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Strength and Erodability of South Carolina Soils for Embankment Construction

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STRENGTH AND ERODIBILITY OF SOUTH CAROLINA SOILS FOR EMBANKMENT CONSTRUCTION

by

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ABSTRACT

When a particular soil from a borrow pit is being used to construct a road embankment, it is important that the strength and erodibility characteristics are known to ensure the embankment has sufficient load capacity and is resistant to erosion by rain or flooding. To evaluate the borrow pit material's strength and erodibility characteristics, forty five-gallon buckets of soils were collected from fourteen borrow pits in South Carolina. Three buckets were collected at three different locations within each pit.

Fifteen sets of direct shear tests were performed on the selected fifteen buckets of soils. There are six buckets of soils from the upstate area, six buckets from the fall zone, and three buckets from the coastal area. They are all typical soils from each area. All of the specimens were remolded to 95% of the maximum dry density with moisture content between -1% to +2% of the optimum moisture content. Based on the results, soils from D2-Anderson-01, B-1 (MH), D2-Abbeville-01, B-1 (SM), D4-York-04, B-2 (SM), D2-Abbeville-01, B-3 (SM), D1-Richland-08, B-2 (ML) and D1-Richland-08, B-1 (ML) are acceptable for embankment construction usage. That is, they have high effective friction angles, which fall in the range specified by the South Carolina Department of Transportation (SCDOT) Geotechnical Design Manual 2008.

Nineteen pinhole tests were performed on all the soils which were suitable to run this test to evaluate the erodibility of the borrow pit soils. Based on the results, most of the soils were non-dispersive soils when compacted to 95% of the maximum dry density with

moisture content between -1% to +2% of the optimum moisture content. Soils from D2-Abbeville-01, B-1 (SM), D1-Lexington-05, B-3 (SC), D6-Berkeley-01, B-2 (SM), D3-Oconee-01, B-3 (CH) and D1-Richland-08, B-3 (CL-ML) are slightly dispersive soils, thus caution should be taken when they are used in embankment construction. For these cases, erosion control mats, retaining structure or specially designed filters may be needed to prevent erosion failure of the embankment.

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CHAPTER 1

INTRODUCTION

1.1 PROBLEM STATEMENT

Unlike other building materials, such as steel and concrete, the properties of soils used in construction cannot be strictly controlled through the production process. Soils by their very nature are heterogeneous materials. The conditions of their formation such as the parent material, the depositional process and environmental factors all affect soil properties and behavior.

The heterogeneous nature of soil requires testing to be performed at each site, and even then some properties may vary considerably in a relatively small area. As shown in Figure 1.1, depositional processes vary across the state of South Carolina, with two clearly defined zones: the coastal area which is rich in sands, and the upstate area which has abundant of residual deposits with silts and clays. Thus, many different soil types occur in the state.

An embankment is a structure built using soil as the construction material to provide a stable surface for a roadway and it is critical to know the properties of the material that are used to construct it. However, as an embankment uses a large amount of soil, it is important for the material to be obtained from local source. In a place where the in situ soil does not fulfill the requirements needed for the design, soil must be obtained from another sources; such sources are known as borrow pits. The South Carolina Department of Transportation (SCDOT) needs to know where suitable borrow pits exist for embankment construction; for this reason this project was undertaken.

1.2 OBJECTIVES AND SCOPE

This thesis is a part of a larger project to develop a geotechnical material database for embankment design and construction. This research focuses on the strength and erodibility of the borrow pit material. The major objectives are:

1. Select typical soils from different borrow pits that are suitable for direct shear tests and pinhole tests. Create a test matrix to carry out the experimental program.

2. Perform direct shear tests to determine the shear strength parameters (c' and ϕ') of the borrow pit soils.

3. Perform pinhole tests to determine the erodibility and dispersion characteristics of the borrow pit soils.

4. Based on the direct shear tests and pinhole tests results, determine the soils types or borrow pit which are suitable for embankment construction.

5. Identify soils that are less suitable for embankments and make recommendations for how to accommodate use of those soils.

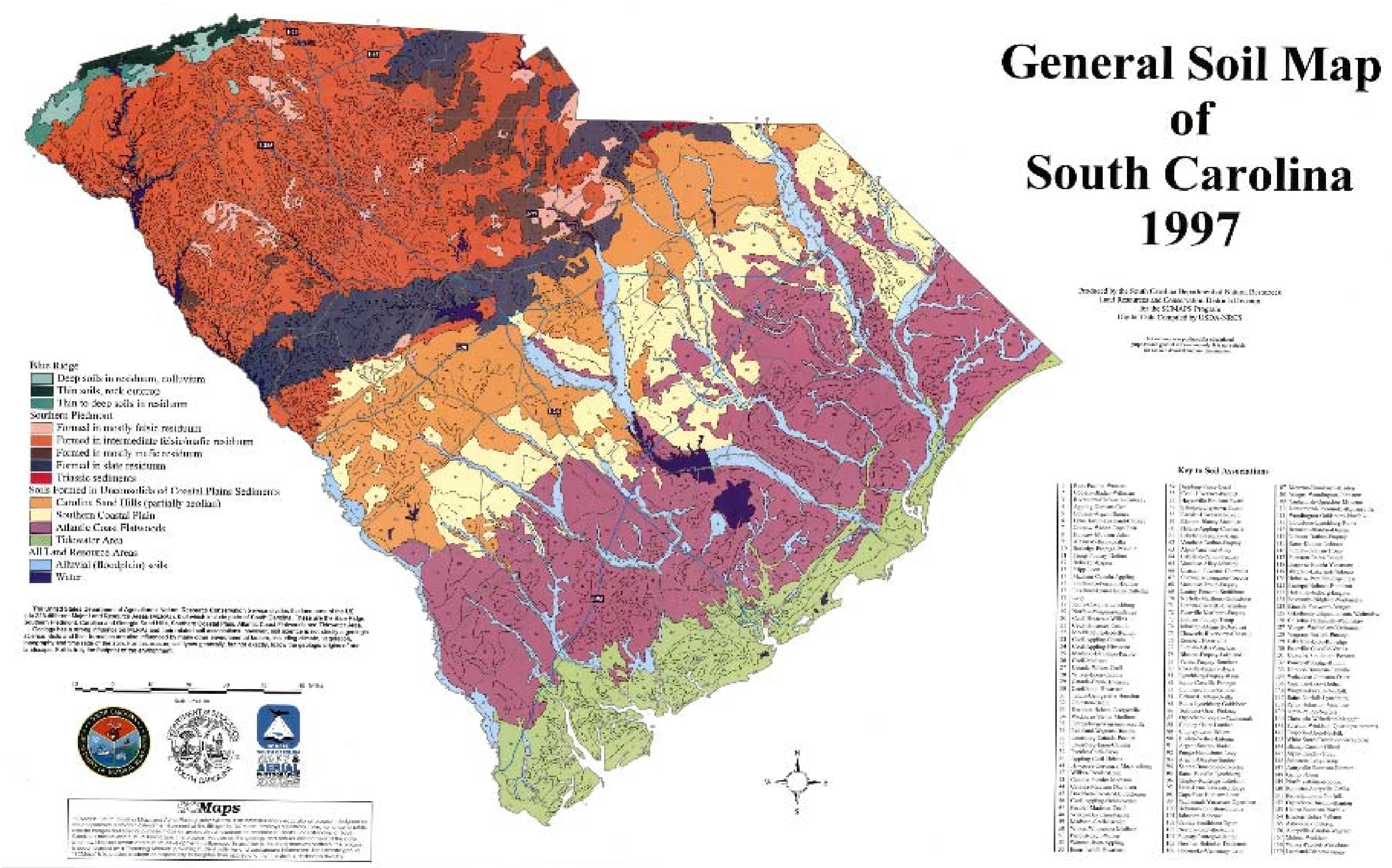


Figure 1.1 General Soil Map of South Carolina (1997) (South Carolina Department of Natural Resources)

1.3 THESIS LAYOUT

Chapter 1 is an introduction chapter, which contains the problem this thesis will investigate and the specific goals that are expected to be achieved in the project.

Chapter 2 is a background chapter. It gives the general information of embankment construction, the embankment soil material specifications given by the SCDOT, and the general borrow pit information in South Carolina. In this chapter, embankment failure modes due to slope stability and erodibility are discussed. It also focuses on the types of shear strength tests and erodibility tests and the factors that affect the results.

Chapter 3 introduces the experimental program. Both direct shear test and pinhole test plans are described. The testing procedures and test specimen preparation are discussed.

Chapter 4 is the test results and analysis chapter. The results of both direct shear tests and pinhole tests are presented and analyzed.

Chapter 5 gives a summary and conclusion of this research. It also points to further research that can be done based on these results.

CHAPTER 2

BACKGROUND

2.1 OVERVIEW OF EMBANKMENT CONSTRUCTION

An embankment, by its definition, is a bank, mound, or dike constructed to hold back water, or carry a roadway. This research is focused on the roadway embankment.

A roadway embankment is frequently constructed along a highway to carry traffic over a valley or low lying area and must be capable of supporting the load of the pavement structure and traffic as well as the weight of the embankment itself.

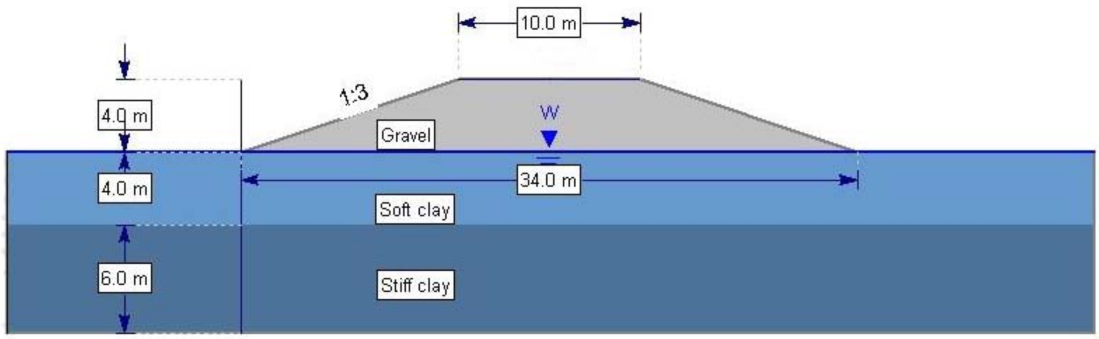


Figure 2.1 Typical Cross Section of a Roadway Embankment (http://geotech.maxit-cms.com/23276/)

Figure 2.1 is a typical cross section of a roadway embankment for a two-lane highway (http://geotech.maxit-cms.com/23276/). In this example, the embankment is

13.1 ft (4 m) high and has a width of 32.8 ft (10 m) at the top. The slope inclination is 1V:3H which gives a total width of the embankment of 111.5 ft (34 m). The foundation soil consists of a 13.1 ft (4 m) thick soft clay layer over a 19.7 ft (6 m) thick stiff clay layer.

According to AASHTO – "A Policy on the Geometric Design of Highways and Streets" 2004, for a typical roadway, 12 ft (3.7m) per lane is the default standard for 'traveled way'. In most states, the highway design manual recommends adding 2 ft (0.6 m) if there is an adjacent curb and adding various additional widths on curved ramps. The design of side slopes shall be governed by slope stability and traffic safety considerations. Side slopes shall not be steeper than 1V: 2H unless soil is retained by suitable soil retaining structures. Where the embankment is more than 9.8 ft (3 m) high and fill material consists of heavy clay or any problematic soil, the embankment stability shall be analyzed and ascertained for safe design.

To make sure the embankment has the adequate ability to support the combined load of pavement structure, traffic and the weight of itself, the South Carolina Department of Transportation Construction Manual 200.2.6 gives the following factors to consider: (1) strength of the soil material under the embankment, (2) engineering characteristics of the embankment material, (3) proper construction of benches and transitions, (4) proper placement of the embankment material in lifts, (5) control of moisture content near optimum during compaction, and (6) compaction of each lift of embankment material to target density, (South Carolina Department of Transportation Construction Manual 200.2.6).

2.2 EMBANKMENT SOIL MATERIAL AND BORROW PIT IN SOUTH CAROLINA

Based on the different soil formation processes, South Carolina soils can be divided into two areas: the Upstate Area and the Coastal Area, as shown in Figure 2.2.

The Upstate Area includes the following counties: Abbeville, Anderson, Cherokee, Chester, Edgefield, Fairfield, Greenville, Greenwood, Lancaster, Laurens, McCormick, Newberry, Oconee, Pickens, Saluda, Spartanburg, Union, and York. Soils for use in embankments and as subgrade collected from this area are indicated as Group A.

The Coastal Area includes the following counties: Aiken, Allendale, Bamberg, Barnwell, Beaufort, Berkeley, Calhoun, Charleston, Chesterfield, Clarendon, Colleton, Darlington, Dillon, Dorchester, Florence, Georgetown, Hampton, Horry, Jasper, Kershaw, Lee, Lexington, Marion, Marlboro, Orangeburg, Richland, Sumter, and Williamsburg. The soils collected from this area are indicated as Group B.

Groups A and B are shown graphically on a South Carolina map in Figure 2.2. Brief geologic descriptions of the surface soils in Groups A and B are provided below.

2.2.1 Group A

This group is located northwest of the “Fall Line” in the Blue Ridge and Piedmont physiographic geologic units. The Blue Ridge unit surface soils typically consist of a residual soil profile consisting of clayey soils near the surface where weathering is more advanced, underlain by sandy silts and silty sands (SCDOT Geotechnical Design Manual 2008 7.12.1).

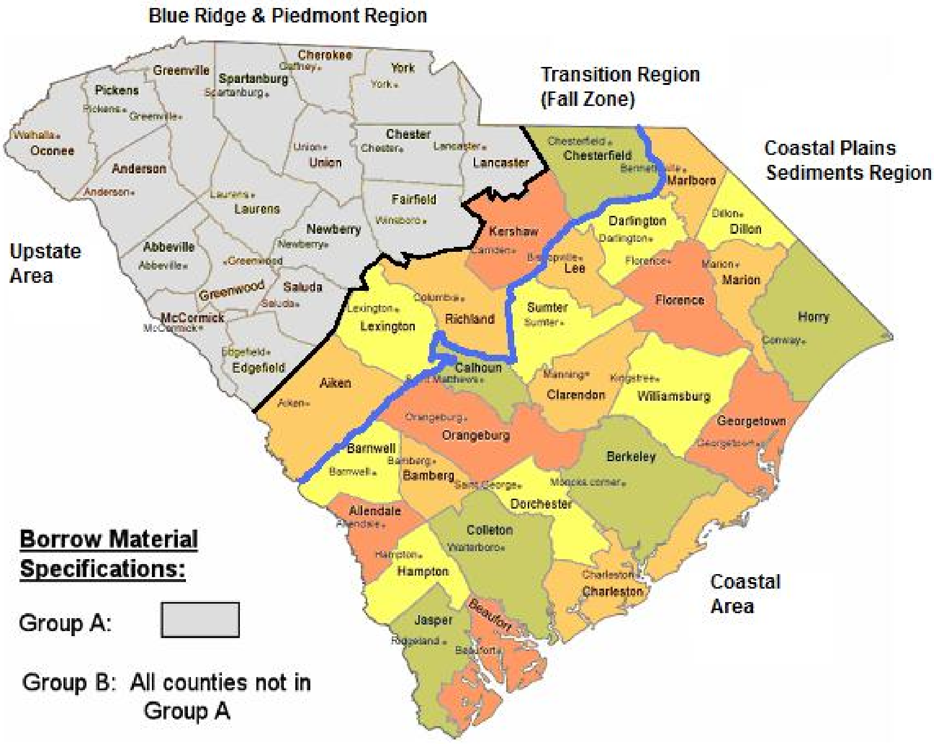


Figure 2.2 Borrow Material Specifications by County

There may be colluvial material from old landslides on the slopes. The Piedmont unit has a residual soil profile that typically consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The residual soil profile exists in areas not disturbed by erosion or the activities of man.

SCDOT experience with borrow material typically found in Group A are Piedmont residual soils. These borrow material are typically described as micaceous clayey silts and micaceous sandy silts, clays, and silty soils in partially drained conditions. These soils may have USCS classifications of either ML or MH and typically have liquid limits (LL) greater than 30.

Published laboratory shear strength testing results for Piedmont residual soils (Sabatini, 2002, Appendix A, page A-40) indicate an average effective friction angle of

35.2° with a ± 1 standard deviation range of 29.9° < ϕ' < 40.5°. A conservative lower bound of 27.3° is also indicated.

2.2.2 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 that consist of unconsolidated sand, clay, gravel, marl, cemented sands, and limestone (SCDOT Geotechnical Design Manual 2008 7.12.1).

SCDOT experience with borrow material typically found in Group B are Coastal Plain soils that are typically uniform fine sands that are sometimes difficult to compact and behave similar to silts. When these soils are encountered, caution should be used in selecting effective soil shear strength friction angles since values typically range from 28° < ϕ' < 32°. (SCDOT Geotechnical Design Manual 2008 7.12.1-7.12.3)

2.3 EMBANKMENT FAILURE MODES

Failures of road embankments can generally be grouped into three classifications: large-scale erosion, small-scale erosion, and structural (insufficient shear strength).

2.3.1 Large-scale Erosion Failures

Large-scale erosion failures from the uncontrolled flow of water over and adjacent to the embankment are due to the erosive action of water on the embankment slopes. These failures happen when there is heavy rain and subsequent flooding near the road. When this kind of failure happens, the soil of the embankment will be washed away. The roadway may be totally destroyed by flooding (Environmental Fact Sheet, WD-DB-4, www.des.nh.gov, 2011).

Case 1:

Heavy rains on May 25, 2011 caused an embankment failure under the railroad tracks that run parallel to Plymouth Road in Ann Arbor, MI, closing the road and buckling the tracks. A 45 ft (13.7 m) long section of the embankment was washed out from beneath the tracks, leaving them unsupported, as shown in Figure 2.3. About 2,000 cubic yards of soil and trees slid across five lanes of Plymouth Road (annarbor.com, 2011).

(a)

(b)

Figure 2.3 Photographs Showing Railroad Embankment Collapse in Ann Arbor (from annarbor.com, photographs courtesy of Angela J. Cesere): (a) the tracks were unsupported and (b) a large amount of soil was washed away

Case 2:

In November 1998, the monsoon season brought unusual heavy rain to the east coast of Sabah, Malaysia. The continuous rain had caused flooding to some areas of the state. Kota Kinabalu – Tambunan road was also affected. A massive road embankment failure had occurred at km 25.5, which caused the road to be cut off from the entire road network. A typical section of failed embankment is shown in Figure 2.4. No vehicle could pass through this section of the road. As a consequence, an emergency work was called in to carry out the immediate repair to the road (Chin Tat Hing et al., 2006).

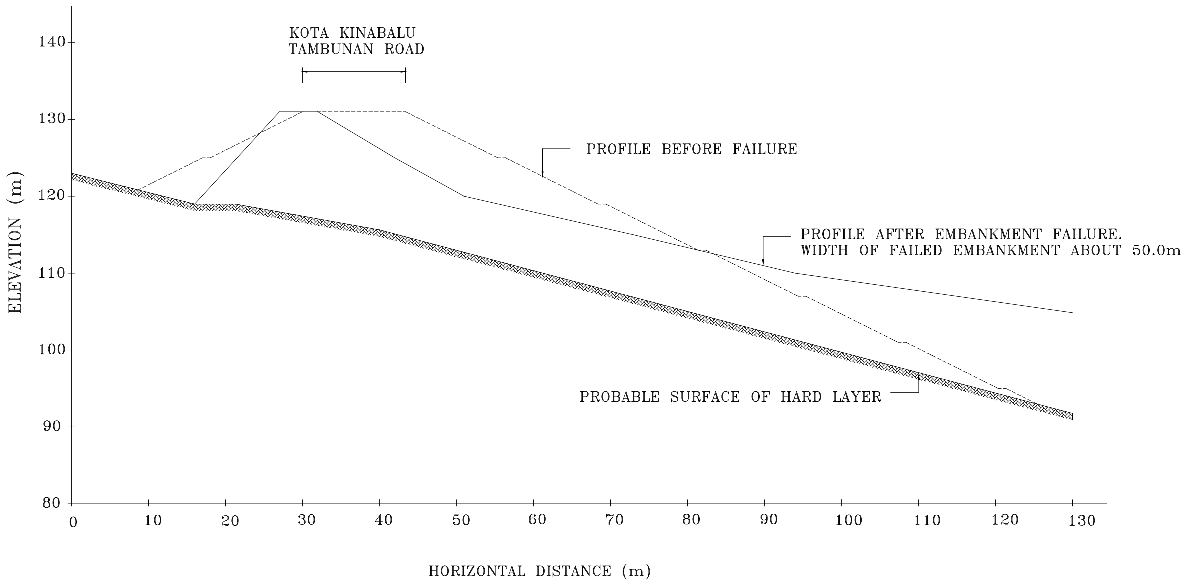


Figure 2.4 Typical Section of Failed Embankment of Kota Kinabalu – Tambunan Road in Malaysia (Chin Tat Hing et al., 2006)

2.3.2 Small-scale Erosion Failures

Small-scale erosion failures are the result of seepage. Most embankments exhibit some seepage when there are heavy rains. However, this seepage must be controlled in velocity and quantity. Seepage, if uncontrolled, can erode fine soil material from the downstream slope to form a pipe or cavity often leading to a failure of the embankment. Seepage can also cause slope failures by saturating the slope material, thereby weakening the properties of the soil and its stability (Environmental Fact Sheet, WD-DB-4, www.des.nh.gov, 2011).

The main failure mode results in cracks due to differential settlements within or beneath the embankment.

