UNIVERSITY OF SOUTH CAROLINA

GEOTECHNICAL MATERIALS DATABASE FOR EMBANKMENT DESIGN AND CONSTRUCTION

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16. Abstract

This project was focused on the assimilation of engineering properties of borrow soils across the state of South Carolina. Extensive data on soils used for embankment construction were evaluated and compared within Group A (Piedmont) and Group B (Coastal Plain) soil deposits. A geotechnical materials database was constructed using three main sources of information: 1) review and synthesis of available soils information from 197 borrow pits gathered from the SCDOT Engineering Districts; 2) experimental testing of representative field samples of soils from 17 known borrow sources to determine the physical, mechanical, and chemical properties of these soils; and 3) triaxial compression test results from soil samples of existing embankments, based on a thorough review and synthesis of multiple project reports supplied by SCDOT. The geographical and geotechnical data for all of the identified borrow sources were compiled into a master spreadsheet using Microsoft® Excel.

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Executive Summary

This report describes the research conducted to develop the South Carolina Department of Transportation (SCDOT) Geotechnical Materials Database (GMD) for embankment design and construction. The identification and selection of local borrow soils with established engineering properties is a critical phase in embankment design and construction. Normally, designers and contractors must conduct expensive and time-consuming geotechnical tests to determine engineering properties, or if available, use their own prior experience. The SCDOT GMD provides an electronic resource with a compilation of the specific engineering properties of potential borrow materials available throughout South Carolina. It was created using data from three sources: 1) available information from the SCDOT Engineering District offices on borrow pits that have been used for embankment construction; 2) available triaxial test data on soil samples acquired from existing embankments; and 3) comprehensive experimental program conducted using bulk samples acquired from a select number of borrow pits representing different regions of the state.

Geographical and geotechnical information were gathered from 197 borrow pits across the state of South Carolina. Geotechnical data were available for 140 of the 197 borrow pits, although in most cases, the data were limited to soil descriptions that often included USCS and/or AASHTO soil classifications. In a few cases, data were provided on particle size distribution and/or soil compaction. It was determined that 37 of the 197 borrow pits were either active or accessible, and seventeen (17) were selected for sampling and testing. Three bulk samples were collected at each borrow pit. The locations of each sampling point were based on soil maps produced using the USDA Web Soil Survey, which delineates the soil units present in each borrow pit. Tests for physical properties included visual manual identification, moisture content, specific gravity, particle size distribution, liquid limit, plastic limit, and soil classification. Tests for mechanical properties included standard Proctor compaction, direct shear, and triaxial compression, which were used to determine the most critical soil properties including maximum dry density ($\gamma_{d,max}$), optimum water content (w_{opt}), effective friction angle, ϕ' and effective cohesion, c'. Tests for chemical properties included soil pH, soil resistivity, chloride content, and sulfate content. Test methods were performed according to AASHTO standard specifications, with two exceptions for chloride and sulfate contents, which were determined using USEPA test methods.

The SCDOT has created two categories of borrow soils, Group A and Group B, based on the geological environment in South Carolina. The 17 borrow pits selected for experimental studies are distributed within these two groups. Group A soils are located north and west of the Fall Line in the Blue Ridge and Piedmont physiographic geologic units. Here, most soils were formed as residuum of the underlying parent rock and therefore reflect the properties of the weathered parent material. These residual soils are often difficult to place and compact during embankment construction, and can be susceptible to erosion. Group B soils are

located south and east of the Fall Line in the Coastal Plain physiographic geologic unit. Coastal Plain units are identified with age and progress from the present coastline, where the youngest deposits reside, northwest toward Columbia. A diverse assortment of sands appears throughout the Coastal Plain region.

The SCDOT GMD shows that the predominant USCS and AASHTO soil classifications differ between Group A and Group B soil deposits, as expected. In general, the soils in Group B have lower fines content than those in Group A. SP-SM and SW-SM soils are common in Group B but are not found in Group A. The fines content of SM and SC soils in Group B does not exceed 32%; whereas, all but one of the SM soils in Group A has at least 35% fines. In terms of AASHTO classifications, Group B soils range from A-1 to A-4 and there are no soils with A-5 or higher classifications. In Group A, the preponderance of soil samples are classified as A-5 or higher.

The compaction characteristics are a function of soil classification. In Group A, the A-2-4 and A-4 soils have the highest $\gamma_{d,max}$ (> 115 pcf in some cases) and lowest w_{opt} required for compaction. The A-5 and A-7-5 soils have the lowest $\gamma_{d,max}$ (< 100 pcf in some cases) and require the highest w_{opt} for compaction. More than half of the Group A soils have $w_{opt} \geq$ 20%. Mica was observed to be present in some of these soil samples. In Group B, the A-1 and A-2 soil groups tend to produce a higher $\gamma_{d,max}$ at lower w_{opt} than the A-3 and A-4 soil groups. All of the Group B soil samples with $\gamma_{d,max}$ of at least 110 pcf are in the A-1 and A-2 soil groups. All of the Group B soils have w_{opt} < 20%.

On average, Group A soils have higher effective friction angles than Group B soils. The results for Group A soils are in agreement with published shear strength parameters for Piedmont residual soils that indicate an average effective friction angle of 35.2° with a ± 1 standard deviation range of $29.9° < \phi' < 40.5°$. In Group B soils, the effective friction angles for SC, SC-SM, CL and ML soils range from $28° < \phi' < 32°$, which is consistent with prior SCDOT experience in the Coastal Plain. Most of the SM soils, however, were found to have higher effective friction angles ranging from $34° < \phi' < 36°$.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
In	inches	25.4	millimeters	mm
Ft	feet	0.305	meters	m
Yd	yards	0.914	meters	m
Mi	miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
Ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	ml
Gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3				
MASS				
Oz	ounces	28.35	grams	g
Lb	pounds	0.454	kilograms	kg
T	Short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact)				
^oF	Fahrenheit temperature	5(F-32)/9 Or (F-32)/1.8	Celsius Temperature	^0C
ILLUMINATION				
Fc	foot-candles	10.76	lux	lx
Fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
Lbf	poundforce	4.45	Newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	Short tons (2000 lb)	T
TEMPERATURE (exact)				
^0C	Celsius Temperature	1.8C+32	Fahrenheit temperature	^oF
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	Newtons	0.225	Poundforce	lbf
kPa	kilopascals	0.145	Poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E38.

Chapter 1 – Introduction

1.1 Problem Statement

Embankment design and construction is one of the most important phases of highway construction. The identification and selection of local borrow soils with established engineering properties is a critical first phase in the process. In the SCDOT Construction Manual (2004a), Section 200.2.6 on Embankment Soil Material, it is recognized that embankment performance is, among other factors, a function of the engineering characteristics of the embankment material, proper placement of the embankment material in lifts, control of moisture content near optimum during compaction, and compaction of each lift of embankment material to target density. To ensure the structural integrity of compacted earthen embankments, the desired mechanical properties must be achieved through appropriate materials selection and careful construction techniques. If unsuitable soils or improper construction techniques are used, uneven settlement or lateral displacement of the embankment can develop to a point that renders the embankment unstable. In some cases, borrow soils might provide adequate short-term soil properties, but their performance can deteriorate with time.

Designers and contractors often must conduct expensive and time-consuming soil and rock testing for engineering properties, or if available, use their own prior experience. Currently, there is a limited compilation of the specific engineering properties of potential borrow material, including soil and rock, available throughout South Carolina for embankment construction. Properties that influence compacted soil behavior include, but are not limited to, particle size distribution, liquid limit (LL) and plasticity index (PI), maximum dry density ($\gamma_{d,max}$) and optimum moisture content (w_{opt}), and drained shear strength parameters, ϕ' and c'. The development of a statewide geotechnical materials database that contains engineering properties of borrow materials would provide designers and contractors with a reliable resource on local soils. With this resource, the time and cost efforts associated with embankment design can be reduced significantly. It will provide a means to evaluate spatial and geological variability of soil deposits within a particular area of the state, and most importantly, facilitate the appropriate selection of soil shear strength parameters.

1.2 Project Objectives

There are three main research objectives for this project:

- Review and synthesize readily available soils information from existing SCDOT archives to determine the distribution of soil classification and engineering properties encountered in Group A (Piedmont) and Group B (Coastal Plain) soil deposits, and then divide and organize soils data according to each one of the seven Engineering Districts and 46 counties;

- Collect representative field samples of soils from known borrow sources at distributed locations within Group A and Group B soil deposits, and then determine the physical, mechanical, and chemical properties of these soils in accordance with applicable AASHTO, ASTM, or SC-T standard specifications; and

- Compile all of the accumulated information in a Microsoft® Excel format that will serve as a geotechnical materials database.

The purpose of the database is to provide information on soil types and soil properties in a given area of the state, such that it can be used to guide the selection of appropriate parameters for embankment design. The database will not be used to provide implicit prior approval of borrow pits.

1.3 Report Organization

This report is organized into eight chapters. Chapter 2 provides background information relevant to the project. Chapters 3 and 4 describe the field sampling and lab testing programs. Chapters 5 and 6 contain the experimental results. Chapter 7 offers conclusions and recommendations for implementation and further studies. Chapter 8 provides the list of references. The content of Chapters 2 through 6 is described in more detail in the following paragraphs.

Chapter 2 provides a brief overview of SCDOT requirements for soil selection and embankment design. It also describes the geological provinces in South Carolina, with an emphasis on residual soil formation in the Piedmont region. Lastly, it provides the borrow material specifications for Group A and B soil deposits, along with guidance on expected soil shear strength parameters for these two soil groups.

Chapter 3 describes the compilation of borrow pits identified across the state, and the sampling program that was developed to obtain soils from borrow pits in Group A and B soil deposits. The methods for identifying a representative subset of borrow pits for sampling are presented first, followed with the field sampling procedures, which include the use of soils maps and surveys to locate sampling points within each pit.

Chapter 4 describes the test methods used in the experimental program to determine the physical, mechanical and chemical properties of the soil obtained from each borrow pit located in Group A and B soil deposits. Tests for physical properties include visual-manual identification, moisture content, specific gravity, particle size distribution, liquid limit, plastic limit, and soil classification. Tests for mechanical properties include standard Proctor compaction, direct shear, and triaxial compression. Standard Proctor compaction tests provide the maximum dry density and optimum moisture content needed for the preparation of direct shear and triaxial compression tests, which are performed to determine shear strength parameters. Tests for chemical properties include soil pH, soil resistivity, chloride content and sulfate contents. Test methods were performed according to AASHTO standard specifications unless otherwise indicated.

Chapter 5 presents the results of the laboratory testing program performed to determine the index properties (specific gravity, particle size distribution, liquid limit, plastic limit, maximum dry density and optimum moisture content), soil classification according to AASHTO and USCS, and the chemical properties (soil pH, soil resistivity, chloride content and sulfate content).

Chapter 6 presents the shear strength parameters (effective friction angle, ϕ', and effective cohesion, c') of Group A and B soil deposits obtained through 1) a synthesis of data provided by the SCDOT for recent embankment projects and 2) a series of consolidated undrained static triaxial compression tests and direct shear tests performed in the USC Geotechnical Laboratory on soil specimens prepared from field samples of borrow pits.

Chapter 2 – Background

2.1 Introduction

Earthen embankments are common and critical elements in transportation infrastructure. Embankments must support the pavement structure and the traffic loads that the pavement transfers into the supporting embankment. A proper embankment design must consider:

- the in-situ and as-placed properties of the fill material to be used for construction;
- properties of foundation materials;
- the local hydrological regime;
- strength and consolidation characteristics of fill and foundation soils; and
- safe slope angles for construction.

Adequate embankment performance depends on:

- strength of the soil material under the embankment;
- engineering characteristics of the embankment material;
- proper construction of benches and transitions;
- proper placement of the embankment material in lifts;
- control of moisture content near optimum during compaction; and
- compaction of each lift of embankment material to target density.

If unsuitable soils or improper construction techniques are used, the embankment can deform requiring slope stabilization and pavement maintenance, like the example shown in Figure 2.1. In extreme cases, the embankment can fail and lead to complete pavement failure, as shown in Figure 2.2. The selection of proper shear strength parameters for the compacted soils in each embankment design is critical.

2.2 SCDOT Requirements for Embankment Design and Construction

According to the SCDOT Geotechnical Design Manual (2010), the soil shear strength design parameters must be locally available, cost effective, and be achievable during construction. The selection of soil shear strength design parameters that require importing materials from outside of the general project area should be avoided.

The method of selecting soil shear strength parameters for compacted soils will be either 1) measured using consolidated undrained triaxial tests with pore pressure measurements, or 2) conservatively selected based on drained soil shear strength parameters typically encountered in South Carolina soils.

Figure 2.1. Evidence of Embankment Slope Deformation (http://mceer.buffalo.edu)

Figure 2.2. Highway Embankment Failure (http://mceer.buffalo.edu)

The selection of soil shear strength parameters for embankment design and construction depends on whether the project is design-build, traditional design-bid-build with existing embankments, or traditional design-bid-build on new alignment. With design-build projects, local borrow soils must be sampled and tested for the following:

- particle size distribution with wash No. 200 sieve;
- liquid limit, plastic limit, and plasticity index (Atterberg Limits);
- standard Proctor compaction; and
- shear strength as determined from triaxial compression (TXC) consolidated undrained (CU) tests with pore pressure measurements. Samples must be remolded and compacted to achieve 95% relative compaction at a moisture content of -1% to +2% of the optimum moisture content.

With traditional design-bid-build projects, the shear strength parameters are based on tests of existing embankment soil samples, as long as similar soils are confirmed to be locally available. With embankments on new alignments, shear strength parameters are pre-selected based on knowledge of local soils and do not require lab testing.

The SCDOT Geotechnical Design Manual (2010) offers guidance on maximum allowable soil shear strength parameters based on soil classification, as shown in Figure 2.3. Maximum total shear strength for cohesive soils is limited to 1,500 psf for CL-ML soils and 2,500 psf for CL and CH soils. However, shear strength parameters exceeding these limits can be used if the specific source of material is identified for the project and enough material is available for construction.

Table 7-17, Maximum Allowable Soil Shear Strengths For Compacted Soils

Soil Description		Effective	
USCS	Description	c' (psf)	ϕ' (degrees)
GW, GP, GM, GC	Stone and Gravel	0	38
SW	Coarse Grained Sand	0	36
SM, SP	Fine Grained Sand	0	34
SP	Uniform Rounded Sand	0	30
ML, MH, SC	Silt, Clayey Sand, Clayey Silt	50	28
SM-ML	Residual Soil	50	24
CL-ML	Clay (Low Plasticity)	50	32
CL, CH	Clay (Med-High Plasticity)	50	26

Figure 2.3. Table 7-17 from the SCDOT Geotechnical Design Manual (2010)

2.3 South Carolina Soils

2.3.1 South Carolina Geological Regions

The geology of South Carolina has yielded a rich variety of minerals and igneous, metamorphic, and sedimentary rocks. Products of weathering from these rocks as well as accumulation of shoreline sediments form the basis for borrow materials used in construction. Products of weathering are often found in the Piedmont region, whereas shoreline sediments have accumulated throughout the Coastal Plain. While South Carolina is divided into these two major geological provinces, each province may be further subdivided into more precise units as shown in Figure 2.4.

While the variety of materials available means that engineers have many grades of materials to work with, it presents a problem in dealing with borrow material in a consistent manner. Many materials with specific engineering properties and behaviors may be available in one part of the state, and not in another part. To assess the suitability of a given borrow material for an intended engineering application, like embankment construction, a useful set of engineering properties should be developed.

Coastal Plain units are identified with age and progress from the present coastline, where the youngest deposits reside, northwest toward Columbia. A diverse assortment of sands appears throughout the Coastal Plain region. Some deposits near the Orangeburg Scarp are dune sands with fairly uniform particle distribution and varying degrees of kaolin interspersed within. Other deposits may be intermixed with alluvial outwash (rounded gravels of varying quality) or near-coastal deposits of calcareous (shell) materials, coquina, or organics. The various depositional processes that have occurred throughout the Coastal Plain lead to an assortment of potential borrow sources.

Further upland, most soils were formed as residuum of the underlying parent rock and therefore reflect the properties of the weathered parent material. These residual soils are often difficult to place and compact during embankment construction, and can be susceptible to erosion. Exceptions in the Piedmont region occur where alluvial valleys have formed and deposited gravels and sands of variable sizes, mineral content, and engineering properties. Areas where clay is mined generally reflect the desired end use. Some clay deposits in South Carolina are ideally suited for porcelain production, others for brick, while the remainder, if carefully handled, can be used as structural fill. Some of these materials may require modification with cement, lime, additional compactive effort, or other means of stabilization.

For example, in SCDOT Instructional Bulletin No. 2004-10 (2004b), it is recognized that certain borrow soils are unsuitable for subbase unless modified with cement. As noted in this bulletin, there are 18 counties generally located in the Piedmont region (e.g. Anderson, Greenville, Spartanburg counties) that have experienced a hardship in readily finding satisfactory borrow material.

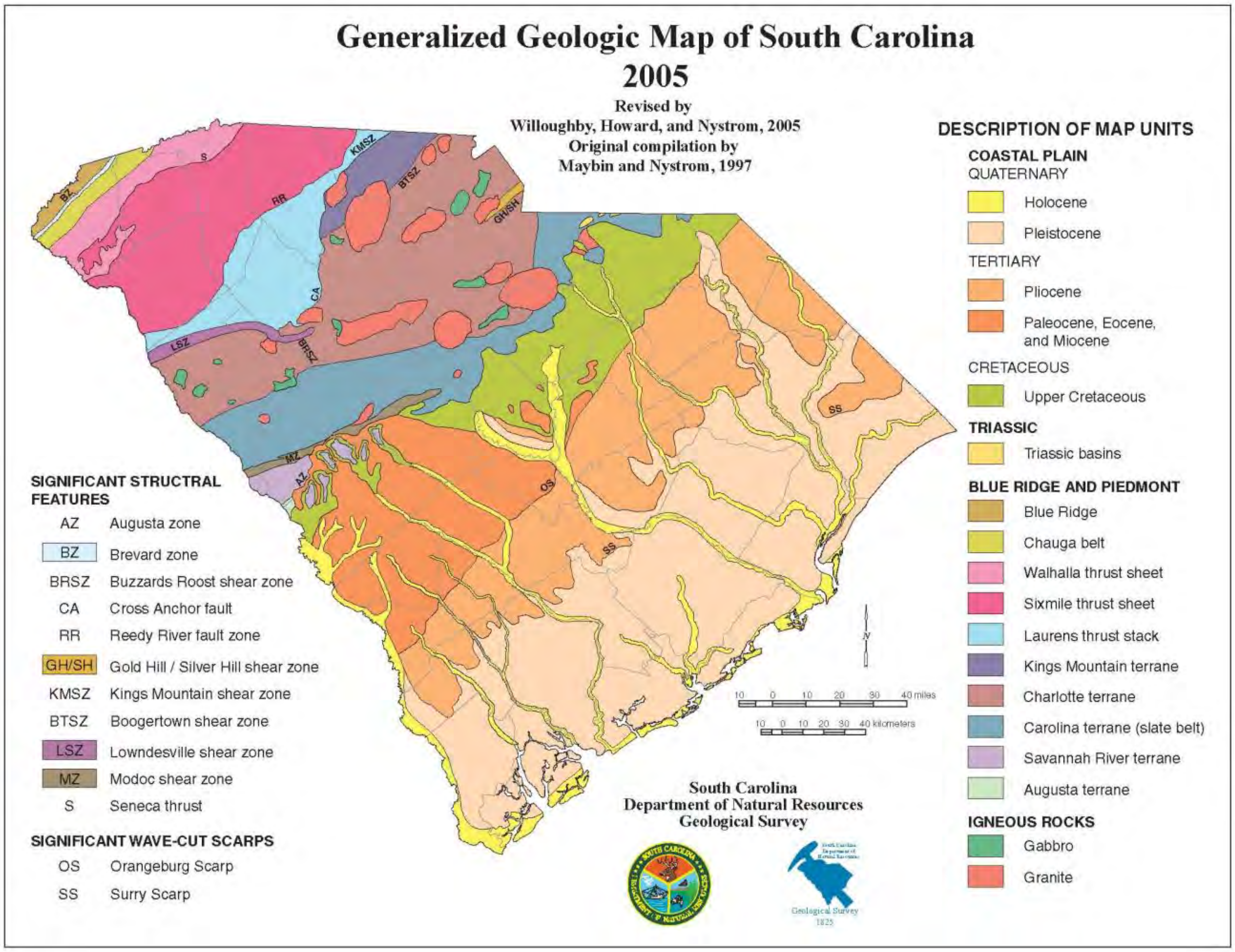

Figure 2.4. Generalized Geologic Map of South Carolina (South Carolina Department of Natural Resources, http://www.dnr.sc.gov/geology/geology.htm)

The Piedmont region is located in the eastern United States, and it is over 800 miles long, covering a 30 mile wide stretch in Maryland to about 125 miles in North Carolina. Lengthwise, it starts in Alabama, runs through Georgia, South and North Carolina, Virginia, Pennsylvania and finishes in New Jersey, as shown in Figure 2.5. The topography of the Piedmont consists of broadly rolling hills, with the hilltops forming flat ridges where major transportation routes are placed. Streams that run through the area form narrow v-shaped valleys that are characterized by shallow water depths and occasional shoals (Sowers 1954). The region drains in a southern and southeastern direction towards the Atlantic Ocean.

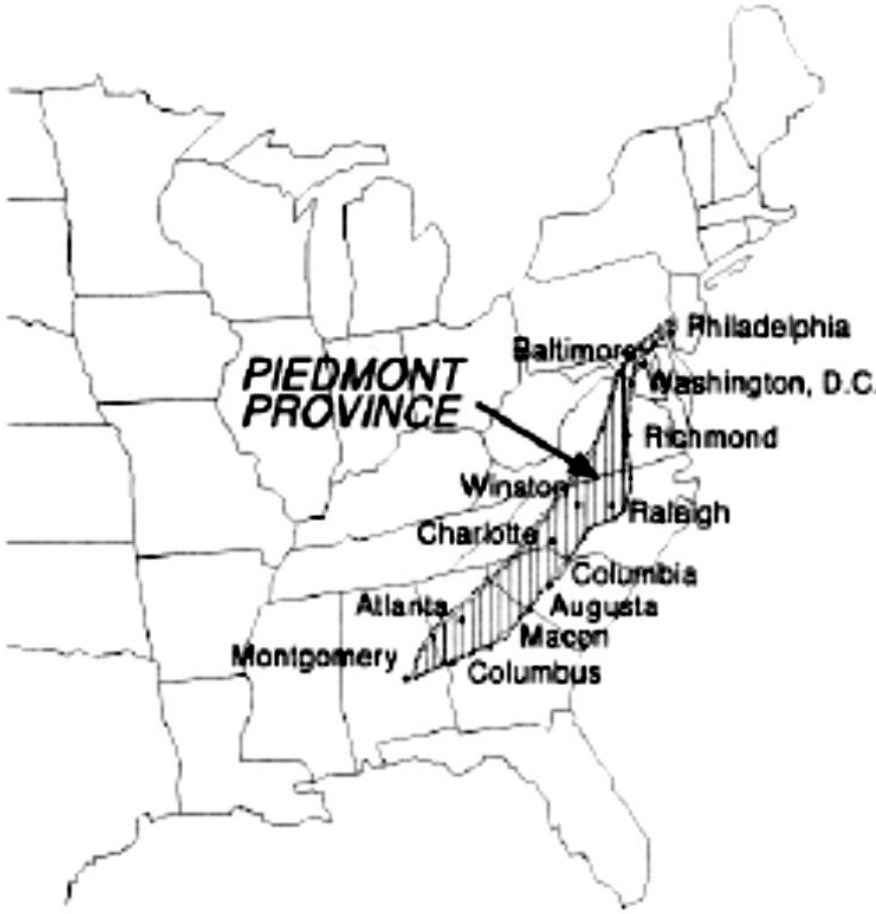

Figure 2.5. Map of the Piedmont Region along the Eastern U.S. (Mayne and Dumas 1997)

The geologic material for the Piedmont region consists mainly of metamorphic rock intruded by igneous rock. There are some unmetamorphosed sedimentary rock formations, but they are rarer. White and Richardson (1987) discuss the geologic formation of rocks in the Piedmont, along with Waisnor et al. (2001) and Sowers (1954). Metamorphic rocks in the Piedmont have mostly metamorphosed from sedimentary rocks of the Precambrian and the lower Paleozoic age, primarily gneisses, schists, amphibolites, phyllites, quartzite, slates and marble. The oldest rocks are gneisses and schists that were formed during the Precambrian era from sedimentary and igneous rocks. Theses rocks, due to the effect of heat and pressure from the metamorphic process, have their minerals segregated into parallel bands. The bands remain parallel and generally dip in one direction, even though the bands themselves appear twisted and swirled.

Due to volume changes and directed stresses during the last major period of deformation, joints formed in the rock. Fluids flowed through these cracks often and deposited minerals, including zeolite, calcite, chlorite and quartz. These joints allow for chemical weathering to occur more easily, and in most parts of the Piedmont these joints control the degree of weathering and resulting topography. The joint set orientations can be described as uniform or random, depending on the area, and faults also exist throughout the region.

