Munsell color designation shall be provided at the end of the rock description.

6.3.3 <u>Grain-size and Shape</u>

Grain-size is dependent on the type of rock as described previously; sedimentary rocks will have a different grain-size and shape, when compared to igneous rocks. Metamorphic rocks may or may not display relict grain-size of the original parent rock. The grain-size description should be classified using the terms presented in Table 6-18. Angularity is a geologic property of particles and is also used in rock classification. Table 6-19 shows the grain shape terms and characteristics used for sedimentary rocks.

Table 6-18, Grain-size Terms

Description	Diameter (mm)	Characteristic
Very coarse-grained	> 4.75	Grain-sizes greater than popcorn kernels
Coarse-grained	2.00 – 4.75	Individual grains easy to distinguish by eye
Medium grained	0.425 – 2.00	Individual grains distinguished by eye
Fine-grained	0.075 – 0.425	Individual grains distinguished with difficulty
Very fine-grained	< 0.075	Individual grains cannot be distinguished by unaided eye

Table 6-19, Grain Shape Terms for Sedimentary Rocks

Description	**Characteristic**
Angular	Shows little wear; edges and corners are sharp, secondary corners are numerous and sharp
Subangular	Shows definite effects of wear; edges and corners are slightly rounded off; secondary corners are less numerous and less sharp than angular grains
Subrounded	Shows considerable wear; edges and corners are rounded to smooth curves; secondary corners greatly reduced and highly rounded
Rounded	Shows extreme wear; edges and corners smoother to broad curves; secondary corners are few and rounded
Well-rounded	Completely worn; edges and corners are not present; no secondary edges

6.3.4 <u>Texture (stratification/foliation)</u>

Significant nonfracture structural features should be described. Stratification refers to the layering effects within sedimentary rocks, while foliation refers to the layering within metamorphic rocks. The thickness of the layering should be described using the terms of Table 6-20. The orientation of the stratification/foliation should be measured from the horizontal with a protractor.

Table 6-20, Stratification/Foliation Thickness Terms

Descriptive Term	Layer Thickness
Very Thickly Bedded	>1.0 m
Thickly Bedded	0.5 to 1.0 m
Thinly Bedded	50 to 500 mm
Very Thinly Bedded	10 to 50 mm
Laminated	2.5 to 10 mm
Thinly Laminated	<2.5 mm

6.3.5 <u>Mineral Composition</u>

The mineral composition shall be identified by a geologist or geotechnical engineer based on experience and the use of appropriate references. The most abundant mineral should be listed first, followed by minerals in decreasing order of abundance. For some common rock types, mineral composition need not be specified (e.g., dolomite and limestone).

6.3.6 Weathering and Alteration

Weathering as defined here (see Table 6-21) is due to physical disintegration of the minerals in the rock by atmospheric processes while alteration is defined here as due to geothermal processes.

Table 6-21, Weathering/Alteration Terms

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered / Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although the original fabric may be intact; material can be granulated by hand
Highly Weathered / Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 1-7/8 inch diameter sample can be easily broken readily by hand across rock fabric
Moderately Weathered / Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 1-7/8 inch diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered / Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering / alteration

6.3.7 Strength

Table 6-22 presents guidelines for common qualitative assessment of strength while mapping or during primary logging of rock cores at the site by using a geologic hammer and pocketknife. The field estimates should be confirmed where appropriate by comparisons with selected laboratory test.

Table 6-22, Rock Strength Terms

Description	Recognition	Approximate Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Can be indented by thumbnail	35 – 150
Very Weak Rock	Can be peeled by pocket knife	150 –700
Weak Rock	Can be peeled with difficulty by pocket knife	700 – 3,500
Medium Strong Rock	Can be indented 3/16 inch with sharp end of pick	3,500 – 7,200
Strong Rock	Requires one hammer blow to fracture	7,200 – 14,500
Very Strong Rock	Requires many hammer blows to fracture	14,500 – 35,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 35,000

A popular classification system based on quantifying discontinuity spacing is known as the RQD (see ASTM D6032 – *Standard Test Method for Determining Rock Quality Designation (RQD) of Rock Core)*. RQD is illustrated in Figure 6-12 and is defined as the total combined length of all the pieces of the intact core that are longer than twice the diameter of the core (normally 2 inches) recovered during the core run divided by the total length of the core run (e.g., the summation of rock pieces greater than 4 inches in length is 4 feet for a 5-foot run indicating an RQD of 80 percent). The RQD can be used to describe the quality of the rock as indicated in Table 6-23. An additional qualitative measure of rock strength is the time to advance the core barrel. The time should be recorded as minutes per foot and should only include the time spent actually advancing the core barrel into the rock mass.

Table 6-23, Rock Quality Description Terms

Description	RQD
Very poor	0 - 25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

The scratch hardness test can also be used to provide an indication of the hardness of a rock sample. The terms to describe rock hardness are provided in Table 6-24.

Table 6-24, Rock Hardness Terms

Description	Characteristic
Soft (S)	Plastic materials only
Friable (F)	Easily crumbled by hand, pulverized or reduced to powder
Low Hardness (LH)	Can be gouged deeply or carved with a pocketknife
Moderately Hard (MH)	Can be readily scratched by a knife blade
Hard (H)	Can be scratched with difficulty
Very Hard (VH)	Cannot be scratched by pocketknife

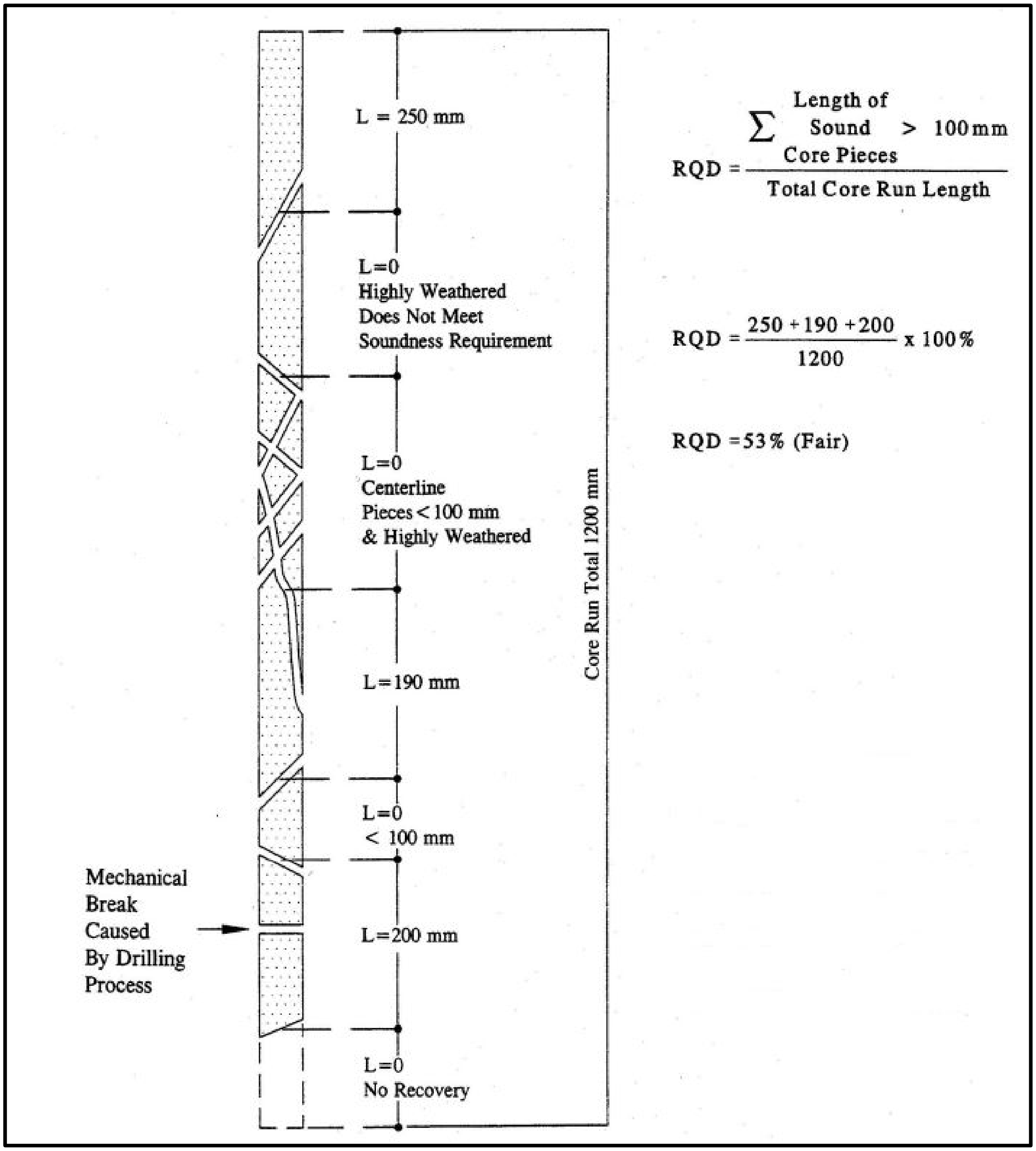


Figure 6-12, RQD Determination
(Mayne, et al., 2002)

6.3.8 Rock Discontinuity

Discontinuity is the general term for any mechanical crack or fissure in a rock mass having no or low tensile strength. It is the collective term for most types of joints, weak bedding planes, weak schistosity planes, weakness zones, and faults. The symbols recommended for the type of rock mass discontinuities are listed in Table 6-25.

Table 6-25, Discontinuity Type

Symbol	Description
F	Fault
J	Joint
Sh	Shear
Fo	Foliation
V	Vein
B	Bedding

The spacing of discontinuities is the perpendicular distance between adjacent discontinuities. The spacing is measured in feet, perpendicular to the planes in the set. Table 6-26 presents guidelines to describe discontinuity.

Table 6-26, Discontinuity Spacing

Symbol	Description
EW	Extremely Wide (> 65 feet)
W	Wide (22 – 65 feet)
M	Moderate (7.5 – 22 feet)
C	Close (2 – 7.5 feet)
VC	Very Close (< 2 feet)

The discontinuities should be described as closed, open, or filled. Aperture is used to describe the perpendicular distance separating the adjacent rock walls of an open discontinuity in which the intervening space is air or water filled. Width is used to describe the distance separating the adjacent rock walls of filled discontinuities. The terms presented in Table 6-27 and Table 6-28 should be used to describe apertures and widths, respectively. Terms such as “wide”, “narrow”, and “tight” are used to describe the width of discontinuities such as thickness of veins, fault gouge filling, or joint openings. For the faults or shears that are not thick enough to be represented on the soil test boring log, the measured thickness is recorded numerically in millimeters (mm).

Table 6-27, Aperture Size Discontinuity Terms

Aperture Opening	Description	
<0.1 mm	Very tight	Closed Features
0.1 – 0.25 mm	Tight	
0.25 – 0.5 mm	Partly open	
0.5 – 2.5 mm	Open	Gapped Features
2.5 – 10 mm	Moderately open	
>10 mm	Wide	
1 – 10 cm	Very wide	Open Features
10 – 100 cm	Extremely wide	
>1m	Cavernous	

Table 6-28, Discontinuity Width Terms

Symbol	Description
W	Wide (12.5 – 50 mm)
MW	Moderately Wide (2.5 – 12.5 mm)
N	Narrow (1.25 – 2.5 mm)
VN	Very Narrow (<1.25 mm)
T	Tight (0 mm)

In addition to the above characterizations, discontinuities are further characterized by the surface shape of the joint and the roughness of its surface (see Tables 6-29 and 6-30).

Table 6-29, Surface Shape of Joint Terms

Symbol	Description
Wa	Wavy
Pl	Planar
St	Stepped
Ir	Irregular

Table 6-30, Surface Roughness Terms

Symbol	Description
Slk	Slickensided (surface has smooth, glassy finish with visual evidence of striations)
S	Smooth (surface appears smooth and feels so to the touch)
SR	Slightly Rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
R	Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive)
VR	Very Rough (near-vertical steps and ridges occur on the discontinuity surface)

Filling is the term for material separating the adjacent rock walls of discontinuities. Filling is characterized by its type, amount, width (i.e., perpendicular distance between adjacent rock walls (see Table 6-28)), and strength. Table 6-31 presents guidelines for characterizing the amount of filling.

Table 6-31, Filling Amount Terms

Symbol	Description
Su	Surface Stain
Sp	Spotty
Pa	Partially Filled
Fi	Filled
No	None

6.3.9 Rock Fracture Description

The location of each naturally occurring fracture and mechanical break should be shown in the fracture column of the rock core log. The naturally occurring fractures are numbered and described using the terminology presented above for discontinuities.

The naturally occurring fractures and mechanical breaks are sketched in the drawing column of the Soil Test Log (see Figures 6-19 and 6-20). Dip angles of fractures shall be measured using a protractor and marked on each log. If the rock is broken into many pieces less than 1 inch long, the log may be crosshatched in that interval or the fracture may be shown schematically. Strike (dip orientation or direction (i.e., north, south, etc.)) should be estimated based on rock cores, local outcrops, and geologic experience in the immediate area.

The number of naturally occurring fractures observed in each 1 foot of core should be recorded in the fracture frequency column. Mechanical breaks, thought to have occurred due to drilling, are not counted. The following criteria can be used to identify natural breaks:

- A rough brittle surface with fresh cleavage planes in individual rock minerals indicates an artificial fracture.
- A generally smooth or somewhat weathered surface with soft coating or infilling materials, such as talc, gypsum, chlorite, mica, or calcite obviously indicates a natural discontinuity.
- In rocks showing foliation, cleavage, or bedding it may be difficult to distinguish between natural discontinuities and artificial fractures when these are parallel with the incipient weakness planes. If drilling has been carried out carefully, then the questionable breaks should be counted as natural features, to be on the conservative side.
- Depending upon the drilling equipment, part of the length of core being drilled may occasionally rotate with the inner barrels in such a way that grinding of the surfaces of discontinuities and fractures occur. In weak rock types, it may be very difficult to decide if the resulting rounded surfaces represent natural or artificial features. When in doubt, the conservative assumption should be made; i.e., assume that the discontinuities are natural.

For projects where knowledge of fractures and strike and dip are important, the GEOR may consider the use of the acoustic televiewer (see Chapter 5 for a description) to obtain this information.

The results of core logging (frequency and RQD) can be strongly time dependent and moisture content dependent in cases of certain varieties of shales and mudstones having relatively weakly developed diagenetic bonds. A frequent problem is "discing", in which an initially intact core separates into discs on incipient planes, the process becoming noticeable perhaps within minutes of core recovery. This phenomenon is experienced in several different forms:

- Stress relief cracking (and swelling) by the initially rapid release of strain energy in cores recovered from areas of high stress, especially in the case of shaley rocks.
- Dehydration cracking experienced in the weaker mudstones and shales which may reduce RQD from 100 percent to 0 percent in a matter of minutes, the initial integrity possibly being due to negative pore pressure.
- Slaking cracking experienced by some of the weaker mudstones and shales when subjected to wetting and drying.