Case 3:

Figure 2.5 shows cracks along the Pentalia road in southwest Cyprus (Hadjigeorgiou et al., 2006). The development of cracks in the road was attributed to higher than usual rainfalls and snowfalls. On the upper bank of the road there is talus material which is composed of chalky and marl chalky fragments in a calcareous sandy silt matrix. The talus material is approximately 13.1 ft (4 m) thick and overlays a clay melange. The melange is a thick deposit of clasts characterised by high plasticity and low shear resistance. It is thus possible that during the intense rainfall period, a perched water table developed along the interface between the two materials causing the instability of the embankment.

(a)

(b)

Figure 2.5 Cracks along the Pentalia Road in the Southwest Cyprus (Hadjigeorgiou et al., 2006): (a) Cracks along the Embankment and (b) Cracks on the Pavement Road Due to Differential Settlements

2.3.3 Structural Failures (Insufficient Shear Strength)

Structural failures involve the separation (rupture) of the embankment material and/or its foundation. This type of failure often happens in a high embankment when soft clay is present. When the gravity of the embankment exceeds the shear strength capacity of the clay layer, failure will occur (Mills and McGinn, 2010).

Case 4:

In July 2006, a large embankment failure occurred during construction of a four-lane divided highway leading to the Canada–U.S. border crossing in St. Stephen, New Brunswick, Canada (Mills and McGinn, 2010), as shown in Figure 2.6. The highway embankment was approximately 40.4 ft (12.3 m) in height, just short of the design height of 46 ft (14 m), when it failed. The cause of the failure was attributed to the rapid rate of construction and the intensity of loading on low-strength foundation soils, consisting of up to 49.2 ft (15 m) of soft marine clay.

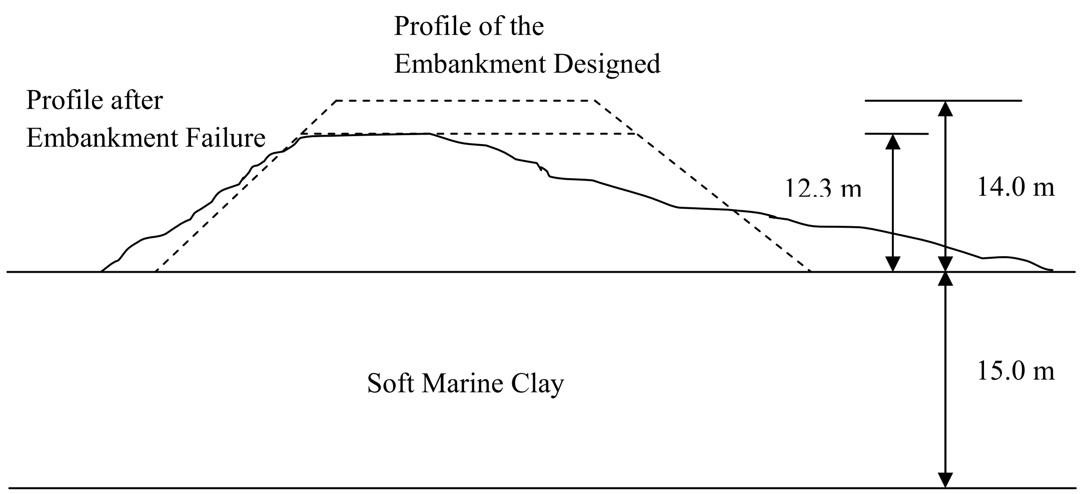


Figure 2.6 Road Embankment Failure in St. Stephen, New Brunswick, Canada (Mills and McGinn, 2010)

2.4 SHEAR STRENGTH TESTS AND FACTORS

The shear strength of a soil is defined as the maximum (or ultimate) shear stress the soil can withstand. The bearing capacity of shallow or deep foundations, slope stability, retaining wall design and, indirectly, pavement design are all affected by the shear strength of soil.

2.4.1 Mohr-Coulomb Failure Criterion

According to Mohr-Coulomb failure criterion, the functional relationship between normal stress and shear stress on a failure plane can be expressed in the following form:

$$\tau_f = c + \sigma \tan\phi \qquad (2\text{-}2)$$

where

c = cohesion

ϕ = angle of internal friction

σ = normal stress on the failure plane

τ_f = shear strength

In saturated soil, the total normal stress at a point is the sum of the effective stress (σ') and pore water pressure (u), or

$$\sigma = \sigma' + u \qquad (2\text{-}3)$$

The effective stress σ' is carried by the soil solids. The Mohr-Coulomb failure criterion, expressed in terms of effective stress, will be of the form

$$\tau_f = c' + \sigma' \tan\phi' \qquad (2\text{-}4)$$

where

c' = cohesion and ϕ' = friction angle, based on effective stress.

Thus, Eq. (2-2) and (2-4) are expressions of shear strength based on total stress and effective stress, respectively. The value of c' for cohesionless soils, such as sand and inorganic silt, is assumed to be 0; however, the presence of clayey fines will contribute to some cohesion. For normally consolidated clays, c' can be approximated as 0. Overconsolidated clays have values of c' that are greater than 0. The angle of friction, ϕ', is sometimes referred to as the drained angle of friction.

2.4.2 Factors that Affect the Shear Strength of Sand and Clay

Since sand is a frictional material, those factors that increase the frictional resistance should lead to increases in the angle of internal friction. The factors that influence ϕ are: void ratio or relative density; particle shape; particle size; grain size distribution; particle surface roughness; water content; intermediate principal stress and pre-consolidation (Holtz and Kovacs, 1981), as summarized in Table 2.1.

Table 2.1 Summary of Factors Affecting ϕ (Holtz and Kovacs, 1981)

Factor	Effect
Void ratio e	$e\uparrow$, $\phi\downarrow$
Angularity A	$A\uparrow$, $\phi\uparrow$
Grain size distribution	$C_u\uparrow$, $\phi\uparrow$
Surface roughness R	$R\uparrow$, $\phi\uparrow$
Moisture content w	$w\uparrow$, $\phi\downarrow$ slightly
Particle size S	No effect (with constant e)
Intermediate principal stress	$\phi_{ps} \geq \phi_{tx}$
Overconsolidation or prestress	Little effect

Typical values of ϕ' for some granular soils are given in Table 2.2 which shows that sand with smaller void ratio and more angular has a larger friction angle ϕ'.

Table 2.2 Typical Values of Drained Angle of Friction for Sands and Silts (Das, 2006)

Soil type	ϕ' (deg)
Sand: Rounded grains	
Loose	27-30
Medium	30-35
Dense	35-38
Sand: Angular grains	
Loose	30-35
Medium	35-40
Dense	40-45
Gravel with some sand	34-48
Silts	26-35

Correlations between ϕ' and dry density, relative density, and soil classifications are shown in Figure 2.7. This figure is useful for estimating the frictional characteristics of granular material.

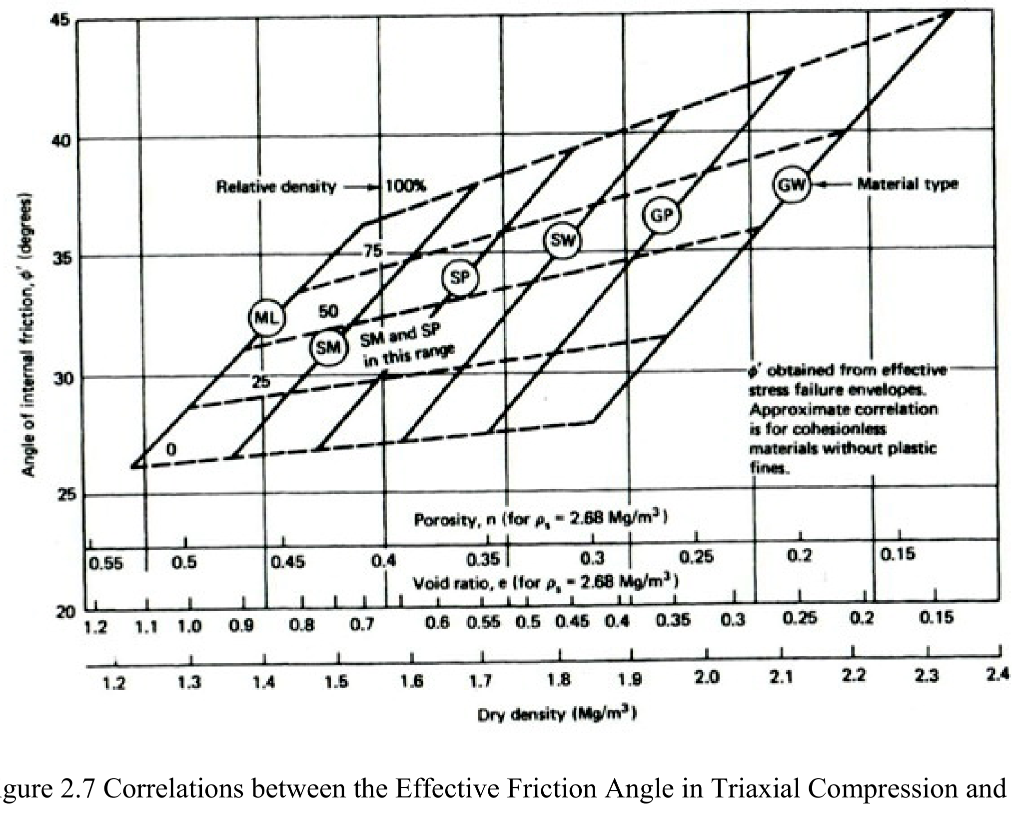


Figure 2.7 Correlations between the Effective Friction Angle in Triaxial Compression and the Dry Density, Relative Density, and Soil Classification (after U. S. Navy, 1971)

For clay, the drained angle of friction, ϕ', generally decreases with the plasticity index (PI) of soil. A best fit nonlinear correlation between sin ϕ' and PI has been developed (Mitchell and Soga 2005) as:

$$\sin \phi' = -0.1 \ln(PI) + 0.8 \tag{2-5}$$

Skempton (1964) provided the results of the variation of the residual angle of friction, ϕ'_r, of a number of clayey soils with the clay-size fraction (≤2 μm) present. Table 2.3 is a summary of these results. (Note that the residual angle of friction, ϕ'_r, is calculated from the residual shear strength of clay; and the term "residual" shear strength of clay is similar to the term "ultimate" shear strength of sand.)

Table 2.3 Values of the Residual Angle of Friction of some Clayey Soils (Skempton, 1964)

Soil	Clay-size fraction (%)	Residual friction angle, ϕ'_r, (deg)
Selset	17.7	29.8
Wiener Tegel	22.8	25.1
Jackfield	35.4	19.1
Oxford Clay	41.9	16.3
Jari	46.5	18.6
London Clay	54.9	16.3
Walton's Wood	67	13.2
Weser-Elbe	63.2	9.3
Little Belt	77.2	11.2
Biotite	100	7.5

2.4.3 Methods to Determine the Shear Strength

The shear strength can be determined by laboratory or in situ tests. In situ methods such as the vane shear test, cone penetrometer test or standard penetration test avoid some of the problems of disturbance associated with the extraction of soil samples from the ground, however, these methods only determine the shear strength indirectly through correlations with laboratory results. Laboratory tests, such as direct shear test and triaxial test, on the other hand, yield the shear strength directly. In addition, valuable information about the stress-strain behavior and the development of pore pressures during shear can often be obtained.

2.4.3.1 Direct Shear Test

The direct shear test is the oldest and simplest form of shear test arrangement. Depending on the equipment, the shear test can be either stress controlled or strain controlled. In stress controlled tests, the shear force is applied in equal increments until the specimen fails. The failure occurs along the horizontal plane of split of the shear box. In strain-controlled tests, a constant rate of shear displacement is applied to one-half of the box by a motor that acts through gears (Das, B. M. 2006).

The direct shear test is simple to perform, but it has some inherent shortcomings. The reliability of the results may be questioned because the soil is not allowed to fail along the weakest plane but is forced to fail along the horizontal plane of split of the shear box. Also, the shear stress distribution over the shear surface of the specimen is not uniform. Despite these shortcomings, the direct shear test is the simplest and most economical for a dry or saturated sandy soil (Das, B. M. 2006).

For cohesive soils, it is very difficult to control the drainage using direct shear test. The hydraulic conductivity of cohesive soils is very small compared with that of sand. When a normal load is applied to a clay soil specimen, a sufficient length of time must elapse for full consolidation – that is, for dissipation of excess pore water pressure. For this reason, the shearing load must be applied very slowly. The test may last from two to five days (Das, B. M. 2006).

2.4.3.2 Triaxial Shear Test

The triaxial shear test is one of the most reliable methods available for determining shear strength parameters. It is used widely for research and conventional testing (Das, B. M. 2006). The triaxial test is more complicated than the direct shear test but also more versatile. Drainage can be well-controlled, and there is no rotation of principal stresses. Stress concentrations still exist, but they are significantly less than in the direct shear test. Also, the failure plane is not predetermined.

There are three standard types of triaxial tests:

1. Consolidated-drained test (CD test)
2. Consolidated-undrained test (CU test)
3. Unconsolidated-undrained test (UU test)

CU test (Consolidated-undrained test) strengths are used for stability problems where the soils have first become fully consolidated and are at equilibrium with the existing stress system. Then, for some reason, additional stresses are applied quickly, with no drainage occurring. Practical examples include rapid drawdown of embankment dams and rapid construction of an embankment on a natural slope. Moreover, it is possible to measure the induced pore pressures in a CU test, calculate the effective stresses in the specimen, and obtain the effective stress strength parameters. CD test (Consolidated-drained test) conditions are the most critical for the long-term steady seepage case for embankment dams and the long-term stability of excavations or slopes in both soft and stiff clays. Effective shear strength parameters needed for long term drained analysis can be obtained from either a CU test with measured pore pressure or a CD test.

2.4.3.3 The Relation of Parameters Obtained from Direct Shear Test and Triaxial Shear Test

The direct shear test (DS) is a plane strain test. The effective friction angles obtained from direct shear test and triaxial test have a relationship as follows:

$$\phi'_{p.s.} = \phi'_{T.X.} + \varepsilon \tag{2-6}$$

where

$\phi'_{\mathrm{p.s.}}$ is the effective friction angle obtained from plane strain test (DS),

$\phi'_{\mathrm{T.X.}}$ is the effective friction angle obtained from triaxial compression test (TXC),

ε is about 10% $\phi'_{\mathrm{T.X.}}$ for contractive material (e.g. loose sand) and is about 4-6% $\phi'_{\mathrm{T.X.}}$ for dilative material (e.g. dense sand).

2.4.3.4 Stress-deformation and Strength Characteristics of Sand and Clay under Direct Shear Test

Figure 2.8 shows a typical plot of shear stress and change in the height of the specimen against shear displacement for dry loose and dense sands (Normal Consolidated Clay and Over Consolidated Clay).

In loose sand (or NC clay), the shear strength increases with shear displacement until a failure shear strength is reached. After that, the shear resistance remains approximately constant for any further increase in the shear displacement (Das, 2006). The height of specimen is decreasing during the shear process. That means that the volume of specimen is decreasing. The specimen behaves as contractive material.

In dense sand, the shear strength increases with shear displacement until it reaches a failure stress which is called the peak shear strength. After failure stress is attained, the shear stress gradually decreases as shear displacement increase until it finally reaches a constant value called the ultimate shear strength (Das, 2006). The height of specimen will decrease at the beginning of shear process, and then it will increase. The volume of specimen will increase at the end of the shear test. The specimen behaves as dilative material.

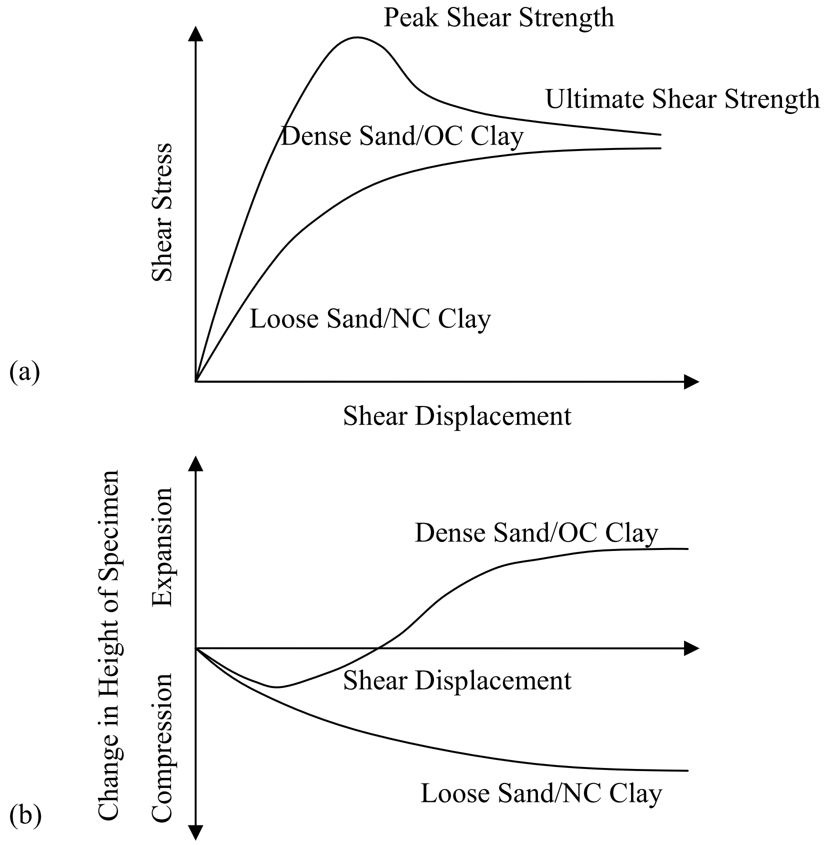


Figure 2.8 General Soil Behavior during Direct Shear: (a) Shear Stress vs. Horizontal Displacement and (b) Change in Height vs. Horizontal Displacement (Das, 2006)

2.5 ERODIBILITY TESTS AND FACTORS

The erodibility of a soil is defined by its resistance to two energy sources: the impact of raindrops on the soil surface, and the shearing action of runoff between clods in grooves or rills. It can be described in terms of behavior in two aspects: 1) the rate of erosion when a given hydraulic shear stress is applied to the soil and 2) the ease of initiating erosion in the soil.

The first studies on the eroibility of material were done by Hjulström in canals (Hjulström, 1935). Figure 2.9 is known as Hjulström's diagram. It shows that there are three sectors, depending on water velocity and the diameter of soil particles.

Hjulström's diagram can be used to obtain important information about a soil's erodibility. First of all, the material most easily dislodged by runoff has a texture close to that of fine sand (100 microns). More clayey material is cohesive and the clayey particles stick together. Coarser particles are heavier and require higher fluid velocity to be moved. Second, as long as the flow is slow (≤ 25 cm/sec), it cannot erode. At last, fine clay particles are easily transported, even at low speeds. When particles are coarser than fine sand, there is a shorter distance from the erosion site to sedimentation site.

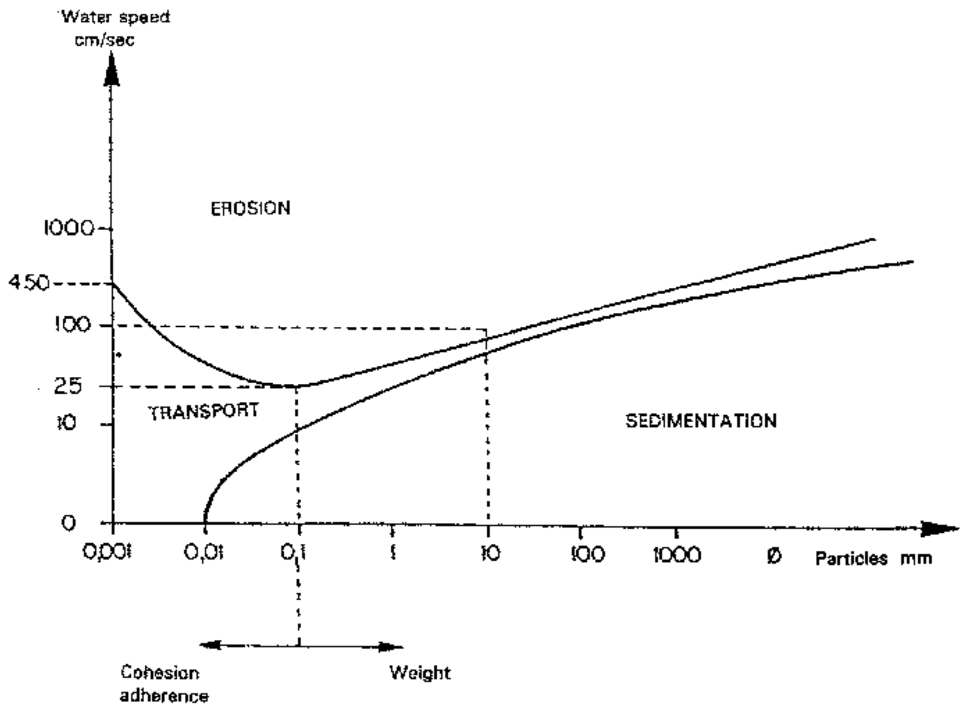


Figure 2.9 Hjulström's diagram (Hjulström, 1935)

The erodibility of a soil is related to soil gradation, soil texture and structure, compaction effort and compaction water content. Poorly graded soil tends to be a more dispersive soil compared with a well graded soil, given that void ratio tends to be higher with poorly graded soils. Compaction water content also plays a major role in determining erodibility. The rate of erosion of the clay soil tested at optimum conditions for the same compaction effort was observed to be 100 times less than the sandy soil. The rate of erosion of either soil was also observed to change more than 10 times by changes in the compaction water content (Hanson and Hunt, 2006). Erodibility decreases with increases in dry density.