Soils in the Piedmont region weather in accordance to other residual soils, with a profile that shows the most advanced weathering at the ground surface and decreasing degrees of weathering with depth. Two example weathering profiles are illustrated in Figure 2.6. Although the boundaries between zones are not well defined and gradual transitions between them are the norm, four zones can be identified:

1. upper zone – completely weathered soil with well-developed soil horizons. This is the part of the soil profile that most represents the source of borrow material;
2. intermediate structure – saprolite that retains the structure of the original rock but also shows soil texture;
3. partly weathered zones – alternate areas of saprolite and partially weathered rock; and
4. bedrock – unaltered or slightly altered rock.

These zones have been defined from an engineering perspective, as shown in Figure 2.7. The boundaries between zones are often not gradual, and the weathering is more advanced adjacent to joints and mineral bands, which leads to variations in soil depth even in small areas. Climate in the Piedmont region is particularly favorable for deep and rapid weathering with high and well distributed rainfall throughout the region, where annual rainfall rates are on the order of 50 in.

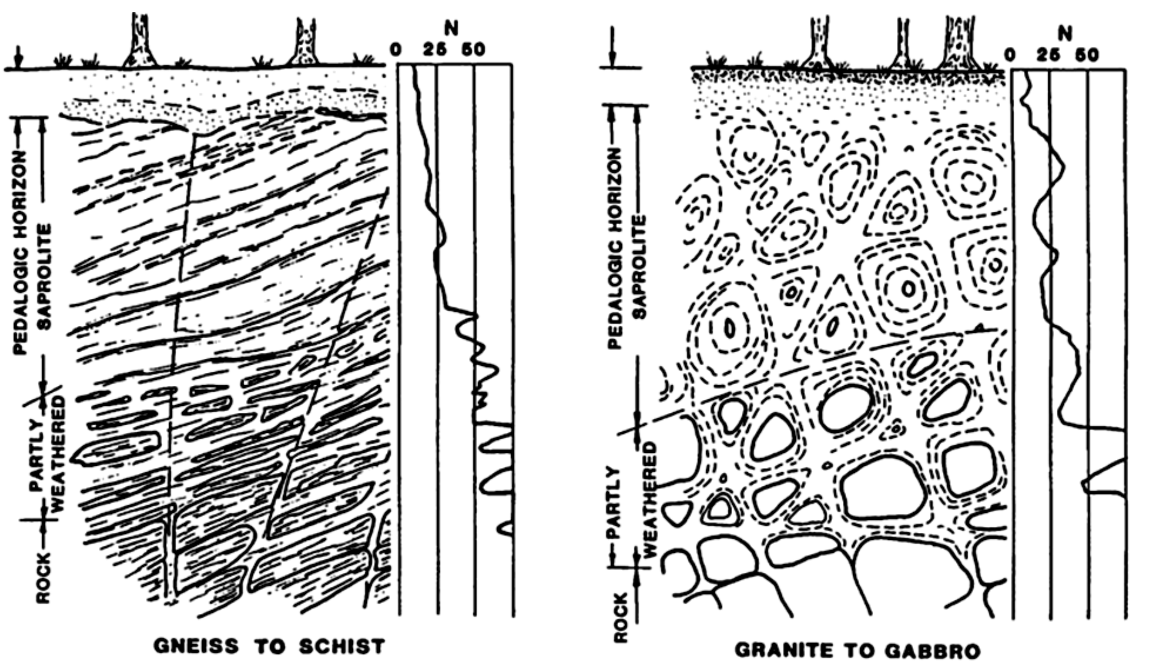

Figure 2.6. Piedmont Weathering Profiles (Sowers 1994)

In the upper zone, these residual soils show the most advanced degree of weathering. The soil minerals include quartz, kaolinitic clays, iron oxides and small amounts of weathered mica. Soils in this zone tend to be classified as CL or CL-ML, and can be stiff because the in situ moisture content is often below its plastic limit. This zone averages 3 to 5 ft in thickness, but can be as much as 10 ft thick.

<table>
<tr><th>Sowers (1963)</th><th colspan="2">Deere & Patton (1971)</th><th>Law/MARTA (Richardson & White, 1980)</th><th>Schnabel Engineering Associates (from Martin, 1977)</th></tr>
<tr><td rowspan="2">Soil
N=5-50</td><td rowspan="3">I
Residual Soil</td><td>IA
A Horizon</td><td rowspan="2">Upper Horizon
No Residual Structure</td><td rowspan="3">Residual Soil
N < 60</td></tr>
<tr><td>IB
B Horizon</td></tr>
<tr><td>Saprolite
N=5-50</td><td>IC
C Horizon</td><td>Saprolite</td></tr>
<tr><td rowspan="2">Partially Weathered Rock - Alternate Hard & Soft Seams
N>50</td><td rowspan="2">II
Weathered Rock</td><td>IIA
Transition From Residual Soil to Partially Weathered Rock</td><td>Partially Weathered Rock
N>100
Core Recovery<50%</td><td>Disintegrated or partially weathered rock
N≥60</td></tr>
<tr><td>IIB
Partly Weathered Rock</td><td>Rock Core Recovery>50%
RQD<50%</td><td rowspan="2">Rock
N≥100/2"
Core For Confirmation</td></tr>
<tr><td>Rock
RQD>75%</td><td colspan="2">III
Unweathered Rock
RQD>75%</td><td>Sound Rock
RQD>50%
Core Recovery>85%</td></tr>
<tr><td colspan="5">RQD = Rock Quality Designation
N=Standard Penetration Test N-Value (blows/foot)</td></tr>
</table>

Figure 2.7. Engineering Definitions for Piedmont Weathering Zones (Wilson and Martin 1996)

Given that the upper zone tends to be shallow, borrow soil might also come from within the transition from the upper zone to the intermediate zone. These soils are somewhat less weathered and are composed of quartz, kaolinitic clays and mica. Mica content in the parent rock can be appreciable, so larger amounts of unweathered mica up to 20 or 30% can be present in the soil. Determination of an accurate liquid limit for these soils is hindered because the soil tends to slide in the cup instead of flow.

2.3.2 South Carolina Borrow Material Specifications

The SCDOT Geotechnical Design Manual (2010) specifies two soil groups for borrow materials, as shown in Figure 2.8. The two groups are designated as Group A and Group B, and are essentially divided at the geological Fall Line.

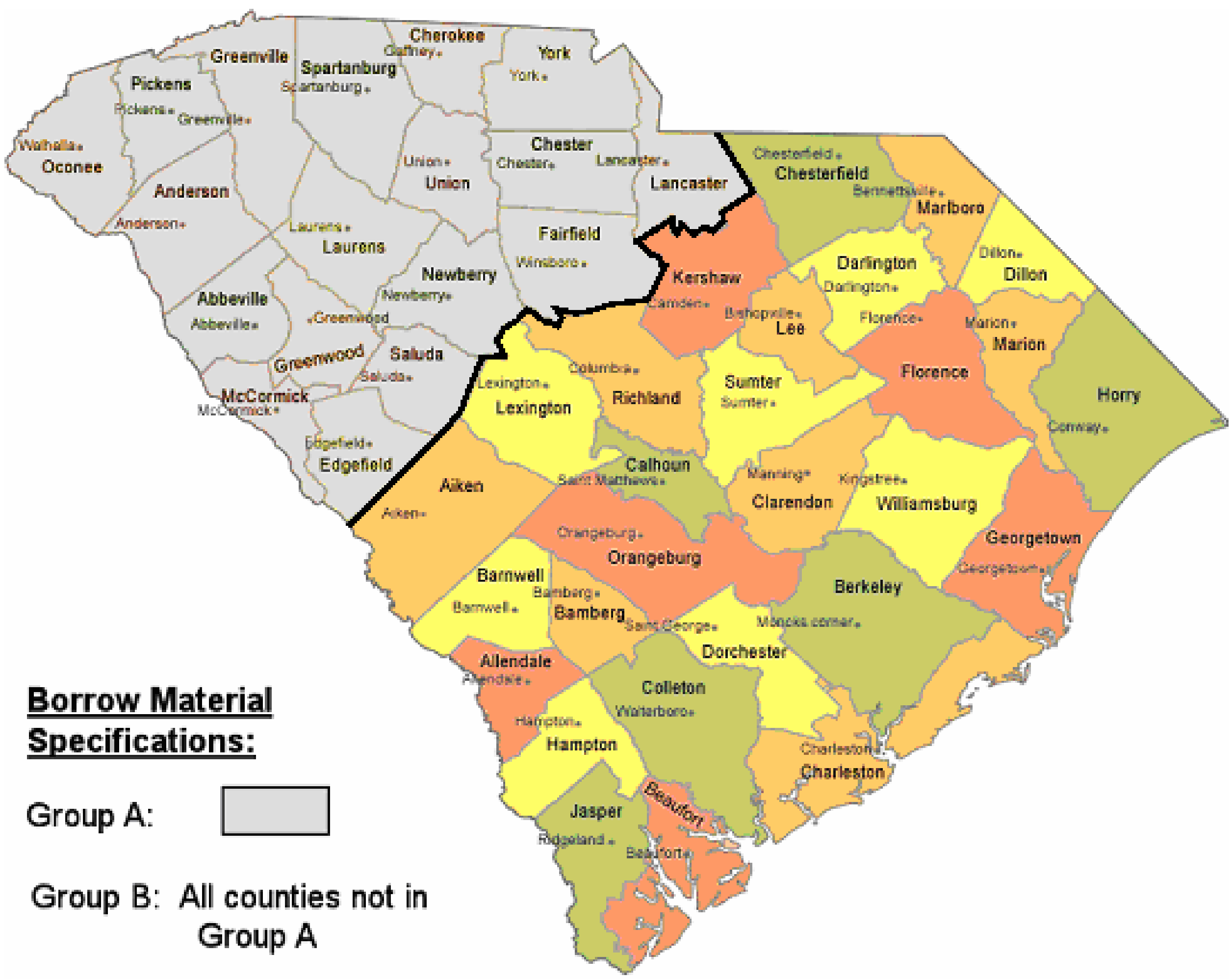

Figure 2.8. South Carolina County Map of Borrow Material Specifications (SCDOT 2010)

Group A: This group is located northwest of the Fall Line in the Blue Ridge and Piedmont physiographic geologic units. The uppermost Blue Ridge unit contains surface soils that show a residual soil profile, with clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands. There are also colluvial deposits on the slopes. The

Piedmont unit has a similar residual soil profile, with clayey soils near the surface and sandy silts and silty sands underneath.

SCDOT experience with borrow materials found in Group A are Piedmont residual soils classified as micaceous clayey silts and micaceous sandy silts, clays, and silty soils in partially drained conditions. These soils tend to have USCS classifications of either ML or MH and typically have liquid limits greater than 30. Published laboratory shear strength testing results for Piedmont residual soils (Sabatini 2002) indicate an average effective friction angle, ϕ', of 35.2° with a ± 1 standard deviation range of $29.9° < \phi' < 40.5°$ and a conservative lower bound of 27.3°.

Group B: This group is located south and east of the Fall Line in the Coastal Plain physiographic geologic unit. Sedimentary soils are found at the surface and consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone, depending on the location.

SCDOT experience with borrow materials found in Group B has shown that, when uniform fine sands are used, these soils can sometimes be difficult to compact and behave similar to silts. When these soils are encountered, caution should be used in selecting effective friction angles since values have been shown to range from $28° < \phi' < 32°$.

Chapter 3 – Sampling Program

This chapter describes the sampling program that was developed to obtain soils from borrow pits in Group A and B soil deposits from which soils have been excavated for the specific purpose of embankment construction. Procedures were developed to identify and locate borrow pits across the state of South Carolina and then select a representative subset for field sampling; these are described first. Then, the field sampling procedures are presented, including the use of soils maps and surveys, selection of sample locations within each pit, and methods for bulk sampling.

3.1 Identification of Borrow Pits

The first step in this project was to locate borrow pits from which soils have been excavated for the specific purpose of embankment construction. The goal was to accumulate information on a sufficient number of borrow pits across the state of South Carolina such that a representative subset could be selected for field sampling. It was expected that the preponderance of borrow pits would be identified within the approximate vicinity of interstate highways and/or high population areas. Figure 3.1 is a state map that illustrates the location of each one of the 46 counties, the current corresponding SCDOT Engineering District 1 through 7, and the interstate highway system. Borrow pits were identified within each SCDOT Engineering District, and the results are summarized in Table 3.1.

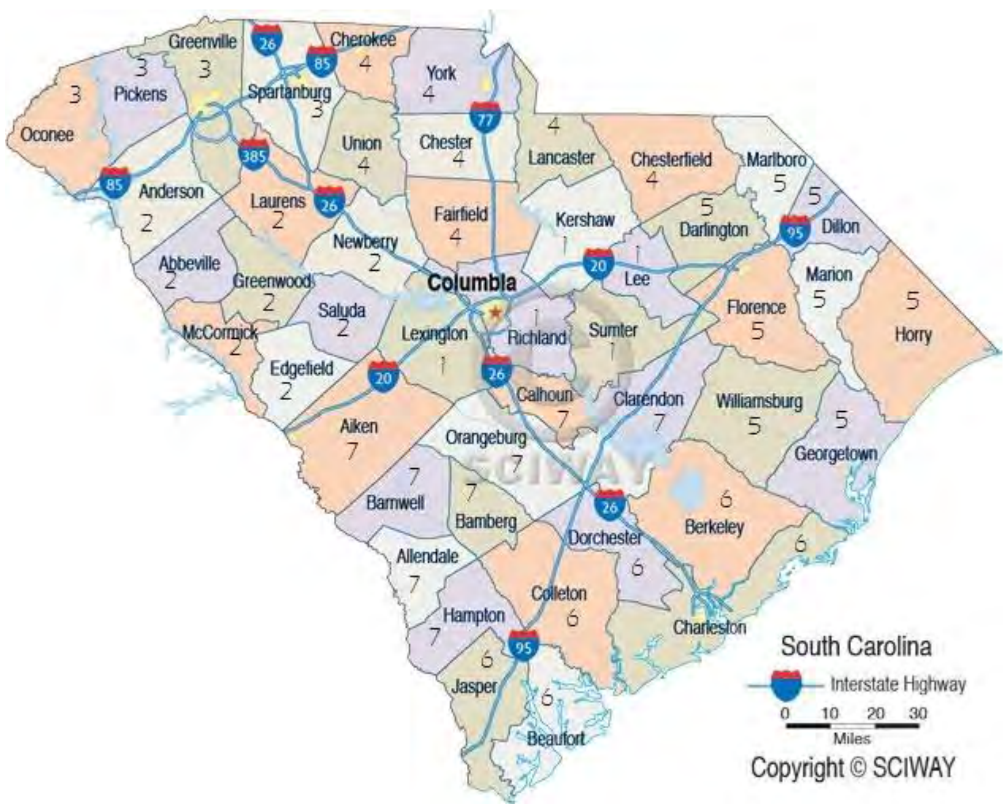

Figure 3.1. Map of South Carolina Counties and Engineering Districts

Table 3.1. Identified Borrow Pits from the Data Provided by the SCDOT

SCDOT District	Number of Pits	Active Pits	Pits with Geotechnical Data	Prominent Geological Region
1	53	14	46	Carolina Sand Hills
2	8	3	2	Southern Piedmont
3	25	7	13	Blue Ridge
4	15	3	2	Southern Piedmont
5	35	5	32	Coastal Plain
6	20	5	8	Coastal Plain
7	40	0	37	Carolina Sand Hills
Total	197	37	140	

Information on borrow pits was solicited from each SCDOT Engineering District. Requests were made for pit location (address and/or GPS coordinates), size in acreage, status (active or inactive), and owner and/or operator contact information. As shown in Table 3.1, a total of 197 borrow pits were identified. Information was also requested on soil classification and soil properties for each pit. Geotechnical data were available for 140 of the 197 borrow pits. In some cases, the geotechnical data were extensive and included particle size distribution and standard Proctor compaction for multiple samples within a given pit. In most cases, however, the data were limited to soil classification or a description of soil type. Each pit was considered to have data if some engineering properties other than soil color were reported.

It was also reported that 37 of the borrow pits were considered to be active, or open, and the remaining pits were considered to be closed. This information was reported from each district and was based on their knowledge or judgement of pit status at that time. For example, the 40 borrow pits identified in District 7 were based on projects from the 1960s and were all considered to be closed. However, owners and/or operators of all pits were contacted to verify pit status. Of the 197 borrow pits, 107 did not have contact information and 36 others were found to have outdated or incorrect contact information, leaving 54 borrow pits with verifiable information. Locations of those remaining 54 borrow pits are shown in Figure 3.2.

Each one of the 197 borrow pits was named using a three-part designation convention as follows:

1. the first part indicates the SCDOT Engineering District in which the borrow pit was located (e.g. D1 represents District 1);
2. the second part indicates the county in which the borrow pit was located (e.g. Lexington); and
3. the third part represents the number of the borrow pit in a running series of pits within the same district and county (e.g. 05 represents the fifth pit in that particular series).

The numerical order of pits was assigned at random. Using the example above, the pit is designated as D1-Lexington-05.

It must be noted that the compilation of borrow pits was initiated in 2008, prior to changes in the counties assigned to each SCDOT Engineering District. At that time, Aiken County was part of District 1, but now it is under the management of District 7. However, in this report, all borrow pits in Aiken County are associated with District 1. This can be seen in Figure 3.2.

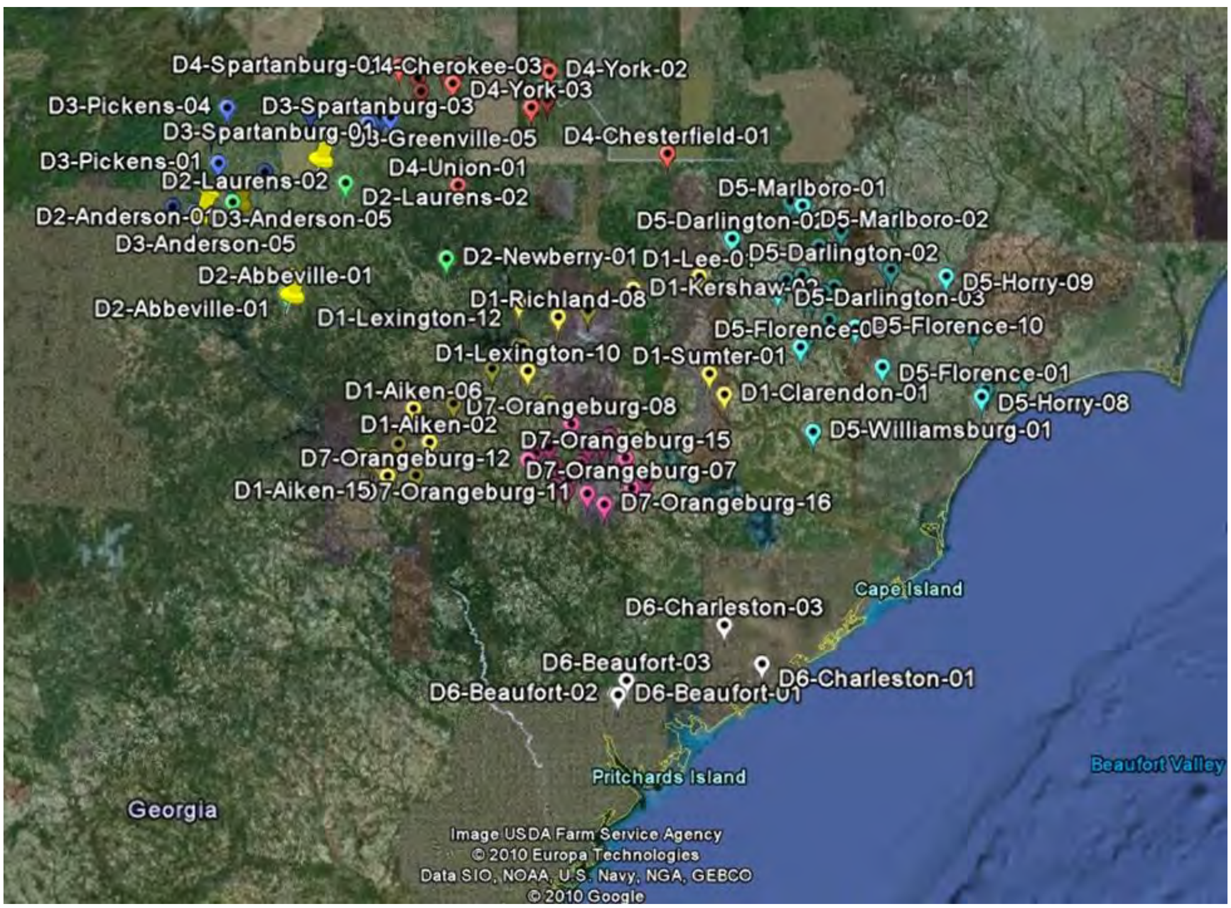

Figure 3.2. Map Showing All Identified Borrow Pits from the Data Provided by SCDOT

Of the 54 borrow pits with accurate contact information, it was determined that 17 pits had been abandoned or closed and used as sites for new construction. The other 37 pits were either active, meaning that soil excavation was current, or accessible. The accessible pits were free from construction and substantial vegetation such as trees, but in some cases there was low-growth vegetative cover present, such as bushes and weeds.

Figure 3.3 marks the locations of 17 borrow pits that were selected for sampling among the 37 active or accessible pits. Six pits were located in the smaller upstate region containing Group A soils, and eleven pits were located in the larger midlands and coastal regions containing Group B soils. SCDOT provided one additional Group B soil sample in the vicinity of the Ace Basin, and this sample location is designated as D6-Beaufort-04; however, this pit was not one of the original borrow pits identified in District 6. There were no active or accessible borrow pits in either District 5 or District 7 (not including pits in Aiken County, which were associated with District 1 at the time). The absence of sample sites in these areas, which stretches along the I-95 corridor, is evident in Figure 3.3.

The designations of borrow pits that were selected for sampling are as follows. In Group A, the following pits were sampled:

- D2-Abbeville-01
- D2-Anderson-01
- D3-Oconee-01
- D3-Greenville-05
- D3-Anderson-05
- D4-York-04

In Group B, the following pits were sampled:

- D1-Richland-08
- D1-Lexington-05
- D1-Lexington-13
- D1-Aiken-05
- D1-Aiken-06
- D1-Aiken-08
- D1-Kershaw-01
- D1-Kershaw-02
- D6-Berkeley-01
- D6-Dorchester-03
- D6-Charleston-06
- D6-Beaufort-04

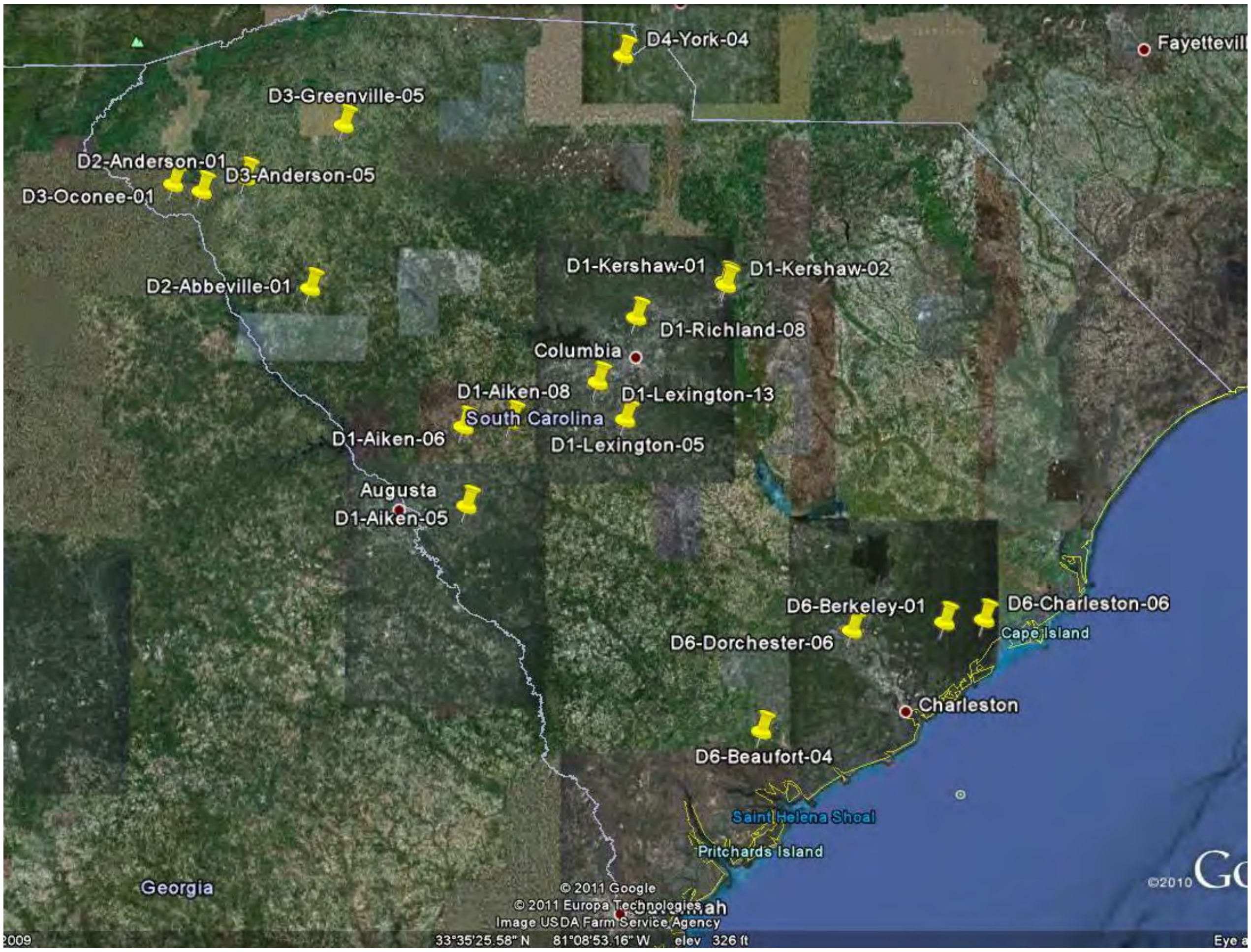

Figure 3.3. Locations of Sampled Borrow Pits

3.2 Field Sampling Procedures

Field sampling was conducted over a period of time beginning in 2008 and ending in 2009. A sampling plan was prepared in advance for each borrow pit to facilitate the acquisition of representative soil samples at each pit. During the preparation stages, soils maps were reviewed to determine if the soils were rather uniform or more variable within each pit. Based on those reviews, three sampling locations were identified for each pit. A step-by-step procedure is described in the following subsections.