All these phenomena may make core logging of fracture frequency and RQD unreliable. Whenever such conditions are anticipated, cores shall be logged by an experienced geologist or geotechnical engineer as it is recovered and at subsequent intervals when the phenomenon is predicted.

6.3.10 <u>Other Pertinent Information</u>

Additional information that adds to the description of the rock may be included. This may include the geologic formation to which the rock belongs. This information should enhance the description.

6.3.11 <u>Geological Strength Index</u>

In the prior versions of this Manual (Version 1.0 and 1.1) the Rock Mass Rating (RMR) was determined and used in the development of the Hoek-Brown criteria used in rock design. In the most recent version of the Hoek-Brown criteria (Hoek, Carranza-Torres and Corkum (2002)), RMR has been replaced by the Geological Strength Index (GSI) classification system. However, the RMR shall still also be determined. According to Marinos, Marinos and Hoek (2005):

> This index *(GSI)* is based upon an assessment of the lithology, structure and condition of discontinuity surfaces in the rock mass and it is estimated from visual examination of the rock mass exposed in outcrops, in surface excavations such as road cuts and in tunnel faces and borehole cores. The GSI, by combining the two fundamental parameters of the geological process, the blockiness of the mass and the conditions of the discontinuities, respects the main geological constraints that govern a formation and is thus a geologically sound index that is simple to apply in the field.

The use of GSI is only applicable to rock masses whose behavior is controlled by the overall mass response and not by failure along pre-existing structural discontinuities. Rock mass is used to describe the system comprised of intact rock, the consolidated and cemented assemblage of mineral particles, and discontinuities, joints, bedding planes, minor faults, or other recurrent planar features. Intact rock characteristics are determined from index and laboratory tests on core samples, while the rock mass properties are estimated from intact rock properties plus the characteristics of discontinuities.

Figure 6-13 provides the chart for determining GSI from rock core samples or exposed outcrops on a site. The GSI is estimated based on, first, the structure of the rock mass and second, on the condition of the rock surfaces. Combining the rock type and the uniaxial compressive (unconfined) strength of intact (q_u) with the GSI provides a practical means to assess rock mass strength and modulus for foundation design.

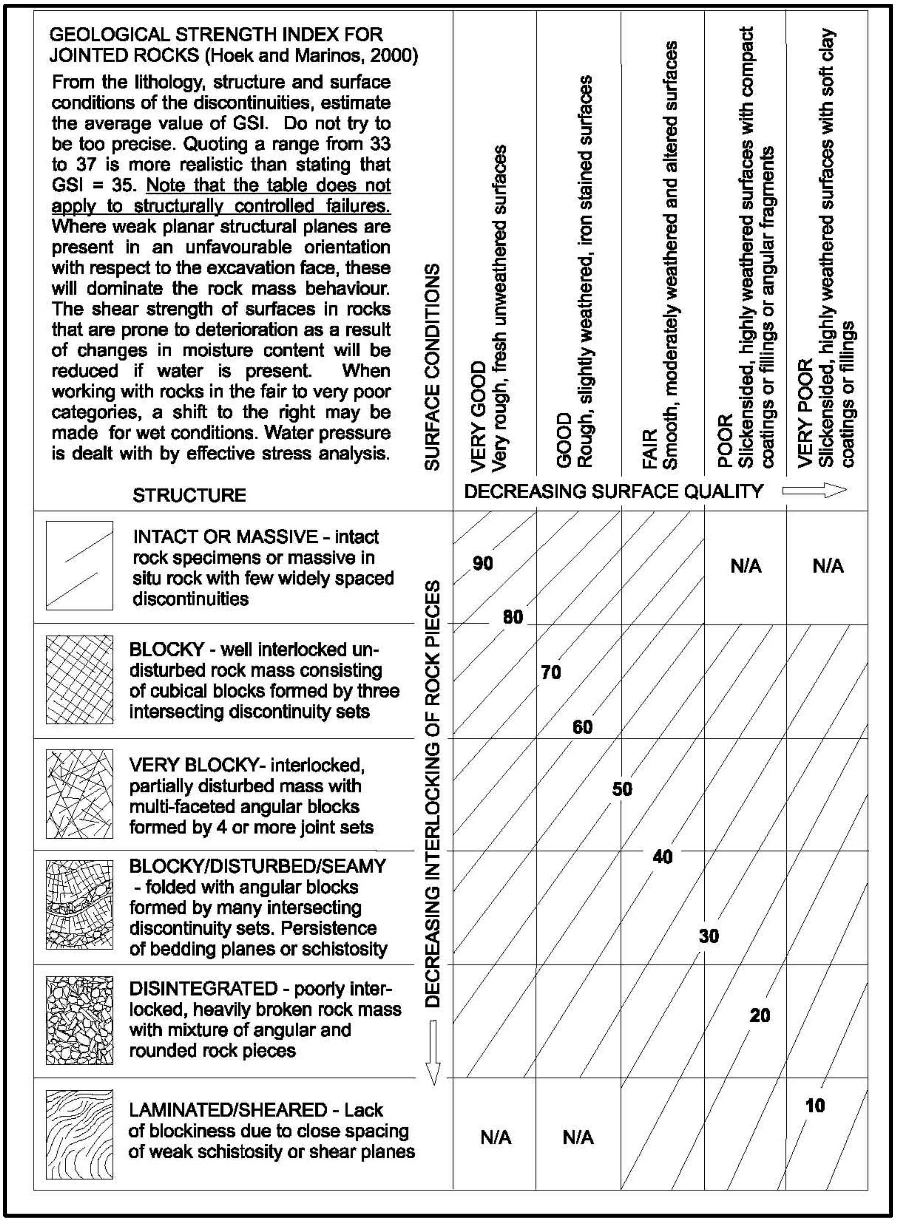


Figure 6-13, GSI Determination
(Brown, Turner and Castelli (2010))

Marinos, et al. (2005) have identified some limitations to the use of the GSI. The GSI classification system should only be applied to those rock masses that are isotropic (i.e., behavior of the rock mass is independent on loading direction). If a clearly defined dominant structural orientation is present (i.e., slate or bedded shales) then the GSI classification system shall not be used. The exception is in slope stability: if the bedding planes are oriented 90° to the slope (i.e., the bedding planes dip into the slope), then the GSI classification system, may be used with caution. Another limitation that needs to be accounted for is the aperture of the discontinuities within the rock mass, since these openings can significantly affect the rock mass properties. The size of the apertures is termed a “disturbance factor” (D) in the latest version of the Hoek-Brown criterion. The disturbance factor ranges from 0 for intact rock to 1 for extremely disturbed rock masses. This factor allows for the disruption of the interlocking on individual rock pieces as result of the opening of the discontinuities. The GSI classification system is a qualitative system that is subjective to the engineer or geologist logging the borehole. Therefore a range of GSI values shall be determined from Figure 6-13.

6.3.12 Rock Mass Rating

The information obtained in the preceding Sections is also used to develop the Rock Mass Rating (RMR). The RMR is used to determine how the mass of rock will behave as opposed to the samples used in unconfined compression, which typically tend to represent the firmest materials available. Discontinuities affect the ability of rock to carry load and to resist deformations. The RMR is the sum of the relative ratings (RR) for 5 parameters adjusted for joint orientations. Table 6-32 provides the 5 parameters and the range of values. The RMR is adjusted to account for joint orientation depending on the favorability of the joint orientation for the specific project. Table 6-33 contains the relative rating adjustments (RRA) for joint orientation. The adjusted RMR is determined using Equation 6-10. The description of the rock mass is based on the adjusted RMR as defined in Table 6-34. The adjusted RMR can be used to estimate the rock mass shear strength and the deformation modulus (see Chapter 7).

$$RMR = RR1 + RR2 + RR3 + RR4 + RR5 + RRA \qquad \textbf{Equation 6-10}$$

Table 6-32, Classification of Rock Masses

	Parameter		Range of Values						
1	Strength of intact rock material	Point load strength index	>1,215 psi	1,215 – 1,100 psi	300 – 1,100 psi	150 – 300 psi	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>30,000 psi	30,000 – 15,000 psi	7,500 – 15,000 psi	3,600 – 7,500 psi	1,500 – 3,600 psi	500 – 1,500 psi	150 – 500 psi
	Relative Rating (RR1)		15	12	7	4	2	1	0
2	Drill core quality RQD		90 – 100%	75 – 90%	50 – 75%	25 – 50%	<25%		
	Relative Rating (RR2)		20	17	13	8	3		
3	Spacing of Joints		>10 ft	3 – 10 ft	1 – 3 ft	2 in – 1 ft	<2 in		
	Relative Rating (RR3)		30	25	20	10	5		
4	Condition of Joints		- Very rough surfaces - Not continuous - No separation - Hard joint wall rock	- Slightly rough surfaces - Separation <0.05 in - Hard joint wall rock	- Slightly rough surfaces - Separation <0.05 in - Soft joint wall rock	- Slicken-sided surfaces or - Gouge <0.2 in thick or - Joints open 0.05 – 0.2 in - Continuous joints	- Soft gouge >0.2 in thick or - Joints open >0.2 in - Continuous joints		
	Relative Rating (RR4)		25	20	12	6	0		
5	Ground water conditions	Ratio – joint water pressure/major principal stress	0	0.0 – 0.2	0.2 – 0.5	>0.5			
		General conditions	Completely dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating (RR5)		10	7	4	0			

Table 6-33, Relative Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Relative Ratings (RRA)	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60

Table 6-34, Rock Mass Class Determination

RMR Rating	81 – 100	61 – 80	41 – 60	21 – 40	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

6.4 FIELD AND LABORATORY TESTING RECORDS

This Section discusses the presentation of field and laboratory data on SCDOT projects. All soil test boring logs and laboratory testing results shall be provided electronically in both a .PDF file and as a gINT® file. In addition, all CPT and DMT data shall be provided electronically as both a .PDF file and as an Excel® spreadsheet following the order provided in Sections 6.2.2 and 6.2.3, respectively. As indicated in Section 6.4.1, the results of shear and compression wave velocity (V_s and V_p) testing shall be presented as a graph in .PDF and Excel® spreadsheet formats including the data table which shall include the V_s, V_p, depth of reading and the estimated unit weight at the reading..

6.4.1 <u>Field Testing Records</u>

The results of Soil Test Borings shall be preliminarily prepared and forwarded to the GEOR for review and editing as well as for the selection of samples for laboratory testing. At the completion of laboratory testing, the preliminary logs shall be corrected to conform to the results of the laboratory testing and final Soil Test Logs shall be prepared and submitted. Figure 6-14 provides the template for the preparation of a soil test log for use on SCDOT projects. Figures 6-15, 6-16 and 6-17 provide the descriptors to be used in preparing the logs. Figure 6-18 provides a template for a manual auger log for use on SCDOT projects. Figures 6-19 and 6-20 provide an example of a completed Soil Test Log. Figure 6-21 presents an example of a completed Manual Auger Log. The results of Field Vane Shear Testing (FVST) shall be presented on soil test boring records as indicated in Figure 6-22, with "FV" inserted after the boring number (i.e., B-1FV). As indicated in Chapter 5, a record is required for Shelby tube (undisturbed, UD) sampling, if the UD is not obtained within a soil test boring. See Figure 6-23 for an example. The record of UD sampling shall consist of the soil test boring designation with a "U" after the number (i.e., B-1U). The results of the CPTu and DMT soundings shall be as presented in Figures 6-24 and 6-25, respectively. The shear and compression wave velocity (V_s and V_p) profiles versus depth shall be presented as indicated in Figure 6-26. In addition, the V_s and V_p profiles versus depth shall also be included in the Excel® spreadsheet as well as provided as a table (see Figure 6-27). In addition, to the information previously indicated, the Soil Test Boring records shall indicate the termination depth, if auger refusal was encountered and what depth. Further, the Soil Test Boring records shall indicate the depth of caving, if encountered and whether the caving was indicated at the completion of the boring or at some other time.

6.4.2 Laboratory Testing Records

In an effort to standardize the appearance of laboratory testing results, all laboratory testing results shall be processed using gINT® as produced by Bentley Systems, Incorporated. Those tests that do not have presentation forms in gINT® shall use the forms currently being used by the GEC. A summary of all laboratory testing results shall be provided (see Figure 6-28). Following the laboratory results summary, provide a graph of index properties (liquid and plastic limits, natural moisture content and percent fines) versus depth. Figure 6-29 provides an example of this graph. The results of moisture-plasticity relationship testing results and grain-size analysis shall also be presented graphically as depicted in Figures 6-30 and 6-31, respectively. The moisture-density relationship testing results shall be depicted as shown in Figure 6-32. In addition, each UD sample is required to have an extraction log (i.e., Shelby Tube Log) indicating the soil encountered in each undisturbed specimen. Further photos of each specimen will also be presented see Figures 6-33, 6-34 and 6-35 for examples. The results of consolidation testing may be shown as depicted in Figure 6-36; however, alternate presentations of consolidation testing results may be presented with prior approval of the OES/GDS. The results of unconfined compression testing may be shown as depicted in Figure 6-37. The results of direct shear testing may be shown as depicted in Figure 6-38. The results of triaxial testing should be shown as indicated in Figures 6-39 and 6-40. In addition, photographs of the triaxial sample immediately after it has been extracted from the Shelby tube, after the sample has been trimmed and placed in the loading cell and after failure shall also be provided. Figure 6-41 provides a summary of the results of rock core testing and Figures 6-42 and 6-43 provide an example of an individual unconfined rock core test result.

6.5 REFERENCES

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SCDOT Soil Test Log

Project ID:	0041401-B01	**County:**	Beaufort/Jasper	**Boring No.:**	B-722		
Site Description:	RBO New River	**Route:**	SC 170/46				
Eng./Geo.:	A. Bore	**Boring Location:**	722+00	**Offset:**	5 ft LT	**Alignment:**	Mainline
Elev.:	1,500 ft	**Latitude:**	34.3750	**Longitude:**	81.0944	**Date Started:**	07/15/03
Total Depth:	45 ft	**Soil Depth:**	39 ft	**Core Depth:**	6 ft	**Date Completed:**	07/16/03
Bore Hole Diameter (in):	4.5	**Sampler Configuration**		**Liner required:**	Y N	**Liner used:**	Y N
Drill Machine:	CME-750	**Drill Method:**	Wash Rotary	**Hammer Type:**	Automatic	**Energy Ratio:**	100%
Core Size:	NQ Wireline	**Driller:**	I. Core	**Groundwater:**	**TOB** 7.5 ft	**24 hr**	15 ft

Depth (feet)	Elevation (ft)	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (feet)	Sample Type / No.	1 st 2 nd 3 rd 4 th	SPT N-Value	● - SPT N-Value (blows / foot) PL MC LL x-------o-------x ▲ - % fines 10 20 30 40 50 60 70 80 90 100
		Soil Description a , b , c , d , e , f , g h , i , j , Munsell , LL PL , PI , NMC , %#200 Munsell = Munsell Color Chart Designation LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index NMC = Natural Moisture Content %#200 = Percent Passing #200 Sieve						
		Rock Description (as required) k , l , m , n , o , p , q r , s , t , u , v , w , x Munsell , RQD , %REC , GSI RMR , q_u , Time Rate Munsell = Munsell Color Chart Designation RQD = Rock Quality Designation %REC = Percent Recovery GSI = Geological Strength Index RMR = Rock Mass Rating q_u = Unconfined Compressive Strength Time Rate = Time required to drill a core						

[1] – The Elevation provided uses NAVD 88.

