SLOPE STABILITY ANALYSIS OF EMBANMENT FILL MATERIAL FROM NORTH CAROLINA COASTAL REGION

by

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ABSTRACT

ASHLEY CATE MCGOVERN, Slope stability analysis of embankment fill material from North Carolina coastal region. (Under the direction of DR. MIGUEL PANDO)

This research involved a comprehensive experimental program was carried out to study compaction characteristics and associated engineering properties of test soils from three actual NCDOT highway projects located within the coastal geologic region of NC. The laboratory testing program helped determine engineering properties such as shear strength parameters, Su, friction, cohesion, and stiffness, as well as classification under United Soil Classification System (USCS). The information gathered from this testing was used to analyze the performance of different highway embankment geometries in terms of slope stability. This analytical component was used to assess the suitability of the current NCDOT selection criteria and to develop recommendations toward an improved criterion. A survey of current practices, in terms of material selection for highway embankments was also carried out as part of this research to set a guidepost showing how NCDOT embankment material compares to the rest of the nation.

The slope stability analyses involved using the slope stability analysis software Slide 7. The different values of the factor of safety computed as a function of engineering properties based on material type, compaction characteristics, and embankment geometry were used to perform a statistical regression analysis with the statistical analysis software SPSS. The regressions obtained were found to be useful to estimate an overall embankment stability number or index that can be used as a basis for an improved material selection process.

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CHAPTER 1: INTRODUCTION

1.1 Overview of the Problem and Motivation

The performance of highway embankments depends primarily on the soil materials used for its construction and the mode of construction. It also depends on other factors such as embankment geometry, (e.g., fill height, embankment width, and side slopes), foundation conditions, traffic loading, and other factors. The main focus of this research is on the soil materials used in the construction of highway embankments. The focus is specifically on soil material, the selection specifications and criteria as well as the specifications for placement and construction. This research was motivated by a research need statement generated by the North Carolina Department of Transportation (NCDOT) and its interest in improving their current material criteria specifications for highway embankment construction.

The current specifications as per the 2012 NCDOT Standard Specifications in Sections 225, 226, 230, and 240 (NCDOT, 2012), uses the soil's Plasticity Index (PI) as the main selection criterion for the suitability of embankment fill material. Although the PI is a useful soil index property, is it is difficult to relate this parameter to a specific soil type. Figure 1.1 below shows the Atterberg plasticity chart commonly used to classify soils using the Unified Soil Classification System (ASTM, 2011). Within this chart there is a rectangular area that shows the NCDOT specification for embankment soil selection for the NC Piedmont region which calls for use of soils with a PI of 25 or less. As shown in Figure 1.1, the highlighted area of acceptable soils includes several types of soil types including: low plastic clays or silts (CL, ML, CL-ML), low plastic organic soils (OL), and high plastic silts and clays (MH, CH). This wide range of soil types is too broad to ensure selection of adequate soil materials for highway embankment construction. Therefore, the current borrow criteria solely based on PI seems to be insufficient to be used effectively in highway embankment construction projects. Other material indices or physical properties to be considered are the unit weight of soil, grain size distribution, shear strength, and compressibility – all of which can affect the performance of the borrow material.

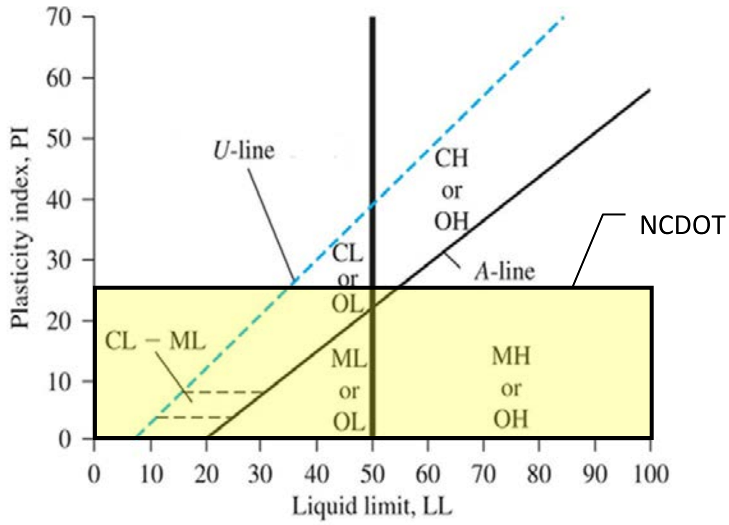


Figure 1.1: Plasticity index chart showing band of soils meeting the current NCDOT PI≤25 borrow material selection criterion

The research need statement by the NCDOT is based on the interest in improving and possibly expanding upon this PI-based borrow material selection criteria. It has been reported that the existing selection criteria has resulted in construction issues such as difficulty of compaction, instability and or slope failures. It has also caused construction delays, disputes, claims and cost overruns. Therefore, this research calls for a rational and scientific study to develop “optimum criteria” for borrow material and unclassified excavation material for embankment construction. The degree of influence of various factors on optimum performance need to be investigated and an engineering analysis of different criteria options should be performed. This MS thesis is part of an ongoing NCDOT funded research project at UNC Charlotte that is focusing on the development of improved highway material selection criteria for soils in the different geologic regions of NC. This thesis gives particular attention to presenting a study of soils representative of the coastal geologic region of NC. The outcome of this project was new, improved material selection criterion that will result in a more rational and economical design of highway earth works, reduce long-term maintenance cost of highways, and avoid construction disputes, claims, delays and cost overruns.

1.2 Objective

The main objective of this research was to develop a rational improved criterion that facilitates the utilization of borrow material for highway embankment construction for the coastal geologic region of NC. This was achieved through a comprehensive laboratory program involving testing of representative soil samples retrieved from three actual NCDOT projects located within the NC coastal region.

The laboratory testing program, performed on the representative materials provided by the NCDOT, helped determine engineering properties such as shear strength parameters, S_u, friction, cohesion, and stiffness, as well as classification under United Soil Classification System (USCS) and the American Association of State Highway and Transportation Officials (AASHTO) guidelines. The information gathered from this testing was used to analyze the performance of different highway embankment geometries in terms of slope stability. This analytical component was used to assess the suitability of the current NCDOT selection criteria and to develop recommendations toward an improved criterion. A survey of current practices, in terms of material selection for highway embankments was also carried out as part of this research to set a guidepost showing how NCDOT embankment material compares to the rest of the nation.

More specific and completed objectives of this research include:

- A survey of the current practices followed by U.S. state DOT's in the selection of embankment construction material
- Laboratory tests on borrow soils that represent current NCDOT construction practices from regions throughout NC. These investigations included material classification as well as stress-strain behavior, deformation and settlement.
- Development of charts and statistical analysis to determine the key engineering properties of the embankment soils based on correlations with slope stability factors of safety.
- Development of contours of index properties of compacted soils to determine acceptable zones within the water content and dry-unit weight graphs.

1.3 Overview of Thesis

This MS thesis is organized into eight chapters and four appendices. Chapter 2 presents a background on compaction and shear strength of compacted soils as it relates to the resulting engineering properties of the soils used for highway construction as it pertains to the NCDOT Embankment Project.

Chapter 3 reviews the current NCDOT standards and specifications for material selection for highway construction. It also presents the results of the survey carried out as part of this research which summarizes embankment construction specifications used by other US State Departments of Transportation (DOT). This also reviews the current U.S. State DOT specifications coverage of slopes stability and embankment loading conditions.

Chapter 4 describes the test materials received from three NCDOT highway embankment projects located in the coastal region of NC. The material description includes results of soil index tests carried out as part of this study in order to classify the soils using USCS and AASHTO soil classification systems.

Chapter 5 discusses the procedures of the main laboratory tests carried out to assess engineering properties and behavior of the three test soils investigated. Specifically, it describes soil compaction and shear strength properties measured using unconsolidated undrained (UU) triaxial compression tests and direct shear testing.

Chapter 6 presents and discusses the results of the experimental program for each test soil. The influence of compaction characteristics, defined in terms of water content of the test soil during compaction. In addition, the amount of compaction energy or effort measured in terms of the resulting dry unit weight of the compacted soil. This is followed by a discussion on the resulting engineering properties (shear strength and stiffness).

Chapter 7 presents the results of an analytical study of the slope stability of different highway embankment geometries, involving four side slopes and four embankment heights, using the engineering properties reported in Chapter 6 for the three coastal soils investigated. The slope stability analyses involved using the slope stability analysis software *Slide 7* (Rocscience, 2015). The different values of the factor of safety as a function of engineering properties based on material type, compaction characteristics, and embankment geometry were used to perform a statistical regression analysis with SPSS statistical analysis software (IBM, 2015). Summaries, conclusions, and recommendations for future work are given in Chapter 8.

This thesis also includes Appendices A through D, which provide additional details of the experimental program such as lab data sheets, and sample photos, as well as details regarding the slope stability analyses and SPSS regression analysis.

CHAPTER 2: BACKGROUND

2.1 Introduction

This MS thesis was carried out as part of an ongoing NCDOT research project entitled: “Improvement of Material Criteria for Highway Embankment Construction (NCDOT RP 2015-05)”. This chapter presents background information related to an NCDOT research project focused on improving soil material selection and placement in roadway embankment specifications. This project background is presented to provide the required context of where this MS thesis fits within the overall NCDOT research project.

Additionally, this chapter provides background information on several topics that relate to the scope of the project. The topics reviewed in this chapter are:

- Compaction of soils (Section 2.3)
- Influence of compaction procedures in the resultant engineering properties of compacted soils (Section 2.4)
- Shear strength testing (Section 2.5)

The reader familiar with the above topics may wish to skip these sections.

2.2 Overview of Improvement of Material Criteria for Highway Embankment Construction (NCDOT RP 2015-05)

This thesis was carried out as part of the NCDOT research project RP2015-05. Its main objective was to assess the current soil selection criteria for highway embankment construction and to improve such criteria. NCDOT currently uses Standard Specifications Section 1018-2 (2012), which primarily uses plasticity index (PI) of the borrow material as a basis for the material selection. This is shown graphically in Figure 2.1. This figure shows the Atterberg plasticity chart used in the USCS. The figure shows two rectangular regions that correspond the to the NCDOT highway embankment soil selection criteria. The taller region corresponds to the acceptance region for NC Piedmont region projects where the soil must have a PI of 25 or less. The second rectangular region corresponds to the NCDOT specification for coastal region projects where the PI of the soil must be 15 or lower.

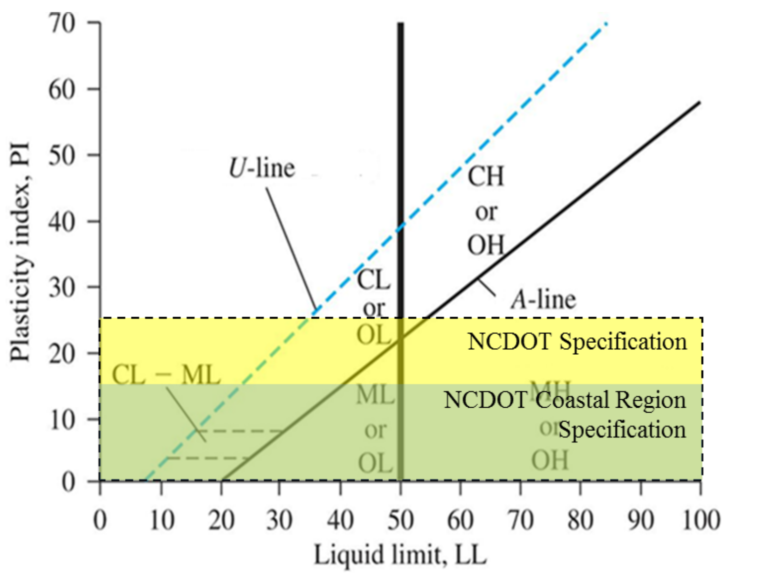


Figure 2.1: NCDOT borrow material specification

In addition to the PI requirement, NCDOT Standard Specifications Section 235 (2012) states that layers of the embankment must be compacted to a minimum dry unit weight to achieve a relative compaction of at least 95% based on the maximum dry unit weight obtained from a compaction test performed in accordance with AASHTO T-99 (equivalent to the Standard Proctor test). In regards to the water of soil during embankment construction, it was noted within the same section that the water content should be increased or decreased to produce the maximum unit weight that will proved a stable grade. There is no further specifications regarding the acceptable range of water contents of the borrow material to be used for highway construction.

NCDOT has reported that the current specifications have resulted in construction issues reported in several NCDOT highway embankment construction projects. Projects that have used borrow materials meeting current NCDOT embankment soil selection specifications have resulted in problems during construction such as compaction difficulty, slope stability failures, construction delays, disputes, claims and cost overruns. Therefore, the main goal of this NCDOT research project is to develop an improved material selection criterion for borrow material and unclassified excavation material for highway embankment construction for the three main geological regions of the state of North Carolina. As noted earlier this MS thesis, which is part of the larger NCDOT project, only focuses on soils from the NC coastal geological region.

The main objective of the NCDOT project is to develop a set of improved material criteria for highway embankment material. The specific objectives are:

1. To survey the current practices followed by U.S. state DOTs in the selection of embankment construction material.

2. To conduct comprehensive laboratory investigation on compacted borrow soils that are representative of NCDOT current construction practices.
3. To develop charts and nomographs estimating key engineering properties based on correlations with index properties.
4. Develop contours defining acceptable zones for slope stability and deformation design considerations in the water content-dry unit weight domain.
5. To develop "overall acceptable zones" for the compacted embankment soils that are based on slope stability (strength) and deformation (stiffness) criteria.
6. To conduct a parametric study of all paramount factors affecting embankment material by means of a detailed regression analyses.
7. To develop a ranking/rating table for the candidate material for embankment construction
8. To determine whether "pond fines" can be used for highway embankment construction material (stand-alone report).
9. Develop a comprehensive set of guidelines for selection and field quality control of highway embankment materials and its suggested compaction conditions.

This MS thesis does not address all of the above objectives of the larger NCDOT project. It primarily focused on the investigation of actual borrow soils obtained from three highway embankment projects located within the coastal region of North Carolina. In addition to investigating these three coastal region soils, the thesis author carried out a survey of current practices followed by U.S. state DOTs in the selection of embankment construction material.

For the three test coastal soils investigated, the following main tasks were performed:

- Conducted a comprehensive geotechnical laboratory investigation to evaluate relevant engineering properties of compacted borrow soils obtained from three NCDOT highway embankment projects.
- Developed charts and contours of key engineering properties of select compacted soils to define acceptable zones for slope stability and deformation design considerations in the water content-dry unit weight domain.
- Performed slope stability analysis to determine the factor of safety related to each point within the water content-dry unit weight curves, as well as the changing embankment dimensions commonly used in embankment construction.
- Performed statistical regression analysis of the slope stability factor of safety and soil engineering properties to determine if a linear relationship can be determined.

2.3 The Compaction of Soils as it Relates to Highway Embankment Construction

Compaction of soils is a classical subject of geotechnical engineering. One of the first studies on this matter was carried out by Proctor (1933) who found an important relationship between the water content of the compacted soil and the achieved dry unit weight. He found that the relationship was curved as shown in Figure 3.1. This figure shows two plots, each representing a different compaction effort or energy. Independent of the compaction energy used, it can be seen in this plot that the dry unit weight initially increases with increasing water content of the compacted soil. This is the case until a maximum dry unit weight is reach at what he termed the optimum water content. Beyond this optimum water content, the dry densities decrease with increasing water content. This parabolic variation of the dry unit weight with the water content of the compacted soil is related to the variation of the resulting arrangement of the soil particles when compacted. It should be noted that the different points that form the compaction curve (parabolic shaped curves with seven points each) are all soils with the phases (i.e., soils, water, and air). In this same figure a curve showing the relationship between computed equivalent dry unit weight and water content for a saturated soil is shown. This line is often called the zero air void line. The position of the zero air void line is a function of the specific gravity of the soil. It is worth pointing out that although this curve represents a fully saturated state (S=100%) the dry unit weight coordinates correspond to computed values that would be obtained if the saturated sample were to be placed in an oven and dried without allowing the total volume to change.

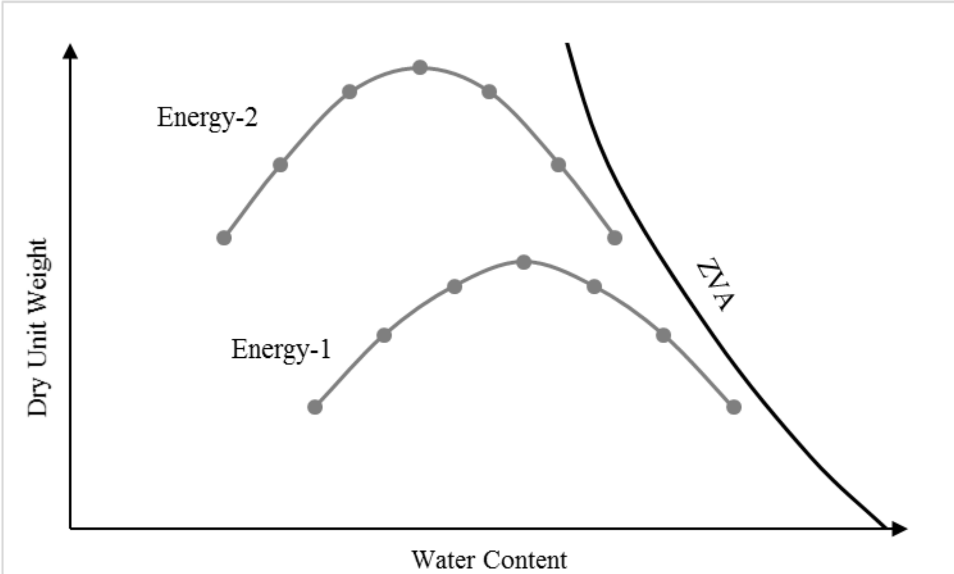


Figure 2.2: Compaction curve (adapted from Proctor, 1933)

Referring to Figure 2.2, it can be seen that the curve labeled Energy-1 is lower and to the right of the curve labeled Energy-2. This is because the compaction effort of Energy-2 is higher than Energy-1 which results in soil samples that are denser. The main observations when comparing the influence of the compaction energy is that by increasing compaction energy, the optimum water content decreases while the maximum dry unit weight increases.

Many researchers have found that the particle arrangements, (i.e. fabric) of compacted soils is heavily influenced by the water content used during compaction. This in turn influences the resulting engineering properties. Figure 2.3 shows how the particle arrangement, or fabric, varies with water content.

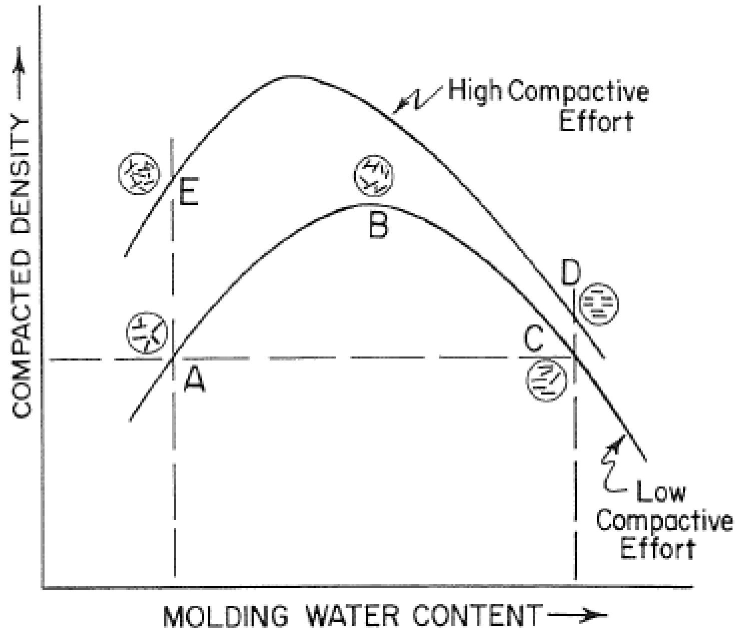


Figure 2.3: Effects of compaction on structure (adapted from Lambe, 1958)

Soil compacted with a low water content is observed to have low to no plasticity, appearing to be very hard and firm. In this state, there remains a large number of voids. To fill the voids, compaction must overcome the frictional resistance between the particles. In finer grained cohesive soils, a film of water surrounds each grain. As the thickness of the film increases, there is a direct effect on the efficiency of soil compaction. The driest stage, called the Hydration stage, is when the effective compaction is most difficult. Water films during this stage are less than 11 millionth of an inch thick (Hogentogler 1936).

Lambe's structural theory discusses the physical-chemical nature of clay, and the effect of the presence of water. Clay particles carry a negative change, which allows for a "double layer" surrounding the particle when immersed in water. This double layer influences the arrangement of the clay particles. When clay is mixed at a low water content, the double layer cannot develop fully. This low quantity of water in fact increases the electrolyte content which depresses the double layer. This depression in the double layer allows for flocculation, which a poor arrangement of the particles and a lower unit weight.

Olson discusses the influence of compaction on effective stress, pore water pressure, and shearing deformations of soil. Loose soil in the dry of optimum state has negative pore water pressure, with little influence on effective stress. During the compaction process, with the addition of water, negative pore-water pressures develop, which increases the effective stress.

Increasing the water content allows lubrication to occur, as well as a capillary force, which draws the particles closer together, thereby increasing the amount of soil in a fixed volume. This increase in the amount of soil particles will increase the resultant unit weight. Soil at this optimum state becomes more plastic, filling remaining voids between particles with water and air that cannot be removed from the compaction force Proctor (1933). As the water increases, the film thickness increases, enabling lubrication. During this lubrication stage, the water film is between 11 to 14 millionth of an inch thick, with more efficient compaction occurring. At maximum unit weight, the film of water reaches a thickness which gives a low cohesive strength at the points of contact that just fails to balance the compaction force (Hogentogler 1936).

Increasing the water content increases the double layer around the particles, reducing the electrolyte concentration. This increase in the double layer decreases the chance of flocculation, allowing a more orderly arrangement of clay particles, increasing the unit weight (Lambe, 1958). Lambe also discusses the effect of compaction effort on the soil structure, stating that an increase in the compactive effort better aligns the particles in a parallel state, and increases the unit weight.

Compaction characteristics of fine grain soils are normally obtained through Proctor compaction, which requires time and material that may not always be available in the field. Upon review of many types of soils, Gertug and Sridharan found a correlation between the plastic limit (PL) of each soil and optimum water content (OMC). This correlation was simply reduced to OMC = 0.92(PL). They also determined the optimum (maximum) dry unit weight correlated to the dry unit weight at the plastic limit water content, $\gamma_{dmax} = 0.98\ \gamma_{dwP}$ (Gertug and Sridharan, 2002).

Further addition of water brings the soil into a saturated state. The number of voids within the compacted soil increases due to water being forced into the sample. This results in a less dense sample, replacing soil in the fixed volume with water (Proctor, 1933). An increase in the double layer even further through the addition of more water, which decreases the attraction between the clay particles. A more orderly arrangement of the particles is created through the addition of water, although the unit weight decreases due to the volume of water lowering the amount of soil available in a fixed volume (Lambe 1958).

When distributions of pore sizes for soil are at equal porosities (void ratio) wet and dry of optimum varied. Dry of optimum soils have larger pores than the wet of optimum soils. An increase of compaction effort at a constant water content dry of optimum causes the pore size to reduce. An increase of compaction effort of wet of optimum soil only slightly reduces pore distribution (Bhasin 1975).

Grain size distribution appears to have little to do with the distribution of pore size. During compaction, gross pores are eliminated, while pores similar in diameter to the particle diameters are reduced. Fine pores are slightly enlarged by higher compaction energies. Pore size distribution remains relevant to the soil type and sample preparation (Sridharen, et al. 1971).

2.3.1 Summary of Compaction of Soils

Extensive research has been done in regards to the compaction of soil and the effects compaction has on the resulting structure and strength of the material. Soil compacted with a low water content displays low densities with large air voids. Soil compacted at optimum water content displays high densities, little to no voids, and better arrangement of the particles. Soil compacted with water above the optimum water content displays swelling from the excess water content, and lower densities.

2.4 Influence of Compaction on Engineering Properties

Prior to construction, a subsurface investigation was performed to determine the properties of the soil, location of groundwater, and identify any future issues that may arise due to the soil conditions. Depending upon the plans of the construction, the soil on site may be used for fill material if determined to be suitable as per the specification. In many cases, funds and time are limited when determining the engineering properties of the soil, which can lead to assumptions made by engineers instead of performing additional testing. USCS has published a table to help these engineers when determining typical cohesion and friction values based on the soil classification.

Table 2.1: USCS recommended typical values for cohesion and friction (Geotechdata.info)

USCS Soil-Class	Description	Cohesion (kPa)	Friction Angle
GW	well-graded gravel, fine to coarse gravel	0	40
GP	poorly graded gravel	0	38
GM	silty gravel	0	36
GC	clayey gravel	0	34
GM-GL	silty gravel	0	35
GC-GL	clayey gravel with many fines	3	29
SW	clayey gravel with many fines	0	38
SP	poorly graded sand	0	36
SM	silty sand	0	34
SC	clayey sand	0	32
SM-SL	silty sand with many fines	0	34
SC-CL	clayey sand with many fines	5	28
ML	silt	0	33
CL	clay of low plasticity, lean clay	20	27
CH	clay of high plasticity, fat clay	25	22
OL	organic silt, organic clay	10	25
OH	organic clay, organic silt	10	22
MH	silt of high plasticity, elastic silt	5	24

Typical values of dry unit weight, Young's modulus, void ratio, and soil bearing capacity are also supplied by USCS for reference. The values supplied are based on average values collected from literature and geotechnical projects. USCS recommends that they be adapted depending on the needs of the project.

Table 2.2: USCS recommended typical values for dry-unit weight (Geotechdata.info)

USCS Soil-Class	Description	Average Value (kN/m^3)
GW	well-graded gravel, fine to coarse gravel	21 ± 1
GP	poorly graded gravel	20.5 ± 1
GM	silty gravel	21.5 ± 1
GC	clayey gravel	19.5 ± 1.5
SW	clayey gravel with many fines	20.5 ± 2
SP	poorly graded sand	19.5 ± 2
SM	silty sand	20.5 ± 2.5
SC	clayey sand	18.5 ± 1.5

Table 2.3: USCS recommended typical values for Young's modulus (MPa) (Geotechdata.info)

USCS Soil-Class	Description	Loose	Medium	Dense
GW, SW	Gravels/Sand well-graded	30-80	80-160	160-320
SP	Sand, uniform	10 - 30	30-50	50-80
GM, SM	Sand/Gravel silty	7 - 12	12-20	20-30

Table 2.4: USCS recommended typical values for Young's modulus (MPa) (Geotechdata.info)

USCS Soil-Class	Description	Very soft to soft	Medium	Stiff to very stiff	Hard
ML	Silts with slight plasticity	2.5 - 8	10 - 15	15 -40	40 - 80
ML, CL	Silts with low plasticity	1.5 - 6	6 -10	10 - 30	30 -60
CL	Clays with low-medium plasticity	0.5 - 5	5 -8	8 - 30	30 - 70
CH	Clays with high plasticity	0.35 - 4	4 -7	7 - 20	20 - 32
OL	Organic silts	-	0.5 -5	-	-

2.5 Shear Strength Testing

2.5.1 Undrained Strength of Compacted Clays

Factors that influence the unconsolidated-undrained (UU) strength of compacted soils in the triaxial testing are: compaction water content, dry unit weight, and minor principle stress. Strength of compacted clays decreases as the water content increases. Strength increases as the dry unit weight increases. Strength increases as the minor principle stress increases, until confining pressure reaches a point that the sample became fully saturated. This occurs when the confining pressure is high enough to cause the air in the sample voids to dissolve into the water (Rutledge 1947).

A silty clay was compacted to a constant dry unit weight. When water content was high and the sample reached near saturation, a small increase in additional pressure dissolved the air into the pores. Further increases in confining pressure was taken on by the pore water and not by the soil structure. When samples have a lower compaction water content, the failure envelope slopes upward and significantly higher pressures are required to compress the specimen voids enough to dissolve the air (Casagrande and Hirshfeld 1960).

In samples with similar structure and compaction water content, strength increases with an increase in unit weight. Undrained strength may also decrease with increasing unit weight at a constant water content, depending on the failure criterion. If failure criterion of 25% strain is adopted, undrained strength increases with increasing unit weight. If a failure criterion of 5% strain is adopted, the undrained strength increases with increasing unit weight up to a point, and then decreases with further increases in unit weight. When significant changes in the soil structure takes place, the strength is significantly reduced despite the increase in unit weight (Seed and Chan 1959).

2.5.2 Summary

It is apparent from the extensive research performed on engineering properties of compacted soils that the strength and stiffness of the compacted soil increases with increasing dry unit weight. However, for a given soil dry unit weight, the strength and stiffness will decrease with increasing water content. Therefore, specifications for compacted engineered fills typically involve specification of the relative compaction (i.e., level of dry unit weight with respect to a maximum value obtained from a specific compaction test) as well as specification of the acceptable range of the placement water contents. This water content range is usually specified with respect to the optimum water content obtained from the same specific compaction test used to define the relative compaction of the soil. This is relevant to this project as the current NCDOT specifications define a minimum relative compaction requirement for highway embankment construction but are less specific when it comes to water content placement requirements.

CHAPTER 3: LITERATURE REVIEW

3.1 Introduction

This chapter presents a detailed literature review on the main subjects that relate to the scope of this thesis as follows:

- Specifications for selecting borrow soils for highway embankments
- Estimation of engineering properties for highway construction
- Slope stability failures of highway embankments

The following sections summarize the results of the literature review for each of these subject.

3.2 Specifications for Selecting Borrow Soils for Highway Embankments

3.2.1 Embankment Construction Specifications

Embankment construction specifications from 48 US Departments of Transportation were reviewed to compile material criteria that is commonly used for selection of fill material for highway embankment construction. Upon reviewing 48 states, there appeared to be a standard practice to refer to an Engineer's judgement to determine what fill soil would be considered acceptable. As part of this study, a survey questionnaire directed to DOT engineers was developed to help confirm the state of practice of highway embankment material selection and placement used across the nation. A copy of the survey is provided in Appendix A.

In addition to referring to an engineer's judgment, several states will specify specific classification requirements for the embankment fill material. For example, Figure 2.1 shows the 14 states highlighted in yellow that currently refer to AASHTO Standard M-57 – Specification for Materials for Embankments and Subgrade (AASHTO, 2012) or that specify gradation or sieve requirements for the embankment material.

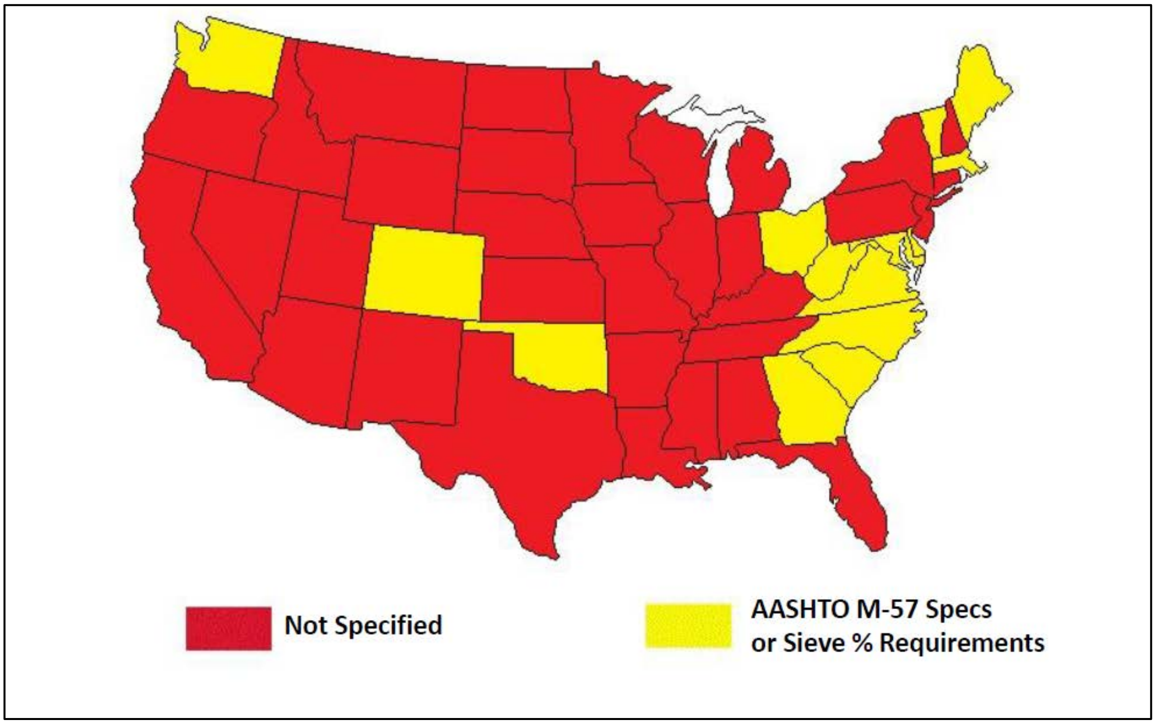


Figure 3.1: Map showing US state DOT's with AASHTO M-57 and/or sieve requirements

3.2.2 Plasticity Index Requirement

From the literature review, six states have specifications on the required plasticity index (PI) of the material used for embankment construction. The six US State DOTs which indicated PI specifications for embankment fill material are shown in Figure 3.2. For instance, Arizona limits the plasticity index to a maximum of 15, but only in the areas built around bridge abutments. Louisiana set a maximum of 20 and 25, for what they classify as usable and selected soil respectively. Oklahoma has set the maximum plasticity index to 15 throughout the embankment. Texas has set maximum and minimum ranges of compaction energy dependent upon the PI value of the material. For example, material with a PI less than or equal to 15, the compaction energy must result in a dry unit weight within 98% of the maximum dry unit weight. Instead of setting a maximum PI, Delaware has specified a maximum liquid limit (LL) of 50. North Carolina currently specifies that the PI of the fill material must stay below 15 in coastal regions, and less than 25 in the Piedmont and western regions.

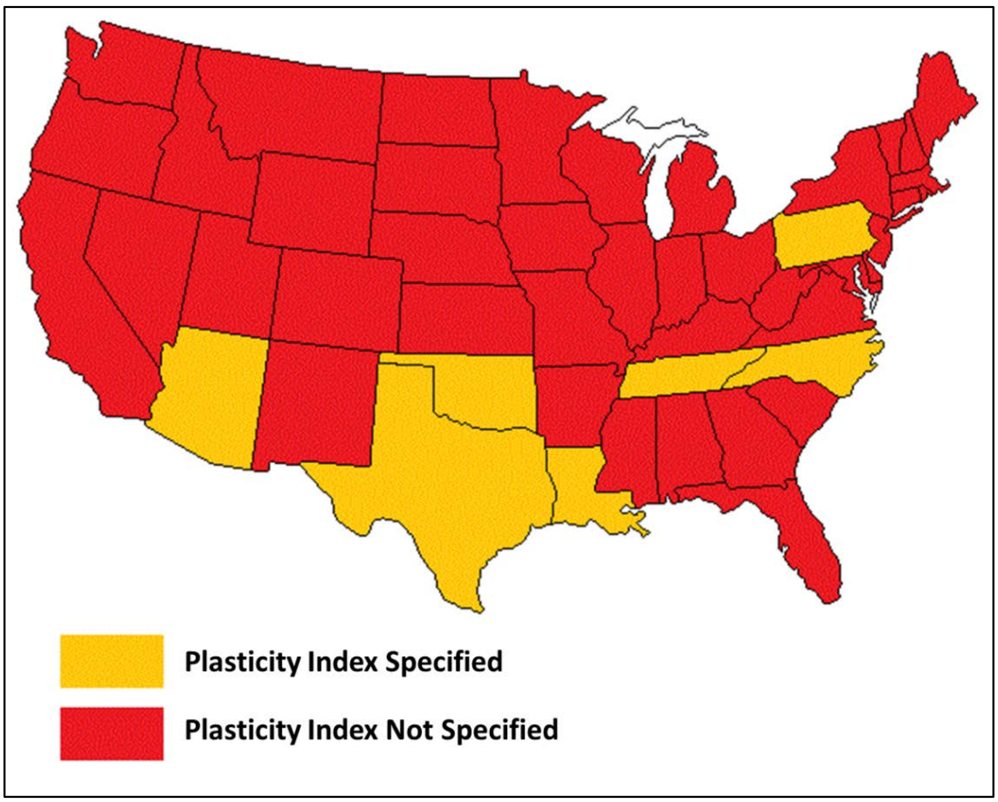


Figure 3.2: State DOT plasticity index requirements

3.2.3 Placement Water Requirement

Twenty-four states have specified a water range near the optimum water content. 12 of those states have specified a range of ±2% with respect to the optimum water content measured using either the Standard or Modified Proctor Compaction Test.

3.2.4 Placement Unit Weight Requirement

Six states have specifications limiting the minimum dry unit weight of the material used in embankment construction. Of these six states, Colorado, Ohio, and Delaware limit the unit weight to a minimum of 95 lb/ft^3. Michigan and Pennsylvania limit the dry unit weight to a minimum of 90 lb/ft^3. Maryland limits the minimum unit weight to 100 lb/ft^3.

3.2.5 Placement Compaction Energy Requirement

Twenty-five states have specified that the relative dry unit weight of the embankment soil should be compacted to an energy of at least 95% of the maximum unit weight determined through laboratory testing. Of all the states compared, 24 specify the exact test method. Nine of those states specify that the maximum unit weight be determined through AASHTO T 180 (modified), while the other 15 states specify the use of AASHTO T 99 (Standard Proctor). States such as Alabama, Colorado, and Arkansas allow both compaction test methods depending on the proposed fill soil.

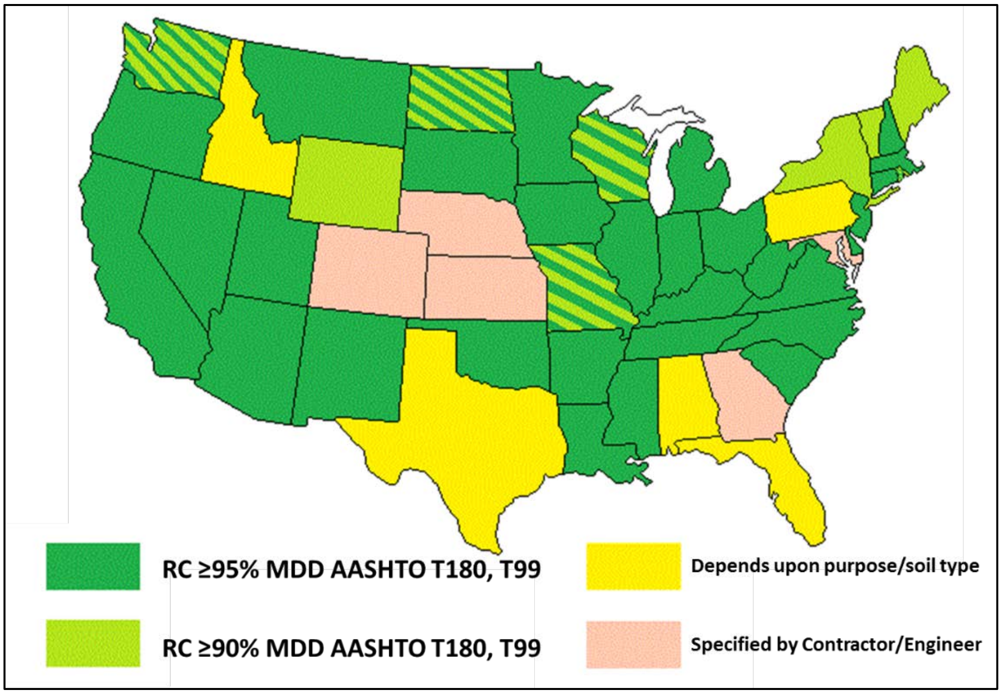


Figure 3.3: State DOT unit weight requirements

3.2.6 Lift Thickness Requirement

A minimum loose lift thickness of eight inches is specified by 31 US DOTs, as shown in Figure 3.4. The remaining DOTs have specifications that specify varying lift thicknesses. For instance, Hawaii specifications state that the top two feet of an embankment must be constructed using loose lift thickness of at most six inches, with the remaining depth having lifts with a maximum loose thickness of nine inches. Louisiana specifies that plastic soils be placed in loose lifts of no more than 12 inches, and granular material is limited to a loose thickness of no more than15 inches. All states except New York and Tennessee have specifications set for the required lift thicknesses in embankment construction.

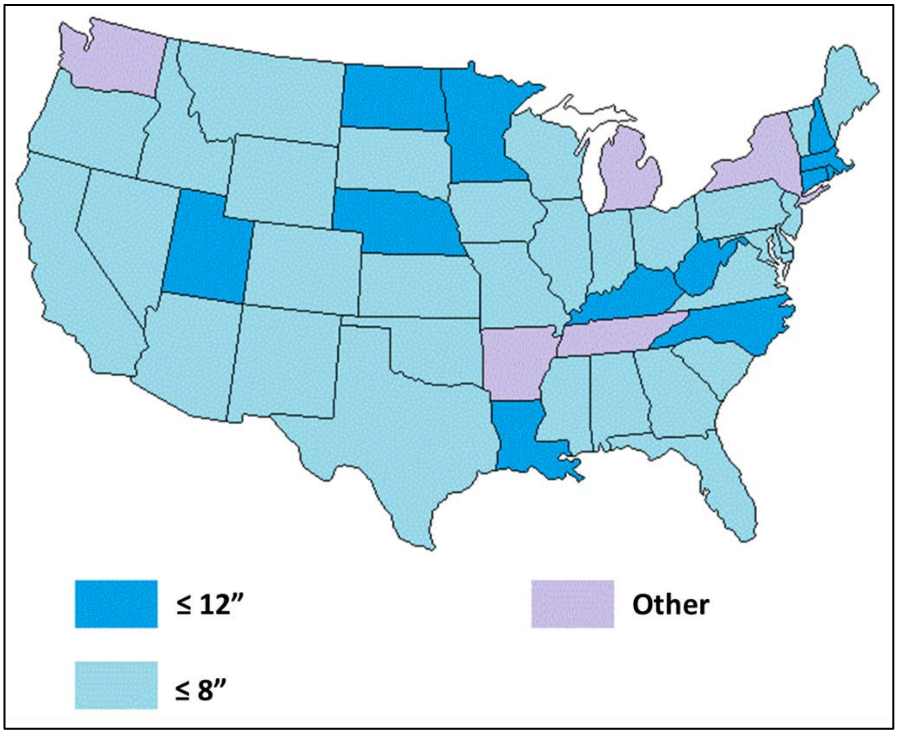


Figure 3.4: State DOT lift thickness requirements

3.2.7 Slope Stability Factors of Safety

Slope stability factors of safety from 1.3 to 1.4 are considered acceptable by the National Cooperative Highway Research Program (NCHRP, 1978). The U.S. Army Corps of Engineers (USACOE, 2003) suggest a minimum safety factor of 1.3 for embankments at the end of construction.

Review of current Department of Transportation embankment construction specifications of all U.S. states was performed to determine current, if any, requirements as to the slope stability factor of safety. All states with their slope requirements can be found in Table 3.2. Some states go further than setting a minimum requirement, and correlate factor of safety requirements to other embankment properties, such as height and slope.

Table 3.1: State DOT slope stability factor of safety

State	Description
Texas	- FS=1.3 for global stability of a slope for both long term and short term performance - For slope or walls that support abutment, buildings, critical utilities use FS=1.5
California	- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25
New York	- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25
Washington	- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25
Iowa	- Typically minimum factors of safety for new embankment slope design ranges from 1.3 to 1.5
Oregon	- All embankments not supporting or potentially impacting structures shall have a minimum safety factor of 1.25

As shown in Table 3.2, the state of Virginia specifies a minimum factor of safety as dependent on whether testing was performed on the material being used in the construction of the embankment (VDOT, 2014).

Table 3.2: Minimum slope stability factor of safety recommended by Virginia DOT (2014)

Basis for Soil Parameters	Factor of Safety	
	Critical Slope	Non-Critical Slope
Site specific in-situ laboratory strength tests of soils	1.5	1.3
No Site specific in-situ laboratory strength tests of soils	N/A	1.5

The Texas DOT specifies a minimum slope stability safety factor of 1.3, although this is dependent on the side slope inclination and the maximum plasticity index of the backfill soil used to build the embankment as determined from laboratory testing. The minimum side slope inclination requirements by Texas DOT are shown in Table 3.3

Table 3.3: Texas DOT (2012) plasticity index range for exposed side slopes required for FS = 1.3 for long term conditions

Maximum Side Slope X:1 (H:V)	Plasticity Index, PI (%)
2.5:1	<5
3.0:1	<20
3.5:1	<35
4.0:1	<55
4.5:1	<85

3.2.8 Side Slope Requirements

NCHRP (1989) states that the standard slope of an embankment is dependent upon the depth of fill and proximity to the roadway. Figure 3.5 illustrates this standard. For an embankment height greater than 30 feet, the side slope should not be any steeper than 2H:1V within 20 feet of the roadway. If the embankment height is less than 30 feet, the slope shall be no steeper than 6H:1V.

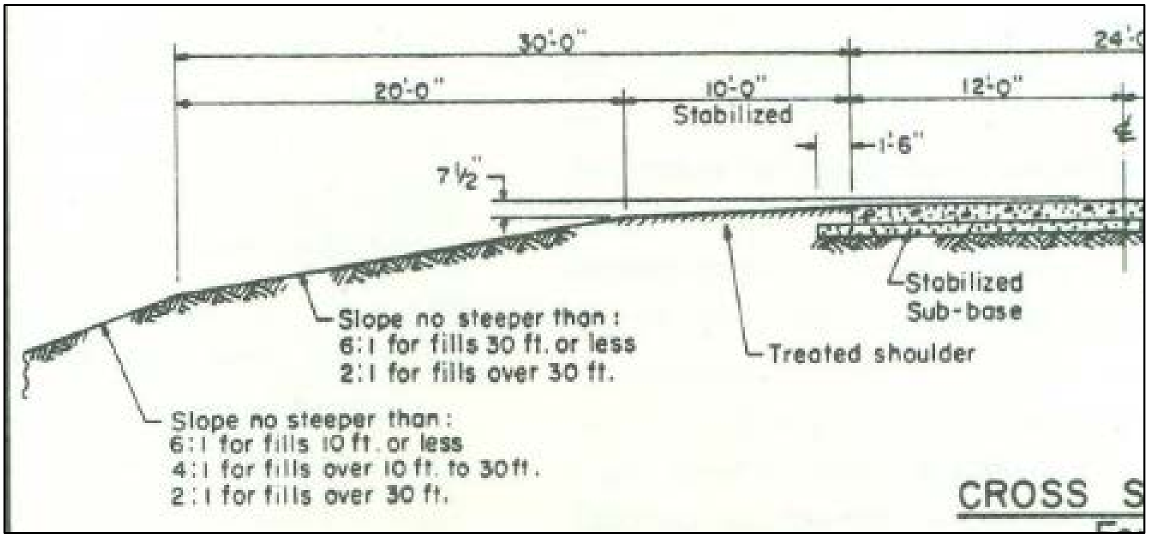


Figure 3.5: NCHRP roadway standard (NCHRP, 1978)

The American Association of State Highway and Transportation Officials (AASHTO, 2002) have embankment side slope recommendations based on the terrain topography of the road construction as well as the embankment height. These recommendations are summarized in Table 3.4.

Table 3.4: AASHTO embankment slope and height recommendations (AASHTO, 2002)

Height of Cut or Fill (m)	Type of Terrain		
	Flat or Rolling	Moderately Steep	Steep
0.0 – 1.2	6:1	4:1	4:1
1.2 – 3.0	4:1	4:1	2:1
3.0 – 4.5	4:1	2.5 : 1	1.75 : 1
4.5 – 6.0	2:1	2:1	1.75 : 1
Over 6.0	2:1	2:1	1.75 : 1

3.2.9 Review of Current Department of Transportation Embankment Construction

Review of specifications was performed to determine current, if any, requirements as to the slope of embankments. All states with their slope requirements can be found in Table 3.5.

Table 3.5: State DOT review slope requirements

State	Slope	Notes
California	2H:1V	If height greater than 10 feet, in depth stability analysis is required
Connecticut	2H:1V to 5H:1V	
Idaho	2H:1V	
Maine	2H:1V	Steeper than 2:1 shall be benched
Maryland	4H:1V	Steeper than 1H:4V shall be benched as the embankment is built up
Michigan		Steeper than 1H:6V shall be benched as the embankment is built up
Mississippi	2H:1V	
Nevada		Steeper than 1H:4V shall be benched as the embankment is built up
New York	2H:1V	If height greater than 10 feet, in depth stability analysis is required

Table 3.5: State DOT review slope requirements (continued)

State	Slope	Notes
North Carolina	None	No specifications
Oregon	2H:1V	If height greater than 10 feet, in depth stability analysis is required
South Dakota		Steeper than 1H:4V shall be benched as the embankment is built up
Washington	2H:1V	If height greater than 10 feet, in depth stability analysis is required
Wyoming		Steeper than 1H:4V shall be benched as the embankment is built up

3.2.10 Embankment Surcharge Loading

The loading applied to an embankment post-construction would primarily be caused by traffic. This loading primarily affects the upper few feet of an embankment (NCHRP, 1971). In many cases, the heavy construction equipment used during construction applies more load than what would be seen during the normal lifetime of a highway embankment.

AASHTO models traffic surcharge primarily for instances of designing appropriate pavement material and subgrade. ASSHTO (2002) recommends using 0.67 m (2 ft) of an 18.9 kN-m^3 (120 lb-ft^3) soil layer at the top of embankment to represent traffic load, with a traffic surcharge of 12.6 kPa (263 lb-ft^2). Therefore, the total surcharge representing pavement and traffic is approximately 35 kPa (730 lb/ft^2).

3.3 Slope Stability Failures of Highway Embankments

Many case studies have been published on embankment failures. The most commonly reported failure types have been slope failures, and excessive settlement. Upon review of these case studies, several causes have been identified. Weak foundation issues appear to be the most common issue discovered at the site. Additionally, long-term slope failures occur at sites where consolidation was able to occur after construction was completed. This thesis covers slope failures that occur in the short term.