3.2.1 Borrow Pit Locations via Google™ Maps

Most of the borrow pits had coordinates but not physical addresses. A street address was needed as input to obtain travel directions via GPS. Coordinates were input into Google™ maps to obtain a street address for each pit. In some cases, the pit location was confirmed at the input coordinates through a visual assessment of the satellite image. In other cases, a pit was not observed at the exact coordinates but was located in the vicinity. When possible, the street address reflected the actual location of the pit; otherwise, the nearest address was recorded. For example, the borrow pit D3-Oconee-01 was listed with coordinates of N 34° 31' 36" and W 82° 59' 30". The pit was visually located in Google™ maps and the nearest address was identified as 718 Edgewood Lane, Fair Play, South Carolina, 29643.

3.2.2 General Soil Conditions

The general soil conditions expected in the area of each borrow pit were identified using a South Carolina soils map, shown in Figure 3.4, and an AASHTO soil classification map, shown in Figure 3.5. Both maps are based on data compiled from the USDA (United States Department of Agriculture). The South Carolina soils map was used to assess geological soil provenience. For example, D3-Oconee-01 is located in the southern Piedmont and the soil formation is classified as intermediate felsic/mafic residuum. The AASHTO soil classification map provided the most common soil types. D3-Oconee-01, for example, is located approximately where there are prevalent A-6 and A-7 soils, depending on the exact location in that area. Information gathered in this step was used as a guideline rather than a definitive assessment.

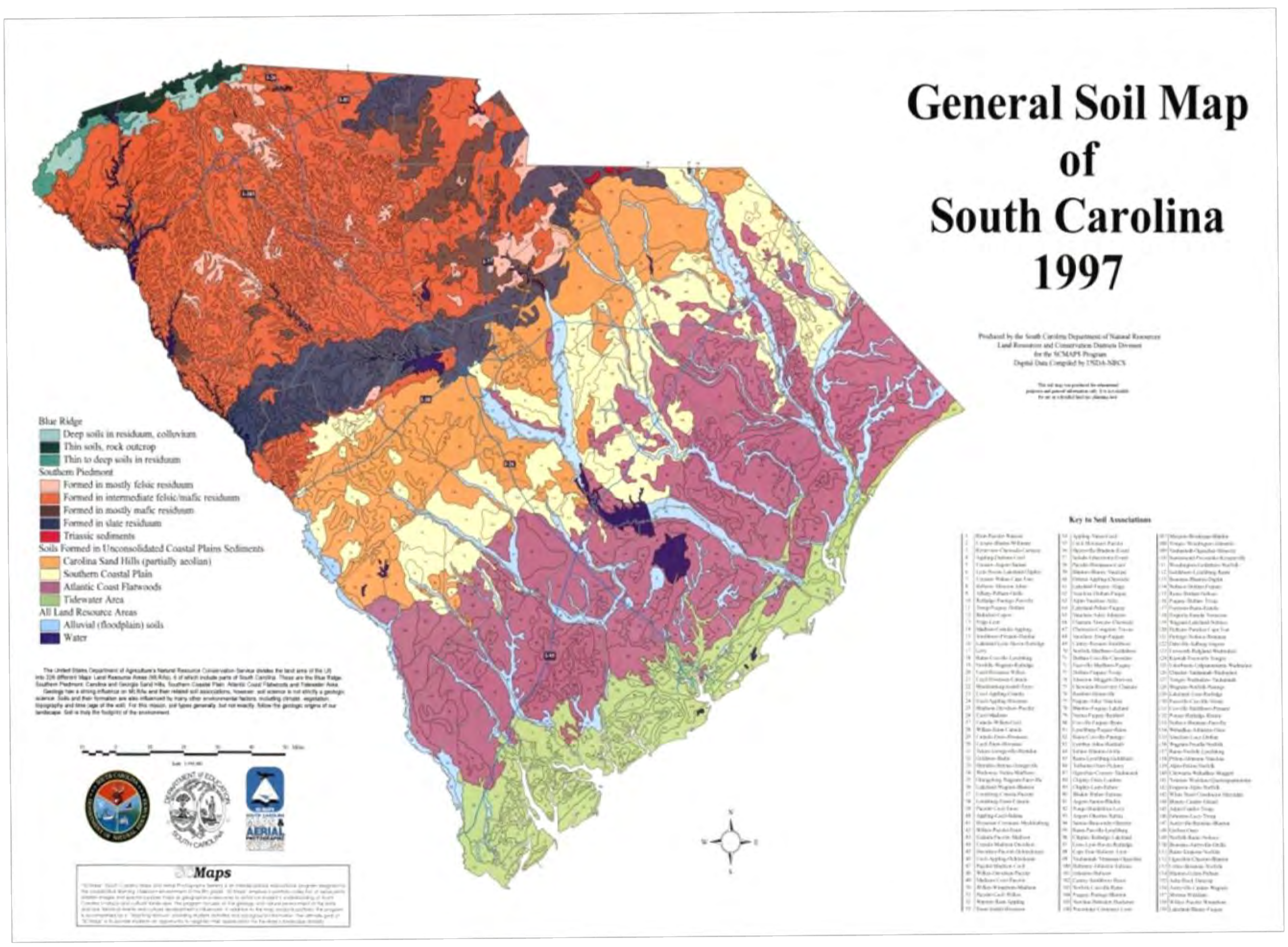

Figure 3.4. General Soil Map for South Carolina

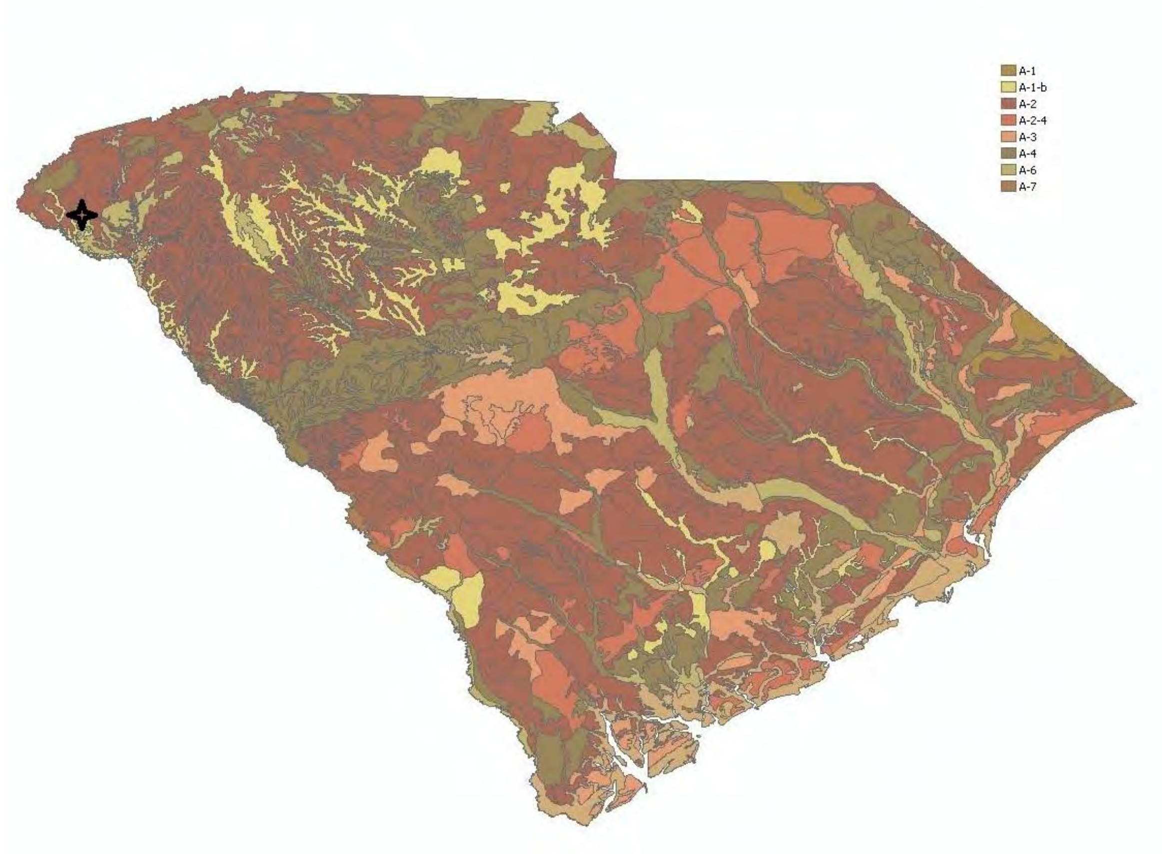

Figure 3.5. AASHTO Soil Classifications of the Soils in South Carolina

3.2.3 Borrow Pit Soil Conditions via Web Soil Survey

The USDA Web Soil Survey tool was utilized to acquire more specific soils information about each borrow pit. Web Soil Survey is a public access, online database located at websoilsurvey.nrcs.usda.gov. This tool allows for a soil survey to be performed at any location in the continental United States, and it can be manipulated to acquire information that includes soil conditions and relative engineering properties. The resolution is sufficiently high to identify the distribution of soils within the boundaries of a given borrow pit.

Figure 3.6 shows an example map produced using Web Soil Survey that delineates the soil units present in that particular borrow pit. The area of interest (AOI) is determined by the user and, in this case, the AOI is captured using the approximate outline of exposed soil in the pit. Table 3.2 summarizes the soil units present within the AOI. In borrow pit D3-Oconee-01, there are three soil units present, although two of the units account for 99% of the area. The first two letters of the soil unit represent the classification, and the third letter indicates the degree of slope at that location. The predominant soil unit is HsC2, which represents a Hiwassee (H) sandy (s) loam; the remaining soil units represent clay loam.

Figure 3.6. Web Soil Survey Map for Borrow Pit D3-Oconee-01 marked with Sampling Locations 1, 2 and 3

Table 3.2. Soil Distribution in Borrow Pit D3-Oconee-01 from Web Soil Survey

Oconee County Area, South Carolina (SC602)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
CcE3	Cecil clay loam, 15 to 25 percent slopes, severely eroded	0.1	1.4%
HsC2	Hiwassee sandy loam, 6 to 10 percent slopes, eroded	5.1	59.4%
LcD3	Lloyd clay loam, 10 to 15 percent slopes, severely eroded	3.4	39.2%
Totals for Area of Interest		**8.6**	**100.0%**

3.2.4 Selection of Individual Sampling Locations within Borrow Pits

The field sampling protocol was designed to secure three diverse, representative soil samples within each borrow pit. Sampling points were preselected on each Web Soil Survey map prior to sampling. In cases where fewer than three distinct soil units were present at the site, at least one sample location was marked within each soil unit. In cases where more than three distinct soil units were present, sample locations were marked to ensure that the three samples captured the full range of soils in that particular pit.

In the D3-Oconee-01 borrow pit, three sample locations were marked within the two major soil units (HsC2 and LcD3), as shown in Figure 3.6. Given that the third unit (CcE3) covered a small fraction of the exposed pit area, it was not selected for sampling. Furthermore, the soil in this lesser unit was classified as clay loam, which was also present in one of the other two units. The largest soil formation, Hiwassee sandy loam, was marked with two sampling points, numbered as 1 and 3. The next largest soil formation, Lloyd clay loam, was marked with one sampling point, shown as number 2.

3.2.5 On-Site Confirmation of Individual Sampling Locations

On site, the Web Soil Survey maps were used to locate the preselected sampling points. At each point, a visual assessment of soil color and texture was performed prior to sampling to confirm the presence of the expected soil type. At the D3-Oconee-01 borrow pit, no changes were made to the sampling plan, so the numbers marked in Figure 3.6 represent the location of each sample. Sometimes at other pits, the preselected location could not be accessed or readily identified. In these cases, a new location was selected and a visual assessment performed to determine whether or not the soil was within the same unit as the preselected sampling location. If the soil was deemed suitable, then a sample was acquired.

3.2.6 Bulk Sampling Practices

At each borrow pit, three bulk samples were acquired manually with the aid of a shovel, spade and pickaxe. Samples were taken from the near-surface soil of the pit itself or, in some cases, from existing stockpiles of soil. Prior to sampling, the uppermost foot of surface soil was first removed and discarded, and the field sample was acquired from the subsurface. If the sample came from a stockpile, the outer soil was removed and a field sample was extracted from the interior of the pile. Each sample was stored and sealed in a five gallon (0.67 ft^3) bucket, which was filled completely with soil. The exact coordinates of each sampling point were recorded using GPS. A field sampling data sheet was placed inside each bucket, and the sample numbers were recorded on each sheet and on the exterior of the bucket.

Chapter 4 – Experimental Program

This chapter describes the test methods used in the experimental program to determine the physical, mechanical and chemical properties of the soil obtained from each borrow pit located in Group A and B soil deposits. Tests for physical properties include visual manual identification, moisture content, specific gravity, particle size distribution, liquid limit, plastic limit, and soil classification. Tests for mechanical properties include standard Proctor compaction, direct shear, and triaxial compression. Standard Proctor compaction tests provide the maximum dry density and optimum moisture content needed for the preparation of direct shear and triaxial compression tests, which are performed to determine shear strength parameters. Tests for chemical properties include soil pH, soil resistivity, chloride content and sulfate content.

Test methods were performed according to American Association of State Highway and Transportation Officials (AASHTO) standard specifications, unless otherwise indicated. Most of the AASHTO standard specifications are comparable, or in some cases, identical to American Society for Testing and Materials (ASTM) standards. United States Environmental Protection Agency (USEPA) test methods were adopted to determine chloride and sulfate contents. Each test method is described herein.

4.1 Physical Properties

4.1.1 Visual-Manual Identification (ASTM D2488)

A visual-manual identification test was performed in accordance to ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). There is no corresponding AASHTO standard specification. Visual-manual identification determines physical soil characteristics such as color, odor, texture, and approximate particle size distribution. For sands and gravels, the predominant particle shapes are determined. For soils with substantial fines content (silts and clays), other properties such as dry strength, dilatancy, plasticity, and toughness are also estimated.

4.1.2 Moisture Content (AASHTO T 265)

The as-received moisture content of soil samples was determined using AASHTO T 265, Laboratory Determination of Moisture Content of Soils, which is comparable to ASTM D2216. This test method was also used to determine moisture content for liquid and plastic limits, standard Proctor compaction, and in the preparation of soil specimens for direct shear and triaxial compression tests.

4.1.3 Specific Gravity (AASHTO T 100)

The specific gravity of soil solids was determined in accordance with AASHTO T 100, Specific Gravity of Soils, which is comparable to ASTM D854. A water pycnometer is utilized in this test method; ASTM D5550 provides an alternative test method with a gas pycnometer, but that method was not used in this experimental program. Specific gravity of soil solids is used in calculations to support hydrometer analyses and standard Proctor compaction tests.

4.1.4 Particle Size Distribution (AASHTO T 88)

Particle size distribution was determined in accordance with AASHTO T 88, Particle Size Analysis of Soils, which is comparable to ASTM D422. The distribution of particle sizes larger than 0.075 mm (No. 200 sieve) is determined by means of mechanical separation in a sieve analysis. The distribution of particle sizes smaller than 0.075 mm (No. 200 sieve) is determined by means of sedimentation in a hydrometer analysis. The coarse (retained on No. 200 sieve) and fine (passing No. 200 sieve) fractions are used in soil classification. The distribution results of both analyses are coupled to produce a particle size distribution curve. Equivalent particle size diameters D_{60}, D_{30}, and D_{10} are determined from each curve and used to calculate the coefficient of uniformity, C_u, and coefficient of curvature, C_c.

4.1.5 Liquid Limit (AASHTO T 89)

The liquid limit, w_{LL} or LL, was determined in accordance with AASHTO T 89, Determining the Liquid Limit of Soils, which is comparable to ASTM D4318. The liquid limit test defines the moisture content of soil at its transition to a liquid state, and it is used in soil classification. The test method is performed on the soil fraction finer than 0.425 mm (No. 40 sieve), and the results are influenced by the amount and mineralogical composition of the soil fraction finer than 0.075 mm (No. 200 sieve).

4.1.6 Plastic Limit (AASHTO T 90)

The plastic limit, w_{PL} or PL, was determined in accordance with AASHTO T 90, Determining the Plastic Limit and Plasticity Index of Soils, which is comparable to ASTM D4318. The plastic limit test defines the moisture content of soil at its transition to a plastic state, and it is used to determine the plasticity index, PI, for soil classification. The test method is performed on the same soil sample that is prepared for the liquid limit test.

4.1.7 Soil Classification (AASHTO M 145 and ASTM D2487)

Soils are classified using AASHTO M 145, Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes, and ASTM D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). The two systems provide distinct classifications, although the AASHTO and USCS classifications for each soil can often be correlated.

4.2 Mechanical Properties

4.2.1 Standard Proctor Compaction (AASHTO T 99)

Standard Proctor compaction tests were performed in accordance with AASHTO T 99, Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop, which is comparable to ASTM D698. The test method utilizes a compaction effort of 12,400 ft-lb/ft (600 kN-m/m) per test, and multiple tests are performed at different moisture contents to generate a compaction curve. The maximum dry density, $\gamma_{d,max}$, and optimum moisture content, w_{opt}, are identified from the peak of the compaction curve. Compaction tests provide information on the physical phase relationships in soil, and the phase relations influence the mechanical behavior. Thus, the mechanical properties of a compacted soil can be correlated with its dry density and moisture content, and depend on whether a soil is compacted at a moisture content that is dry of optimum ($w < w_{opt}$) or wet of optimum ($w > w_{opt}$).

4.2.2 Direct Shear (AASHTO T 236)

Direct shear tests were performed in accordance with AASHTO T 236, Direct Shear Test of Soils under Consolidated Drained Conditions, which is equivalent to ASTM D3080. Tests were performed using a strain controlled, direct shear apparatus manufactured by Wykeham Farrance. The dimensions of the shear box are 2.5 in. × 2.5 in. (6.4 cm x 6.4 cm) square by 1.2 in. (3.1 cm) high. Specimens were prepared in the shear box to a target density of 95% maximum standard Proctor density at the optimum moisture content. The soil was placed in three equal lifts. Each compacted surface was scarified before the next lift of soil was placed.

Three tests were performed on each sample at normal stresses equal to 7 psi (48.3 kPa), 14 psi (96.5 kPa), and 21 psi (144.8 kPa). These stresses were selected to simulate the range of stresses present in a typical highway embankment (i.e. depth range of 8.2 ft (2.5 m) to 24.3 ft (7.4 m)). During shear, the load ring deformation, horizontal displacement and vertical displacement were measured. The shear rate ranged from 0.0001 to 1.2 mm/min.

For each test, the relationship between the shear stress and horizontal displacement and the relationship between horizontal displacement and vertical displacement are plotted to determine the shear stress and normal stress at failure (defined as peak stress). Then, the shear stress and normal stress at failure are plotted for each of the three tests to determine the slope (effective friction angle, ϕ') and intercept (effective cohesion, c') from the best linear fit of the data.

4.2.3 Triaxial Compression (AASHTO T 297)

Triaxial compression tests were performed according to AASHTO T 297, Consolidated Undrained Triaxial Compression Test on Cohesive Soils, which is equivalent to ASTM D4767. The GDS Electromechanical Dynamic Triaxial Testing System (DYNTTS) used for these tests is shown in Figure 4.1 and was recently acquired by the Department of Civil and Environmental

Engineering in November 2010. It is a software-driven system that includes a triaxial cell capable of providing a confining pressure up to 1 MPa, a digital processor for controlling cell pressure, and a digital processor for controlling back pressure. The digital processors maintain pressures to within 1 kPa. The unit is equipped with an analog pore pressure transducer and an analog load cell transducer. The displacement is digitally-controlled through an encoder in a stepper motor. The system has a maximum axial load capacity of 10 kN.

Specimens for triaxial testing were prepared from the soils obtained from the borrow pits across the state. Specimens were compacted to a target of 95% maximum standard Proctor density at optimum moisture content. The specimens were 2.0 in. (50.1 mm) in diameter and 4.0 in. (101.6 mm) in height. The corresponding height-to-diameter ratio of the specimens was H/D = 2. Two specimen preparation methods were used. Soils with some cohesion (i.e. CL, ML) were compacted in a standard Proctor mold per Section 4.2.1 at optimum moisture content, extracted from the mold, and trimmed to size. Cohesionless soils (i.e. SP) were prepared inside a 2.0 in. (50.1 mm) diameter split mold. The soil was placed in thin lifts and tamped into place to achieve the target density. Each compacted surface was scarified before the next lift of soil was placed.

Following sample preparation, samples were saturated in the triaxial cell using a two step process: 1) primary saturation in which specimens were flushed with de-aired water and 2) back pressure saturation by applying a back pressure on the specimen to drive water into the specimen and force the entrapped air into aqueous solution. The degree of saturation was found by a B-value check using a cell pressure increment of 25 kPa. Once a satisfactory B-value was achieved, the specimens were isotropically consolidated to 48, 96, or 144 kPa (1, 2, or 4 ksf). After consolidation, specimens were sheared undrained in triaxial compression under strain controlled conditions. The rate of axial strain was selected based on AASHTO T 297 (ASTM D4767) and is a function of specimen permeability. Pore pressures were measured during shearing.

For each test, the relationships between the principal stress difference and axial strain and the excess pore pressure and axial strain are plotted. These data are also used to plot the stress path in q-p space for each test. The effective friction angle, ϕ', and the effective cohesion, c', are determined from the slope, ψ, and intercept, a, of the linear portion of the stress path using the following relationships: $c'=a/\cos\phi'$ and $\tan\psi=\sin\phi'$.

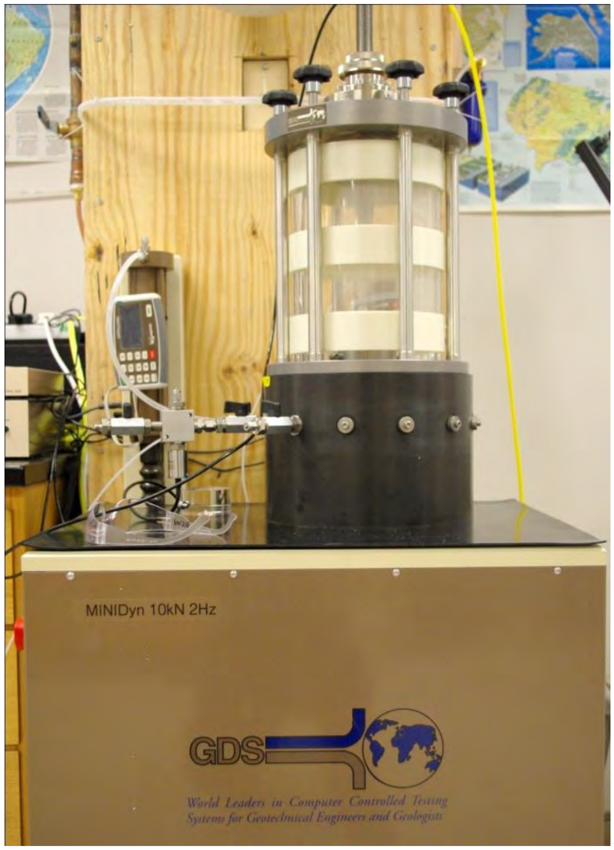

Figure 4.1. GDS Electromechanical Dynamic Triaxial Testing System (DYNTTS) in the Department of Civil and Environmental Engineering Advanced Geotechnical Laboratory.

4.3 Chemical Properties

4.3.1 pH (AASHTO T 289)

pH of the soil samples was determined in accordance with AASHTO T 289, Determining pH of Soil for Use in Corrosion Testing. ASTM D4972 provides another test method that covers the measurement of pH for uses other than corrosion testing. The test measures the concentration of H^+ ions in a soil sample to determine its degree of acidity or alkalinity.

4.3.2 Resistivity (AASHTO T 288)

The minimum electrical soil resistivity was determined in accordance with AASHTO T 288, Determining Minimum Laboratory Soil Resistivity, which is similar to ASTM G187. These test methods employ a two-electrode soil box to accommodate lab bench-scale tests. Soil resistivity is a function of moisture content, and multiple tests are performed at different moisture contents to determine the minimum resistivity. The corrosion potential of a soil can be correlated with its minimum electrical resistivity, which corresponds to its maximum electrical conductivity.