There are several tests that have been devised to identify dispersive soils: pinhole test, crumb test, double hydrometer test and cylinder dispersion test.

2.5.1 Pinhole Test

The pinhole test was described by Sherard, et al. (1976). Water flows through a 1 mm diameter hole punched in a specimen of compacted clay. The water emerging from dispersive clay carries a suspension of colloidal particles. There are several categories of dispersive and non-dispersive soils, depending on the cloudiness of the water. ASTM test method D 4647 – 06 states that the test provides a direct qualitative measurement of the dispersibility or deflocculation and consequent erodibility of clay soils. It claims that the comparison of results from the pinhole test and other indirect tests indicates that the results of the pinhole test have the best correlation with the erosional performance of clays in nature.

2.5.2 Crumb Test

The crumb test described by Emerson (1954, 1964) is a quick test for dispersive soils and is similar to the traditional puddle clay soaking test in which a hand rolled ball of clay is immersed in water for 48 hours (Bishop 1946). A few crumbs of soil 6-10 mm in diameter are placed in a large volume of distilled water or sodium hydroxide solution. It is noted whether or not a colloidal cloud extends around the crumbs. There are several categories of dispersive and non-dispersive properties, depending on the cloudiness caused by colloids in suspension.

2.5.3 Double Hydrometer Test

The U.S. Soil Conservation Service developed a dispersion test procedure based on the hydrometer test for the particle size analysis of soils. In this double hydrometer test, two identical samples undergo the standard hydrometer sedimentation test. In one test dispersant is used; in the other test, dispersant and mechanical stirring are omitted. The dispersion is the ratio of the percentage of clay particles with dispersant, and is expressed as a percentage. Decker and Dunnigan (1976) stated that 85% of soils which show 30% or more dispersion are subject to dispersive erosion.

2.5.4 Cylinder Dispersion Test

The cylinder dispersion test described by Atkinson, et al. (1990) is a simple test to examine erosion and dispersion of soils from the surfaces of cracks into water. This test is an extension of the crumb test and it was designed to examine the behavior of soils at zero effective stresses by submerging a saturated sample in water. When saturated soil samples are submerged in water, one of three basic characteristic types of behavior will be observed, which indicates three different categories: type N: Non-dispersive, cohesionless; type C: Non-dispersive, cohesive and type D: Dispersive.

CHAPTER 3

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

Based on the soils information provided in Section 2.2, there are two areas in South Carolina: Upstate Area and Coastal Area. However, it is more reasonable to divide the state of South Carolina into three regions based on the different soil formation processes. From Figure 1.1, in the upstate area, soils of Blue Ridge are found in the northwest of the state. The soils from most of this area are Piedmont soils. So this region is named as Blue Ridge and Piedmont Region.

Considering the soil variability within the Coastal Area, this area was divided into two regions for the purpose of this research: Transition Region (which also known as Fall Zone) and Coastal Plains Sediments Region, as shown in Figure 2.2. The soils types in the Fall Zone region are more variable. This region contains the following counties: Aiken, Chesterfield, Kershaw, Lexington, and Richland.

For this project, forty (40) five-gallon buckets of soils were collected from fourteen (14) borrow pit in South Carolina. These borrow pit are shown in Figure 3.1.

The place marks with “P”, “T”, and “C” are located in the Blue Ridge and Piedmont Region, Transition Region, and Coastal Plains Sediments Region respectively.

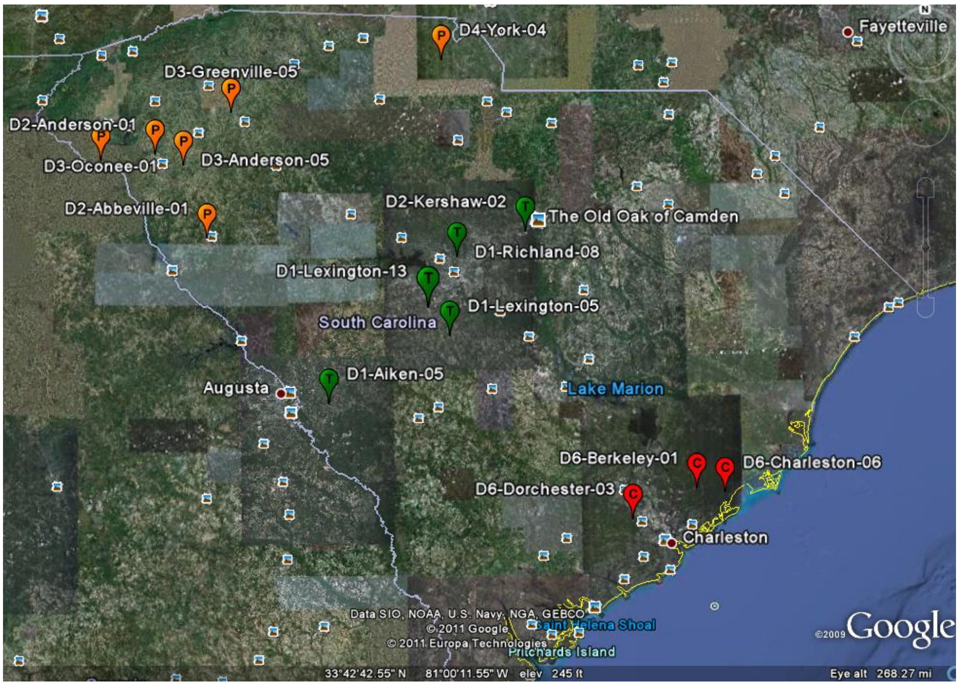


Figure 3.1 Borrow Pit Locations

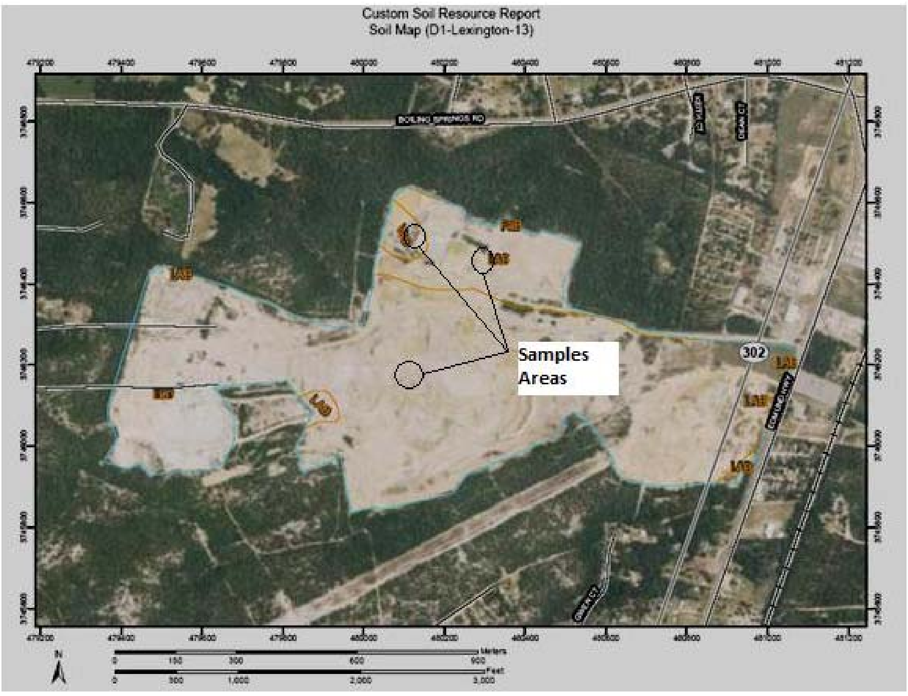


Figure 3.2 Web Soil Survey Results for D1-Lexington-13

In most cases, three buckets samples were collected at three different locations within each pit based on the Web Soil Survey (http://websoilsurvey.nrcs.usda.gov/app/WebSoilSurvey.aspx) results. Sampling points were selected based upon location, prevalent soil types according to Web Soil Survey, and pit size. For example, samples from D1-Lexington-13 were collected on July 7, 2008. Figure 3.2 shows that there are three different soil regions within this pit. One bucket sample was collected from each one of these different regions.

3.2 SOIL PROPERTIES

3.2.1 Group A Soils Properties

The following Group A soils were sampled: D4-York-04, D2-Anderson-01, D3-Anderson-05, D2-Abbeville-01, D3-Greenville-05 and D3-Oconee-01. The properties of these soils were determined by Pierce et al. 2011 and are listed in Table 3.1 and Table 3.2.

In Table 3.1, the classification information is provided by using two criteria - USCS and AASHTO. The last two columns are Atterberg Limits. There are two columns of fines content. The second one is used to determine if the soil is suitable for pinhole dispersion test. If a soil with less than 12% fraction finer than 0.0002 in. (0.005 mm) and with a plasticity index less than or equal to 4, this soil is not suitable for pinhole test.

Table 3.2 contains the information of soils' specific gravity and the results of standard proctor compaction tests.

Table 3.1 Group A Soil Properties (for Classification) (Pierce et al. 2011)

Pit	Bucket No.	Classification		Fines Content (%)		Atterberg Limits	
		USCS	AASHTO	Finer than 0.075 mm	Finer than 0.005 mm	Liquid Limit, LL	Plasticity Index, PI
D4-York-04	1	SM	A-2-4	21	6	NP	NP
	2	SM	A-4	40	9	NP	NP
	3	SM	A-4	37	9	NP	NP
D2-Anderson-01	1	MH	A-7-5	53	34	54	18
	2	SM	A-7-5	46	29	55	24
	3	ML	A-7-6	50	27	42	13
D3-Anderson-05	1	MH	A-7-5	53	33	55	13
	2	SM	A-7-6	46	20	41	16
	3	SM	A-5	44	18	48	10
D2-Abbeville-01	1	SM	A-7-6	36	9	45	17
	2	MH	A-7-5	68	28	58	21
	3	SM	A-2-4	35	4	38	1
D3-Greenville-05	1	SM	A-5	48	8	44	5
	2	MH	A-7-5	60	29	53	22
	3	SM	A-4	41	5	NP	NP
D3-Oconee-01	1	CH	A-7-6	58	34	52	27
	2	MH	A-7-5	53	34	72	23
	3	CH	A-7-6	57	34	65	42

*NP means "Non-plastic".

Table 3.2 Group A Soil Properties (for Compaction) (Pierce et al. 2011)

Pit	Bucket No.	Specific Gravity, G_s	Standard Proctor Compaction			
			Maximum Dry Density, $\gamma_{d\ max}$ (pcf)	Optimum Water Content, w_{opt} (%)	Optimum Degree of Saturation S_{opt} (%)	Optimum Void Ratio, e_{opt}
D4-York-04	1	2.59	119.5	11.0	81	0.35
	2	2.63	114.0	15.5	92	0.44
	3	2.61	116.0	13.5	87	0.40
D2-Anderson-01	1	2.74	97.0	24.0	86	0.77
	2	2.69	104.0	22.0	96	0.62
	3	2.71	103.5	19.0	81	0.63
D3-Anderson-05	1	2.79	99.0	23.0	85	0.76
	2	2.8	106.5	20.0	87	0.64
	3	2.8	99.5	23.0	85	0.76
D2-Abbeville-01	1	2.67	109.5	15.0	77	0.52
	2	2.83	91.0	28.0	84	0.94
	3	2.64	110.0	14.0	74	0.50
D3-Greenville-05	1	2.76	98.0	23.0	84	0.76
	2	2.74	100.0	23.0	89	0.71
	3	2.75	93.0	20.0	65	0.85
D3-Oconee-01	1	2.75	102.5	23.5	96	0.67
	2	2.63	102.5	22.5	98	0.60
	3	2.62	106.0	19.0	92	0.54

3.2.2 Group B Soils Properties

The following Group B soils were sampled: D1-Lexington-05, D1-Lexington-13, D1-Richland-08, D1-Kershaw-02, D1-Aiken-05, D6-Charleston-06, D6-Berkeley-01 and D6-Dorchester-03. The properties of these soils were determined by Pierce et al. 2011 and are listed in Table 3.3 and Table 3.4.

The classification, fines content and Atterberg Limits are listed in Table 3.3. And the specific gravity and standard proctor compaction tests results are listed in Table 3.4.

Table 3.3 Group B Soil Properties (for Classification) (Pierce et al. 2011)

Pit	Bucket No.	Classification		Fines Content (%)		Atterberg Limits	
		USCS	AASHTO	Finer than 0.075 mm	Finer than 0.005 mm	Liquid Limit, LL	Plasticity Index, PI
D1-Lexington-05	1	SC	A-2-6	18	9	31	12
	2	SM	A-2-4	21	13	NP	NP
	3	SC	A-2-7	28	17	49	28
D1-Lexington-13	1	SW-SM	A-1-b	6	4	NP	NP
	2	SW-SM	A-1-b	7	2	NP	NP
	3	SW-SM	A-1-b	8	4	NP	NP
D1-Richland-08	1	ML	A-4	75	10	31	7
	2	ML	A-4	88	8	35	1
	3	CL-ML	A-4	52	17	17	4
D1-Kershaw-02	1	NOT TESTED					
	2	SW-SM	A-2-4	12	3	NP	NP
	3	SM	A-2-4	29	17	NP	NP
D1-Aiken-05	1	SP	A-3	3	2	NP	NP
	2	SP-SM	A-3	6	3	NP	NP
	3	SP-SM	A-2-4	12	8	NP	NP
D6-Charleston-06	1	SM	A-2-4	20	7	NP	NP
	2	SP-SM	A-3	7	3	NP	NP
	3	SP-SM	A-3	8	4	NP	NP
D6-Berkeley-01	1	SP-SM	A-2-4	11	7	NP	NP
	2	SM	A-2-4	32	19	NP	NP
	3	SP-SM	A-3	10	5	NP	NP
D6-Dorchester-03	1	SP-SM	A-2-4	12	5	NP	NP

*NP means “Non-plastic”.

Table 3.4 Group B Soil Properties (for Compaction) (Pierce et al. 2011)

Pit	Bucket No.	Specific Gravity, G_s	Standard Proctor Compaction			
			Maximum Dry Density, $\gamma_{d\,max}$ (pcf)	Optimum Water Content, w_{opt} (%)	Optimum Degree of Saturation, S_{opt} (%)	Optimum Void Ratio, e_{opt}
D1-Lexington-05	1	2.76	122.0	13.0	87	0.41
	2	2.67	122.0	11.0	80	0.37
	3	2.74	116.0	12.5	72	0.47
D1-Lexington-13	1	2.69	122.0	10.5	75	0.38
	2	2.71	122.0	10.5	74	0.39
	3	2.73	122.0	10.5	72	0.40
D1-Richland-08	1	2.86	106.0	17.0	71	0.68
	2	2.89	109.5	15.0	66	0.65
	3	2.83	109.5	15.0	68	0.61
D1-Kershaw-02	1	NOT TESTED				
	2	2.74	101.0	14.0	55	0.69
	3	2.75	111.0	14.0	71	0.55
D1-Aiken-05	1	2.71	100.0	18.0	71	0.69
	2	2.66	102.5	16.0	68	0.62
	3	2.67	111.5	12.0	65	0.50
D6-Charleston-06	1	2.53	106.0	16.5	85	0.49
	2	2.61	102.0	16.0	70	0.60
	3	2.64	108.5	12.5	64	0.52
D6-Berkeley-01	1	2.57	105.0	12.5	61	0.52
	2	2.54	113.5	12.5	79	0.40
	3	2.67	104.0	17.5	77	0.61
D6-Dorchester-03	1	2.71	107.0	12.0	56	0.58

3.3 DIRECT SHEAR TEST

In this project, direct shear tests were performed on fifteen soils collected from borrow pit in South Carolina. Most soil types we collected are sandy silts (ML), silty sand (SM) or sand (SW-SM). So the direct shear tests were selected to determine the shear strength. The results of these tests provide the range of the shear strength (c', ϕ') from different borrow pits, and can be used to draw conclusions about which kinds of soils are suitable for embankment construction.

The test plan is listed in Table 3.5.

In the Blue Ridge and Piedmont Region, the typical soil types are clayey soils, sandy silts, and silty sand. Six samples were chosen for testing, which contain soil types CH, MH, and SM.

In the Transition Region (which also known as Fall Zone), the soil types are more variable. Six samples were chosen for testing, which contain soil types ML, SC, SP-SM, and SW-SM. They almost covered all typical soil types found in this area.

The typical soil in the Coastal Plains Sediments Region is poorly graded sand. Three samples were chosen for testing. Two of them are SM. The other one is SP-SM.

Table 3.5 Test Plan

Region	Pit	Bucket No.	Classification		Direct Shear Test	Pinhole Dispersion Test
			USCS	AASHTO		
Blue Ridge and Piedmont	D4-York-04	1	SM	A-2-4		
		2	SM	A-4	YES	
		3	SM	A-4		
	D2-Anderson-01	1	MH	A-7-5	YES	YES
		2	SM	A-7-5		YES
		3	ML	A-7-6		YES
	D3-Anderson-05	1	MH	A-7-5	YES	YES
		2	SM	A-7-6		YES
		3	SM	A-5		YES
	D2-Abbeville-01	1	SM	A-7-6	YES	YES
		2	MH	A-7-5		YES
		3	SM	A-2-4	YES	
	D3-Greenville-05	1	SM	A-5		YES
		2	MH	A-7-5		YES
		3	SM	A-4		
	D3-Oconee-01	1	CH	A-7-6	YES	YES
		2	MH	A-7-5		YES
		3	CH	A-7-6		YES
Transition (Fall Zone)	D1-Lexington-05	1	SC	A-2-6	YES	
		2	SM	A-2-4		
		3	SC	A-2-7		YES
	D1-Lexington-13	1	SW-SM	A-1-b	YES	
		2	SW-SM	A-1-b		
		3	SW-SM	A-1-b		
	D1-Richland-08	1	ML	A-4	YES	YES
		2	ML	A-4	YES	YES
		3	CL-ML	A-4		YES
	D1-Kershaw-02	1	NOT TESTED			
		2	SW-SM	A-2-4	YES	
		3	SM	A-2-4		YES
	D1-Aiken-05	1	SP	A-3		
		2	SP-SM	A-3	YES	
		3	SP-SM	A-2-4		
Coastal Plains Sediments	D6-Charleston-06	1	SM	A-2-4	YES	
		2	SP-SM	A-3		
		3	SP-SM	A-3		
	D6-Berkeley-01	1	SP-SM	A-2-4		
		2	SM	A-2-4	YES	YES
		3	SP-SM	A-3		
	D6-Dorchester-03	1	SP-SM	A-2-4	YES	

3.3.1 Direct Shear Test Procedure

The test procedure was according to ASTM D3080-04, Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

Direct shear tests were performed on a Wykeham Farrance apparatus as shown in Figure 3.3. This device is strain controlled. The shear rate can range from 0.000004 to 0.047 in./min (0.0001 to 1.2 mm/min). The shear box has a square shape with dimensions of 2.5 in. × 2.5 in. (63.5 mm × 63.5 mm). The height of the shear box is 2.0 in. (50.8 mm).

Figure 3.3 Direct Shear Test Apparatus

There are three dials in this device which are used to measure load ring deformation, horizontal displacement and vertical displacement, respectively. They are shown in Figure 3.4. The accuracy of these three dials are 0.00008 in. (0.002 mm), 0.001 in. (0.0254 mm), and 0.0001 in.(0.00254 mm), respectively.

(a) (b) (c)

Figure 3.4 Photographs of Dials in the Direct Shear Test Device: (a) Load Ring Dial; (b) Horizontal Displacement Dial and (c) Vertical Displacement Dial

3.3.1.1 Shear Rate Selection

The shear rate should be slow enough so that no excess pore pressure would exist at failure. The following equation was used as a guide to determine the estimated minimum time required from the start of the test to failure:

$$t_f = 50\, t_{50} \tag{3-1}$$

where

t_f = total estimated elapsed time to failure, min,

t_{50} = time required for the specimen to achieve 50 percent consolidation under the specified normal stress (or increments thereof), min.

Then the appropriate displacement rate can be determined from the following equation:

$$d_r = d_f / t_f \tag{3-2}$$

where

d_r = displacement rate (in./min, mm/min),

d_f = estimated horizontal displacement at failure (in., mm),

t_f = total estimated elapsed time to failure, min.

In this research, most of the soils are sand or silt. The consolidation process is very quick. According to the changing of the vertical displacement during shear, most soils will achieve 50 percent consolidation within 1 minute. That is, we can assume t_{50} = 1 min. Then t_f = 50 min. We can use d_f = 0.5 in. (12 mm), because most of the soils are normally consolidated coarse-grained. So:

$$\mathrm{d_r} = \frac{\mathrm{d_f}}{\mathrm{t_f}} = \frac{12\ \mathrm{mm}}{50\ \mathrm{min}} = 0.24\ \mathrm{mm/min}$$

The actual displacement rate is 0.164 mm/min for most of the tests except D3-Oconee-01, Bucket 1. For the soil D3-Oconee-01, Bucket 1, the soil type is CH. It will take longer to achieve 50 percent consolidation. According to the vertical displacement record, using t_{50} = 2 min is reasonable. Then the displacement rate should be 0.12 mm/min. The actual displacement rate is 0.113 mm/min for this soil.

3.3.1.2 Normal Load Determination

The vertical loads of these tests are 44 lbf (196.2 N), 88 lbf (392.4 N), and 132 lbf (588.6 N) respectively. That is, the initial normal stresses are 7 psi (48.3 kPa), 14 psi (96.5 kPa), and 21 psi (144.8 kPa).