3.4 Summary

The literature review presented in this chapter highlights the wide variety of specifications used by US DOTs for selected fill material for highway embankment construction and for construction and slope stability analysis and design. NCDOT specifications for embankment material selection and placement in general appeared to be less stringent when compared to most other US DOTs in regards to lift thickness requirements. No NCDOT specifications for side slope dimensions or recommended slope stability factors of safety.

CHAPTER 4: MATERIALS

4.1 Introduction

This chapter describes the three coastal test soils investigated in this project as well as the experimental procedures used to classify these soils.

4.2 Test Soils

The three test soils investigated as part of this research were provided by NCDOT from highway embankments located in the coastal region of North Carolina. The locations of these three test soils can be seen in Figure 4.1. From each location at least one 55-gallon drum was obtained. The test soils were named after the county of North Carolina from where they were recovered, i.e., Wayne, Kinston, and Brunswick.

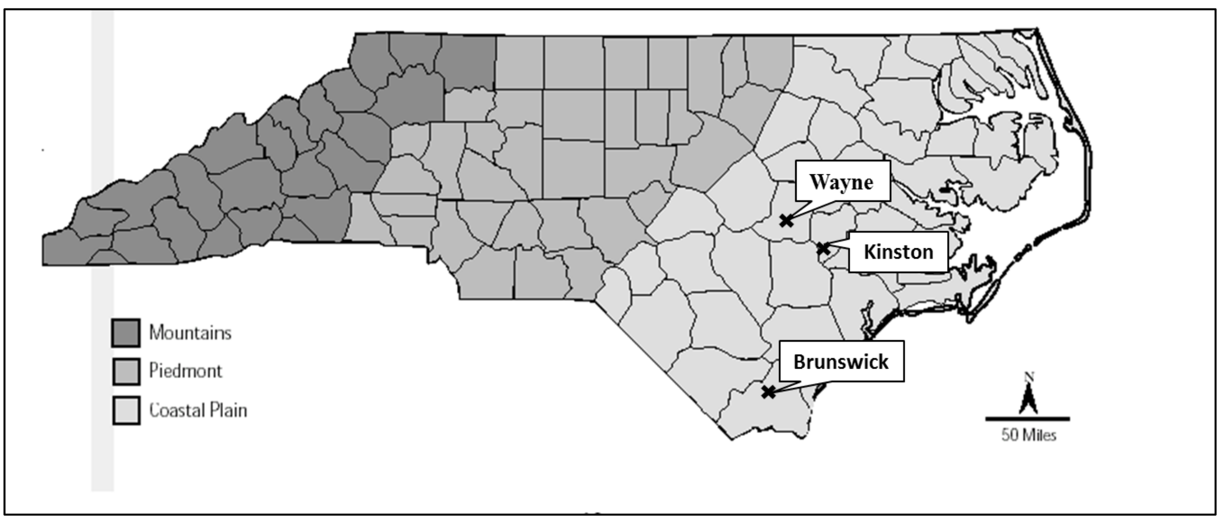


Figure 4.1: North Carolina county locations

The test soils were classified using the American Association of State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System(USCS) classification procedures. The soil classification was determined by performing geotechnical laboratory tests as per recommendations of ASTM D2487 (ASTM, 2011). Table 4.1 summarizes the soil classification and results of the different index tests carried out for the three test soils.

Table 4.1: Summary of index properties and classification of test soils

		Test Soil		
Item Description	ASTM Standard	Wayne	Kinston	Brunswick
USCS Classification	D2484	SM	SM	SP
AASHTO Classification		A-2-4 (0)	A-4 (0)	A-3 (1)
Atterberg Limits	D418-10	NP	NP	NP
Specific Gravity	D854-14	2.72	2.72	2.69
Grain Size Distribution	D422-63			
% of Sand sizes (4.75 mm – 0.075 mm)		87.2	61.7	97.9
% Silt sizes (0.075 – 0.005 mm)		7.8	20.3	1.1
Clay fraction (<0.005 mm)		5	18	1
D_{50} (mm)		0.22	0.11	0.33
C_u		6.3	127.5	2.2
C_c		2.6	5.3	1.1

The Wayne soil consisted of a light tan fine sand with some silts and traces of non-plastic clays. A photo of this test soil is shown in Figure 4.2. The Kinston test soil was a dark greyish brown, non-plastic, silty fine sand with some clays as shown in Figure 4.3 The Brunswick soil was a light tannish brown, poorly graded fine sand, with traces of silt and clay. A representative photo of this soil is shown in Figure 4.4.

Figure 4.2: Wayne test soil

Figure 4.3: Kinston test soil

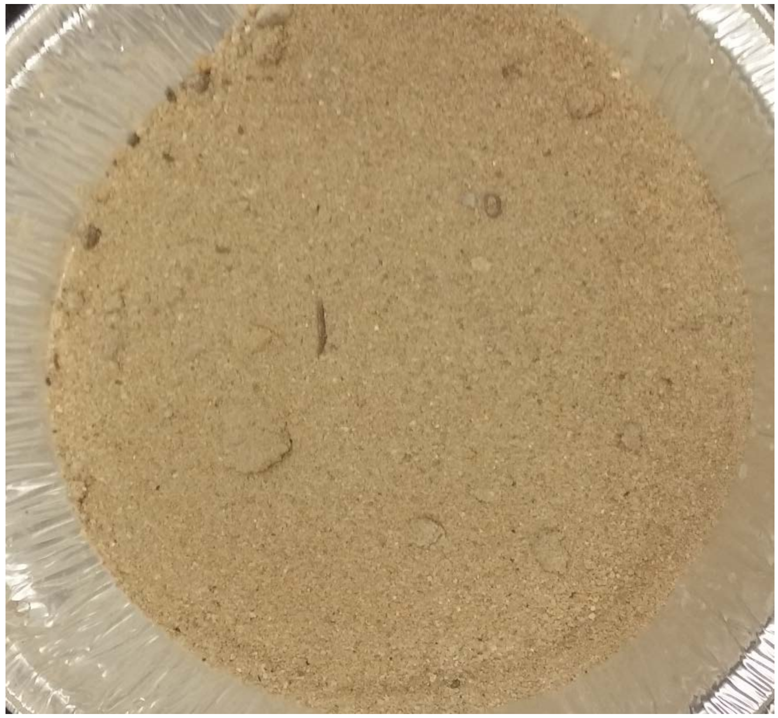


Figure 4.4: Brunswick test soil

4.3 Test Methods to Classify Soils

4.3.1 Grain Size Distribution

Grain size distribution curves for the three test soils were developed following recommendations of ASTM D6913 (ASTM, 2009). A representative sample of each test soil was air dried then mechanically sieved. The mechanical sieving involved using sieve sizes ranging from a maximum opening size of 1 ½ inch (37.5 mm) to a minimum of 0.075 mm (Sieve No. 200). For a determination of the particle distribution for soil particles smaller than that which could go through the No. 200 sieve, by the sedimentation process using a hydrometer. Representative gradation curves for the three test soils are shown in Figure 4.5

Table 4.1 lists the median grain size (D_{50}), and the effective grain size (D_{10}). The coefficient of uniformity ($C_u = D_{60}/D_{10}$) and coefficient of curvature ($Cc = D_{30}^2/(D_{10} \times D_{60})$) were determined and also reported in Table 4.1.

All three test soils had more than 50% of their soil particles by weight larger than the No. 200 sieve (i.e., coarser fraction was more than 50%). The Wayne, Kinston, and Brunswick test soils had USC classifications of SM, SM, and SP, respectively which correspond to predominantly sandy soils.

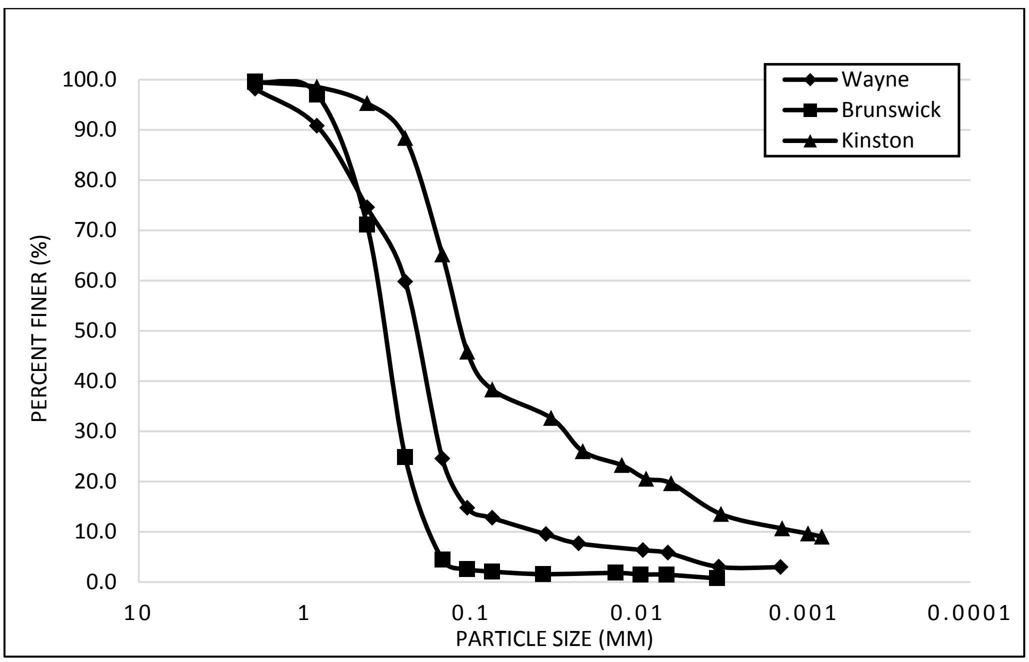


Figure 4.5: Grain size distribution

4.3.2 Atterberg Limits

The Atterberg limits, liquid limit and plastic limit, were determined for each test soil with test procedures in general accordance with ASTM Standard D4318 (ASTM, 2010). The Atterberg limits was evaluated only on the portion of the material passing the No. 40 (0.425 mm) sieve, i.e. the material retained by the No. 40 sieve was removed from each sample to be tested. As per ASTM D4318, the device known as the Casagrande cup, as shown in Figure 4.6, was used to measure the liquid limit of each test soil. Water was added to the soil to obtain a near liquid state. The wetted soil was then placed into the cup using a spatula, and divided using a grooving tool. The cup sits on a mechanical device that drops the cup from a specific height, causing the sample to gradually close the divide. Each drop of the cup is recorded until the divide in soil closes to an approximate ½ inch length. This is performed several times until a water content at 25 drops can be determined. No liquid limit was found to be measurable for any of the three test soils. This is common for non-plastic soils.

Figure 4.6: Casagrande’s liquid limit cup and grooving tool

According to ASTM D4318 the plastic limit is determined by rolling the wetted sample into 1/8-inch threads repeatedly until the soil has just the right water content for which the 1/8 inch threads just begin to form fissures or cracks. This test was attempted several times for all three test soils but the soils were found to be non-plastic.

4.3.3 Specific Gravity

The specific gravity of each test soil was determined in general accordance with the recommendations of ASTM D854-10 (ASTM, 2010). This test method involves using a 250 mL pycnometer that is calibrated in accordance with steps outlined in this same standard. A sample from the fraction of the test soil passing the No. 4 (4.75 mm) sieve is weighed and then funneled into the pycnometer. Then de-aired water is added to create a slurry. A stopper is then placed on the top of the pycnometer, with a hose attached to a vacuum to help remove the air from the sample by continuously agitating the pycnometer with the soil sample. After all the air has been removed, the weight of the pycnometer, water and soil is recorded. Finally, the pycnometer is fully emptied into a pan and then oven-dried for 24 hours at a temperature of 110 ^{o}C. The final dried weight of the soils is then recorded and used to calculate the specific gravity. This procedure was performed several times to reach an average specific gravity value. The specific gravity values obtained for each test soil are reported in Table 4.1.

CHAPTER 5: EXPERIMENTAL PROGRAM

5.1 Introduction

This chapter describes the experimental program of the tests performed on the three test soils to obtain compaction properties and engineering properties such as shear strength and stiffness. The initial proposed work plan called for Undrained Unconsolidated (UU) triaxial compression tests for all three test soils. However due to the non-plastic, poorly graded sand classification of the Brunswick test soil, direct shear tests had to be performed for this material instead of the UU triaxial tests.

5.2 Experimental Program

5.2.1 Water-Dry Unit Weight Relationship from Compaction Testing

Compaction testing was conducted for each test soil to determine the maximum dry unit weight ($\gamma_{d,max}$) and optimum water content (w_{opt}) at three compaction energies. All compaction tests were performed using a 4-inch Proctor mold, with a 5.5-pound hammer for standard energy, and a 10-pound hammer for intermediate and modified energy. The mold and two hammer types used are shown in Figure 5.1.

Figure 5.1: Proctor mold with standard and modified proctor hammers

The lowest compaction energy corresponded to a specific compaction energy of 600 kN.m/m^3 which corresponds to the Standard Proctor test method as per ASTM D698 (ASTM, 2012). The highest compaction energy involved a specific compaction energy 2,700 kN.m/m^3 which corresponds to the Modified Proctor compaction test as per ASTM D1557 (ASTM, 2012). The intermediate compaction effort used was 1,500 kN.m/m^3 which was achieved by using the 10-pound hammer and applying 23 blows per layer using three layers. The compaction test details used for the three selected compaction energies are summarized in Table 5.1.

Table 5.1: Compaction methods used in this research

Compaction Method	**Specific Energy**	**Mold Diameter**	**Hammer Weight**	**Drop Height**	**No. Layers**	**Blows per Layer**
	Lb-ft/ft^3 (kN.m/m^3)	**inch (mm)**	**lb**	**inch (mm)**		
Standard	12400 (600)	4 (101.6)	5.5	12 (305)	3	25
Intermediate	31000 (1500)	4 (101.6)	10	18 (457.2)	3	23
Modified	56000 (2700)	4 (101.6)	10	18 (457.2)	5	25

The compaction curve for each compaction energy was defined with at least five points. The points in the compaction curves show the correlation between molding water content and dry unit weight. Both parameters are necessary considerations for comparison to the standards set by the North Carolina Department of Transportation (NCDOT) on embankment construction. These parameters are also used when conducting slope stability analysis.

Due to the nature of the soils after compaction, in some cases a smaller mold was necessary to create samples that were easier to transfer onto the triaxial cell platform. It was found that during the process of driving tubes into the Proctor mold, the specimens inside to tube had changed in height, creating a sample with a dry unit weight less than what was desired. A small mold was used, as well as a smaller hammer to create individual specimens, instead of creating 3 specimens simultaneously. Each specimen was prepared immediately prior to loading of the triaxial cell, which decreased the time available for the soil to lose water.

Prior to compaction of the specimens in the smaller mold, standard, intermediate, and modified compaction curves were developed following ASTM D698 (ASTM, 2012), which represents the standard effort (12,400 lb-lbf^3(600 kN-m/m^3)) as well as ASTM D1557 which represents modified effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)). Through the use of these curves, the ideal weight for each specimen was calculated to reach the ideal dry unit weight at each water content. Through an iterative process, the number of blows per layer of soil placed within the small mold was determined to ensure the same energy was applied to each specimen (Table 5.2).

Table 5.2: Compaction information for small 1.4-inch dimeter mold

Target Energy	**Number of Layers**	**Number of Blows per layer**
Standard	5	25
Intermediate	5	30
Modified	5	35

Note: Mold dimeter = 1.4 in., hammer weight = 1 lb., drop height = 4 in.

5.3 Shear Strength Testing Program and Procedures

5.3.1 Unconsolidated Undrained Triaxial Compression Tests

Unconsolidated undrained (UU) triaxial tests were performed in general accordance to ASTM 2850 for two of the three sandy materials investigated, namely the Wayne and Kinston test soils.

As mentioned in the previous section a total of three compaction energies (Standard Proctor, Intermediate Proctor, and Modified Proctor) were considered and samples were prepared dry and wet of the optimum water content (preferably two samples mixed dry of optimum, two wet of optimum, and the fifth at or approximate to the optimum water content). The samples for UU triaxial testing were obtained by pushing sharpened stainless steel tubes into the compacted samples in the Proctor mold. This was achieved by using a Universal Testing Machine that was lowered very slowly to carefully push the tubes into the compacted sample. The steel tubes had dimensions of 152.4 mm (6 in.) long, and a 35.6 mm (1.4 in.) inside diameter. The result of this procedure is shown in Figure 5.2.

Figure 5.2: Steel tubes containing samples after removal from proctor mold

The levels of confining pressures applied to each specimen removed from the Proctor mold were 25, 50, and 100 kPa. After the confining pressure was applied, specimen was loaded with a deviator stress at an axial strain controlled rate of 1%/minute. Undrained shearing of the test samples continued until the specimen reached a 15% strain rate. Upon completion of the test, specimen conditions were recorded and then were removed from the triaxial chamber for water content determination.

The main output of each UU triaxial test was a deviatory stress versus axial strain curve. For each stress-strain curve the maximum deviator stress was recorded and used as the main failure criterion for the determination of shear strength parameters. It should be pointed out that the pushing of the steel tubes into the compacted soil inside the Proctor mold was expected to induce some sample disturbance. In order to assess UU sample disturbance due to this procedure the sample unit weight change was measured and compared with respect to the dry unit weight recorded from the sample in the Proctor mold. This is summarized in Table 5.3.

Table 5.3: Proctor and specimen average dry unit weight

Sample type	Dry Unit Weight (kN/m^3)								
Proctor	18.3	18.5	17.8	17.8	16.7	17.7	18.8	19.9	19.5
Average Specimen	18.0	18.2	17.9	17.9	17.1	18.6	18.8	19.6	19.3
Deviation	-0.3	-0.3	+0.1	+0.1	+0.4	+0.9	0.0	-0.3	-0.2

5.3.2 Direct Shear Tests

To obtain the shear strength parameters for the Brunswick test soil, it was decided to conduct direct shear tests due to its inability to produce stable samples.

Direct shear tests were carried out following procedures outlined in ASTM Standard D4253 (ASTM, 2000) and ASTM D4254 (ASTM, 2000). Samples were prepared using a compaction procedure using moist Brunswick soil at the target water content (dry, wet or at or close to optimum) and with a compaction energy obtained by trial and error in order to obtain the target relative compaction (or dry unit weight). However due to the poorly graded sand characteristics this soil was better suited to be specified based on a relative unit weight using the ASTM specifications for maximum and minimum void ratios (or the corresponding maximum and minimum dry unit weights). These were obtained as per ASTM D4253 (ASTM, 2000). The corresponding minimum and maximum dry unit weight values obtained were 14.68 and 14.94 kN/m^3, respectively.

The direct shear test device used to perform these experiments is shown in Figure 5.3. The consolidated drained tests were carried out with the soil specimen placed in the shearing box with a circular cross section as shown in Figure 5.4.

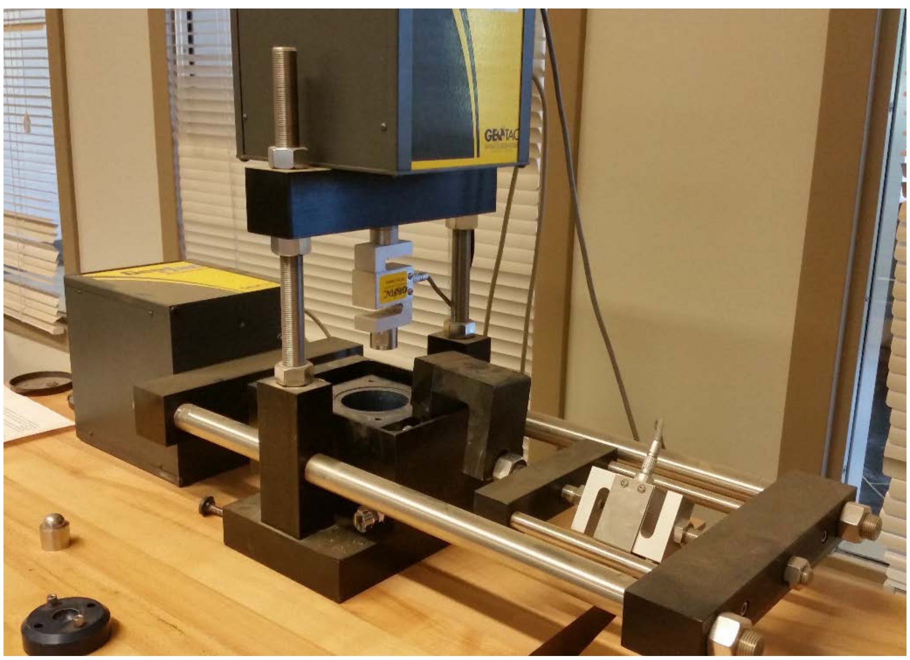


Figure 5.3: Direct shear test device

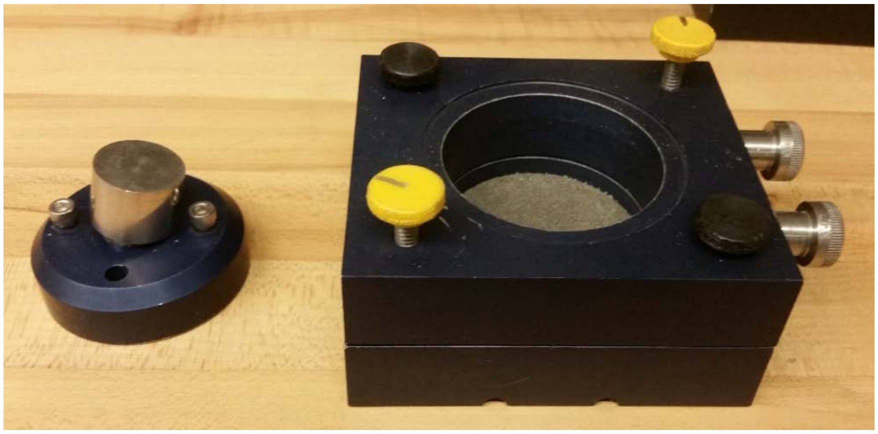


Figure 5.4: Direct shear box

The direct shear tests were sheared at a controlled rate of 0.254 mm/min (0.01 in/min) to ensure a drained shear condition. Three levels of applied normal stresses were used: 25, 50, and 100 kPa.

As the circular cross section of the specimen at the top and bottom halves of the shear box moves, the net contact area of the shearing surface decreases. The corrected shear contact area can be calculated using the following equation:

$$A_c = A \times \frac{2}{\pi} \times \cos\left(\frac{\Delta H}{D}\right) - \left(\frac{\Delta H}{D}\right) \times \sqrt{1 - \left(\frac{\Delta H}{D}\right)^2} \qquad (5.1)$$

where:

$A_c = Corrected\ Area\ (in^2)$

$A = Initial\ Area\ (in^2)$

$\Delta H = Horizontal\ Displacement\ (in)$

$D = Diameter\ (in)$

An example of a typical shear stress versus horizontal displacement graph can be seen in Figure 5.5. This figure shows three curves corresponding to three different normal stress levels (25 kPa, 50 kPa, and 100 kPa). By plotting the Mohr circles of the failure points of these three figures one can determine the shear strength envelope which in turn can be used to define the friction angle and cohesion intercept (if any) for the test soil at a particular compaction water content and compacted dry unit weight.

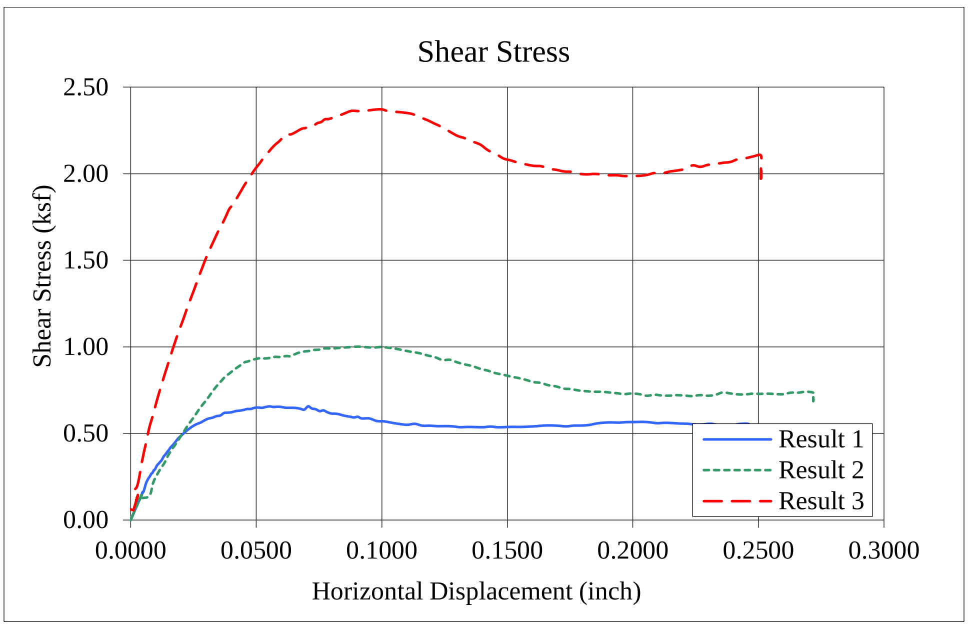


Figure 5.5: Example of shear stress versus horizontal displacement curves from three direct shear tests at three normal stress levels

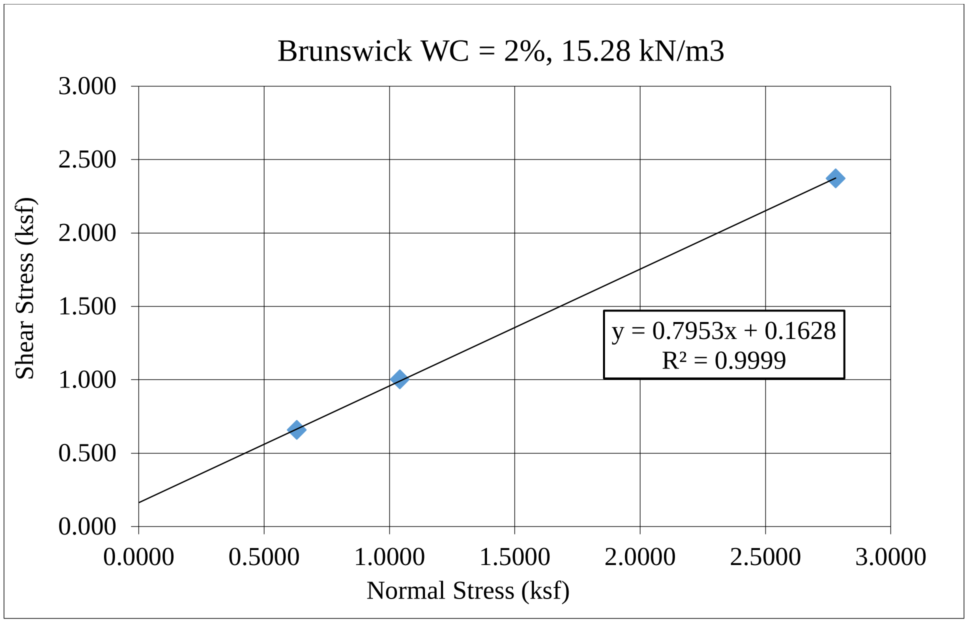


Figure 5.6: Example of shear strength envelope from a direct shear test

CHAPTER 6: EXPERIMENTAL RESULTS

6.1 Introduction

The procedure for the experiments performed are discussed in Chapter 4. This chapter discusses the results of these experiments. Unconsolidated undrained triaxial testing was performed on Wayne and Kinston soils to determine shear strength, friction and cohesion values. After attempts to perform unconsolidated undrained triaxial testing on Brunswick soil, it was determined that direct shear testing would be best for determining the cohesion and friction values.

6.2 Compaction Test Results

As stated in in Chapter 5, the engineering properties and the anticipated field performance of compacted soils used for highway embankment construction will depend on the soil characteristics, type, index properties, as well as the way it is compacted in the field. Related to compaction in the field it is important to first determine in the laboratory the dry unit weight-water relationships for borrow soils to be used in construction. The next step is to define a target zone which will be used also as the acceptance zone for QC in the field.

As mentioned in Chapter 5, three compaction efforts were used to assess the compaction characteristics for the test soils. The lowest and highest compaction energies correspond to the compaction efforts specified for the Standard and Modified Proctor tests, respectively. The compaction test results for the Wayne and Kinston test soils are presented in Figures 6.1 and 6.2 respectively.

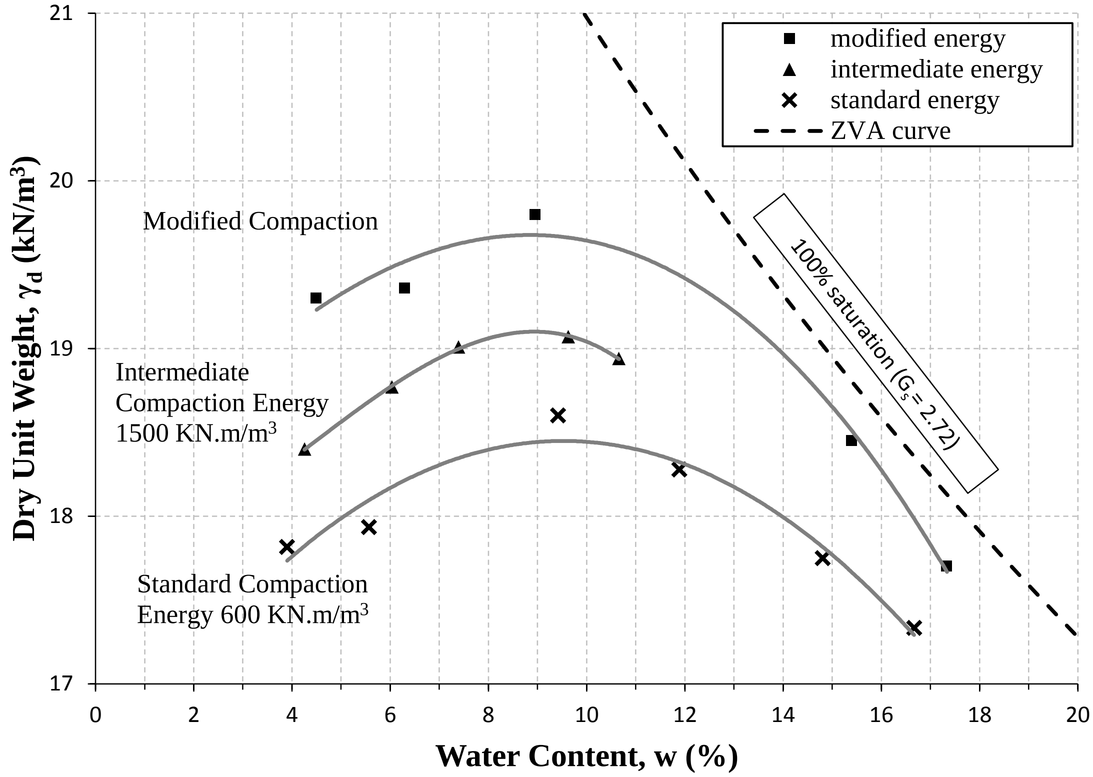


Figure 6.1: Wayne test soil compaction curves

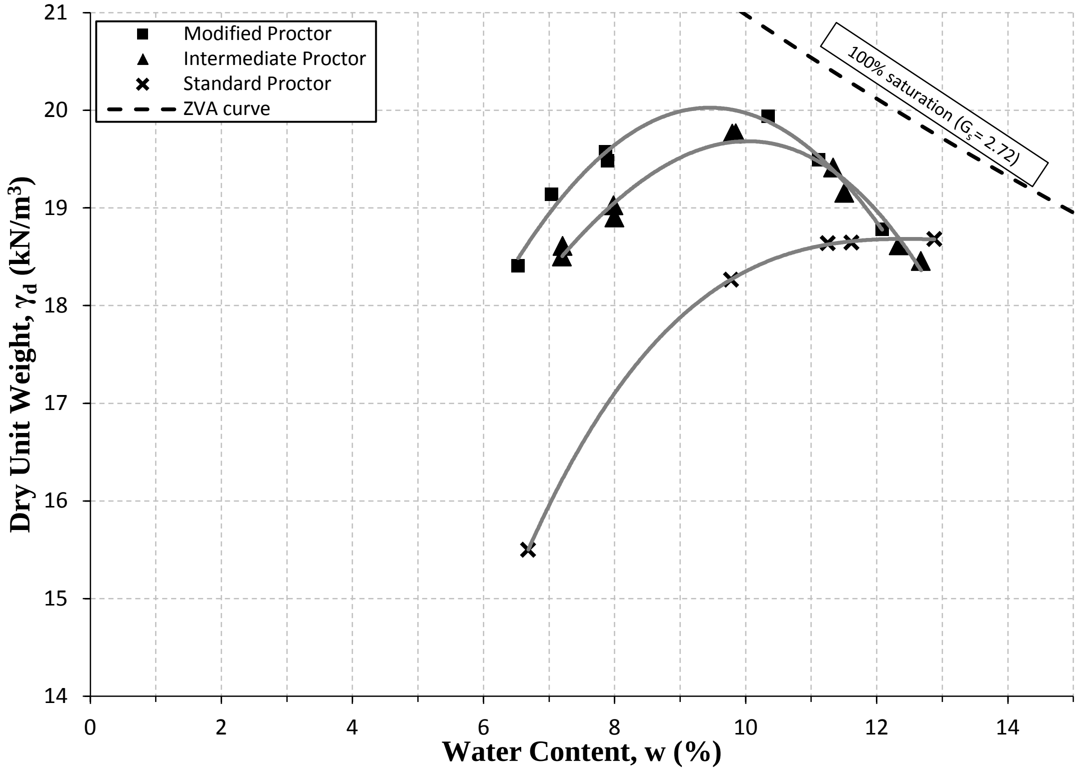


Figure 6.2 Kinston test soil compaction curves

Attempts were made to compact Brunswick test soil following the recommendations of ASTM D698-12, which gives the instructions for standard effort compaction. Due to the poor gradation and apparent low cohesive properties of the soil, the particles of soil could not be compacted into a more proper arrangement to allow for a higher dry unit weight. Figure 6.2 shows the more linear water content – dry unit weight graph created during standard Proctor compaction.

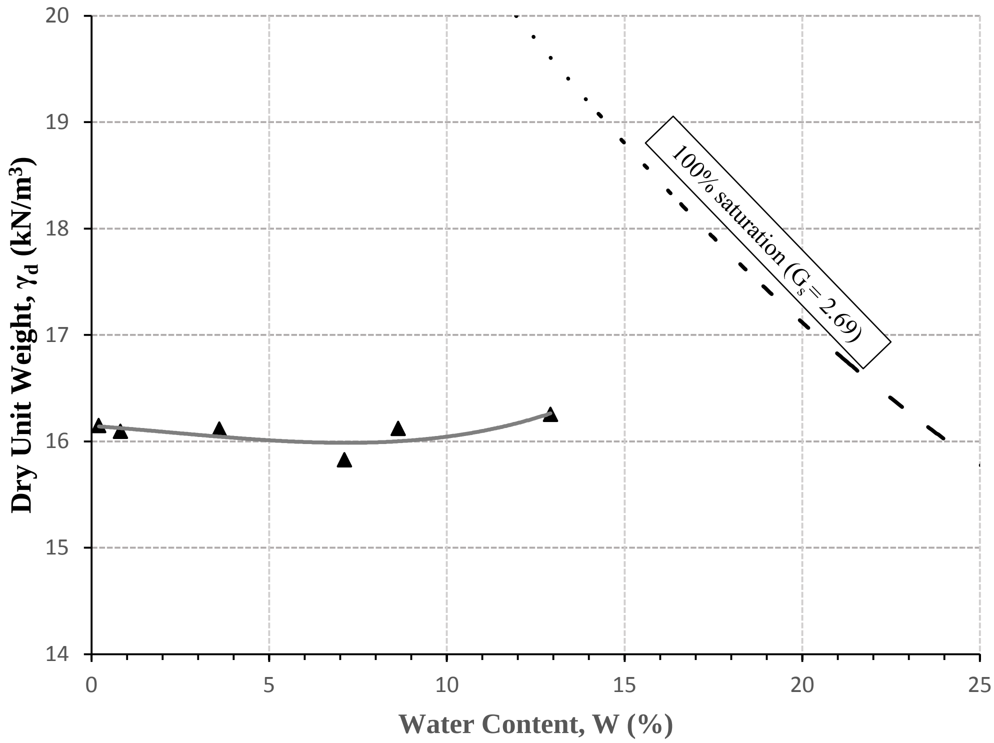


Figure 6.3: Brunswick test soil standard compaction curve

6.3 Shear strength parameters and stiffness of test soils

6.3.1 Unconsolidated Undrained Test Results

6.3.1.1 Maximum Deviator Stress

Data sheets from each UU triaxial test are shown in Appendix A, including figures of the deviator stress versus axial strain for each triaxial test performed on Wayne and Kinston test soils and the p-q plots. Results from the tests are shown in Tables 6.1 – 6.6. From these results, it was observed that as the water content decreased, the maximum deviator stress tended to increased. Also, as compaction energy increased, the maximum deviator stress for similar water contents also increased.

Table 6.1 Wayne test soil – summary of UU data for standard compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
1			144.0
2	11.9	18.3	228.8
3			427.4
16			82.0
17	14.8	17.7	173.0
18			447.5
19			64.8
20	16.7	17.3	132.6
21			439.1
31			144.1
32	9.4	18.6	205.0
33			NA
58			275.3
59	3.9	17.8	384.0
60			550.4
67			255.0
68	5.6	17.9	348.7
69			501.5

Table 6.2 Wayne test soil - summary of UU data for intermediate compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
61			408.0
62	4.3	18.3	577.5
63			595.1
70			378.0
71	6.0	18.8	508.9
72			584.8
79			195.8
80	9.6	19.1	373.7
81			591.5
82			270.5
83	10.7	19.0	333.5
84			646.0
88			379.2
89	7.4	19.0	331.6
90			526.7

Table 6.3 Wayne test soil - summary of UU data for modified compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
34			235.8
35	8.9	19.8	270.7
36			558.6
22			90.4
23	15.4	18.5	132.5
24			292.6
64			577.3
65	4.5	19.3	605.2
66			732.9
73			384.4
74	6.3	19.4	456.9
75			736.5

Table 6.4 Kinston test soil- summary of UU data for standard compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
1			291.9
2	11.8	18.7	353.4
3			NA
3			238.8
4	12.2	18.8	341.2
5			426.8
6			477.7
7	10.1	18.5	451.6
8			656.9
15			192.5
16	11.6	18.7	278.0
17			363.1
21			420.2
22	7.3	17.0	367.2
23			593.7
54			206.6
55	11.3	18.6	348.8
56			473.2

Table 6.5 Kinston test soil - summary of UU data for intermediate compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
33			689.5
34	7.2	18.5	477.8
35			809.0
42			648.2
43	8.0	19.3	691.3
44			813.2
45			640.9
46	9.8	19.8	522.6
47			660.0
51			237.1
52	12.3	18.6	266.9
53			256.9
57			178.4
58	11.3	19.3	358.5
59			558.1
66			279.0
67	11.5	19.2	363.8
68			441.5

Table 6.6 Kinston test Soil - summary of UU data for modified compaction energy

Test Number	Average Water Content (%)	Average Dry Unit Weight (kN/m^3)	Maximum Deviator Stress (kPa)
9			208.4
10	12.2	18.8	166.8
11			382.1
12			420.9
13	10.3	19.9	528.6
14			696.6
18			307.8
19	11.1	19.5	382.4
20			554.5
24			804.8
25	6.5	18.4	926.2
26			962.5
27			622.4
28	7.9	19.6	804.1
29			947.8
60			309.5
61	11.0	19.2	371.2
62			336.9

Each sample that was compacted at the specified dry-unit-weight and water content contained three specimens that were tested at increasing confining pressures. The stress-strain graphs recorded (See Appendix A) in general showed an increasing maximum deviator stress with increasing confining pressure.

The total shear strength parameters (friction ϕ and cohesion c) were computed for each compaction condition listed in Tables 6.1 through 6.6. These strength parameters were obtained using the k_f failure lines in the p-q space (see Appendix A). The slope of these K_f line is used to determine the friction angle of the soil as follows:

$$\phi = sin^{-1}(\tan(\alpha)) = sin^{-1}(\frac{\Delta p}{\Delta q}) \qquad (6.1)$$

Where: tan (α) = slope of k_f line,

$p = \frac{\sigma_1 + \sigma_3}{2}$,

$q = \frac{\sigma_d}{2}$,

$\sigma_1 = \sigma_d + \sigma_3$

The cohesion intercept is computed as follows:

$$c = \frac{d}{\cos\phi} \qquad (6.2)$$

Where: d is the intercept of the k_f line and ϕ is the friction angle from Eq. (6.1).

For each sample tested, the resulting friction and cohesion values were linked to each correlating water content and dry-unit weight (Tables 6.7 and 6.8). By making these correlations, a contour curve showing the friction and cohesive values could be developed for each soil. Contour plots of cohesion and friction angle for the Wayne and Kinston test soils are shown in Figures 6.4 through 6.7.

Table 6.7: Wayne test soil friction and cohesion values

Compaction	Dry Unit Weight (kN/m^3)	Water Content (%)	Friction (°)	Cohesion (kPa)
1	18.3	11.9	41	10
2	17.7	14.8	46	13
3	17.3	16.7	47	21
4	18.6	9.4	34	21
5	17.8	3.9	40	44
6	17.9	5.6	38	43
7	18.9	5.8	35	85
9	19.2	9.9	46	17
10	19.0	10.7	47	21
11	19.8	8.9	35	40
14	19.3	4.5	32	142
15	19.4	6.3	36	70

Table 6.8: Kinston test soil friction and cohesion values

Compaction	Unit Weight (kN/m^3)	Water Content (%)	Friction (°)	Cohesion (kPa)
1	18.8	12.2	30	56
2	18.5	10.1	37	89
3	17.0	7.3	37	70
5	18.5	7.2	47	68
6	18.9	8.0	32	163
7	19.8	9.8	29	145
8	18.5	12.8	44	10
10	18.8	11.9	34	16
11	19.9	10.3	40	79
12	19.5	11.1	35	56
13	18.4	6.5	28	239
14	19.5	7.9	43	114

Dry unit weight values determined from compaction of Wayne range from 17.3 to 19.8 kN/m^3. Kinston soil dry unit weight values ranged from 17.0 to 19.9 kN/m^3. Both Wayne and Kinston dry unit weight values fell within the ranges discussed in Chapter 2, recommended by USCS. Friction values of Wayne and Kinston soils summarized in

Tables 6.7 and 6.8 range from 32° to 47°, Kinston soil's friction values ranged from 29° to 44°, depending upon the water content and dry unit weight. As recommended by USCS, the average friction value for the Wayne and Kinston soil type (SM – silty sand) is 34°.

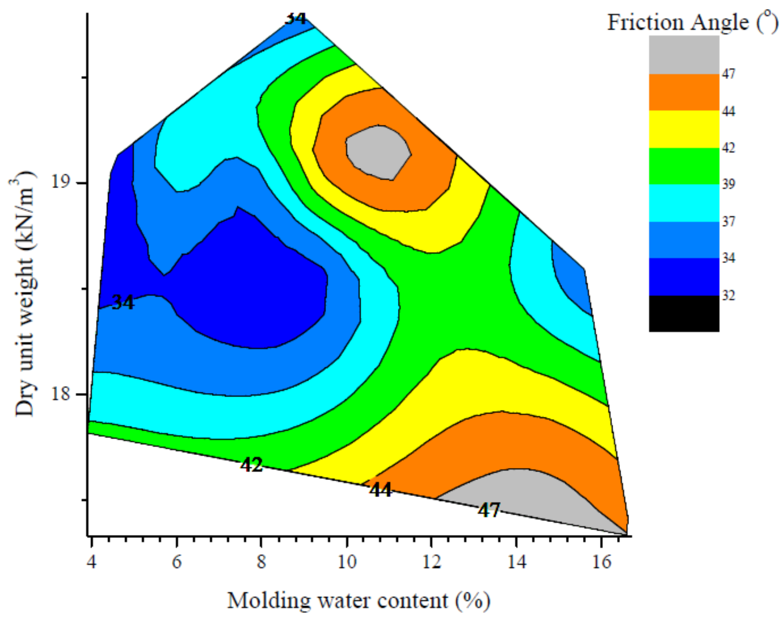


Figure 6.4: Wayne test soil friction

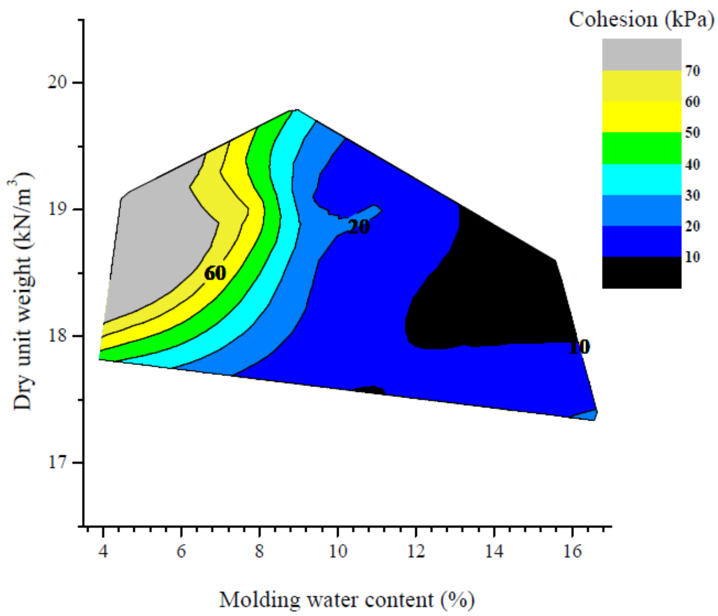


Figure 6.5: Wayne test soil cohesion

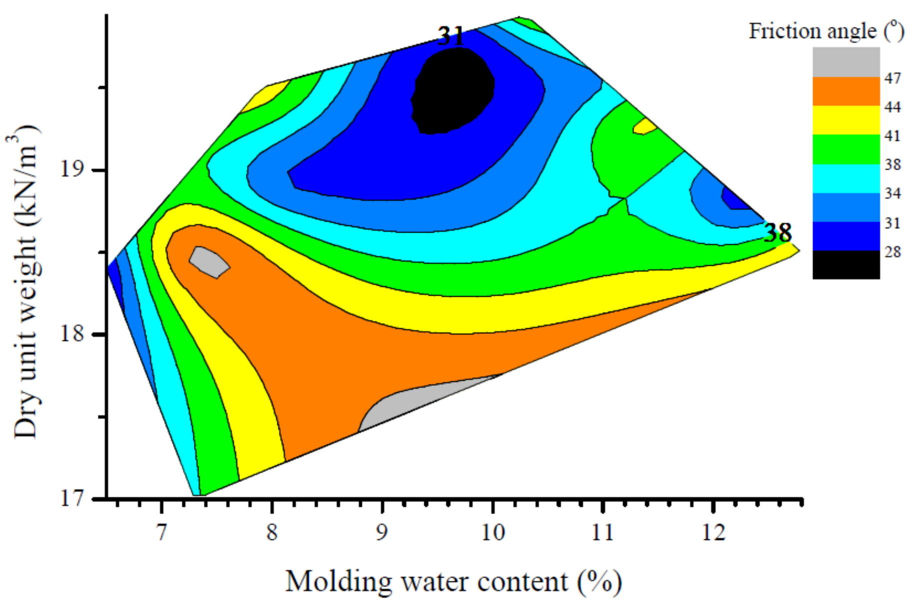


Figure 6.6: Kinston test soil friction

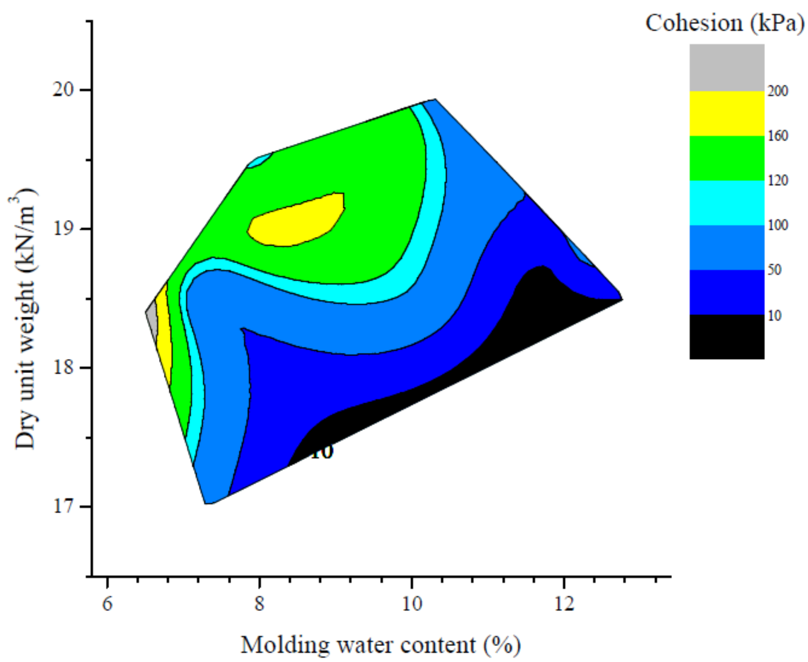


Figure 6.7: Kinston test soil cohesion

6.3.1.2 Soil Stiffness

The Young's Modulus of Elasticity (stiffness) was also determined through the results provided by the UU triaxial tests on Wayne and Kinston test soils. As previously discussed in Chapter 3, there are several methods for determining the elasticity values for soil. In this project it was decided to use the E_{50} value calculated from the following equation:

$$E_{50} = \frac{0.5 \times \sigma_{d,max}}{S_{50}} \qquad (6.3)$$

Where $\sigma_{d,max} = maximum\ deviator\ stress$, and

$S_{50} = strain\ value\ at\ 50\%\ maximum\ deviator\ stress$

Tables 6.9 through 6.14 summarize the stiffness values resulting from the triaxial tests on Wayne and Kinston soils. By comparison to the USCS average values discussed in Chapter 2, only a few values fall within the average ranges.