4.3.3 Chloride Content (USEPA Method 8225)

The chloride content of soil samples was determined using an adaptation of USEPA Method 8225, Silver Nitrate Buret Titration Method for Chloride. This method was compared to AASHTO T 291, Determining Water-Soluble Chloride Ion Content in Soil, and it was deemed to be similar to AASHTO Method A. The differences in test procedures are confined to sample preparation and chemical requirements. The USEPA method was found to be acceptable for use in this investigation, and this change was approved by the SCDOT.

In this test method, silver nitrate is titrated into the soil sample until a specific color change is noted. The amount of silver nitrate required to change color is correlated to the chloride content. Results are reported as chloride content in mg/L.

4.3.4 Sulfate Content (USEPA Method 8051)

Like chloride content, a USEPA method was also approved and adopted for determining sulfate content of soil samples. USEPA Method 8051, SulfaVer 4 Method for Sulfate, was found to be similar to Method B of AASHTO T 290, Determining Water-Soluble Sulfate Ion Content in Soil, which involves forming a precipitate of barium sulfate. The main differences are with sample preparation, the wavelength required to measure turbidity and the provenience of chemical compounds to provoke the reaction. Results are reported as sulfate content in parts per million (ppm).

Chapter 5 – Results: Soil Classification, Index Properties and Chemical Properties

This chapter presents the results of the laboratory testing program performed to determine the properties of the soils obtained from each borrow pit located in Group A and B soil deposits. Results of tests performed to determine the index properties (specific gravity, particle size distribution, liquid limit, plastic limit, maximum dry density and optimum moisture content) are presented first and used to classify the soils according to AASHTO and USCS. These results are followed by the environmental properties determined from the chemical tests (soil pH, soil resistivity, chloride content and sulfate content). Results of the strength tests will be presented in Chapter 6.

5.1 Soil Classification and Index Properties

5.1.1 Specific Gravity

Figures 5.1 and 5.2 illustrate the specific gravity of soil solids, G_s, measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. In a given soil sample, G_s depends on the mineralogical composition and organic content. Most inorganic soils contain a mixture of minerals such that the composite G_s commonly ranges from 2.50 to 2.80. Table 5.1 lists common soils minerals and their values of specific gravity. Quartz and feldspars are common minerals in gravels, sands, and non-plastic silts. Kaolinite, illite, and montmorillonite are the most common clay minerals. Table 5.1 shows that the G_s for clay minerals can be lower and more variable than non-clay minerals, such that G_s for soils containing significant amounts of clay minerals can depend on the distribution of mineral types.

Table 5.1. Values of Specific Gravity for Common Soil Minerals

Mineral	Specific Gravity, G_s
Quartz	2.65
Orthoclase Feldspar	2.57
Plagioclase Feldspar	2.62 - 2.76
Muscovite Mica	2.76 - 3.10
Biotite Mica	2.80 - 3.20
Kaolinite	2.16 - 2.68
Illite	2.65
Montmorillonite	1.70 - 2.00
Chlorite	2.60 - 3.30
Halloysite	2.53

Values of G_s for soils from borrow pits in South Carolina fall well within the common range of 2.50 to 2.80. No sample falls below a G_s of 2.50; the lowest measured G_s is 2.53 from D6-Charleston-06. There are four samples that exceed 2.80. One of the samples is from D2-Abbeville-01, where a G_s of 2.83 from soil in Bucket Sample 2 is noticeably higher than G_s of 2.64 and 2.67 measured from the other two bucket samples. The higher value of G_s suggests that the mineralogical composition of soil at the location corresponding to Bucket Sample 2 is sufficiently different from soil at the other two sampling locations. All three soil samples taken from D1-Richland-08 contained solids with G_s ranging from 2.83 to 2.89. As expected, some of the borrow pits, like D1-Richland-08, had less variation in G_s than other pits, like D2-Abbeville-01. Borrow pits with soil samples that varied more than a tenth ($G_s \pm 0.1$) include D3-Oconee-01, D6-Charleston-06, and D6-Berkeley-01. It should be noted that these observations are based on a limited number of three samples acquired at each pit.

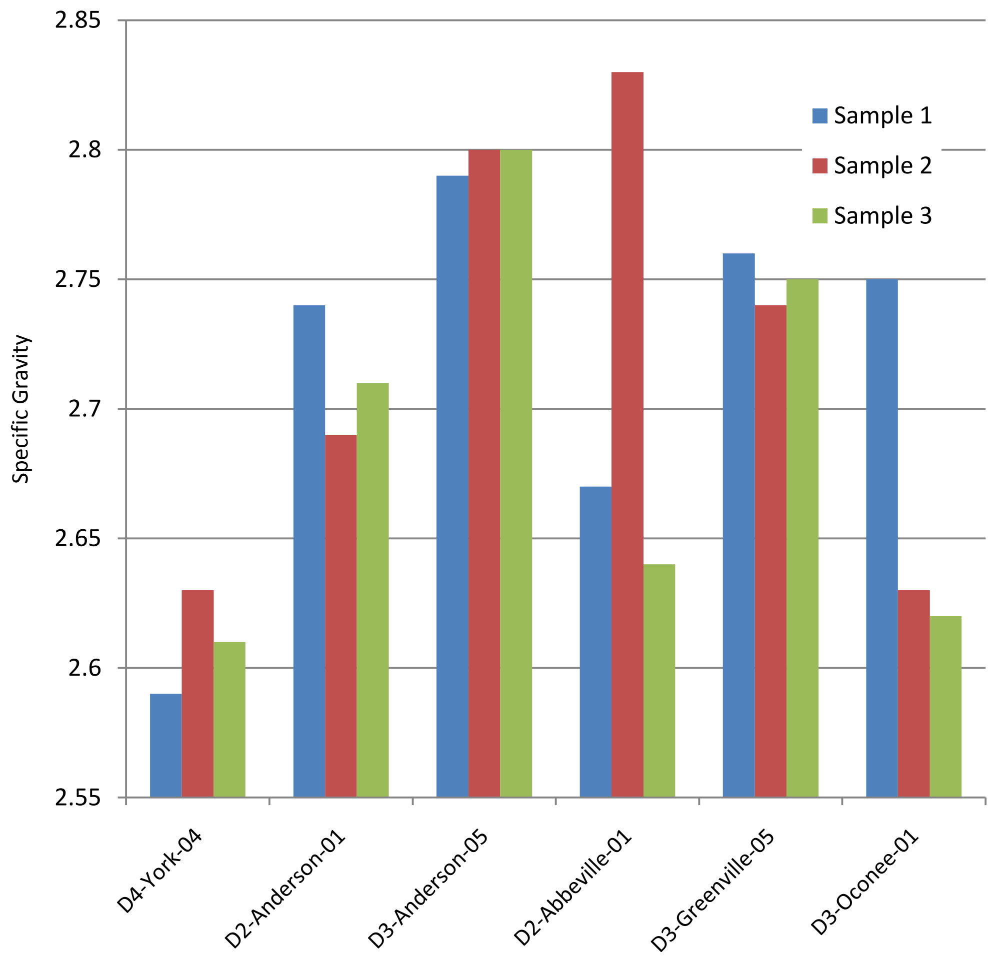

Figure 5.1. Specific Gravity, G_s, of Group A Soils

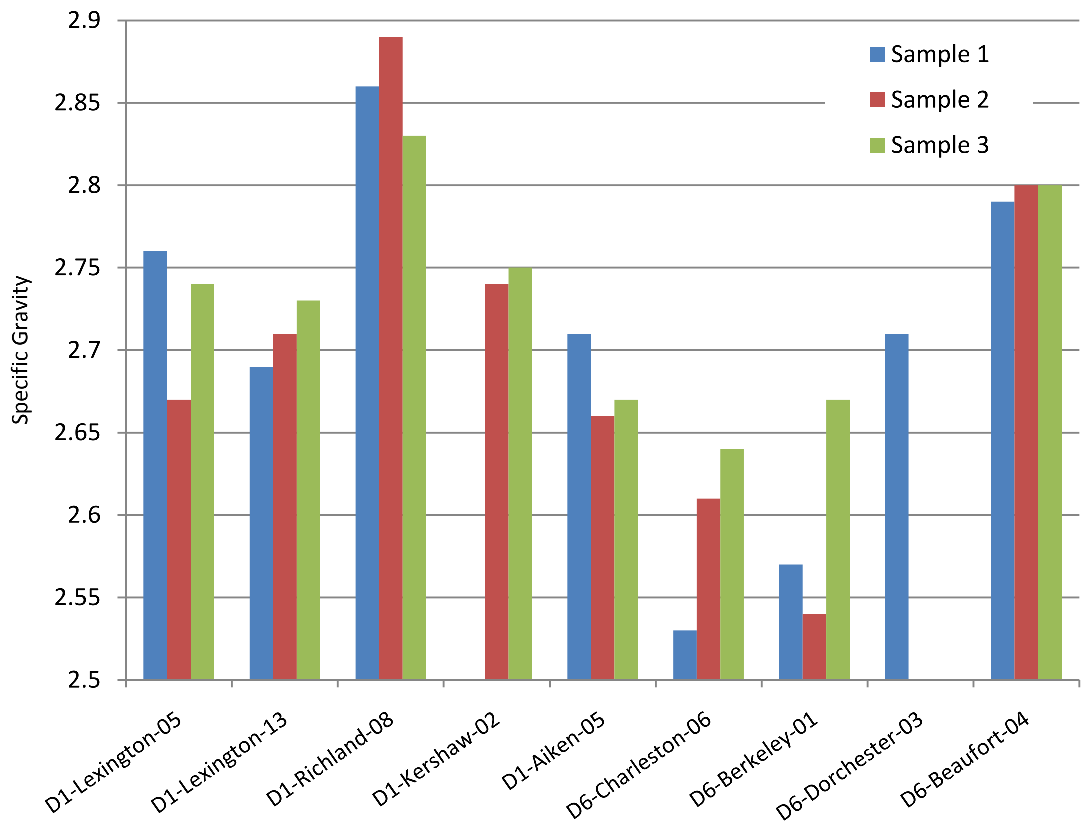

Figure 5.2. Specific Gravity, G_s, of Group B Soils

5.1.2 Particle Size Distribution

Table 5.2 summarizes the USCS and AASHTO soil classifications for each bucket sample from borrow pits located in Group A. Soil classifications are based on the results of sieve and hydrometer analyses to determine particle size distribution and liquid and plastic limit tests to determine plasticity. The fines content, liquid limit (LL), and plasticity index (PI) are provided for each soil sample.

A review of the soil classifications shows that silty sands, SM, are prevalent and account for 10 of the 18 soil samples. Each borrow pit has at least one SM soil sample, except for D3-Oconee-01. The next most common soils are high plasticity silts, MH, which are also present in all but one borrow pit. The pit without SM soils, D3-Oconee-01, is also the singular pit where clay (CH) soils were classified. It should be noted that all soils contain a mixture of coarse and fine grains, as the fines content illustrates in Table 5.2. The fines content ranges from 21% to 68%; furthermore, 13 of the 18 soil samples have fines content of 50% ± 10%. This suggests that some, but not all, of the SM and MH soils should have comparable mechanical properties. In fact, the AASHTO soil classifications reflect some of the similarities between SM, MH, and CH soils. There are 11 A-7-5 and A-7-6 soils classified in five of the six borrow pits. D4-York-04 is the lone exception, where the fines content tends to be lower than in the other pits such that A-2-4 and A-4 soils are present.

Table 5.2 Soil Classification for Borrow Pits in Group A

Borrow Pit	No.	Soil Classification		Fines Content (% < 75 μm)	LL[1]	PI[1]
		USCS	AASHTO			
D4-York-04	1	SM	A-2-4	21	NP	NP
	2	SM	A-4	40	NP	NP
	3	SM	A-4	37	NP	NP
D2-Anderson-01	1	MH	A-7-5	53	54	18
	2	SM	A-7-5	46	55	24
	3	ML	A-7-6	50	42	13
D3-Anderson-05	1	MH	A-7-5	53	55	13
	2	SM	A-7-6	46	41	16
	3	SM	A-5	44	48	10
D2-Abbeville-01	1	SM	A-7-6	36	45	17
	2	MH	A-7-5	68	58	21
	3	SM	A-2-4	35	38	1
D3-Greenville-05	1	SM	A-5	48	44	5
	2	MH	A-7-5	60	53	22
	3	SM	A-4	41	NP	NP
D3-Oconee-01	1	CH	A-7-6	58	52	27
	2	MH	A-7-5	53	72	23
	3	CH	A-7-6	57	65	42

[1]NP means non-plastic.

Table 5.3 summarizes the USCS and AASHTO soil classifications for each bucket sample from borrow pits located in Group B. In general, the soils in Group B have lower fines content than those in Group A, and the soil classifications differ accordingly. SP-SM and SW-SM soils are found in 11 of the 21 samples. By definition, these soils must contain between 5% and 12% fines. There are six SM and SC soils, but the fines content does not exceed 32%; whereas, all but one of the SM soils in Group A has at least 35% fines. The difference in fines content is also reflected in the distribution of AASHTO soil classifications. In Group B, the soils range from A-

1 to A-4 and there are no soils with A-5 or higher classifications. In Group A, 13 of the 18 soil samples are classified as A-5 or higher.

Table 5.3 Soil Classification for Borrow Pits in Group B

Borrow Pit	No.	Soil Classification		Fines Content (% < 75 µm)	LL[1]	PI[1]
		USCS	AASHTO			
D1-Lexington-05	1	SC	A-2-6	18	31	12
	2	SM	A-2-4	21	NP	NP
	3	SC	A-2-7	28	49	28
D1-Lexington-13	1	SW-SM	A-1-b	6	NP	NP
	2	SW-SM	A-1-b	7	NP	NP
	3	SW-SM	A-1-b	8	NP	NP
D1-Richland-08	1	ML	A-4	75	31	7
	2	ML	A-4	88	35	1
	3	CL-ML	A-4	52	17	4
D1-Kershaw-02	2	SW-SM	A-2-4	12	NP	NP
	3	SM	A-2-4	29	NP	NP
D1-Aiken-05	1	SP	A-3	3	NP	NP
	2	SP-SM	A-3	6	NP	NP
	3	SP-SM	A-2-4	12	NP	NP
D6-Charleston-06	1	SM	A-2-4	20	NP	NP
	2	SP-SM	A-3	7	NP	NP
	3	SP-SM	A-3	8	NP	NP
D6-Berkeley-01	1	SP-SM	A-2-4	11	NP	NP
	2	SM	A-2-4	32	NP	NP
	3	SP-SM	A-3	10	NP	NP
D6-Dorchester-03	1	SP-SM	A-2-4	12	NP	NP

[1]NP means non-plastic.

5.1.3 Atterberg Limits

Figures 5.3 and 5.4 illustrate the plasticity index, PI, measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. Given the geographical and geological distinctions of each soil group, it is expected that Group A soils would have some measurable plasticity and that Group B soils would not, except for soils within the Fall Zone close to the division of Group A and B soils. Figure 5.3 shows that most soil samples have a PI between 10 and 25, with a few exceptions. Soil samples collected from D4-York-04 were determined to be non-plastic. The highest PI of 42 was measured from soil in D3-Oconee-01. It is also observed that some borrow pits with more variable PI measurements, such as D3-Oconee-01 and D2-Abbeville-01, also had more variable G_s measurements, as discussed in the prior section. These correlations suggest that the differences in specific gravity and plasticity are a function of changes in the amount and/or type of clay minerals present in the soil samples. For Group B soils, Figure 5.4 shows that, except for two borrow pits, the fines content is limited and/or was determined to be non-plastic. The two borrow pits containing soils with some measureable plasticity are located in Lexington and Richland counties within the Fall Zone.

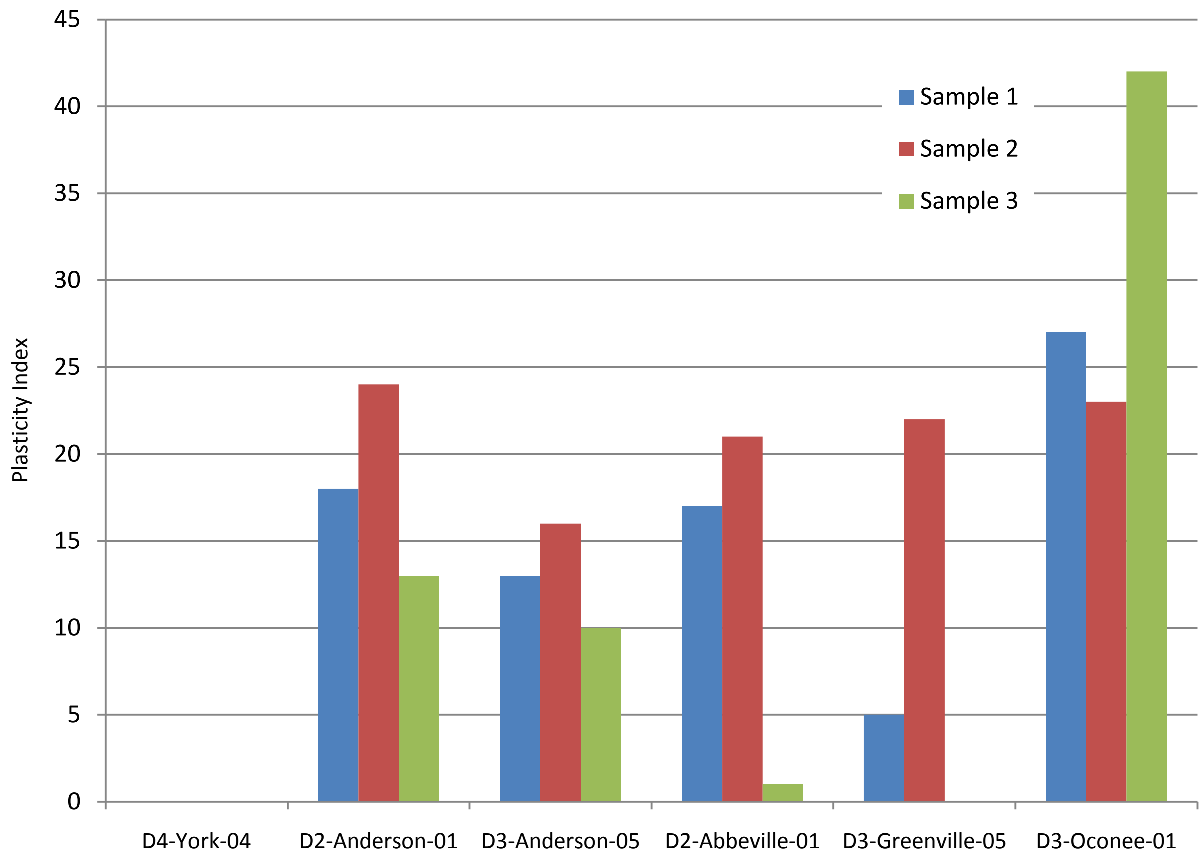

Figure 5.3. Plasticity Index, PI, of Group A Soils

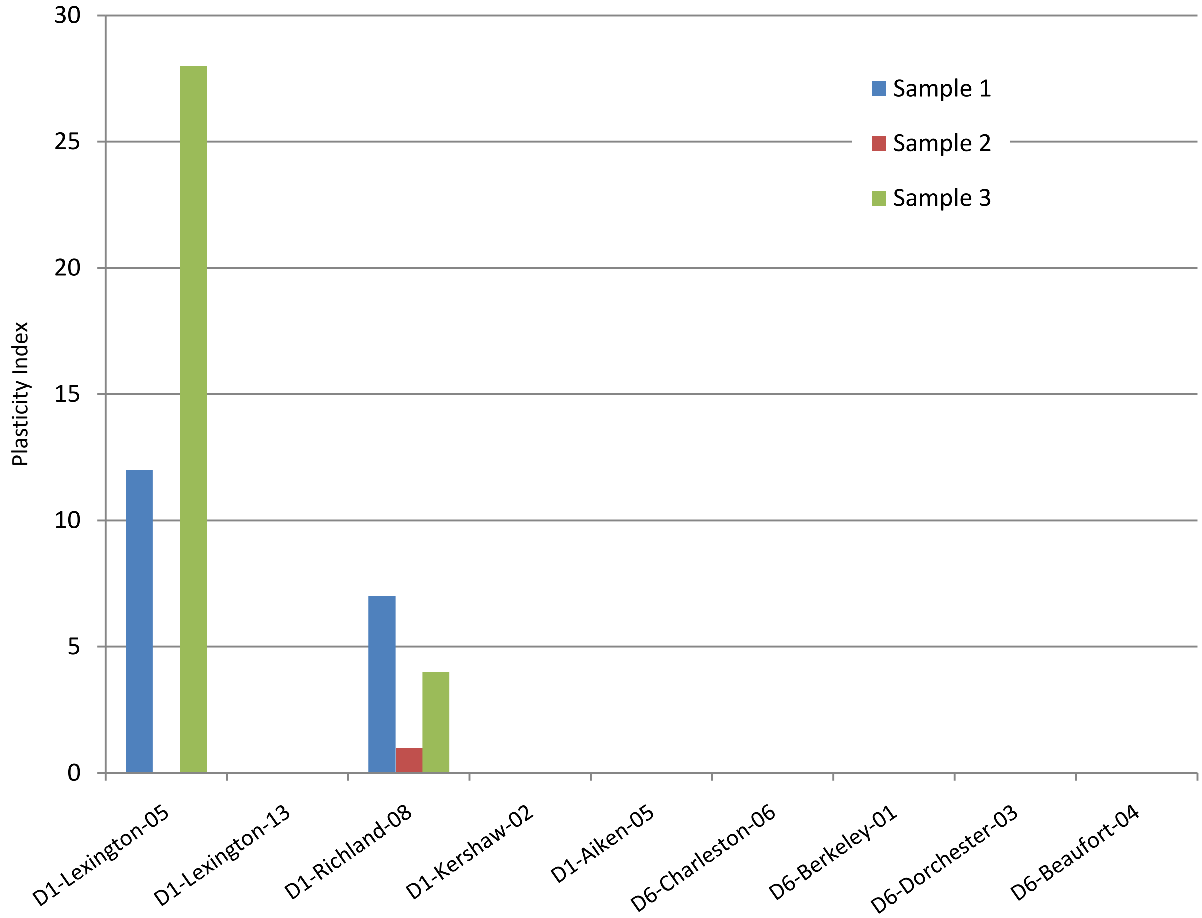

Figure 5.4. Plasticity Index, PI, of Group B Soils

5.1.4 Compaction Characteristics

Figures 5.5 and 5.6 illustrate the maximum dry density, $\gamma_{d,max}$, measured from standard Proctor compaction tests performed on each bucket sample from each borrow pit located in Group A and Group B soil deposits, respectively. On average, soils in Group B can be compacted to a higher dry density than soils in Group A. In Group A, $\gamma_{d,max}$ ranges from 91 to 119.5 pcf, and a minimum of 100 pcf is achieved in 12 of the 18 soil samples. Samples from D4-York-04 represent the only three soils that exceeded 110 pcf. Most soils with lower $\gamma_{d,max}$ were observed to contain mica particles, which have been shown to reduce the maximum dry density of compacted soils. In Group B, all of the soil samples achieved a minimum $\gamma_{d,max}$ of 100 pcf, and 11 of the soil samples exceeded 110 pcf.