Figure 6-14, SCDOT Soil Test Log Template

SCDOT Soil Test Log Descriptors

a - Relative Density / Consistency Terms

Relative Density[1]

Descriptive Term	Relative Density	SPT Blow Count
Very Loose	0 to 15%	< 4
Loose	16 to 35%	5 to 10
Medium Dense	36 to 65%	11 to 30
Dense	66 to 85%	31 to 50
Very Dense	86to 100%	>51

Consistency[2]

Descriptive Term	Unconfined Compression Strength (q_u) (tsf)	SPT Blow Count
Very Soft	<0.25	<2
Soft	0.26 to 0.50	3 to 4
Firm	0.51 to 1.00	5 to 8
Stiff	1.01 to 2.00	9 to 15
Very Stiff	2.01 to 4.00	16 to 30
Hard	>4.01	> 31

b Moisture Condition

Descriptive Term	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually in coarse-grained soils below the water table

c Color

Describe the sample color while sample is still moist, using Munsell color chart.

d Angularity[1]

Descriptive Term	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

e HCl Reaction[3]

Descriptive Term	Criteria
None Reactive	No visible reaction
Weakly Reactive	Some reaction, with bubbles forming slowly
Strongly Reactive	Violent reaction, with bubbles forming immediately

f Cementation[3]

Descriptive Term	Criteria
Weakly Cemented	Crumbles or breaks with handling or little finger pressure
Moderately Cemented	Crumbles or breaks with considerable finger pressure
Strongly Cemented	Will not crumble or break with finger pressure

g Particle-Size Range[1]

Gravel

	mm	Sieve size
Fine	4.76 to 19.1	#4 to ¾ inch
Coarse	19.1 to 76.2	¾ inch to 3 inch

Sand

	mm	Sieve size
Fine	0.074 to 0.42	#200 to #40
Medium	0.42 to 2.00	#40 to #10
Coarse	4.00 to 4.76	#10 to #4

h Primary Soil Type[1, 2]

The primary soil type will be shown in all capital letters

i USCS Soil Designation

Indicate USCS soil designation as defined in ASTM D-2487 and D-2488

j AASHTO Soil Designation

Indicate AASHTO soil designation as defined in AASHTO M-145 and ASTM D-3282

[1]Applies to coarse-grained soils (major portion retained on No. 200 sieve)
[2]Appiles to fine-grained soils (major portion passing No. 200 sieve)
[3]Use as required

Figure 6-15, SCDOT Soil Test Log Descriptors – Soil

SCDOT Soil Test Log Descriptors

k **Rock Type**
Indicate type of rock encountered (i.e. granite, limestone, shale, slate, etc.)

l **Color**
Describe the sample color while sample is still moist, using Munsell color chart.

m **Texture**
Describe the nonfracture structural features. Stratification is the layering of sedimentary rock and foliation is the layering of metaphoric rock

Descriptive Term	Criteria
Very Thickly Bedded	> 1.0 m
Thickly Bedded	0.5 to 1.0 m
Thinly Bedded	50 to 500 mm
Very Thinly Bedded	10 to 50 mm
Laminated	2.5 to 10 mm
Thinly Laminated	< 2.5 mm

n **Grain Size and Shape**
Describe the size and shape of all visible grains, typically used on sedimentary rock.

Size

Descriptor	mm	Sieve size
Very coarse grained	> 4.75	Grain sizes greater than popcorn kernels
Coarse grained	2.00 – 4.75	Individual grains easy to distinguish by eye
Medium grained	0.425 – 2.00	Individual grains distinguished by eye
Fine grained	0.075 – 0.425	Individual grains distinguished with difficulty
Very Fine grained	< 0.075	Individual grains cannot be distinguished by unaided eye

Shape

Descriptive Term	Criteria
Angular	Shows little wear: edges and corners are sharp
Subangular	Shows definite effects of wear; edges and corners are slightly rounded off
Subrounded	Shows considerable wear; edges and corners are rounded to smooth curves
Rounded	Shows extreme wear; edges and corners are smoother to broad curves
Well-rounded	Completely worn; edges and corners are not present

o **Weathering / Alteration**
Weathering is the physical disintegration of the minerals by atmospheric processes. Alteration is disintegration of the minerals by geothermal processes.

Description	Recognition
Residual Soil	Original minerals of rock have been entirely decomposed to secondary minerals, and original rock fabric is not apparent; material can be easily broken by hand
Completely Weathered / Altered	Original minerals of rock have been almost entirely decomposed to secondary minerals, although the original fabric may be intact; material can be granulated by hand
Highly Weathered / Altered	More than half of the rock is decomposed; rock is weakened so that a minimum 1-7/8 inch diameter sample can be easily broken readily by hand across rock fabric
Moderately Weathered / Altered	Rock is discolored and noticeably weakened, but less than half is decomposed; a minimum 1-7/8 inch diameter sample cannot be broken readily by hand across rock fabric
Slightly Weathered / Altered	Rock is slightly discolored, but not noticeably lower in strength than fresh rock
Fresh	Rock shows no discoloration, loss of strength, or other effect of weathering / alteration

Figure 6-16, SCDOT Soil Test Log Descriptors – Rock

SCDOT Soil Test Log Descriptors

p **Rock Strength**

Provide a qualitative assessment of the rock strength using either a geologic hammer or knife.

Description	Recognition	Approximately Uniaxial Compressive Strength (psi)
Extremely Weak Rock	Can be indented by thumbnail	35 – 150
Very Weak Rock	Can be peeled by pocket knife	150 –700
Weak Rock	Can be peeled with difficulty by pocket knife	700 – 3,500
Medium Strong Rock	Can be indented 3/16 inch with sharp end of pick	3,500 – 7,200
Strong Rock	Requires one hammer blow to fracture	7,200 – 14,500
Very Strong Rock	Requires many hammer blows to fracture	14,500 – 35,000
Extremely Strong Rock	Can only be chipped with hammer blows	> 35,000

q **Strike and Dip**

Dip of fracture surface measured relative to horizontal with bearing and direction (i.e. N30°down, etc.)

r **Discontinuity Type**

F - Fault
J - Joint
Sh - Shear
Fo - Foliation
V - Vein
B - Bedding

s **Discontinuity Width (millimeters)**

W - Wide (12.5 – 50)
MW - Moderately Wide (2.5 – 12.5)
N - Narrow (1.25 – 2.5)
VN - Very Narrow (< 1.25)
T - Tight (0)

t **Amount of Infilling**

Su - Surface Stain
Sp - Spotty
Pa - Partially Filled
Fi - Filled
No - None

u **Type of Infilling**

Cl - Clay
Ca - Calcite
Ch - Chloride
Fe - Iron Oxide
Gy - Gypsum/Talc
H - Healed
No - None
Py - Pyrite
Qz - Quartz
Sd - Sand

v **Surface Shape of Joint**

Wa - Wavy
Pl - Planar
St - Stepped
Ir - Irregular

w **Discontinuity Spacing (feet)**

EW Extremely Wide (> 65)
W Wide (22 – 65)
M Moderate (7.5 – 22)
C Close (2 – 7.5)
VC Very Close (< 2)

x **Roughness of Surface**

Slk - Slickensided (surface has smooth, glassy finish with visual evidence of striations)
S - Smooth (surface appears smooth and feels so to the touch)
SR - Slightly Rough (asperities on the discontinuity surfaces are distinguishable and can be felt)
R - Rough (some ridges and side-angle steps are evident; asperities are clearly visible, and discontinuity surface feels very abrasive
VR - Very Rough (near-vertical steps and ridges occur on the discontinuity surface)

Figure 6-17, SCDOT Soil Test Log Descriptors – Rock (con't)

SCDOT Manual Auger Log

Project ID:	0041401-B01			**County:**	Beaufort/Jasper		**Boring No.:**	MA-1
Site Description:	RBO New River						**Route:**	SC 170/46
Driller:	A. Bore	**Boring Location:**	722+00	**Offset:**	5 ft LT		**Alignment:**	Mainline
Elev.:	1,500 ft	**Latitude:**	34.3750	**Longitude:**	81.0944		**Date Started:**	07/15/03
Total Depth:	5 ft	**Groundwater:**	**TOB** 5 ft	**24 hr**	3 ft		**Date Completed:**	07/16/03
Dynamic Cone Penetrometer Test Procedure:		**Sowers and Hedges (1966)**				**ASTM D6951**		

Depth (feet)	Elevation (ft)	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (feet)	Sample Type / No.	1 st 2 nd 3 rd	DCP N-Value	● - DCP N-Value (blows / foot) PL MC LL x-------o-------x ▲ - % fines 10 20 30 40 50 60 70 80 90 100
		Soil Description a , b , c , d , e , f , g h , i , j , Munsell , LL PL , PI , NMC , %#200 Munsell = Munsell Color Chart Designation LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index NMC = Natural Moisture Content %#200 = Percent Passing #200 Sieve						

[1] – The Elevation provided uses NAVD 88.

Figure 6-18, SCDOT Manual Auger Log Template

SCDOT Soil Test Log

Project ID:	0041401-B01	**County:**	Lexington	**Boring No.:**	B-1		
Site Description:	gINT Example	**Route:**	SC 160				
Eng./Geo.:	Alfred Boring	**Boring Location:**	100+50	**Offset:**	30 L	**Alignment:**	Mainline
Elev.:	351.0 ft	**Latitude:**	34.0654	**Longitude:**	80.2211	**Date Started:**	7/14/2006
Total Depth:	55.75 ft	**Soil Depth:**	39 ft	**Core Depth:**	16.75 ft	**Date Completed:**	7/15/2006
Bore Hole Diameter (in):	4.5	**Sampler Configuration**		**Liner Required:**	Y N	**Liner Used:**	Y N
Drill Machine:	CME-750	**Drill Method:**	HSA/RC	**Hammer Type:**	Automatic	**Energy Ratio:**	85%
Core Size:	NQ	**Driller:**	T.Reid	**Groundwater:** TOB	7.5 ft	**24HR**	15 ft

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Sample Depth (ft)	Sample No./Type	1st 6"	2nd 6"	3rd 6"	4th 6"	N Value
	0.0	Top of ground; flat and dry							
		Loose, moist, reddish brown, Silty fine to medium SAND (SM) (A-2-4), 2.5YR5/4	0.0	SS-1	2	3	6	7	9
		LL=40, PL=30, PI=10, NMC=25, %200=14	1.5	SS-2	2	3	4	5	7
	3.0	LL=NP, PL=NP, PI=NP, NMC=18, %200=16	3.5	SS-3	3	4	4	5	8
346.0	5.5	Loose, moist, reddish brown, fine to medium SAND with Clay (SP-SC) (A-2-6), 5YR5/4	6.0	SS-4	4	6	7	8	13
341.0		LL=35, PL=15, PI=20, NMC=17, %200=12	8.5	SS-5	4	7	9	10	16
	12.0	Medium dense, moist, dark brown, fine to medium SAND with Silt (SP-SM) (A-2-4), 7.5YR4/4	13.5	SS-6	3	10	9	10	19
336.0		LL=8, PL=8, PI=0, NMC=25, %200=10							
		LL=4, PL=4, PI=0, NMC=22, %200=8							
		Medium dense, moist, dark brown, Silty fine to medium SAND (SM) (A-2-4), 7.5YR4/3	18.5	SS-7	5	8	15	16	23
331.0		LL=16, PL=13, PI=4, NMC=37, %200=32							
		LL=10, PL=10, PI=0, NMC=56, %200=15							
	22.0	Medium dense, moist, verk dark grayish brown, Clayey fine to medium SAND (SC) (A-6), 10YR3/2	23.5	SS-8	7	8	20	21	28
326.0		LL=40, PL=12, PI=28, NMC=42, %200=40							
	27.0	Hard, moist, very dark brown, Sandy fat CLAY (CH) (A-7-6(13)), 10YR2/2	28.5	SS-9	9	19	17	18	36
321.0		LL=67, PL=27, PI=40, NMC=12, %200=57							
	32.0	Hard, moist, dark brown, Sandy SILT (ML) (A-5(8)), 10YR3/3	33.5	SS-10	12	20	18	19	38
316.0		LL=45, PL=30, PI=15, NMC=14, %200=58							
	37.0	Hard, moist, reddish brown, elastic SILT with Sand (MH) (A-7-5(16)), 5YR5/3	38.5	SS-11	50/4"				50/4"
311.0		LL=55, PL=35, PI=20, NMC=15, %200=72							
	42.0	LIMESTONE, tan, thickly bedded, hihgly to moderately weathered, weak rock, Sh, VN, No, PI, M, SR, 10YR7/3	42.0	NQ-1					REC=55%, RQD=20%
306.0		%REC=55, RQD=20, GSI=35, RMR=50, 10 min/ft, qu=8,000psi							
		%REC=75, RQD=30, GSI=35, RMR=60, 12 min/ft, qu=3,000psi	47.0	NQ-2					REC=75%, RQD=30%

● SPT N VALUE ● PL MC LL ▲ FINES CONTENT (%) 0 10 20 30 40 50 60 70 80 90

LEGEND

Continued Next Page

SAMPLER TYPE		DRILLING METHOD	
SS - Split Spoon	NQ - Rock Core, 1-7/8"	HSA - Hollow Stem Auger	RW - Rotary Wash
UD - Undisturbed Sample	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core
AWG - Rock Core, 1-1/8"	CT - Continuous Tube	DC - Driving Casing	

SC_DOT_SC_DOT_A_07_20_2016.GPJ SCDOT DATA TEMPLATE_01_30_2015.GDT 10/14/16

Figure 6-19, Soil Test Log Example

SCDOT Soil Test Log

Project ID:	0041401-B01	**County:**	Lexington	**Boring No.:**	B-1		
Site Description:	gINT Example	**Route:**	SC 160				
Eng./Geo.:	Alfred Boring	**Boring Location:**	100+50	**Offset:**	30 L	**Alignment:**	Mainline
Elev.:	351.0 ft	**Latitude:**	34.0654	**Longitude:**	80.2211	**Date Started:**	7/14/2006
Total Depth:	55.75 ft	**Soil Depth:**	39 ft	**Core Depth:**	16.75 ft	**Date Completed:**	7/15/2006
Bore Hole Diameter (in):	4.5	**Sampler Configuration**		**Liner Required:**	Y N	**Liner Used:**	Y N
Drill Machine:	CME-750	**Drill Method:**	HSA/RC	**Hammer Type:**	Automatic	**Energy Ratio:**	85%
Core Size:	NQ	**Driller:**	T.Reid	**Groundwater:**	**TOB** 7.5 ft	**24HR**	15 ft