Table 6.9: Wayne test soil, standard compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
1 - 3	3.5	15.8	13
16 - 18	0.7	4.2	21.6
19 - 21	0.2	0.3	21.4
31 - 33	0.4	0.5	-
41 - 43	0.5	0.8	1.1
58 - 60	11.7	24.7	29.5
67 - 69	17.1	24.1	22.5

Table 6.10: Wayne test soil, intermediate compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
61 – 63	22	38.7	50.5
70 – 72	21.5	33.7	38.4
76 – 78	19.1	28.2	41.6
79 – 81	14.3	28.7	41.5
82 – 84	18	17.9	39.1
88 - 89	23.4	27.6	43.3

Table 6.11: Wayne test soil, modified compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
22 - 24	0.6	0.8	2.1
34 – 36	3.9	4.4	6.6
64 - 66	29.9	24.3	29.8
73 - 75	17.9	44.9	43.1
85 - 87	21.2	28.3	37.8

Table 6.12: Kinston test Soil, standard compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
1 – 2	5.0	6.0	-
3 - 5	1.7	2.2	3.0
6 - 8	11.8	7.9	10.5
15 - 17	31.2	17.2	22.7
21 - 23	2.7	4.6	4.6
54 - 55	1.7	2.4	4.1

Table 6.13: Kinston test soil, intermediate compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
33 – 35	1.8	1.7	1.5
42 – 44	38.9	45.4	59.8
45 – 46	55.8	44.1	33.5
51 – 53	1.3	0.9	3.2
57 – 59	1.7	2.8	3.7
66 – 68	2.1	2.3	2.9

Table 6.14: Kinston test soil, modified compaction, confining pressure and E_{50} (MPa)

Test Numbers	25 kPa	50 kPa	100 kPa
9 – 11	1.2	0.9	2.3
12 – 14	3.8	4.9	6.5
18 – 20	2.0	2.1	3.5
24 – 26	2.0	51.8	43.2
27 – 29	23.0	39.6	44.2
60 - 62	2.7	3.4	4.7

6.3.2 Direct Shear Test Results

Due to the coarser grain size distribution, direct shear tests were performed on the Brunswick test soil. The procedure for the direct shear tests was described in Chapter 5. For each compaction condition considered a set of three shear stress vs. horizontal displacement curves was obtained with each corresponding to an applied normal stress level. The results also included sample height change versus horizontal displacement. The shear strength parameters were obtained using the maximum shear stress and corresponding normal stress. Since the results are in the normal stress (σ) versus shear stress (τ) space the regression lines already correspond to the Mohr-Coulomb shear strength envelopes with the slope of this line corresponding to the frictional angle (ϕ) and the y-axis intercept the cohesion (c). The contour plots for the friction angle and cohesion obtained for the Brunswick soil are shown in Figures 6.8 and 6.9, respectively.

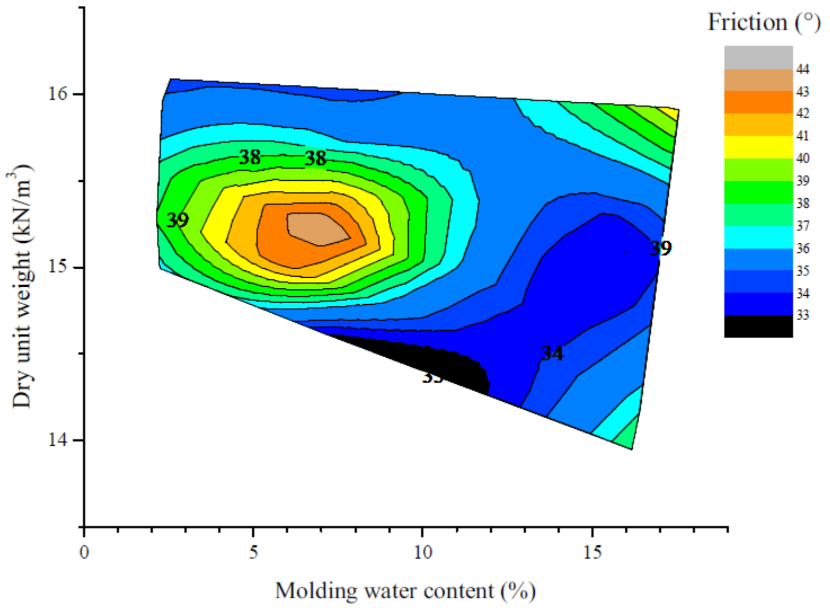


Figure 6.8: Brunswick test soil friction contours

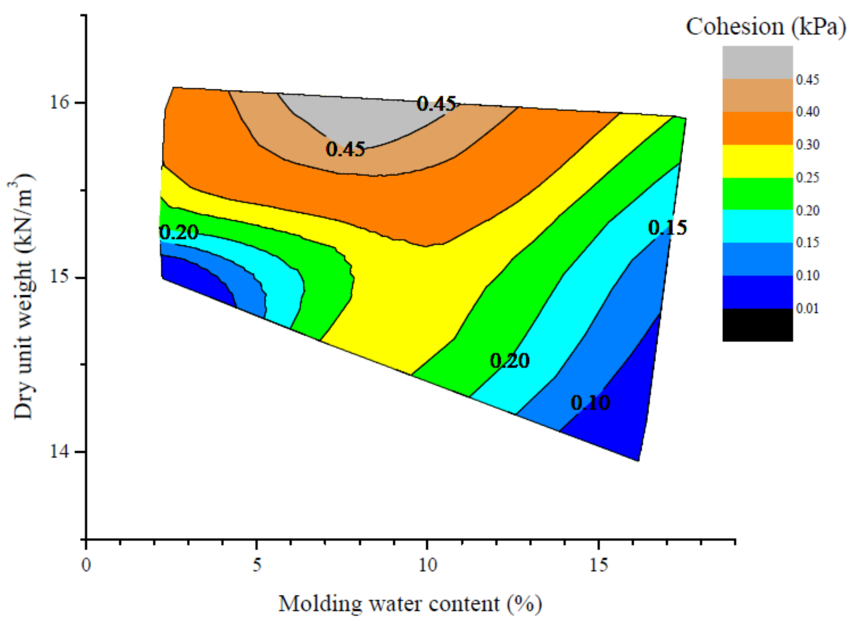


Figure 6.9: Brunswick test soil cohesion contours

Brunswick soil, a poorly graded sand (SP), had friction values ranging from 32° to 44°, based on gradation considerations the average friction values expected for a soil with this USCS is about 36°. Cohesion values obtained for the Brunswick test soil were exceptionally low, which is as expected given the coarser gradation and USCS classification.

CHAPTER 7: SLOPE STABILITY ANALYSES AND ESN

7.1 Introduction

Slope stability of an embankment was an indicator of failure of the material presented in this research. As previously discussed in Chapter 3, slope safety factor of less than 1.3 is considered unacceptable by many current USDOT standards as well as national standards. This limit has always been the benchmark when analyzing the large range of embankment dimensions presented in this chapter.

7.2 Description of Slope Stability Analysis

Slope stability is usually evaluated in two major ways: limit equilibrium or deformation analysis. In this thesis the analysis was carried out using the limit equilibrium analysis (LEA) approach. Specifically, the Bishop Modified Method (Bishop, 1955) was used for evaluating the stability of the highway embankment slopes considered. The slope stability analyses involved assessment of several embankment geometries (heights and side slopes) and were assumed built using the three test soils described in Chapters 4 and 5. The shear strength properties assigned to the embankment soil were assigned assuming the soils were placed at different relative compaction levels and at different water contents to cover a wide range of scenarios that could be encountered in actual embankment construction. This includes low and high compaction efforts that would be reflected as low or high dry unit weights, respectively. Soil placement under wet or dry conditions, correspond to soils wet or dry of optimum water content values discussed in Chapter 2. A matrix that summarizes the different slope stability analyses considered, in terms of geometry and soil type, is provided in Table 7.1. As shown in this table height/side slope scenarios were considered for each soil type. The heights evaluated were 10, 20, 30 and 40 ft. The side slopes evaluated for the highway embankment had ratios of 1H:1V, 2H:1V, 3H:1V, and 4H:1V. These side slopes were selected to reflect NCDOT design practice.

The slope stability analyses summarized in this table were limited to a fixed slip surface that was kept constant to facilitate comparison of factor of safety values for the different geometries and soil parameters. The slip surface considered intersected the top of the embankment 2.5 feet from the crest, and exit the embankment exactly at the toe. The selected slip surfaces for two highway embankment geometries are shown in Figure 7.1 which shows the center and radius change for each case. The center and radius were computed using simple geometry considerations in order to meet these conditions for each highway embankment geometry.

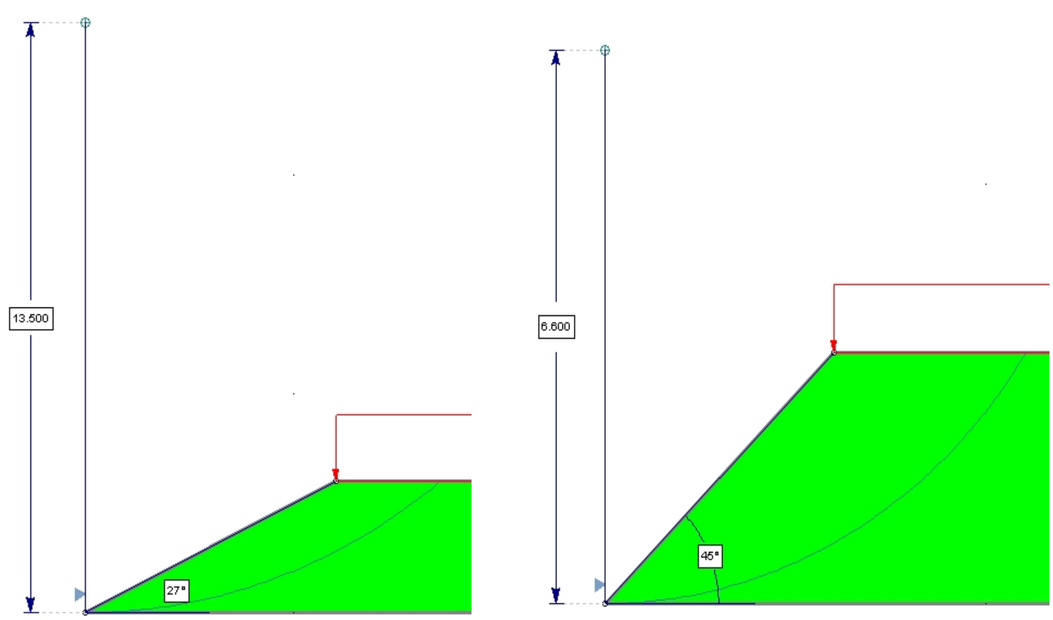


Figure 7.1: Examples of change in surface radius with change in embankment geometry (not to scale)

The matrix of analyses in Table 7.1 corresponds to 16 base geometries for each soil type. For each base soil the engineering properties were also varied to reflect possible variations in compaction conditions (energy, achieved dry unit weight, and water content). This was reflected in the slope stability analyses in terms the soil total unit weight and shear strength properties assigned based on the location in the water content versus dry unit weight space shown schematically in Figure 7.1.

Table 7.1: Embankment geometries

Base soil	Side Slope	Height (ft)	Analysis ID	No.
Wayne/Kinston/Brunswick	1H:1V	10	A-1-1-10	1
		20	A-1-1-20	2
		30	A-1-1-30	3
		40	A-1-1-40	4
	2H:1V	10	A-2-1-10	5
		20	A-2-1-20	6
		30	A-2-1-30	7
		40	A-2-1-40	8
	3H:1V	10	A-3-1-10	9
		20	A-3-1-20	10
		30	A-3-1-30	11
		40	A-3-1-40	12
	4H:1V	10	A-4-1-10	13
		20	A-4-1-20	14
		30	A-4-1-30	15
		40	A-4-1-40	16

All slope stability analyses were carried out using the slope stability software *SLIDE 7* (Rocscience, 2015). As mentioned earlier the analyses were carried out using the Bishop modified method. The highway embankments were considered not submerged and the foundation soils were considered competent enough not to influence the slope stability of the embankment. This is one of the reasons the slip surface selected for this slope stability study was made to exit at the toe of the embankment without intersecting the foundation soils.

The slope stability analyses also considered a uniform surcharge loading at the top to represent traffic loading. This surcharge loading was estimated to be approximately 35 kN/m^2.

7.3 Slope Stability Analyses Results

Upon completion of the slope stability analysis, the resulting factor of safety for each combination of dimensions was applied as contours to the dry unit weight – water content curves previously created for each soil. These contours show the change in factor of safety as the water content and dry unit weight change.

Upon review of the results of the slope stability analyses performed with *SLIDE*, each dimension and soil property condition resulted in a factor of safety remaining above 1.3 for test soils Wayne and Kinston. Test soil Brunswick had several cases in which the factor of safety dropped below 1.3 when the slope was 1H:1V. The remaining slope conditions resulting in factors of safety above 1.3. Table 7.3 lists the results from the slope stability analysis of test soil Brunswick with a 1H:1V slope. Factors of safety in bold are those below 1.3.

Table 7.2: Brunswick test soil, slope 1H:1V stability results

Test	Height (m)	Total Unit Weight (kN/m^3)	Water Content (%)	Cohesion (kPa)	Friction (°)	Bishop FS
Test 1	10	15.61	2.14	0.2	38.5	1.336
Test 2	10	15.32	2.16	0.01	37	**1.252**
Test 3	10	17.55	15.83	0.16	32.7	**1.094**
Test 4	10	16.21	16.25	0.01	38.4	1.326
Test 5	10	16.13	6.89	0.22	44	1.627
Test 6	10	15.65	7.05	0.26	32.8	**1.089**
Test 7	10	17.02	7.65	0.47	35.4	**1.226**
Test 8	10	18.72	17.57	0.24	41	1.497
Test 9	10	16.47	2.34	0.33	34	**1.151**
Test 1	20	15.61	2.14	0.2	38.5	**1.185**
Test 2	20	15.32	2.16	0.01	37	**1.110**
Test 3	20	17.55	15.83	0.16	32.7	**0.960**
Test 4	20	16.21	16.25	0.01	38.4	**1.176**
Test 5	20	16.13	6.89	0.22	44	1.441
Test 6	20	15.65	7.05	0.26	32.8	**0.964**
Test 7	20	17.02	7.65	0.47	35.4	**1.081**
Test 8	20	18.72	17.57	0.24	41	1.320
Test 9	20	16.47	2.34	0.33	34	**1.015**
Test 1	30	15.61	2.14	0.2	38.5	**1.180**
Test 2	30	15.32	2.16	0.01	37	**1.108**
Test 3	30	17.55	15.83	0.16	32.7	**0.959**
Test 4	30	16.21	16.25	0.01	38.4	**1.171**
Test 5	30	16.13	6.89	0.22	44	1.433
Test 6	30	15.65	7.05	0.26	32.8	**0.959**
Test 7	30	17.02	7.65	0.47	35.4	**1.070**
Test 8	30	18.72	17.57	0.24	41	1.306
Test 9	30	16.47	2.34	0.33	34	**1.008**
Test 1	40	15.61	2.14	0.2	38.5	**1.187**
Test 2	40	15.32	2.16	0.01	37	**1.116**
Test 3	40	17.55	15.83	0.16	32.7	**0.962**
Test 4	40	16.21	16.25	0.01	38.4	**1.178**
Test 5	40	16.13	6.89	0.22	44	1.440
Test 6	40	15.65	7.05	0.26	32.8	**0.964**
Test 7	40	17.02	7.65	0.47	35.4	**1.072**
Test 8	40	18.72	17.57	0.24	41	1.308
Test 9	40	16.47	2.34	0.33	34	**1.011**

7.4 Statistical Analysis

As part of the proposed research objectives for the NCDOT Embankment Project, a statistical regression analysis was performed to determine the degree of influence of the soil parameters from a proposed borrow site, in relation to the resulting factor of safety. The statistical software SPSS was used to perform linear regression analysis of the multiple variables used to determine the factor of safety. These variables include: soil dry unit weight, water content, friction, cohesion, and embankment height. Details of the regression can be found in Appendix D.

As previously discussed, the factor of safety was determined prior to this statistical analysis. Through use of *SLIDE 7*, the factor of safety was calculated using the Bishop Modified Method. Due to the varying nature of the factor of safety in relation to the failure surface, a fixed slip surface intersecting the embankment was assumed. This surface intersected the top of the embankment approximately 2.5 feet from the crest, developing through the embankment fill and exiting at the embankment toe.

Four different heights and slopes were evaluated in the slope stability analysis, each varying independently from the other. In total there were 16 different dimensions to be considered for each point covered from the dry unit weight – water content compaction curves. For example, on the standard compaction curve of Wayne soil, the first point on the standard compaction curve has the following properties:

Table 7.3 Wayne compaction point properties

Water content (%)	Total Unit Weight (kN/m^3)	Friction (°)	Cohesion (kPa)
3.9	18.51	40	44

The above variables remained the same regardless of the dimensions of the embankment, although the resulting factor of safety changed with each change in the embankment height and slope.

Table 7.4: Factor of safety with changes in height and slope

		Height (ft)			
		10	20	30	40
Side Slope	1H:1V	3.969	2.898	2.787	2.522
	2H:1V	6.031	4.938	4.331	3.890
	3H:1V	8.725	7.216	6.264	5.592
	4H:1V	11.918	9.798	7.726	7.421

In total, 192 values related to each compaction point were listed in this analysis, resulting in each variable being assigned a coefficient relative to the degree of influence each variable had on the resulting factor of safety (Appendix D). Initial interpolation of equations 7.1 through 7.3 was conducted by assuming the dependent variable as the $\log_{10}$ value of the factor of safety. The $\log_{10}$ value of each factor of safety was used during linear regression due to the resulting high values of reliability, which is discussed later in Chapter 7. With the use of algebraic rearrangement, the final equations were created displaying the factor of safety as the resultant of each equation. The final linear equation can be used for the specific soil type. For example, after analysis of Wayne County soil, the resulting equation was developed:

$$FS_{Wayne} = \frac{10^{.304} \times 10^{.153S} \times .10^{.004\phi} \times 10^{.004C}}{10^{.006H} \times 10^{.010\gamma d} \times 10^{.003WC}} \quad (7.1)$$

Where:

$S = Slope\ (H/V)$,

$H = Height\ (m)$,

$\gamma_d = Dry\ Unit\ Weight\ (\frac{kN}{m^3})$,

$WC = Water\ Content$,

$\phi = Friction$ (°), and

$C = Cohesion$ (kPa)

Equation 7.1 was developed using all variables that were considered when performing slope stability analysis. Variables such as height and slope vary independently with one another, although the soil dry unit weight, water content, friction, and cohesion remain the same as height and slope change. It is evident from Equation 7.1 that the embankment slope has the most influence on the resulting factor of safety and has the greatest coefficient among the variables considered. This equation indicates that as the slope, friction and cohesion values increase, the factor of safety increases. As the height, total unit weight, and water content increase, the factor of safety decrease.

The reliability of the regression analysis was determined in SPSS by evaluating the R-Square value. This R-Square value represents the strength of the association between the dependent variable (factor of safety), and the independent variables (height, slope, dry unit weight, water content, friction, and cohesion). In the case of Wayne

County soil, the R-Square value was 0.958. This suggests that approximately 95.8% of the resulting factor of safety is determined by independent variables.

Statistical analyses were also performed on Kinston and Brunswick test soils. Kinston and Brunswick had R-Square values of 0.95 and 0.99 respectively, indicating that both regression equations are reliable.

$$FS_{Kinston} = \frac{10^{.088} \times 10^{.160S} \times .10^{.001\phi} \times 10^{.003C} \times 10^{.036\gamma d}}{10^{.007H} \times 10^{.046WC}} \qquad (7.2)$$

$$FS_{Brunswick} = \frac{10^{.135S} \times .10^{.016\phi} \times 10^{.019C} \times 10^{.002\gamma d}}{10^{.670} \times 10^{.003WC}} \qquad (7.3)$$

7.5 Embankment Stability Number (ESN)

The above regressions were used to estimate the factor of safety of an arbitrary failure surface (see Figure 7.1) that although useful may not be the critical slip surface. Therefore it is desirable to report an embankment stability number (ESN) as opposed to a factor of safety of an arbitrary slip surface. This arbitrary slip surface is useful to define an ESN. The preliminary ESN index is proposed to be based on the following qualitative slope stability conditions for the fixed slip surface selected for this study:

Table 7.4. Hypothetical Borrow Material Rating based on proposed ESN

Factor of Safety for Selected Arbitrary Slip Surface	**RATING**	**ESN**	**Comments**
> 2.0	Excellent	10	Acceptable
2.0 – 1.5	Good	8-10	Acceptable
1.25 – 1.5	Fair	5-8	Marginal
1.0 – 1.25	Poor	1-5	Not acceptable
< 1.0	Unsatisfactory	0	Not acceptable

Note: ESN value can be computed using linear interpolation for each rating category.

CHAPTER 8: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS FOR FUTURE WORK

8.1 Summary and Conclusions

The stress-strain behavior and engineering properties of three North Carolina soils were studied. UU triaxial compression tests and direct shear tests on compacted samples at different confining pressures, water contents, and dry-unit weights were the main focus of the experimental program. Index tests as well as sieve analyses were performed to determine the classification of each soil.

In addition to experimental testing, analyses were performed on the slope stability of embankments of varying dimensions comprised of the NCDOT-supplied material. Upon review of the factor of safety for each scenario, statistical linear regression analysis was performed to determine any correlations between the material properties and embankment dimensions with the resulting factor of safety. These tests led to the following important conclusions:

- A linear relationship was discovered between the log value of the factor of safety and the embankment dimensions, as well as the material properties. For each soil, a resulting R^2 value greater than 0.9 was reached.
- As the slope of the embankment increased in steepness, the factor of safety decreased. In addition, the factor of safety decreased as the embankment height increased.

- Values of cohesion for Brunswick soil decreased as the dry unit weight decreased, appearing to be independent of the water content of the soil.
- Values of friction for Brunswick soil reached a maximum when the water content was dry of the optimum water content value.
- Cohesion of Kinston soil increased as the dry unit weight increased, increased as the water content decreased.
- Friction of Kinston soil appeared to reach a minimum value at the maximum dry-unit weight and optimum water content, but continued to increase as the dry-unit weight decreased.
- Wayne soil cohesion values increased as the water content decreased, appearing to be independent of the dry-unit weight.
- Wayne soil friction values reached a maximum at the maximum dry unit weight and optimum water content, but decreased as the unit weight and water content values less or greater than optimum water content and dry unit weight. Friction values appeared to also reach a maximum as the water content increased and dry unit weight decreased.

8.2 Recommendations for Future Work

The soils presented in this thesis were supplied by the NCDOT and they met current specification requirements for embankment fill material by maintaining a Plasticity Index below 15%. Slope stability analysis of these soils determined that with proper compaction, an acceptable factor of safety by current standards could be reached. To determine the threshold at which a failure would occur, material with higher PI values from the coastal region should be analyzed in a similar manner. Additional slope stability analyses with other failure surfaces should be considered as well.

REFERENCES

ADOT (2008), “Arizona Department of Transportation Standard Specifications for Road and Bridge Construction”, Arizona DOT.

Ahmed, S., Lovell, C.W., and Diamond, S. (1974), "Pore sizes and strength of compacted clay", Journal of the Geotechnical Engineering Division, vol. 100 (4), pp. 407-425.

AKDOT (2015), “Alaska Department of Transportation and Public Facilities Standard Specifications for Highway Construction”, Alaska State DOT.

ALDOT (2012), “Alabama Department of Transportation Standard Specification for Highway Construction”, Alabama State DOT.

ARDOT (2014), “Arkansas Standard Specification for Highway Construction”, Arkansas State DOT.

ASSHTO (2002), "Standard Specifications for Highway Bridges", American Association of State Highway and Transportation Officials, Washington, DC.

ASTM D1557-12 Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D3080 (2011) Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D422-63 (2007) Standard Test Method for Particle-Size Analysis of Soils. In *Annual Book of ASTM Standards* (Vol. 04.08)

ASTM D4253-00 Standard Test Methods for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D4254-00 Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D4318-10 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D698-12 (2012) Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

ASTM D854-10 Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer. In *Annual Book of ASTM Standards* (Vol. 04.08). West Conshohocken, Pennsylvania.

Bhasin, R.N. (1975), "Pore Size Distribution of Compacted Soils after Critical Region Drying", PhD Dissertation, Purdue University.

Bishop, A.W. (1955), "The use of the slip circle in the stability analysis of slopes", Geotechnique, vol. 5 (1), pp. 7-17.

Bishop, A.W., Alpan, I., Blight, G., and Donald, I. (1960), "Factors controlling the strength of partly saturated cohesive soils", in Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, Colorado, United States, 1960, pp. 503-532.

Bromhead, E.N. (1986), "The Stability of Slopes", New York: Surrey University Press/Chapman and Hall.

CADOT (2010), “Standard Specifications State of California”, California State DOT.

Carrier, W.D. (2000), "Compressibility of a compacted sand", Journal of geotechnical and geoenvironmental engineering, vol. 126 (3), pp. 273-275.

Casagrande, A. and Hirschfeld, R.C. (1960), "First Progress Report on Investigation of Stress-deformation and Strength Characteristics of Compacted Clays", Harvard Soil Mechanics Series, No. 61.

Casagrande, A. and Hirschfeld, R.C. (1962), "Second Progress Report on Investigation of Stress-deformation and Strength Characteristics of Compacted Clays", Harvard Soil Mechanics Series, No. 65.

Chen, Y. (2010), "An experimental investigation of the behavior of compacted clay/sand mixtures", M.Sc., University of Delaware, advisor: C. L. Meehan.

Chen, Y. (2010), "An experimental investigation of the behavior of compacted clay/sand mixtures", M.Sc. thesis, University of Delaware, advisor: C. L. Meehan.

Chen, Y. and Meehan, C.L. (2011), "Undrained strength characteristics of compacted bentonite/sand mixtures", in Geo-Frontiers 2011, 2011, pp. 2699-2708.

Chirapuntu, S. and Duncan, J.M. (1976), "The role of fill strength in the stability of embankments on soft clay foundations", Office of Research Services, University of California Berkeley, Prepared for: Army Engineer Waterways Experiment Station.

CODOT (2011), “Colorado Department of Transportation Standard Specifications for Road and Bridge Construction”, Colorado State DOT.

CTDOT (2004), “State of Connecticut Department of Transportation Standard Specifications for Roads, Bridges and Incidental Construction”, Connecticut State DOT.

DaCruz, P.T. (1963), "Shear strength characteristics of some residual compacted clays", in 2nd Pan-American Conference on Soil Mechanics and Foundation Engineering, 1963, pp. 73-102.

DEDOT (2001), “The Delaware Department of Transportation Standard Specifications for Road and Bridge Construction”, Delaware State DOT.

Diamond, S. (1971), "Microstructure and pore structure of impact-compacted clays", Clays and Clay minerals, vol. 19 pp. 239-249.

Engineers, U.S.A.C.o. (2003), "Slope Stability", U.S. Army Corps of Engineers, EM 1110-2-1902.

FLDOT (2014), “Florida Department of Transportation Standard Specifications for Road and Bridge Construction”, Florida State DOT.

GDOT (2013), “Standard Specifications Construction of Transportation Systems”, Georgia State DOT.

Geotechdata.info (2013), http://geotechdata.info/parameter.html

Han, X. (2010), "Shear strength and stability of highway embankments in Ohio", M.Sc. Thesis, Ohio University, advisor: T. Masada.

HIDOT (2005), “Standard Specifications and Special Provisions”, Hawaii State DOT.

Hogentogler, C.A. (1936), "Essentials of Soil Compaction", in Sixteenth Annual Meeting of the Highway Research Board, Washington, D.C., 1936, pp. 309-316.

IADOT (2012), “Iowa Department of Transportation Standard Specifications”, Iowa State DOT.

IDDOT (2012), “Standard Specifications for Highway Construction”, Idaho State DOT.

ILDOT (2012), “Standard Specifications for Road and Bridge Construction”, Illinois State DOT.

INDOT (2014), “Indiana Department of Transportation Standard Specifications”, Indiana State DOT.

KSDOT (2007), “Standard Specifications for State Road and Bridge Construction,” Kansas State DOT.

KYDOT “Kentucky Standard Specifications for Road and Bridge Construction”, Kentucky State DOT.

Ladd, C.C. and Foott, R., (1977), "Foundation design of embankments constructed on varved clays", U. S. D. o. Transportation.

LADOTD (2006), “Louisiana Standard Specifications for Roads and Bridges”, Louisiana State DOT.

Lambe, T.W. (1958), "The Structure of Compacted Clay", Soil Mechanics and Foundations Division, ASCE, vol. 84 (SM2)

Lambe, T.W. and Whitman, R.V. (1969), "Soil Mechanics", New York: John Wiley & Sons.

Liang, Y. and Lovell, C. (1982), "Strength of field compacted clayey embankments", Joint Highway Research Project: FHWA and Purdue University, FHWA/IN/JHRP-82/1.

Long, J., Olson, S., Stark, T., and Samara, E. (1998), "Differential movement at embankment-bridge structure interface in Illinois", Transportation Research Record: Journal of the Transportation Research Board, vol. 1633 pp. 53-60.

Masada, T. and Han, X. (2011), "Shear strength and stability of highway embankment slopes in Ohio", in Geo-Frontiers Congress 2011, 2011, pp. 3686-3695.

MASSDOT (2014), “Massachusetts Department of Transportation Construction Standard Details”, Massachusetts State DOT.

MDDOT (2008), “Maryland Standard Specifications for Construction and Materials”, Maryland DOT.

MEDOT (2014), “Maine DOT Standard Specifications”, Maine State DOT.

MIDOT (2012), “Standard Specifications for Construction”, Michigan State DOT

Miller, E.A. and Sowers, G.F. (1957), "Strength Characteristics of Soil-Aggregate Mixtures", Highway Research Board Bulletin, vol. 183 pp. 16-23.

MNDOT (2014), “Department of Transportation Standard Specifications for Construction”, Minnesota State DOT.

MODOT (2011), “Missouri Standard Specification Book for Highway Construction”, Missouri State DOT.

Morgenstern, N. and Price, V.E. (1965), "The analysis of the stability of general slip surfaces", Geotechnique, vol. 15 (1), pp. 79-93.

Morse, R.F. (1941), "Studies of compaction and other properties of soils for low cost roads", PhD Dissertation, Cornel University.

MSDOT (2004), “Mississippi Standard Specifications for Road and Bridge Construction”, Mississippi State DOT.

MTDOT (2014), “Montana Department of Transportation Standard Specifications for Road and Bridge Construction”, Montana State DOT.

NCDOT (2012), "Pavement Condition Survey Manual", North Carolina Department of Transportation.

NCDOT (2012), “Standard Specifications and Provisions”, North Carolina State DOT.

NCHRP (1971), "NCHRP Synthesis of Highway Practice 8: Construction of Embankments", Transportation Research Board, Washington, D.C.

NCHRP (1975), "NCHRP Synthesis of Highway Practice 29: Treatment of Soft Foundations for Highway Embankments", Transportation Research Board, Washington, D.C.

NCHRP (1989), "NCHRP Synthesis of Highway Practice 147: Treatment of problem foundations for highway embankments", Transportation Research Board, Washington, D.C.

NCHRP (1990), "NCHRP Synthesis of Highway Practice 159: Design and Construction of Bridge Approaches", Transportation Research Board, Washington, D.C.

NCHRP (2004), "NCHRP Report 529: Guideline and Recommended Standard for Geofoam Applications in Highway Embankments", Transportation Research Board, Washington, D.C.

NDDOT “Standard Specifications for Road and Bridge Construction”, North Dakota State DOT.

NEDOT (2012), “Nebraska Department of Roads Geotechnical Policies and Procedures Manual”, Nebraska State DOT.

NHDOT (2010). “New Hampshire Standard Specifications for Road and Bridge Construction”, New Hampshire State DOT

NVDOT (2010), “Nevada Department of Transportation Standard Plans for Road and Bridge Construction”, Nevada State DOT.

NYSDOT (2012), "NYSDOT Geotechnical Design Manual", New York State DOT.

ODOT (2014), "Oregon DOT Geotechnical Design Manual", Oregon DOT.

ODOT, (2010), "Shear Strength of Proposed Embankments", ODOT, vol. Geotechnical Bulletin (GB 6), pp. 1-7. Available: http://www.dot.state.oh.us/Divisions/Engineering/Geotechnical/Geotechnical_Documents/GB6_Shear_Strength.pdf.

OKDOT (2009), “Oklahoma Department of Transportation Transportation Commission”, Oklahoma State DOT.

Olson, R.E. (1963), "Effective stress theory of soil compaction", Journal of the Soil Mechanics and Foundations Division, vol. 89 (2), pp. 27-46.

ORDOT (2015), “Oregon Standard Specifications for Construction”, Oregon State DOT.

PENNDOT (2010), “Roadway Construction Standards”, Pennsylvania State DOT.

Proctor, R.R. (1933), "Fundamental Principles of Soil Compaction", Engineering News-Record, vol. 111 (9), pp. 245-248.

RIDOT (2009), “Rhode Island Department of Transportation Standard Specifications for Road and Bridge Construction”, Rhode Island State DOT.

Rutledge, P.C. (1947), "Cooperative Triaxial Shear Research Program", Progress Report on Soil Mechanics Fact Finding Survey, U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

SCDOT (2007), “South Carolina Department of Transportation Standard Specifications for Highway Construction”, South Carolina State DOT.

SDDOT (2015), “South Dakota Standard Specifications for Roads and Bridges”, South Dakota State DOT.

Seed, H.B. and Chan, C.K. (1959), "Structure and Strength Characteristics of Compacted Clays", Journal of the Soil Mechanics and Foundations Division, vol. 85 (5), pp. 87-128.

Seed, H.B., Mitchell, J.K., and Chan, C.K. (1960), "The strength of compacted cohesive soils", in Research Conference on Shear Strength of Cohesive Soils, University of Colorado, Boulder, Colorado, United States, 1960, pp. 877-964.

Spencer, E. (1967), "A method of analysis of the stability of embankments assuming parallel inter-slice forces", Geotechnique, vol. 17 (1), pp. 11-26.

Sridharan, A., Altschaeffl, A., and Diamond, S. (1971), "Pore size distribution studies", Journal of the soil mechanics and foundations division, vol. 97 (5), pp. 771-787.

T.D., S., Olson, S., and Long, J. (1995), "Differential movement at the embankment/structure interface-mitigation and rehabilitation", Illinois DOT, Springfield, Illinois, LAB-H1 FY93.

TDOT (2012), "TDOT Geotechnical Manual", Texas State DOT.

Terzaghi, K., Peck, R.B., and Mesri, G. (1996), "Soil mechanics in engineering practice", 3rd ed.: John Wiley & Sons.

TNDOT (2006), “Tennessee Standard Specifications for Road and Bridge Construction”, Tennessee State DOT.

UTDOT (2014), “UDOT geotechnical Manual of Instruction”, Utah State DOT.

VDOT (2014), "Virginia Manual of Instructions-Chapter 3: Geotechnical Engineering", Virginia DOT.

VTDOT (2011), “Standard Specifications for the Construction Book”, Vermont State DOT.

WADOT (2014), “Standard Specifications for Road, Bridge, and Municipal Construction”, Washington State DOT

White, D. (1980), "The fabric of a medium plastic clay compacted in the laboratory and in the field", M.Sc. Thesis, Purdue University.

WIDOT, (2014), “Standard Specifications”, Wisconsin State DOT.

Wright, S.G. (1969), "A study of slope stability and the undrained shear strength of clay shales", PhD Dissertation, University of California, Berkeley.

WSDOT (2015), "WSDOT Geotechnical Design Manual", Washington State DOT, M 46-03.11.

WVDOT (2010), “West Virginia Division of Highways Standard Specifications Roads and Bridges”, West Virginia State DOT.

WYDOT (2010), “State of Wyoming Standard Specifications for Road and Bridge Construction”, Wyoming State DOT.

Yin, J.-H. (1999), "Properties and behaviour of Hong Kong marine deposits with different clay contents", Canadian Geotechnical Journal, vol. 36 (6), pp. 1085-1095.

APPENDIX A: UU TRIAXIAL TESTS

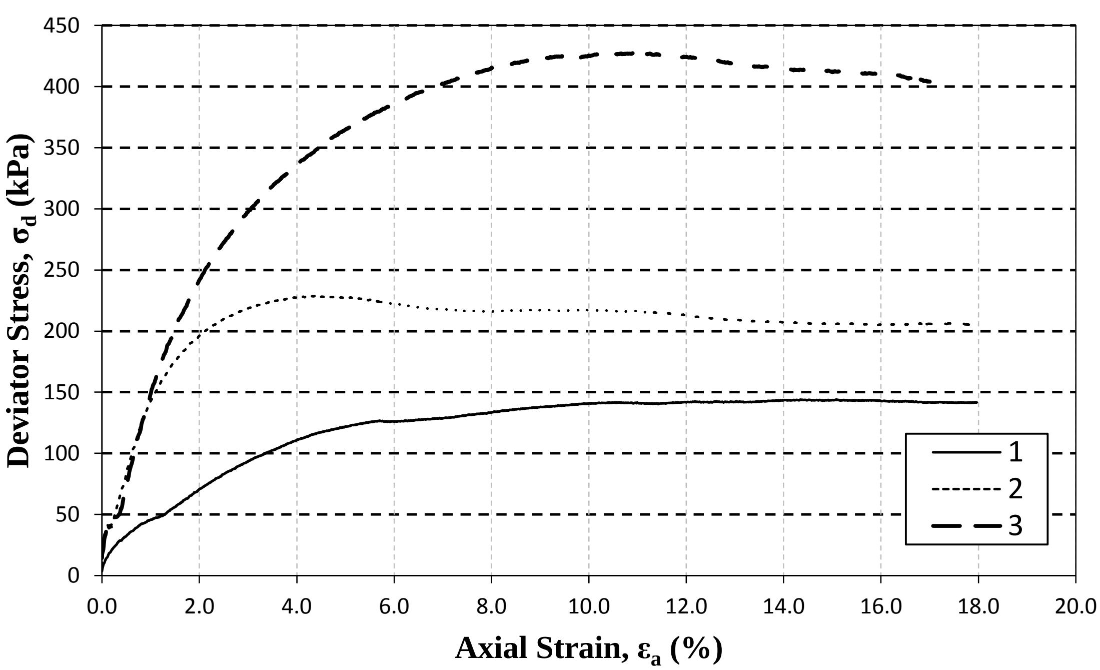
Project Name: Improvement of Material Criteria for Highway Embankment Construction					
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	5/21/2015

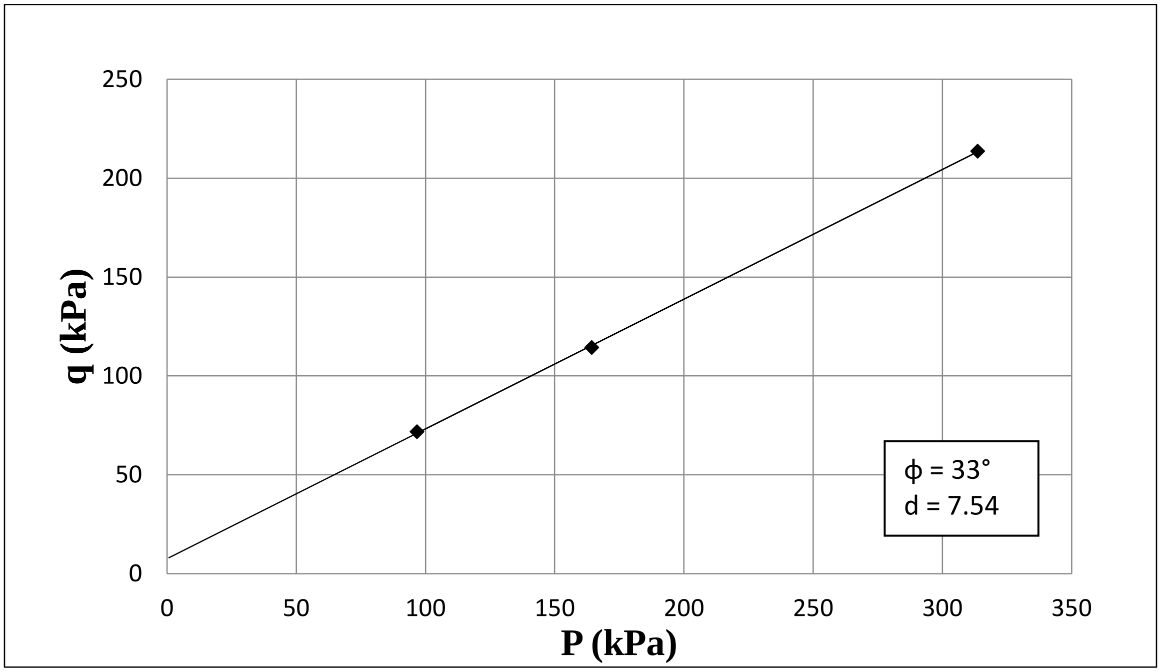
Sample No.	1	2	3
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.77	38.17	35.86
Average Height (mm)	88.99	83.88	90.59
Weight (g)	188.03	194.18	188.04
Water Content (%)	12.4	13	12.3
Dry Unit Weight (kN/m^3)	18.34	17.61	17.93
Saturation (%)	74	69	69
Strain at Failure (%)	15	4.65	11.7
Max Deviator Stress (kPa)	144	228.8	427.4

Test 1	Test 2	Test 3

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS: SM		Gs:	2.72
Strain Rate:	1%/min	AASHTO: A-2-4 (0)		Date:	5/21/2015





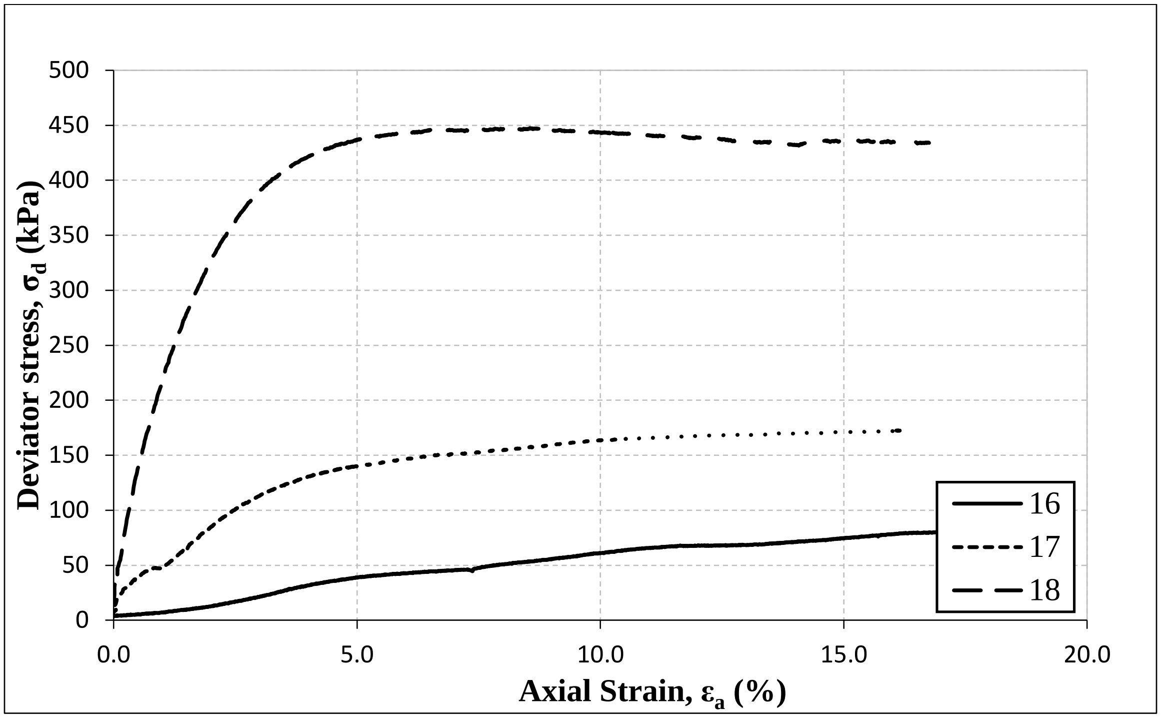
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/2/2015

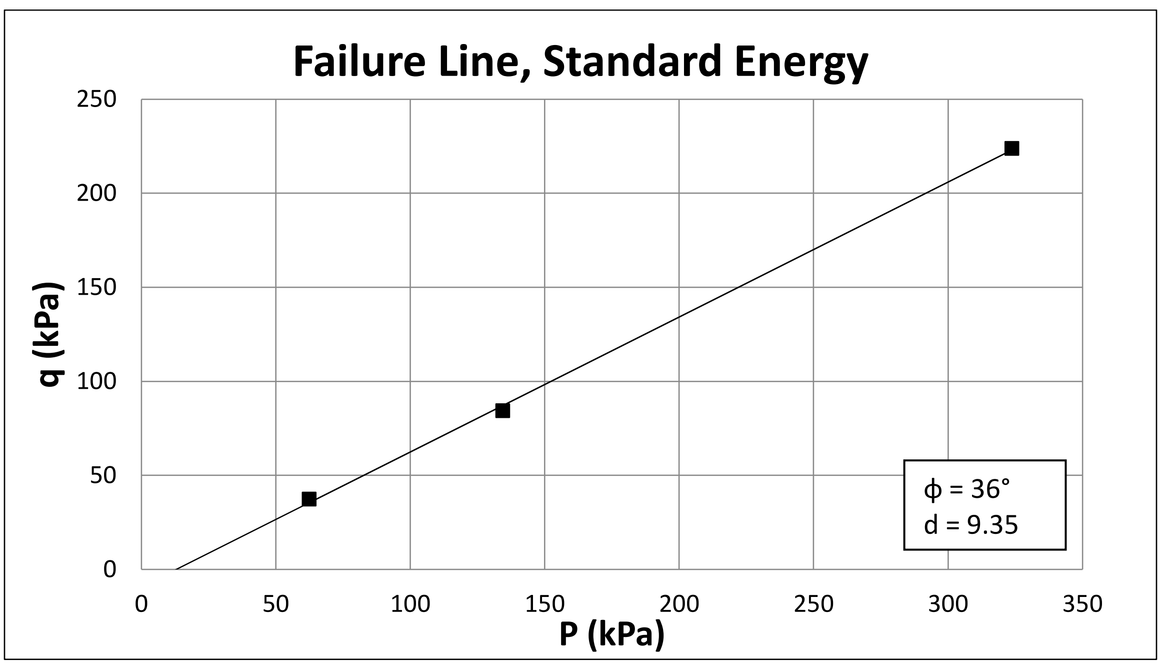
Sample No.	16	17	18
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	34.00	34.83	35.86
Average Height (mm)	87.94	85.26	90.65
Weight (g)	176.54	166.07	182.80
Water Content (%)	14.3	14.1	14.3
Dry Unit Weight (kN/m^3)	18.98	17.57	17.13
Saturation (%)	95	74	69
Strain at Failure (%)	15.00	15.00	9.41
Max Deviator Stress (kPa)	82.0	173.0	447.5

Test 16	Test 17	Test 18

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/2/2015



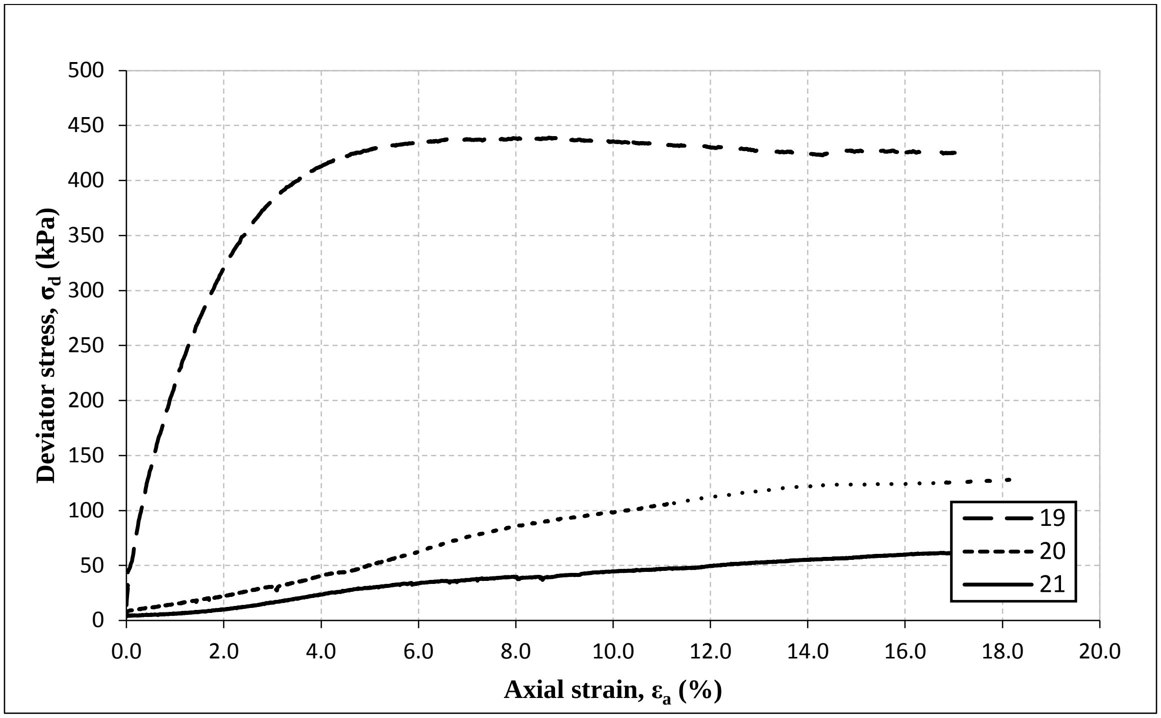


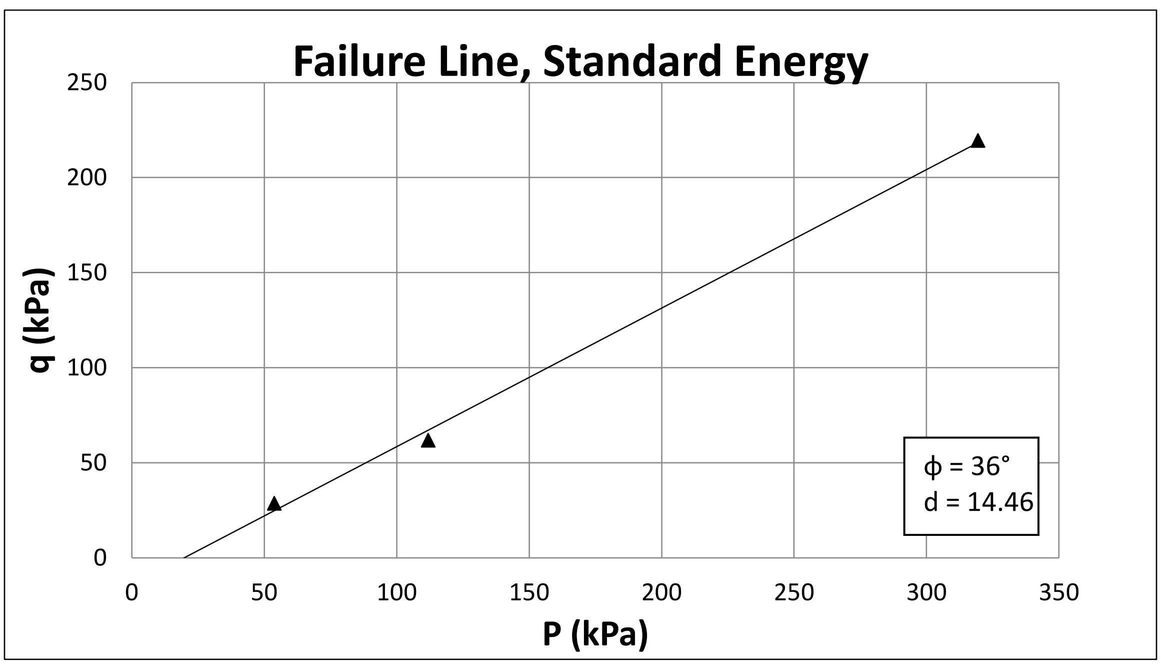
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/3/15