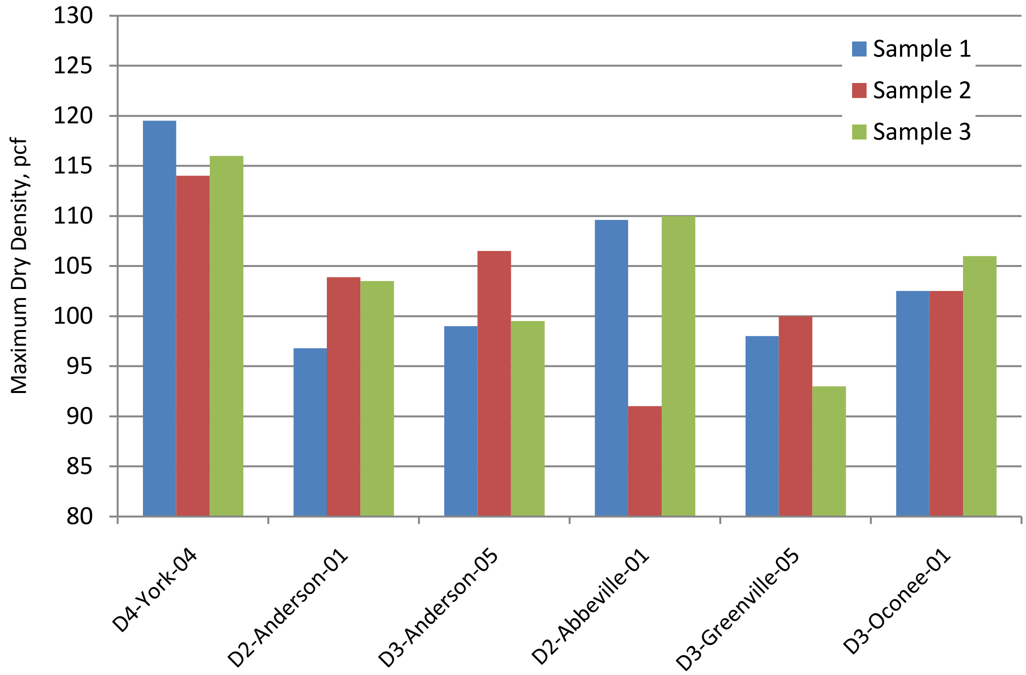

Figure 5.5. Maximum Dry Density of Group A Soils

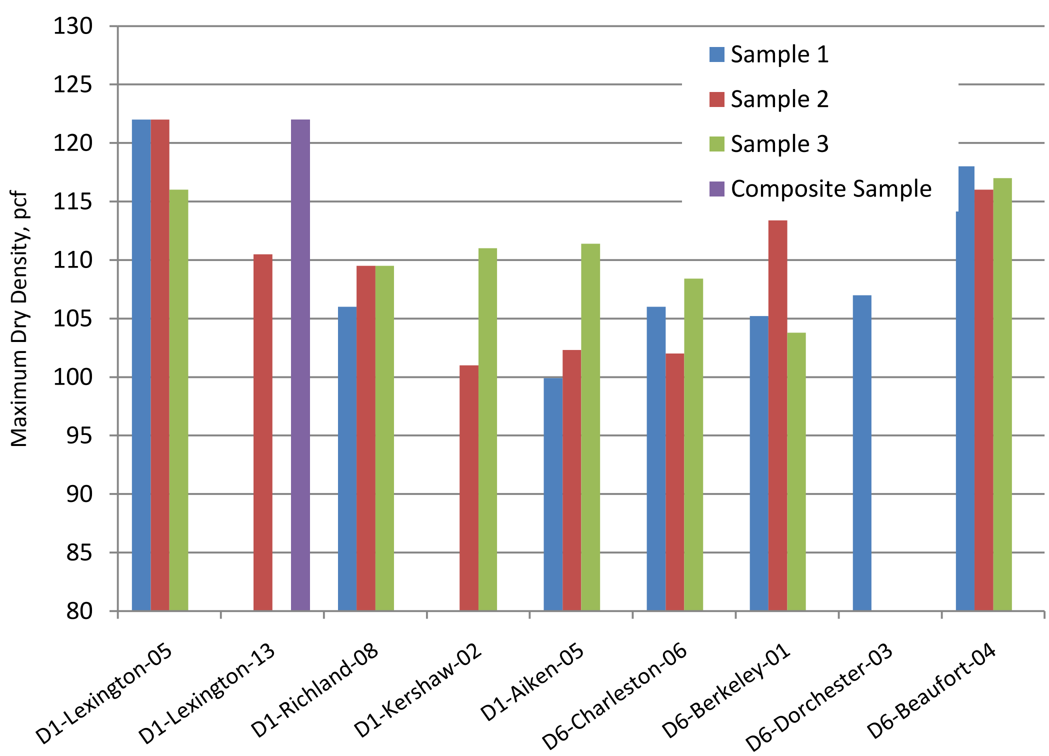

Figure 5.6. Maximum Dry Density of Group B Soils

Tables 5.4 and 5.5 compare the compaction characteristics within each AASHTO soil classification present in Group A and Group B soil deposits, respectively.

In Group A, the A-2-4 and A-4 soils have the highest $\gamma_{d,max}$ and lowest w_{opt} required for compaction. There is one exception in D3-Greenville-05, where mica was observed in the soil samples. The A-7-6 soils have the next highest $\gamma_{d,max}$, with values ranging between 102 and 110 pcf. The A-5 and A-7-5 soils have the lowest $\gamma_{d,max}$ and require the highest w_{opt} for compaction. Mica was observed to be present in some of these soil samples.

In Group B, the A-1 and A-2 soil groups tend to produce a higher $\gamma_{d,max}$ at lower w_{opt} than the A-3 and A-4 soil groups. All of the Group B soil samples with $\gamma_{d,max}$ of at least 110 pcf are in the A-1 and A-2 soil groups. There are also noticeable differences in the w_{opt} required for compaction. All of the A-1 and A-2 soils, except one, have $w_{opt} \leq 14\%$. Conversely, all of the A-3 and A-4 soils, except one, have $w_{opt} \geq 14\%$. It should be noted that all of the Group B soils have $w_{opt} < 20\%$; whereas, 11 of the 18 Group A soils have $w_{opt} \geq 20\%$.

Table 5.4 Compaction Characteristics of Group A Soils

AASHTO Soil Classification	Borrow Pit and Bucket Sample Nos.	G_s	Standard Proctor Compaction	
			$\gamma_{d\ max}$ (pcf)	w_{opt} (%)
A-2-4	D4-York-04-B1	2.59	119.5	11
	D2-Abbeville-01-B3	2.64	110	14
A-4	D4-York-04-B2	2.63	114	15.4
	D4-York-04-B3	2.61	116	13.5
	D3-Greenville-05-B3	2.75	93	20
A-5	D3-Anderson-05-B3	2.80	99.5	23
	D3-Greenville-05-B1	2.76	98	23
A-7-5	D2-Anderson-01-B1	2.74	96.8	24.1
	D2-Anderson-01-B2	2.69	103.9	22
	D3-Anderson-05-B1	2.79	99	23
	D2-Abbeville-01-B2	2.83	91	28
	D3-Greenville-05-B2	2.74	100	23
	D3-Oconee-01-B2	2.63	102.5	22.5
A-7-6	D2-Anderson-01-B3	2.71	103.5	19
	D3-Anderson-05-B2	2.80	106.5	20
	D2-Abbeville-01-B1	2.67	109.6	15
	D3-Oconee-01-B1	2.75	102.5	23.5
	D3-Oconee-01-B3	2.62	106	19

Table 5.5 Compaction Characteristics of Group B Soils

AASHTO Soil Classification	Borrow Pit and Bucket Sample Nos.	G_s	Standard Proctor Compaction	
			$\gamma_{d\,max}$ (pcf)	w_{opt} (%)
A-1-b	D1-Lexington-13-B1	2.69	122	10.5
	D1-Lexington-13-B2	2.71	122	10.5
	D1-Lexington-13-B3	2.73	122	10.5
A-2-4	D1-Lexington-05-B2	2.67	122	11
	D1-Kershaw-02-B2	2.74	101	14
	D1-Kershaw-02-B3	2.75	111	14
	D1-Aiken-05-B3	2.67	111.4	12
	D6-Charleston-06-B1	2.53	106	16.5
	D6-Berkeley-01-B1	2.57	105.2	12.5
	D6-Berkeley-01-B2	2.54	113.4	12.3
	D6-Dorchester-03-B1	2.71	107	12
A-2-6	D1-Lexington-05-B1	2.76	122	13
A-2-7	D1-Lexington-05-B3	2.74	116	12.5
A-3	D1-Aiken-05-B1	2.71	99.9	18.2
	D1-Aiken-05-B2	2.66	102.3	16
	D6-Charleston-06-B2	2.61	102	16
	D6-Charleston-06-B3	2.64	108.4	12.6
	D6-Berkeley-01-B3	2.67	103.8	17.5
A-4	D1-Richland-08-B1	2.86	106	17
	D1-Richland-08-B2	2.89	109.5	14.7
	D1-Richland-08-B3	2.83	109.5	14.7

5.2 Chemical Properties

5.2.1 Soil pH

Figures 5.7 and 5.8 illustrate the soil pH measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. Table 5.6 provides a summary of pH values for all measurements.

Soil pH is a function of soil deposition and geographic location, which in turns impacts the presence and thickness of soil horizons within soil deposits. In general, soil pH can range from 2.8 to 10.0. However, most soils in the southeastern U.S. tend to be acidic. With the exception of four samples, soils from borrow pits across South Carolina range from very strongly acid to slightly acid, as shown in Table 5.6. The four samples with pH of neutral to moderately alkaline are from borrow pits in Kershaw (Group B) and York (Group A) counties. Even though these pits represent two different soil groups, the pits are relatively close to each other and are just separated by Lancaster County. None of the soil samples have pH on the most extreme ends of the acid-alkaline scale.

pH of soils within a borrow pit can be quite consistent, as shown in Figure 5.7 for D3-Greenville-05 and D3-Oconee-01. However in most cases shown in Figures 5.7 and 5.8, the measured pH values are variable, which is expected given the natural soil variation expected at a given site. For each borrow pit sampled, the difference in maximum and minimum pH was ≤ 1 except for D4-York-04 (2.1 difference), D2-Anderson-01 (1.1 difference), and D6-Berkeley-01 (1.3 difference).

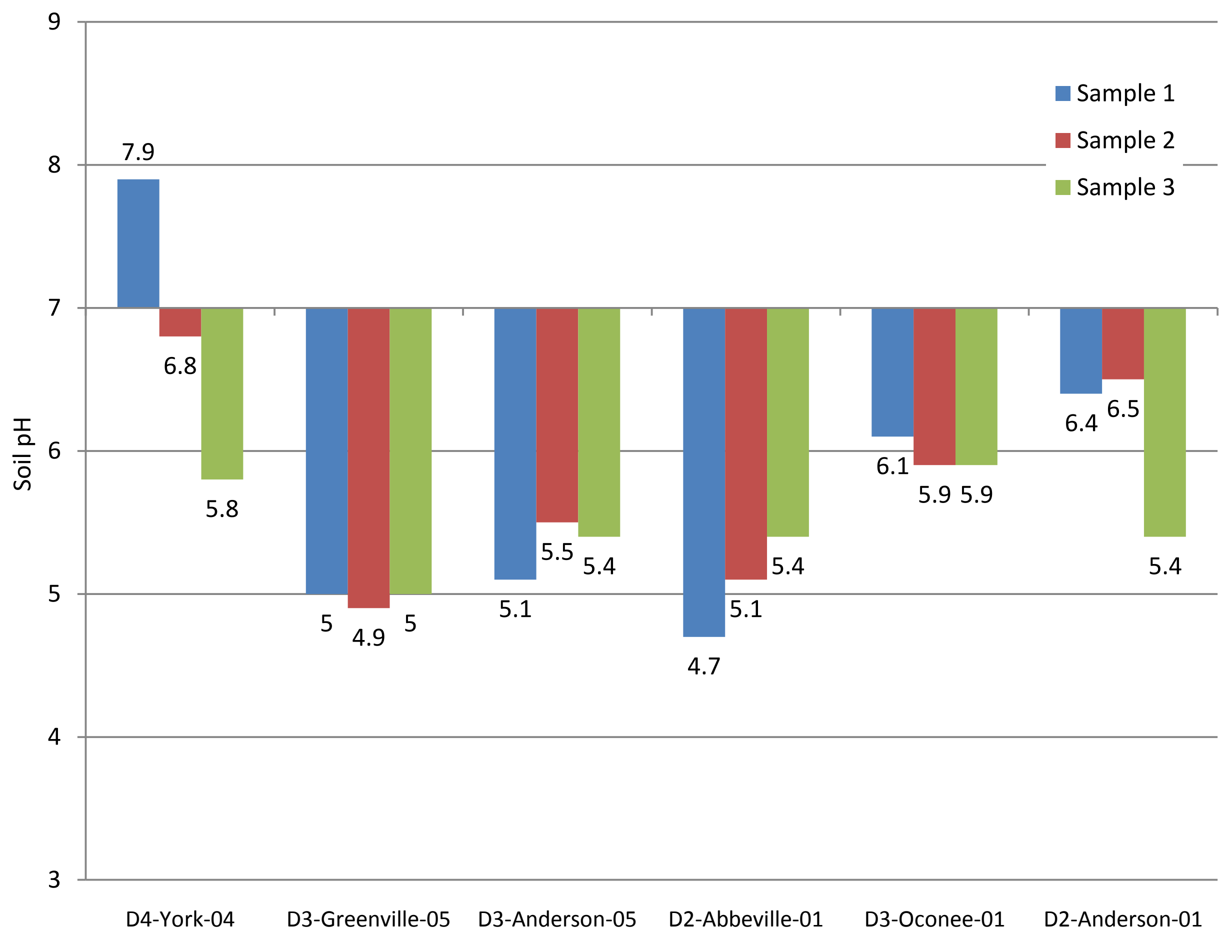

Figure 5.7. Soil pH for Group A Soils

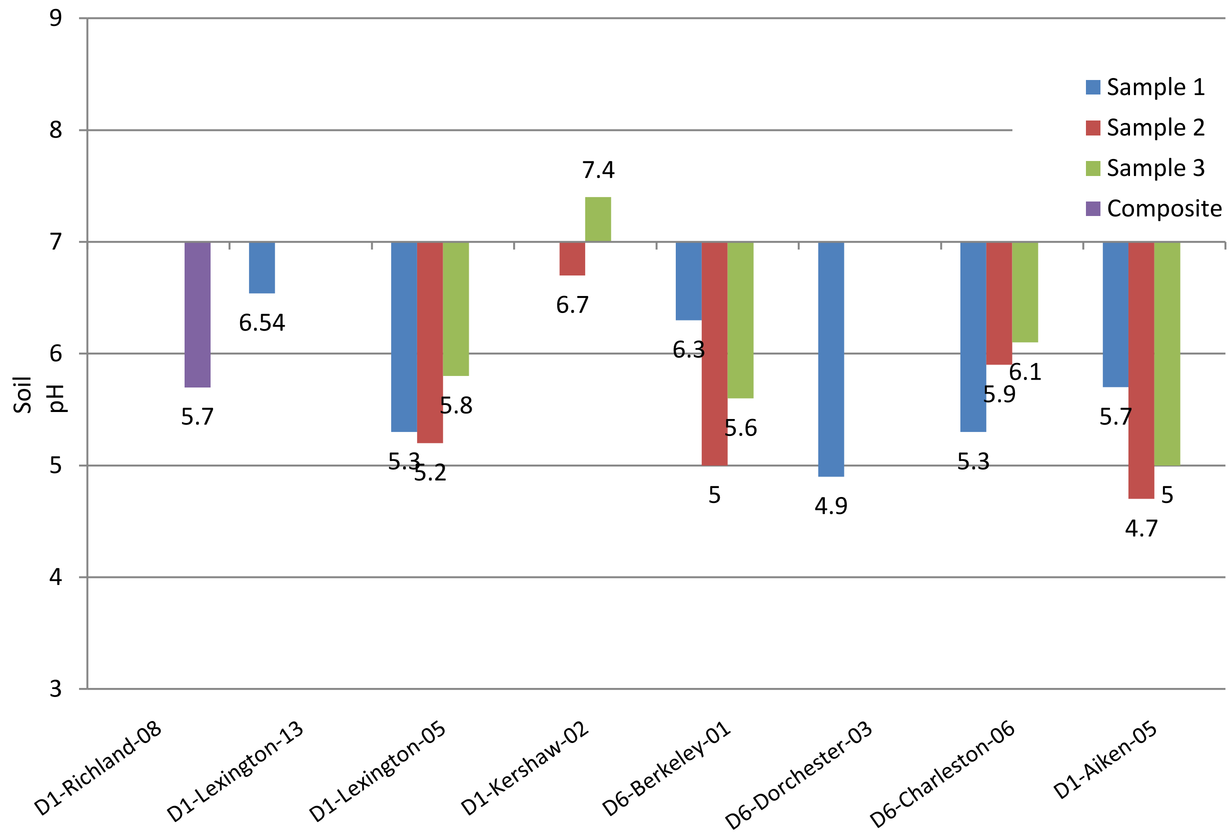

Figure 5.8. Soil pH for Group B Soils

Table 5.6. Description of pH measurements for Group A and B Soils

Description (Soil Survey Division 1993)	**pH Range**	**No. of Borrow Pit Samples**	
		Group A	**Group B**
Ultra acid	< 3.5	---	---
Extremely acid	3.5 - 4.4	---	---
Very strongly acid	4.5 - 5.0	4	4
Strongly acid	5.1 - 5.5	6	3
Moderately acid	5.6 - 6.0	3	5
Slightly acid	6.1 - 6.5	3	3
Neutral	6.6 - 7.3	1	1
Slightly alkaline	7.4 - 7.8	---	1
Moderately alkaline	7.9 - 8.4	1	---
Strongly alkaline	8.5 - 9.0	---	---
Very strongly alkaline	> 9.0	---	---

5.2.2 Soil Resistivity

Figures 5.9 and 5.10 illustrate the minimum soil resistivity measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. Table 5.7 summarizes corrosivity ratings of these soils based on minimum soil resistivity values according to Roberge (2006).

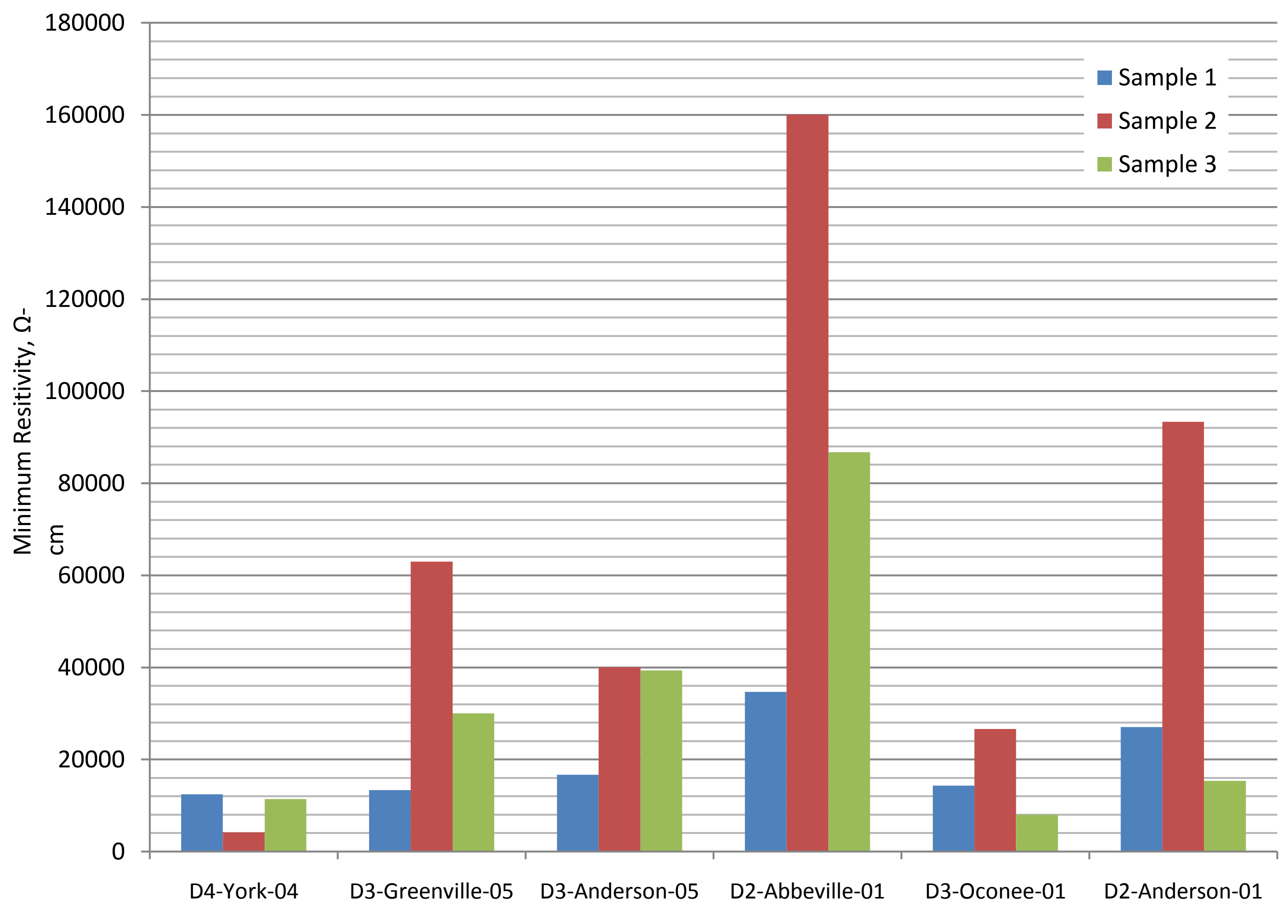

Figure 5.9. Soil Resistivity for Group A Soils

Soil resistivity is a function of its moisture content, salt content, and temperature. When a given soil contains a higher moisture content, electrical conductivity increases and resistivity decreases because the soil matrix provides an improved conduit for transmission. Because of the dependence on moisture content, grain size distribution is also an important factor that determines the minimum soil resistivity. In general, soil resistivity is expected to range from 10^2 to 10^3 Ω-cm for clays, 10^3 to 10^5 Ω-cm for sands, and 10^4 to 10^6 Ω-cm for gravels. None of the soil samples collected from borrow pits in South Carolina contain significant gravel content; rather, most soils are sands with variable fines content, although several samples contain more fines than sand. Thus, the expected range of soil resistivity in this investigation is on the order of 10^3 to 10^5 Ω-cm, which is confirmed by the results shown in Figures 5.9 and 5.10. Salinity also increases electrical conductivity and decreases resistivity. In South Carolina, soils located along or near the coast (Group B) might contain elevated salt content.

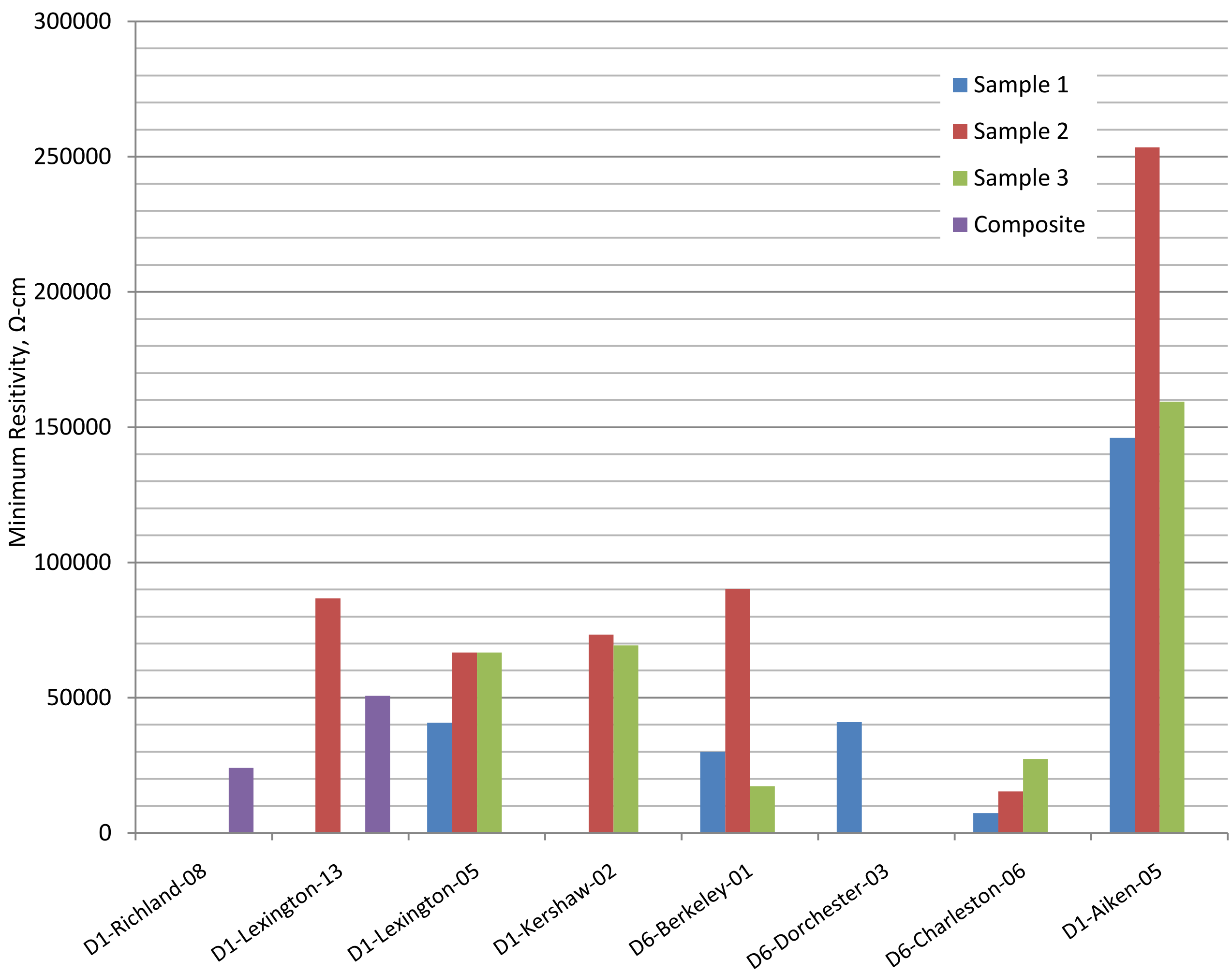

Figure 5.10. Soil Resistivity for Group B Soils

Table 5.7 shows that almost 70% of the soil samples (25 of 36) are classified as non-corrosive and another 20% (8 of 36) are classified as mildly corrosive. The remaining three samples are considered to be moderately corrosive to corrosive and are located in D3-Oconee-01 (8,000 Ω-cm), D6-Charleston-06 (7,300 Ω-cm), and D4-York-04 (4,160 Ω-cm). The soil samples in D3-Oconee-01 (Bucket Sample 3) and D4-York-04 (Bucket Sample 2) are classified as CH and SM with fines contents of 57% and 40%, respectively. The finer grain size distribution of these two soil samples contributes to a lower soil resistivity. The lower soil resistivity of the sample from Charleston, which is classified as SP-SM, is a function of higher salt content; Figure 5.12 shows this particular sample (Bucket Sample 1) contained the highest chloride content of all soil samples tested for chlorides. In general, soils from Group A soil deposits tend to have lower soil resistivity than soils from Group B soil deposits.