Based on the Standard Proctor Test results (ASTM D 698 - 07e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3))), the maximum dry density range is from 93 pcf (14.6

kN/m^3) to 122 pcf (19.2 kN/m^3). The range of the total unit weight of the soils at the optimum moisture content is from 112 pcf (17.6 kN/m^3) to 138 pcf (21.7 kN/m^3).

By using the average value γ = 125 pcf (19.6 kN/m^3), the stress range caused by the vertical load 44 lbf (196.2 N), 88 lbf (392.4 N), and 132 lbf (588.6 N) represents a depth range of 8.2 ft (2.5 m) to 24.3 ft (7.4 m). These depths are reasonable for most embankments.

3.3.2 Direct Shear Test Specimen Preparation

All specimens tested were remolded using the compaction procedures described as follows:

(1) Collect a sample of soil that is sufficient to produce at least three specimens. A mass of 2.2 lbs (1 kg) was normally collected.

(2) Measure the moisture content of the soil according to ASTM D 2216 – 05, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.

(3) Adjust the moisture by adding distilled water or air drying to optimum moisture content. The actual moisture content should be -1 % to +2 % of the optimum moisture content (SCDOT Geo_manual, 2008). The optimum moisture content is determined according to ASTM D 698 - 07e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

(4) Calculate the moist weight of soil needed based on the height of the specimen as 1.2 in (30.5 mm). This was determined by calculating the density of the soil first, by assuming the final dry density to be 95% maximum dry density. The maximum dry density is determined according to ASTM D 698 - 07e1, Standard Test Methods for

Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

(5) Weigh an exact amount of moist soil based on the calculation of step (4).

(6) Divide the soil into three equal amounts. They should be compacted in three layers, and each layer should be 0.4 in. (10.2 mm).

(7) Assemble and secure the shear box. Compact each layer of soil until the desired depth is achieved by using a circular tamping rod. The top surface of each layer shall be scarified prior to the addition of material for the next layer.

(8) Continue placing and compacting soil until the full height of 1.2 in. (30.5 mm) is achieved.

3.4 PINHOLE DISPERSION TEST

In this research, pinhole dispersion tests were conducted to evaluate the erodibility of soils collected from different borrow pit. This test is used to evaluate the erodibility of soils with high fines content by flowing water through a small hole that is drilled through the compacted specimen.

According to ASTM D4647-06, Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test, this method is not applicable to soils with less than 12% fraction finer than 0.005 mm and with a plasticity index less than or equal to 4. Such soils generally have low resistance to erosion regardless of dispersive characteristics. There are nineteen (19) soils with more than 12% fraction finer than 0.005 mm or with a plasticity index more than 4. That is, nineteen (19) sets of pinhole tests were conducted in this research. A test plan was made on the soils sampled from different borrow pits, shown in Table 3.5.

3.4.1 Pinhole Test Procedure

The procedure was followed according to ASTM D4647-06, Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Test. In this research, Method A was used to evaluate the erodibility of soils. Method A of the pinhole test requires the evaluation of cloudiness of effluent, final size of the pinhole, and computation of flow rates through the pinhole in order to classify the dispersive characteristics of the soil. It will classify soils into six categories of dispersiveness as: dispersive (D1, D2), slight to moderately dispersive (ND4, ND3), and nondispersive (ND2, ND1).

The apparatus used in the Pinhole Test is shown in Figure 3.5 and Figure 3.6. The test chamber has a unique clamping ring for holding the stainless steel mold to the base while compacting the sample. Included with the chamber are screens, base stand, constant head reservoir, tubing, connections, pipet and a needle for drilling the pinhole. The end cap has a pilot hole for drilling the 1.0 mm (diameter) hole through the sample. The depth of the hole equals the height of the sample.

(a)

(b)

Figure 3.5 Chamber of the Pinhole Dispersion Test Apparatus: (a)Mold, Screens, Nipple, and Needle; (b) The Assembled Chamber and Cylinders

Figure 3.6 The Assembled Pinhole Dispersion Test Apparatus

3.4.2 Pinhole Test Specimen Preparation and Method A Procedure

All the specimens used in the tests were compacted to approximately 95% of maximum standard dry unit weight at the optimum moisture content. The procedures for specimen preparation are as follows:

(1) Collect a sample of soil that is sufficient to produce one specimen. A mass of 0.11 lbs (50 g) was normally collected.

(2) Measure the moisture content of the soil according to ASTM D 2216 – 05, Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass.

(3) Adjust the moisture by adding distilled water or air drying to optimum moisture content. The actual moisture content should be -1 % to +2 % of the optimum moisture

content (SCDOT Geo_manual, 2008). The optimum moisture content is determined according to ASTM D 698 - 07e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

(4) Calculate the moist weight of soil needed based on the diameter and the length of the specimen as 1.3 in. (33 mm) and 1.0 in. (25 mm), respectively. This means the soil volume will be 1.33 in.^{3} (21382 mm^3). Next, calculate the density of the soil, by assuming the final dry density to be 95% maximum dry density. The maximum dry density is determined according to ASTM D 698 - 07e1, Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

(5) Weigh an exact amount of moist soil based on the calculation of step (4).

(6) Assemble and secure the pinhole chamber.

(7) Divide the soil into five equal amounts. Compact each amount of soil layer by layer. The height of each layer is supposed to be 0.2 in. (5 mm). Compact each layer of soil into the pinhole test cylinder on top of the coarse sand and wire screen, which have been previously placed in the cylinder, until the desired depth is achieved. The top surface of each layer shall be scarified prior to the addition of material for the next layer.

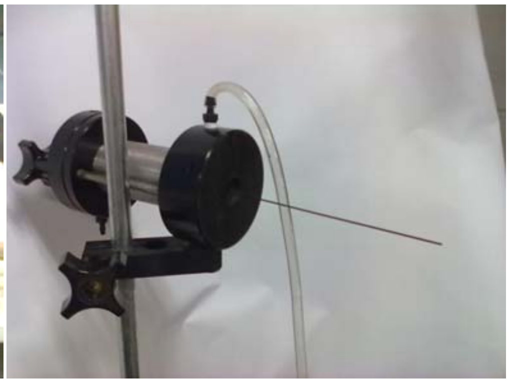
(8) Continue placing and compacting soil until the entire specimen is compacted.

(9) Carefully place the wire screen on top of the specimen and fill the remaining void in the top of the test cylinder with coarse sand.

(10) Assemble the top plate and connect the head (distilled water) source. Place assembled apparatus in horizontal position as shown in Figure 3.7.

(a) (b)

(c) (d)



(e) (f)

Figure 3.7 The Pinhole Test Specimen Preparation Procedure: (a) Weigh an Amount of Soil; (b) Coarse Sand Used as Filter Layer at the Bottom of Sample; (c) Compact Soil to 95% Maximum Dry Density; (d) Control the Height of Each Layer; (e) Assemble the Chamber; (f) Punch a Hole Use the 1-mm Diameter Pin

(11) Open the screw using a screw driver. Insert the 1.0-mm diameter wire punch into the centering guide and punch or force it through the soil specimen. Force the punch in a continuous motion through the soil specimen; it then should penetrate into the underlying sand. Close the screw.

(12) Start the test by introducing distilled water into the apparatus so that a hydraulic head at the level of the pinhole is 50 mm (2 in.).

(13) Record the time at start of test (or start the stop watch).

(14) With an appropriate graduated cylinder, begin measuring the quantity of effluent flow as it emerges from the specimen. If no flow occurs when the test is started, stop the test, dismantle the top of the apparatus, and repunch the hole.

(15) Observe the cloudiness of the effluent for each measured discharge by looking both through the side of the cylinder and vertically through the column of fluid in the cylinder. Record the cloudiness of the effluent in the cylinder as very dark, dark, moderately dark, slightly dark, barely visible, or completely clear (ASTM D 4647-06).

(16) Continue the test under the 2 in. (50 mm) head for 5 min. If, at the end of 5 min, the effluent is very dark and flow rates have gradually increased to 1.0 to 1.4 mL/s, the test is complete (ASTM D 4647-06).

(17) Dismantle the apparatus and extrude the soil specimen from the cylinder. Break or cut open the specimen, transversely and longitudinally, and measure the size of the hole by comparing against the needle used to punch the hole (ASTM D 4647-06).

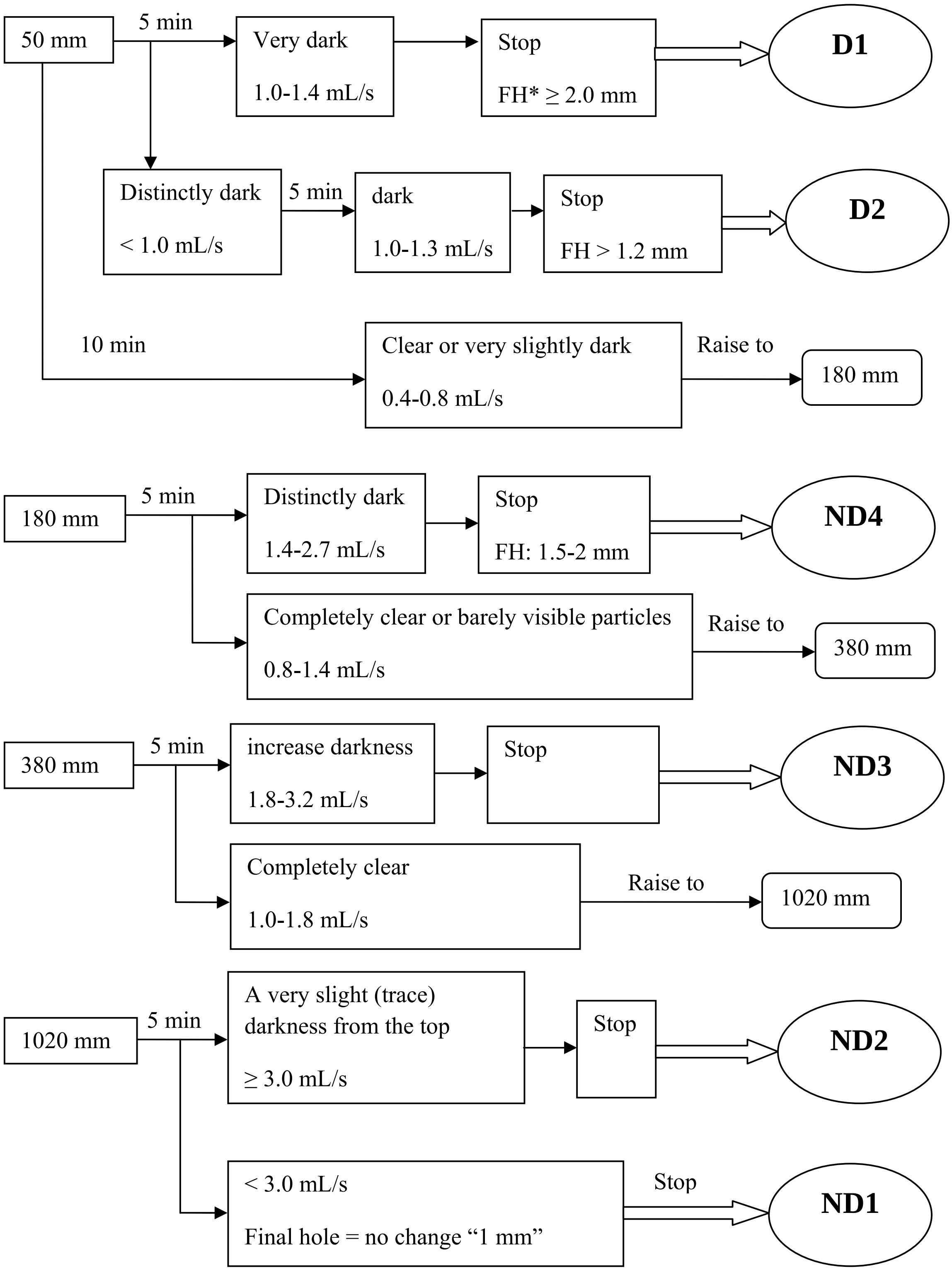
(18) If the final hole size is greater than twice the needle punch diameter, classify the soil as highly dispersive, D1. Otherwise, the flow rate and the hole size are inconsistent and the test should be done again (ASTM D 4647-06).

(19) If the effluent from the 2 in. (50 mm) head is distinctly dark and the flow rate does not exceed 1.0 mL/s at the end of 5 min., continue the test an additional 5 min for a total of 10 min. At the end of 10 min, if the effluent is still dark, stop the test and determine the hole size. Classify the soil as dispersive D2 if the final flow rate is 1.0 to 1.3 mL/s (ASTM D 4647-06).

(20) If the effluent under the 2 in. (50 mm) head is clear or is very slightly dark at the end of 10 min. and the flow rate is 0.40 to 0.80 mL/s, raise the head to 7 in. (180 mm) as shown in Figure 3.8. Under the 7 in. head, if the effluent is distinctly dark and the rate of flow has increased rapidly to 1.4 to 2.7 mL/s, stop the test and examine the hole diameter. If the hole diameter is equal to or greater than 1.5 to 2 needle diameters, classify the soil as slightly to moderately dispersive, ND4 (ASTM D 4647-06).

(21) If the flow under the 7 in. head continues to flow completely clear or has particles that are barely visible after 5 min and the flow rate is 0.8 to 1.4 mL/s, raise the head to 15 in. (380 mm). After 5 min. under the 15 in. head, if the flow has increased darkness or the flow rate has increased to 1.8 to 3.2 mL/s, stop the test and classfy the soil as slightly dispersive, ND3 (ASTM D 4647-06).

(22) If, after 5 min., the flow under the 15 in. head is completely clear and the flow rate is 1.0 to 1.8 mL/s, raise the head to 40 in. (1020 mm). If the flow under 40 in. head after 5 min. has a very slight (trace) darkness from the top of the cylinder or the flow rate exceeds 3.0 mL/s, classify the soil as nondispersive, ND2. Otherwise classify the soil as nondispersive, ND1. The flow rate for ND1 soils under 40 in. head will generally be less than 3.0 mL/s and the size of the hole at the end of the test should not be measurably larger than the needle punch (ASTM D 4647-06).



*FH means final hole diameter at the end of the test.

Figure 3.8 The Flow Chart of the Pinhole Test

CHAPTER 4

TEST RESULTS AND ANALYSIS

4.1 DIRECT SHEAR TEST

4.1.1 Test Results

The experimental conditions and the results of the fifteen (15) groups of direct shear tests are summarized in Table 4.1, Table 4.2 and Table 4.3.

In Table 4.1 and Table 4.2, pit name and bucket number, classification (USCS and AASHTO), specific gravity G_s, specimen number, optimum moisture content w_{opt}, the initial moisture content of the specimen w_{actual}, maximum dry density $\gamma_{d\ max}$, 95% maximum dry density, the dry density of the specimen $\gamma_{d\ actual}$, and the initial void ratio e_0 are listed in columns 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, respectively.

According to SCDOT Goetechnical Design Manual 2008, the samples used in the lab tests should be remolded to 95% of Standard Proctor with moisture -1 % to +2 % of the optimum moisture content. From Table 4.1 and 4.2, most of the specimens met this standard, except the third specimen of D2-Anderson-01; B-1

The initial actual moisture content was measured by using the leftovers after making three specimens. The actual unit weight was calculated by using the follow equation:

$$\gamma = \frac{W}{V} \tag{4-1}$$

where W is the weight of the moist soil and V is the volume of the specimen.

Then $\gamma_{d\ actual}$ was found from the following equation:

$$\gamma_{d\ actual} = \frac{\gamma}{1+w_{actual}} \tag{4-2}$$

where w_{actual} is the initial moisture content of the specimen.

The initial void ratio is calculated by using the following equation:

$$e = \frac{G_s \gamma_w}{\gamma_{d\ actual}} - 1 \tag{4-3}$$

where G_s is the specific gravity and γ_w is the unit weight of water (62.4 pcf).

In Table 4.3, pit name and bucket number, classification (USCS and AASHTO), specific gravity, coarse grain shape description, coefficient of uniformity C_u, coefficient of curvature C_c, fines content (percent of particles less than 0.075 mm), liquid limit (LL) and plasticity index (PI) are listed in column 1, 2, 3, 4, 5, 6, 7, 8 and 9, respectively. From column 10 to column 13 are strength indices – effective cohesion c′ and effective friction angle ϕ′. For those soils which are non-plastic, two methods were used to determine the strength indices. Values in column 10 and 11 were obtained from the least square fitting method (first degree polynomial).

For each direct shear test, the relationship of shear stress and normal stress at failure, the relationship of shear stress and horizontal displacement, and the relationship of horizontal displacement and vertical displacement are shown in Figures 4.1 to 4.15. In this project, the shear stress is defined by using peak shear stress.

In the figure of shear stress versus normal stress, there are three points which represent three specimens being sheared under three different normal stresses 7 psi (48.3

kPa), 14 psi (96.5 kPa), and 21 psi (144.8 kPa). These three normal stresses represent three different depths of an embankment, which was discussed in Section 3.3.1.2.

Table 4.1 Direct Shear Test Condition (Blue Ridge and Piedmont Region)

Pit & Bucket No.	Classification		Specific Gravity	Speci -men No.	w% (opt)	w% (actu -al)	$\gamma_{d\,max}$ (pcf)	95% $\gamma_{d\,max}$ (pcf)	$\gamma_{d\,actual}$ (pcf)	e_0
	USCS	AASHTO								
D4-York-04; B-2	SM	A-4	2.63	1	15.4	14.4	114	108.3	109.7	0.50
			2.63	2	15.4	14.4	114	108.3	110.0	0.49
			2.63	3	15.4	14.9	114	108.3	109.5	0.50
D2-Anderson-01; B-1	MH	A-7-5	2.74	1	24.1	25.0	97	92.0	91.8	0.86
			2.74	2	24.1	26.0	97	92.0	91.0	0.88
			2.74	3	24.1	27.1	97	92.0	90.3	0.89
D3-Anderson-05; B-1	MH	A-7-5	2.79	1	23.0	22.5	99	94.1	94.9	0.83
			2.79	2	23.0	22.6	99	94.1	94.9	0.84
			2.79	3	23.0	21.7	99	94.1	95.5	0.82
D2-Abbeville-01; B-1	SM	A-7-6	2.67	1	15.0	15.7	110	104.1	103.9	0.60
			2.67	2	15.0	16.5	110	104.1	103.2	0.62
			2.67	3	15.0	15.9	110	104.1	103.8	0.61
D2-Abbeville-01; B-3	SM	A-2-4	2.64	1	14.0	14.3	110	104.5	104.6	0.57
			2.64	2	14.0	14.6	110	104.5	104.5	0.58
			2.64	3	14.0	14.3	110	104.5	104.8	0.57
D3-Oconee-01; B-1	CH	A-7-6	2.75	1	23.5	24.1	103	97.4	97.3	0.76
			2.75	2	23.5	24.2	103	97.4	97.3	0.76
			2.75	3	23.5	22.7	103	97.4	98.4	0.74

Table 4.2 Direct Shear Test Condition (Transition and Coastal Plains Sediments Region)

Pit & Bucket No.	Classification		Specific Gravity	Speci -men No.	w% (opt)	w% (actu -al)	$\gamma_{d\ max}$ (pcf)	95% $\gamma_{d\ max}$ (pcf)	γ_{d} actual (pcf)	e_0
	US CS	AASHTO								
D1-Lexington-05; B-1	SC	A-2-6	2.76	1	13	12.7	122	115.9	116.7	0.48
			2.76	2	13	12.3	122	115.9	117.1	0.47
			2.76	3	13	12.3	122	115.9	117.0	0.47
D1-Lexington-13; B-1	SP-SM	A-1-b	2.69	1	10.5	10	122	115.9	116.9	0.44
			2.69	2	10.5	9.9	122	115.9	117.0	0.44
			2.69	3	10.5	10.7	122	115.9	116.2	0.44
D1-Richland-08; B-1	ML	A-4	2.86	1	17	17.3	106	100.7	101.0	0.77
			2.86	2	17	17.3	106	100.7	101.1	0.77
			2.86	3	17	16.9	106	100.7	101.3	0.76
D1-Richland-08; B-2	ML	A-4	2.89	1	14.7	15.7	110	104.0	103.7	0.74
			2.89	2	14.7	15.6	110	104.0	103.7	0.74
			2.89	3	14.7	15.3	110	104.0	104.0	0.73
D1-Kershaw-02; B-2	SW -SM	A-2-4	2.74	1	14	13.2	101	96.0	97.3	0.76
			2.74	2	14	13	101	96.0	97.3	0.76
			2.74	3	14	13.1	101	96.0	97.3	0.76
D1-Aiken-05; B-2	SP-SM	A-3	2.66	1	16	16.7	102	97.2	97.2	0.71
			2.66	2	16	16.5	102	97.2	97.4	0.70
			2.66	3	16	16.8	102	97.2	97.0	0.71
D6-Charleston-06; B-1	SM	A-2-4	2.53	1	16.5	17.4	106	100.7	100.5	0.57
			2.53	2	16.5	17.8	106	100.7	100.1	0.58
			2.53	3	16.5	17.3	106	100.7	100.7	0.57
D6-Berkeley-01; B-2	SM	A-2-4	2.54	1	12.3	12.7	113	107.7	107.9	0.47
			2.54	2	12.3	12.8	113	107.7	107.7	0.47
			2.54	3	12.3	12.5	113	107.7	108.1	0.47
D6-Dorchester-03; B-1	SP-SM	A-2-4	2.71	1	12	13.9	107	101.7	100.6	0.68
			2.71	2	12	13.8	107	101.7	100.8	0.68
			2.71	3	12	13.8	107	101.7	100.7	0.68