Sample No.	19	20	21
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	36.02	35.79	36.17
Average Height (mm)	92.38	85.15	89.07
Weight (g)	187.91	172.78	186.55
Water Content (%)	15.8	15.5	15.6
Dry Unit Weight (kN/m^3)	16.91	17.12	17.30
Saturation (%)	74	76	78
Strain at Failure (%)	15.00	15.00	9.58
Max Deviator Stress (kPa)	64.8	132.6	439.1

Test 19	Test 20	Test 21
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/3/15



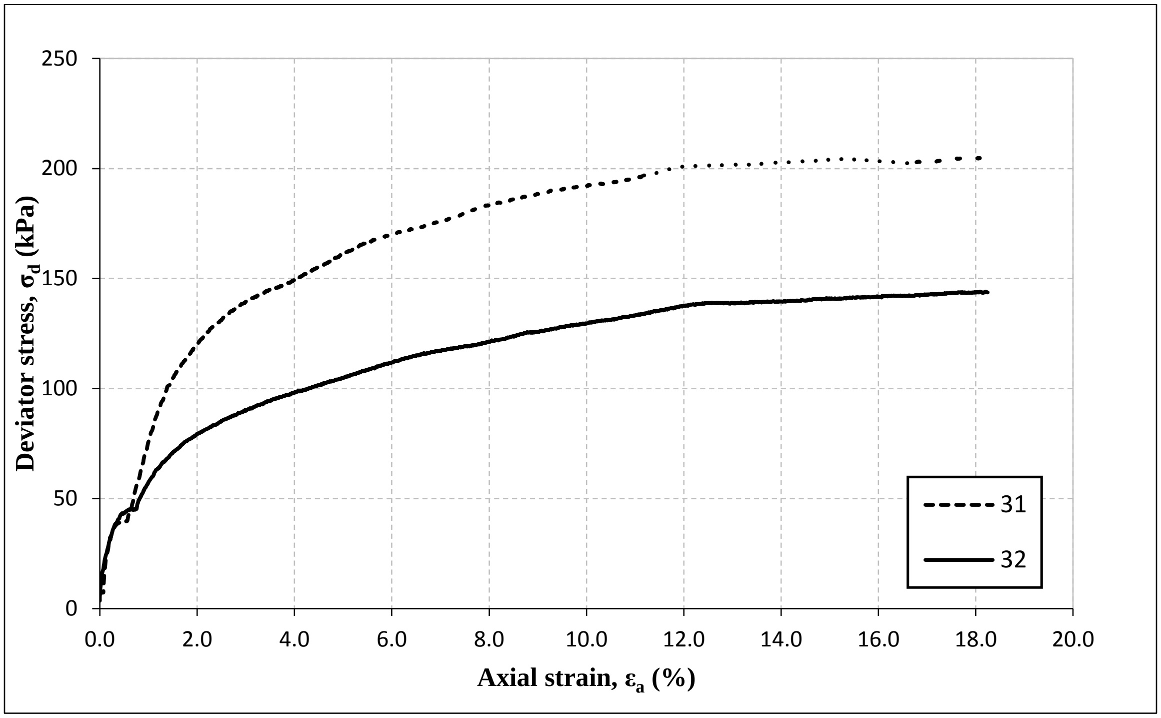


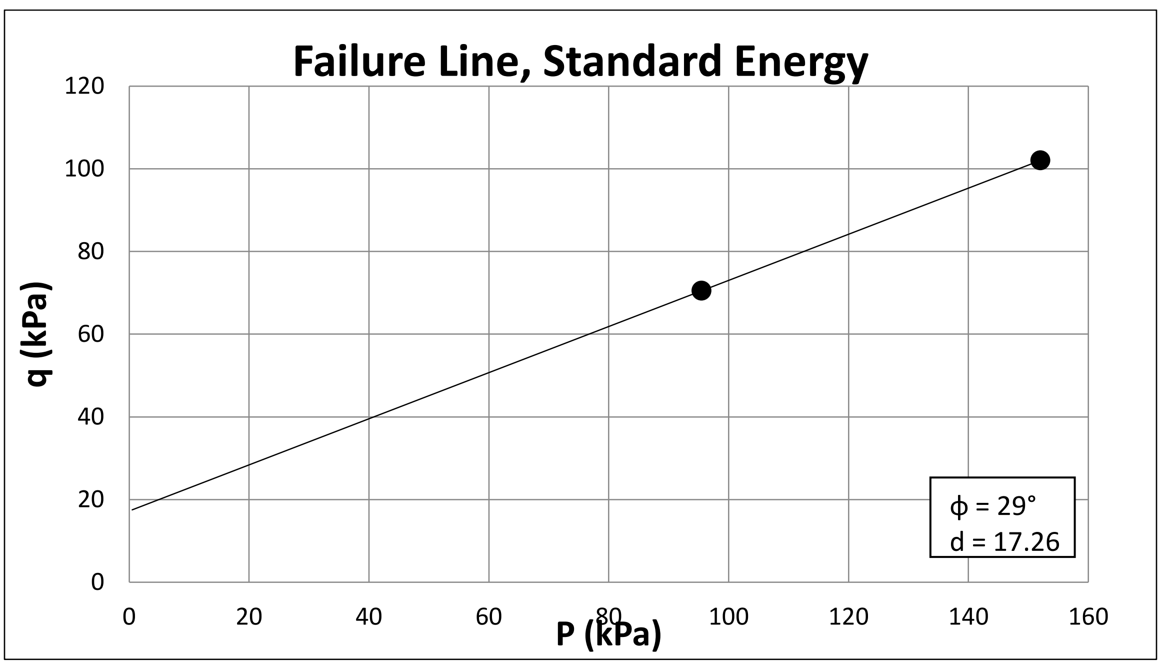
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/10/15

Sample No.	31	32	
Cell Pressure (kPa)	25	50	
Average Diameter (mm)	36.05	38.17	
Average Height (mm)	89.06	83.66	
Weight (g)	167.55	175.74	
Water Content (%)	9.2	9.5	
Dry Unit Weight (kN/m^3)	16.56	16.44	
Saturation (%)	41	42	
Strain at Failure (%)	15.00	15.00	
Max Deviator Stress (kPa)	144.1	205.0	

Test 31	Test 32	

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/10/15



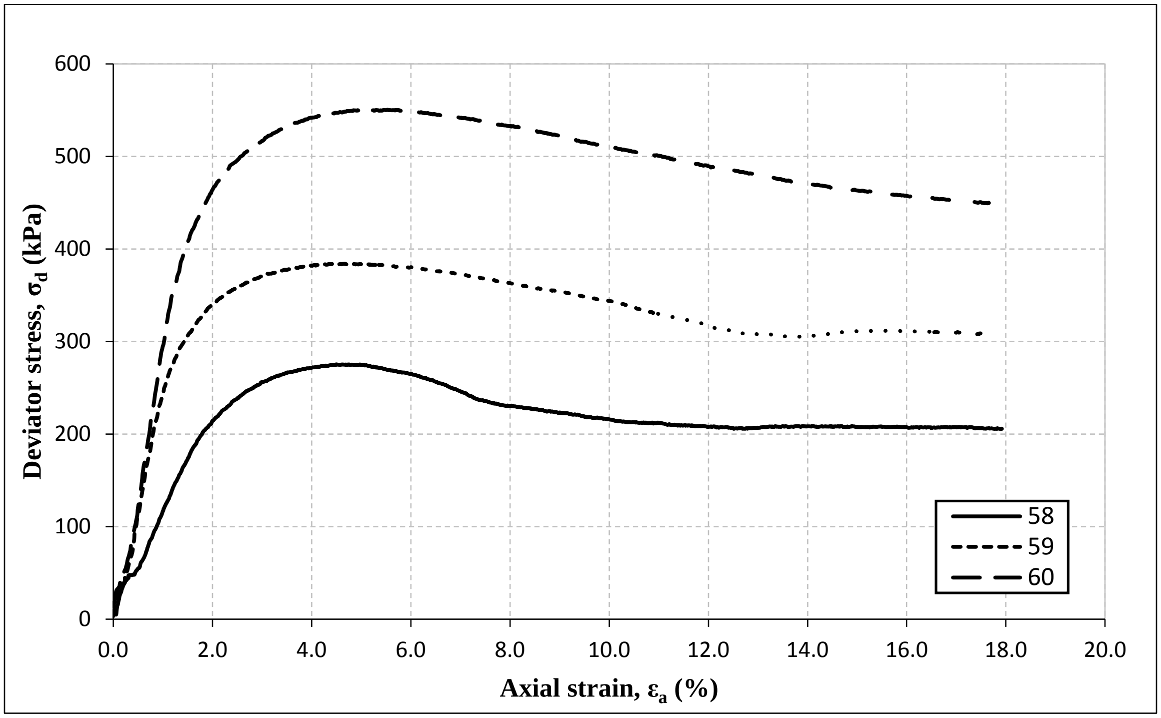


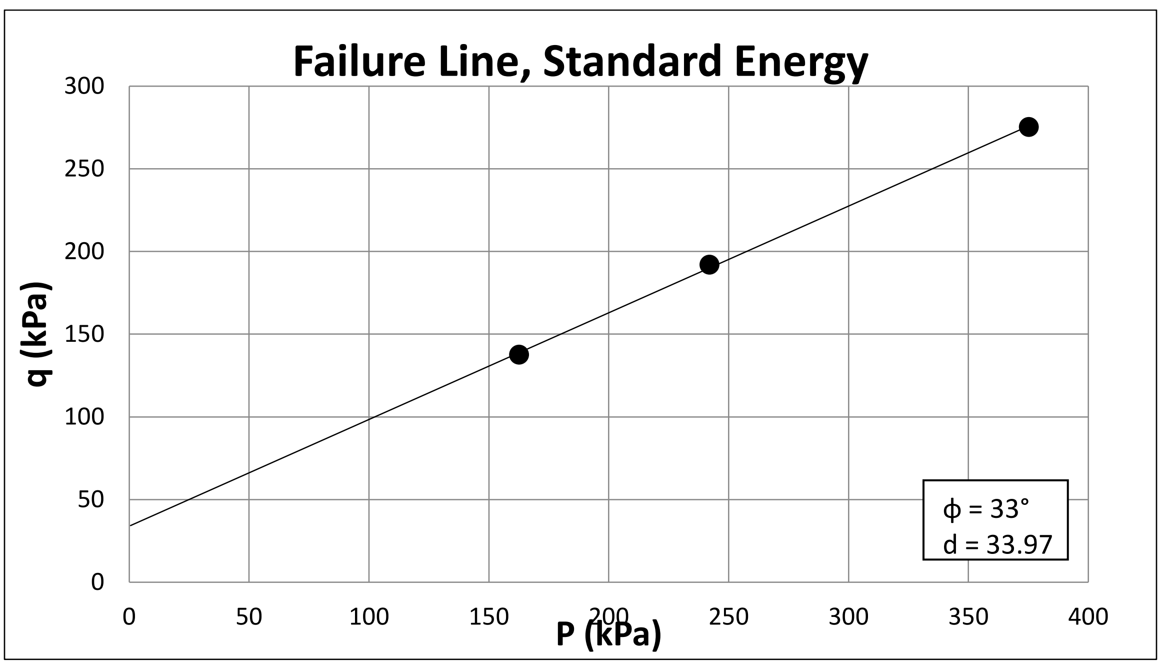
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/12/2015

Sample No.	58	59	60
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.57	35.73	35.54
Average Height (mm)	71.73	71.50	71.63
Weight (g)	134.69	134.66	134.41
Water Content (%)	4.0	3.9	3.8
Dry Unit Weight (kN/m^3)	17.82	17.73	17.87
Saturation (%)	22	21	21
Strain at Failure (%)	4.69	4.80	5.83
Max Deviator Stress (kPa)	275.3	384.0	550.4

Test 58	Test 59	Test 60

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/12/2015



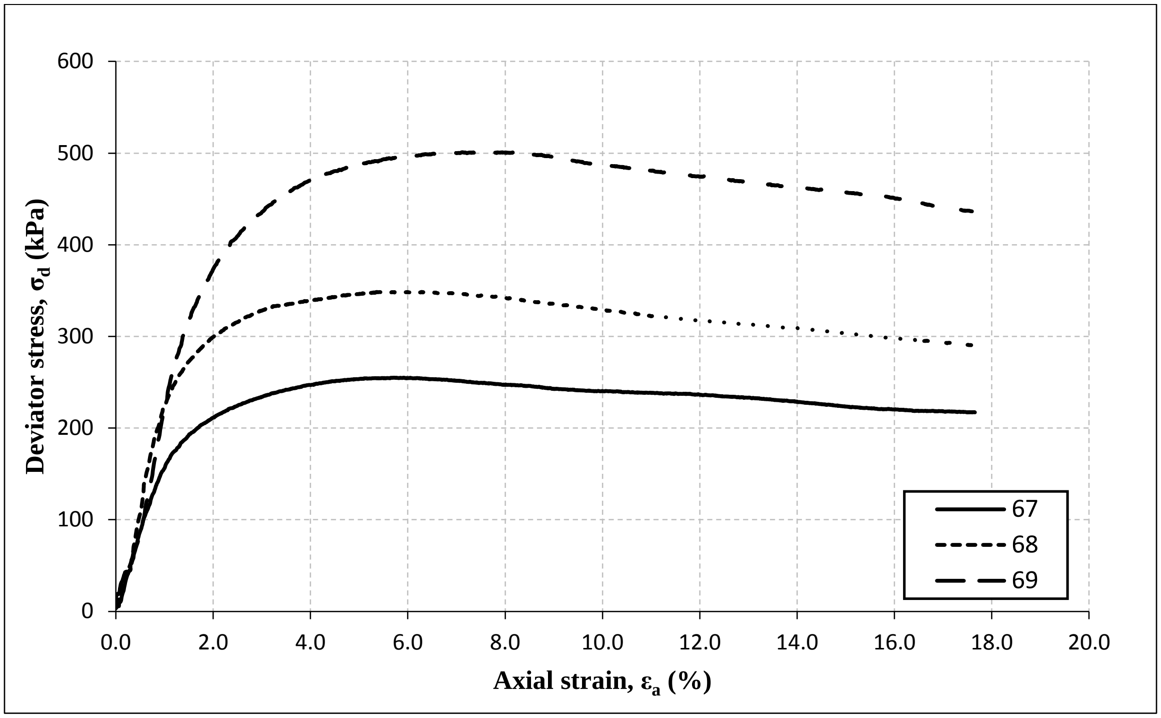


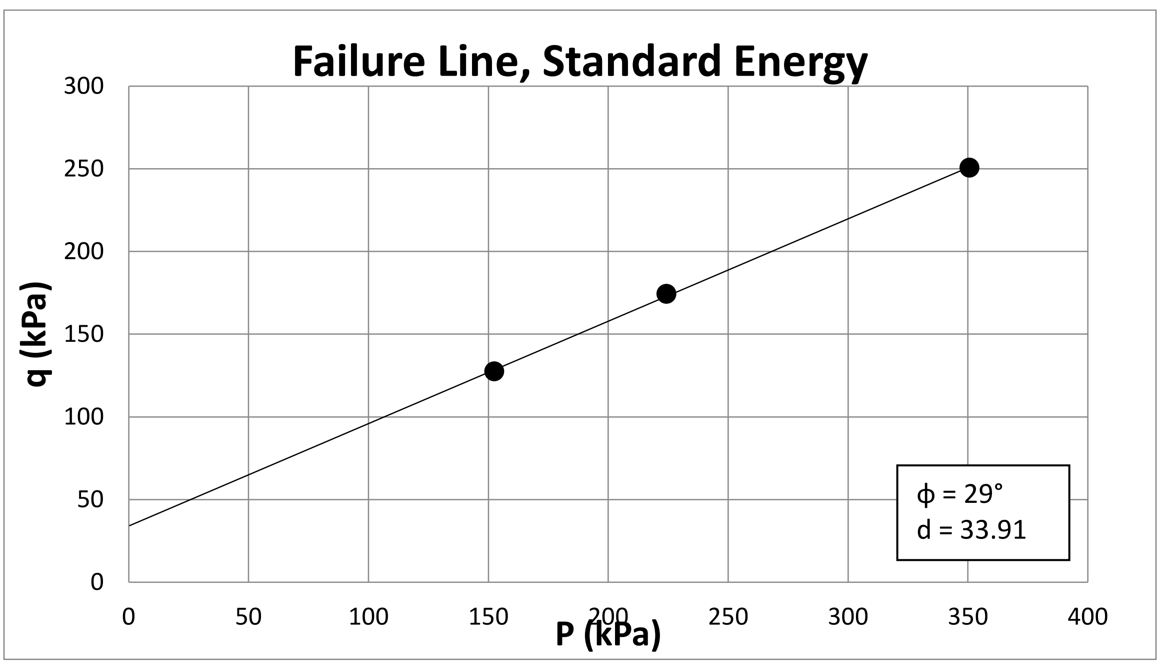
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/2015

Sample No.	67	68	69
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.71	35.73	35.43
Average Height (mm)	71.70	71.50	71.77
Weight (g)	138.83	138.63	136.27
Water Content (%)	5.9	5.4	5.4
Dry Unit Weight (kN/m^3)	17.9	17.9	17.9
Saturation (%)	33	30	30
Strain at Failure (%)	6.42	5.83	8.33
Max Deviator Stress (kPa)	255.0	348.7	501.5

Test 67	Test 68	Test 69

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/2015



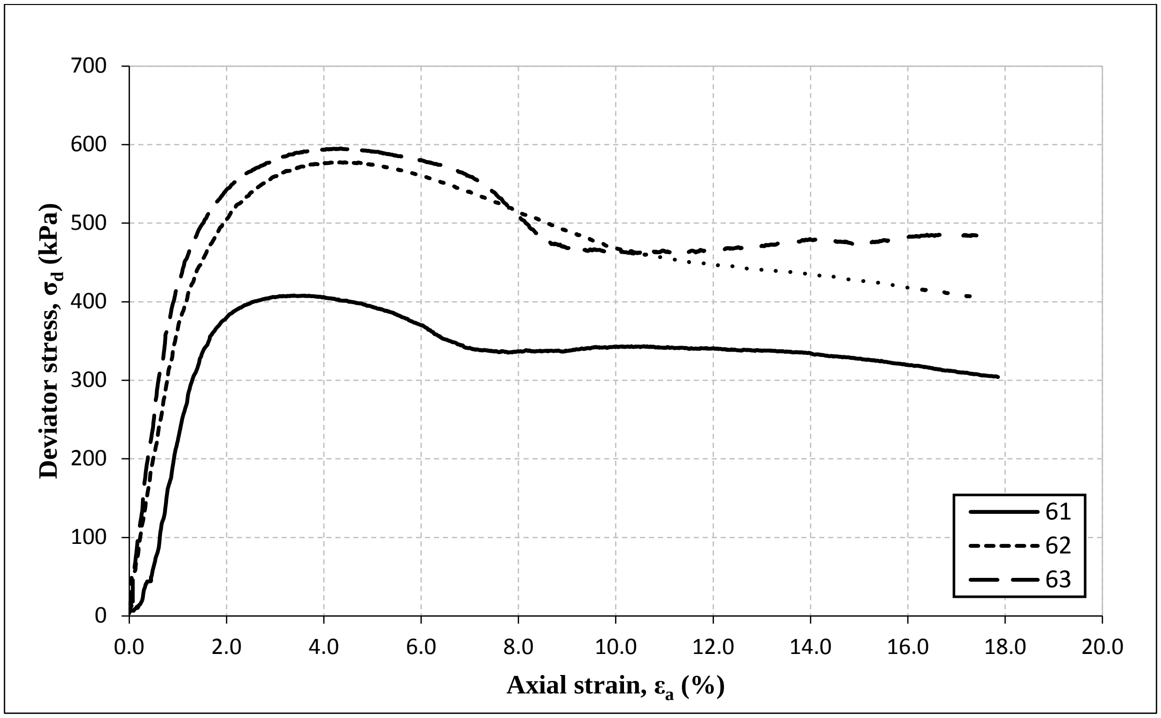


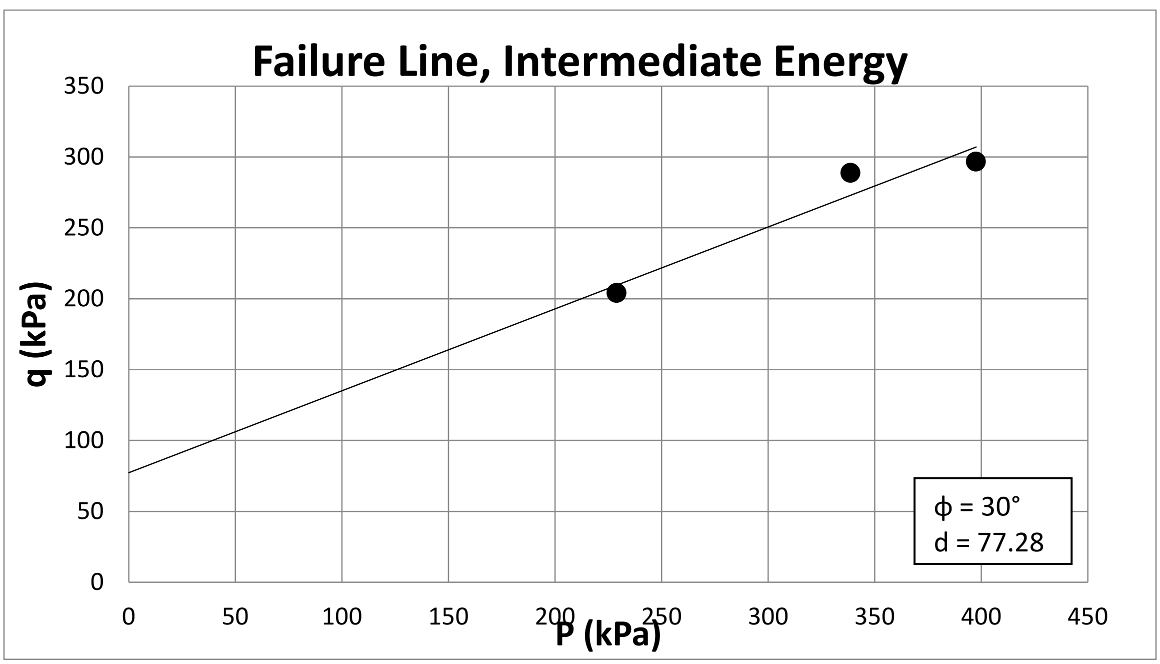
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/12/15

Sample No.	61	62	63
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.87	35.73	35.54
Average Height (mm)	71.64	71.50	71.71
Weight (g)	142.22	139.03	139.24
Water Content (%)	4.3	3.8	4.7
Dry Unit Weight (kN/m^3)	17.54	18.32	18.34
Saturation (%)	22	23	28
Strain at Failure (%)	3.68	5.20	4.85
Max Deviator Stress (kPa)	408.0	577.5	595.1

Test 61	Test 62	Test 63

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/12/15



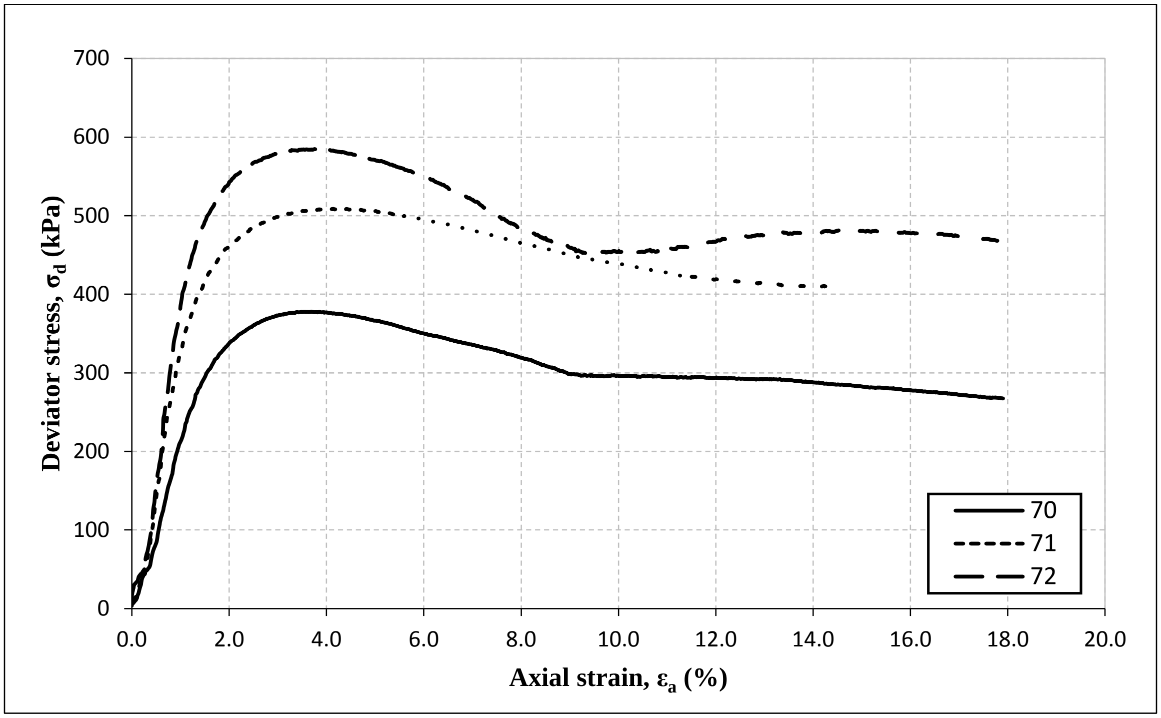


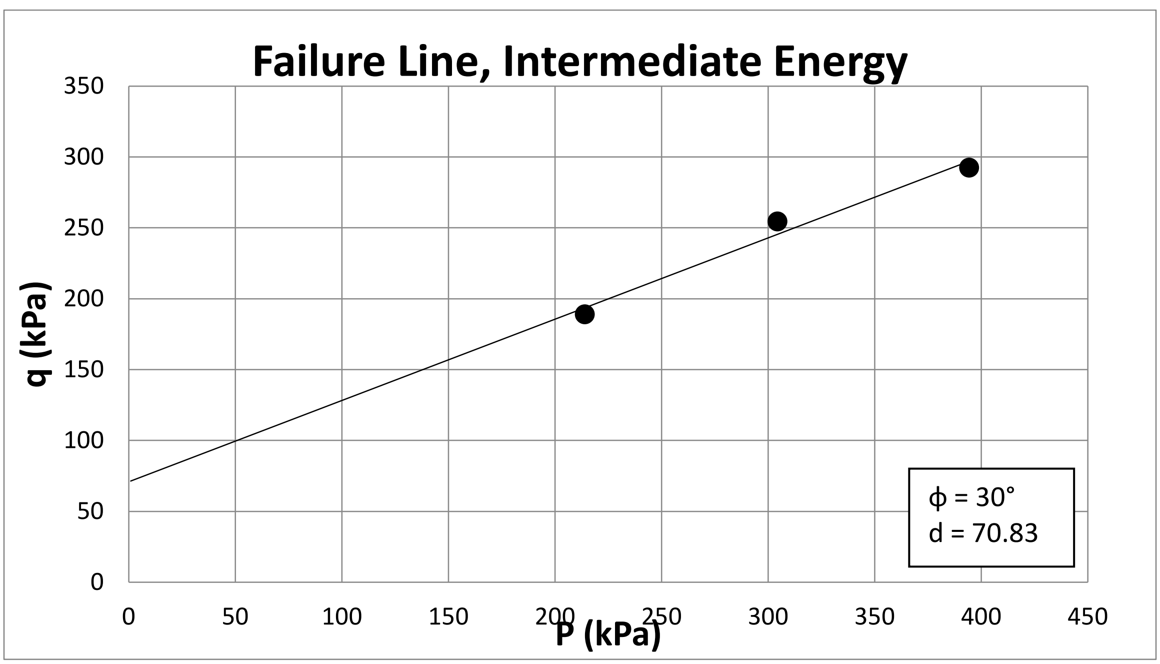
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/15

Sample No.	70	71	72
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.60	35.73	35.63
Average Height (mm)	71.68	71.50	71.84
Weight (g)	145.49	144.58	143.90
Water Content (%)	5.9	6.4	5.8
Dry Unit Weight (kN/m^3)	18.89	18.58	18.62
Saturation (%)	39	40	36
Strain at Failure (%)	3.93	8.52	4.03
Max Deviator Stress (kPa)	378.0	508.9	584.8

Test 70	Test 71	Test 72

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/15



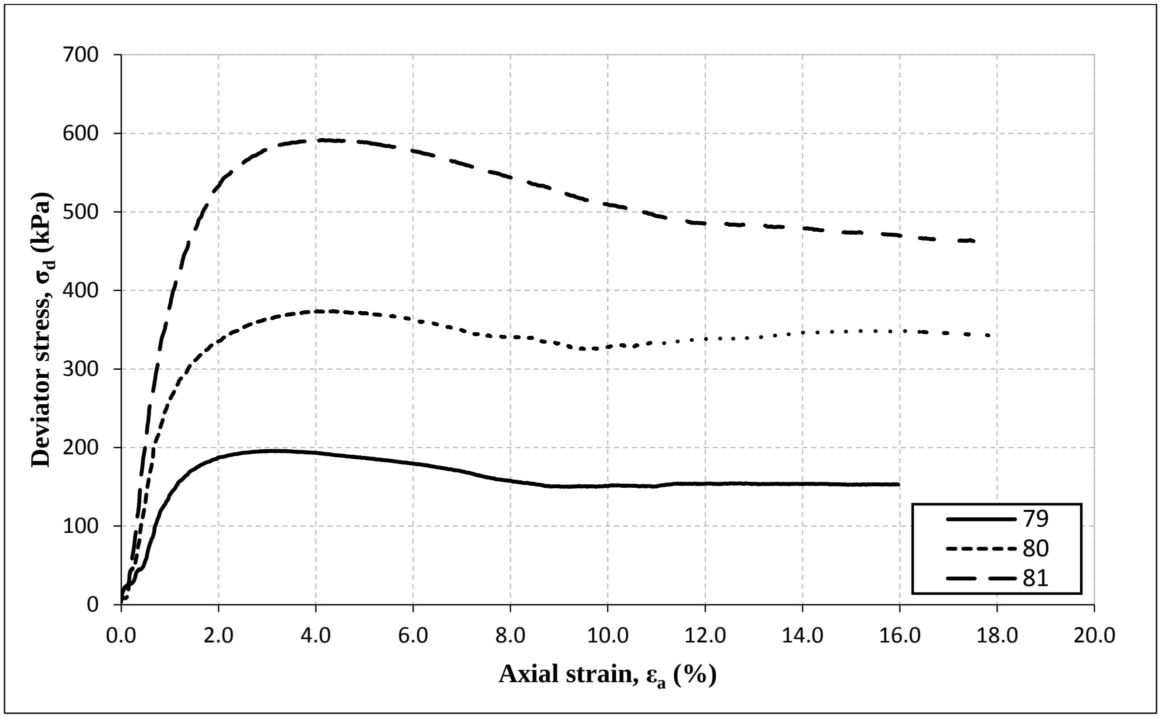


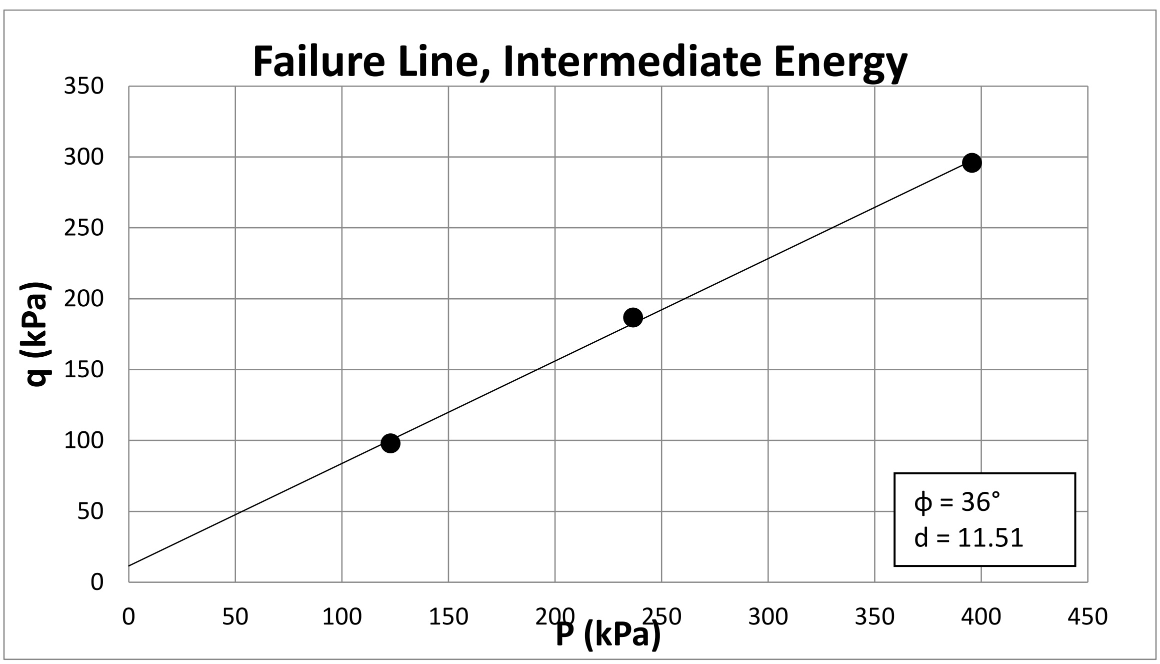
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/21/15

Sample No.	79	80	81
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.56	35.73	35.58
Average Height (mm)	71.32	71.50	71.40
Weight (g)	151.63	150.37	153.29
Water Content (%)	9.8	9.0	10.0
Dry Unit Weight (kN/m^3)	19.12	18.87	19.25
Saturation (%)	67	59	71
Strain at Failure (%)	3.46	4.69	4.51
Max Deviator Stress (kPa)	195.8	373.7	591.5

Test 79	Test 80	Test 81

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Wayne, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/21/15



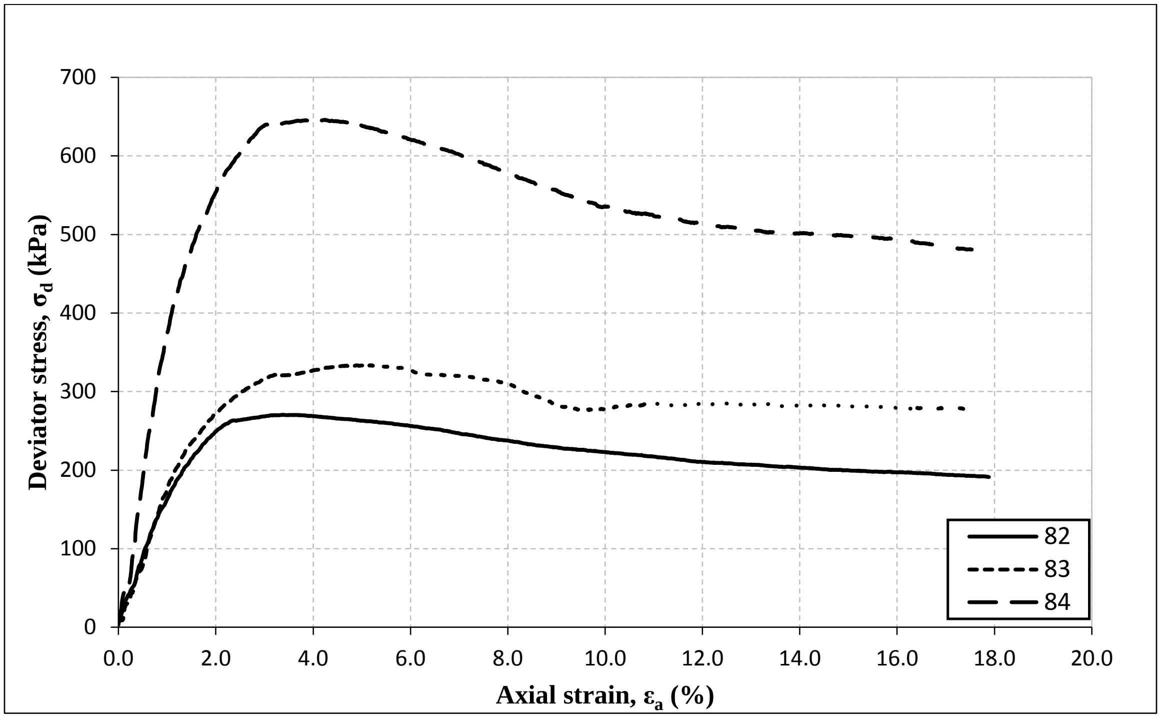


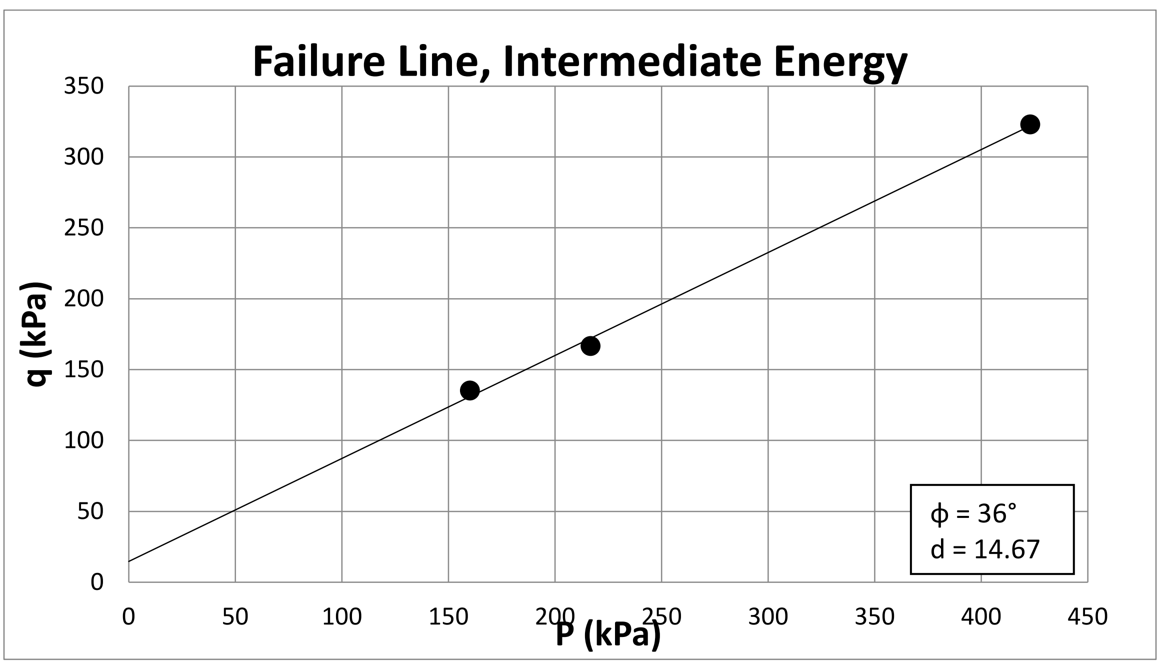
Project Name: Improvement of Material Criteria for Highway Embankment Construction			
Sample: Wayne, Intermediate Compaction			
Specimen Type: Compacted		USCS: SM	Gs: 2.72
Strain Rate: 1%/min		AASHTO: A-2-4 (0)	Date: 8/28/15

Sample No.	82	83	84
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.74	35.73	35.56
Average Height (mm)	71.48	71.50	71.07
Weight (g)	152.22	153.36	154.04
Water Content (%)	10.9	10.4	10.7
Dry Unit Weight (kN/m^3)	18.78	19.00	19.34
Saturation (%)	71	70	77
Strain at Failure (%)	3.66	5.60	4.76
Max Deviator Stress (kPa)	270.5	333.5	646.0

Test 82	Test 83	Test 84

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/28/15



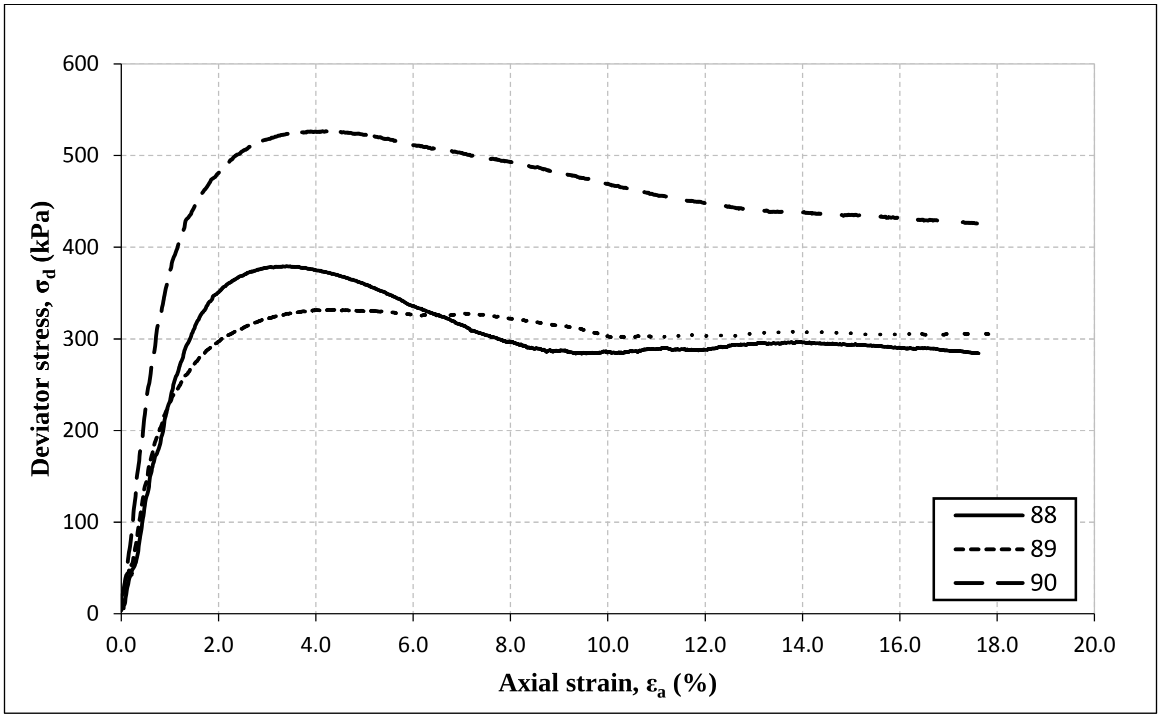


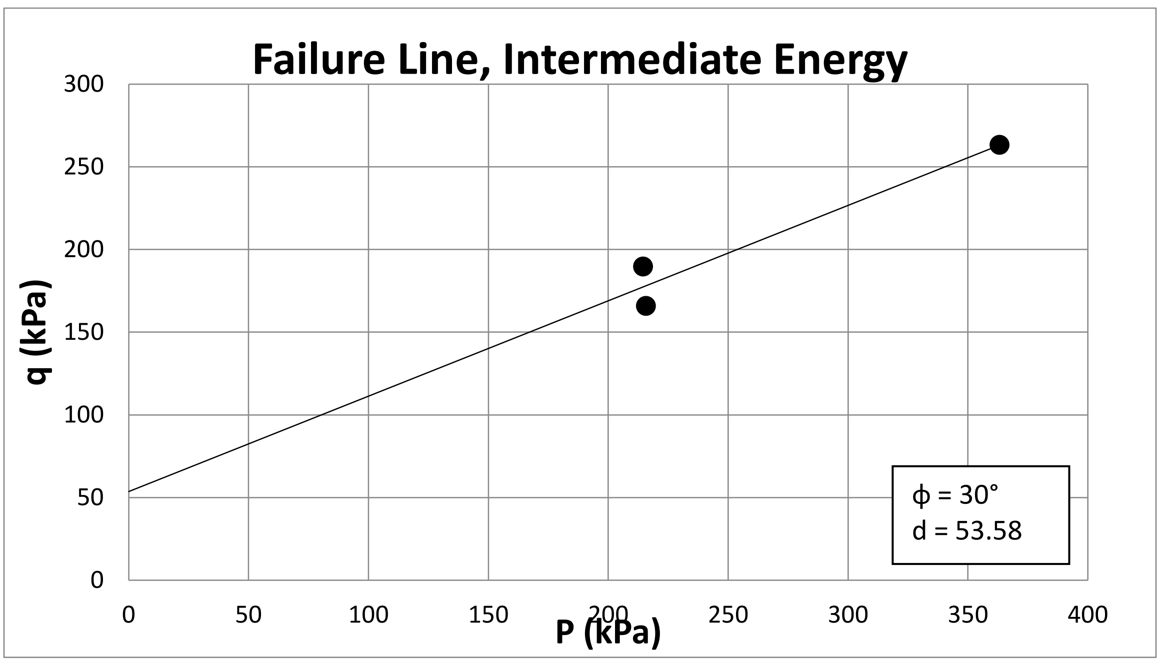
Project Name:	Improvement of Material Criteria for Highway Embankment Construction		
Sample: Wayne, Intermediate Compaction			
Specimen Type:	Compacted	USCS: SM	Gs: 2.72
Strain Rate:	1%/min	AASHTO: A-2-4 (0)	Date: 9/2/15

Sample No.	88	89	90
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.51	35.50	35.40
Average Height (mm)	71.26	71.46	71.29
Weight (g)	147.89	146.63	146.13
Water Content (%)	7.3	7.3	7.6
Dry Unit Weight (kN/m^3)	19.16	18.96	18.97
Saturation (%)	51	49	51
Strain at Failure (%)	4.01	4.72	4.68
Max Deviator Stress (kPa)	379.2	331.6	526.7

Test 88	Test 89	Test 90

Project Name:	Improvement of Material Criteria for Highway Embankment Construction		
Sample: Wayne, Intermediate Compaction			
Specimen Type: Compacted	USCS: SM	Gs: 2.72	
Strain Rate: 1%/min	AASHTO: A-2-4 (0)	Date: 9/2/15	