Table 5.7. Corrosivity Rating for Group A and B Soils

Corrosivity Rating (Roberge 2006)	Resistivity Range (Ω-cm)	No. of Borrow Pit Samples	
		Group A	Group B
Essentially non-corrosive	> 20,000	10	15
Mildly corrosive	10,000 – 20,000	6	2
Moderately corrosive	5,000 – 10,000	1	1
Corrosive	3,000 – 5,000	1	---
Highly corrosive	1,000 – 3,000	---	---
Extremely corrosive	< 1,000	---	---

5.2.3 Soil Chloride Content

Figures 5.11 and 5.12 illustrate the chloride content measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. None of the measurements exceed 1,000 ppm (0.1%) of chloride, and all but four measurements show chloride contents ≤ 500 ppm (0.05%). The highest chloride content of 960 ppm occurred in a soil sample from Charleston.

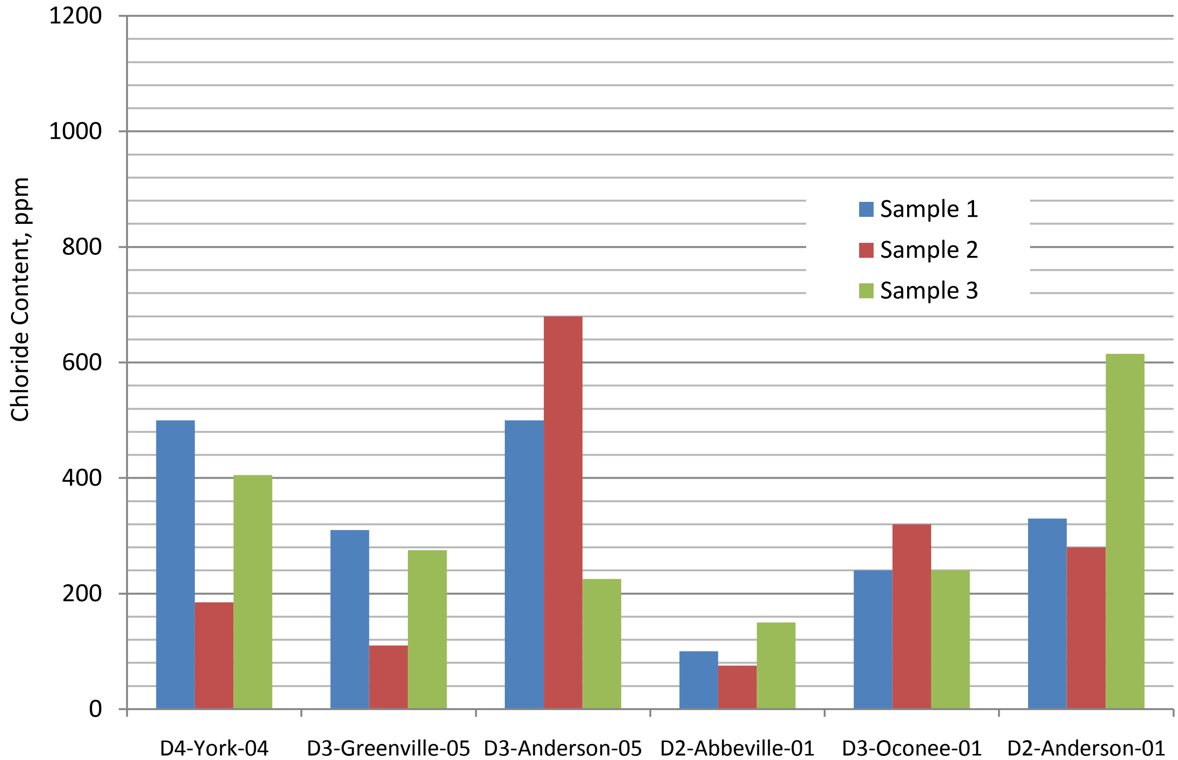

Figure 5.11. Chloride Content for Group A Soils

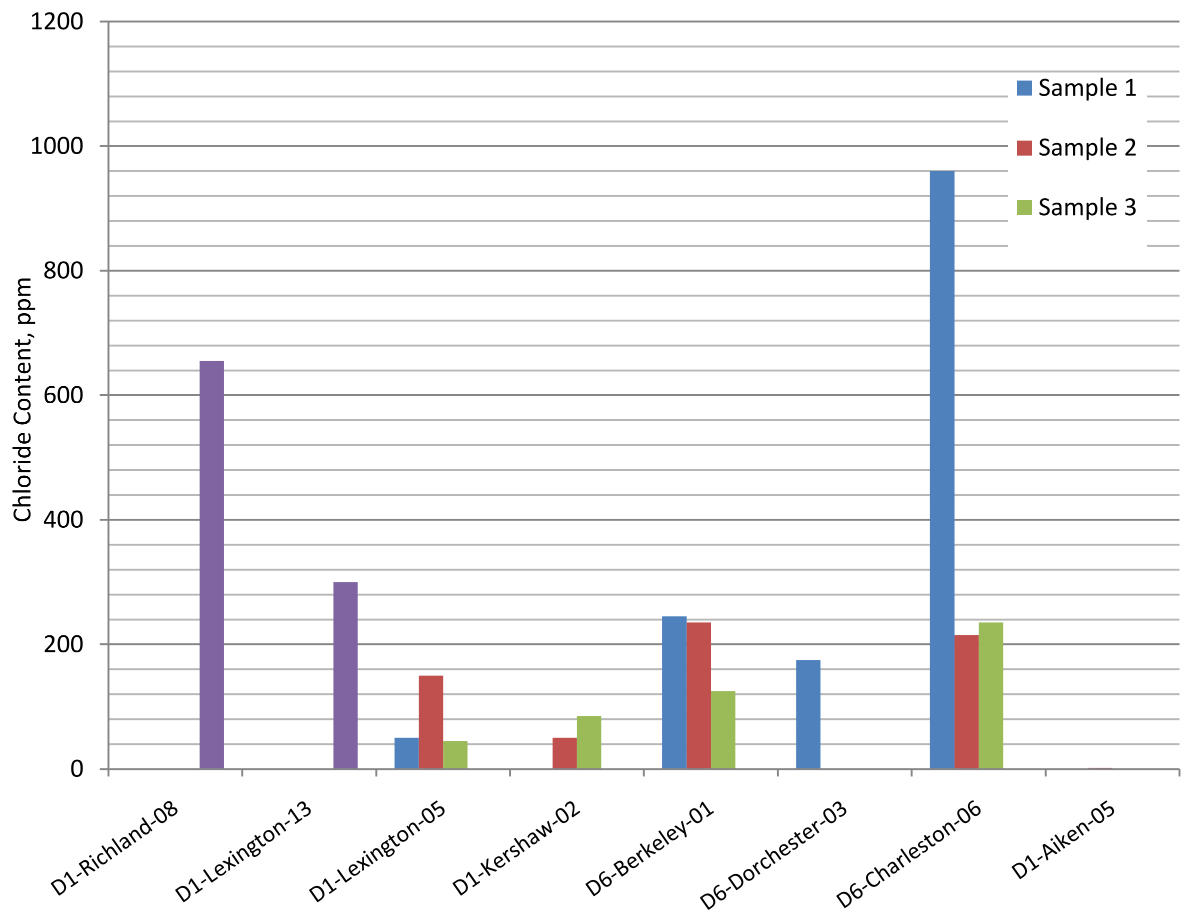

Figure 5.12. Chloride Content for Group B Soils

Table 5.8 compares the chloride concentrations of soil samples in this investigation to those prepared by Maslehuddin et al. (2007) for corrosion tests of steel reinforcement in concrete specimens prepared using Type I Portland cement, Type V Portland cement, and silica fume cement. In their studies, soils were prepared with chloride concentrations of 0.05, 0.1, 0.2, 0.5, 1.0, 2.0 and 3.0%. None of the reinforced concrete specimens exposed to a soil chloride concentration of 500 ppm showed signs of steel corrosion. Concrete containing Type I Portland cement did not shows signs of corrosion at chloride concentrations as high as 2,000 ppm. All of the soils sampled from borrow pits in South Carolina have chloride concentrations well below this limit.

Table 5.8. Corrosion Rating for Group A and B Soils

Corrosion Rating (Maslehuddin et al. 2007)	Chloride Concentration (ppm)	No. of Borrow Pit Samples	
		Group A	Group B
No corrosion [all cements]	500	16 ≤ 500 ppm	18 ≤ 500 ppm
No corrosion [Type I and silica fume cements] Minor corrosion [Type V cement]	1,000 and 2,000	None (2 > 500 ppm)	None (2 > 500 ppm)
No corrosion [Silica fume cements] Minor to moderate corrosion [Type I and V cements]	5,000	---	---

5.2.4 Soil Sulfate Content

Figures 5.13 and 5.14 illustrate the sulfate content measured from each bucket sample of each borrow pit located in Group A and Group B soil deposits, respectively. Sulfate contents of Group A soils are, on average, quite low and do not exceed 35 mg/L. All but one sample have sulfate contents ≤ 20 mg/L, and some samples were below detection limits. Sulfate contents of Group B soils are, on average, higher than Group A soils. However, it is clear that there are two distinct groupings of soil sulfate contents. Soils from borrow pits within the Fall Zone, which include Aiken, Kershaw, Lexington and Richland counties, have sulfate contents greater than those measured from Group A soils. Most of these soils have sulfate contents of at least 100 mg/L, and the soil samples from D1-Aiken-05 range from just below 250 mg/L to just above 300 mg/L. However, soil samples from the Coastal Plain, which include Berkeley, Charleston and Dorchester counties, had sulfate contents below detection limits except for one measurement of 13 mg/L.

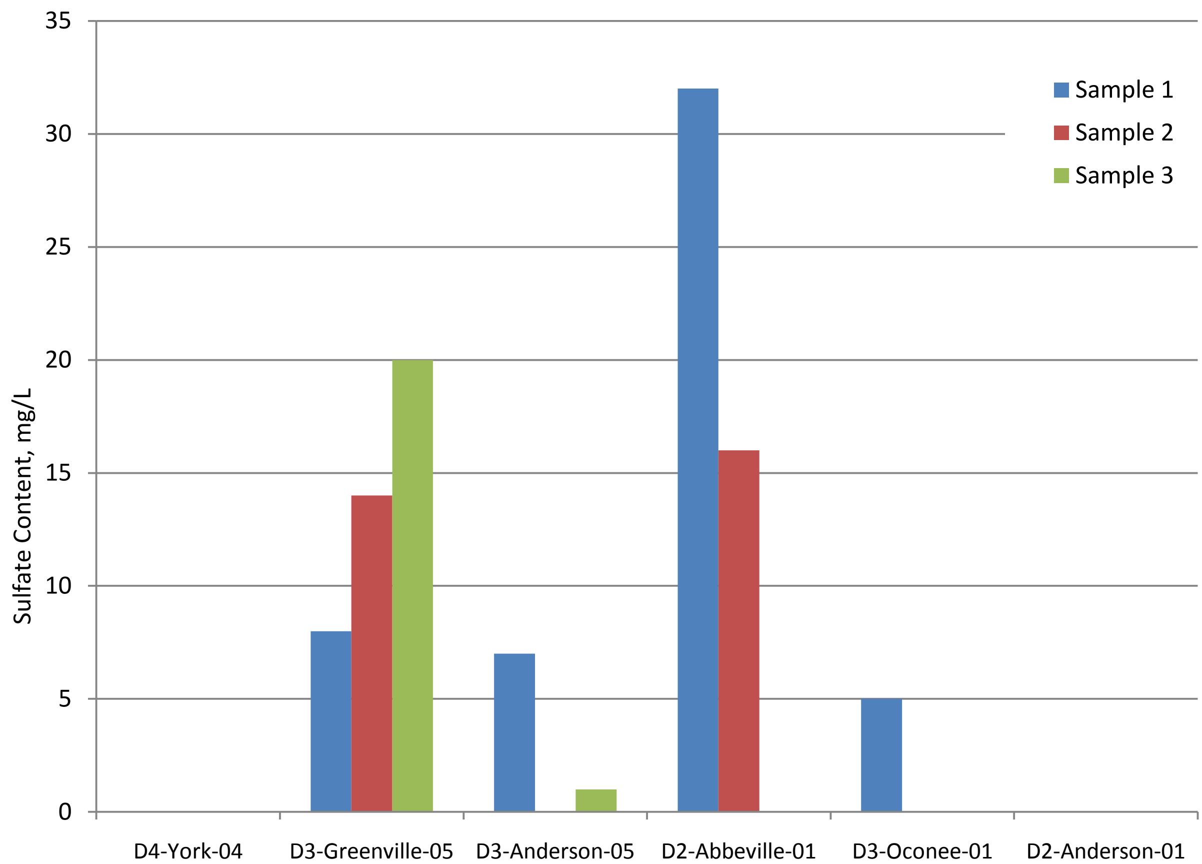

Figure 5.13. Sulfate Content for Group A Soils

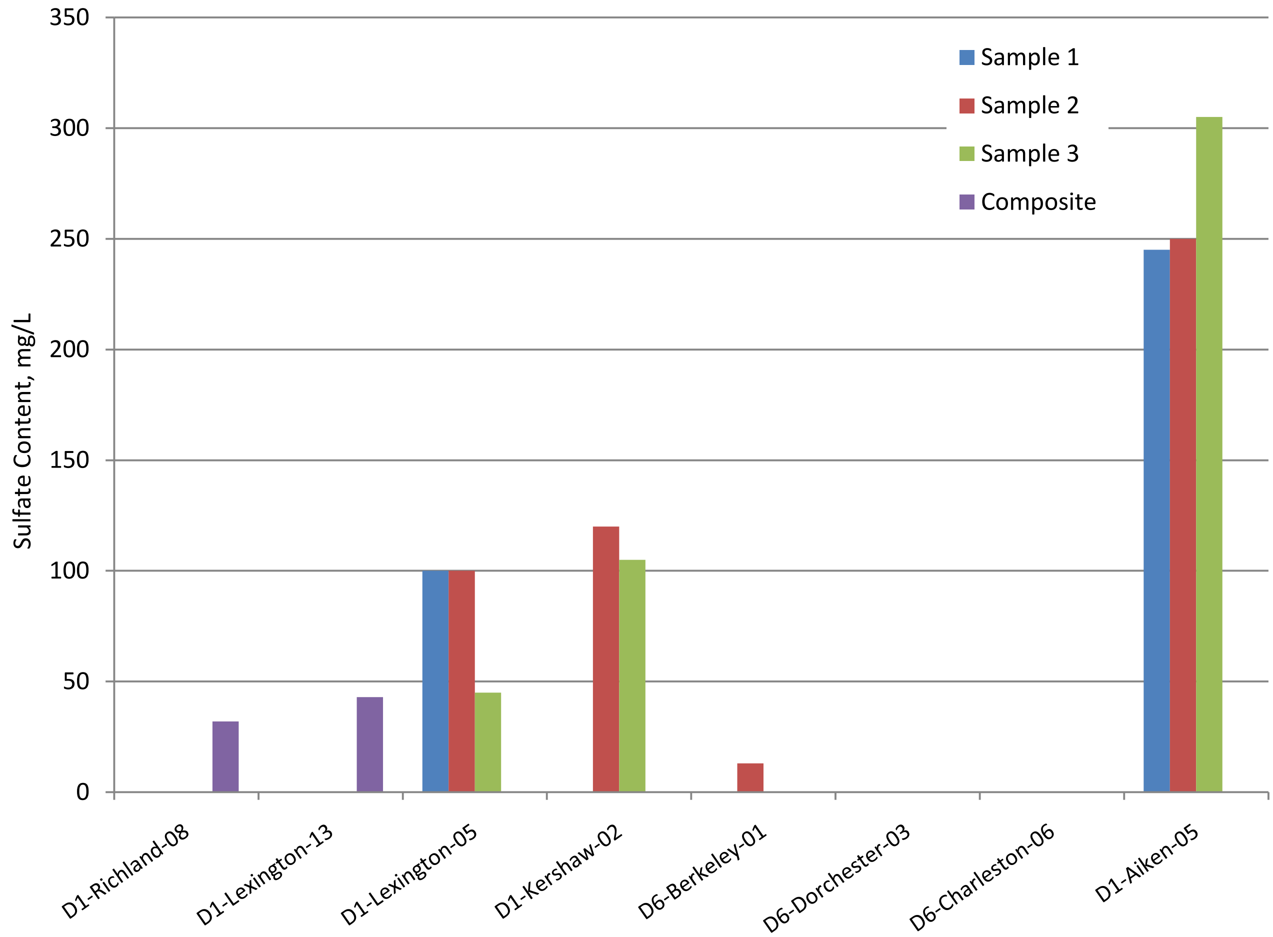

Figure 5.14. Sulfate Content for Group B Soils

Chapter 6 – Results: Strength Parameters

This chapter presents the strength parameters (effective friction angle, ϕ', and effective cohesion, c') of Group A and B soil deposits obtained through 1) a synthesis of data provided by the SCDOT for existing embankment projects and 2) a series of consolidated undrained static triaxial compression tests and direct shear tests performed in the USC Geotechnical Laboratory on soils obtained from borrow pits located in Group A and B soil deposits.

6.1 Synthesis of Triaxial Test Data from Existing SCDOT Embankments

Isotropically consolidated undrained triaxial data for soils used in SCDOT projects with embankments were provided to the investigators by the SCDOT. The data were sorted by county as shown in Table 6.1 for Group A soils and in Table 6.2 for Group B soils. For each SCDOT project file number, boring logs and triaxial data sheets were provided. From this information, the sample type, sample depth, material description and effective strength parameters were tabulated.

The locations of the embankments from which the soils were tested were inferred from the SCDOT Project File numbers and boring logs and are shown in Figure 6.1. While the location of the borrow pit from which each embankment soil was obtained and tested is unknown, it can be assumed to be within the vicinity of the embankment for most cases, thus embankments to the west of the Fall Line are assumed to be constructed from Group A soils and embankments to the east of the Fall Line are assumed to be constructed from Group B soils.

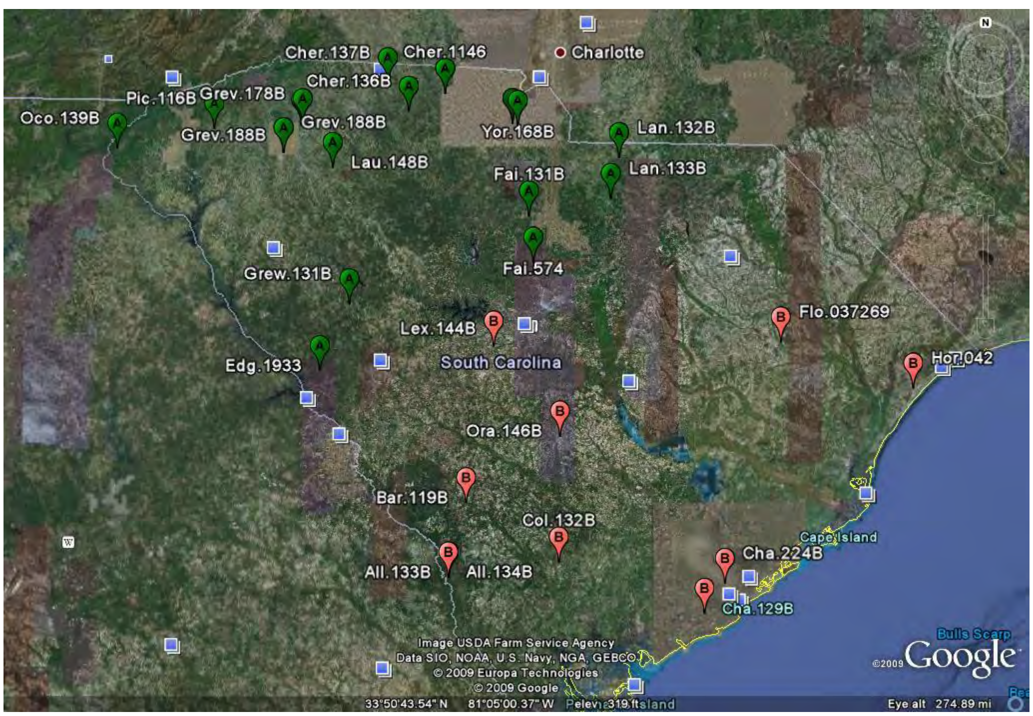

Figure 6.1. Locations of Triaxial Tests Provided by SCDOT

Table 6.1. SCDOT Project CIU Triaxial Results for Group A Soils

County	SCDOT Project File #	Boring No.	Sample No. /Type	Sample Depth	Soil Description	Triaxial Test Results		
						c'		phi'
				(ft)		(kPa)	(psf)	(deg)
Cherokee	11.136B.1	B-5	Bag-1	0-5	Brown-Gray SILT with Sand (ML)	0	0	38
Cherokee	11.137B.1	B-1	Bag-1	0-5	Silty Medium to Fine SAND (SM)	0	0	38
Cherokee/York	1146.101B.1	B-7	Bulk 1	0-5	Black-Brown Silty Medium to Fine SAND	0	0	39
Edgefield/ McCormick	1933.100B	405+00	Bulk-1	0-5	Red-Tan-Orange Lean CLAY with Sand (CL)	10	200	34
Fairfield	20.131B.1	Station 543+00	BS-1/Bulk	0-3	Brown & Yellowish Brown Sandy Lean Clay	5	100	37
Greenville	23.178B	B-2	Bulk	0-3	Reddish Brown, Brown, and White Silty Fine to Medium Sand with Partially Weathered Rock	0	0	43
Greenville	23.178B	B-24	Bulk	0-5	Light Brownish Red Silty Fine to Medium Sand	0	0	37
Greenville	23.188B	B-2	Bulk	0-5	Reddish Brown Clayey Fine to Medium Sand	5	100	36
Greenville	23.188B	B-8	Bulk	0-5	Brownish Red Sandy Lean Clay	0	0	39
Greenwood	24.131B	B-2	Bulk	0-5	Brown Clayey Fine to Coarse Sand	0	0	35
Greenwood	24.131B	B-14	Bulk	0-5	Light Brownish Yellow Clayey Fine to Medium Sand	0	0	38
Lancaster	29.132B	B-2	S-1	0-1.5	Tan-Brown Silty Sandy CLAY (CL)	4	73	29.4
Lancaster	29.132B	B-12	S-2	1.5-3.0	Tan & Gray Silty Sandy CLAY (CL)	3	62	28.5
Lancaster	29.133B.01	B-3	Bulk	0-6	Orange Silt	2	40	32
Lancaster	29.133B.01	B-11	Bulk	0-6	Red Silt with Rock	11	220	32.3
Lancaster/ Chesterfield	1329.100B.01	B-3	Bulk S-1	0-7	Tan-Olive Silty Sandy CLAY with Gravel (CL)	7	144	30.4
Lancaster/ Chesterfield	1329.100B.01	B-16	Bulk S-2	0-7	Tan-Brown Silty Clayey SAND (SC)	8	161	27.8
Laurens	30.148B	St 71+50	Bulk	0-2	Brownish Red Sandy Silt	5	100	36
Oconee	37.139B	B-6	Bulk	0-3	Brown Silty Fine to Medium Sand	0	0	40
Pickens	39.116B	B-1 (96+00)	Bulk	0-2	Brownish Red Silty, Clayey Fine to Medium Sand	0	0	39
Pickens	39.116B	St 76+00; 20' Left of CL	Bulk		Brownish Red Silty, Clayey Fine to Medium Sand	0	0	39
Pickens	2339.100B.1	St 15+50	Bulk	0-5	Dark Brown Silty Sand	0	0	35.1
York	46.168B	BS-1	Bulk	0-5	Reddish Brown and Brown Sandy Lean Clay	5	100	36
York	46.169B	BS-1 (64+00)	Bulk	0-5	Dark Brown Clayey Fine to Medium Sand	5	100	38

Table 6.2. SCDOT Project CIU Triaxial Results for Group B Soils

County	SCDOT Project File #	Boring No.	Sample No. /Type	Sample Depth	Soil Description	Triaxial Test Results		
						c'		phi'
				(ft)		(kPa)	(psf)	(deg)
Allendale	3.133B.1	116+00	Bulk 1	0-5	Brown Fine to Medium Sand	0	0	35.2
Allendale	3.134B.1	169+50	Bulk 1	0-5	Red tan silty fine to coarse sand	0	0	35.7
Barnwell	6.119B.1	392+00 B-7	Bulk S-1	4-15	Red Silty Clayey Sand (SC)	13	274	30.8
Charleston	10.129B	B-6	Bulk	0-5	Tan Silty Sand	29	600	20
Charleston	10.224B	B-2A	SCI B-2A, 24'/ST	23-25	CH (gray, fat clay)	25	518	22.5
Charleston	10.224B	B-2A	SCI B-2A, 36'/ST	35-37	CH	25	518	22.5
Charleston	10.224B	B-33A	GTX B-33A, 21'/SS	20-22	gray, elastic silt (MH)	7	144	16
Charleston	10.224B	B-33A	GTX B-33A, 31'/SS	30-32	CH	5	101	17.5
Charleston	10.224B	B-33A	GTX B-33A, 51'/SS	50-52	Olive-gray, elastic SILT with sand (MH)	36	749	27
Charleston	10.224B	B-23 Alt-1	SCI B-23 Alt-1, 51'/SS	51	Yellow brown, silty SAND (SM)	40	835	38
Charleston	10.224B	B-39A	GTX B-39A, 16'/ST	15-17	Gray, fat CLAY (CH)	11	230	18.5
Charleston	10.224B	B-55B	GTX B-55B, 36'/SS	35-37	Gray, fat CLAY (CH)	35	734	12
Charleston	10.224B	B-74A	S&ME B-74A, 66'/ST	64-66	Yellow-brown, sandy elastic SILT (MH)	4	86	41
Colleton	15.132B.1	B-10	Bulk	0-5	Olive-Gray Silty Clayey SAND (SM)	0	0	35.8
Florence	21.037269	B-1	Bulk 2	0-10	Brown with Tan Clayey Sand (SC-SM, A-2-4)	7	150	30.5
Florence	21.037269	B-1	UD/ST	12-14	Clayey Sand-Gray (SC, A-2-6)	5	110	31.8
Florence	21.037269	B-1	UD/ST	33-35	Fat Clay with Sand-Dark (CH, A-7-6)	49	1030	17.9
Florence	21.037269	B-7	UD/ST	15-17	Clayey Sand -Gray (SC, A-6)	0	0	31.2

Notes: ST: Shelby Tube; UD: Undisturbed; SS: Split Spoon

Table 6.2. (cont.) SCDOT Project CIU Triaxial Results for Group B Soils

County	SCDOT Project File #	Boring No.	Sample No. /Type	Sample Depth	Soil Description	Triaxial Test Results		
						c'		phi'
				(ft)		(kPa)	(psf)	(deg)
Florence	21.037269	B-7	UD/ST	33.5-35.5	Sandy Lean Clay-Gray (CL, A-7-6)	11	220	27.3
Florence	21.216B	AB-1 (95+00)	Bulk 1	0-5	Brown Silty Clayey SAND (SC)	15	305	30
Horry	6211-03-042	B-1	UD/ST	13.5-15	Grey Clayey Sand with Shell Pieces	3	60	27.1
Horry	6211-03-042	RB-26	UD/ST	8-10	Light Grey Silty Clay	9	170	2.6
Lancaster	29.132B	B-2	Bulk S-1	0-1.5	Tan-Brown Silty Sandy CLAY (CL)	4	73	29.4
Lancaster	29.132B	B-12	Bulk S-2	1.5-3.0	Tan & Gray Silty Sandy CLAY (CL)	3	62	28.5
Lexington	32.144B	B-3	Bulk	0-5	Brown gray silty medium to fine sand (A-2-4)	2	43	34.5
Lexington	32.144B	B-21	Bulk	0-5	Gray tan silty fine to medium sand (A-1-b)	1	14	35.9
Orangeburg	38.146B	B-2	UD	19-21.5	n/a	17	360	32.2
Orangeburg	38.146B	G11962/ B4	UD	19-21.5	n/a	12	259	28.1
Orangeburg	38.146B	G11961/ B2	UD	19-21.5	n/a	17	360	32.2
Notes: ST: Shelby Tube; UD: Undisturbed								

The data sheets indicate that both bulk samples and Shelby tube samples were obtained. The bulk samples are from existing embankment soils and presumably represent local borrow sources. There were also soils taken at depths below the natural ground (Shelby tube/ UD samples) and are included in the tables for reference. According to the data sheets, bulk samples were remolded to 95% MDD at optimum moisture content in most cases. Data sheets for Laurens, Greenville and Greenwood reported remolding at 2% wet of optimum.