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (ft)	Sample No./Type	1st 6"	2nd 6"	3rd 6"	4th 6"	N Value	● SPT N VALUE ● PL MC LL ▲ FINES CONTENT (%) 0 10 20 30 40 50 60 70 80 90
	52.0			52.0							
296.0		SILTSTONE, tan, thickly bedded, moderately weathered, strong rock, Fo, T,No, Wa, W, R, 10YR7/3			NQ-3						REC=95%; RQD=100%
	55.8	%REC= 95, RQD=100, GSI=80, RMR=100, 20min/ft, qu=12,000psi									
		Boring Terminated at 55.75 feet									
291.0											
286.0											
281.0											
276.0											
271.0											
266.0											
261.0											
256.0											

SC_DOT_SC_DOT_A_07_20_2016.GPJ SCDOT DATA TEMPLATE_01_30_2015.GDT 10/14/16

LEGEND

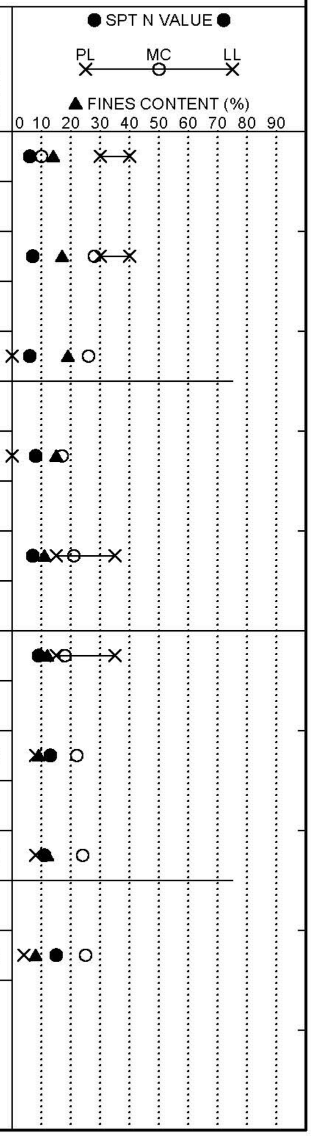
SAMPLER TYPE		DRILLING METHOD	
SS - Split Spoon	NQ - Rock Core, 1-7/8"	HSA - Hollow Stem Auger	RW - Rotary Wash
UD - Undisturbed Sample	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core
AWG - Rock Core, 1-1/8"	CT - Continuous Tube	DC - Driving Casing	

Figure 6-20, Soil Test Log Example (con't)

SCDOT Soil Test Log

Project ID:	0041401-B01			**County:**	Lexington	**Boring No.:**	MA-1
Site Description:	gINT Example					**Route:**	SC 160
Eng./Geo.:	Alfred Boring	**Boring Location:**	100+50	**Offset:**	30 R	**Alignment:**	Mainline
Elev.:	351.0 ft	**Latitude:**	34.0654	**Longitude:**	80.2211	**Date Started:**	7/16/2006
Total Depth:	8.5 ft	**Soil Depth:**	8 ft	**Core Depth:**	ft	**Date Completed:**	7/16/2006
Bore Hole Diameter (in):	4	**Sampler Configuration**		**Liner Required:**	Y N	**Liner Used:**	Y N
Drill Machine:		**Drill Method:**	HA	**Hammer Type:**		**Energy Ratio:**	
Core Size:		**Driller:**	T.Reid	**Groundwater:**	**TOB** NE	**24HR**	4 ft

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Sample Depth (ft)	Sample No./Type	1st 6"	2nd 6"	3rd 6"	4th 6"	N Value
	0.0	Loose, moist, reddish brown Silty fine to medium SAND (SM) (A-2-4), 2.5YR5/4 LL=40, PL=30, PI=10, NMC=10, %200=14	0.0	DCP-1	2	5	7		6
		LL=40, PL=30, PI=10, NMC=28, %200=17	1.0	DCP-2	1	7	6		7
		LL=0, PL=0, PI=0, NMC=26, %200=19	2.0	DCP-3	3	4	8		6
	3.5	LL=0, PL=0, PI=0, NMC=17, %200=15	3.0	DCP-4	2	8	8		8
		Loose, moist, reddish brown, fine to medium SAND with Clay (SP-SC) (A-2-6), 5YR5/4 LL=35, PL=15, PI= 20, NMC=21, %200=11	4.0	DCP-5	2	5	9		7
346.0	5.5	LL=35, PL=15, PI= 20, NMC=18, %200=12	5.0	DCP-6	3	6	12		9
		Loose, moist, dark, brown, fine to medium SAND with Silt (SP-SM) LL=8, PL=8, PI=0, NMC=22, %200=9	6.0	DCP-7	1	5	20		13
		LL=8, PL=8, PI=0, NMC=24, %200=12	7.0	DCP-8	1	5	16		11
	8.5	LL=4, PL=4, PI=0, NMC=25, %200=8	8.0	DCP-9	5	9	21		15
		Manual Auger Terminated at 8.5 feet.							



LEGEND

SAMPLER TYPE		DRILLING METHOD	
SS - Split Spoon	NQ - Rock Core, 1-7/8"	HSA - Hollow Stem Auger	RW - Rotary Wash
UD - Undisturbed Sample	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core
AWG - Rock Core, 1-1/8"	CT - Continuous Tube	DC - Driving Casing	

SC_DOT_SC_DOT_A_01_30_2015.GPJ SCDOT DATA TEMPLATE_01_30_2015.GDT 10/14/16

Figure 6-21, Manual Auger Log Example

SCDOT Soil Test Log

Project ID: 0041401-B01		**County:** Lexington		**Boring No.:** B-1FV			
Site Description: gINT Example				**Route:** SC 160			
Eng./Geo.: Alfred Boring	**Boring Location:** 100+50		**Offset:** 25 L	**Alignment:** Mainline			
Elev.: 351.0 ft	**Latitude:** 34.0654	**Longitude:** 80.2211		**Date Started:** 7/17/2006			
Total Depth: 31.5 ft	**Soil Depth:** 35 ft	**Core Depth:** ft		**Date Completed:** 7/17/2006			
Bore Hole Diameter (in): 4.5	**Sampler Configuration**	**Liner Required:** Y N		**Liner Used:** Y N			
Drill Machine: CME-750	**Drill Method:** HSA	**Hammer Type:**		**Energy Ratio:**			
Core Size:	**Driller:** T.Reid	**Groundwater:** **TOB** 7.5 ft		**24HR** 15 ft			

Elevation (ft)	Depth (ft) 0.0	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (ft)	Sample No./Type	1st 6"	2nd 6"	3rd 6"	4th 6"	N Value	● SPT N VALUE ● PL MC LL ▲ FINES CONTENT (%) 0 10 20 30 40 50 60 70 80 90
346.0		See Soil Test Boring B-1 for soils									
341.0											
336.0											
331.0		$(S_u)_{peak}$=500 psf $(S_u)_{rem}$=100psf		18.0	FV-1						
326.0											
321.0	31.5	$(S_u)_{peak}$=1,500 psf $(S_u)_{rem}$=250psf Boring Terminated at 31.5 feet.		31.0	FV-2						
316.0											
311.0											
306.0											

LEGEND

SAMPLER TYPE		DRILLING METHOD	
SS - Split Spoon	NQ - Rock Core, 1-7/8"	HSA - Hollow Stem Auger	RW - Rotary Wash
UD - Undisturbed Sample	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core
AWG - Rock Core, 1-1/8"	CT - Continuous Tube	DC - Driving Casing	

SC_DOT SC_DOT_A_07_20_2016.GPJ SCDOT DATA TEMPLATE_01_30_2015.GDT 10/14/16

Figure 6-22, Field Vane Shear Testing Log Example

SCDOT Soil Test Log

Project ID: 0041401-B01			**County:** Lexington		**Boring No.:** B-1U		
Site Description: gINT Example					**Route:** SC 160		
Eng./Geo.: Alfred Boring	**Boring Location:** 100+55		**Offset:** 30 L		**Alignment:** Mainline		
Elev.: 351.0 ft	**Latitude:** 34.0654		**Longitude:** 80.2211		**Date Started:** 7/16/2006		
Total Depth: 34 ft	**Soil Depth:** 35 ft		**Core Depth:** ft		**Date Completed:** 7/16/2006		
Bore Hole Diameter (in): 4.5	**Sampler Configuration**		**Liner Required:** Y N		**Liner Used:** Y N		
Drill Machine: CME-750	**Drill Method:** HSA		**Hammer Type:**		**Energy Ratio:**		
Core Size:	**Driller:** T.Reid		**Groundwater:**	**TOB** 7.5 ft	**24HR** 15 ft		

Elevation (ft)	Depth (ft)	MATERIAL DESCRIPTION	Graphic Log	Sample Depth (ft)	Sample No./Type	1st 6"	2nd 6"	3rd 6"	4th 6"	N Value	● SPT N VALUE ● PL MC LL ▲ FINES CONTENT (%) 0 10 20 30 40 50 60 70 80 90
	0.0										
346.0											
341.0											
336.0											
331.0		See Soil Test Boring Log B-1 for Soils REC=100%		20.0	UD-1						
326.0											
321.0											
	34.0	See Soil Test Boring Log B-1 for Soils REC=100%		32.0	UD-2						
316.0		Boring Terminated at 34 feet.									
311.0											
306.0											

LEGEND

SAMPLER TYPE		DRILLING METHOD	
SS - Split Spoon	NQ - Rock Core, 1-7/8"	HSA - Hollow Stem Auger	RW - Rotary Wash
UD - Undisturbed Sample	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core
AWG - Rock Core, 1-1/8"	CT - Continuous Tube	DC - Driving Casing	