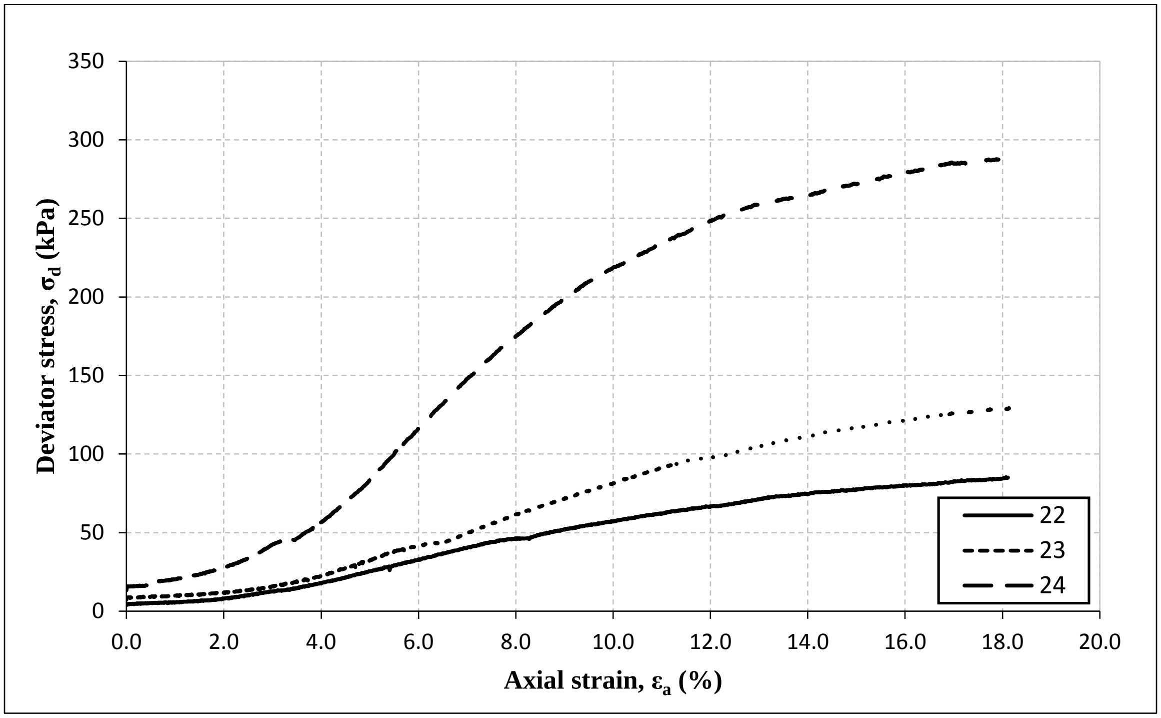


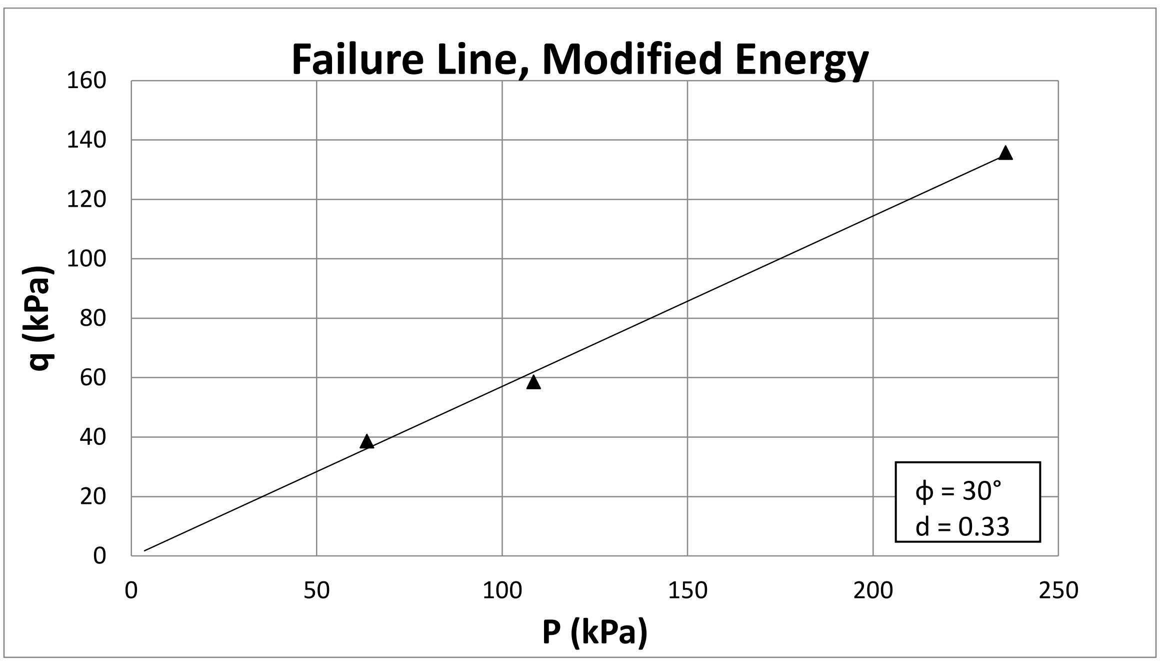
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/2/15

Sample No.	22	23	24
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	34.90	35.88	35.39
Average Height (mm)	86.60	80.45	91.89
Weight (g)	183.36	179.22	196.10
Water Content (%)	15.6	15.8	15.4
Dry Unit Weight (kN/m^3)	18.68	18.65	18.44
Saturation (%)	99	100	94
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	90.4	132.5	292.6

Test 22	Test 23	Test 24
		

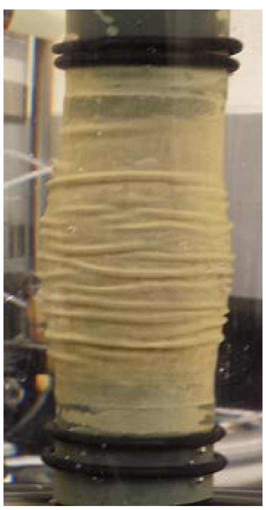
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/2/15



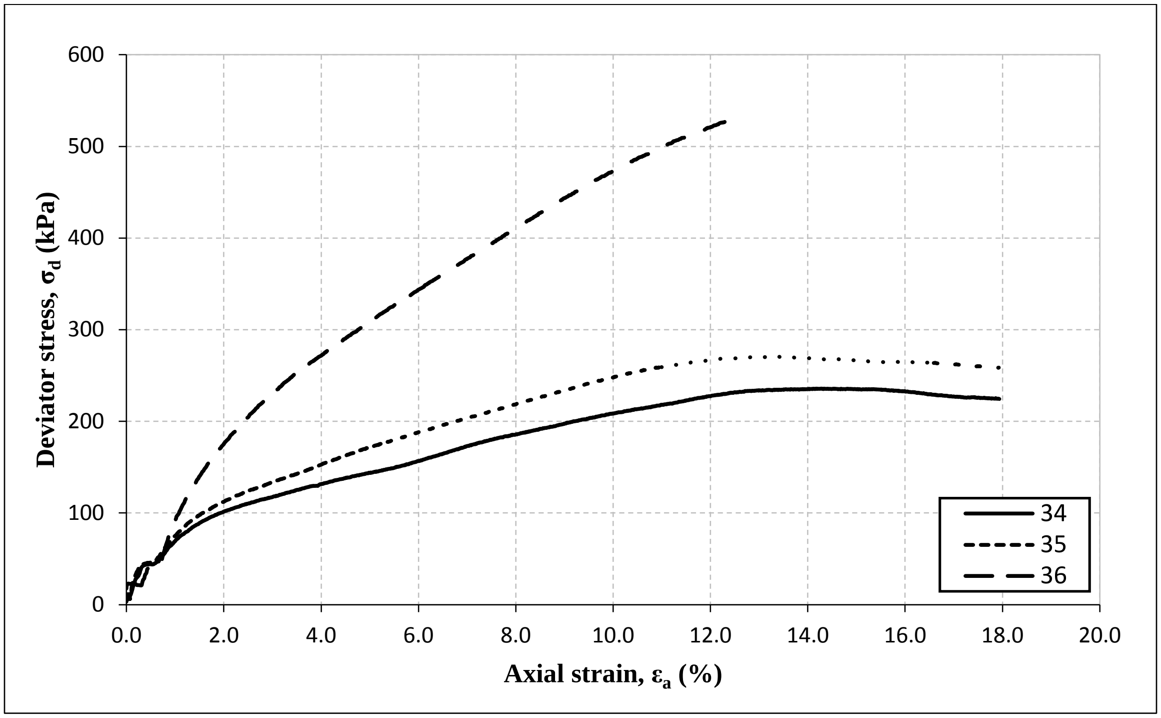


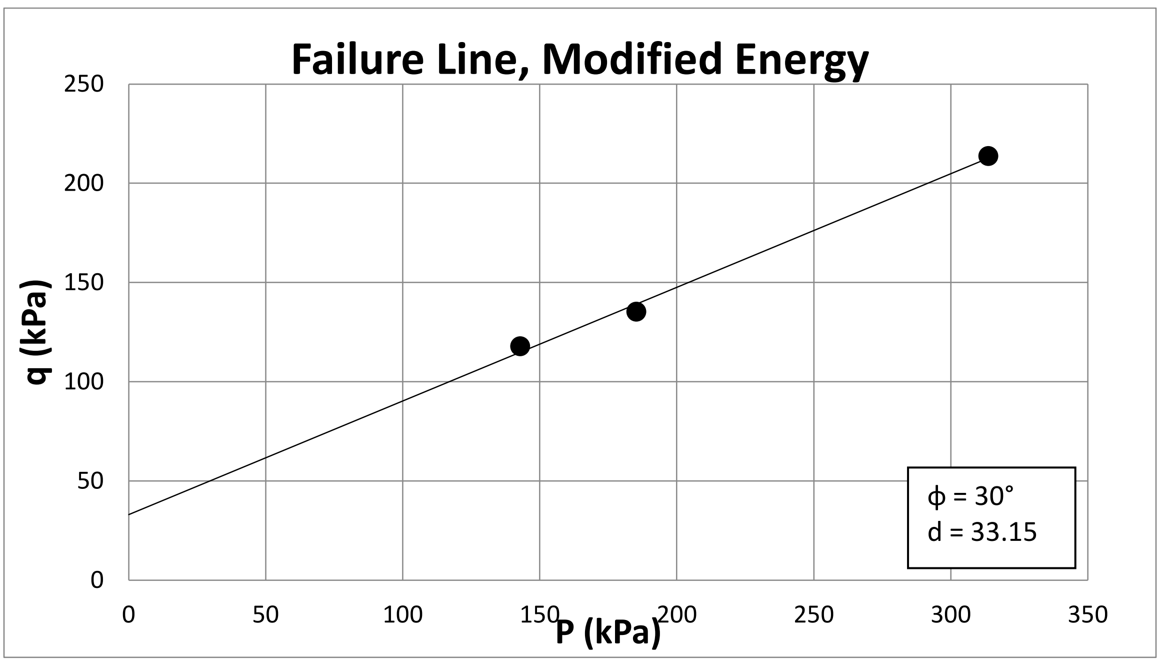
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/10/15

Sample No.	34	35	36
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	36.19	35.97	36.28
Average Height (mm)	94.88	89.44	92.76
Weight (g)	186.75	172.55	187.86
Water Content (%)	9.3	9.9	9.1
Dry Unit Weight (kN/m^3)	17.18	16.94	17.60
Saturation (%)	46	47	48
Strain at Failure (%)	14.47	13.19	15.00
Max Deviator Stress (kPa)	235.8	270.7	558.6

Test 34	Test 35	Test 36
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	6/10/15



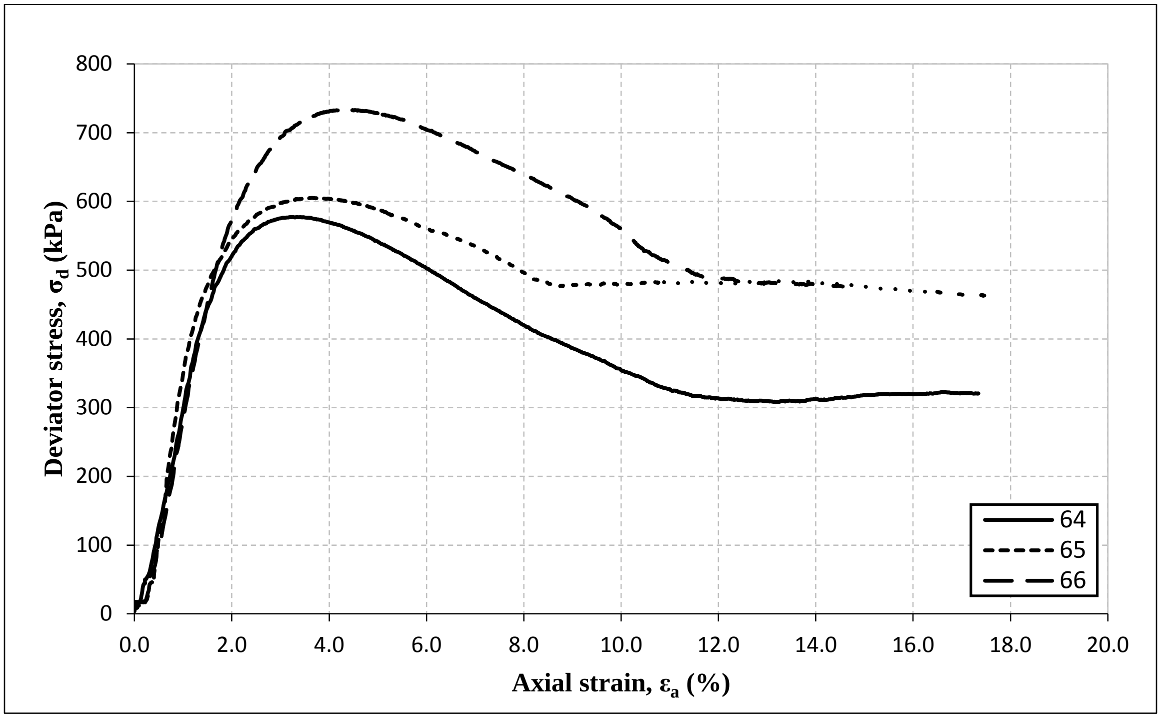


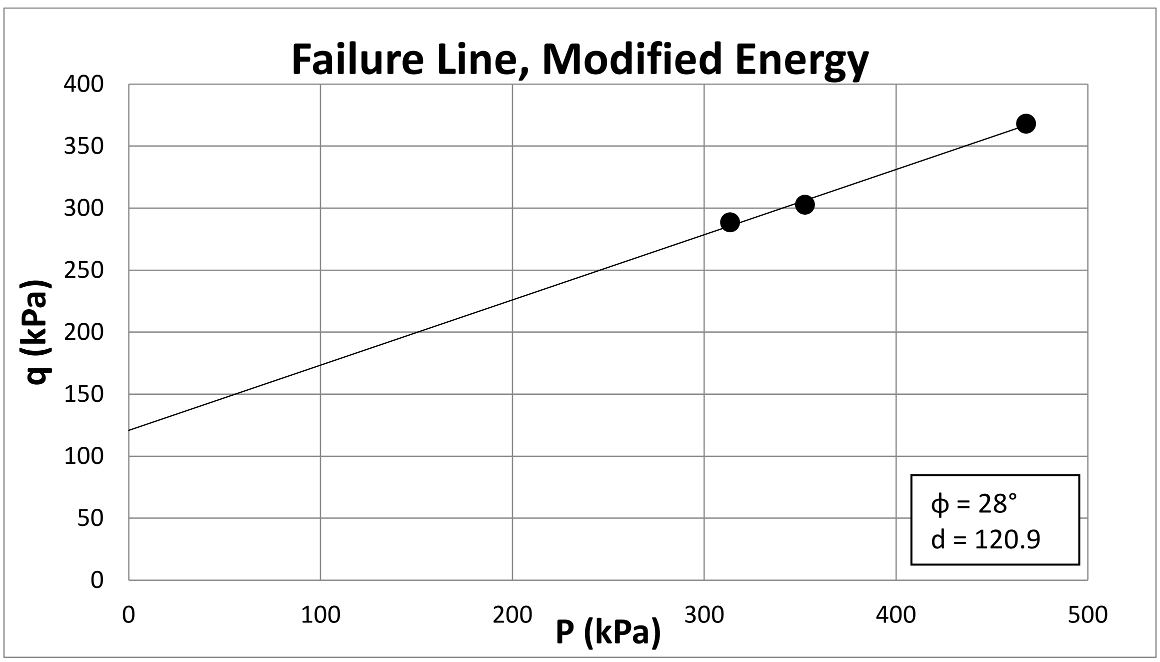
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/12/15

Sample No.	64	65	66
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.65	35.71	35.67
Average Height (mm)	71.60	71.50	72.13
Weight (g)	144.92	146.99	146.16
Water Content (%)	4.6	4.0	4.8
Dry Unit Weight (kN/m^3)	19.00	19.35	18.97
Saturation (%)	31	29	32
Strain at Failure (%)	4.32	4.15	4.76
Max Deviator Stress (kPa)	577.3	605.2	732.9

Test 64	Test 65	Test 66

Project Name: Improvement of Material Criteria for Highway Embankment Construction			
Sample: Wayne, Modified Compaction			
Specimen Type:	Compacted	USCS: SM	Gs: 2.72
Strain Rate:	1%/min	AASHTO: A-2-4 (0)	Date: 8/12/15



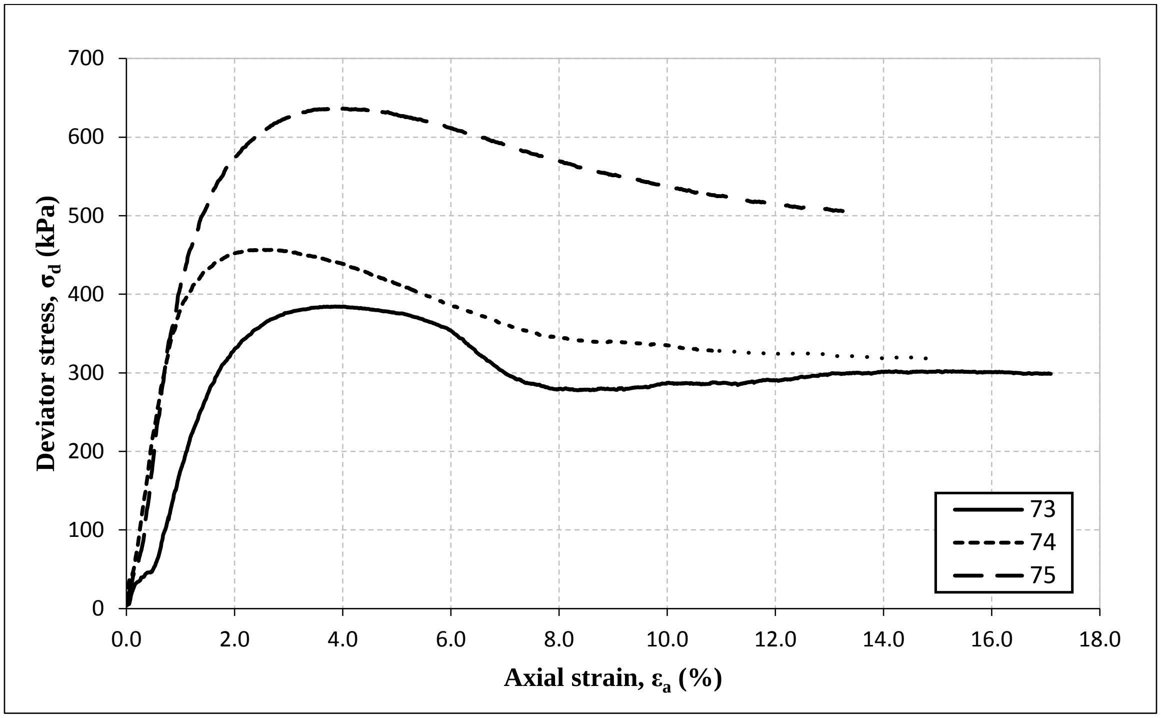


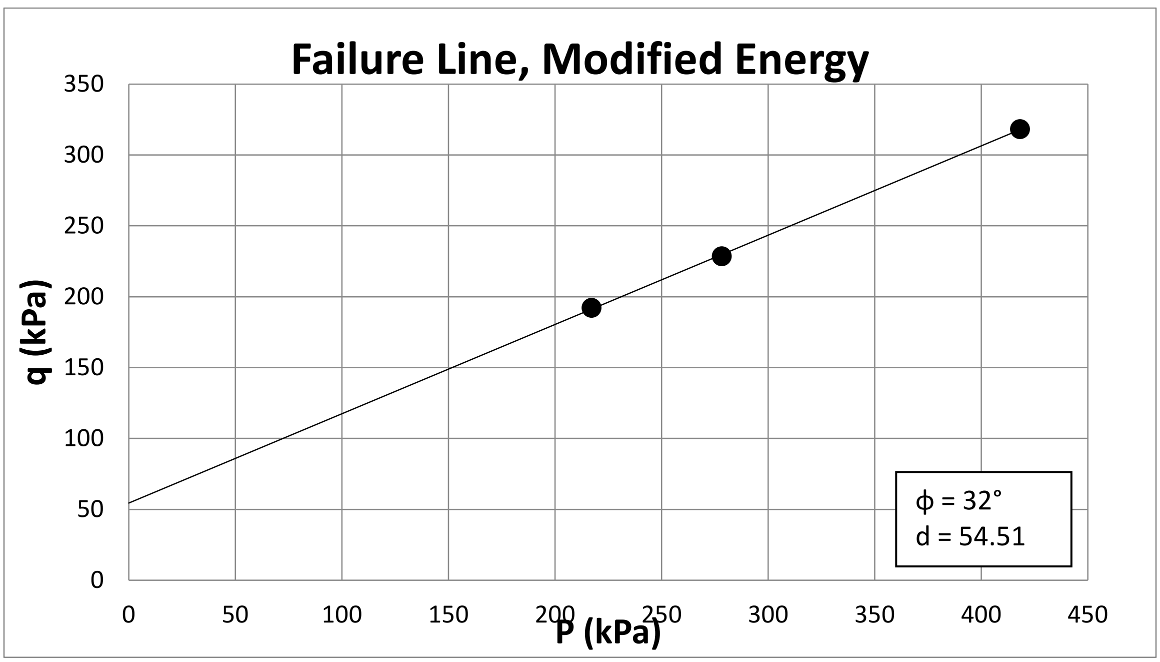
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/15

Sample No.	73	74	75
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.62	35.64	35.56
Average Height (mm)	71.98	72.13	71.69
Weight (g)	148.95	148.43	146.95
Water Content (%)	6.3	5.8	6.3
Dry Unit Weight (kN/m^3)	19.16	19.12	19.05
Saturation (%)	44	40	43
Strain at Failure (%)	4.16	2.77	4.17
Max Deviator Stress (kPa)	384.4	456.9	736.5

Test 73	Test 74	Test 75

Project Name:	Improvement of Material Criteria for Highway Embankment Construction		
Sample: Wayne, Modified Compaction			
Specimen Type: Compacted	USCS: SM	Gs: 2.72	
Strain Rate: 1%/min	AASHTO: A-2-4 (0)	Date: 8/13/15	



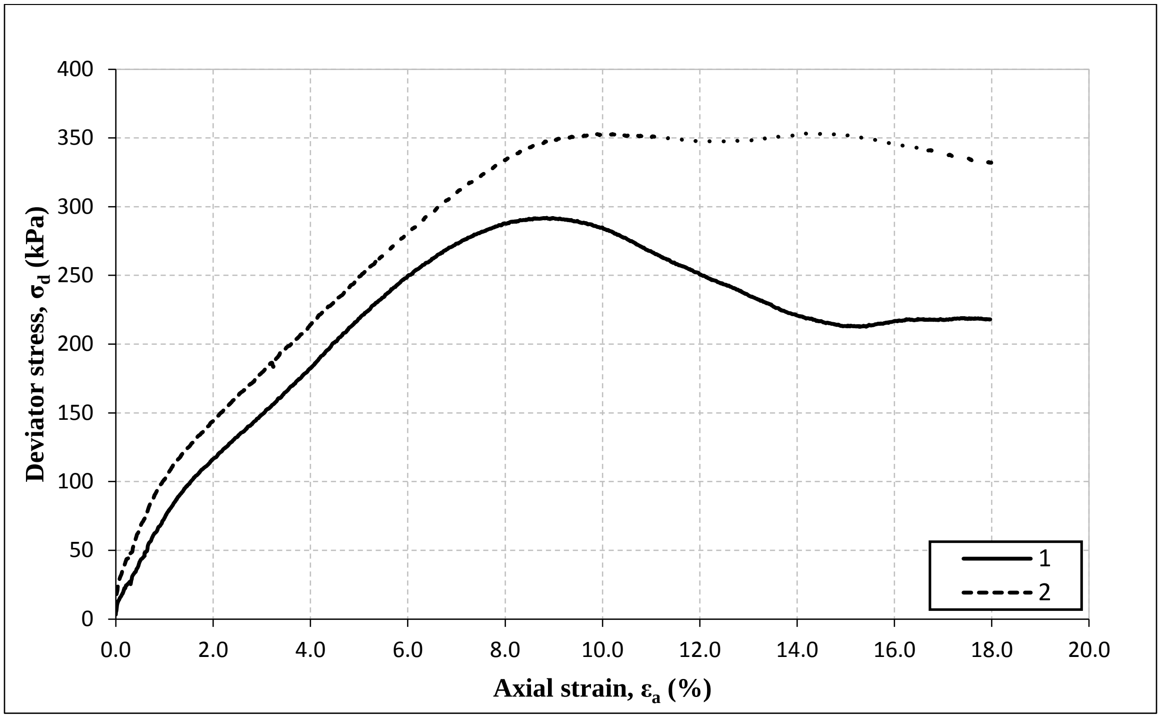


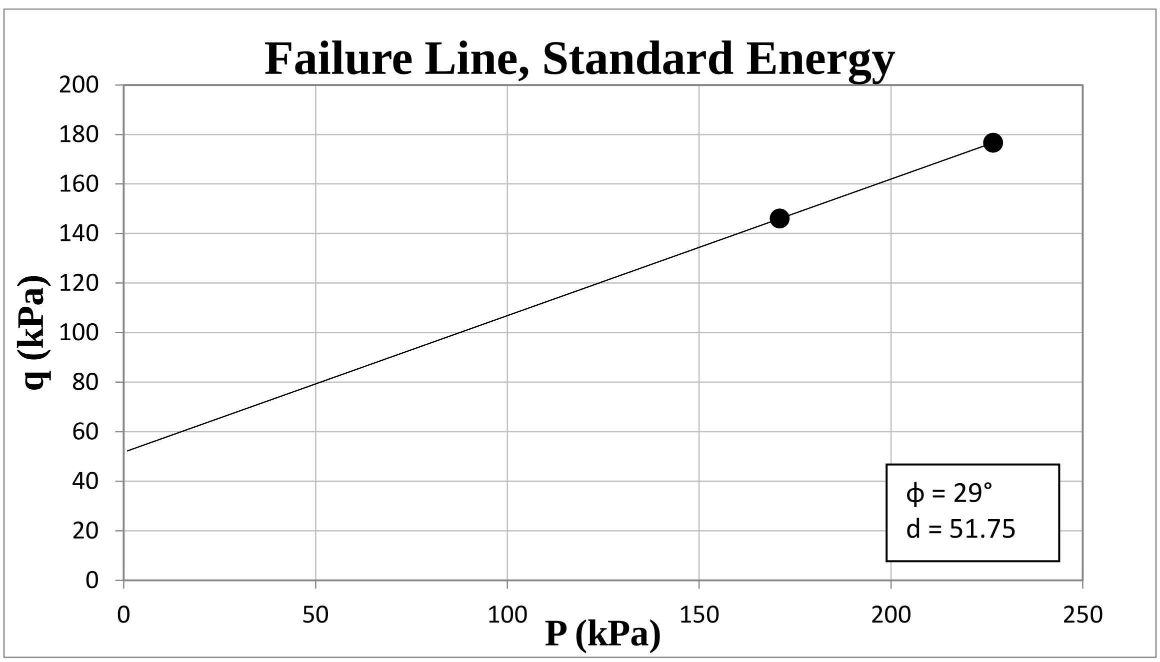
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/25/15

Sample No.	1	2	
Cell Pressure (kPa)	25	50	
Average Diameter (mm)	35.78	35.77	
Average Height (mm)	87.78	84.11	
Weight (g)	185.69	181.04	
Water Content (%)	11.4	12.1	
Dry Unit Weight (kN/m^3)	18.52	18.75	
Saturation (%)	70	78	
Strain at Failure (%)	9.14	14.48	
Max Deviator Stress (kPa)	291.9	353.4	

Test 1	Test 2	
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction		
Sample: Kinston, Standard Compaction			
Specimen Type: Compacted	USCS: SM	Gs: 2.72	
Strain Rate: 1%/min	AASHTO: A-2-4 (0)	Date: 6/25/15	



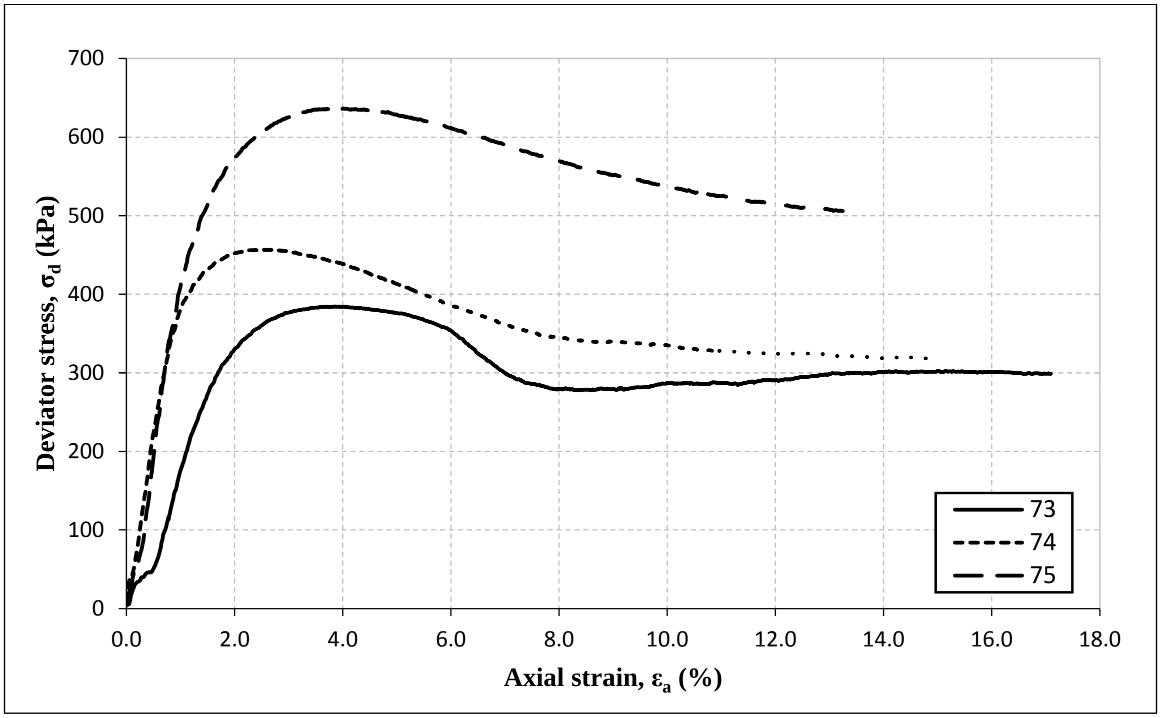


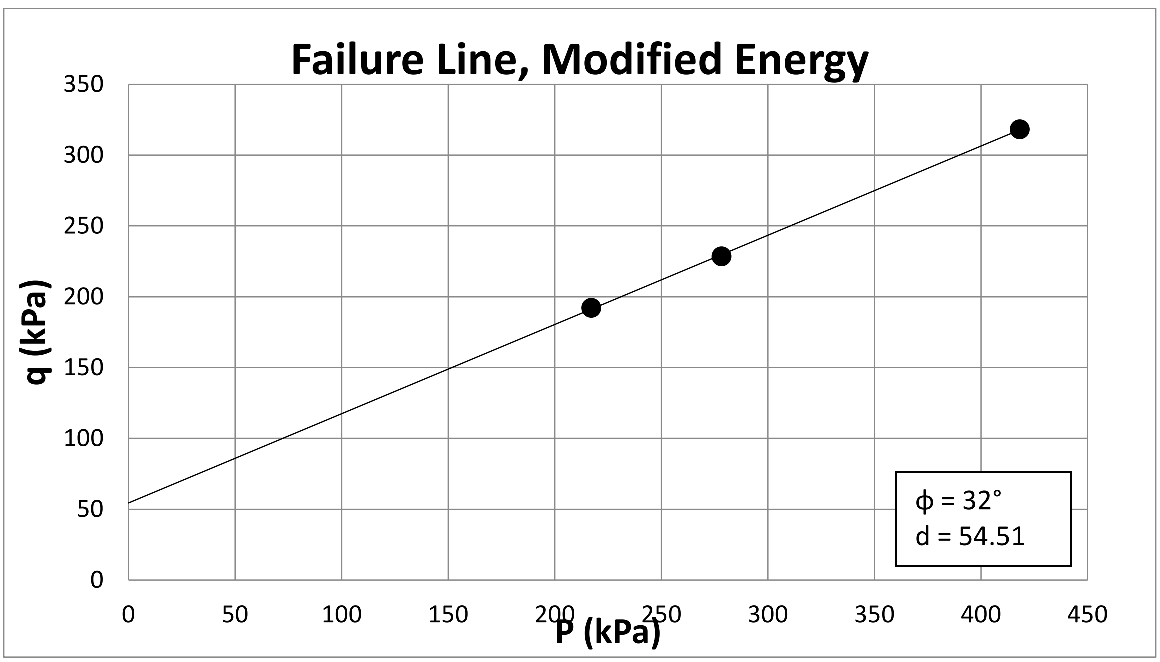
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/15

Sample No.	73	74	75
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.62	35.64	35.56
Average Height (mm)	71.98	72.13	71.69
Weight (g)	148.95	148.43	146.95
Water Content (%)	6.3	5.8	6.3
Dry Unit Weight (kN/m^3)	19.16	19.12	19.05
Saturation (%)	44	40	43
Strain at Failure (%)	4.16	2.77	4.17
Max Deviator Stress (kPa)	384.4	456.9	736.5

Test 73	Test 74	Test 75

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Wayne, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-2-4 (0)	Date:	8/13/15



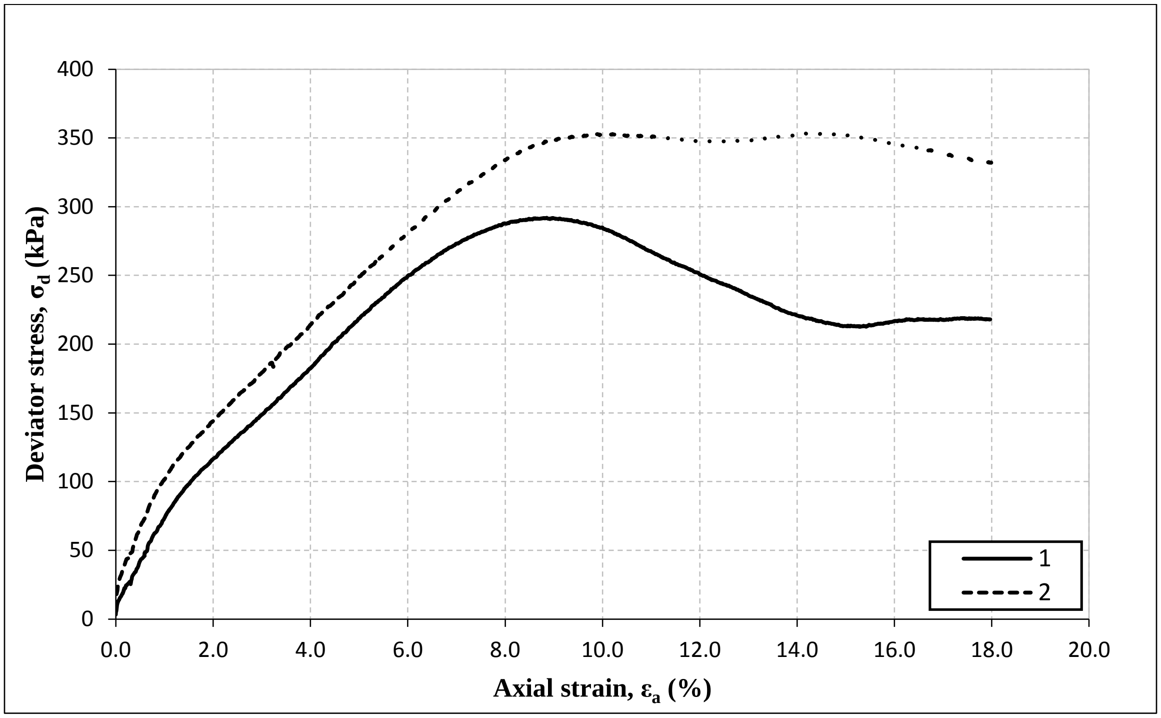


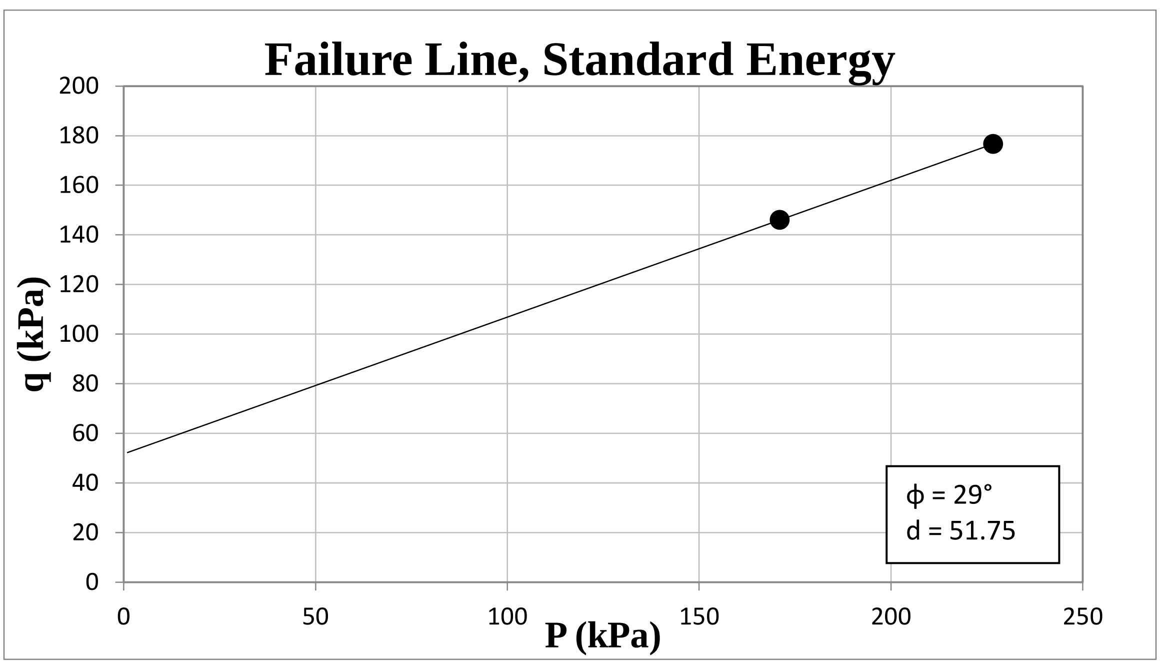
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/25/15

Sample No.	1	2	
Cell Pressure (kPa)	25	50	
Average Diameter (mm)	35.78	35.77	
Average Height (mm)	87.78	84.11	
Weight (g)	185.69	181.04	
Water Content (%)	11.4	12.1	
Dry Unit Weight (kN/m^3)	18.52	18.75	
Saturation (%)	70	78	
Strain at Failure (%)	9.14	14.48	
Max Deviator Stress (kPa)	291.9	353.4	

Test 1	Test 2	
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS: SM		Gs:	2.72
Strain Rate:	1%/min	AASHTO: A-2-4 (0)		Date:	6/25/15



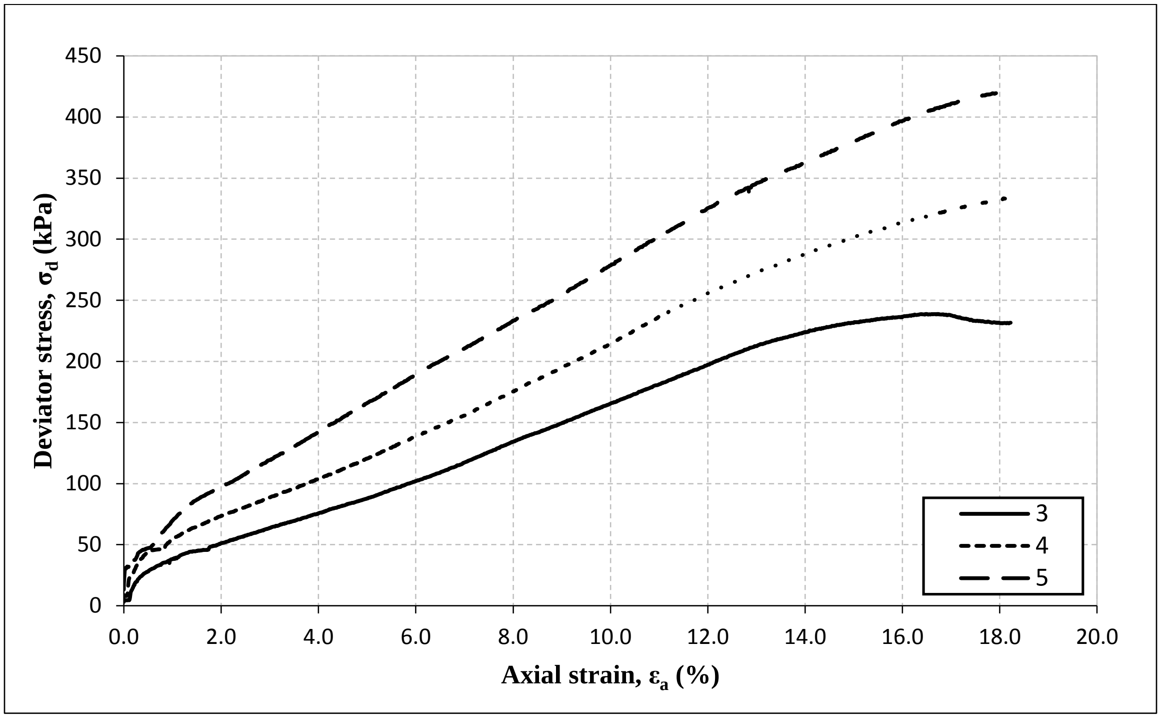


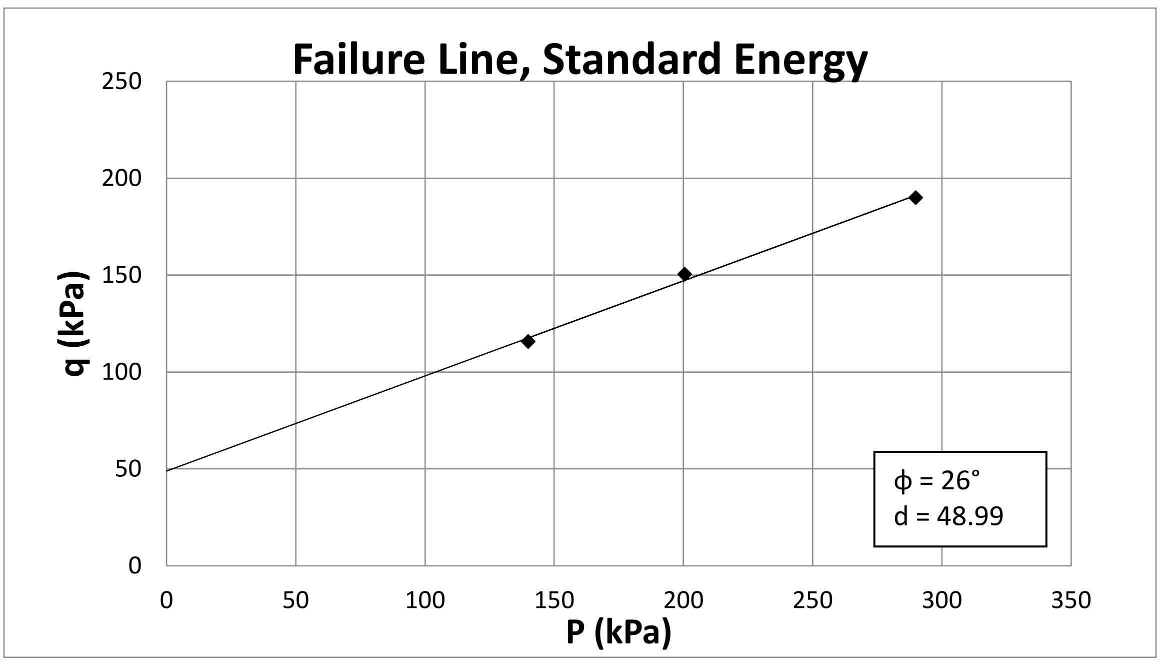
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/30/15

Sample No.	3	4	5
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.76	35.62	35.73
Average Height (mm)	95.46	88.85	92.61
Weight (g)	204.50	191.93	198.42
Water Content (%)	12.4	12.2	12.1
Dry Unit Weight (kN/m^3)	18.62	18.94	18.70
Saturation (%)	78	81	77
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	238.8	341.2	426.8

Test 3	Test 4	Test 5
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/30/15



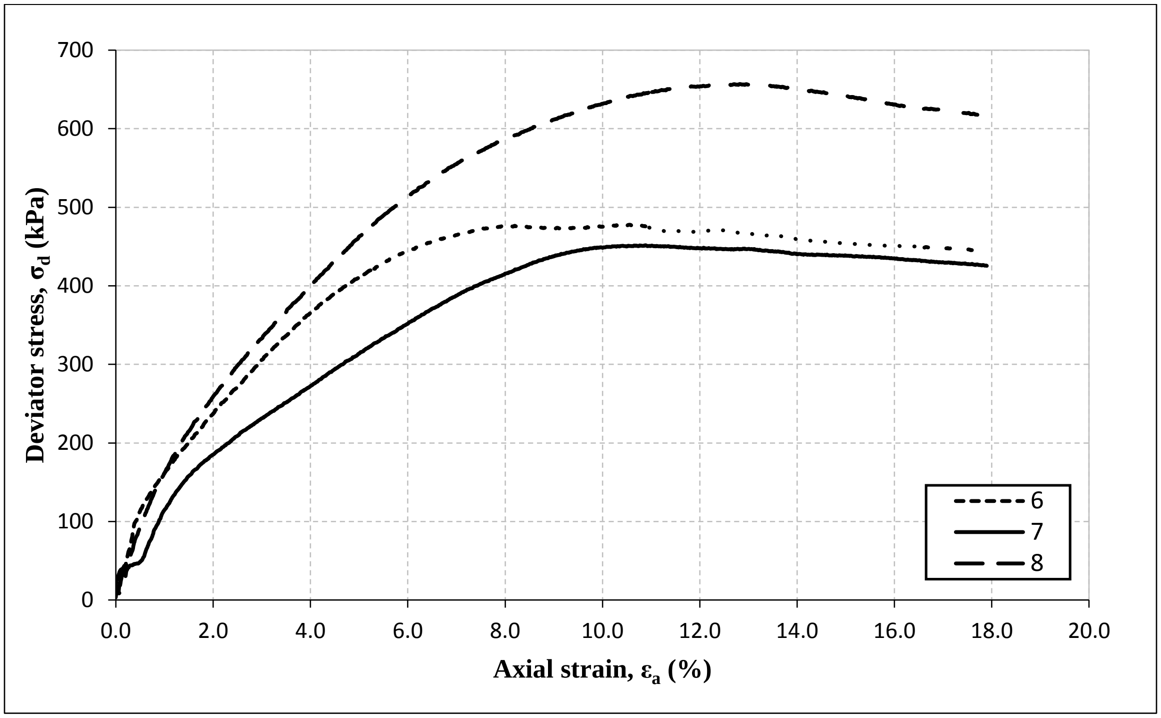


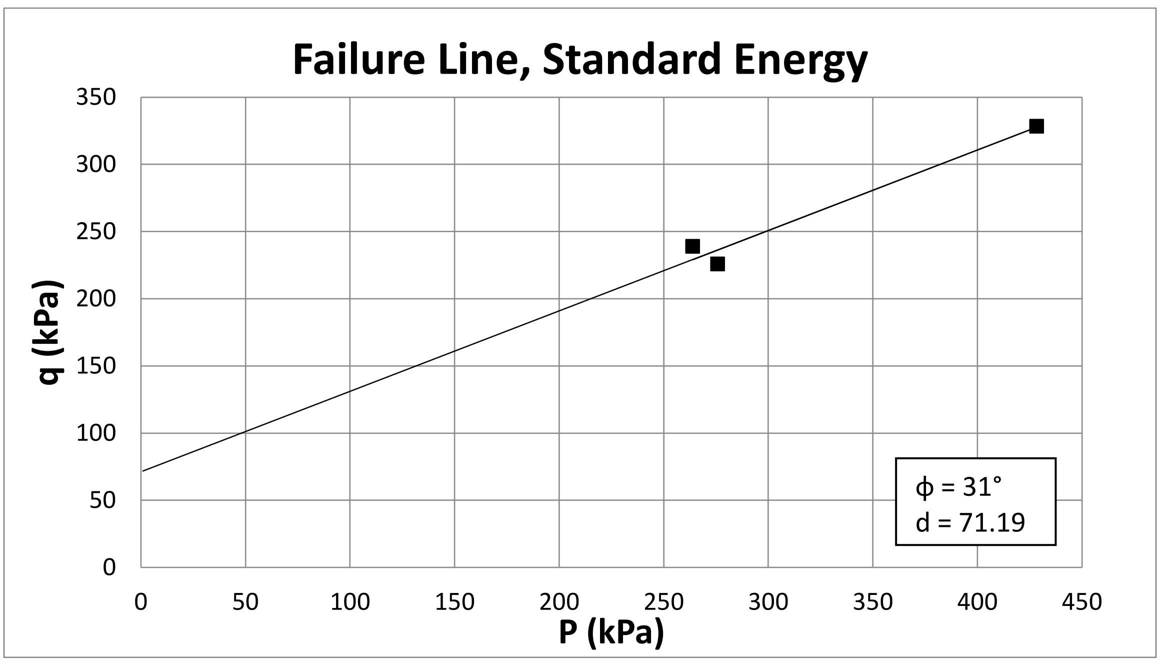
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/30/15

Sample No.	6	7	8
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.75	35.90	35.84
Average Height (mm)	86.91	92.44	87.55
Weight (g)	185.69	192.08	182.02
Water Content (%)	10.2	10.3	10.2
Dry Unit Weight (kN/m^3)	18.95	18.26	18.34
Saturation (%)	68	61	61
Strain at Failure (%)	10.82	11.29	13.02
Max Deviator Stress (kPa)	477.7	451.6	656.9