Note that the CH soils in Table 6.2 are from borings at depth and these are not soils that are anticipated for use in embankment construction.

The range of effective strength parameters for each reported USCS soil type and associated county are summarized in Table 6.3. The table represents tests performed on bulk samples and does not include data from tests performed on samples taken at depths below the natural ground. Note that on average Group A soils have higher effective friction angles than Group B soils.

For Group A embankment soils, the effective friction angles, ϕ', for CL, ML, SC and SM soils range from 29 to 39°, 32 to 38°, 28 to 39° and 35 to 43°, respectively. Comparing these results to the maximum allowable effective friction angles for compacted soils (26°, 28°, 28° and 34° for CL, ML, SC and SM soils, respectively) per Table 7-17 in the SCDOT GDM (see Figure

2.3), it is observed that all of the effective friction angles determined from the CIU triaxial tests are above the maximum allowed friction angles, indicating that the maximum angles in the table are conservative based on the CIU data to date. The effective cohesion, c', reported in Table 6.3 ranges from 0 to 220 psf and indicates that in some cases, the effective cohesion of the Group A soils can exceed the maximum allowed effective cohesion (50 psf per Table 7-17).

Furthermore, these results for Group A soils are in agreement with the published laboratory shear strength testing results for Piedmont residual soils (Sabatini 2002) that indicate an average effective friction angle of 35.2° with a ±1 standard deviation range of 29.9°< φ' < 40.5° and a conservative lower bound of 27.3° and more specifically define the range of expected effective friction angles for each of the CL, ML, SC and SM soils found in the state of South Carolina.

For Group B embankment soils summarized in Table 6.3, the effective friction angles for CL, SC, SC-SM and SM soils range from 28.5 to 29.4°, 30 to 30.8°, 30.5° and 20 to 35.9°, respectively. Note that the minimum friction angle of 20° for the SM soils is for one embankment and is inconsistent with the other 5 SM samples that have friction angles in a more narrow range of 34 to 35.9°. Comparing these results to the maximum allowable effective friction angles for compacted soils (26°, 28° and 34° for CL, SC, and SM soils, respectively (no data is provided for SC-SM soils)) per Table 7-17 in the SCDOT GDM (see Figure 2.3), it is observed that all of the effective friction angles are above the maximum allowed friction angles as they were for the Group A soils. Also, the effective cohesion, c', of the Group B soils (e.g. up to 305 psf for SC soils) can exceed the maximum allowed effective cohesion (50 psf per Table 7-17).

Furthermore, for the CL, SC and SC-SM soils found in Lancaster, Barnwell and Florence counties, these results are in agreement with SCDOT experience that borrow materials typically found in Group B have effective friction angles ranging from 28° < φ' < 32°. However, effective friction angles for SM soils found in Allendale, Colleton, Charleston and Lexington counties have friction angles in the range of 34 to 35.9° which exceed the typically expected range. Also note that the CL soils from Lancaster have low friction angles (28.5 to 29.4°) and little to no cohesion.

Table 6.3. Summary of Effective Strength Parameters for Soils Used in Embankments on SCDOT Projects

Soil Type	Counties		c' (kPa)	c' (psf)	phi' (deg)
Group A					
CL	Chesterfield	Lancaster	0 to 10	0 to 200	29 to 39
	Edgefield	McCormick			
	Fairfield	York			
	Greenville				
ML	Cherokee		0 to 11	0 to 220	32 to 38
	Lancaster				
	Lancaster				
SC	Chesterfield	Lancaster	0 to 8	0 to 161	28 to 39
	Greenville	Pickens			
	Greenwood	York			
	Greenwood				
SM	Cherokee	Oconee	0 to 5	0 to 100	35 to 43
	Greenville	Pickens			
	Greenville	Pickens			
	Laurens	York			
Group B					
CL	Lancaster		3 to 4	62 to 73	28.5 to 29.4
SC	Barnwell		13 to 15	274 to 305	30 to 30.8
	Florence				
SC-SM	Florence		7	150	30.5
SM	Allendale	Colleton	0 to 29	0 to 600	20 to 35.9
	Charleston	Lexington			

6.2 Strength Tests Performed at USC

6.2.1 Consolidated Undrained Triaxial Tests

Consolidated undrained (CIU) triaxial tests with pore pressure measurements were performed at USC on samples of the soils obtained from the borrow pits listed in Table 6.4. Tests were performed on samples from York and Anderson counties to study SM and MH soils from Group A; tests were performed on samples from Lexington and Richland counties to study SC and ML soils in Group B near the Fall Line; and, tests were performed on samples from Berkeley and Dorchester Counties to study SM and SP-SM soils in Group B near the coastal area. Selection of samples in these counties further expands the database of CIU triaxial test results synthesized in Section 6.1.

Specimens were prepared in accordance with the procedures in Section 4.2.3 and the properties of the specimens are summarized in Table 6.4 along with the pre-shearing parameters. The moisture content, w_i, of each specimen was within 1 to 2% of w_{opt} in all cases and the dry unit weight, γ_d, exceeded 95% maximum standard Proctor density in all cases. A B-value greater than 0.95 was achieved in all cases, indicating saturated conditions.

Table 6.4. Specimen Properties and Pre-Shearing Parameters for Triaxial Tests Performed at USC

Pit & Bucket No.	Classification	Specimen No.	Sample Properties		Pre-Shearing Parameters		
			w_i	γ_d	σ'_{3c}		B Value
			(%)	(lb/ft^3)	(kPa)	(ksf)	
Group A (Upstate Area)							
D4-York-04; B-2	SM	1	14.2	110.0	48	1	1.00
		2	15.4	110.1	98	2	1.00
D2-Anderson-01; B-1	MH	1	25.3	92.7	96	2	0.99
		2	25.0	94.8	48	1	0.98
D2-Abbeville-01; B-1	SM	1	14.9	102.2	96	2	0.98
		2	14.6	102.9	48	1	0.97
Group B (Fall Line)							
D1-Lexington-05; B-1	SC	1	12.1	114.4	96	2	0.97
		2	10.9	114.9	48	1	0.96
D1-Richland-08; B-1	ML	1	17.4	103.3	48	1	0.96
		2	16.1	101.3	96	2	1.00
Group B (Coastal)							
D6-Berkeley-01; B-2	SM	1	12.2	112.6	48	1	0.95
		2	12.3	116.8	96	2	0.97
D6-Dorchester-03; B-1	SP-SM	1	11.3	104.0	96	2	0.96
		2	13.2	104.2	48	1	0.95
		3	12.8	103.6	144	3	0.96
Charleston; B-1	SP	1	9.8	113.8	48	1	0.96
		2	10.9	111.3	96	2	0.98

The triaxial test results for each of the borrow pit soils in Table 6.4 are shown in Figures 6.2 to 6.7. Each figure includes a plot of the principal stress difference versus axial strain, excess pore pressure versus axial strain, and the effective and total stress paths in q-p space. The effective friction angle, ϕ', and effective cohesion, c', obtained from these tests are summarized in Table 6.5. The effective friction angle, ϕ', for Group A soils ranges from 26 to 34°. For Group B soils, it ranges from 27 to 33° near the Fall Line and from 30 to 37° near the coast.

Table 6.5. Strength Parameters from Triaxial Tests Performed at USC

Pit & Bucket No.	Classification	Specimen No.	Strain Rate, dδ		ϕ'	c'	
			(in./min)	(mm/min)	(deg)	(kPa)	(psf)
Group A (Upstate Area)							
D4-York-04; B-2	SM	1	0.003	0.076	34°	6	125
		2	0.003	0.076	34°	18	376
D2-Anderson-01; B-1	MH	1	0.003	0.076	26°	28	585
		2	0.003	0.076	26°	28	585
D2-Abbeville-01; B-1	SM	1	0.010	0.254	31°	0	0
		2	0.010	0.254	34°	5	104
Group B (Fall Line)							
D1-Lexington-05; B-1	SC	1	0.005	0.127	33°	14	292
		2	0.075	1.905	33°	18	376
D1-Richland-08; B-1	ML	1	0.005	0.127	28°	12	251
		2	0.005	0.127	27°	20	418
Group B (Coastal)							
D6-Berkeley-01; B-2	SM	1	0.003	0.076	30°	21	439
		2	0.003	0.076	34°	0	0
D6-Dorchester-03; B-1	SP-SM	1	0.010	0.254	37°	25	522
		2	0.010	0.254	35°	0	0
		3	0.010	0.254	37°	25	522
Charleston; B-1	SP	1	0.009	0.229	37°	0	0
		2	0.009	0.229	34°	0	0

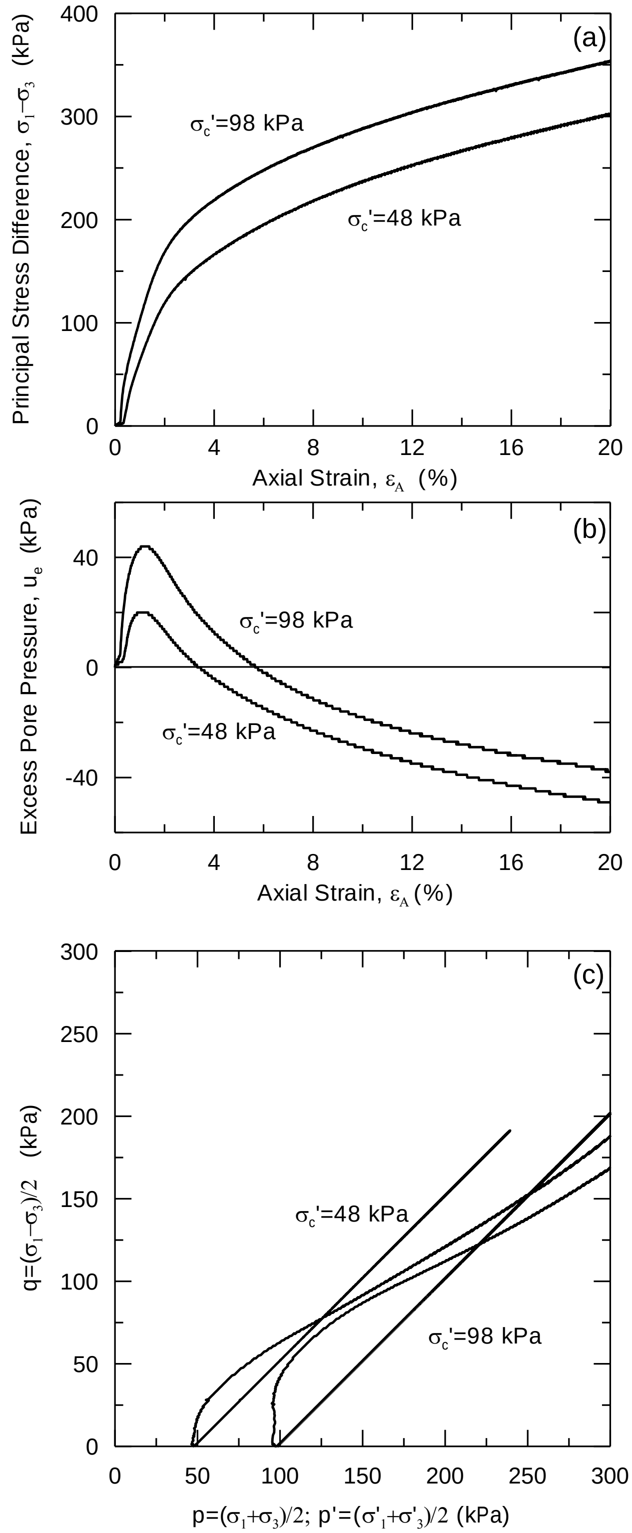

Figure 6.2. Static Triaxial Test Results for D4-York-04; B-2: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

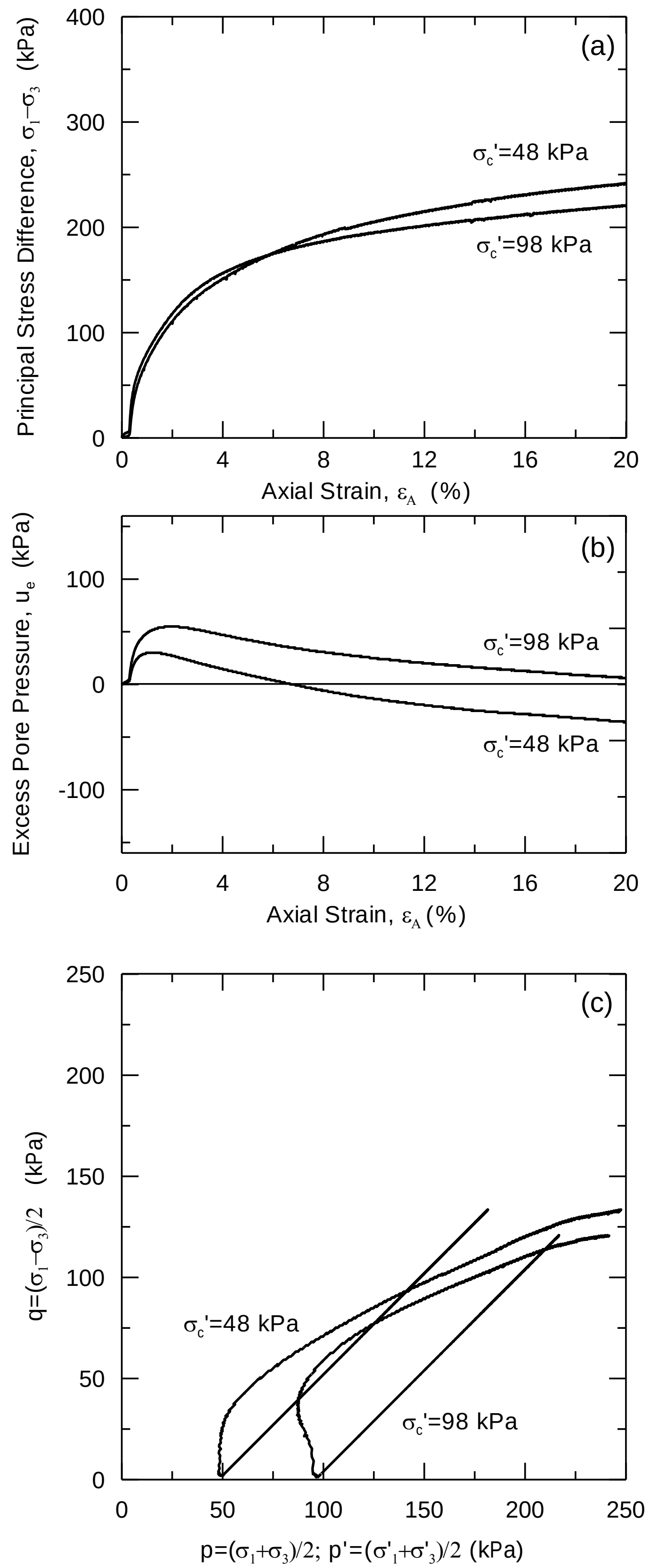

Figure 6.3. Static Triaxial Test Results for D2-Anderson-01; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

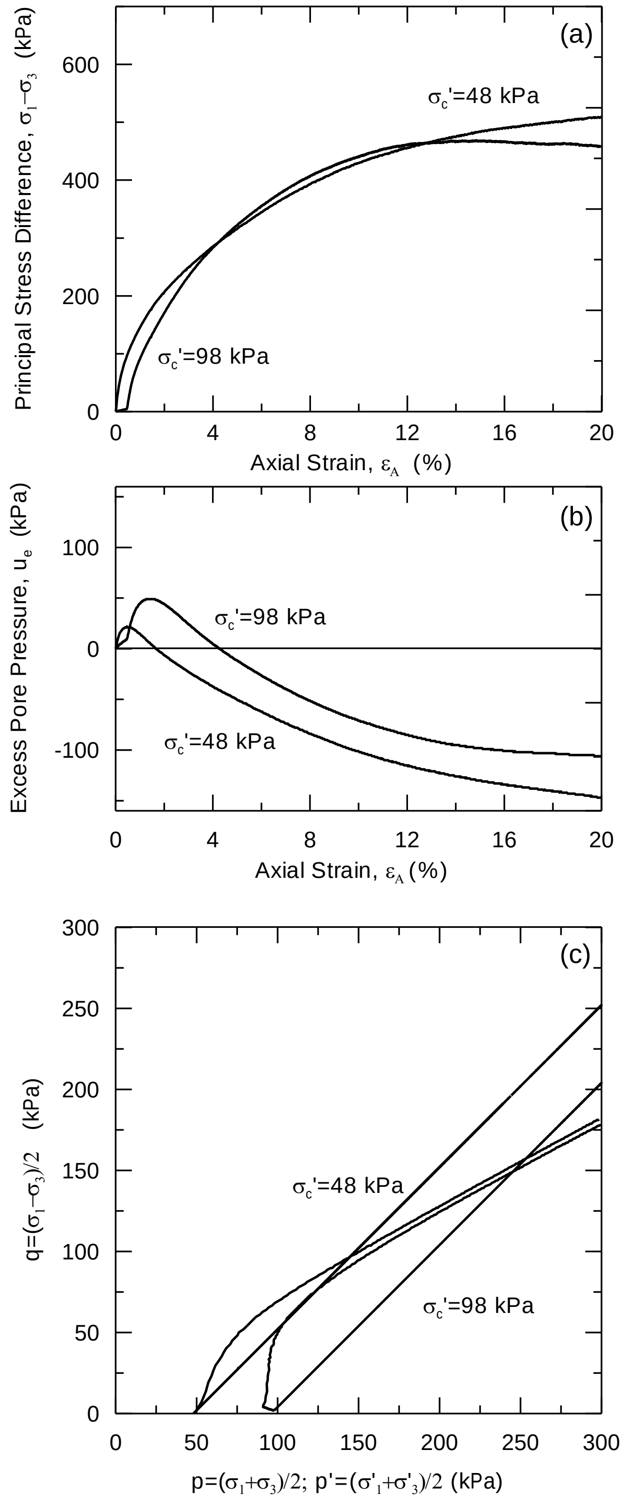

Figure 6.4. Static Triaxial Test Results for D1-Lexington-05; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

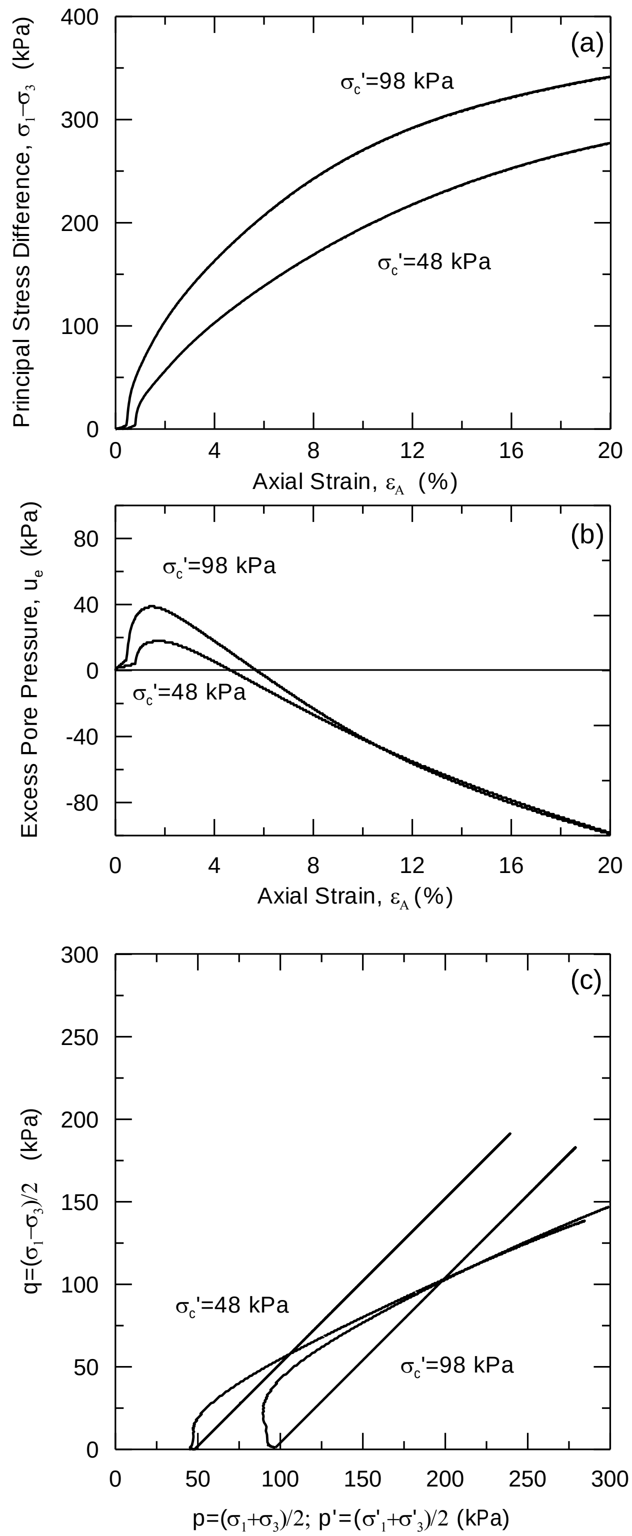

Figure 6.5. Static Triaxial Test Results for D1-Richland-08; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