SC_DOT SC_DOT_A_07_20_2016.GPJ SCDOT DATA TEMPLATE_01_30_2015.GDT 10/14/16

Figure 6-23, Undisturbed Sampling Log Example

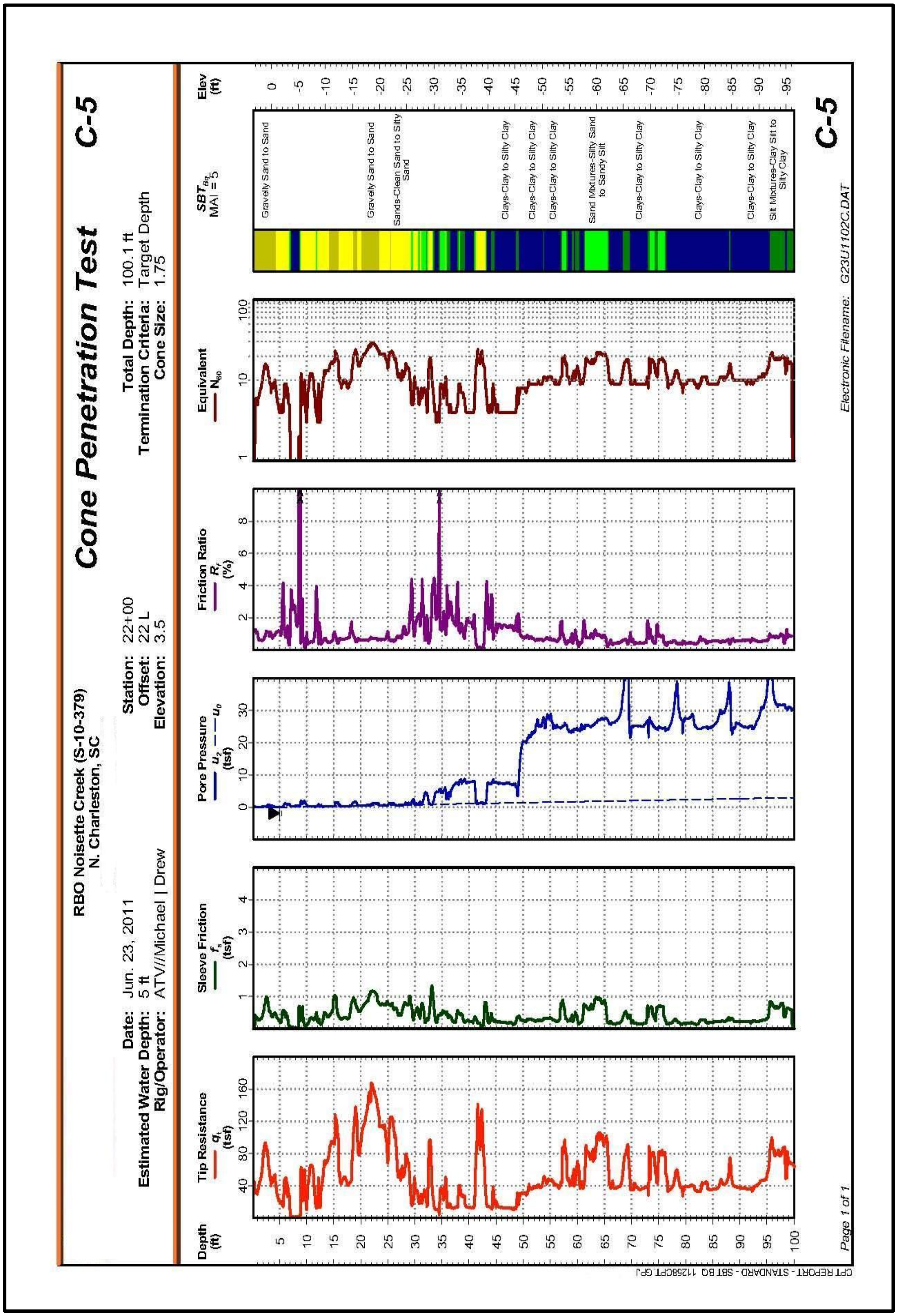


Figure 6-24, Electro-Piezocone Sounding Record Example

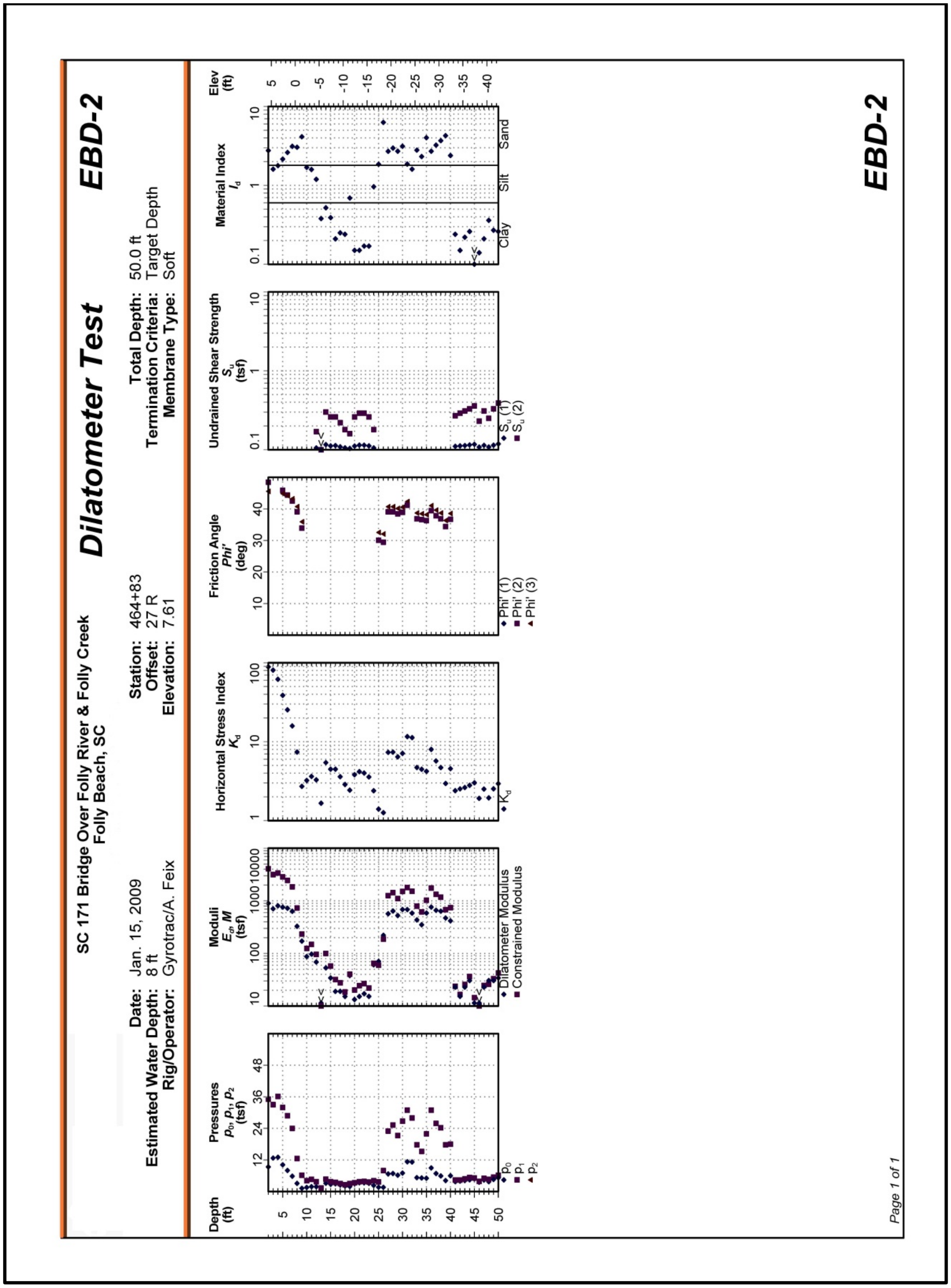


Figure 6-25, Dilatometer Sounding Record Example

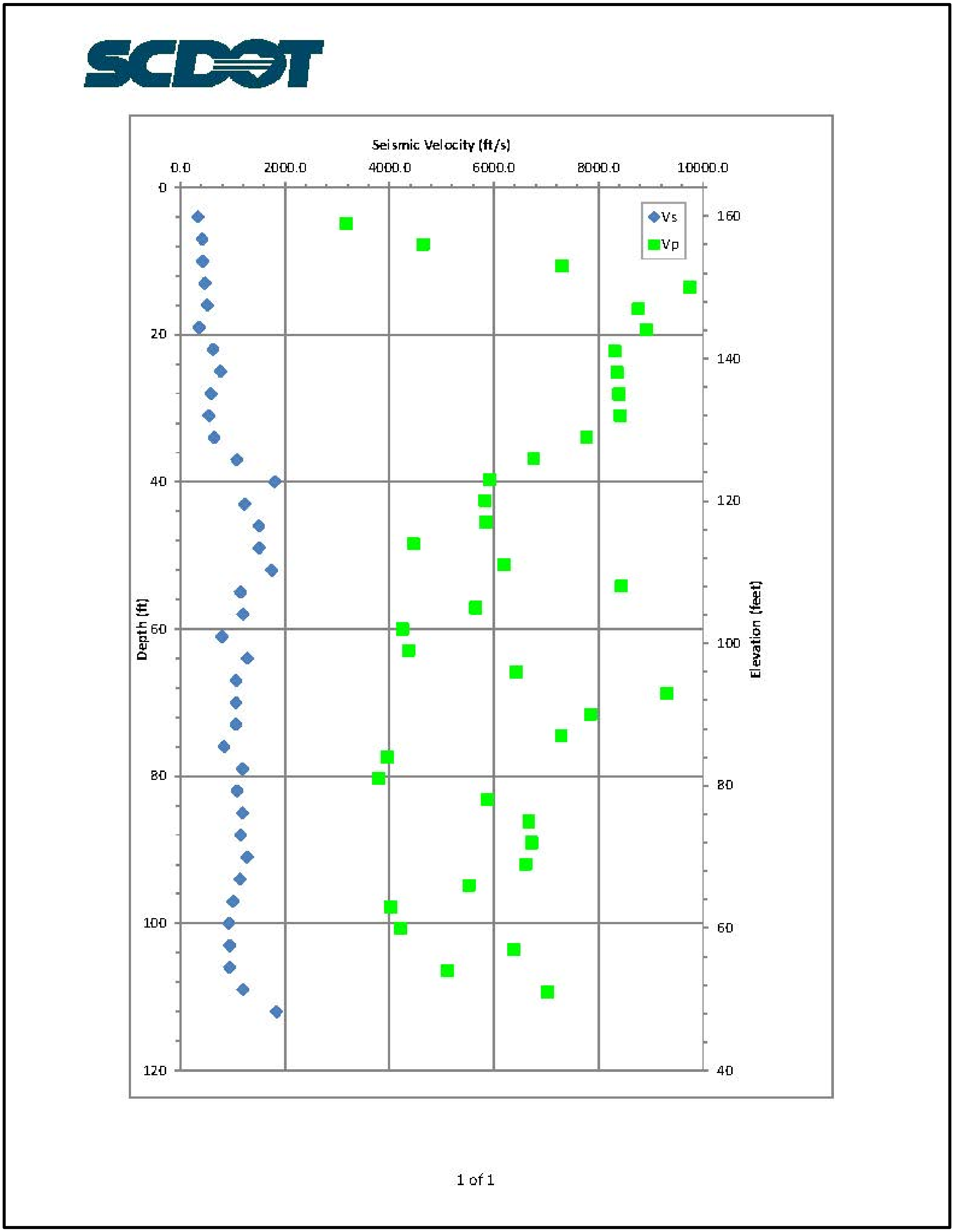


Figure 6-26, Shear and Compression Wave Velocity Profile vs. Depth

Project Name:	Bridgeway Project MASW Testing			
Project Number:	73215035			
Line No.:	1			
Depth	S-wave velocity	P-wave velocity	Density	
ft.	ft/sec.	Ft/Sec.	g/cc	pcf
0	639.578342	4946.396828	1.802648	112.54
4.3	633.3	4944.6	1.8	112.54
9.2	627.8	4943.4	1.8	112.54
14.8	720.8	5044.7	1.8	112.92
21.1	943.5	5276.7	1.8	113.75
28.0	1201.3	5548.7	1.8	114.87
35.6	1238.9	5589.0	1.8	115.02
43.8	1414.8	5798.5	1.9	116.69
52.7	1413.8	5819.8	1.9	117.53
62.3	1348.8	5764.2	1.9	117.79
72.5	1496.9	5924.8	1.9	118.80
83.4	1614.4	6034.7	1.9	119.04
94.9	1663.3	6066.7	1.9	118.51
107.1	1905.4	6319.3	1.9	119.20
145.7	1905.4	6325.2	1.9	119.20

Figure 6-27, Shear and Compression Wave Velocity Profile Table

SCDOT

SUMMARY OF LABORATORY RESULTS

PAGE 1 OF 1

PROJECT ID 0041401-B01 PROJECT NAME gINT Example

PROJECT COUNTY Lexington

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	0.0	40	30	10	4.75	14	SM	25.0			
B-1	1.5	NP	NP	NP	4.75	16	SM	18.0			
B-1	5.0	35	15	20	4.75	12	SP-SC	17.0			
B-1	7.5	8	8	NP	9.5	10	SP-SM	25.0	76.7		
B-1	10.0	4	4	NP	4.75	8	SP-SM	22.0	101.2		
B-1	15.0	16	13	3	4.75	15	SM	37.0			
B-1	20.0	10	10	NP	4.75	15	SM	56.0			
B-1	25.0	40	12	28	2.36	40	SC	42.0	81.6		
B-1	30.0	67	27	40	2.36	57	CH	12.0			
B-1	35.0	45	30	15	1.18	58	ML	14.0			
B-1	40.0	55	35	20	1.18	72	MH	15.0			
MA-2	0.0	40	30	10	4.75	14	SM	10.0			
MA-2	1.5	40	30	10	4.75	17	SM	28.0			
MA-2	2.5	NP	NP	NP	4.75	19	SM	26.0			
MA-2	3.0	NP	NP	NP	4.75	15	SM	17.0			
MA-2	4.0	35	15	20	4.75	11	SP-SC	21.0			
MA-2	5.0	35	15	20	4.75	12	SP-SC	18.0			
MA-2	6.5	8	8	NP	4.75	9	SP-SM	22.0			
MA-2	7.5	8	8	NP	4.75	12	SP-SM	24.0			
MA-2	8.0	4	4	NP	4.75	8	SP-SM	25.0			

LAB SUMMARY SCDOT_GINT_EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

Figure 6-28, Summary of Laboratory Testing Results

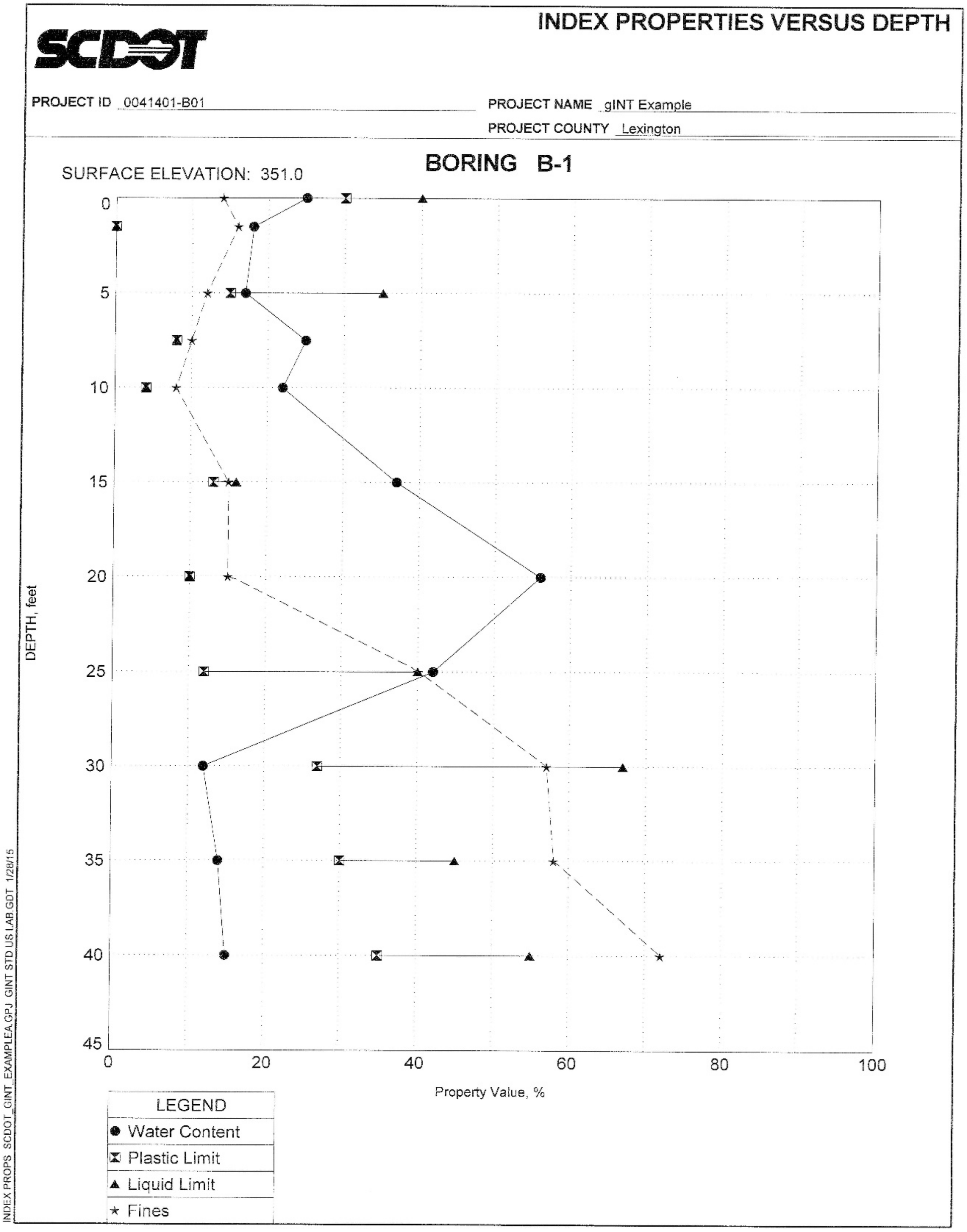


Figure 6-29, Index Properties versus Depth

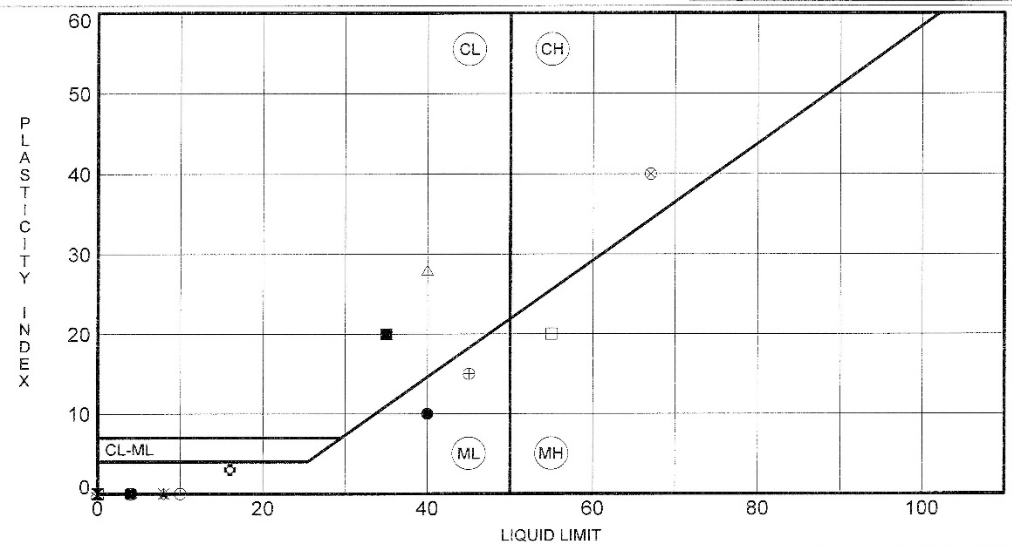
SCDOT

ATTERBERG LIMITS' RESULTS

PROJECT ID 0041401-B01

PROJECT NAME gINT Example

PROJECT COUNTY Lexington



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	B-1	0.0	40	30	10	14	Reddish brown Silty f to m SAND
⧗	B-1	1.5	NP	NP	NP	16	Reddish brown Silty f to m SAND
▲	B-1	5.0	35	15	20	12	Reddish brown f to m SAND with Clay
★	B-1	7.5	8	8	NP	10	Dark brown f to m SAND with Silt
⊙	B-1	10.0	4	4	NP	8	Dark brown f to m SAND with Silt
✪	B-1	15.0	16	13	3	15	Dark brown Silty f to m SAND
○	B-1	20.0	10	10	NP	15	Dark brown Silty f to m SAND
△	B-1	25.0	40	12	28	40	Very dark grayish brown Clayey SAND
⊗	B-1	30.0	67	27	40	57	Very dark brown Sandy fat CLAY
⦶	B-1	35.0	45	30	15	58	Dark brown Sandy SILT
□	B-1	40.0	55	35	20	72	Reddish brown elastic SILT with Sand
●	MA-2	0.0	40	30	10	14	Reddish brown Silty f to m SAND
◐	MA-2	1.5	40	30	10	17	Reddish brown Silty f to m SAND
☆	MA-2	2.5	NP	NP	NP	19	Reddish brown Silty f to m SAND
⌑	MA-2	3.0	NP	NP	NP	15	Reddish brown Silty f to m SAND
■	MA-2	4.0	35	15	20	11	Reddish brown f to m SAND with Clay
◆	MA-2	5.0	35	15	20	12	Reddish brown f to m SAND with Clay
◇	MA-2	6.5	8	8	NP	9	Dark brown f to m SAND with Silt
×	MA-2	7.5	8	8	NP	12	Dark brown f to m SAND with Silt
✖	MA-2	8.0	4	4	NP	8	Dark brown f to m SAND with Silt