Test 6	Test 7	Test 8
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/30/15



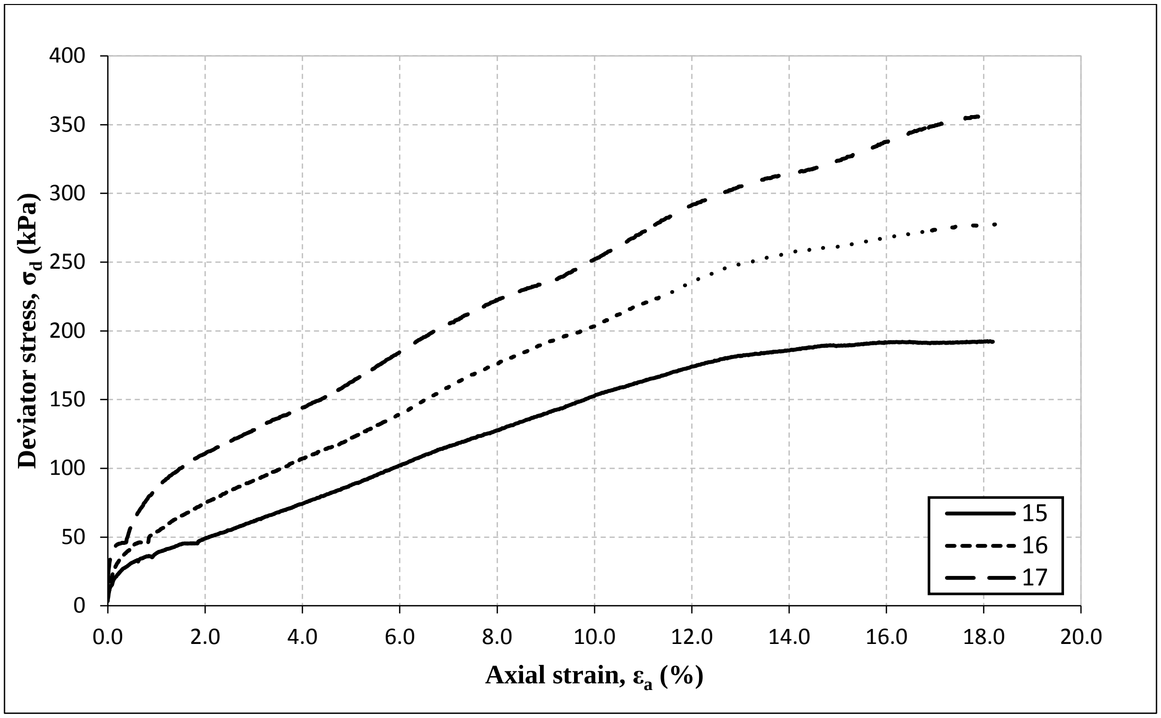


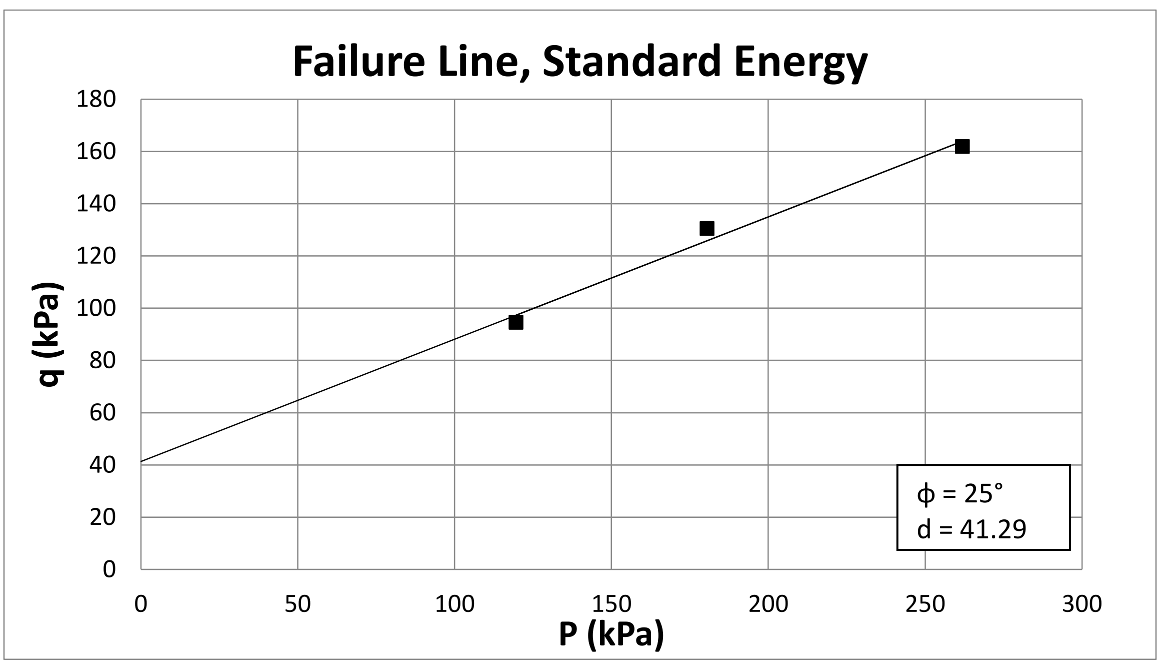
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/28/15

Sample No.	15	16	17
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.80	35.74	35.82
Average Height (mm)	89.67	102.10	102.00
Weight (g)	191.00	220.11	217.66
Water Content (%)	11.7	11.6	11.5
Dry Unit Weight (kN/m^3)	18.57	18.88	18.63
Saturation (%)	73	76	72
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	192.5	278.0	363.1

Test 15	Test 16	Test 17

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/28/15



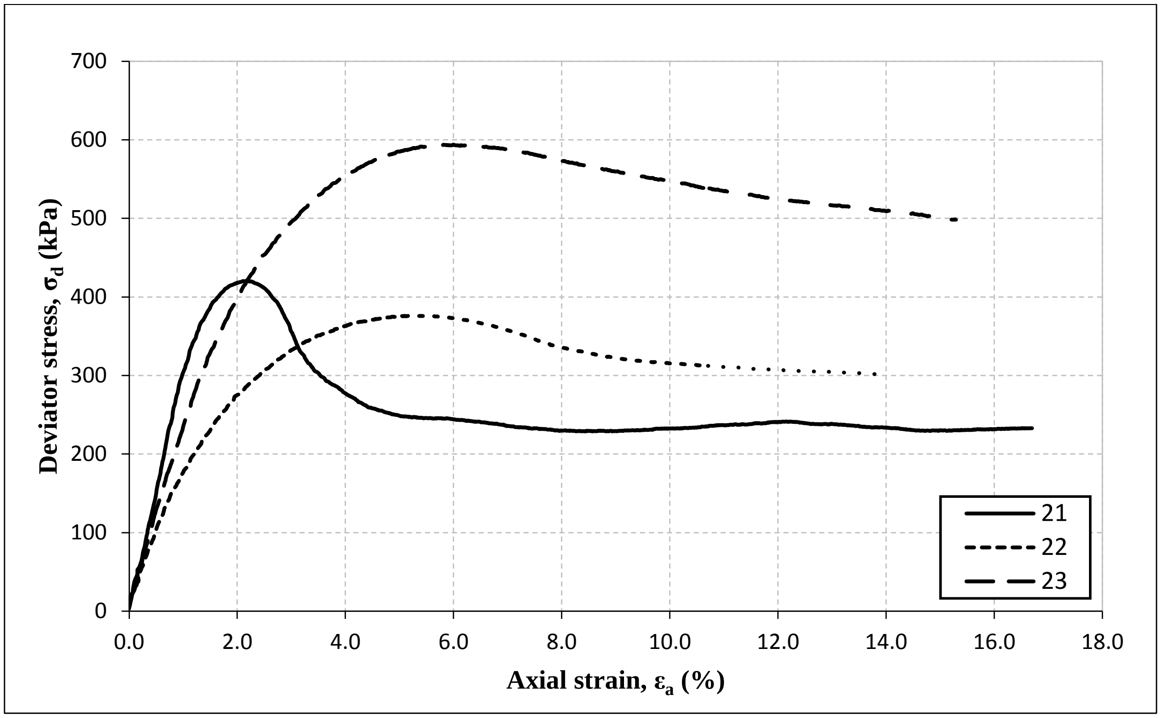


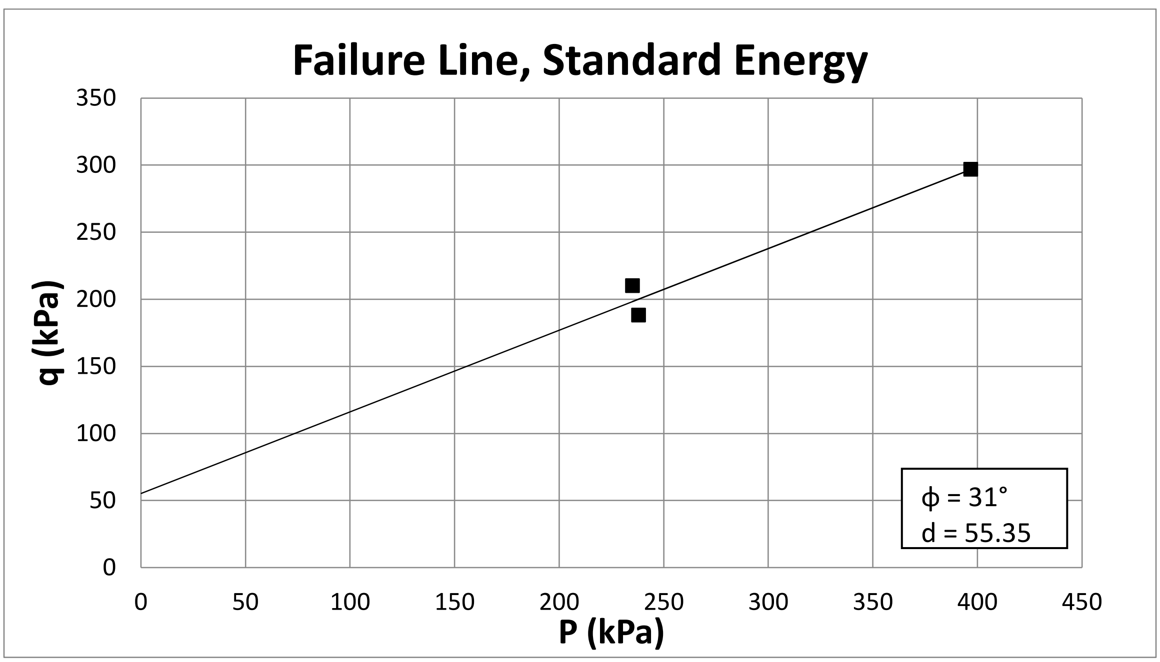
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/29/15

Sample No.	21	22	23
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.67	35.70	35.78
Average Height (mm)	71.29	71.23	71.42
Weight (g)	132.18	132.14	134.50
Water Content (%)	6.7	7.7	7.4
Dry Unit Weight (kN/m^3)	17.06	16.88	17.11
Saturation (%)	32	36	36
Strain at Failure (%)	2.58	5.81	6.36
Max Deviator Stress (kPa)	420.2	367.2	593.7

Test 21	Test 22	Test 23

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/29/15



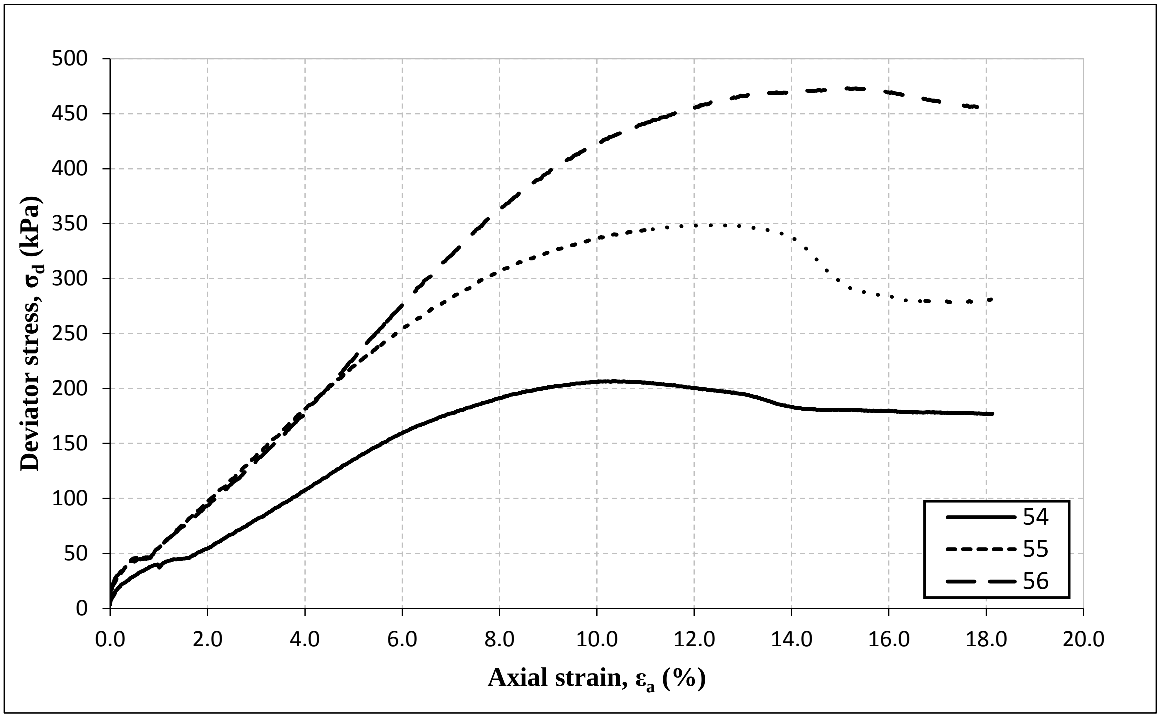


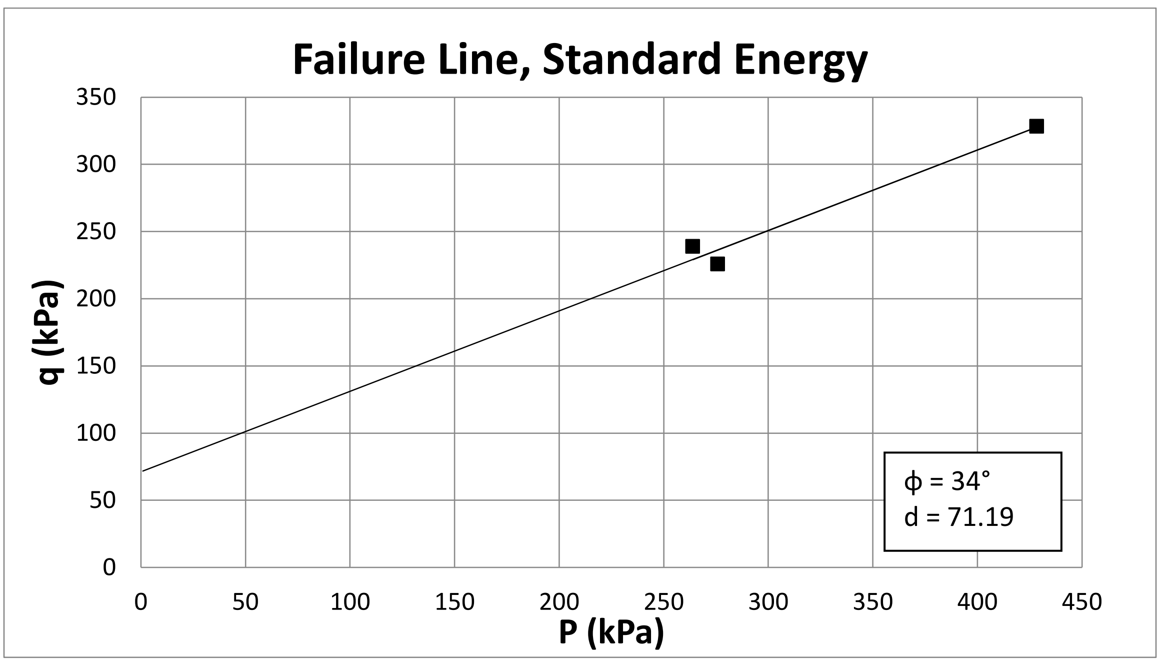
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/15/15

Sample No.	54	55	56
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.65	35.54	35.64
Average Height (mm)	71.23	71.51	71.14
Weight (g)	155.18	154.91	155.40
Water Content (%)	11.5	11.2	11.0
Dry Unit Weight (kN/m^3)	19.20	19.25	19.34
Saturation (%)	80	79	79
Strain at Failure (%)	10.36	12.49	15.00
Max Deviator Stress (kPa)	206.6	348.8	473.2

Test 54	Test 55	Test 56

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Standard Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/15/15



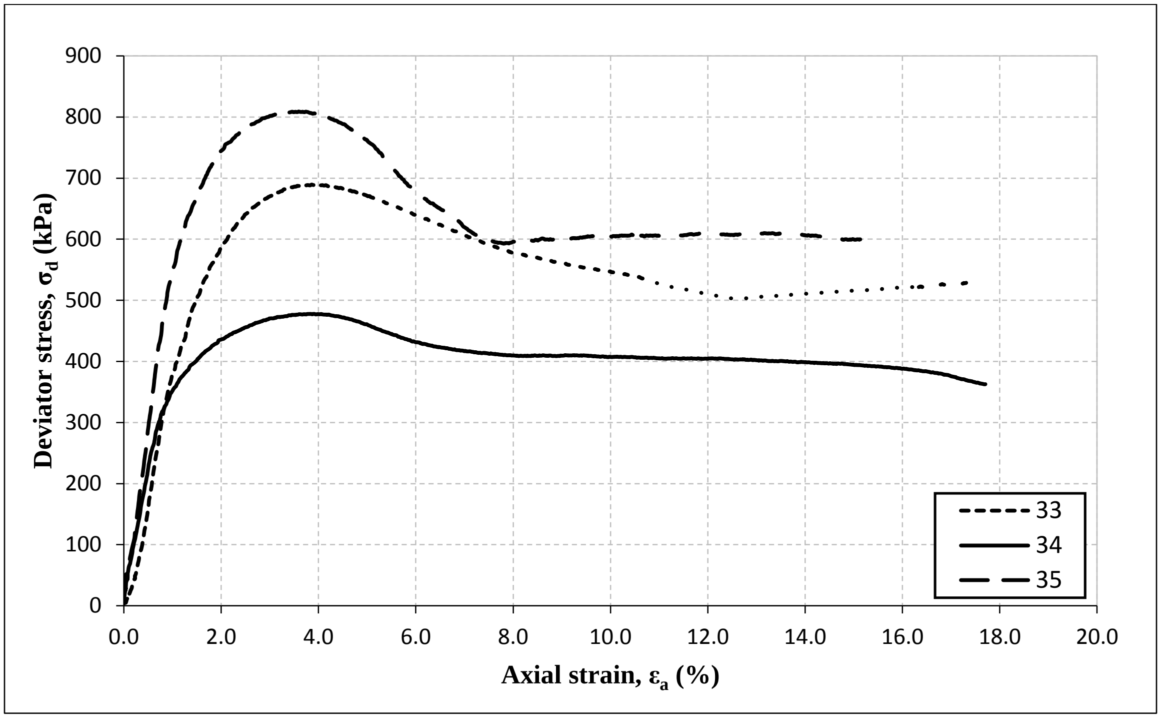


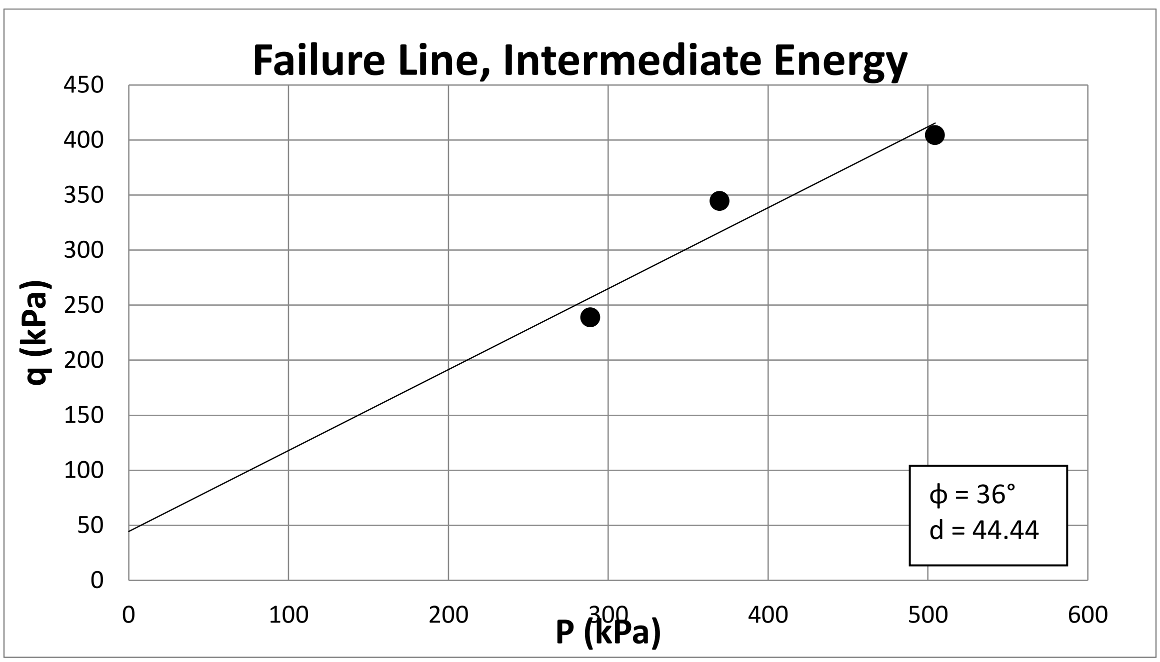
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/18/15

Sample No.	33	34	35
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.70	35.75	35.89
Average Height (mm)	71.64	71.39	71.41
Weight (g)	144.89	145.25	145.93
Water Content (%)	6.9	7.5	7.2
Dry Unit Weight (kN/m^3)	18.54	18.48	18.48
Saturation (%)	43	46	44
Strain at Failure (%)	4.43	4.30	4.17
Max Deviator Stress (kPa)	689.5	477.80	809.0

Test 33	Test 34	Test 35

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/18/15



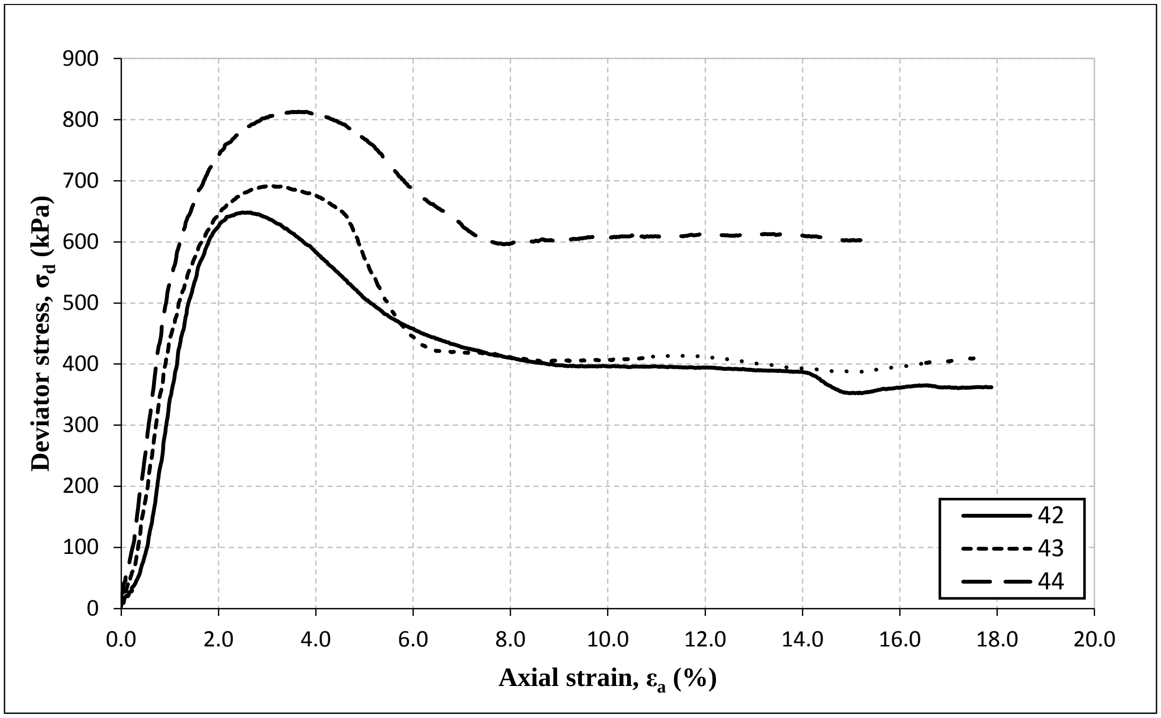


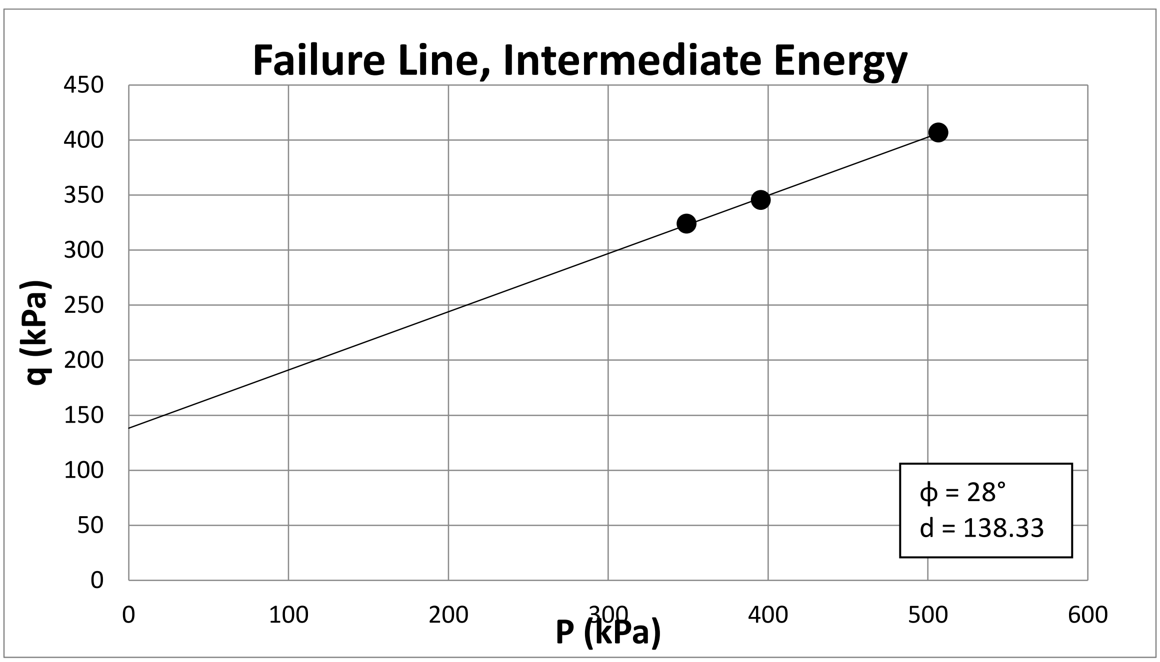
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/21/15

Sample No.	42	43	44
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.67	35.72	35.79
Average Height (mm)	71.57	71.37	71.34
Weight (g)	148.07	149.68	149.76
Water Content (%)	7.8	8.2	8.1
Dry Unit Weight (kN/m^3)	18.84	18.98	18.93
Saturation (%)	51	55	54
Strain at Failure (%)	2.76	3.45	4.18
Max Deviator Stress (kPa)	648.2	691.3	813.2

Test 42	Test 43	Test 44

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/21/15



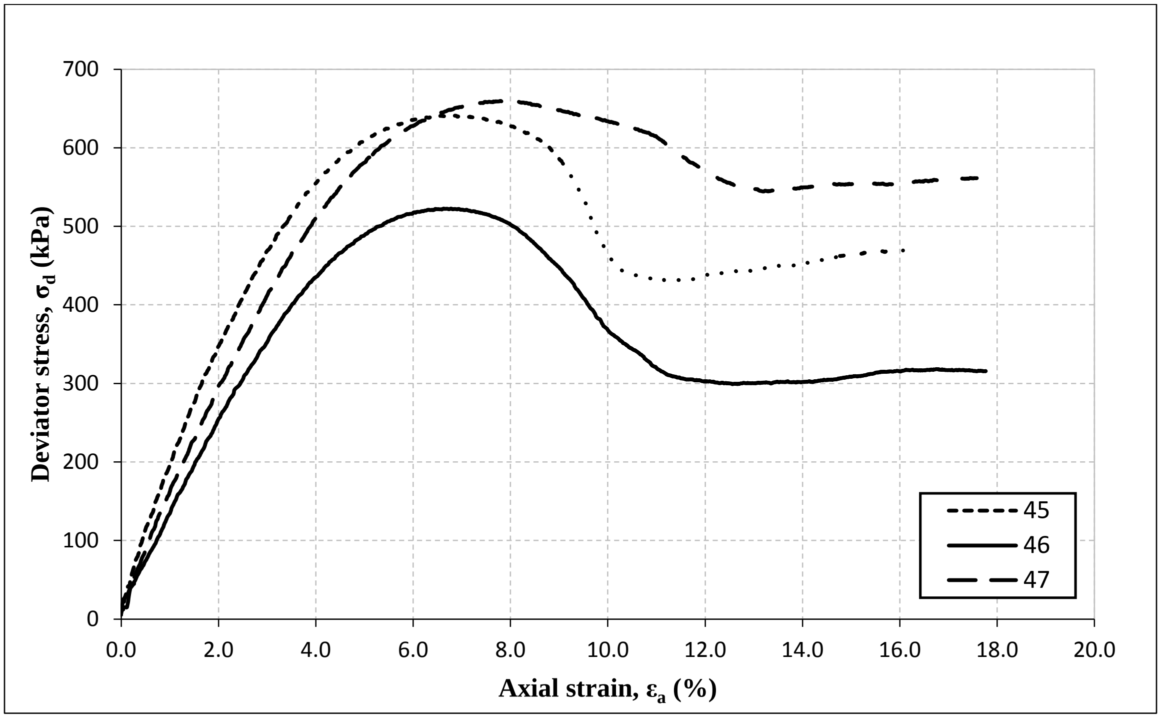


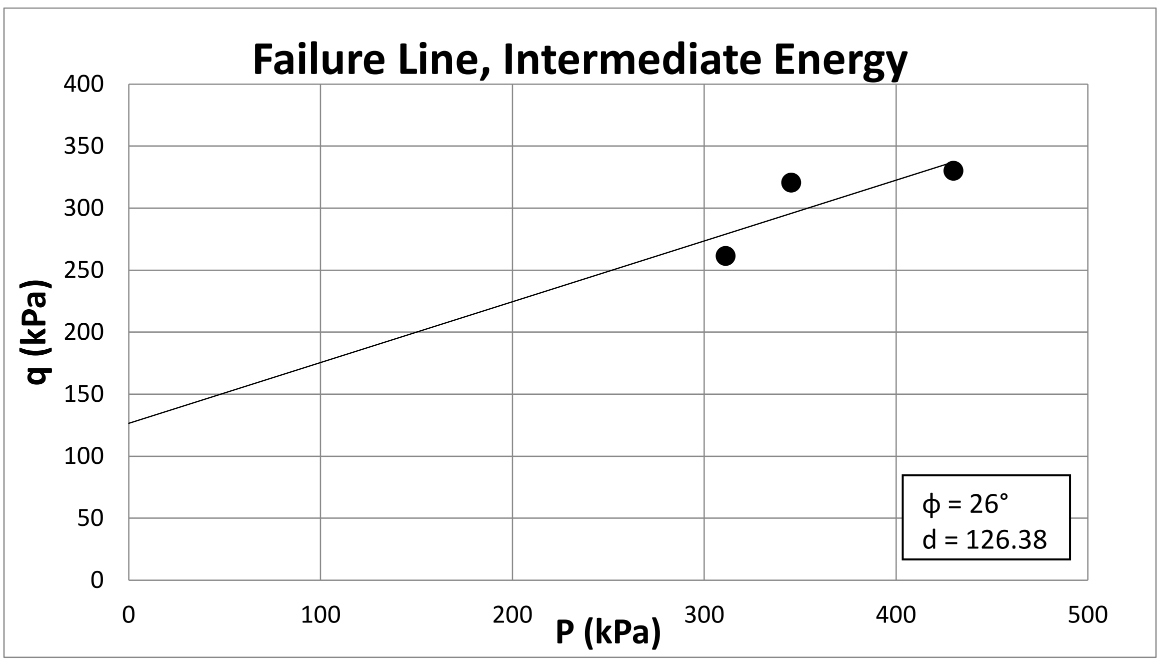
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample:					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/21/15

Sample No.	45	46	47
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.64	35.66	35.58
Average Height (mm)	71.60	71.65	71.92
Weight (g)	158.10	158.33	158.10
Water Content (%)	9.6	10.0	9.8
Dry Unit Weight (kN/m^3)	19.80	19.73	19.74
Saturation (%)	75	77	76
Strain at Failure (%)	9.21	7.14	8.37
Max Deviator Stress (kPa)	640.9	522.6	660.0

Test 45	Test 46	Test 47

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample:					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/21/15



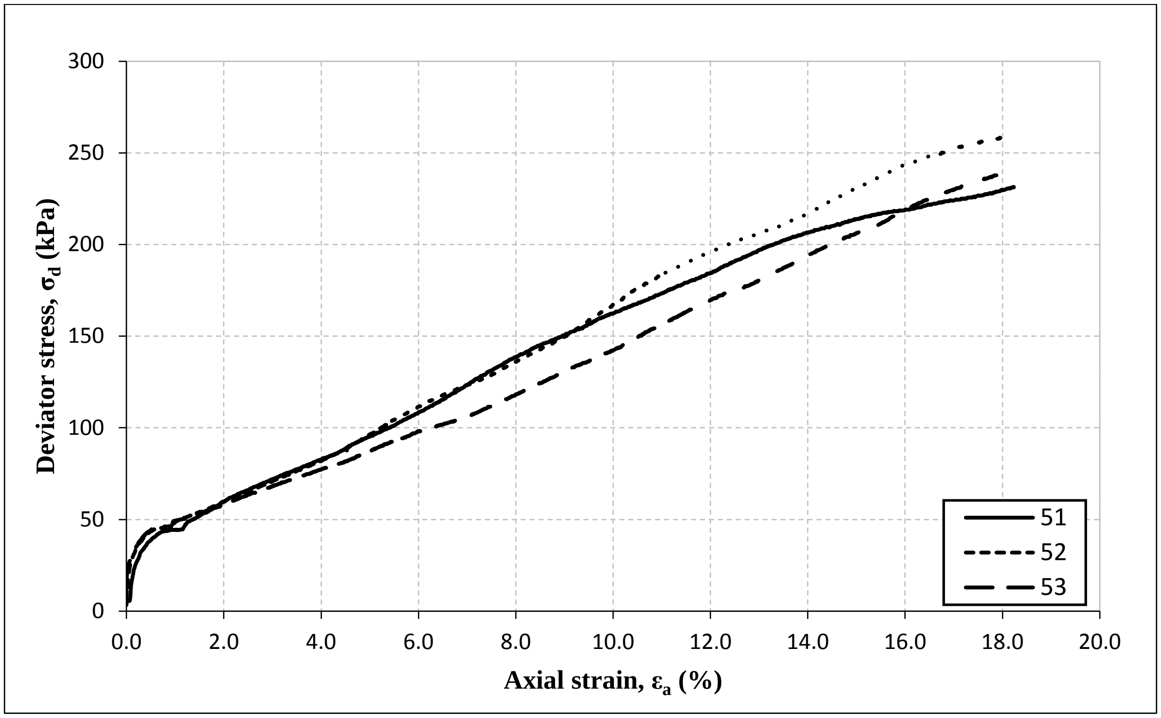


Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/3/2015

Sample No.	51	52	53
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.55	35.66	35.77
Average Height (mm)	71.27	71.42	71.29
Weight (g)	151.08	151.31	158.10
Water Content (%)	13.1	13.5	13.1
Dry Unit Weight (kN/m^3)	18.53	18.33	19.35
Saturation (%)	81	81	86
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	237.1	266.9	256.9

Test 51	Test 52	Test 53

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	6/3/2015

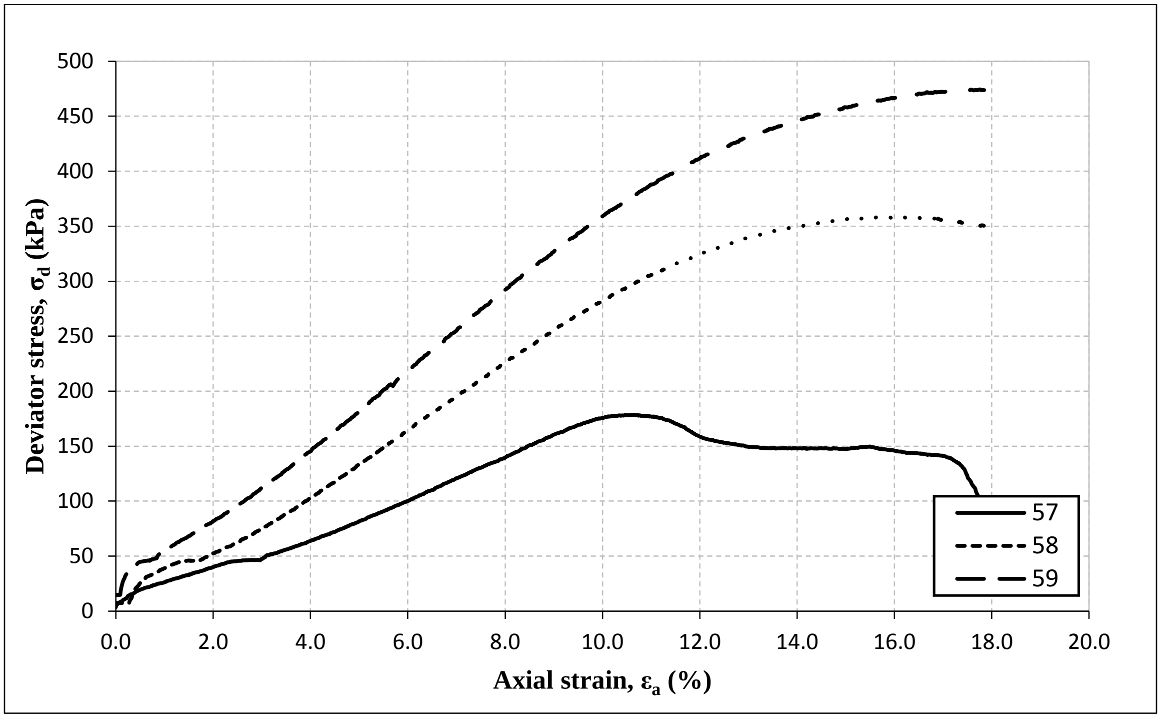


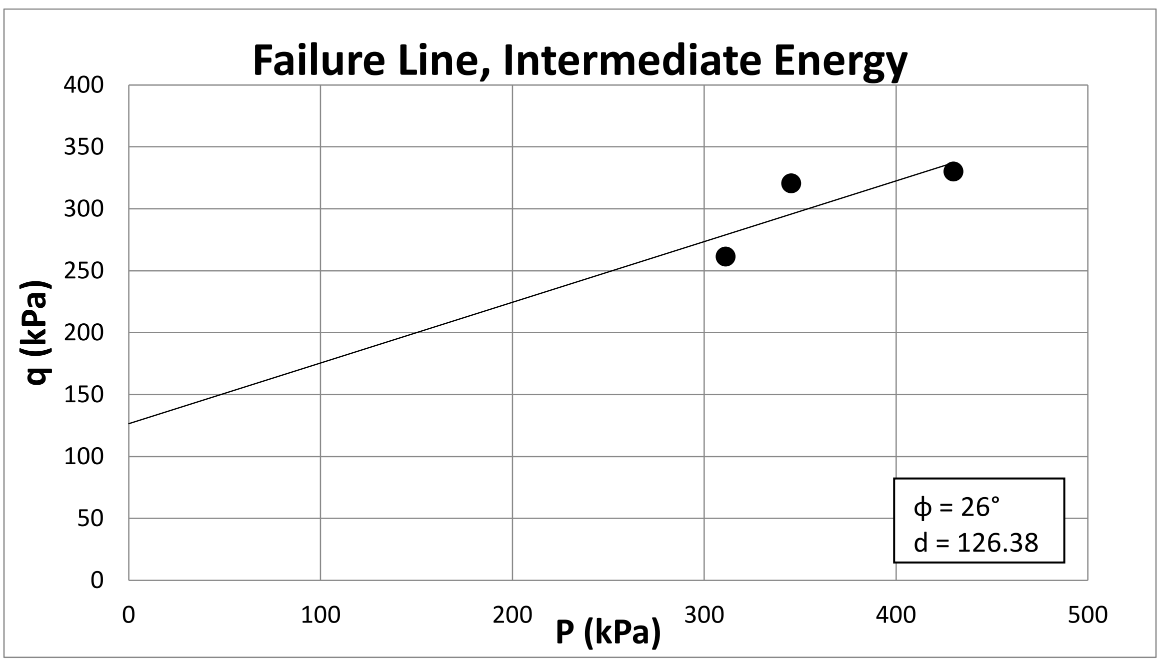
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/15/15

Sample No.	57	58	59
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.5	35.57	35.59
Average Height (mm)	71.23	71.1	71.07
Weight (g)	155.8	154.40	154.84
Water Content (%)	11.4	11.3	11.2
Dry Unit Weight (kN/m^3)	19.40	19.25	19.31
Saturation (%)	83	80	80
Strain at Failure (%)	10.61	15.00	15.00
Max Deviator Stress (kPa)	178.4	358.5	558.1

Test 57	Test 58	Test 59

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/15/15





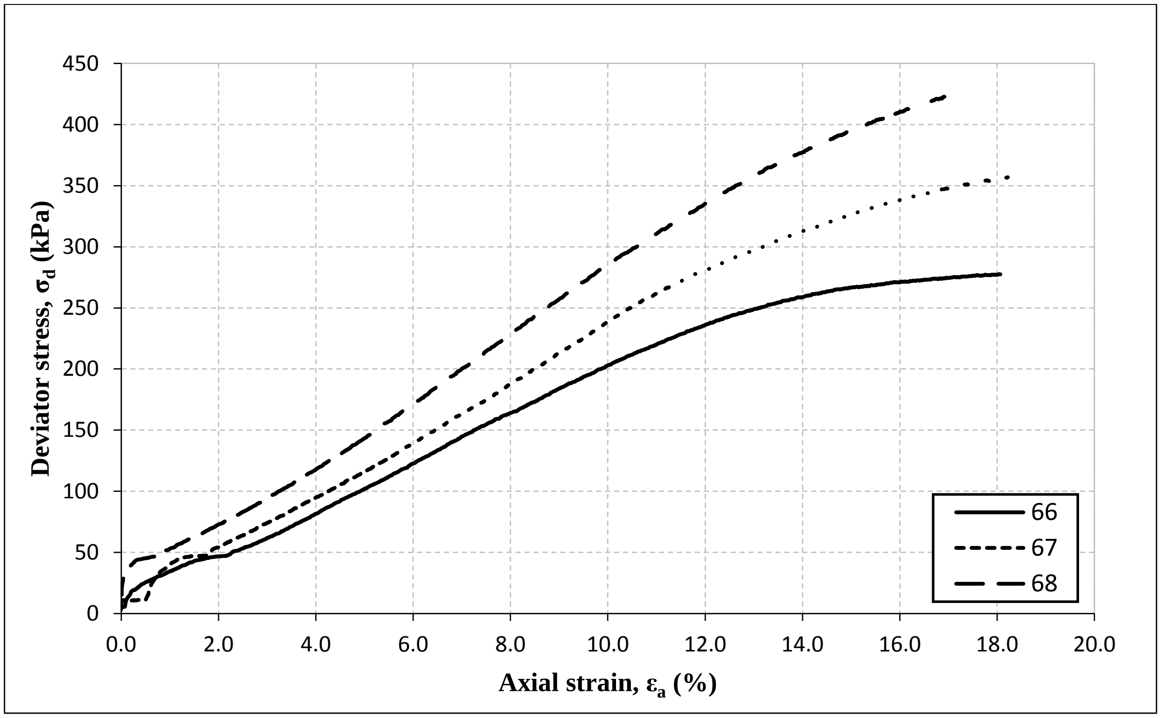
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/20/15

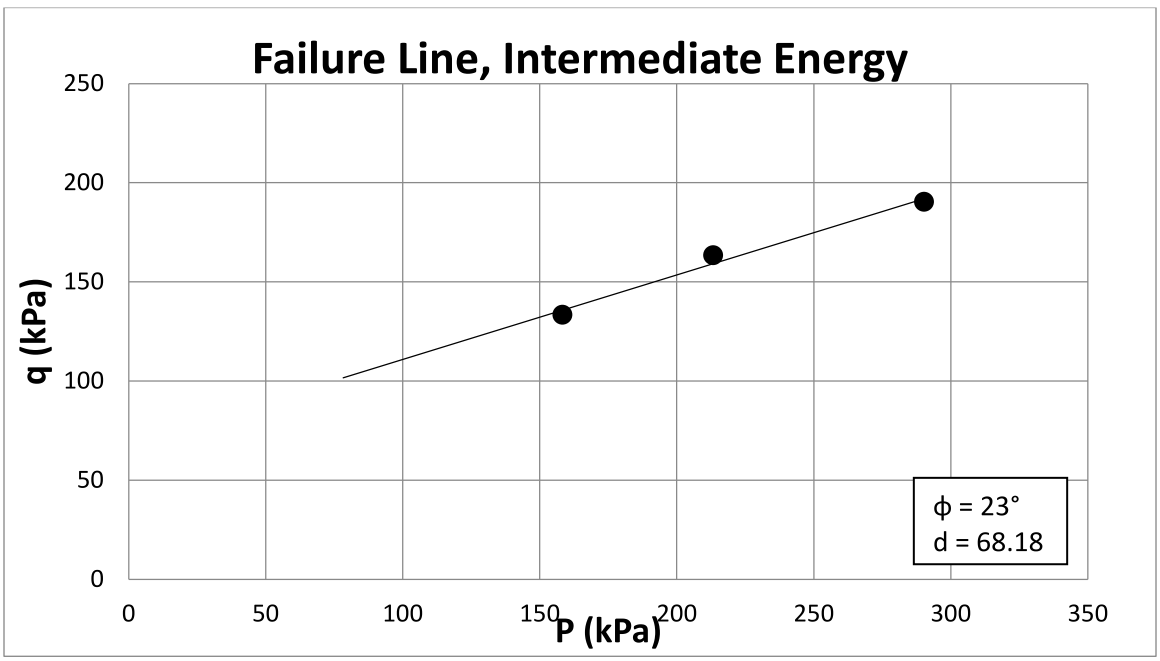
Sample No.	66	67	68
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.65	35.58	35.66
Average Height (mm)	71.08	70.82	71.07
Weight (g)	154.10	154.00	154.49
Water Content (%)	11.4	11.5	11.6
Dry Unit Weight (kN/m^3)	19.13	19.23	19.13
Saturation (%)	79	81	80
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	279.0	363.8	441.5

Test 66	Test 67	Test 68

Project Name:	Improvement of Material Criteria for Highway Embankment Construction

Sample: Kinston, Intermediate Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/20/15





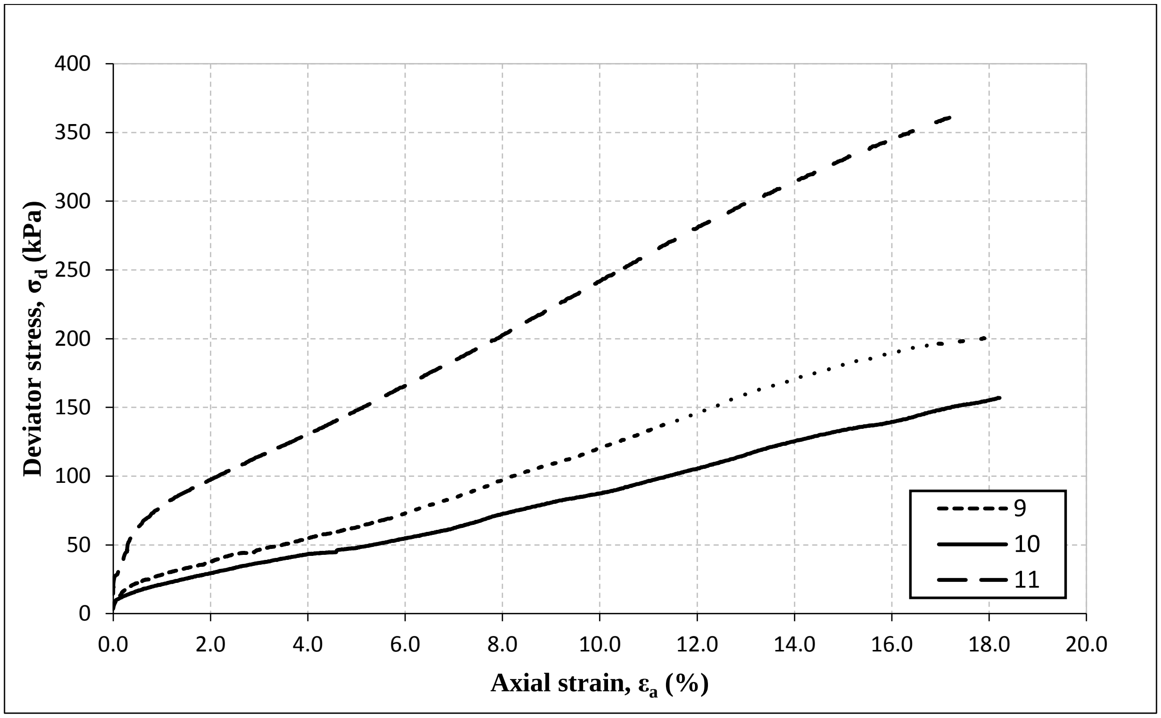
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/1/15