Table 4.3 Direct Shear Test Results and Related Soil Indices

Pit & Bucket No.	Classification		Specific Gravity	Coarse Grain Shape Description	C_u	C_c	Fines Content	LL	PI	c'	ϕ'
	USCS	AASHTO					%	%	%	psi	deg
D4-York-04, B-2	SM	A-4	2.63	Subangular	60.0	4.400	40	NP	NP	8.84	34.2
D2-Anderson-01, B-1	MH	A-7-5	2.74	-	-	-	53	54	18	6.42	39.8
D3-Anderson-05, B-1	MH	A-7-5	2.79	-	-	-	53	55	13	8.51	30.1
D2-Abbeville-01, B-1	SM	A-7-6	2.67	Subangular	72.1	1.801	36	45	17	6.79	37.2
D2-Abbeville-01, B-3	SM	A-2-4	2.64	Subrounded	17.9	1.607	35	38	1	9.73	32.2
D3-Oconee-01, B-1	CH	A-7-6	2.75	-	-	-	58	52	27	4.26	31.7
D1-Lexington-05, B-1	SC	A-2-6	2.76	Subangular to Subrounded	35.4	4.833	18	31	12	3.60	40.2
D1-Lexington-13, B-1	SP-SM	A-1-b	2.69	Subangular	5.1	1.269	6	NP	NP	3.31	46.2
D1-Richland-08, B-1	ML	A-4	2.86	-	-	-	75	31	7	6.68	32.5
D1-Richland-08, B-2	ML	A-4	2.89	-	-	-	88	35	1	8.06	33.5
D1-Kershaw-02, B-2	SW-SM	A-2-4	2.74	Subangular to Rounded	6.1	1.608	12	NP	NP	0.85	46.3
D1-Aiken-05, B-2	SP-SM	A-3	2.66	Angular	2.2	0.992	6	NP	NP	3.59	36.9
D6-Charleston-06, B-1	SM	A-2-4	2.53	Subangular to Subrounded	11.0	5.114	20	NP	NP	7.57	37
D6-Berkeley-01, B-2	SM	A-2-4	2.54	Subrounded to Rounded	51.4	10.15 9	32	NP	NP	8.76	30.6
D6-Dorchester-03, B-1	SP-SM	A-2-4	2.71	Subrounded	4.8	2.961	12	NP	NP	2.08	37.9

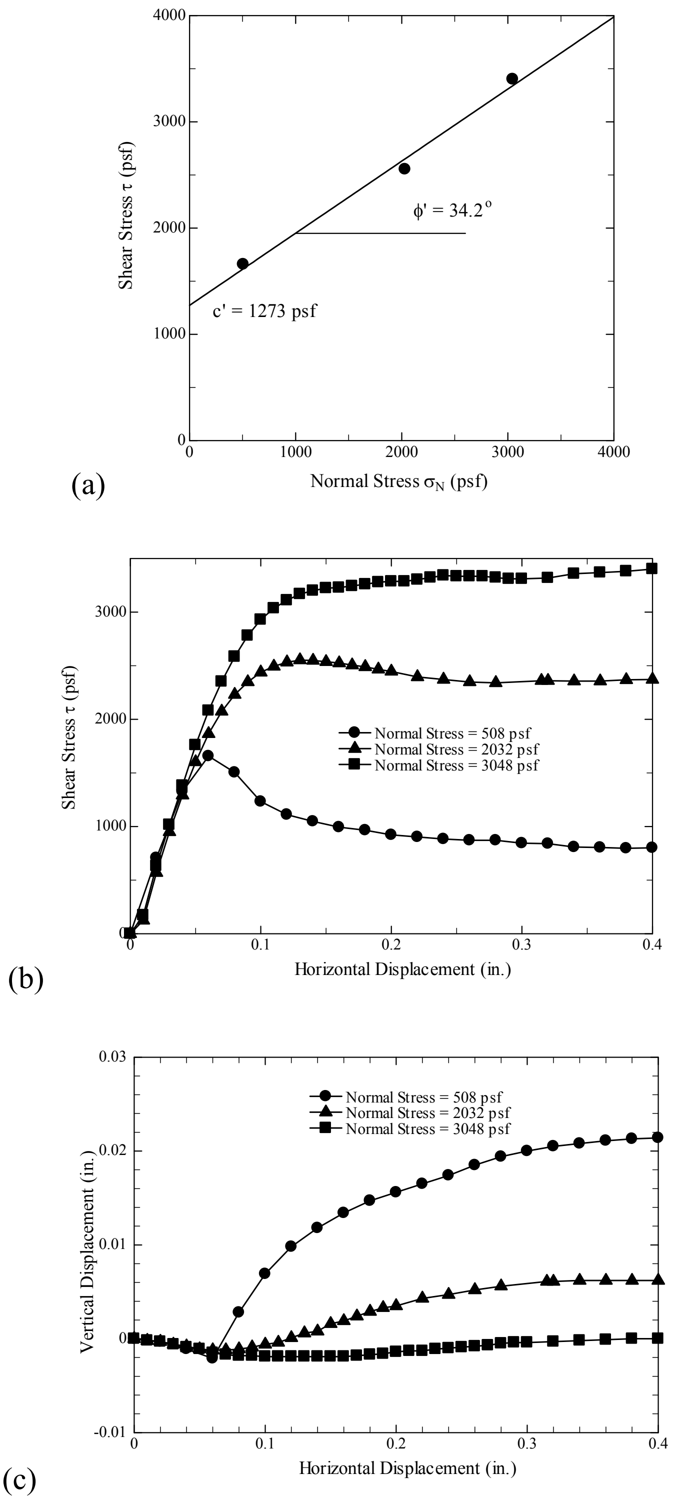


Figure 4.1 Results for D4-York-04, Bucket 2: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

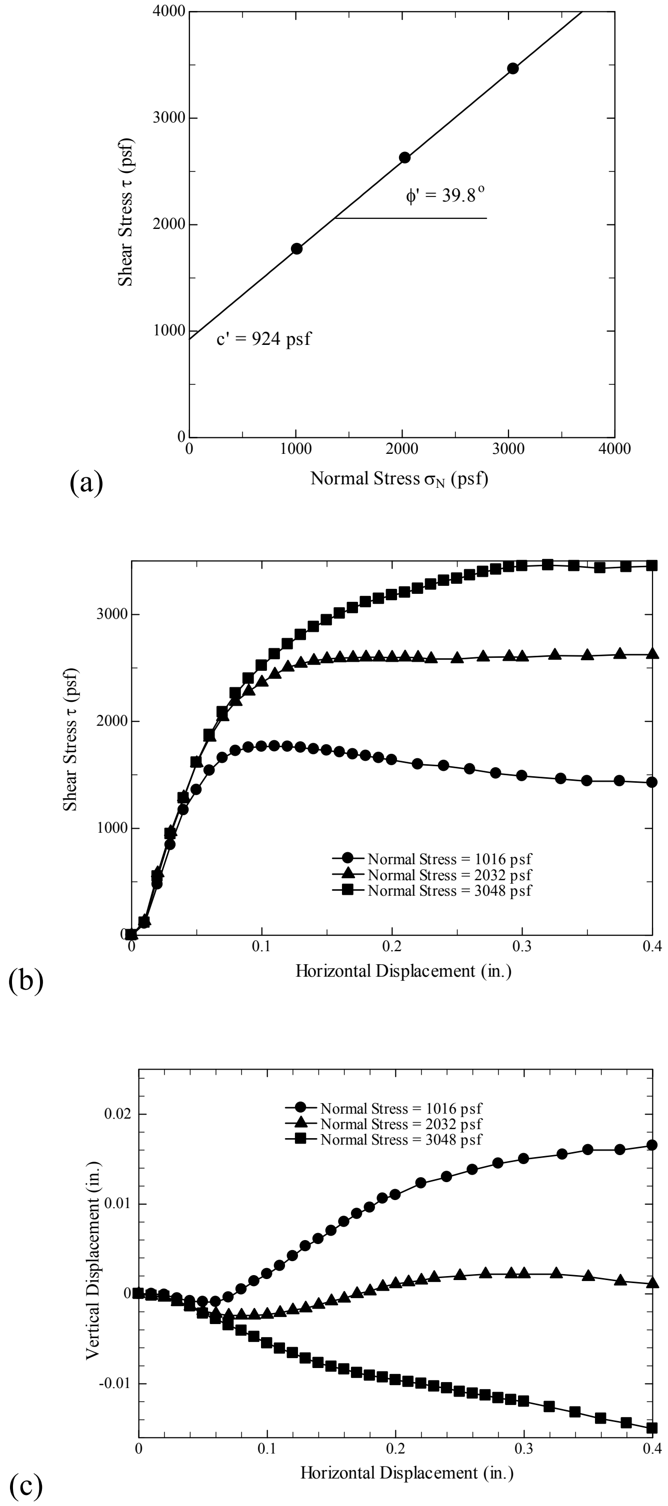
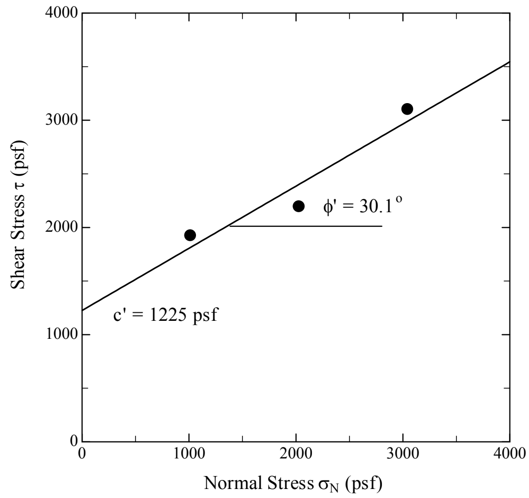
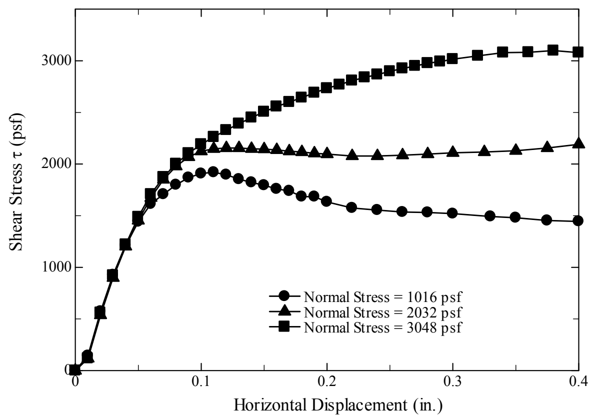


Figure 4.2 Results for D2-Anderson-01, Bucket 1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



(a)



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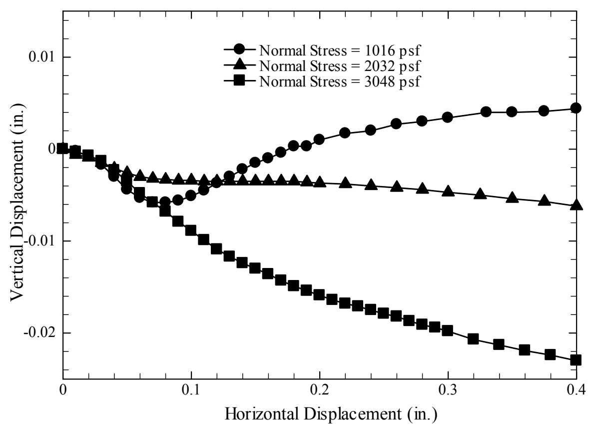
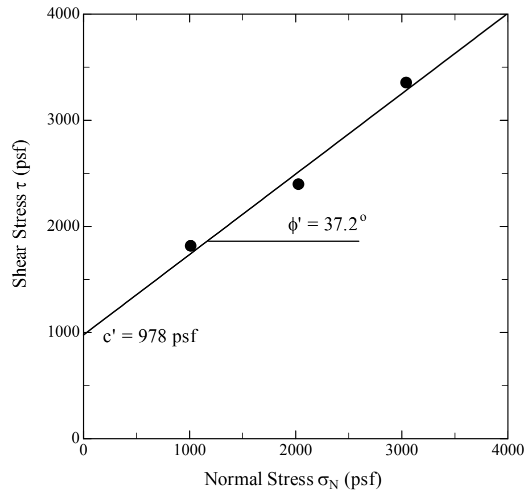
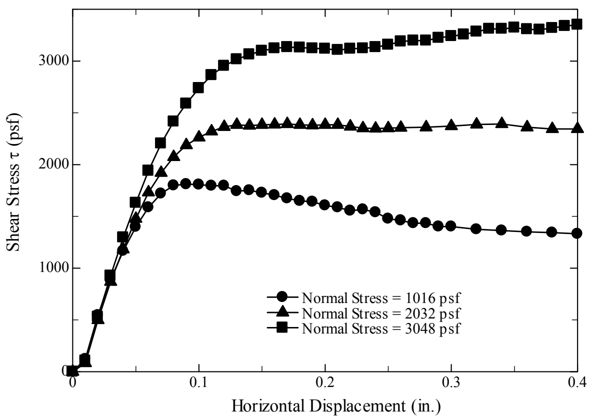


(c)

Figure 4.3 Results for D3-Anderson-05, Bucket 1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



(a)



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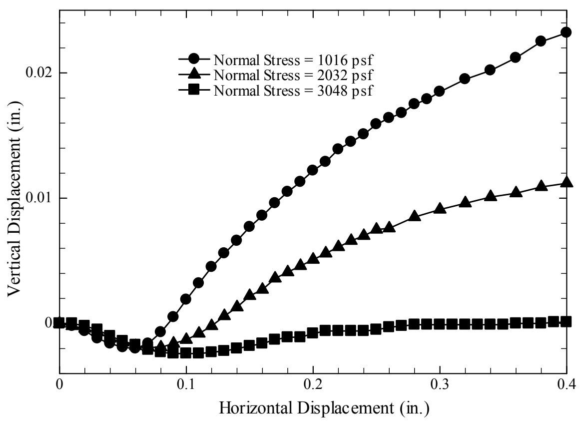


(c)

Figure 4.4 Results for D2-Abbeville-01, Bucket 1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

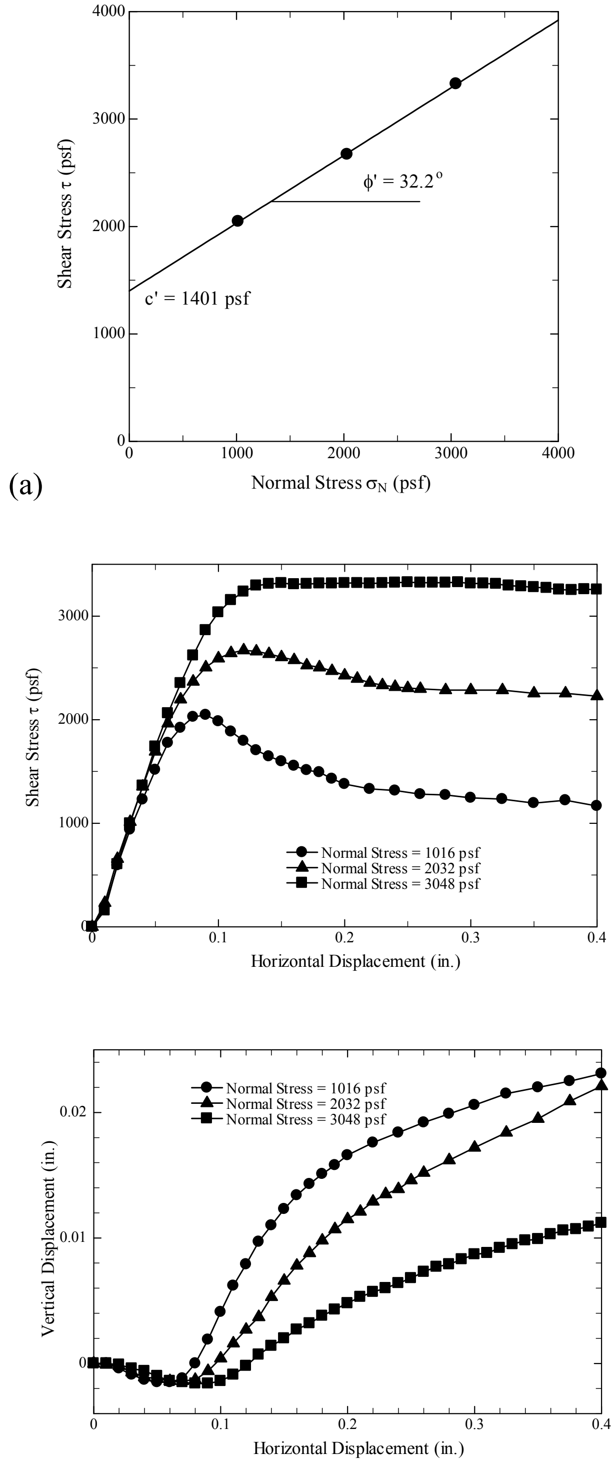
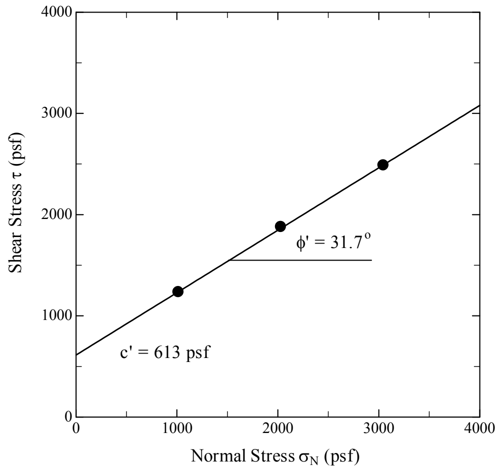
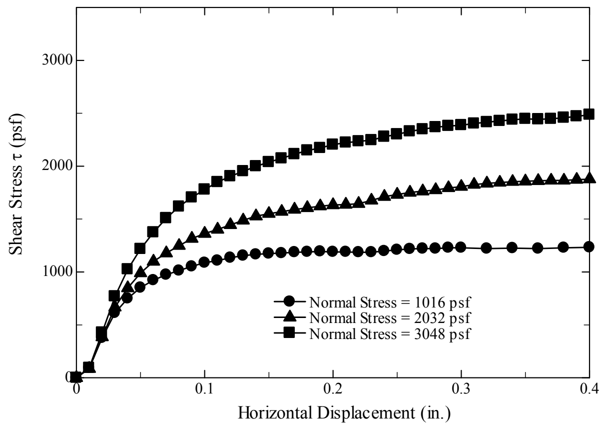


Figure 4.5 Results for D2-Abbeville-01, Bucket 3: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



(a)



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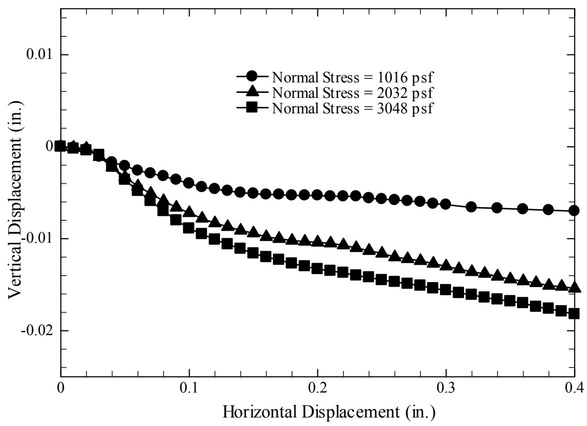


(c)

Figure 4.6 Results for D3-Oconee-01, Bucket 1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