ATTERBERG LIMITS SCDOT_GINT_EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

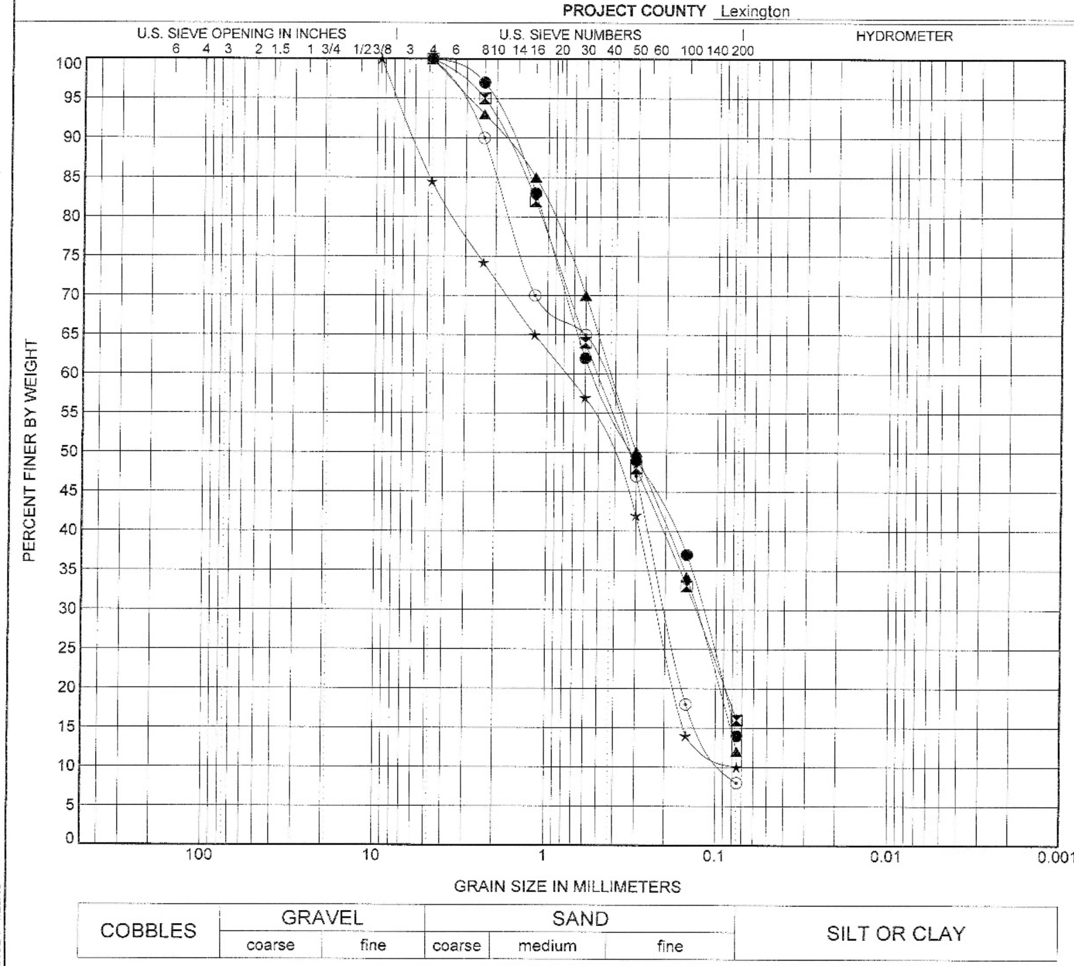
Figure 6-30, Moisture-Plasticity Relationship Testing Results

SCDOT

GRAIN SIZE DISTRIBUTION

PROJECT ID 0041401-B01 PROJECT NAME gINT Example

PROJECT COUNTY Lexington



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	B-1	0.0	Reddish brown Silty f to m SAND	40	30	10		
⧗	B-1	1.5	Reddish brown Silty f to m SAND	NP	NP	NP		
▲	B-1	5.0	Reddish brown f to m SAND with Clay	35	15	20	0.58	6.01
★	B-1	7.5	Dark brown f to m SAND with Silt	8	8	NP	0.86	10.31
⊙	B-1	10.0	Dark brown f to m SAND with Silt	4	4	NP	0.94	5.74

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	0.0	4.75	0.539	0.121		0.0	86.0	14.0	
⧗	B-1	1.5	4.75	0.505	0.133		0.0	84.0	16.0	
▲	B-1	5.0	4.75	0.424	0.132		0.0	88.0	12.0	
★	B-1	7.5	9.5	0.773	0.223	0.075	15.5	74.5	10.0	
⊙	B-1	10.0	4.75	0.495	0.2	0.086	0.0	92.0	8.0	

GRAIN SIZE SCDOT_GINT_EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

Figure 6-31, Grain-Size Analysis Results

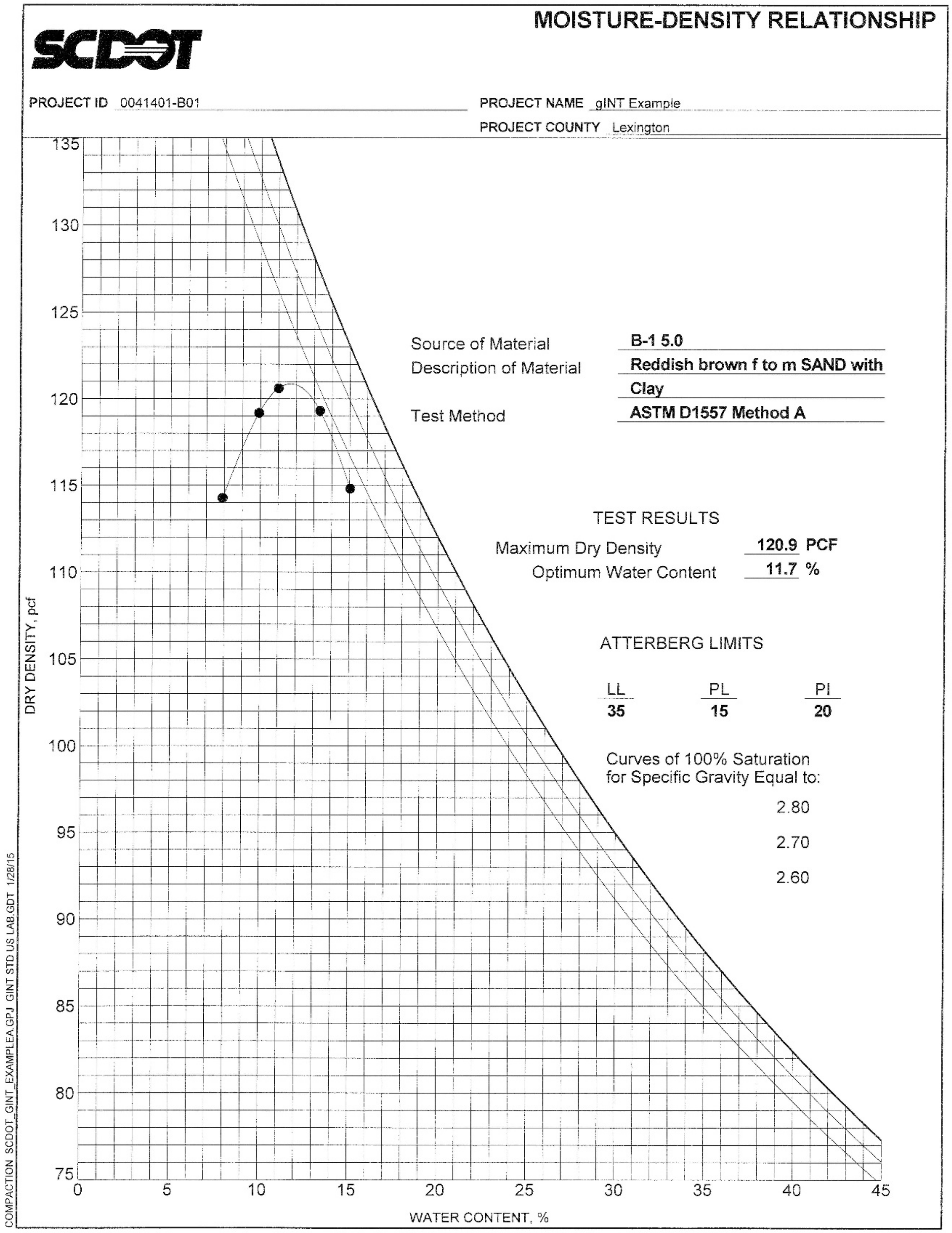
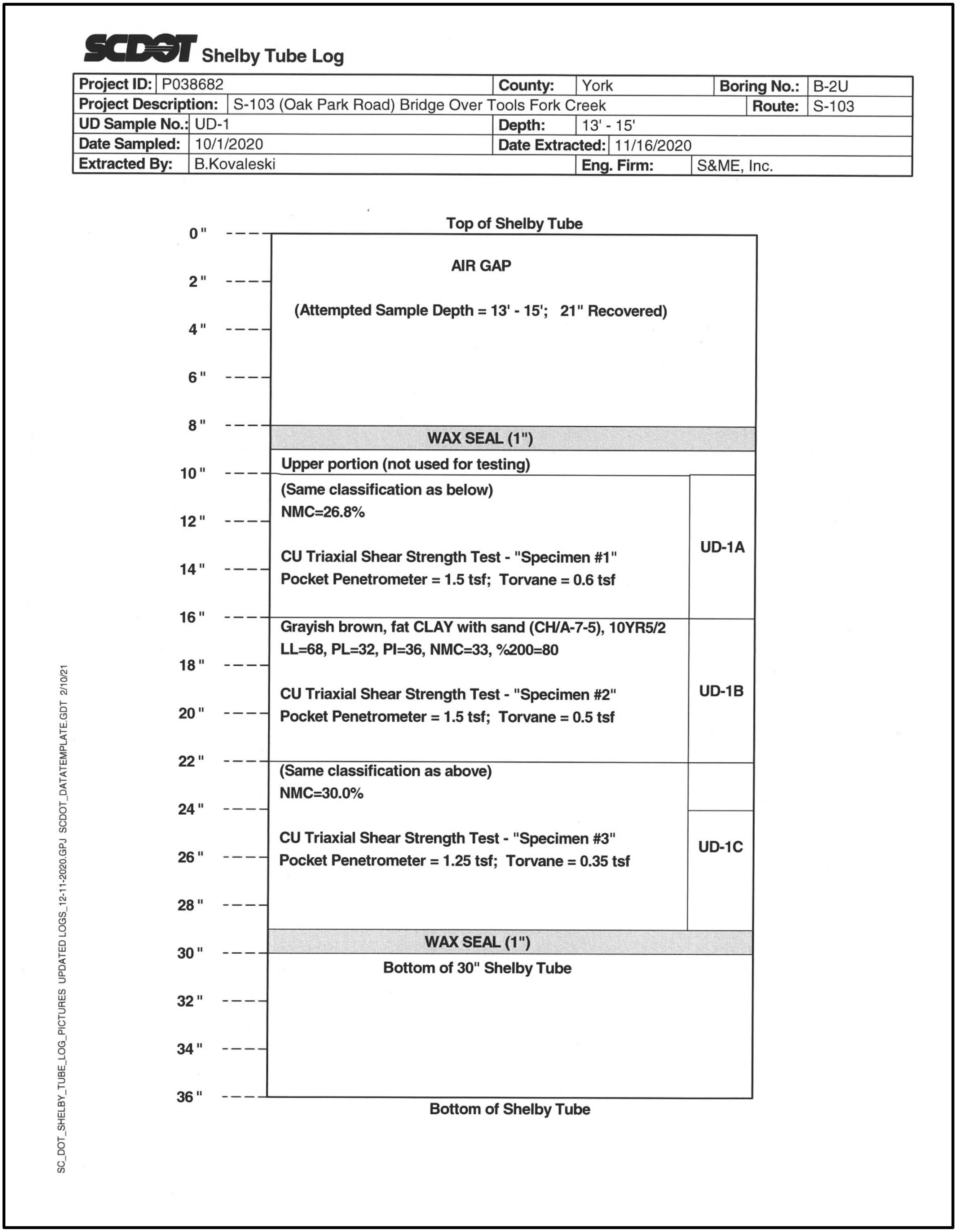


Figure 6-32, Moisture-Density Relationship Testing Results



SCDOT Shelby Tube Log

Project ID:	P038682	**County:**	York	**Boring No.:**	B-2U
Project Description:	S-103 (Oak Park Road) Bridge Over Tools Fork Creek			**Route:**	S-103
UD Sample No.:	UD-1	**Depth:**	13' - 15'		
Date Sampled:	10/1/2020	**Date Extracted:**	11/16/2020		
Extracted By:	B.Kovaleski	**Eng. Firm:**	S&ME, Inc.		