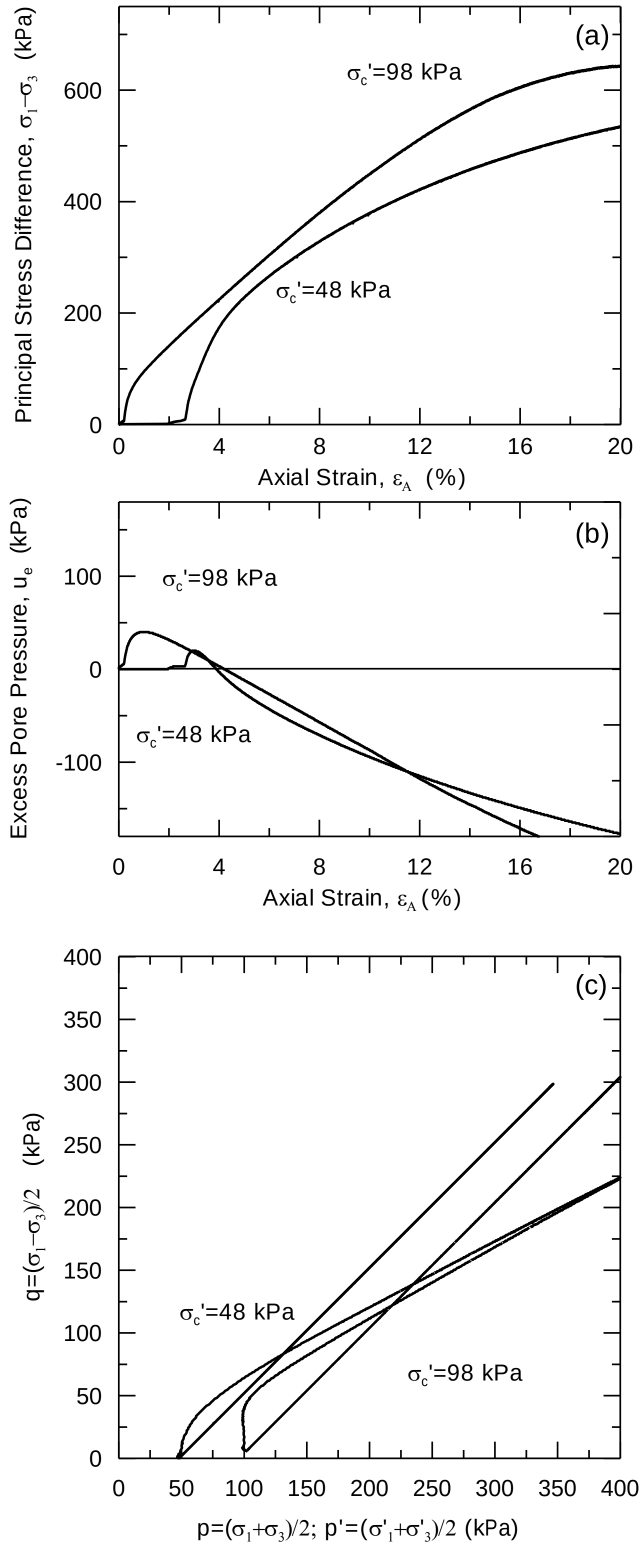

Figure 6.6. Static Triaxial Test Results for D6-Berkely-01; B-2: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

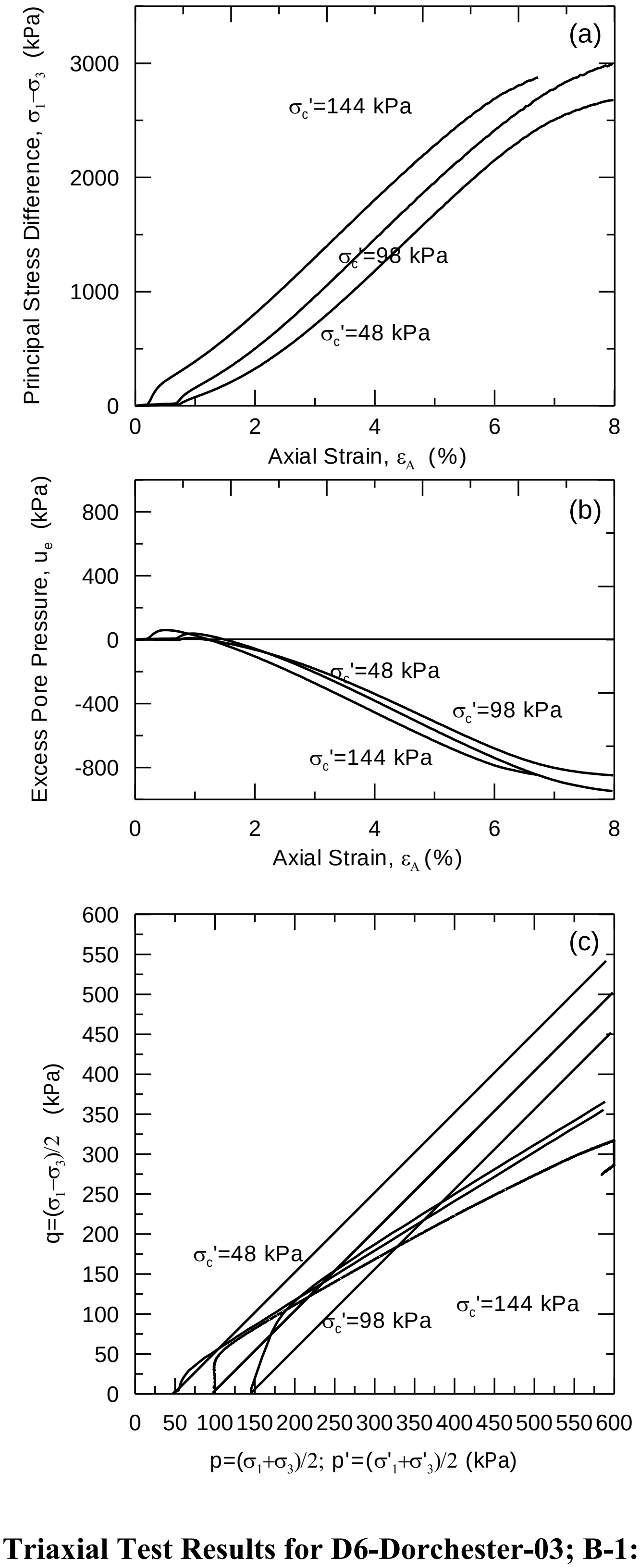

Figure 6.7. Static Triaxial Test Results for D6-Dorchester-03; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

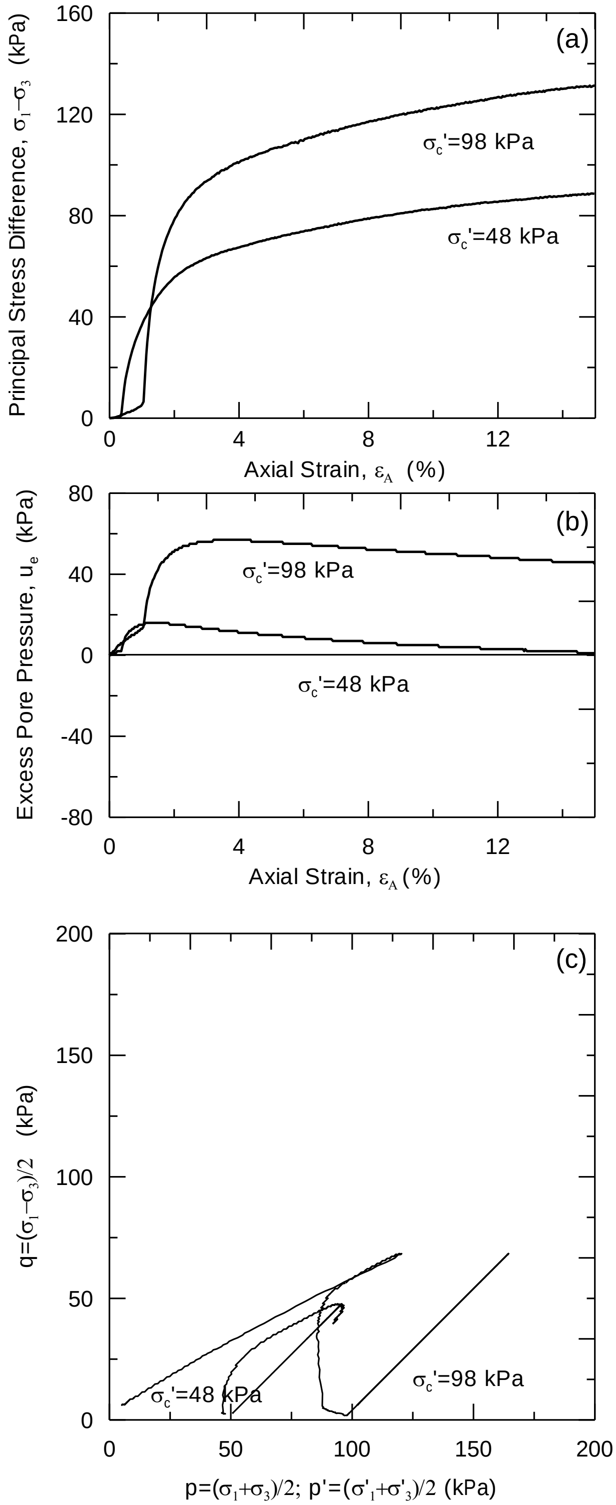

Figure 6.8. Static Triaxial Test Results for D2-Abbeville-01; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

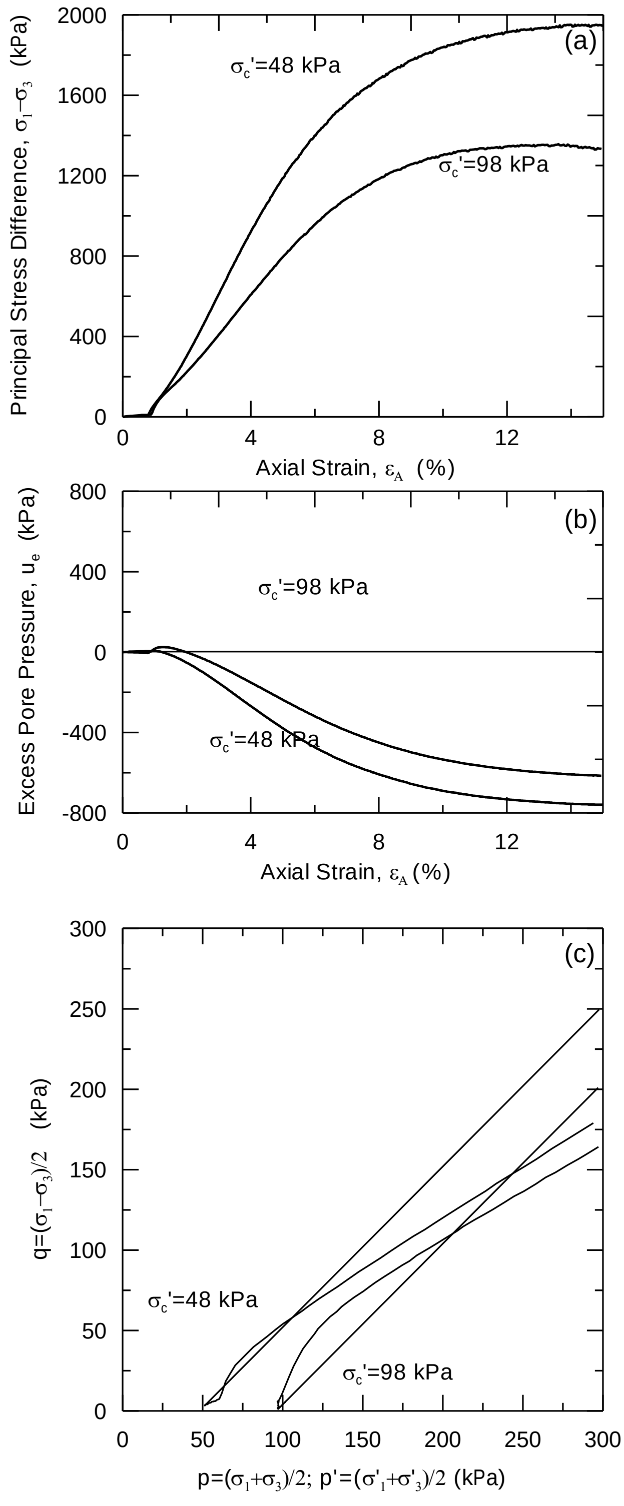

Figure 6.9. Static Triaxial Test Results for Charleston; B-1: a) Principal Stress Difference versus Axial Strain, b) Excess Pore Pressure versus Axial Strain, and c) Effective and Total Stress Paths.

6.2.2 *Direct Shear Test Results*

Direct shear tests were performed at USC on samples of the soils obtained from the borrow pits listed in Table 6.6. Tests were performed on samples from York, Anderson, Abbeville and Oconee counties to study SM, MH and CH soils from Group A; tests were performed on samples from Lexington, Richland, Kershaw and Aiken counties to study SC, SP-SM, ML, SW-SM and SP-SM soils in Group B near the Fall Line; and, tests were performed on samples from Charleston, Berkeley and Dorchester Counties to study SM and SP-SM soils in Group B near the coastal area. The index properties of each of the soils tested is summarized in Table 6.6.

Table 6.6. Index Properties for Soils Tested in Direct Shear

Pit & Bucket No.	USCS	Specific Gravity	Grain Shape of Coarse Fraction	C_u	C_c	Fines Content	LL	PI	w_{opt}	$\gamma_{d,max}$	95% $\gamma_{d,max}$
						%	%	%	%	(pcf)	(pcf)
Group A (Upstate Area)											
D4-York-04, B-2	SM	2.63	Subangular	60	4.4	40	NP	NP	15.4	114	108
D2-Anderson-01, B-1	MH	2.74	Subrounded	141.7	0.007	53	54	18	24.1	97	92
D3-Anderson-05, B-1	MH	2.79	Subrounded	160	0.03	53	55	13	23	99	94
D2-Abbeville-01, B-1	SM	2.67	Subangular	72.1	1.8	36	45	17	15	110	105
D2-Abbeville-01, B-3	SM	2.64	Subrounded	17.9	1.6	35	38	1	14	110	105
D3-Oconee-01, B-1	CH	2.75	Subrounded to Rounded	100	0.04	58	52	27	23.5	103	98
Group B (Fall Line)											
D1-Lexington-05, B-1	SC	2.76	Subangular to Subrounded	35.4	4.8	18	31	12	13	122	116
D1-Lexington-13, B-1	SP-SM	2.69	Subangular	5.1	1.3	6	NP	NP	10.5	122	116
D1-Richland-08, B-1	ML	2.86	Subrounded	8.8	1.8	75	31	7	17	106	101
D1-Richland-08, B-2	ML	2.89	Subrounded	5.5	1.2	88	35	1	14.7	110	105
D1-Kershaw-02, B-2	SW-SM	2.74	Subangular to Rounded	6.1	1.6	12	NP	NP	14	101	96
D1-Aiken-05, B-2	SP-SM	2.66	Angular	2.2	1.0	6	NP	NP	16	102	97
Group B (Coastal)											
D6-Charleston-06, B-1	SM	2.53	Subangular to Subrounded	11	5.1	20	NP	NP	16.5	106	101
D6-Berkeley-01, B-2	SM	2.54	Subrounded to Rounded	51.4	10.2	32	NP	NP	12.3	113	107
D6-Dorchester-03, B-1	SP-SM	2.71	Subrounded	4.8	3.0	12	NP	NP	12	107	102

Specimens were prepared in accordance with the procedures in Section 4.2.2 and the properties of the specimens are summarized in Table 6.7. The moisture content, w_i, of each specimen was within 1 to 2% of w_{opt} in all cases and the dry unit weight, γ_d, exceeded 95% maximum standard Proctor density in all cases.

The effective friction angle, ϕ', and effective cohesion, c', obtained from the direct shear tests is summarized in Table 6.7. The effective friction angles, ϕ', for Group A soils range from 31.7 to 39.8°. For Group B soils, they range from 32.5 to 46.2° near the Fall Line and from 30.6 to 37.9° near the coast. In general, effective friction angles from direct shear tests are higher than those obtained from triaxial testing because the direct shear test is performed under plane strain conditions.

Table 6.7. Direct Shear Test Results: Specimen Properties and Strength Parameters

Pit & Bucket No.	USCS	Specimen No.	w_i	γ_d	φ'	c'	
			(%)	(pcf)	deg	(kPa)	(psf)
Group A (Upstate)							
D4-York-04; B-2	SM	1	14.4	110	34.2	61	1273
		2	14.4	110			
		3	14.9	110			
D2-Anderson-01; B-1	MH	1	25	92	39.8	44	924
		2	26	91			
		3	27.1	90			
D3-Anderson-05; B-1	MH	1	22.5	95	30.1	59	1225
		2	22.6	95			
		3	21.7	96			
D2-Abbeville-01; B-1	SM	1	15.7	104	37.2	47	978
		2	16.5	103			
		3	15.9	104			
D2-Abbeville-01; B-3	SM	1	14.3	105	32.2	67	1401
		2	14.6	104			
		3	14.3	105			
D3-Oconee-01; B-1	CH	1	24.1	97	31.7	29	613
		2	24.2	97			
		3	22.7	98			
Group B (Fall Line)							
D1-Lexington-05; B-1	SC	1	12.7	117	40.2	25	518
		2	12.3	117			
		3	12.3	117			
D1-Lexington-13; B-1	SP-SM	1	10	117	46.2	23	476
		2	9.9	117			
		3	10.7	116			
D1-Richland-08; B-1	ML	1	17.3	101	32.5	46	962
		2	17.3	101			
		3	16.9	101			
D1-Richland-08; B-2	ML	1	15.7	104	33.5	56	1160
		2	15.6	104			
		3	15.3	104			
D1-Kershaw-02; B-2	SW-SM	1	13.2	97	46.3	6	123
		2	13	97			
		3	13.1	97			
D1-Aiken-05; B-2	SP-SM	1	16.7	97	36.9	25	517
		2	16.5	97			
		3	16.8	97			
Group B (Coastal)							
D6-Charleston-06; B-1	SM	1	17.4	100	37	52	1090
		2	17.8	100			
		3	17.3	101			
D6-Berkeley-01; B-2	SM	1	12.7	108	30.6	60	1262
		2	12.8	108			
		3	12.5	108			
D6-Dorchester-03; B-1	SP-SM	1	13.9	101	37.9	14	300
		2	13.8	101			
		3	13.8	101			

Chapter 7 – Conclusions and Recommendations

7.1 Project Summary

This project was focused on the assimilation of engineering properties of borrow soils across the state of South Carolina. Extensive data on soils used for embankment construction were evaluated and compared within Group A (Piedmont) and Group B (Coastal Plain) soil deposits. A geotechnical materials database was constructed using three main sources of information: 1) review and synthesis of available soils information from 197 borrow pits gathered from the SCDOT Engineering Districts; 2) experimental testing of representative field samples of soils from 17 known borrow sources to determine the physical, mechanical, and chemical properties of these soils; and 3) triaxial compression test results from soil samples of existing embankments, based on a thorough review and synthesis of multiple project reports supplied by SCDOT.

In this investigation, tests for physical properties included visual manual identification, moisture content, specific gravity (G_s), particle size distribution, liquid limit (LL), plastic limit (PL), and AASHTO and USCS soil classifications. Tests for mechanical properties included standard Proctor compaction, direct shear, and triaxial compression. Standard Proctor compaction tests provided the maximum dry density ($\gamma_{d,max}$) and optimum moisture content (w_{opt}) needed for the preparation of direct shear and triaxial compression tests, which were performed to determine the effective friction angle, ϕ', and effective cohesion, c'. Tests for chemical properties included soil pH, soil resistivity, chloride content and sulfate contents. Test methods were performed according to AASHTO standard specifications unless otherwise indicated.

7.2 Conclusions

Based on the contents of this project report, the following major conclusions are put forth:

1. The physical properties of soils sampled from borrow pits across South Carolina are reasonable and do not suggest that unusual specimens or unique mineralogical compositions were acquired. For example, values of G_s for most soil samples fall within a common range of 2.50 to 2.80. No sample falls below a G_s of 2.50 and just four samples (three from the same pit) exceed 2.80. In several of the residual soils acquired from borrow pits in Group A, mica was observed in variable quantities and over a range of particle sizes from coarse to fine. Some samples appeared to have trace amounts of mica, while others clearly had higher contents.

2. The predominant USCS and AASHTO soil classifications differ between Group A and Group B soil deposits. In general, the soils in Group B have lower fines content than those in Group A. SP-SM and SW-SM soils are common in Group B but are not found in Group A. The fines content of SM and SC soils in Group B does not exceed 32%;

whereas, all but one of the SM soils in Group A has at least 35% fines. In terms of AASHTO classifications, Group B soils range from A-1 to A-4 and there are no soils with A-5 or higher classifications. In Group A, the preponderance of soil samples are classified as A-5 or higher.

3. Compaction characteristics are a function of soil classification. In Group A, the A-2-4 and A-4 soils have the highest $\gamma_{d,max}$ (> 115 pcf in some cases) and lowest w_{opt} required for compaction. The A-5 and A-7-5 soils have the lowest $\gamma_{d,max}$ (< 100 pcf in some cases) and require the highest w_{opt} for compaction. Mica was observed to be present in some of these soil samples. In Group B, the A-1 and A-2 soil groups tend to produce a higher $\gamma_{d,max}$ at lower w_{opt} than the A-3 and A-4 soil groups. All of the Group B soil samples with $\gamma_{d,max}$ of at least 110 pcf are in the A-1 and A-2 soil groups. There are also noticeable differences in the w_{opt} required for compaction. All of the A-1 and A-2 soils, except one, have $w_{opt} \leq 14\%$. Conversely, all of the A-3 and A-4 soils, except one, have $w_{opt} \geq 14\%$. It should be noted that all of the Group B soils have $w_{opt} < 20\%$; whereas, more than half of the Group A soils have $w_{opt} \geq 20\%$.

4. On average, Group A soils have higher effective friction angles than Group B soils. The results for Group A soils (CL, ML, SC, and SM) are in agreement with the published shear strength testing results for Piedmont residual soils (Sabatini 2002) that indicate an average effective friction angle of 35.2° with a ± 1 standard deviation range of $29.9° < \varphi' < 40.5°$. None of these test results fell below 29° and thus did not approach the conservative lower bound of 27.3°. However, there was one MH soil sample tested (Anderson), and it had an effective friction angle of 26°; there were no strength test results for other MH soils and, thus, no means for comparison. It is recognized that MH soils can be difficult to compact and might not be suitable for embankment construction.

5. In Group B soils, the effective friction angles of SC, SC-SM, CL and ML soils found in Barnwell, Florence, Lancaster and Richland counties are consistent with SCDOT experience of soils with effective friction angles ranging from $28° < \varphi' < 32°$. The effective friction angle of SC soil in Lexington was just above this range with $\varphi' = 33°$. Most of the SM soils have higher effective friction angles in a narrow range of 34° to 36°. One of the soil samples tested in this project, SP-SM soil from Dorchester, had effective friction angles ranging from 35° to 37°. The SP soil from Charleston also had an effective friction angle ranging from 34° to 37°.

6. For all soils in both Group A and Group B deposits, the triaxial compression test results provide effective friction angles that meet or exceed the maximum allowable friction angles (26°, 28°, 28° and 34° for CL, ML, SC and SM soils, respectively) per Table 7-17 in the SCDOT Geotechnical Design Manual. In some cases, the effective cohesion, c', is

also found to exceed the maximum allowable effective cohesion of 50 psf. These observations confirm that the values provided in Table 7-17 offer conservative guidance for design when soil information is not available.

7. The corrosion potential of most soils from borrow sources is low. Based on minimum soil resistivity, almost 70% of the soil samples are classified as non-corrosive and another 20% are classified as mildly corrosive. Soil pH tends to range from very strongly acid to slightly acid, with a few exceptions. The chloride and sulfate contents are also low, with a few exceptions in specific locations.

7.3 Recommendations

Based on the findings presented in this project report, the following recommendations are put forth for implementation in SCDOT practices or for consideration of further studies that could benefit SCDOT. These recommendations are not prioritized or presented in a particular order.

1. Update relevant sections of the SCDOT Geotechnical Design Manual to reflect the expected soil properties, in particular the shear strength parameters, for borrow soils in Group A and Group B soil deposits. Post the Microsoft® Excel geotechnical materials database on the SCDOT website (www.scdot.org) for public access. If desired, monitor the number of views or downloads of the database and/or provide a forum for comments that can be used for continuous improvement (see Recommendation 5).

2. Consider expanding the borrow material specifications from two to three soil groups to distinguish the engineering properties of some soils in the transitional Fall Zone to those in the remaining Coastal Plain region. For example, Group C might include all or some of the following counties: Aiken, Chesterfield, Kershaw, Lexington, and Richland (however, there was limited information gathered in this report on soils in Chesterfield county). More soils data from these particular counties might be desired to warrant such a change (see Recommendation 5).

   Alternatively, appropriate notes could be added to the discussion of Group B soils (see Recommendation 1) to indicate the differences between those soils located within or near the Fall Zone, particularly in Lexington and Richland counties, and those soils located along or near the coast.

3. Incorporate information regarding chemical properties of soils into appropriate sections of the SCDOT Geotechnical Design Manual, such as those sections regarding compacted soil for bridge abutments, MSE walls, buried pipe, and others where compacted soil is in contact with reinforced concrete or metallic elements.

If desired, conduct more targeted field and lab studies on chemical properties of soils to further evaluate corrosion potential in geographical areas of concern (e.g. soils in Coastal Plain with higher chloride contents and/or soils in Fall Zone with higher sulfate contents).

4. Conduct a research project to evaluate the effects of mica content on engineering properties of micaceous, residual soils in the Piedmont region, and determine what, if any, impacts mica content has on long-term embankment performance. As part of this investigation, existing embankments known to contain micaceous soils could be sampled and tested. Mica content is known to have an effect on soil characteristics such as plasticity, compaction density, void ratio, permeability, compressibility and shear strength. The presence of mica can also contribute to an increase in erosion potential.

5. Develop a protocol for growth and continuous maintenance of the geotechnical materials database, such that the number of borrow pits or embankments can be expanded, especially in those counties or engineering districts where there is limited soils information. As more soils and engineering properties are added, the correlations between soil classifications and expected shear strength parameters in Group A and Group B soils can be further refined.

   As part of the protocol, plans for converting the current Microsoft® Excel database into a GIS format could be considered.

Chapter 8 – References

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