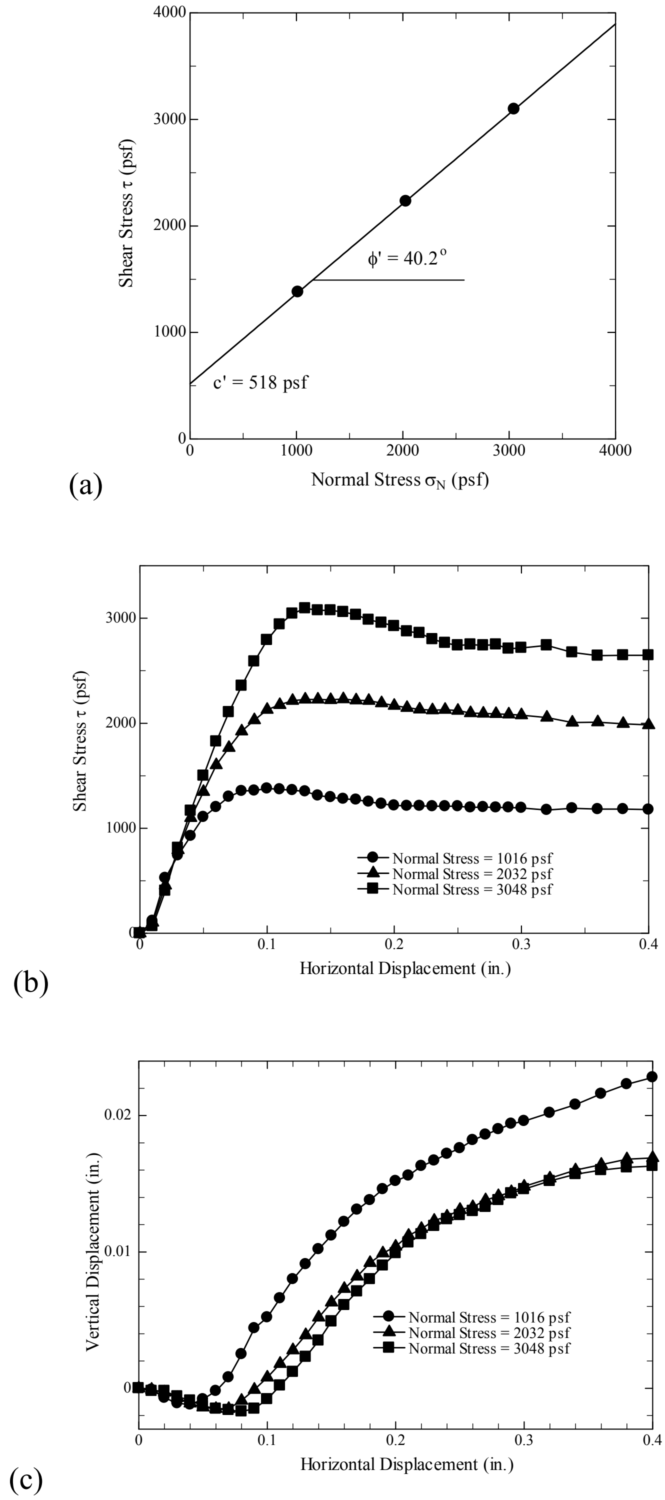
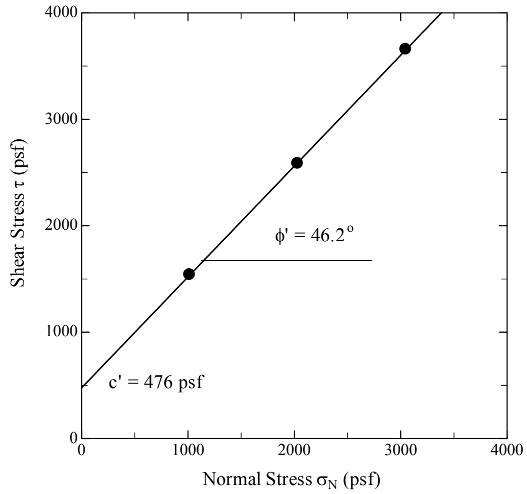
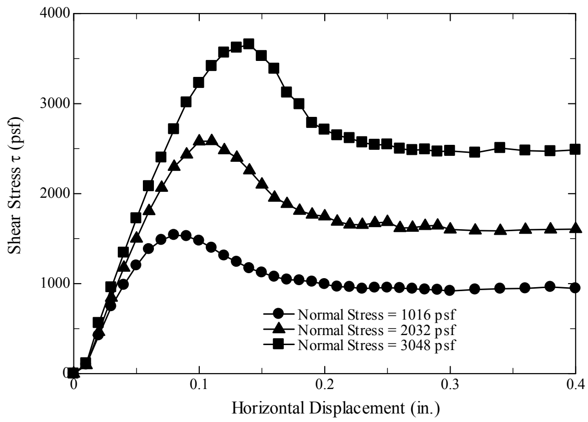


Figure 4.7 Results for D1-Lexington-05, Bucket1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



(a)



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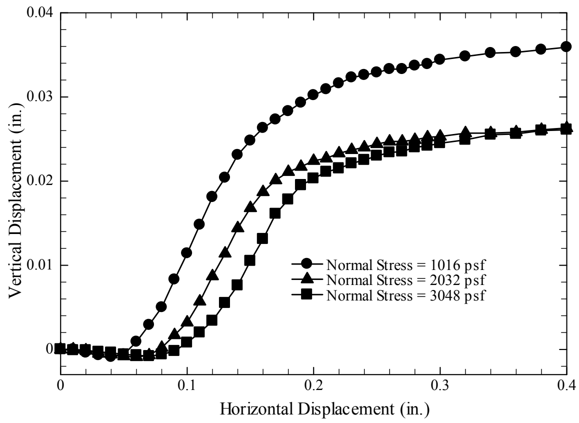
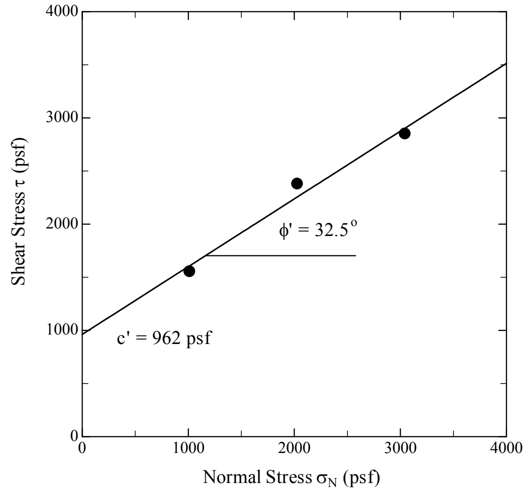
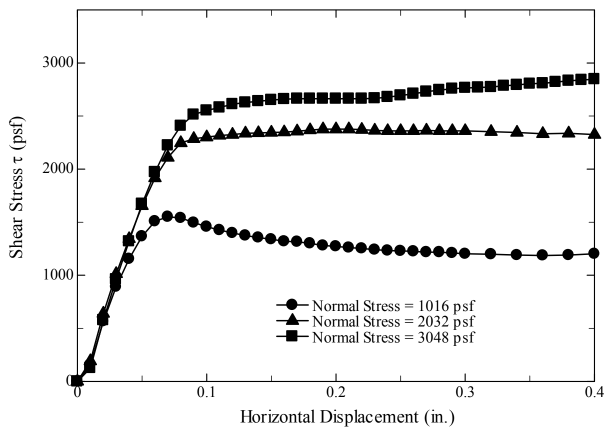


(c)

Figure 4.8 Results for D1-Lexington-13, Bucket1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



(a)



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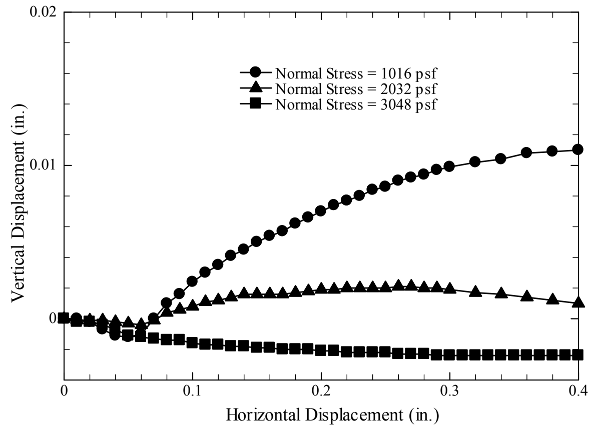
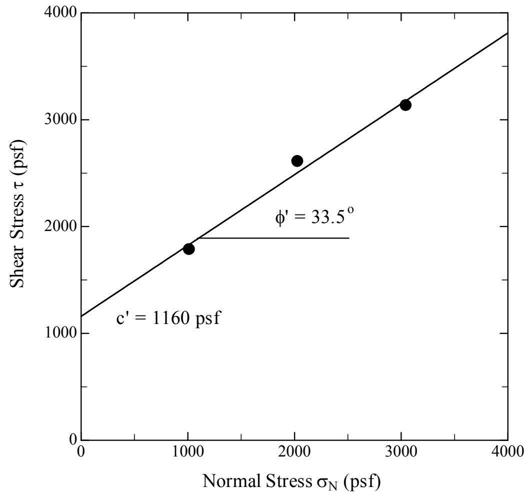


(c)

Figure 4.9 Results for D1-Richland-08, Bucket 1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



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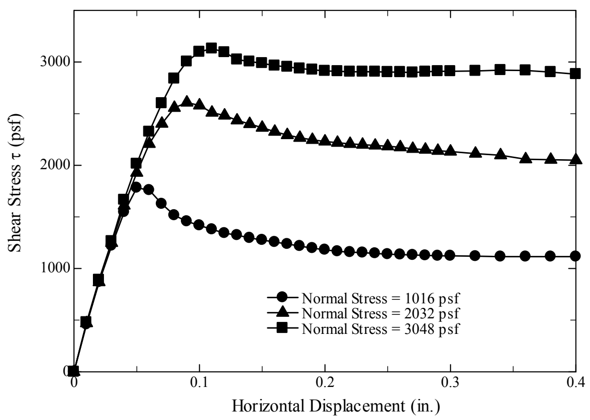

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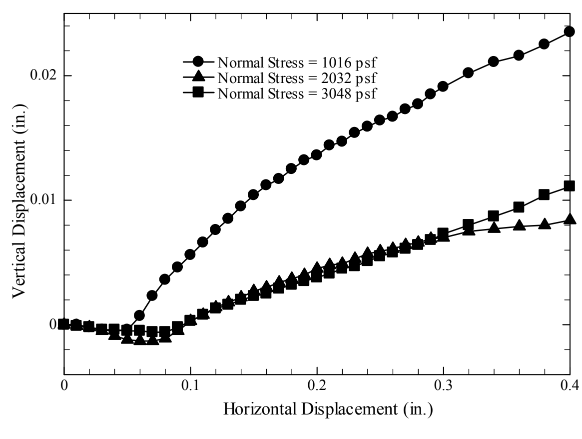
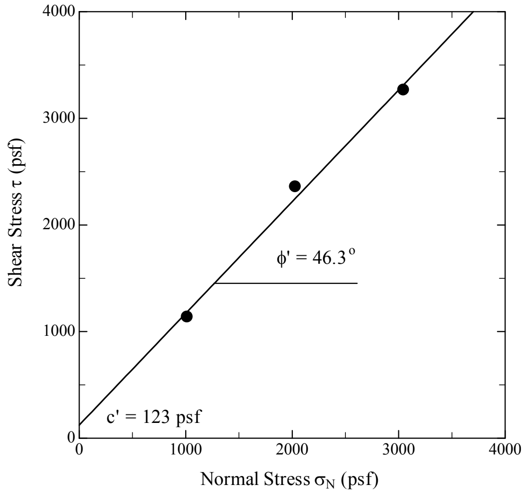


(c)

Figure 4.10 Results for D1-Richland-08, Bucket 2: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement



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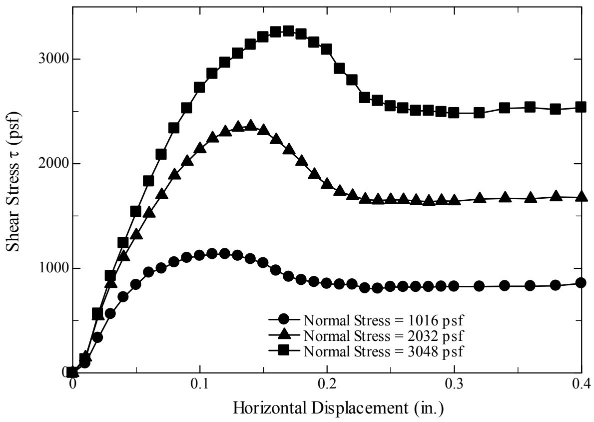

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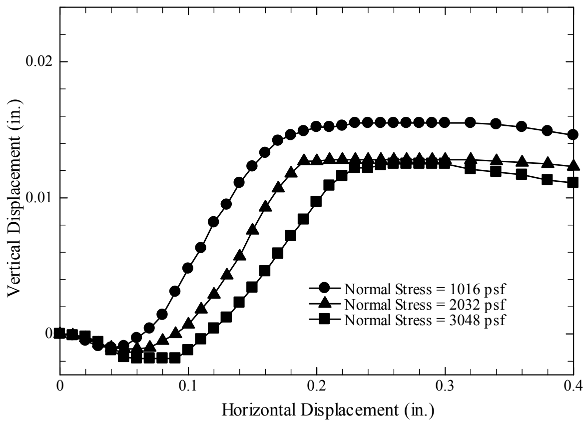

(c)

Figure 4.11 Results for D1-Kershaw-02, Bucket 2: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

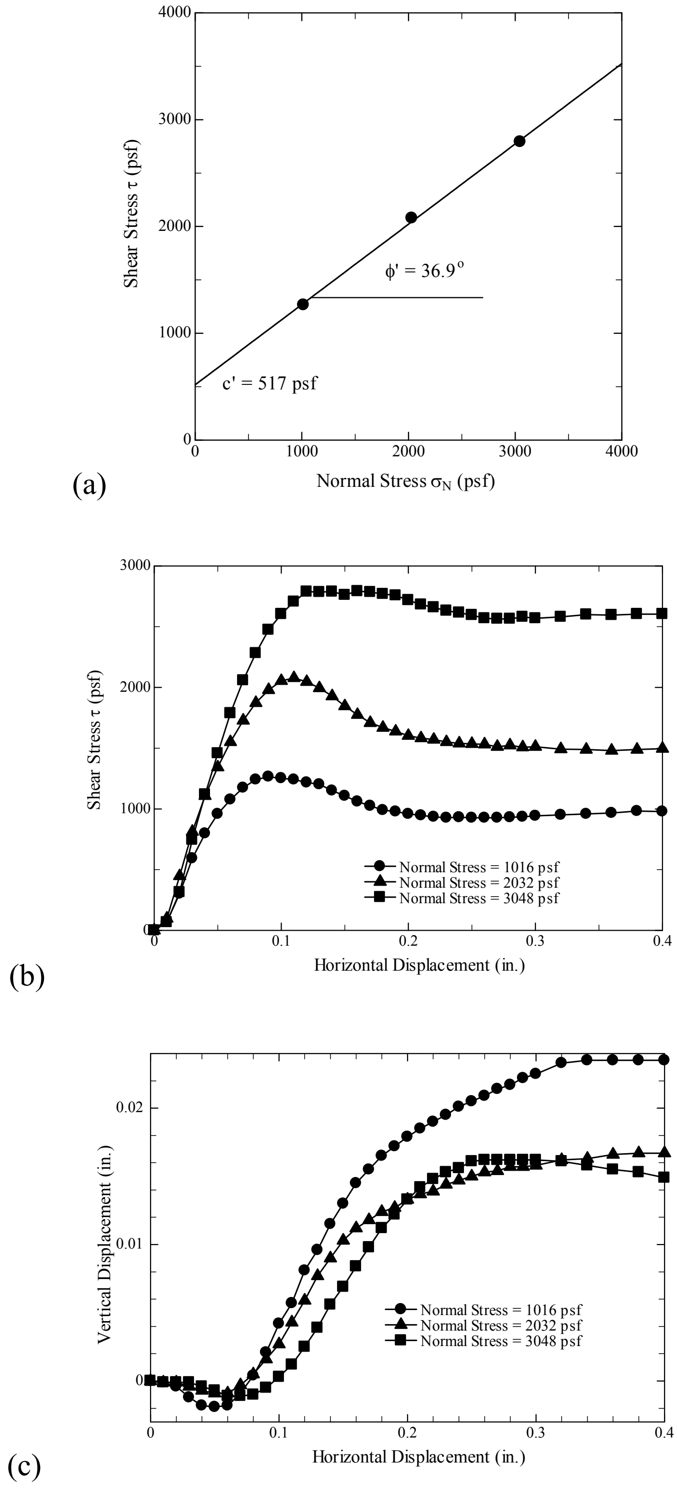

Figure 4.12 Results for D1-Aiken-05, Bucket 2: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

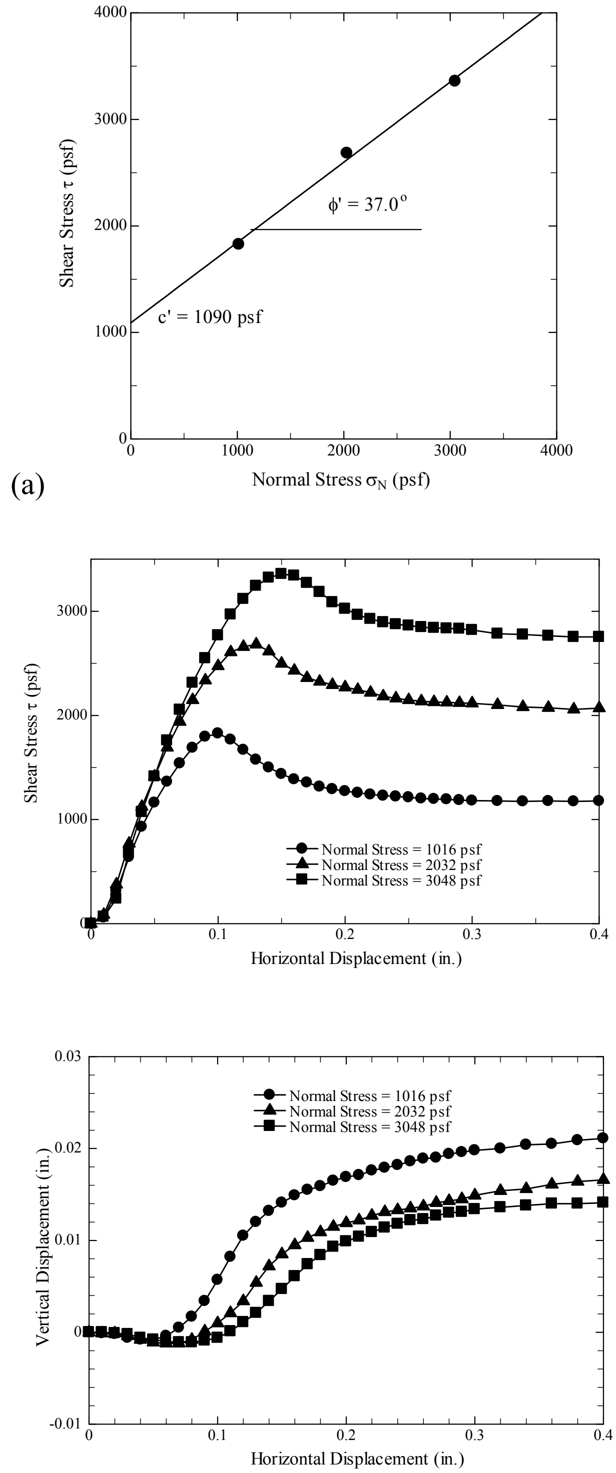

Figure 4.13 Results for D6-Charleston-06,Bucket1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

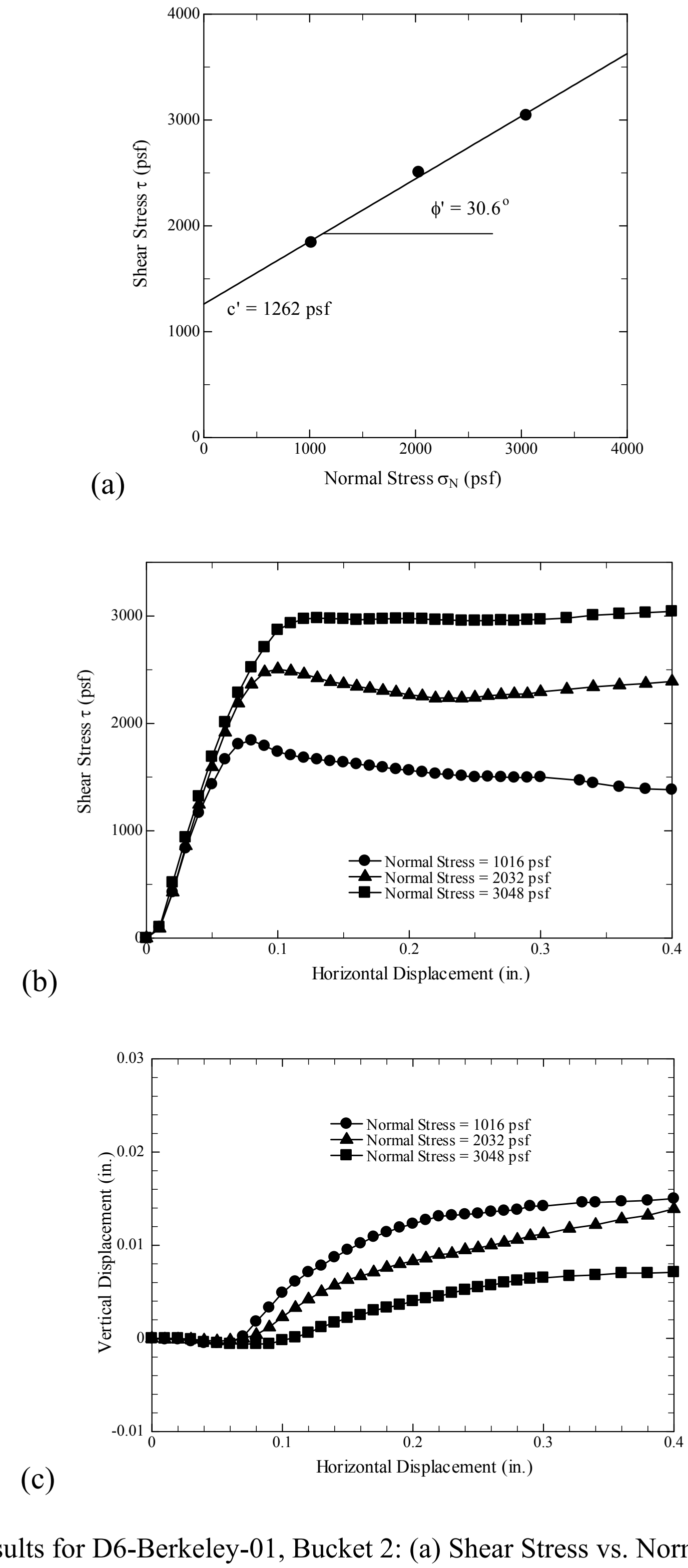

Figure 4.14 Results for D6-Berkeley-01, Bucket 2: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

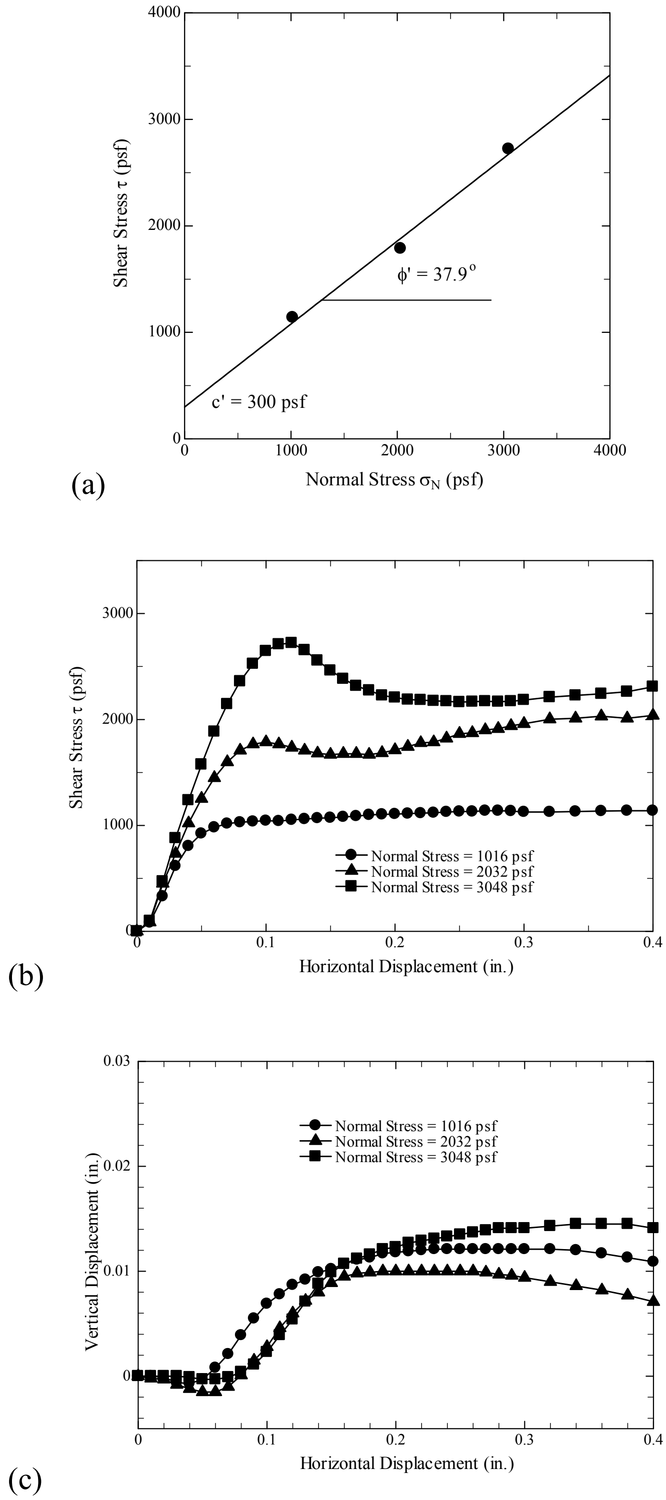

Figure 4.15 Results for D6-Dorchester-03,Bucket1: (a) Shear Stress vs. Normal Stress, (b) Shear Stress vs. Horizontal Displacement, (c) Vertical Displacement vs. Horizontal Displacement

4.1.2 Test Analysis

As discussed in Section 2.4.3.3, the effective friction angle obtained from direct shear test and triaxial test have a relationship shown in Equation (2-6).