0" 2" 4" 6" 8" 10" 12" 14" 16" 18" 20" 22" 24" 26" 28" 30" 32" 34" 36"

Top of Shelby Tube

AIR GAP

(Attempted Sample Depth = 13' - 15'; 21" Recovered)

WAX SEAL (1")

Upper portion (not used for testing)

(Same classification as below)
NMC=26.8%

CU Triaxial Shear Strength Test - "Specimen #1"
Pocket Penetrometer = 1.5 tsf; Torvane = 0.6 tsf

UD-1A

Grayish brown, fat CLAY with sand (CH/A-7-5), 10YR5/2
LL=68, PL=32, PI=36, NMC=33, %200=80

CU Triaxial Shear Strength Test - "Specimen #2"
Pocket Penetrometer = 1.5 tsf; Torvane = 0.5 tsf

UD-1B

(Same classification as above)
NMC=30.0%

CU Triaxial Shear Strength Test - "Specimen #3"
Pocket Penetrometer = 1.25 tsf; Torvane = 0.35 tsf

UD-1C

WAX SEAL (1")

Bottom of 30" Shelby Tube

Bottom of Shelby Tube

SC_DOT_SHELBY_TUBE_LOG_PICTURES UPDATED LOGS_12-11-2020.GPJ SCDOT_DATATEMPLATE.GDT 2/10/21

Figure 6-33, Shelby Tube Log Example

SCDOT Undisturbed Sample Pictures

Project ID:	P037125	County:	25 - Hampton	Boring No:	STB-2A
Site Description:	S-140 Camp Branch		Route:	S-25-140	
UD Sample No.:	ST-1	Depth:	25.0' - 27.0'		
Date Sampled:	8/23/2019	Date Extracted:	9/4/2019		
Extracted By:	D. Schmidt		Eng. Firm:	HDR	

Specimen No. ST-1.A

Specimen No. ST-1.B

Figure 6-34, Shelby Tube Log Photograph Example

SCDOT Undisturbed Sample Pictures

Project ID:	P037125	County:	25 - Hampton	Boring No:	STB-2A
Site Description:	S-140 Camp Branch		Route:	S-25-140	
UD Sample No.:	ST-1	Depth:	25.0' - 27.0'		
Date Sampled:	8/23/2019	Date Extracted:	9/4/2019		
Extracted By:	D. Schmidt	Eng. Firm:	HDR		

Specimen No. ST-1.C

Specimen No. ST-1.D

Figure 6-35, Shelby Tube Log Photograph Example

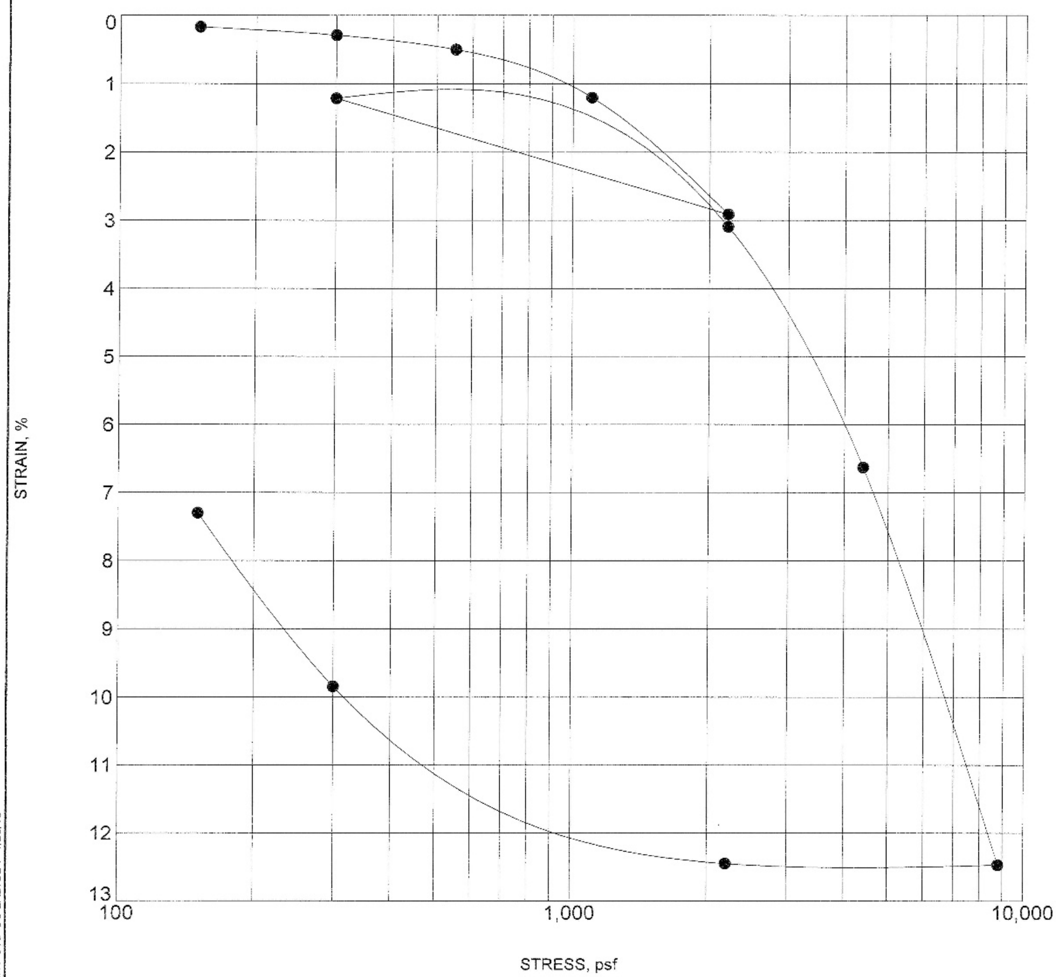
SCDOT

CONSOLIDATION TEST

PROJECT ID 0041401-B01

PROJECT NAME gINT Example

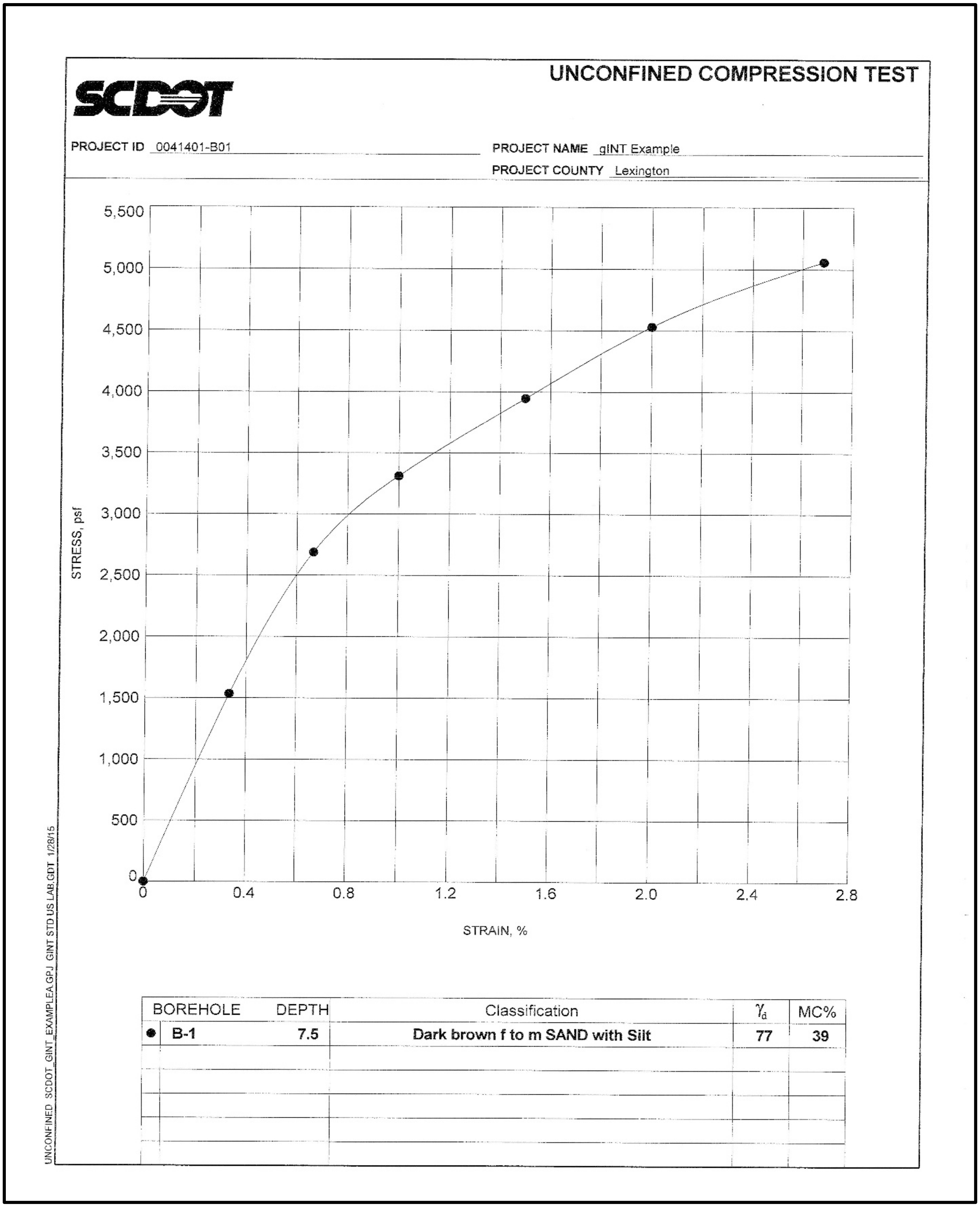
PROJECT COUNTY Lexington



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-1**	**25.0**	**Very dark grayish brown Clayey SAND**	**82**	**36**

CONSOL STRAIN SCDOT_GINT_EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

Figure 6-36, Consolidation Testing Results


SCDOT

UNCONFINED COMPRESSION TEST

PROJECT ID 0041401-B01

PROJECT NAME gINT Example

PROJECT COUNTY Lexington

	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	B-1	7.5	Dark brown f to m SAND with Silt	77	39

UNCONFINED SCDOT_GINT_EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

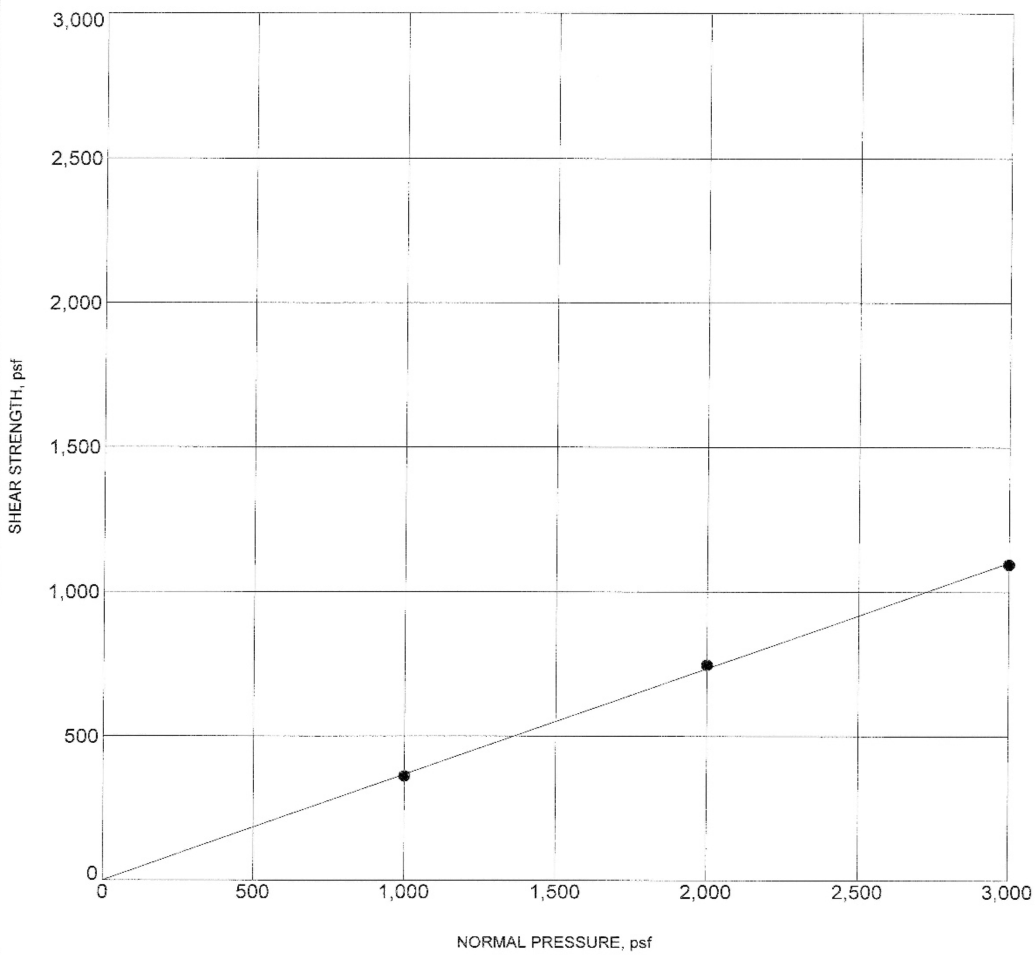
Figure 6-37, Unconfined Compression Testing Results

SCDOT

DIRECT SHEAR TEST

PROJECT ID 0041401-B01 PROJECT NAME gINT Example

PROJECT COUNTY Lexington



	BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ
●	**B-1**	**10.0**	**Dark brown f to m SAND with Silt**	**101**	**20**	**0.3**	**20**

DIRECT SHEAR SCDOT GINT EXAMPLEA.GPJ GINT STD US LAB.GDT 1/28/15

Figure 6-38, Direct Shear Testing Results

TRIAXIAL SHEAR TEST REPORT
(ASTM D 4767)

01-05-16LEXUS401

Project No :	1463-16-004A	Route :	US-76	Review Date :	7/7/16
Site Description: :	US 76 Lynches Bridge R.	Sample Date :	04/04/16	Reviewed By:	A. Boring
Sample ID :	TX-7 BS-2	Depth. :	1-5 feet -	Performed By :	T. Reid
Client :	SCDOT	Sample Type :	BULK - 2	Test Date :	04/05/16
Soil Description :	dark grayish br. & strong br., CLAYEY SAND (SC, A-6), 10YR 3/1			Specific Gravity, Gs:	2.71
Liquid Limit, %:	25	PI, % :	12	Fines, % :	40