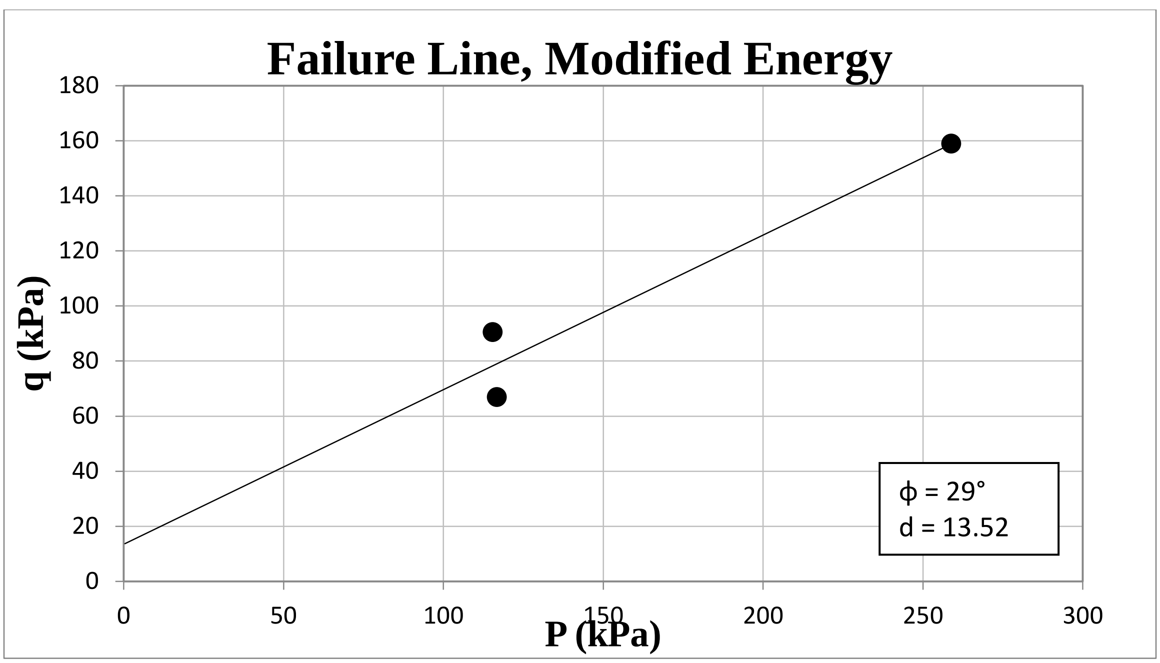
Sample No.	9	10	11
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.73	35.62	35.61
Average Height (mm)	103.64	94.94	88.23
Weight (g)	223.85	204.63	188.71.
Water Content (%)	12.2	12.2	12.1
Dry Unit Weight (kN/m^3)	18.82	18.91	18.78
Saturation (%)	80	81	78
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	208.4	166.8	382.1

Test 9	Test 10	Test 11
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction

Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/1/15



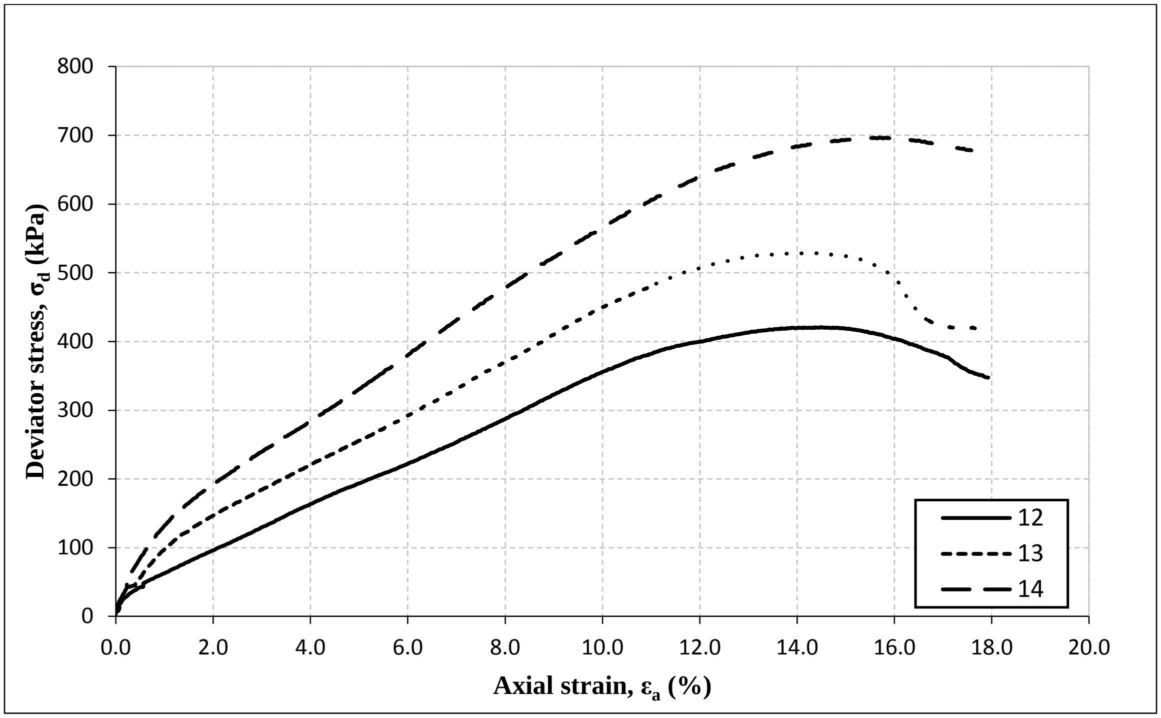


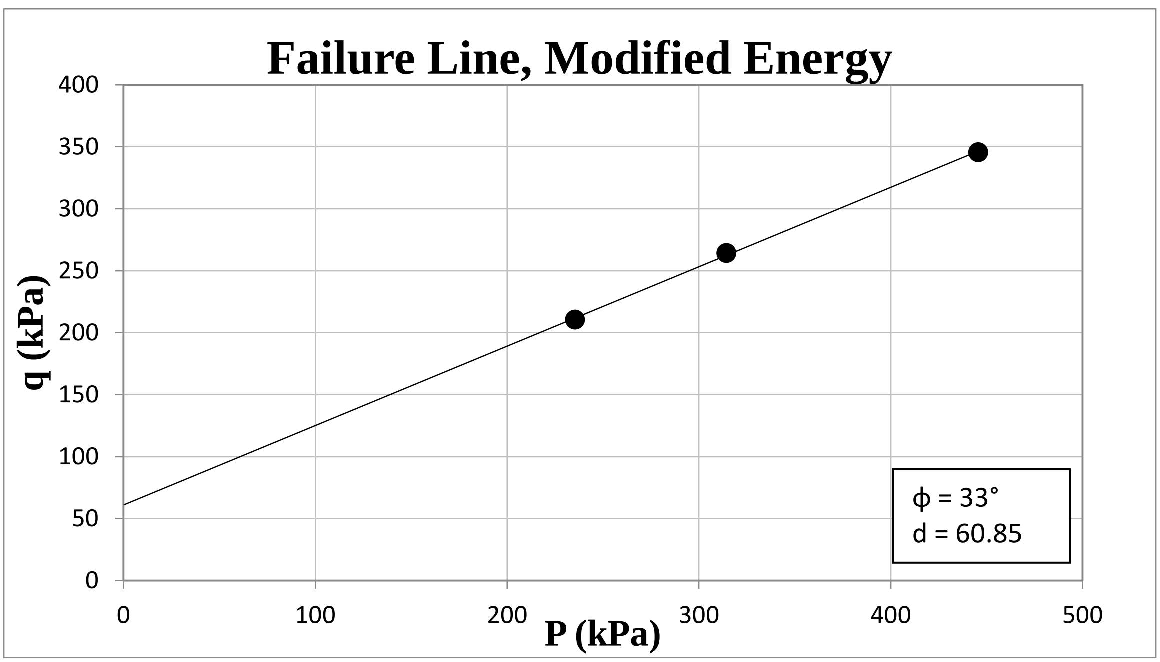
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/7/15

Sample No.	12	13	14
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.78	35.79	35.81
Average Height (mm)	92.08	96.70	94.72
Weight (g)	204.32	212.66	203.38
Water Content (%)	10.2	9.8	9.9
Dry Unit Weight (kN/m^3)	19.64	19.52	19.48
Saturation (%)	77	73	73
Strain at Failure (%)	14.78	14.25	15.00
Max Deviator Stress (kPa)	420.9	528.6	696.1

Test 12	Test 13	Test 14
		

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/7/15



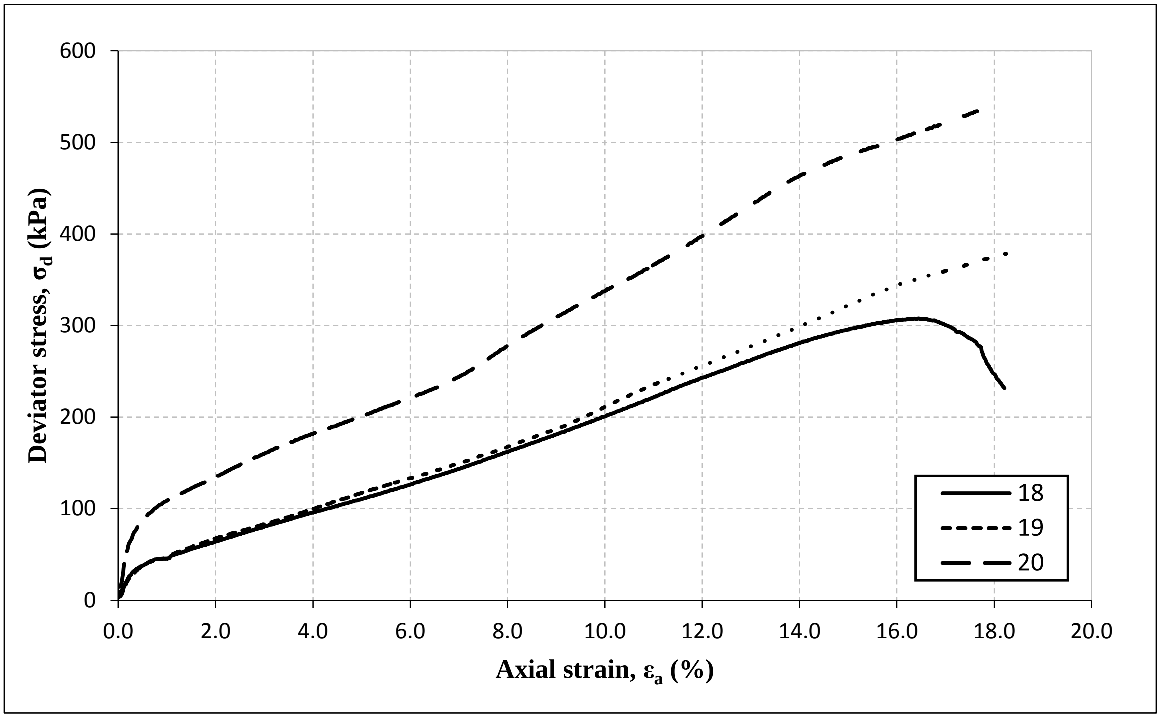


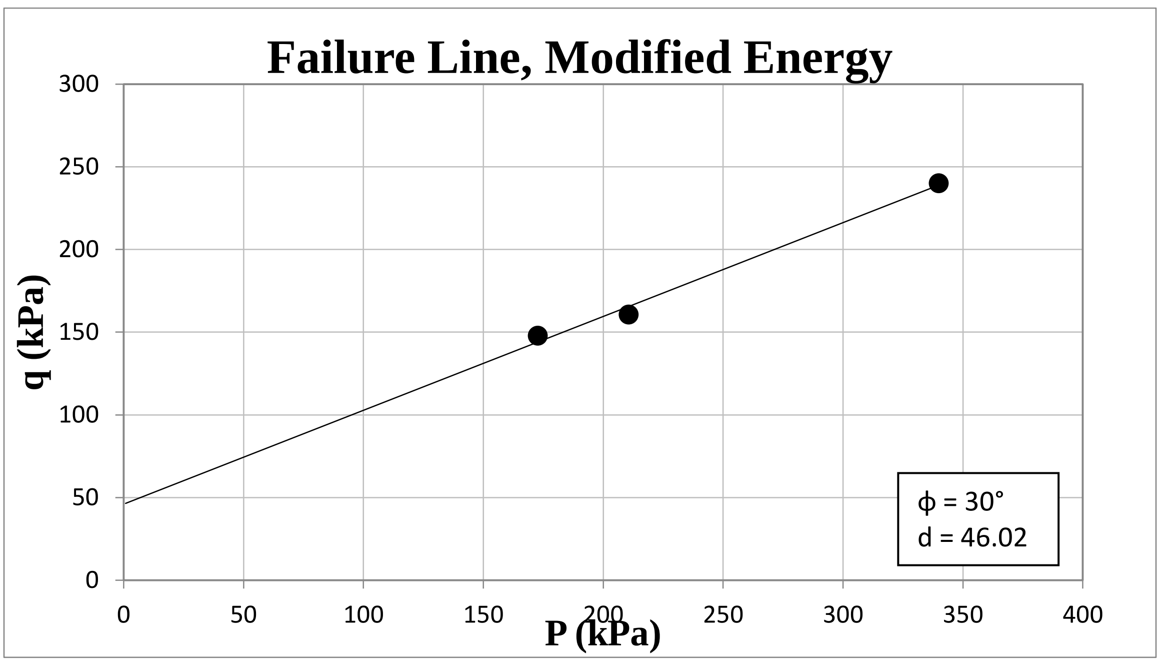
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/29/16

Sample No.	18	19	20
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.69	35.79	35.72
Average Height (mm)	101.80	97.76	103.71
Weight (g)	223.33	214.25	227.30
Water Content (%)	10.9	10.9	10.8
Dry Unit Weight (kN/m^3)	19.39	19.27	19.36
Saturation (%)	79	77	78
Strain at Failure (%)	15.00	15.00	15.00
Max Deviator Stress (kPa)	307.8	382.4	554.5

Test 18	Test 19	Test 20

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	7/29/16



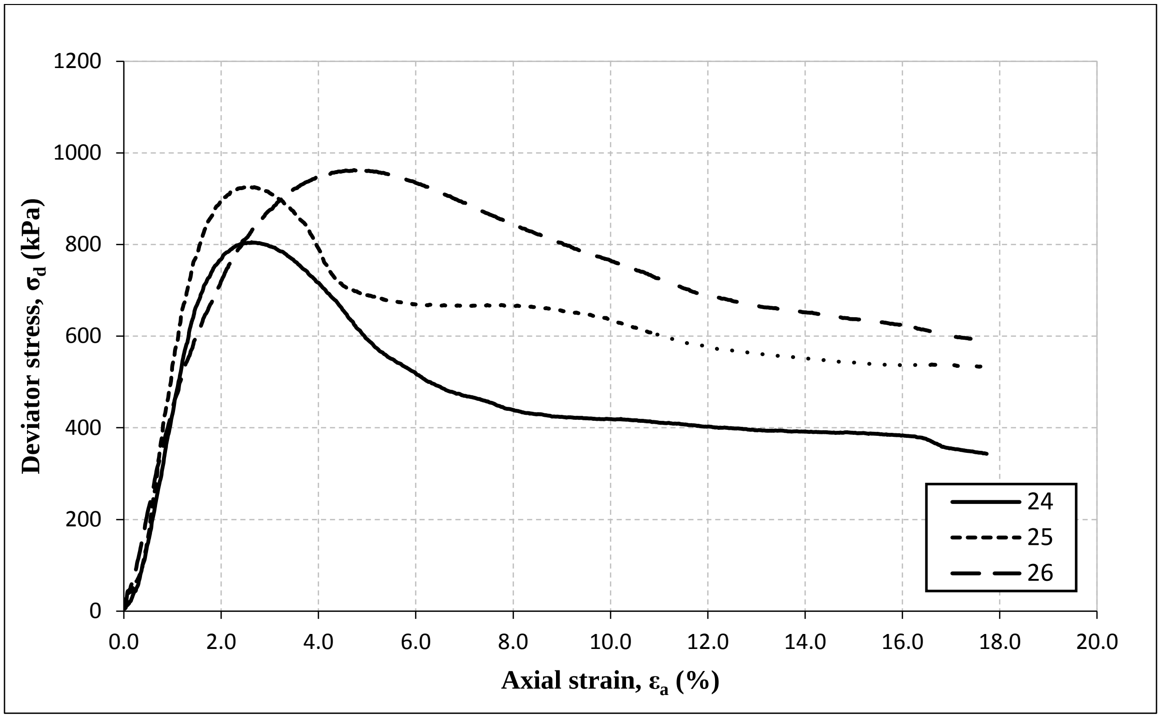


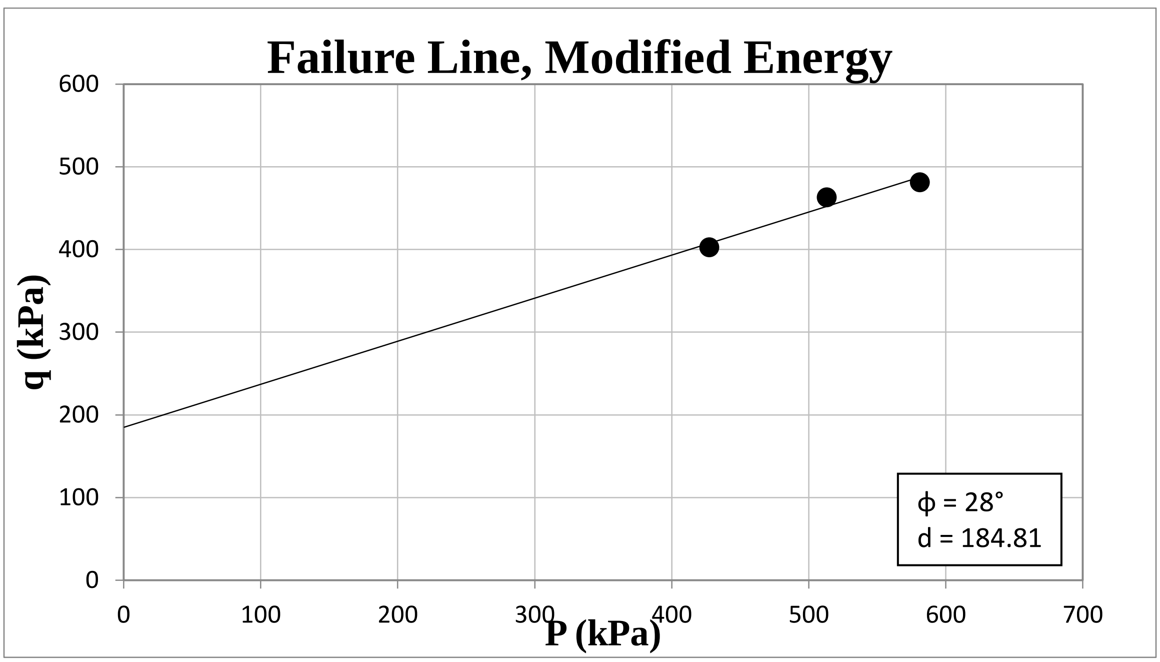
Project Name:	Improvement of Material Criteria for Highway Embankment Construction		
Sample: Kinston, Modified Compaction			
Specimen Type: Compacted	USCS: SM	Gs: 2.72	
Strain Rate: 1%/min	AASHTO: A-4 (0)	Date: 8/3/15	

Sample No.	24	25	26
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.89	35.83	35.97
Average Height (mm)	71.33	71.45	71.44
Weight (g)	145.66	144.36	142.68
Water Content (%)	6.2	6.7	6.7
Dry Unit Weight (kN/m^3)	18.63	18.43	18.06
Saturation (%)	39	41	38
Strain at Failure (%)	2.98	2.80	5.37
Max Deviator Stress (kPa)	804.8	926.2	962.5

Test 24	Test 25	Test 26

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/3/15



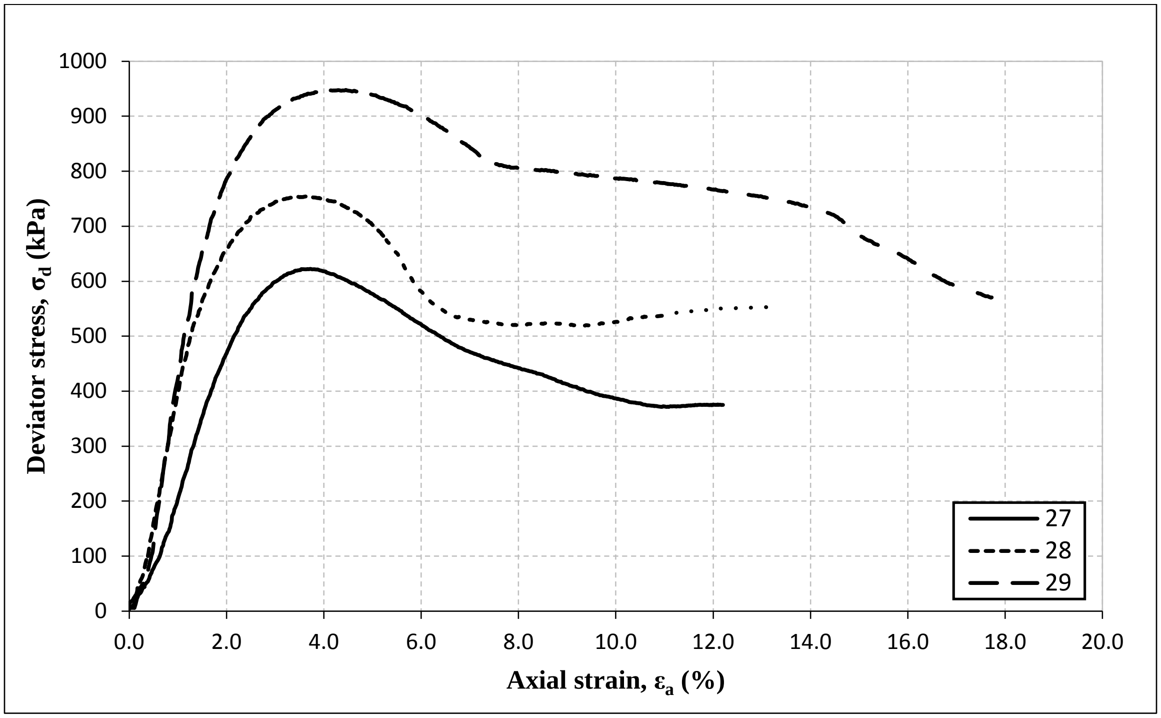


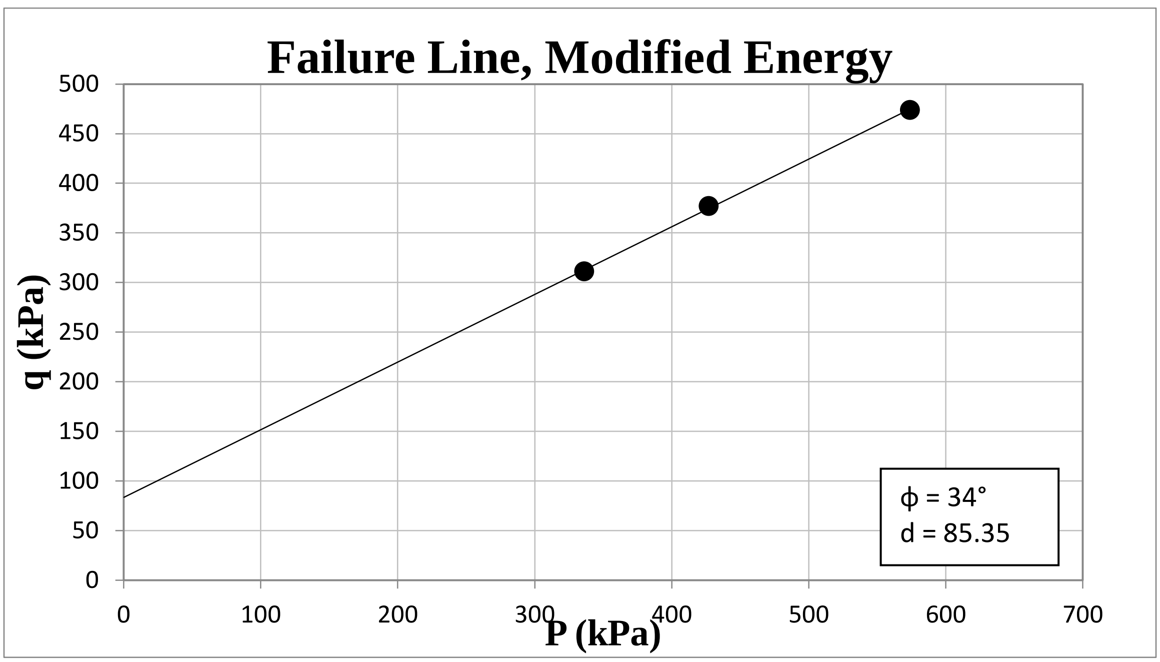
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/5/15

Sample No.	27	28	29
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	36.00	35.93	35.94
Average Height (mm)	71.58	71.45	71.46
Weight (g)	156.07	155.05	155.49
Water Content (%)	7.9	7.8	7.9
Dry Unit Weight (kN/m^3)	19.47	19.48	19.49
Saturation (%)	58	57	58
Strain at Failure (%)	4.02	3.97	4.70
Max Deviator Stress (kPa)	622.4	804.1	947.8

Test 27	Test 28	Test 29

Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	8/5/15



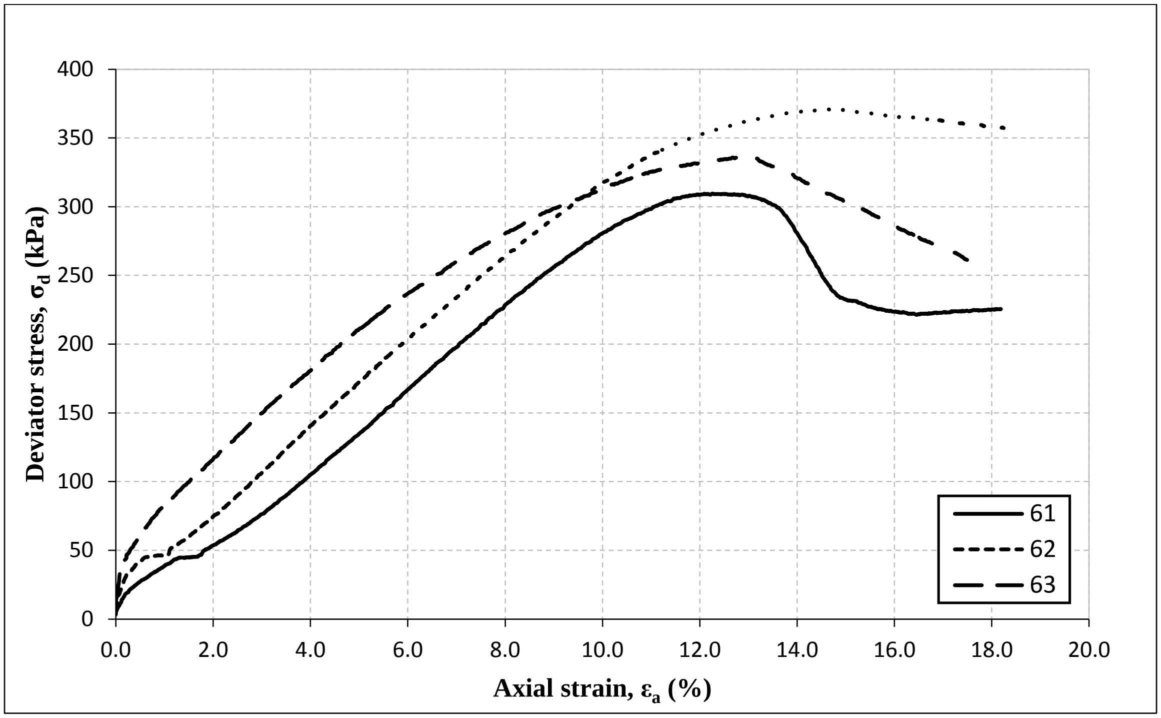


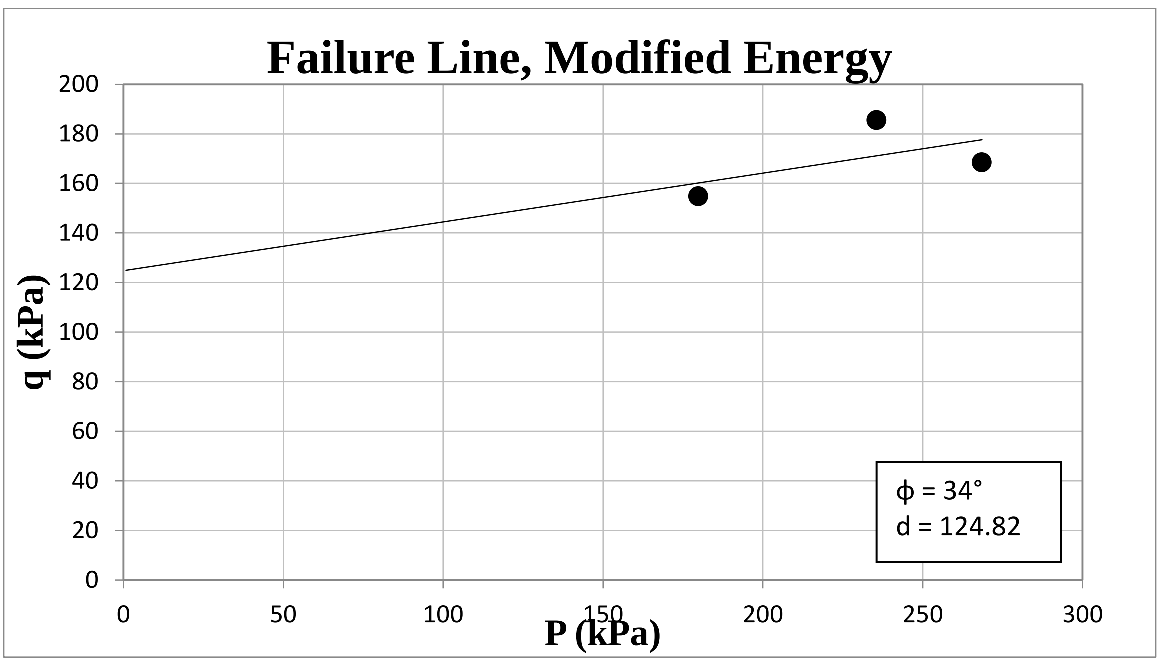
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/17/15

Sample No.	60	61	62
Cell Pressure (kPa)	25	50	100
Average Diameter (mm)	35.81	35.76	35.60
Average Height (mm)	70.97	71.89	71.23
Weight (g)	156.48	155.00	155.82
Water Content (%)	10.9	11.0	11.00
Dry Unit Weight (kN/m^3)	19.35	18.97	19.41
Saturation (%)	78	74	80
Strain at Failure (%)	12.26	14.55	13.36
Max Deviator Stress (kPa)	309.5	371.2	336.9

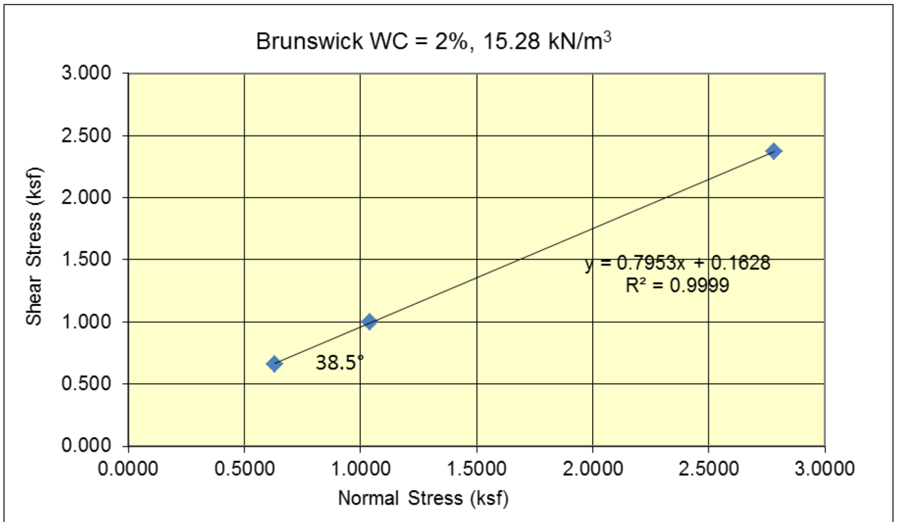
Test 60	Test 61	Test 62

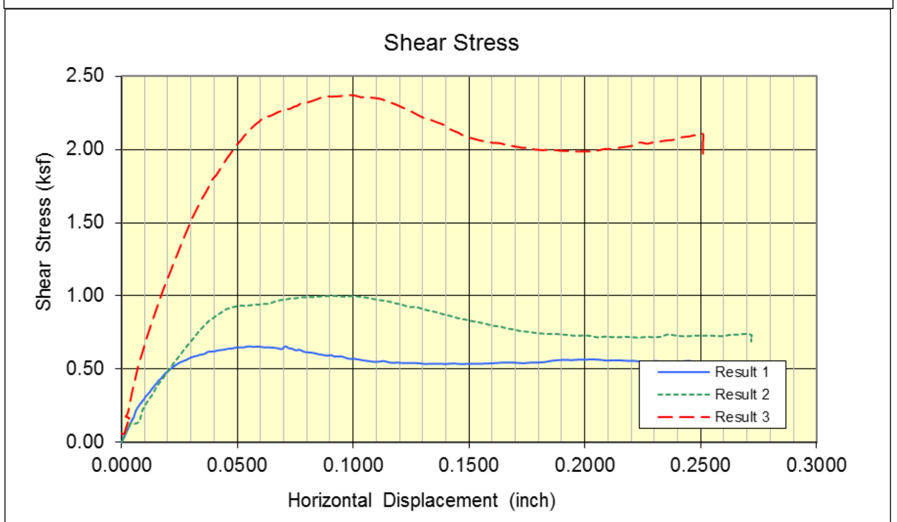
Project Name:	Improvement of Material Criteria for Highway Embankment Construction				
Sample: Kinston, Modified Compaction					
Specimen Type:	Compacted	USCS:	SM	Gs:	2.72
Strain Rate:	1%/min	AASHTO:	A-4 (0)	Date:	9/17/15

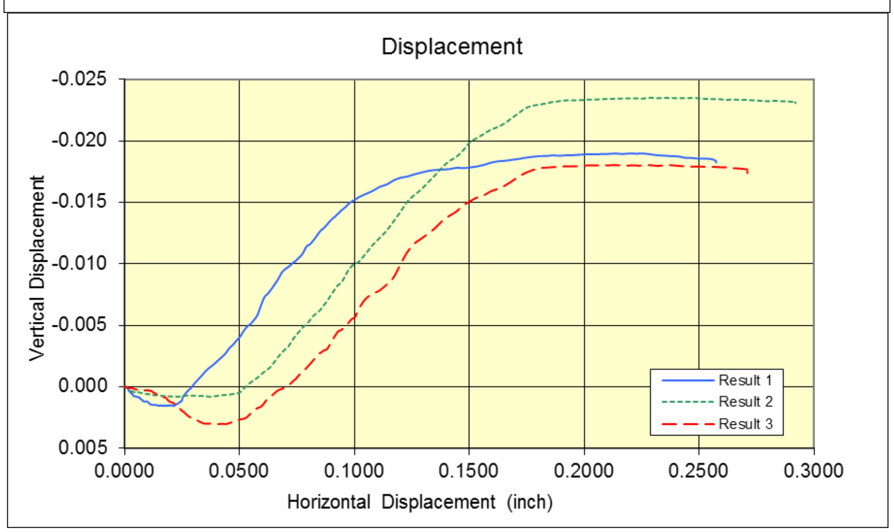


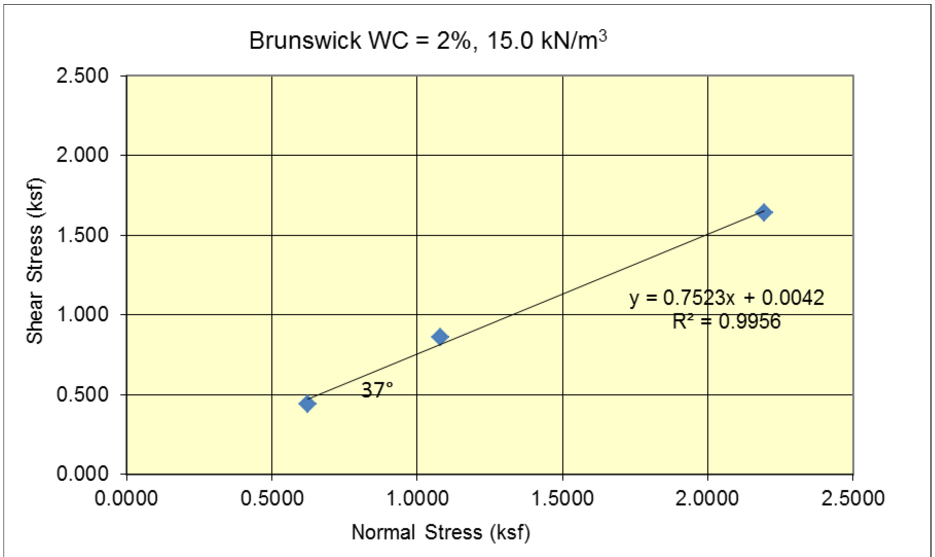


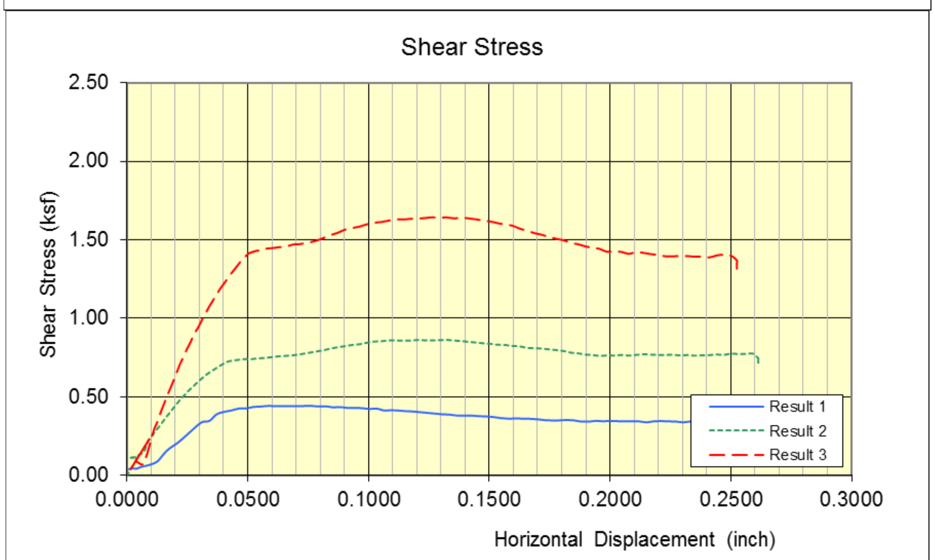
APPENDIX B: DIRECT SHEAR

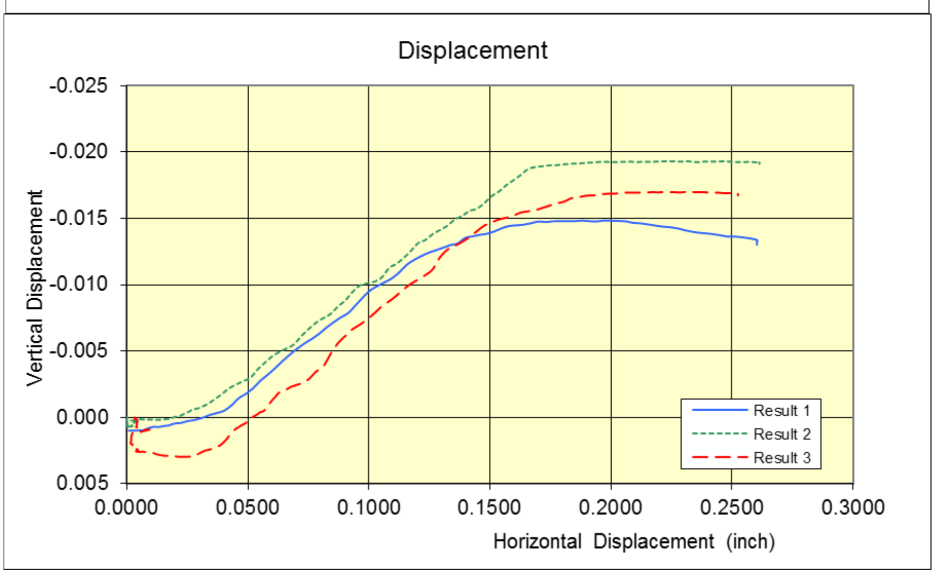


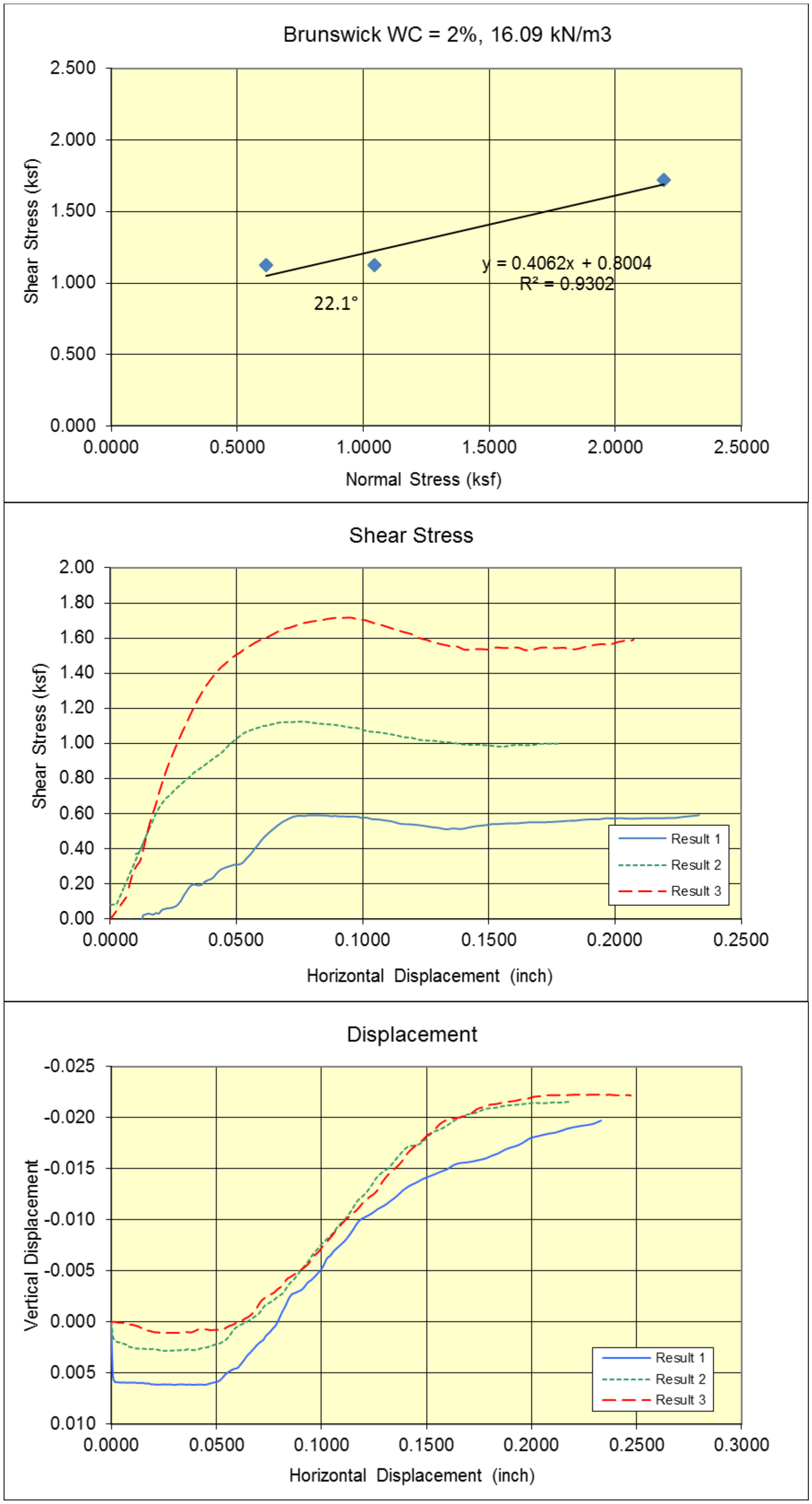




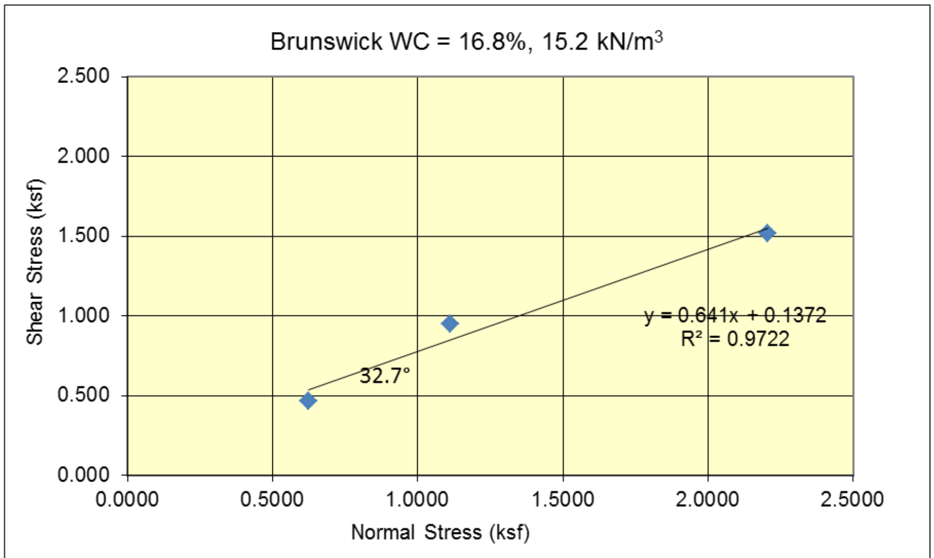

Brunswick WC = 2%, 15.0 kN/m3
y = 0.7523x + 0.0042
R² = 0.9956
37°
Shear Stress (ksf)
Normal Stress (ksf)
2.500
2.000
1.500
1.000
0.500
0.000
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000

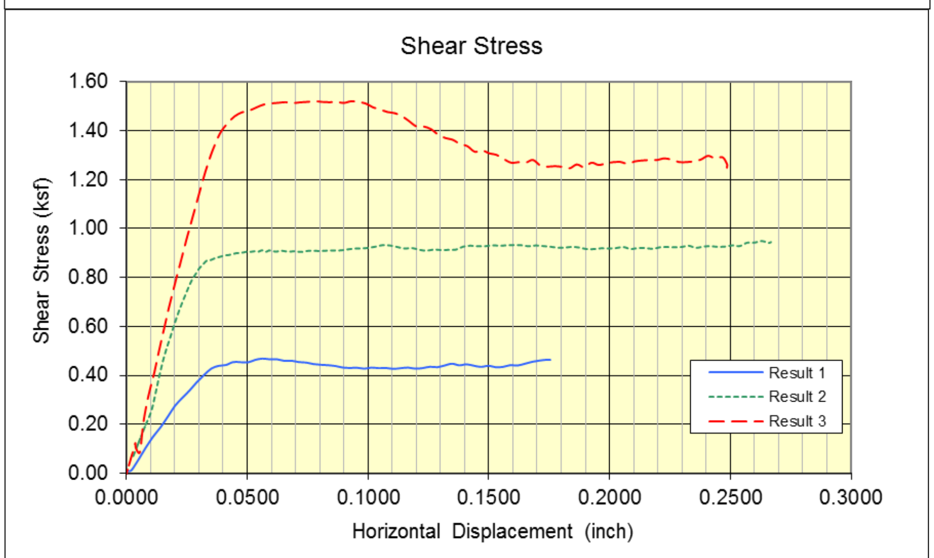

Shear Stress
Shear Stress (ksf)
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
2.50
2.00
1.50
1.00
0.50
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000

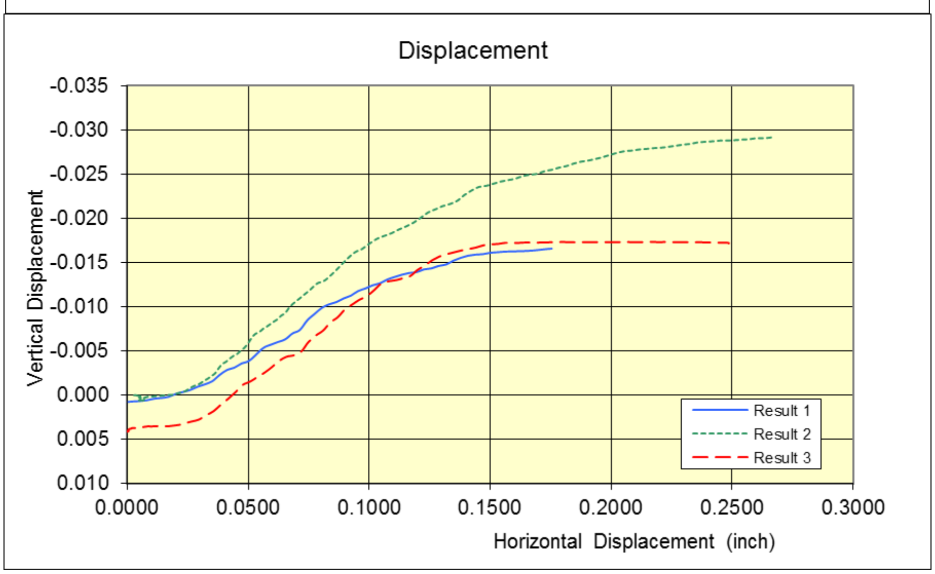

Displacement
Vertical Displacement
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000