Then, $\phi'_{T.X.}$ was calculated by using Equation (2-6). Most of the soils are dilative material, which can be seen from the diagram of vertical displacement vs. horizontal displacement, except one soil – D3-Oconee-01, B-1. So for most soils, ε = 5% $\phi_{T.X}'$. For D3-Oconee-01, B-1, ε = 10% $\phi_{T.X}'$. The values of $\phi_{T.X}'$ are shown in Table 4.4. Table 4.4 was given by combining useful information (such as soil classification, specific gravity, grain shape, and fines content) from Table 4.1, Table 4.2 and Table 4.3.

The last column in Table 4.4 is the average initial void ratio of the three specimens tested in the direct shear test.

Table 4.4 Direct Shear Test Results and Soil Indices (revised)

Pit & Bucket No.	Classif-ication (USCS)	Specific Gravity	Grain Shape Description	Fines Content	c'	ϕ'	ϕ_{TX}' [a]	$\bar{e}_0$
				%	psi	deg	deg	
D4-York-04, B-2	SM	2.63	Subangular	40	8.84	34.2	32.6	0.50
D2-Anderson-01, B-1	MH	2.74	-	53	6.42	39.8	37.9	0.88
D3-Anderson-05, B-1	MH	2.79	-	53	8.51	30.1	28.7	0.83
D2-Abbeville-01, B-1	SM	2.67	Subangular	36	6.79	37.2	35.4	0.61
D2-Abbeville-01, B-3	SM	2.64	Subrounded	35	9.73	32.2	30.7	0.57
D3-Oconee-01, B-1	CH	2.75	-	58	4.26	31.7	28.8	0.76
D1-Lexington-05, B-1	SC	2.76	Subangular to Subrounded	18	3.60	40.2	38.3	0.47
D1-Lexington-13, B-1	SP-SM	2.69	Subangular	6	3.31	46.2	44.0	0.44
D1-Richland-08, B-1	ML	2.86	-	75	6.68	32.5	31.0	0.76
D1-Richland-08, B-2	ML	2.89	-	88	8.06	33.5	31.9	0.74
D1-Kershaw-02, B-2	SW-SM	2.74	Subangular to Rounded	12	0.85	46.3	44.1	0.76
D1-Aiken-05, B-2	SP-SM	2.66	Angular	6	3.59	36.9	35.1	0.71
D6-Charleston-06, B-1	SM	2.53	subangular to Subrounded	20	7.57	37.0	35.2	0.57
D6-Berkeley-01, B-2	SM	2.54	Subrounded to Rounded	32	8.76	30.6	29.1	0.47
D6-Dorchester-03, B-1	SP-SM	2.71	Subrounded	12	2.08	37.9	36.1	0.68

a. Calculated from Equation 2-6.

4.1.2.1 Void Ratio and Effective Friction Angle

Void ratio, related to the density of the sand, is perhaps the most important single parameter that affects the strength of sands. Generally speaking, for drained tests in the direct shear apparatus, the lower the void ratio, the higher the shear strength.

For sands, samples from D1-Lexington-13, B-1 and D1-Aiken-05, B-2, both of them are SP-SM. They also have similar specific gravity, and the same fines content. The initial void ratio for D1-Lexington-13, B-1 is 0.438 which is lower than that for D1-

Aiken-05, B-2. As shown in Figure 4.16, the soil from D1-Lexington-13, B-1 has a larger friction angle 44.0° than that of D1-Aiken-05, B-2, 35.1°.

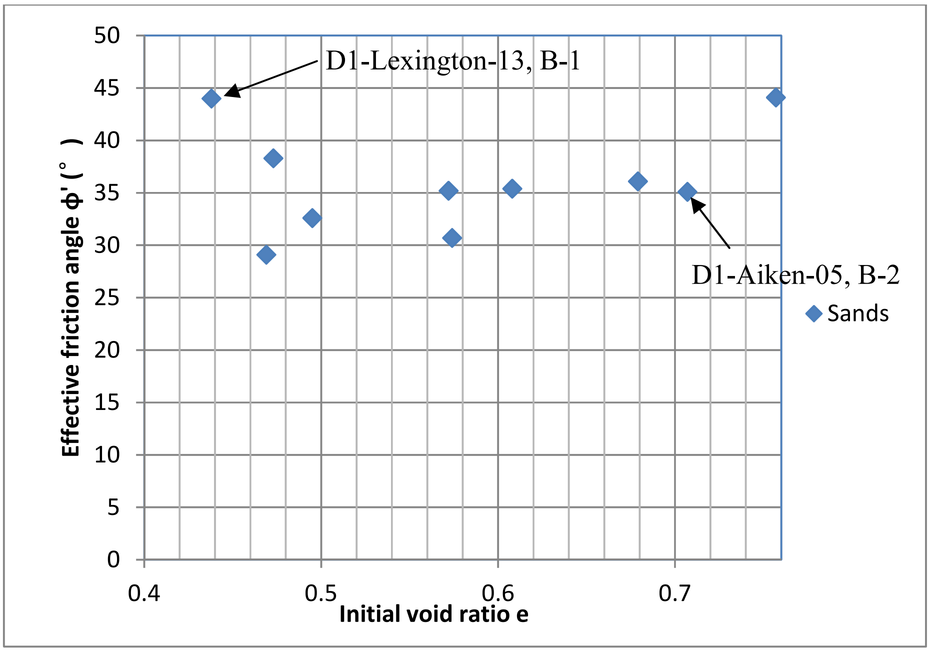

Figure 4.16 Effective Friction Angle vs. Initial Void Ratio

4.1.2.2 Grain Shape and Effective Friction Angle

Generally speaking, effective friction angle increases with increasing angularity with all else constant.

For example, samples from D2-Abbeville-01, B-3 and D6-Charleston-06, B-1 are both SM. And they have similarly specific gravity, fines content, and initial void ratio. The grain shape of the sample from D2-Abbeville-01, B-3 is subrounded, while the grain shape of the sample from D6-Charleston-06, B-1 is subangular to subrounded. As shown in Figure 4.17, the soil from D6-Charleston-06, B-1 has a larger friction angle 35.2° than that of D2-Abbeville-01, B-3, 30.7°.

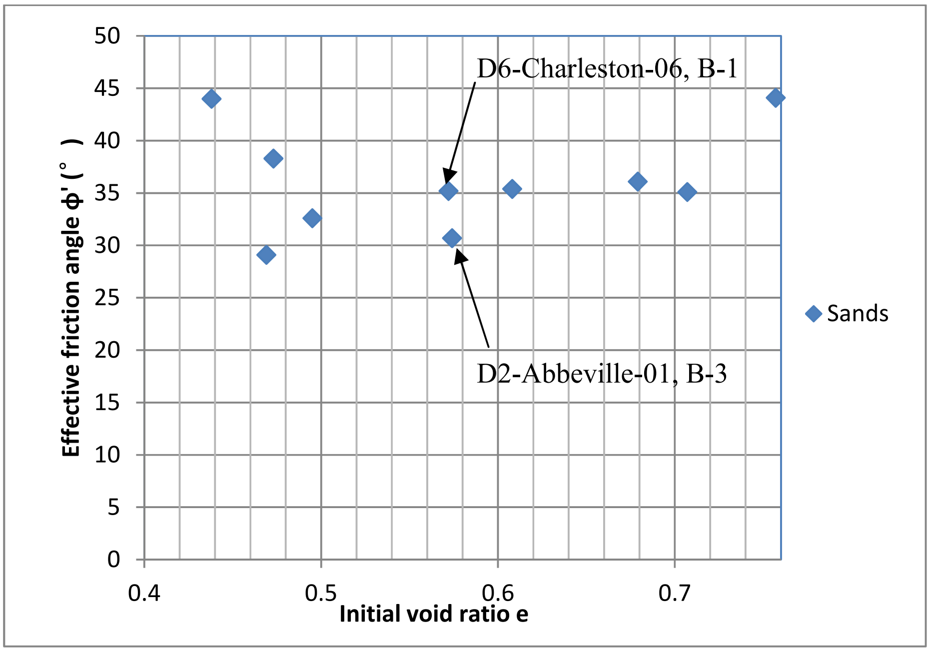

Figure 4.17 Effective Friction Angle vs. Initial Void Ratio (the Effect of Grain Shape)

4.1.2.3 Grain Size Distribution and Effective Friction Angle

If two sands have the same relative density, the soil that is more well graded should have a larger effective friction angle.

For example, samples from D1-Kershaw-02, B-2 and D6-Dorchester-03, B-1 have a similar specific gravity, grain shape, fines content and initial void ratio. The classification of the soil from D1-Kershaw-02, B-2 is SW-SM, while the classification of the soil from D6-Dorchester-03, B-1 is SP-SM. Therefore the soil from D1-Kershaw-02, B-2 has a larger effective friction angle 44.1° than that of D6-Dorchester-03, B-1, 36.1°.

4.1.2.4 Fines Content and Shear Strength Parameters

Generally speaking, a higher fines content will produce a higher cohesion and a lower effective friction angle.

For example, samples from D1-Lexington-05, B-1 and D6-Berkeley-01, B-2, they have similarly specific gravity, grain shape, and initial void ratio. The fines content of the sample from D1-Lexington-05, B-1 is 18%, while the fines content of the sample from D6- Berkeley-01, B-2, is 32%. As shown in Figure 4.18, the soil from D1-Lexington-05, B-1 has a larger friction angle 38.3° and a lower cohesion 3.60 psi than that of D6-Berkeley-01, B-2, 29.1° and 8.76 psi.

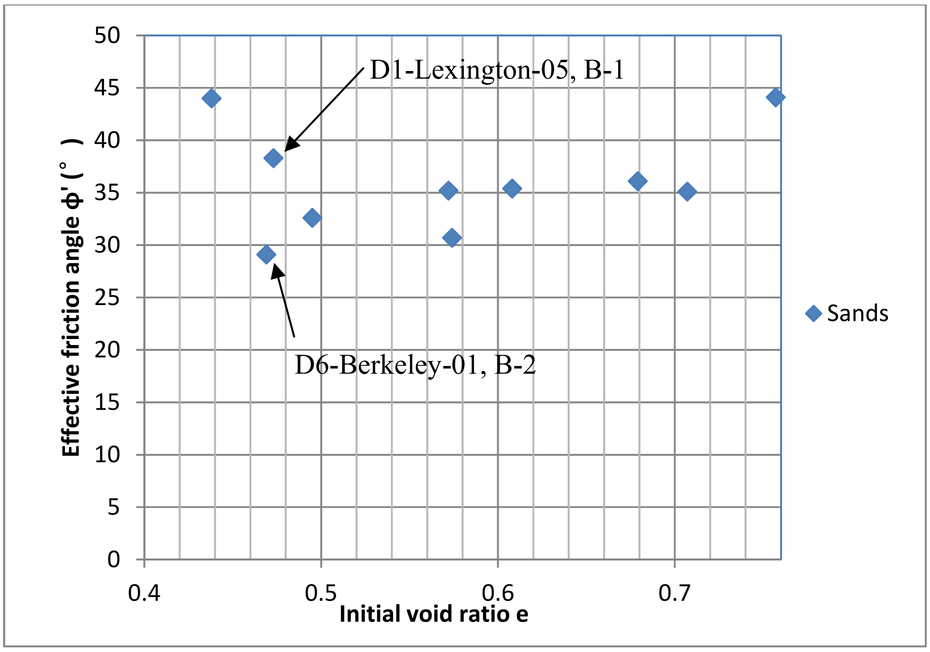

Figure 4.18 Effective Friction Angle vs. Initial Void Ratio (the Effect of Fines Content)

4.1.3 Summary

According to SCDOT Geotechnical Design Manual 2008, and based on the analysis above, several conclusions are listed below:

1. The silt (MH) in Group A, that is, soils from D2-Anderson-01, B-1 has an effective friction angle (ϕ_{TX}') of 37.9°. It falls in the range of 29.9°< ϕ' < 40.5° which is

given by SCDOT Geotechnical Design Manual 2008 for Group A. So this soil is suitable for use in embankments or as subgrade material.

2. The silty sand (SM) in Group A, that is, soils from D4-York-04, B-2, D2-Abbeville-01, B-1 and D2-Abbeville-01, B-3 have an effective friction angle (ϕ_{TX}') range of 30.7° to 35.4°. These soils are not typical Group A soils. But they do exist in Group A. The friction angle range still falls in the range of 29.9°< ϕ' < 40.5°. So these soils are considered as suitable for embankment construction.

3. For sands (contain SC, SM, SP-SM, SW-SM) in Group B, they have an effective friction angle range (ϕ_{TX}') of 29.1° to 44.1°. Soils from D1-Richland-08, B-1 and B-2, and D6-Berkeley-01, B-2 are in the range of 28°< ϕ' < 32° given by SCDOT Geotechnical Design Manual 2008. That is, these three soils are acceptable for embankment construction usage.

4. All the samples are compacted to 95% maximum dry density, that is, they are dense soils. So, high ϕ' are expected. For example, ϕ_{TX}' is 44.0° for soil from D1-Lexington-13, B-1; ϕ_{TX}' is 44.1° for soil from D1-Kershaw-02, B-2.

4.2 PINHOLE DISPERSION TEST

4.2.1 Pinhole Test Results

In this research, nineteen (19) pinhole tests were performed to evaluate the erodibility of soils collected from different borrow pits in South Carolina. The results are listed in Table 4.5. The initial moisture content w_{actual} is measured by using the leftovers after making the specimen. The dry unit weight $\gamma_{d\ actual}$ and initial moisture content w_{actual} were obtained using equation (4-1, 4-2, and 4-3).

In Table 4.5, pit name and bucket number, classification (USCS), optimum moisture content w_{opt}, the actual initial moisture content w_{actual}, maximum dry density $\gamma_{d\ max}$, 95% maximum dry density, the actual dry density of the specimen $\gamma_{d\ actual}$, specific gravity G_s, fines content and the dispersion classification (Method A) are listed in columns 1, 2, 3, 4, 5, 6, 7, 8, 9 and 10, respectively.

According to SCDOT Geotechnical Design Manual 2008, the samples used in the lab tests should be remolded to 95% of Standard Proctor with moisture -1% to +2% of the optimum moisture content. From Table 4.5, most of the specimens met this standard.

As discussed in Section 2.5, the erodibility of a soil is related to soil gradation, soil texture and structure, compaction effort and compaction water content. As all samples were compacted to 95% of Standard Proctor with moisture -1% to +2% of optimum moisture content, that is, they were all well compacted, therefore, most of them were non-dispersive soils or slightly dispersive soils.

Table 4.6 shows pinhole test data of D2-Anderson-01, B-1. All the pinhole tests data are listed in the Appendix.

Table 4.5 Pinhole Test Results

Pit & Bucket No.	USCS	w% (opt)	w% (actual)	$\gamma_{d\ max}$ (pcf)	95% $\gamma_{d\ max}$ (pcf)	$\gamma_{d\ actual}$ (pcf)	Specific Gravity	% fines	Dispersion Classification (Method A)
D2-Anderson-01, B-1	MH	24.1	24	96.8	92.0	92.1	2.74	53	ND1
D2-Anderson-01, B-2	SM	22	22	103.9	98.7	98.8	2.69	46	ND1
D2-Anderson-01, B-3	ML	19	19	103.5	98.3	99.4	2.71	50	ND1
D3-Anderson-05, B-1	MH	23	26	99	94.1	93.6	2.79	53	ND1
D3-Anderson-05, B-2	SM	20	20	106.5	101.2	101.7	2.80	46	ND1
D3-Anderson-05, B-3	SM	23	22.5	99.5	94.5	95.6	2.80	44	ND1
D3-Greenville-05,B-1	SM	23	21.3	98	93.1	95.3	2.76	48	ND1
D1-Kershaw-02, B-3	SM	14	13.8	111	105.5	107.6	2.75	29	ND1
D2-Abbeville-01, B-2	MH	28	28	91	86.5	87.4	2.83	68	ND2
D3-Greenville-05,B-2	MH	23	23	100	95.0	96.1	2.74	60	ND2
D3-Oconee-01, B-1	CH	23.5	24.1	102.5	97.4	98.6	2.75	58	ND2
D3-Oconee-01, B-2	MH	23	23	102.5	97.4	98.5	2.63	53	ND2
D1-Richland-08, B-1	ML	17	16.2	106	100.7	103.0	2.86	75	ND2
D1-Richland-08, B-2	ML	14.7	15	109.5	104.0	103.8	2.89	88	ND2
D2-Abbeville-01, B-1	SM	15	15.9	109.6	104.1	105.0	2.67	36	ND3
D3-Oconee-01, B-3	CH	19	19.2	106	100.7	102.1	2.62	57	ND3
D1-Lexington-05,B-3	SC	12.5	12.5	116	110.2	111.3	2.74	28	ND3
D1-Richland-08, B-3	CL-ML	14.7	15	109.5	104.0	104.3	2.83	52	ND3
D6-Berkeley-01, B-2	SM	12.3	17.4	113.4	107.7	105.4	2.54	32	ND3

* Dispersiveness categories: dispersive (D1, D2), slight to moderately dispersive (ND4, ND3), and nondispersive (ND2, ND1).

Table 4.6 Pinhole Test Data of D2-Anderson-01, B-1 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
6:08	2"	10	67	0.15					√			
		10	67	0.15						√	√	
		25	164	0.15						√	√	
		25	164	0.15						√	√	
6:22	7"	25	38	0.66					√			
		25	35	0.71					√			
		25	34	0.74					√			
6:26		25	34	0.74					√			
6:29	15"	25	17	1.47					√			
		25	17	1.47					√			
		50	35	1.43						√		
6:34		50	35	1.43						√	√	
6:38	40"	50	18	2.78						√	√	
		100	38	2.63						√	√	
		100	38	2.63						√	√	
		100	38	2.63						√	√	
6:43		100	38	2.63						√	√	Classificat-ion: ND1

4.2.2 Pinhole Test Analysis

All soils which were designated as ND1 have a fines content ranging from 44% to 53%, except D1-Kershaw-02, B-3. Thus soils with 50% fines content are more resistant to erosion when compacted to 95% $\gamma_{d\ max}$. For D1-Kershaw-02, B-3, the sample has a very high compaction density ($\gamma_{d\ actual}$ = 107.6 pcf) and lower water content (w_{actual} = 13.8 %). This is why it has a very high resistance to erosion.

For those soils which were designated as ND2, there is also a common characteristic. That is, most of these soils are silt (ML and MH, and one is a clay, CH) and have a higher fines content (53 to 88%) than that of ND1 soils.