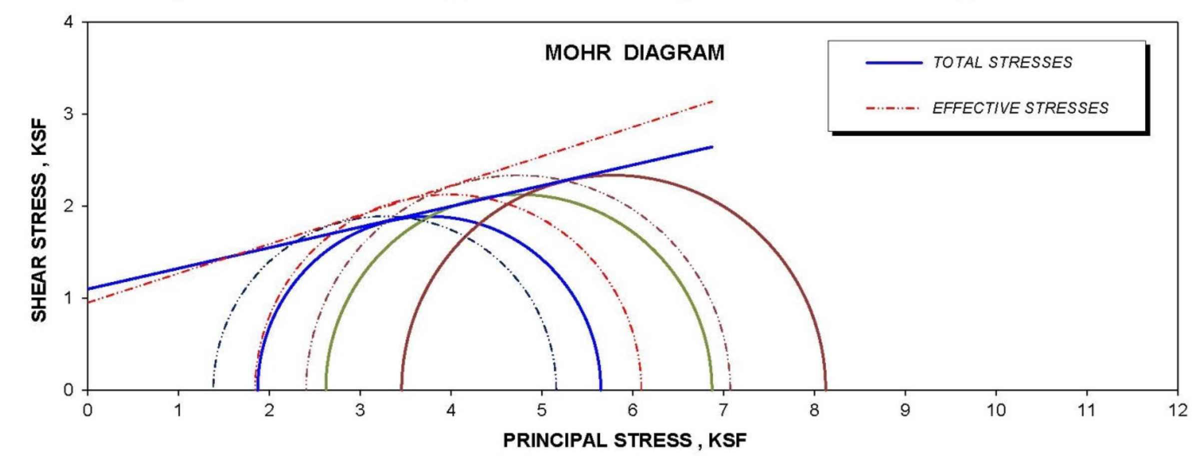
SPECIMEN PROPERTIES								
		INITIAL			AFTER CONSOLIDATION			
SPECIMEN NO.		1	2	3		1	2	3
DIAMETER , INCHES	D_o	2.80	2.81	2.81	D_c	2.79	2.79	2.79
HEIGHT , INCHES	H_o	6.03	6.02	6.03	H_c	6.01	6.00	5.99
WATER CONTENT, %	W_o	10.7	10.6	10.6	W_c	15.7	15.6	15.3
DRY DENSITY, PCF	γ_{dryo}	117.4	117.4	117.5	γ_{dryc}	118.7	118.9	119.5
SATURATION ,%	S_o	66	65	65	S_c	100	100	100
VOID RATIO	e_o	0.44	0.44	0.44	e_c	0.42	0.42	0.42

TEST PARAMETERS , TEST TYPE : CU/PP				
SPECIMEN NO.		1	2	3
B Value		0.96	0.95	0.95
BACK PRESSURE, ksf	U_o	14.38	13.10	13.24
CONFINING PRESSURE , ksf	σ_3	1.87	2.62	3.46
FAILURE DEVIATOR STRESS, ksf	$\sigma_1-\sigma_3$	3.78	4.25	4.67
ULT. DEVIATOR STRESS , ksf	$\sigma_1-\sigma_3$	3.78	4.25	4.67
STRAIN AT FAILURE	ε	15.0%	15.0%	15.0%
Specimen Shape @ Failure: See attached				

Strain: 0.012% per minute T50, Minutes = 2

FAILURE CRITERIA 2 Ult. Deviator Stress

SHEAR STRENGTH PARAMETERS	TOTAL		EFFECTIVE	
	COHESION, C (ksf):	1.096	APPARENT COHESION , C' (ksf):	0.950
	ANGLE OF INTER. FRICTION, ϕ :	12.7°	ANGLE OF INTER. FRICTION, ϕ' :	17.7°



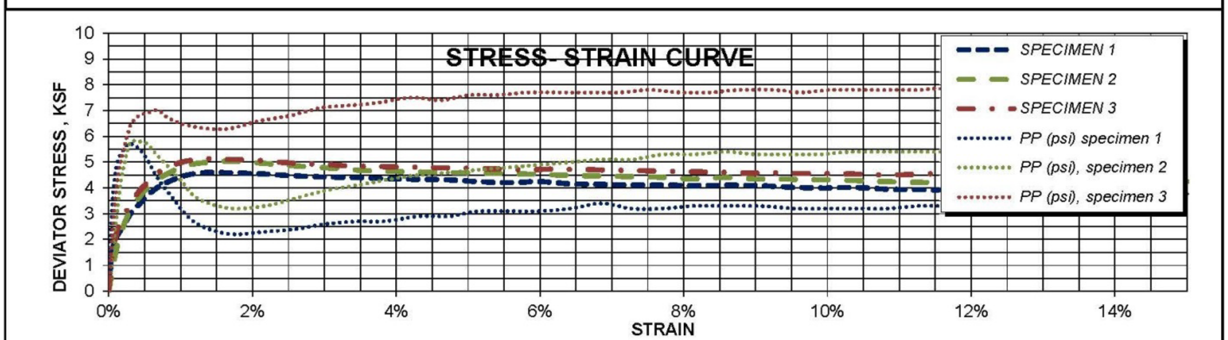


Figure 6-39, Triaxial Shear Testing Results

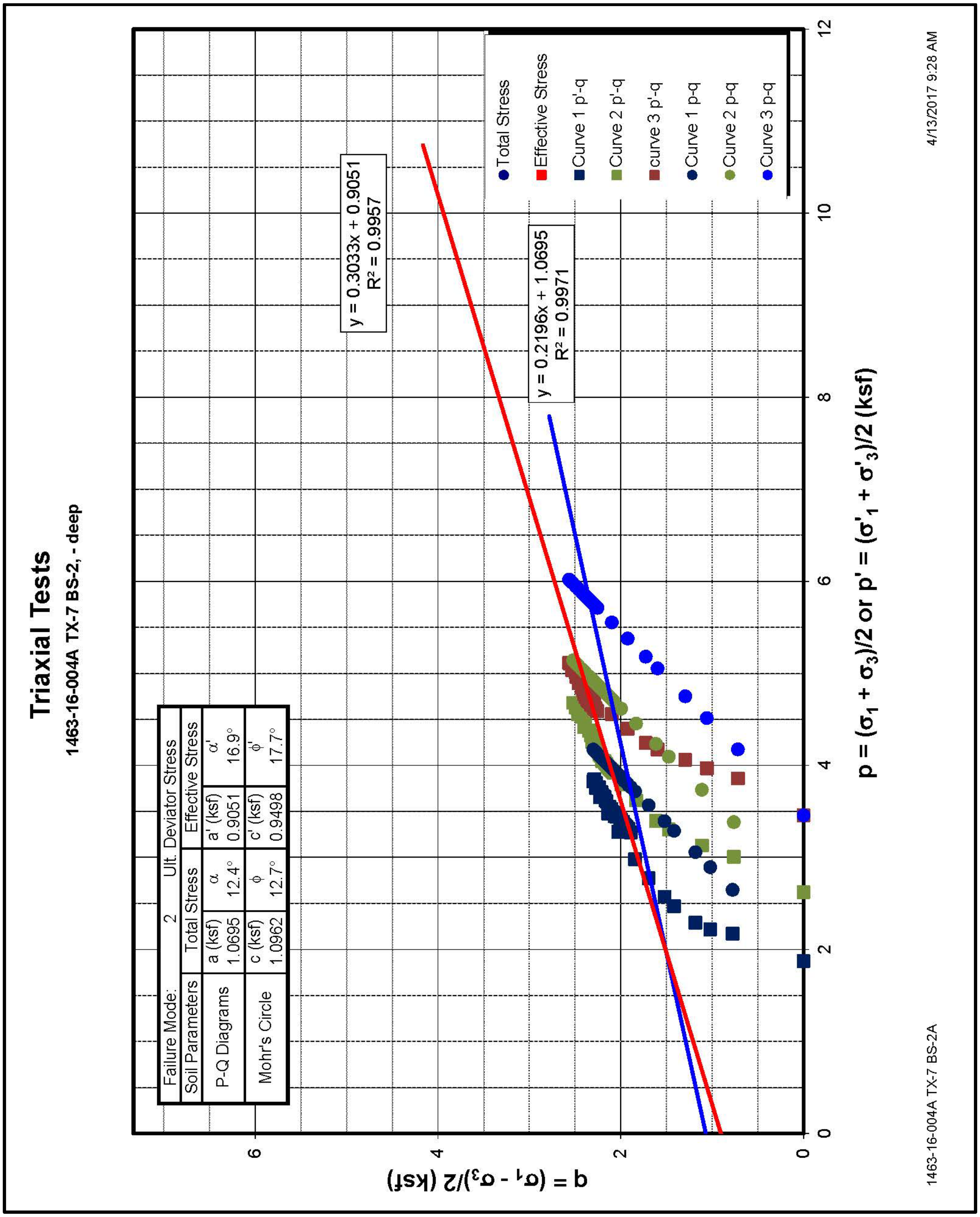


Figure 6-40, p-q Plot - Triaxial Shear Testing

Rock Coring Summary

Project ID: ____________________

Project Name: ____________________

Project County: ____________________

Borehole	Core Run Number	Core Run Top Depth (ft)	REC (%)	RQD (%)	q_u (psi)	Poisson's Ratio	Secant Modulus (ksi)	Unit Weight (pcf)	RMR / GSI

Page 1 of 1

Figure 6-41, Rock Coring Summary

Project	SC-823 BRO Little River	**Diameter, in.:**	1.99	**Date:**	5/10/2016
Project No.:	1461-15-030	**Length, in.:**	4.49	**Tested by:**	BKP
Boring Id:	B-7	**Unit Weight, pcf:**	189.5	**Reviewed by:**	JBB
Sample No.:	Run 1	**Moisture Content, %:**	0.1		
Depth (ft):	22.9-23.6	**Load Rate, psi/sec:**	70		

Data Point	Strain(10^{-6})		Load (lb)	Compressive stress (psi)	Secant Modulus $x10^6$ (psi)	Poisson's Ratio	Remarks Failure
	axial	radial					
1	0	0	0	0	0.00	0.00	
2	-50	12	2,000	643	12.86	0.24	
3	-94	27	4,000	1,286	13.68	0.29	
4	-146	39	6,000	1,929	13.21	0.27	
5	-198	54	8,000	2,572	12.99	0.27	
6	-253	68	10,000	3,215	12.71	0.27	
7	-302	82	12,000	3,859	12.78	0.27	
8	-355	97	14,000	4,502	12.68	0.27	
9	-404	113	16,000	5,145	12.73	0.28	
10	-462	130	18,000	5,788	12.53	0.28	
11	-513	145	20,000	6,431	12.54	0.28	
12	-569	161	22,000	7,074	12.43	0.28	
13	-623	179	24000	7,717	12.39	0.29	
14	-679	196	26,000	8,360	12.31	0.29	
15	-732	212	28,000	9,003	12.30	0.29	
16	-790	231	30,000	9,646	12.21	0.29	
17	-849	249	32,000	10,289	12.12	0.29	
18	-961	287	36,000	11,576	12.05	0.30	
19	-1,078	324	40,000	12,862	11.93	0.30	
20	-1,197	366	44,000	14,148	11.82	0.31	
21	-1,321	410	48,000	15,434	11.68	0.31	
22	-1,443	459	52,000	16,720	11.59	0.32	
23	-1,577	513	56,000	18,006	11.42	0.33	
24	-1,710	571	60,000	19,293	11.28	0.33	
25	-1,843	638	64,000	20,579	11.17	0.35	
28	-1,989	714	68,000	21,865	10.99	0.36	
29	-2,131	801	72,000	23,151	10.86	0.38	
30	-2,287	906	76,000	24,437	10.69	0.40	
31	-2,457	1,048	80,000	25,724	10.47	0.43	
32	-2,627	1,221	84,000	27,010	10.28	0.46	
33	-2,829	1,541	88,000	28,296	10.00	0.54	
34			89,530	28,788			Failure

Figure 6-42, Rock Core Testing Results

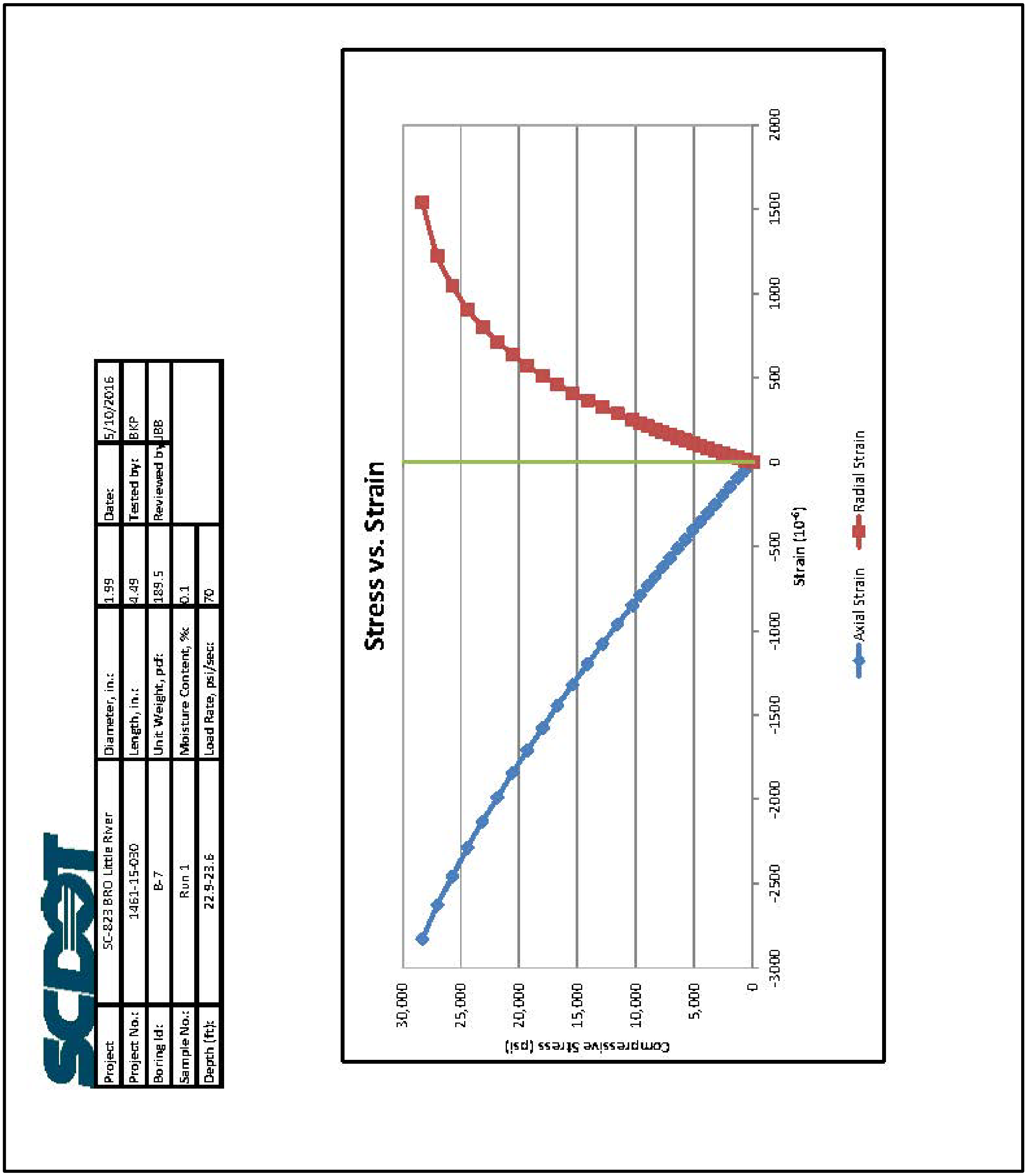


Figure 6-43, Rock Core Testing Stress versus Strain Graph