For those soils which were designated as ND3, they are slightly dispersive soils. There are two situations for these soils. For soils from D2-Abbeville-01, B-1, D1-Lexington-05, B-3 and D6-Berkeley-01, B-2, their classifications are SM, SC and SM. The fines contents are around 30%. At this level, the fines will be more easily taken away by water than that of ND1 and ND2 soils. On the other hand, for soils from D3-Oconee-01, B-3 and D1-Richland-08, B-3, their classifications are CH and CL-ML, and fines contents are 57% and 52%. For these soils, the fines contents are around 50%, but there are more clay particles. They will be more easily taken away by water when mixed with sand.

4.2.3 Summary

Based on the analysis above, several conclusions are listed below:

1. For those silt soils (MH, ML) and silty sand soils (SM) which were designated as ND1, and whose fines contents are around 50%, they are non-dispersive soils. They can be used in embankment construction with proper compaction without dispersion consideration.

2. For those silt soils (MH, ML) and clay (CH) which were designated as ND2, and whose fines content range from 60% to 90%, they are also non-dispersive soils. They also can be used in embankment construction with proper compaction.

3. For those silty sand soils (SM), clayey sand soils (SC) which were designated as ND3, and whose fines contents are around 30%, they are slightly dispersive soils. By considering long-term effects, these soils should be avoided for use in embankment construction. If they have to be used in embankment construction under certain circumstances, the erosion failure should be considered in embankment design. Or some method should be used to prevent erosion failure. For example, reinforcement construction or specially designed filters can be used in this situation to prevent erosion failure of the embankment.

4. For those sandy clay soils (CL, CL-ML), which were designated as ND3, and whose fines contents are around 50%, they are slightly dispersive soils. They should be avoided in embankment construction usage. If they have to be used under certain situations, the same method should be used as conclusion 3.

4.3 GOOD EMBANKMENT SOILS VS. NOT AS GOOD SOILS

Based on the discussion above, the good embankment soils should have high strength and low dispersion. In the other hand, the not as good soils may have lower strength, or higher dispersion, or both.

Table 4.8 shows the priority of the good embankment soils. In the upstate area, soil from D2-Anderson-01, B-1 have an effective friction angle ϕ_{TX}' as 37.9°, which is in the range of 29.9°< ϕ' < 40.5° which is given by SCDOT Geotechnical Design Manual 2008 for Group A and it is also nondispersive. So it is the most suitable soil (five-star priority) for embankment in this area. Soils from D2-Abbeville-01, B-1, D4-York-04, B-2, and D2-Abbeville-01, B-3, have high effective friction angles, but the dispersion classifications are not available (These soils are not suitable for pinhole tests). So they are four-star priority soils. Soil from D3-Anderson-05, B-1 has a lower effective friction angle, and is nondispersive. So it's three-star priority. Soil from D3-Oconee-01, B-1 has a lower effective friction angle, and is ND2. So it's two-star priority soil.

By using the same method, the priority all soils from fall zone and coastal area were determined, as shown in Table 4.8.

Table 4.7 Priority of the Good Embankment Soils

Pit & Bucket No.	Classification USCS	c′ psi	ϕ′ deg	ϕ_{TX}' deg	$\bar{e}_0$	Dispersion Classification	Priority
D2-Anderson-01, B-1	MH	6.42	39.8	37.9	0.88	ND1	*****
D2-Abbeville-01, B-1	SM	6.79	37.2	35.4	0.61	N/A	****
D4-York-04, B-2	SM	8.84	34.2	32.6	0.5	N/A	****
D2-Abbeville-01, B-3	SM	9.73	32.2	30.7	0.57	N/A	****
D3-Anderson-05, B-1	MH	8.51	30.1	28.7	0.83	ND1	***
D3-Oconee-01, B-1	CH	4.26	31.7	28.8	0.76	ND2	**
D1-Richland-08, B-2	ML	8.06	33.5	31.9	0.74	ND2	****
D1-Richland-08, B-1	ML	6.68	32.5	31.0	0.76	ND2	****
D1-Lexington-13, B-1	SP-SM	3.31	46.2	44.0	0.44	N/A	***
D1-Kershaw-02, B-2	SW-SM	0.85	46.3	44.1	0.76	N/A	***
D1-Aiken-05, B-2	SP-SM	3.59	36.9	35.1	0.71	N/A	***
D1-Lexington-05, B-1	SC	3.60	40.2	38.3	0.47	N/A	***
D6-Dorchester-03, B-1	SP-SM	2.08	37.9	36.1	0.68	N/A	***
D6-Charleston-06, B-1	SM	7.57	37.0	35.2	0.57	N/A	***
D6-Berkeley-01, B-2	SM	8.76	30.6	29.1	0.47	ND3	**

CHAPTER 5

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

5.1 SUMMARY

To evaluate strength and erodibility characteristics of borrow soils for embankment construction, forty (40) five-gallon buckets of soil samples were collected from fourteen (14) borrow pits in South Carolina. In most cases, three bucket samples were collected at three different locations within each pit.

Fifteen (15) sets of direct shear tests were performed on the selected fifteen bucket samples. There were six samples from the upstate area, six samples from the fall zone area, and three samples from the coastal area. They represent typical soils from each area. All of the specimens were remolded to 95% of the maximum dry density at a moisture content within -1 % to +2 % of the optimum water content.

Nineteen (19) pinhole tests were performed to evaluate the erodibility of all the soils which were suitable for this test. Based on the results, most of the soils were non-dispersive when compacted to 95% of the maximum dry density at a moisture content within -1 % to +2 % of the optimum water content.

5.2 CONCLUSIONS

Based on the direct shear test and pinhole test results, several conclusions are listed as follows:

1. The trends in observed shear strength from the direct shear tests were as expected:

(a) Soils with lower void ratios had higher effective friction angles.

(b) Soils with higher angularity had higher effective friction angles with all other parameters being the same.

(c) Soils that were more well graded had a larger effective friction angle for soils with the same void ratio.

(d) Soils with higher fines content had higher cohesion and lower effective friction angles.

2. Based on the friction angles (ϕ_{TX}') calculated from the direct shear tests, Group A soils have higher effective friction angles than Group B soils. The results for Group A soils (effective friction angles ranging from 28.7° to 37.9°) 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° < ϕ' < 40.5°. The effective friction angles of soils from D3-Anderson-05, B-1 and D3-Oconee-01, B-1 fell below 29°, but they did not approach the conservative lower bound of 27.3°.

3. In Group B soils, the effective friction angles of ML and SM soils found in Richland and Berkeley counties are consistent with SCDOT experience of soils with effective friction angles ranging from 28° < ϕ' < 32°. The effective friction angles of SM

soils in Charleston county is just above this range with $\phi' = 35.2°$. Most of the SC, SP-SM and SW-SM soils have higher effective friction angles in a range of 35.1° to 44.1°.

4. The friction angles from the direct shear tests were consistently higher than those from the triaxial tests performed in a parallel study. That is ϕ' (DS) > ϕ' (TX). (Pierce et al. 2011).

5. High ϕ' are expected, because all the samples are compacted to 95% maximum dry density, that is, they are dense soils. For example, ϕ_{TX}' is 44.0° for soil from D1-Lexington-13, B-1; ϕ_{TX}' is 44.1° for soil from D1-Kershaw-02, B-2.

6. The erosion potential of most soils from borrow sources is low. In Group A, most soils are non-dispersive (ND1 and ND2). Only two samples from Abbeville and Oconee are slight-dispersive (ND3). In Group B, the SM and ML soils found in Kershaw and Richland are non-dispersive (ND1 and ND2). The SC, CL-ML, and SM soils in Lexington, Richland, and Berkeley counties are slight-dispersive (ND3).

5.3 RECOMMENDATIONS FOR FURTHER WORK

If this research will continue, several recommendations are listed below:

1. More samples should be collected from Fall Zone and Coastal Area. Because soils from Fall Zone are variable, and only three samples were collected from Coastal Area.

2. More direct shear tests should be conducted on soils from other borrow pits with different soils, or even from the same borrow pit due to the soils variability. This effort will provide more information to develop a comprehensive geotechnical materials database to serve as a guide for further embankment construction.

3. Refer to the SCDOT project report made by Pierce et al. 2011, some Triaxial tests were run, but should run more on all soils obtained. After all, the triaxial test is a more accurate test, and it is more suitable for testing soils being considered for embankment construction.

4. Other dispersion tests, such as crumb test, double hydrometer test and cylinder dispersion test, should be conducted to compare to the pinhole test results.

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APPENDIX

This appendix contains all the pinhole tests results. A.4.1 to A.4.13 are the results of Group A soils; A.4.14 to A.4.15 are the results of Group B soils.

A.4.1 Pinhole Test Data of D2-Anderson-01, B-1 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
18:08	2"	10	67	0.15					√			
		10	67	0.15						√	√	
		25	164	0.15						√	√	
		25	164	0.15						√	√	
18:22	7"	25	38	0.66					√			
		25	35	0.71					√			
		25	34	0.74					√			
18:26		25	34	0.74					√			
18:29	15"	25	17	1.47					√			
		25	17	1.47					√			
		50	35	1.43						√		
18:34		50	35	1.43						√	√	
18:38	40"	50	18	2.78						√	√	
		100	38	2.63						√	√	
		100	38	2.63						√	√	
		100	38	2.63						√	√	
18:43		100	38	2.63						√	√	Classificat-ion: ND1

A.4.2 Pinhole Test Data of D2-Anderson-01, B-2 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
16:27	2"	10	25	0.40					√			
		10	30	0.33					√			
		25	58	0.43						√	√	
16:37		25	60	0.42						√	√	
16:39	7"	25	25	1.00					√			
		25	26	0.96						√	√	
		25	26	0.96						√	√	
16:44		25	26	0.96						√	√	
16:45	15"	50	32	1.56					√			
		50	32	1.56						√	√	
		50	32	1.56						√	√	
16:51		50	32	1.56						√	√	
16:54	40"	50	19	2.63					√			
		50	19	2.63						√	√	
		100	39	2.56						√	√	
		100	39	2.56						√	√	
17:00		100	40	2.50						√	√	Classificat-ion: ND1

A.4.3 Pinhole Test Data of D2-Anderson-01, B-3 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
14:13	2"	10	31	0.32					√			
		10	31	0.32						√	√	
		25	72	0.35						√	√	
14:23		25	73	0.34						√	√	
14:25	7"	25	70	0.36						√	√	
		25	70	0.36						√	√	
		25	70	0.36						√	√	
14:30		25	71	0.35						√	√	
14:41	15"	50	30	1.67					√			
		50	30	1.67						√	√	
		50	31	1.61						√	√	
14:46		50	30	1.67						√	√	
14:50	40"	50	18	2.78					√			
		100	36	2.78						√	√	
		100	35	2.86						√	√	
		100	35	2.86						√	√	
14:55		100	35	2.86						√	√	Classificat-ion: ND1

A.4.4 Pinhole Test Data of D3-Anderson-05, B-1 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
17:34	2"	10	28	0.36					√			
		10	28	0.36					√			
		25	70	0.36					√			
17:44		25	70	0.36						√	√	
17:45	7"	25	32	0.78				√				
		25	27	0.93					√			
		25	27	0.93						√	√	
17:51		25	27	0.93						√	√	
17:52	15"	50	21	2.38					√			
		50	39	1.28						√	√	
		50	39	1.28						√	√	
17:57		50	38	1.32						√	√	
18:02	40"	50	38	1.32					√			
		100	90	1.11						√	√	
		100	91	1.10						√	√	
18:07		100	92	1.09						√	√	Classificat-ion: ND1

A.4.5 Pinhole Test Data of D3-Anderson-05, B-2 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
16:54	2"	10	10	1.00				√				
		10	14	0.71					√			
		25	39	0.64						√		
17:04		25	40	0.63						√	√	
17:06	7"	25	21	1.19				√				
		25	22	1.14					√			
		25	21	1.19						√		
17:11		25	22	1.14						√	√	
17:13	15"	50	47	1.06				√				
		50	47	1.06						√		
		50	47	1.06						√	√	
17:18		50	48	1.04						√	√	
17:22	40"	50	42	1.19					√			
		100	86	1.16						√		
		100	88	1.14						√	√	
17:27		100	89	1.12						√	√	Classificat-ion: ND1

A.4.6 Pinhole Test Data of D3-Anderson-05, B-3 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
19:41	2"	10	40	0.25					√			
		10	32	0.31						√		
		25	76	0.33						√		
19:51		25	77	0.32						√	√	
19:54	7"	25	32	0.78						√		
		25	40	0.63						√	√	
		25	33	0.76						√	√	
19:59		25	33	0.76						√	√	
20:00	15"	50	45	1.11					√			
		50	42	1.19						√		
		50	41	1.22						√	√	
20:05		50	40	1.25						√	√	
20:08	40"	50	32	1.56					√			
		100	66	1.52						√		
		100	70	1.43						√	√	
20:13		100	73	1.37						√	√	Classificat-ion: ND1

A.4.7 Pinhole Test Data of D3-Greenville-05, B-1 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
16:48	2"	10	23	0.43					√			
		10	26	0.38					√			
		25	75	0.33						√		
16:58		25	76	0.33						√	√	
17:02	7"	25	28	0.89					√			
		25	37	0.68						√		
		25	36	0.69						√		
17:07		25	36	0.69						√	√	
17:09	15"	50	43	1.16					√			
		50	45	1.11						√		
		50	45	1.11						√	√	
17:14		50	45	1.11						√	√	
17:18	40"	50	26	1.92					√			
		100	62	1.61						√		
		100	66	1.52						√	√	
17:23		100	68	1.47						√	√	Classificat-ion: ND1

A.4.8 Pinhole Test Data of D2-Abbeville-01, B-2 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
15:27	2"	10	35	0.29					√			
		10	35	0.29						√		
		25	87	0.29						√	√	
15:37		25	89	0.28						√	√	
15:40	7"	25	30	0.83					√			
		25	25	1.00						√	√	
		25	26	0.96						√	√	
15:45		25	26	0.96						√	√	
15:46	15"	50	14	3.57					√			
		50	27	1.85						√		
		50	26	1.92						√	√	
15:51		50	26	1.92						√	√	
15:58	40"	50	13	3.85						√	√	
		100	27	3.70						√	√	
		100	27	3.70						√	√	
16:03		100	27	3.70						√	√	Classificat-ion: ND2

A.4.9 Pinhole Test Data of D3-Greenville-05, B-2 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
17:20	2"	10	25	0.40					√			
		10	23	0.43						√		
		25	57	0.44						√		
		25	55	0.45						√	√	
17:30		25	54	0.46						√	√	
17:32	7"	25	28	0.89					√			
		25	25	1.00						√		
		25	24	1.04						√	√	
17:37		25	24	1.04						√	√	
17:38	15"	50	27	1.85					√			
		50	26	1.92						√		
		50	26	1.92						√		
17:43		50	26	1.92						√	√	
17:47	40"	50	13	3.85					√			
		100	26	3.85						√		
		100	26	3.85						√	√	
		100	26	3.85						√	√	
17:52		100	26	3.85						√	√	Classificat-ion: ND2

A.4.10 Pinhole Test Data of D3-Oconee-01, B-1 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
19:12	2"	10	19	0.53					√			
		10	19	0.53						√	√	
		25	55	0.45						√	√	
19:22		25	54	0.46						√	√	
	7"	25	22	1.14				√				
		25	23	1.09					√			
		25	22	1.14						√		
19:27		25	23	1.09						√	√	
	15"	50	27	1.85				√				
		50	26	1.92					√			
		50	26	1.92						√		
19:35		50	26	1.92						√	√	
	40"	50	14	3.57				√				
		100	28	3.57						√		
		100	28	3.57						√		
19:48		100	27	3.70						√	√	Classificat-ion: ND2

A.4.11 Pinhole Test Data of D3-Oconee-01, B-2 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
15:53	2"	10	60	0.17					√			
		10	46	0.22						√		
		25	117	0.21						√		
16:03		25	113	0.22						√	√	
16:07	7"	25	39	0.64					√			
		25	36	0.69						√		
		25	37	0.68						√		
16:12		25	37	0.68						√		
16:14	15"	50	29	1.72					√			
		50	30	1.67						√		
		50	29	1.72						√		
16:19		50	29	1.72						√	√	
16:24	40"	50	15	3.33					√			
		100	31	3.23						√		
		100	31	3.23						√		
		100	31	3.23						√		
16:29		100	31	3.23						√		Classificat-ion: ND2

A.4.12 Pinhole Test Data of D2-Abbeville-01, B-1 (ND3)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
22:25	2"	10	20	0.50				√				
		10	16	0.63					√			
		25	43	0.58						√		
22:35		25	44	0.57						√		
22:37	7"	25	15	1.67				√				
		25	15	1.67					√			
		25	16	1.56						√		
22:42		25	16	1.56						√		
22:44	15"	50	20	2.50				√				
		50	20	2.50					√			
		50	20	2.50						√		
22:50		50	20	2.50						√		Classificat-ion: ND3

A.4.13 Pinhole Test Data of D3-Oconee-01, B-3 (ND3)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
15:30	2"	10	17	0.59				√				
		10	16	0.63					√			
		25	41	0.61					√			
15:41		25	39	0.64						√		
15:43	7"	25	16	1.56				√				
		25	17	1.47						√		
		25	16	1.56						√	√	
15:48		25	16	1.56						√	√	
15:50	15"	50	20	2.50			√					
		50	19	2.63					√			
		50	20	2.50						√		
15:55		50	19	2.63						√		Classificat-ion: ND3

A.4.14 Pinhole Test Data of D1-Kershaw-02, B-3 (ND1)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
18:25	2"	5	175	0.03					√			
		5	175	0.03					√			
18:10		5	244	0.02						√	√	
18:14	7"	5	94	0.05					√			
		5	95	0.05						√		
18:19		5	96	0.05						√	√	
18:21	15"	10	81	0.12					√			
		10	82	0.12						√		
		10	79	0.13						√		
18:27		10	75	0.13						√		
18:29	40"	10	20	0.50			√					
		10	19	0.53				√				
		25	43	0.58				√				
18:34		25	38	0.66					√			Classificat-ion:ND1

A.4.15 Pinhole Test Data of D1-Richland-08, B-1 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
17:25	2"	10	25	0.40				√				
		10	32	0.31					√			
		25	86	0.29						√		
17:35		25	82	0.30						√	√	
17:38	7"	25	41	0.61					√			
		25	41	0.61						√		
		25	41	0.61						√	√	
17:43		25	41	0.61						√	√	
17:46	15"	25	48	0.52					√			
		50	47	1.06						√		
		50	47	1.06						√	√	
17:51		50	46	1.09						√	√	
17:55	40"	50	13	3.85			√					
		100	28	3.57				√				
		100	25	4.00				√				
		100	23	4.35				√				
18:00		100	25	4.00				√				Classificat-ion:ND2

A.4.16 Pinhole Test Data of D1-Richland-08, B-2 (ND2)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
18:15	2"	10	37	0.27					√			
		10	40	0.25						√		
		25	95	0.26						√		
18:25		25	99	0.25						√		
18:27	7"	25	48	0.52						√		
		25	44	0.57						√		
		25	43	0.58						√		
18:31		25	42	0.60						√		
18:33	15"	25	48	0.52					√			
		50	46	1.09						√		
		50	45	1.11						√		
18:38		50	44	1.14						√		
18:43	40"	50	20	2.50				√				
		50	16	3.13				√				
		100	23	4.35				√				
		100	23	4.35				√				
		100	22	4.55				√				Classificat-ion:ND2

A.4.17 Pinhole Test Data of D1-Lexington-05, B-3 (ND3)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
17:59	2"	10	9	1.11				√				
		10	12	0.83					√			
		25	28	0.89						√		
18:09		25	29	0.86						√		
18:10	7"	25	15	1.67						√		
		25	15	1.67						√		
		25	15	1.67						√		
18:16		25	15	1.67						√	√	
18:17	15"	50	20	2.50					√			
		50	20	2.50						√	√	
		50	20	2.50						√	√	
18:22		50	21	2.38						√	√	Classificat-ion: ND3
18:28	40"	50	12	4.17					√			
		100	25	4.00						√		
		100	25	4.00						√		
		100	26	3.85						√	√	
18:33		100	26	3.85						√	√	

A.4.18 Pinhole Test Data of D1-Richland-08, B-3 (ND3)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
18:49	2"	10	12	0.83				√				
		10	13	0.77					√			
		25	34	0.74						√		
18:59		25	35	0.71						√	√	
19:01	7"	25	17	1.47						√		
		25	18	1.39						√	√	
		25	18	1.39						√	√	
19:06		25	18	1.39						√	√	
19:07	15"	25	11	2.27					√			
		25	12	2.08						√		
		25	11	2.27						√		
		50	22	2.27						√	√	
19:12		50	22	2.27						√	√	Classificat-ion: ND3
19:16	40"	50	11	4.55				√				
		100	25	4.00					√			
		100	23	4.35						√		
		100	23	4.35						√		
		100	24	4.17						√		

A.4.19 Pinhole Test Data of D6-Berkeley-01, B-2 (ND3)

Clock Time	Head	Flow		Flow Rate	Turbidity from side						Comp-letely Clear from top	Remarks
		mL.	sec.	mL/s	Very Dark	Dark	Moderately Dark	Slightly Dark	Barely Visible	Completely Clear		
12:48	2"	10	23	0.43					√			
		10	23	0.43						√		
		25	53	0.47						√		
12:58		25	53	0.47						√	√	
13:00	7"	25	22	1.14					√			
		25	20	1.25						√		
		25	21	1.19						√		
13:05		25	19	1.32						√	√	
13:06	15"	50	24	2.08					√			
		50	21	2.38						√		
		50	21	2.38						√	√	
13:11		50	21	2.38						√	√	Classificat-ion:ND3