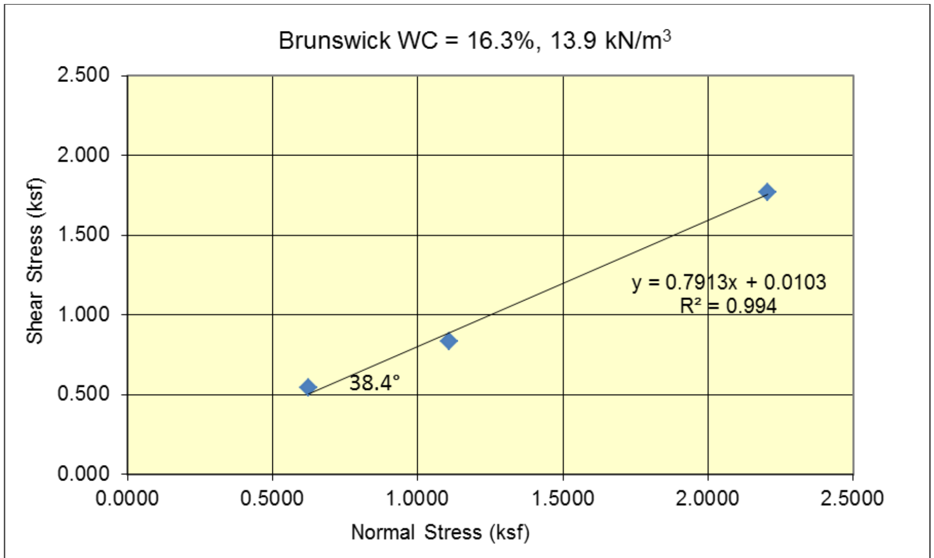


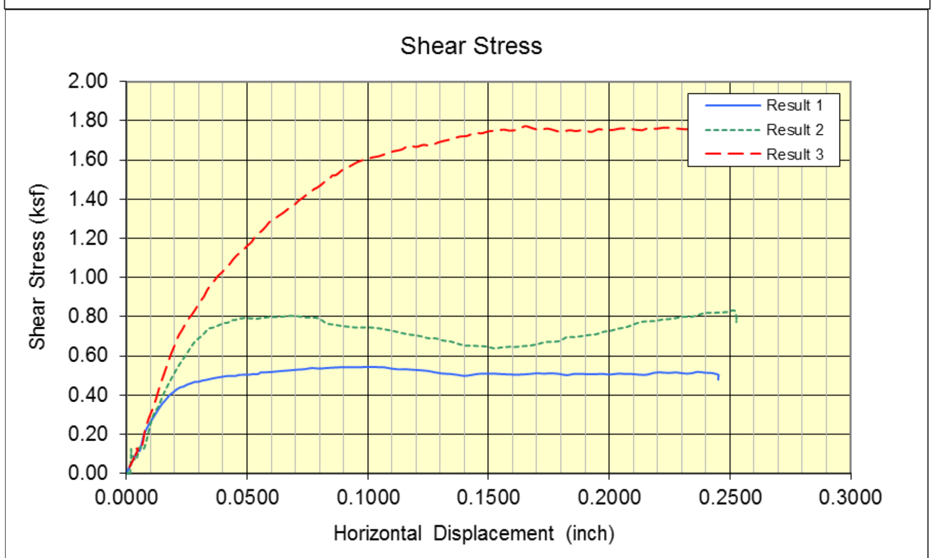
Brunswick WC = 2%, 16.09 kN/m3
Shear Stress (ksf)
Normal Stress (ksf)
y = 0.4062x + 0.8004
R² = 0.9302
22.1°
0.000
0.500
1.000
1.500
2.000
2.500
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000
Shear Stress
Shear Stress (ksf)
Horizontal Displacement (inch)
0.00
0.20
0.40
0.60
0.80
1.00
1.20
1.40
1.60
1.80
2.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
Result 1
Result 2
Result 3
Displacement
Vertical Displacement
Horizontal Displacement (inch)
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Result 1
Result 2
Result 3

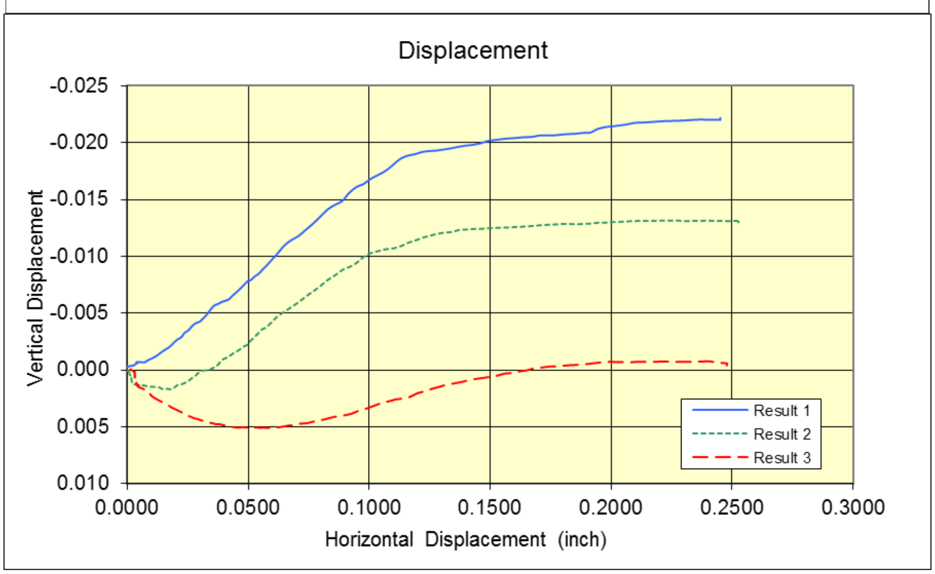

Brunswick WC = 16.8%, 15.2 kN/m3
Shear Stress (ksf)
2.500
2.000
1.500
1.000
0.500
0.000
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000
Normal Stress (ksf)
y = 0.641x + 0.1372
R² = 0.9722
32.7°

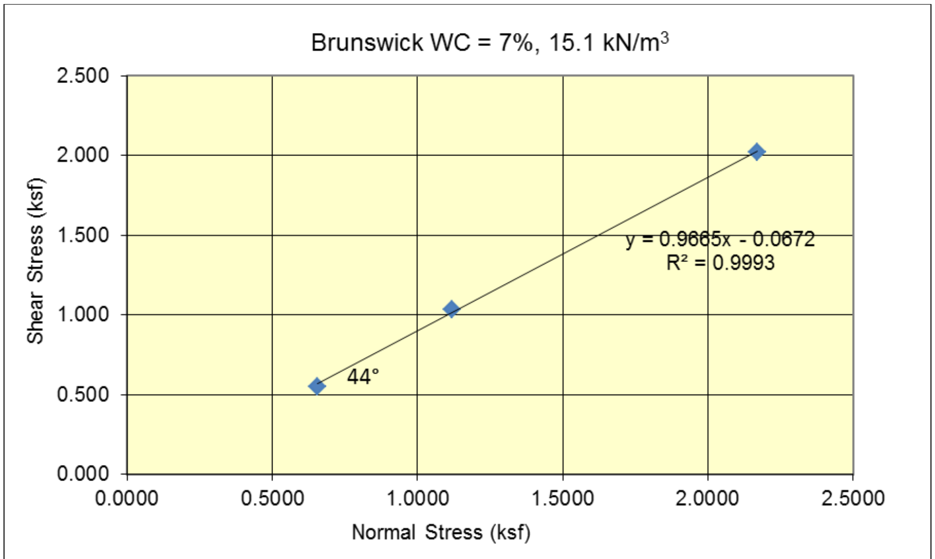

Shear Stress
Shear Stress (ksf)
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Horizontal Displacement (inch)
Result 1
Result 2
Result 3

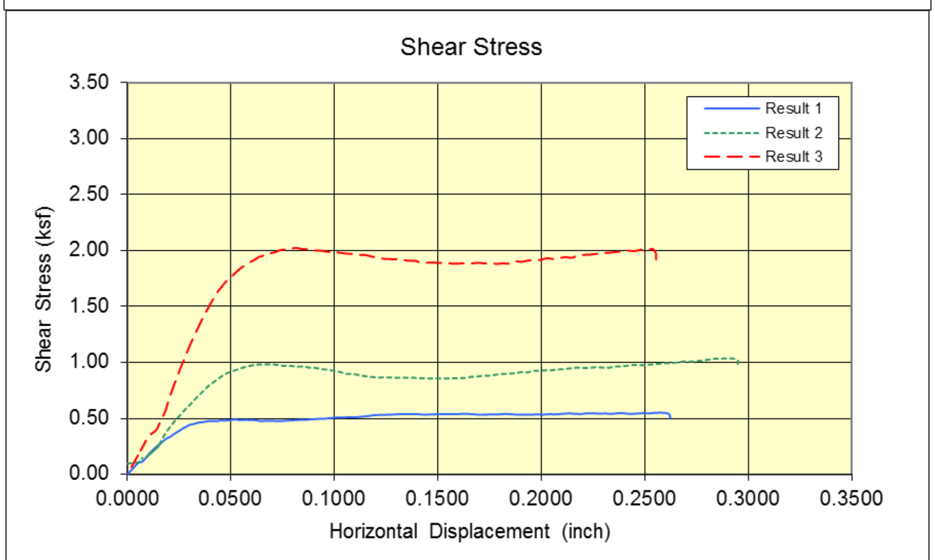

Displacement
Vertical Displacement
-0.035
-0.030
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Horizontal Displacement (inch)
Result 1
Result 2
Result 3

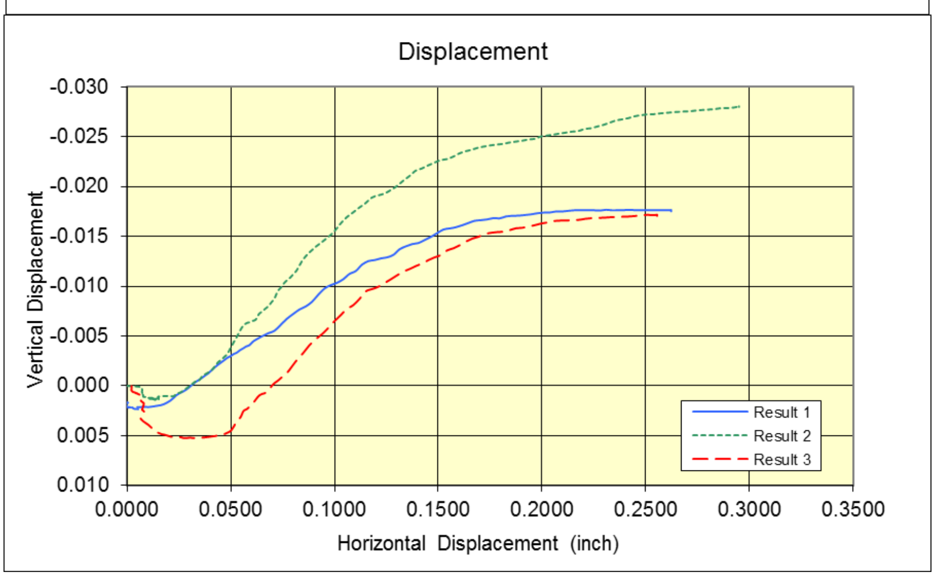

Brunswick WC = 16.3%, 13.9 kN/m3
Shear Stress (ksf)
Normal Stress (ksf)
y = 0.7913x + 0.0103
R² = 0.994
38.4°
2.500
2.000
1.500
1.000
0.500
0.000
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000

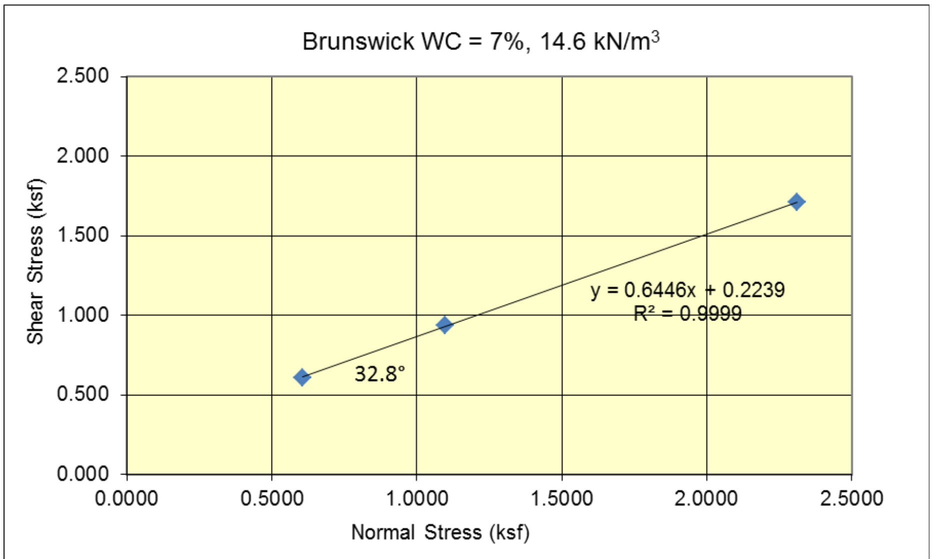

Shear Stress
Shear Stress (ksf)
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
2.00
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000

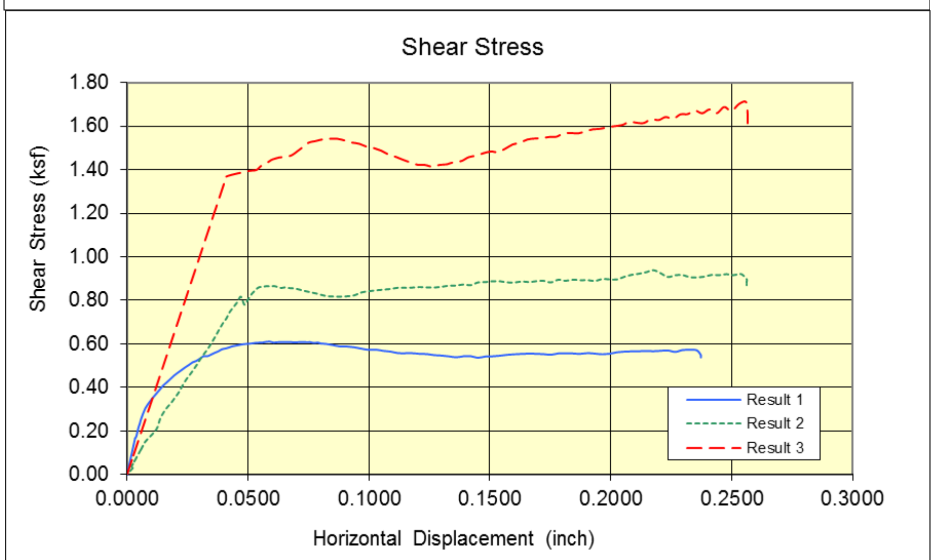

Displacement
Vertical Displacement
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000

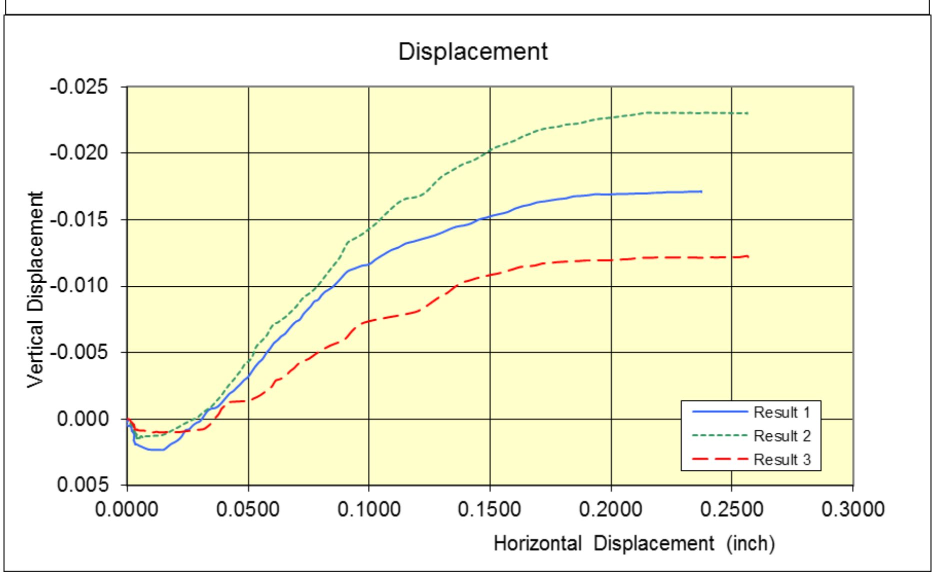

Brunswick WC = 7%, 15.1 kN/m3
y = 0.9665x - 0.0672
R² = 0.9993
44°
Shear Stress (ksf)
Normal Stress (ksf)
2.500
2.000
1.500
1.000
0.500
0.000
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000

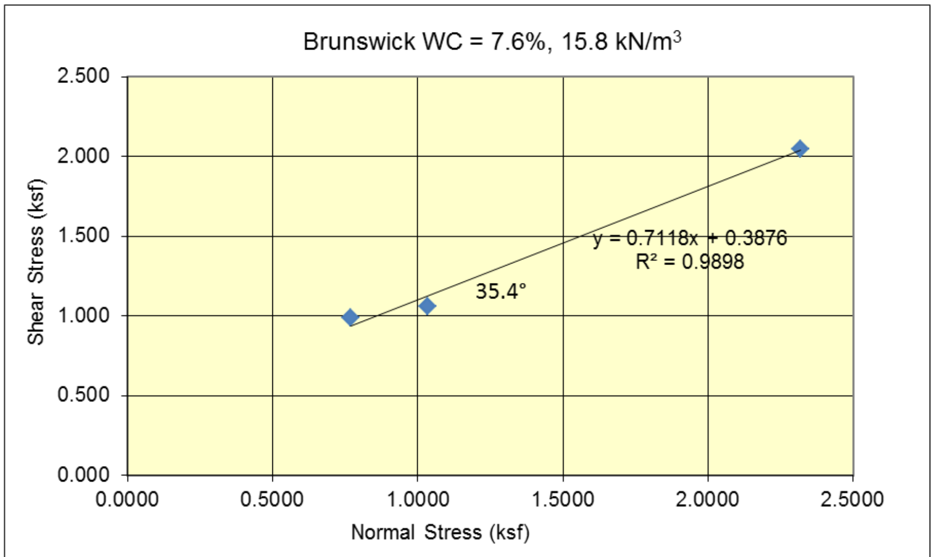

Shear Stress
Result 1
Result 2
Result 3
Shear Stress (ksf)
Horizontal Displacement (inch)
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500

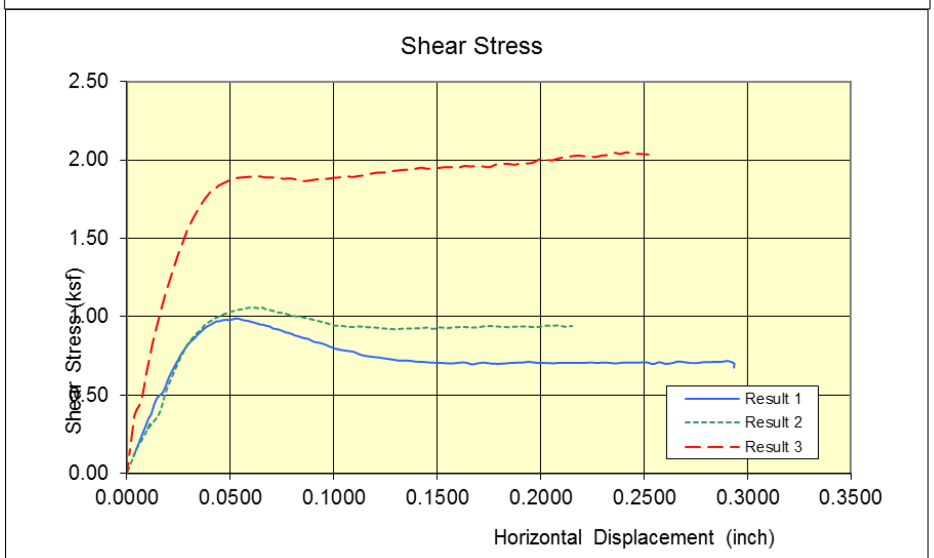

Displacement
Result 1
Result 2
Result 3
Vertical Displacement
Horizontal Displacement (inch)
-0.030
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500

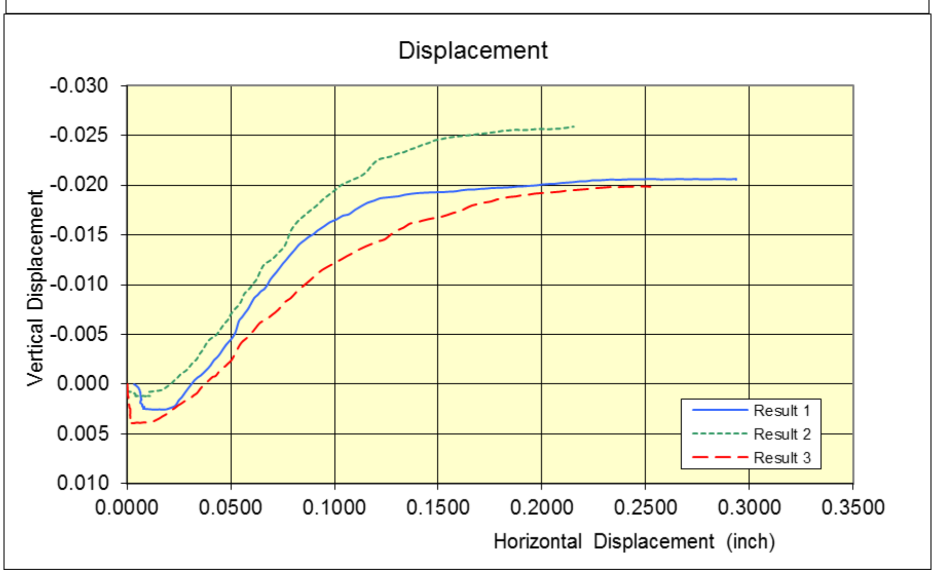

Brunswick WC = 7%, 14.6 kN/m3
Shear Stress (ksf)
2.500
2.000
1.500
1.000
0.500
0.000
y = 0.6446x + 0.2239
R² = 0.9999
32.8°
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000
Normal Stress (ksf)

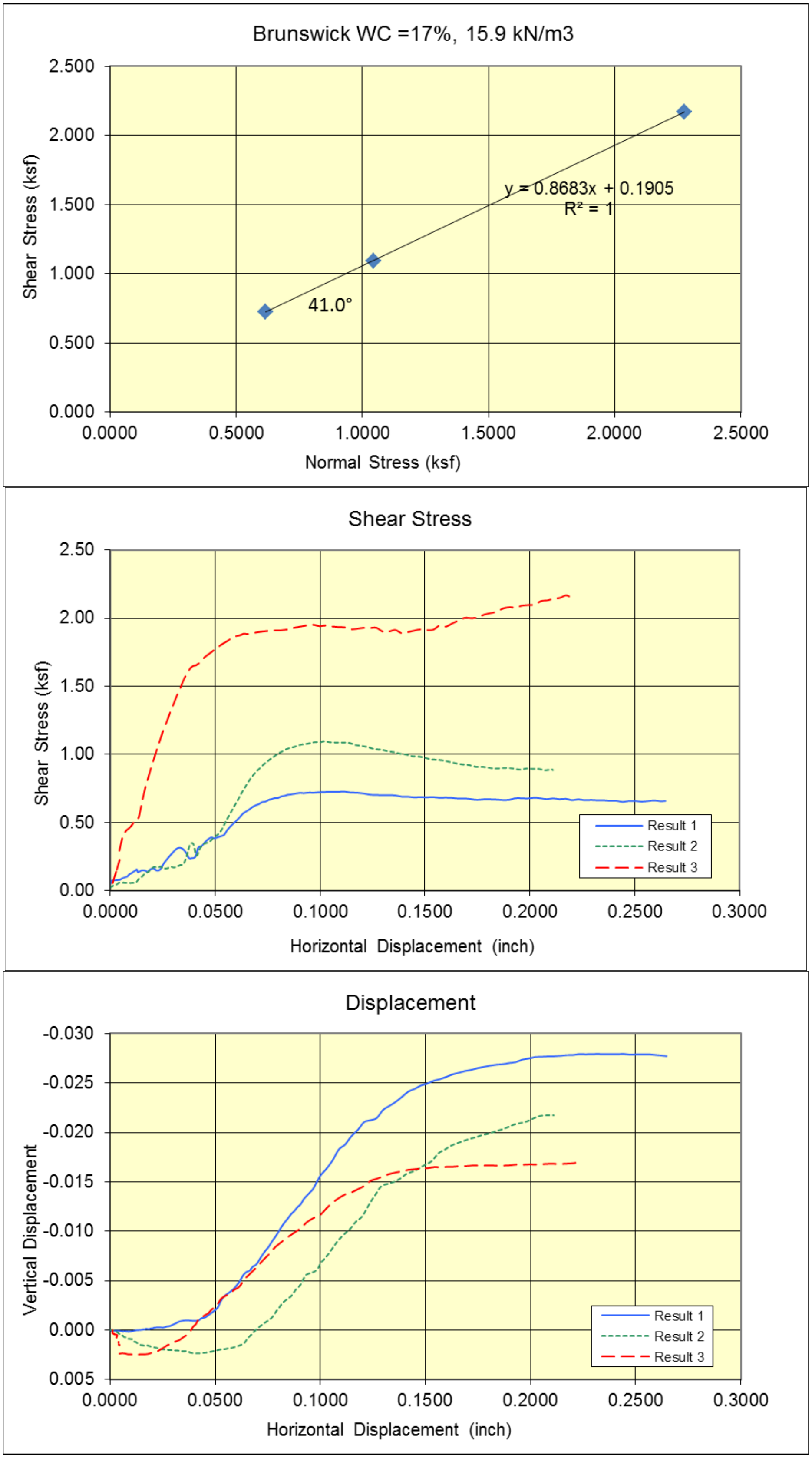

Shear Stress
Shear Stress (ksf)
1.80
1.60
1.40
1.20
1.00
0.80
0.60
0.40
0.20
0.00
Result 1
Result 2
Result 3
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Horizontal Displacement (inch)


Displacement
Vertical Displacement
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
Result 1
Result 2
Result 3
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Horizontal Displacement (inch)


Brunswick WC = 7.6%, 15.8 kN/m3
Shear Stress (ksf)
Normal Stress (ksf)
y = 0.7118x + 0.3876
R² = 0.9898
35.4°
2.500
2.000
1.500
1.000
0.500
0.000
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000

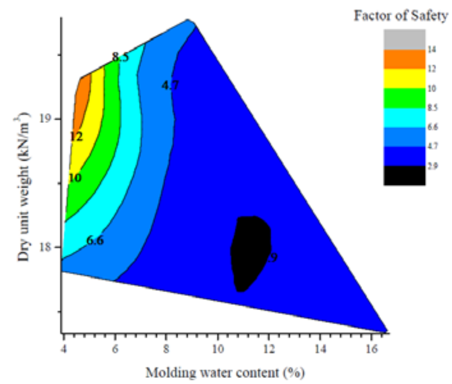

Shear Stress
Shear Stress (ksf)
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
2.50
2.00
1.50
1.00
0.50
0.00
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500


Displacement
Vertical Displacement
Horizontal Displacement (inch)
Result 1
Result 2
Result 3
-0.030
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.010
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
0.3500

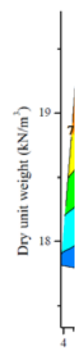

Brunswick WC =17%, 15.9 kN/m3
Shear Stress (ksf)
Normal Stress (ksf)
y = 0.8683x + 0.1905
R² = 1
41.0°
0.000
0.500
1.000
1.500
2.000
2.500
0.0000
0.5000
1.0000
1.5000
2.0000
2.5000
Shear Stress
Shear Stress (ksf)
Horizontal Displacement (inch)
0.00
0.50
1.00
1.50
2.00
2.50
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Result 1
Result 2
Result 3
Displacement
Vertical Displacement
Horizontal Displacement (inch)
-0.030
-0.025
-0.020
-0.015
-0.010
-0.005
0.000
0.005
0.0000
0.0500
0.1000
0.1500
0.2000
0.2500
0.3000
Result 1
Result 2
Result 3

APPENDIX C: SLOPE STABILITY

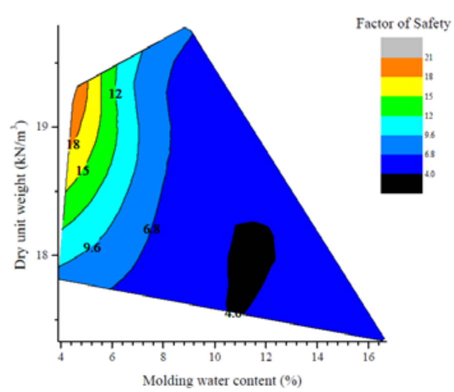
Wayne Soil: Embankment Height = 10 feet



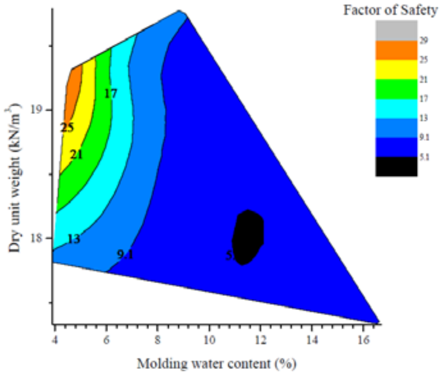
1H:1V Slope



2H:1V Slope

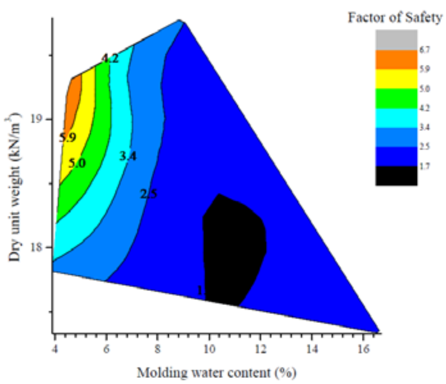


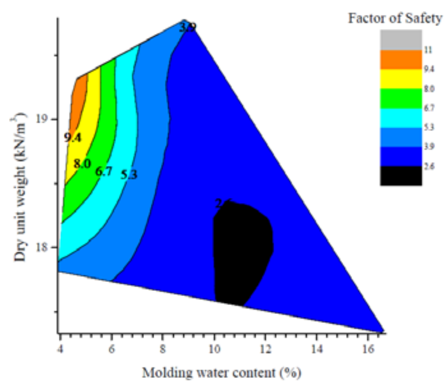
3H:1V Slope



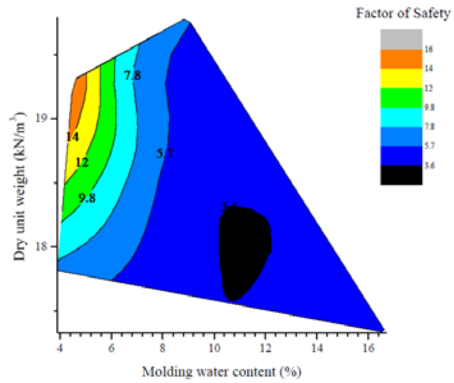
4H:1V Slope

Wayne Soil: Embankment Height = 20 feet



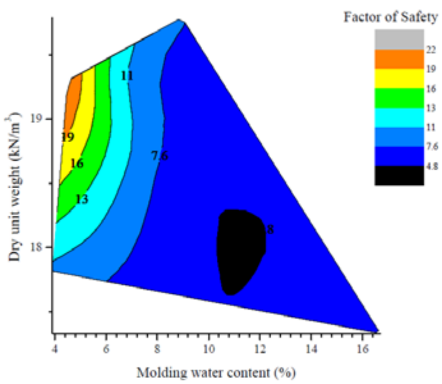


1H:1V Slope

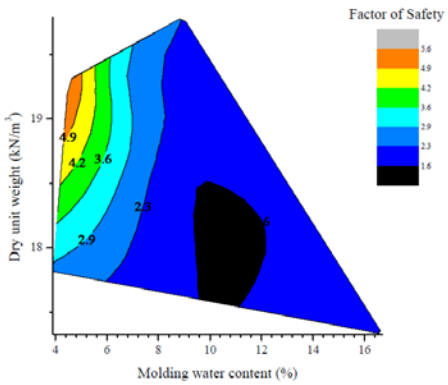


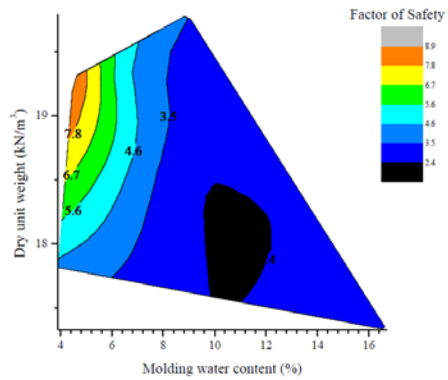
3H:1V Slope

4H:1V Slope



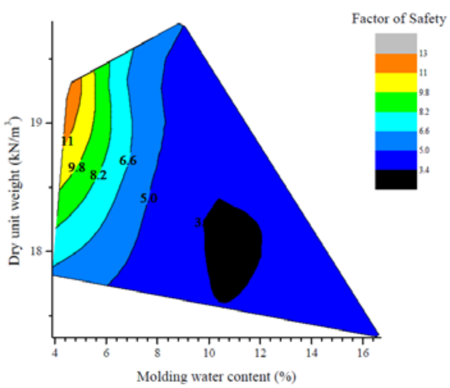
Wayne Soil: Embankment Height = 30 feet

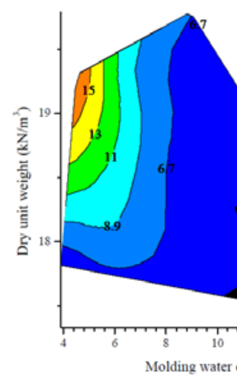




1H:1V Slope

2H:1V Slope



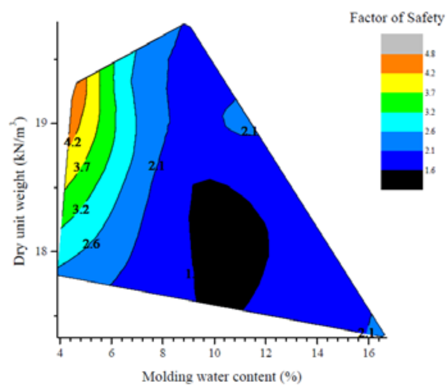


3H:1V Slope

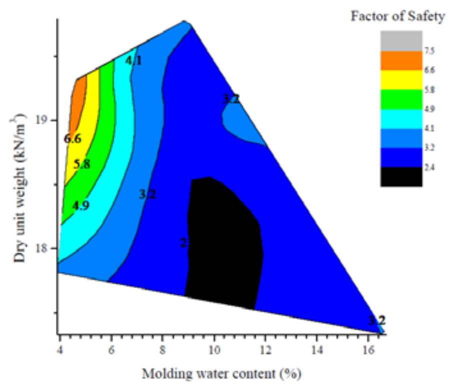
4H:1V Slope

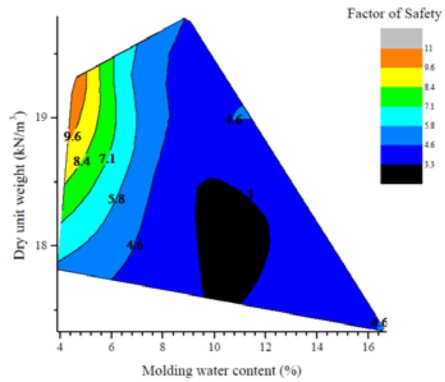
Wayne Soil: Embankment Height = 40 feet

2H:1V Slope

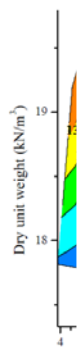


1H:1V Slope



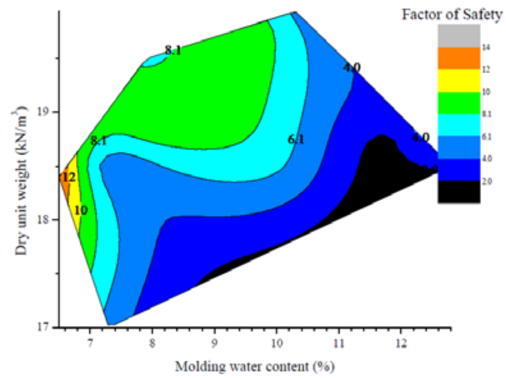


3H:1V Slope



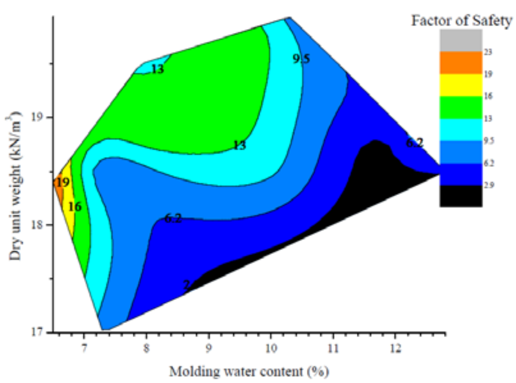
4H:1V Slope

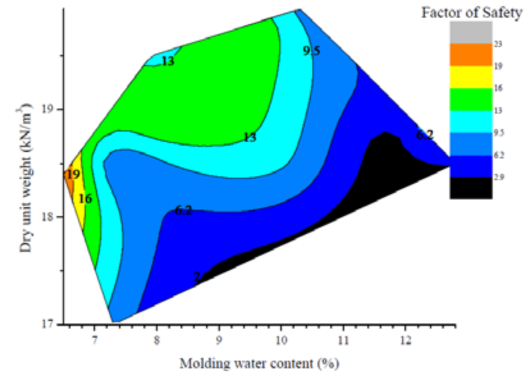
Kinston Soil: Embankment Height = 10 feet



1H:1V Slope

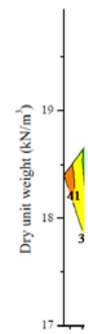
2H:1V Slope



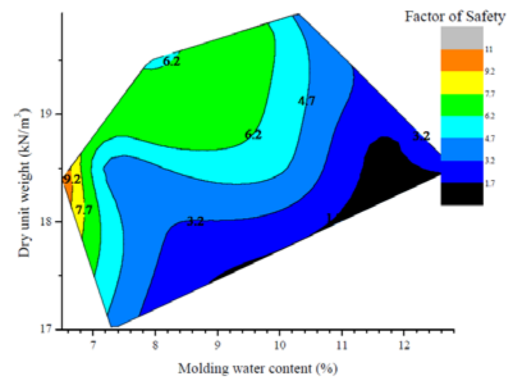


3H:1V Slope

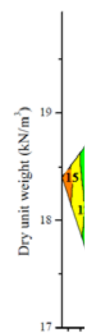
4H:1V Slope



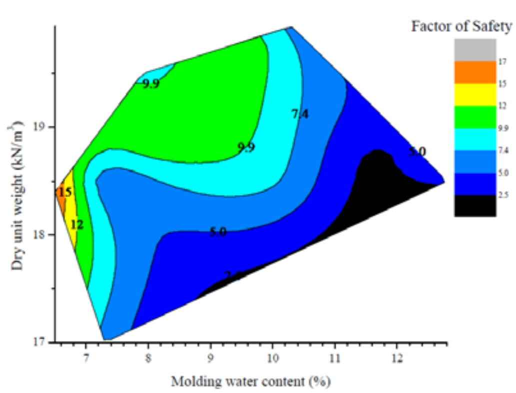
Kinston Soil: Embankment Height = 20 feet



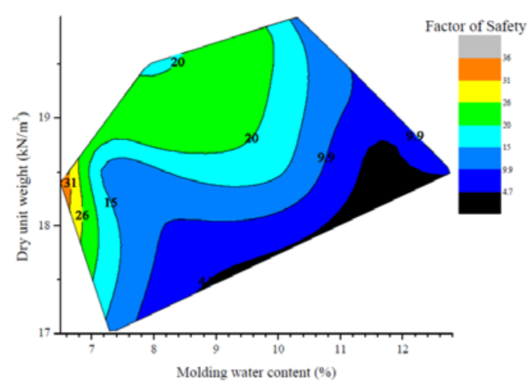
1H:1V Slope



2H:1V Slope

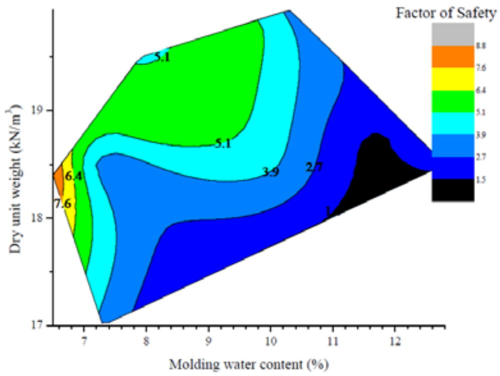


3H:1V Slope



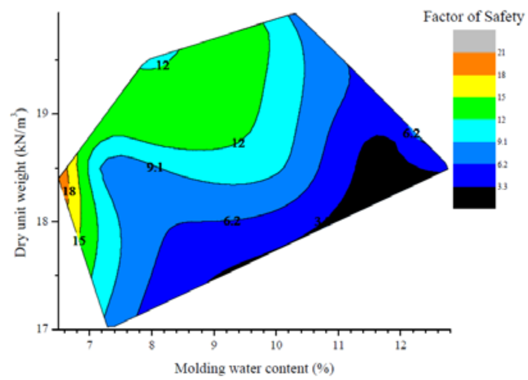
4H:1V Slope

Kinston Soil: Embankment Height = 30 feet



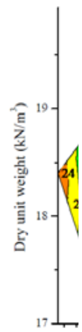
2H:1V Slope

1H:1V Slope

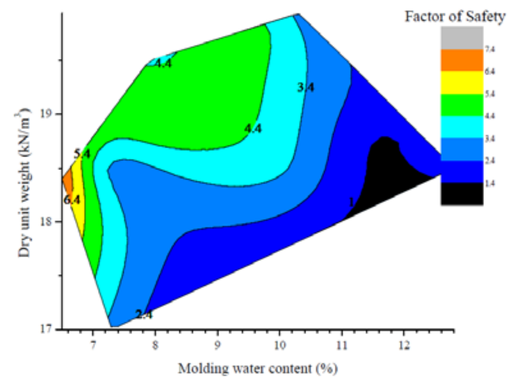


3H:1V Slope

4H:1V Slope

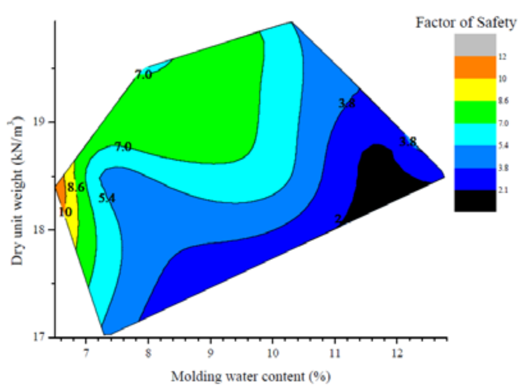


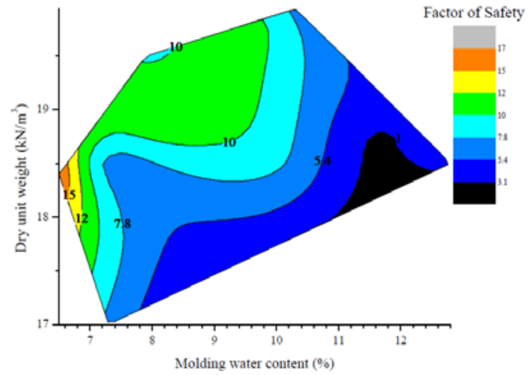
Kinston Soil: Embankment Height = 40 feet



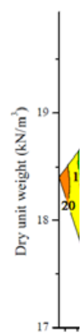
1H:1V Slope

2H:1V Slope



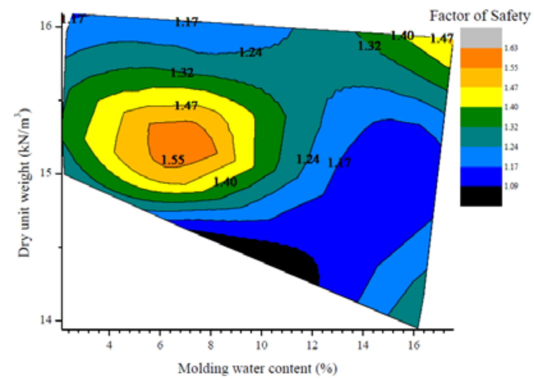


3H:1V Slope



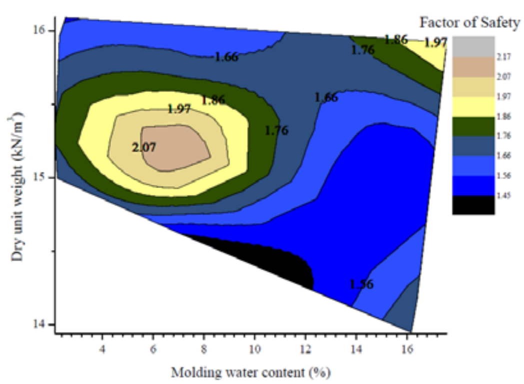
4H:1V Slope

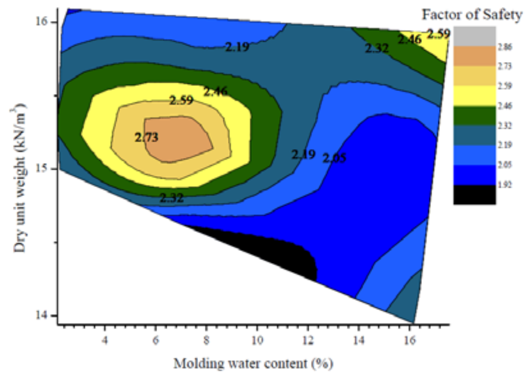
Brunswick Soil: Embankment Height = 10 feet



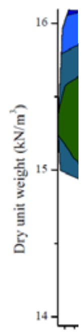
1H:1V Slope

2H:1V Slope



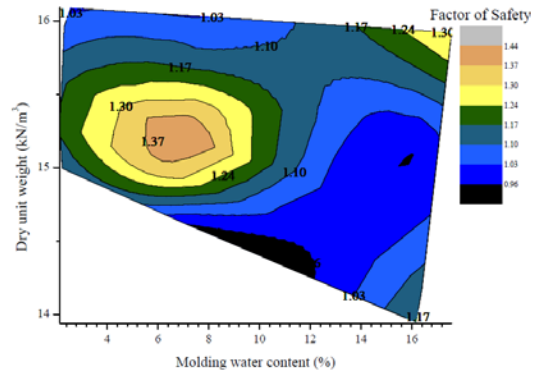


3H:1V Slope



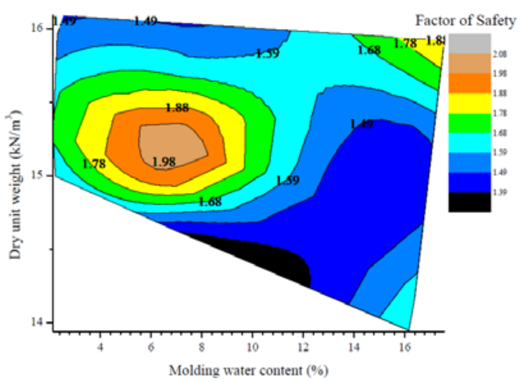
4H:1V Slope

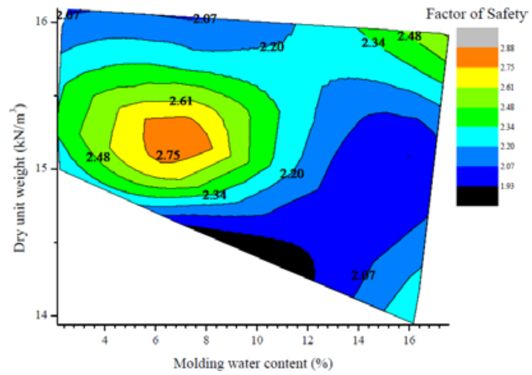
Bunswick Soil: Embankment Height = 20 feet



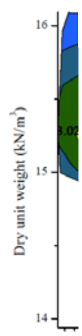
1H:1V Slope

2H:1V Slope



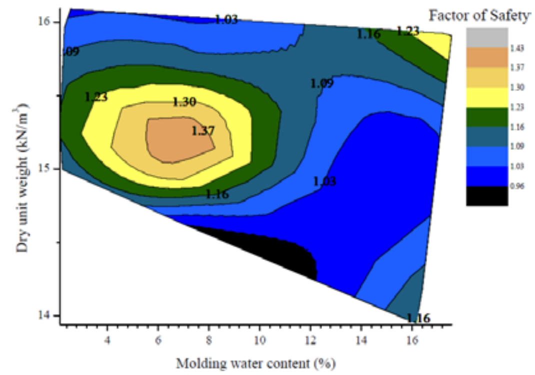


3H:1V Slope



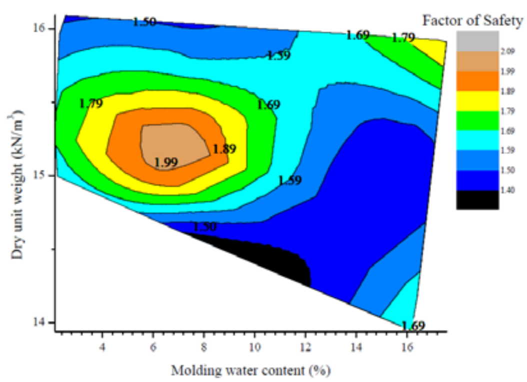
4H:1V Slope

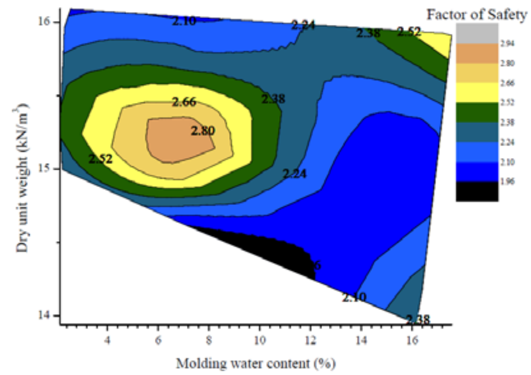
Brunswick Soil: Embankment Height = 30 feet



1H:1V Slope

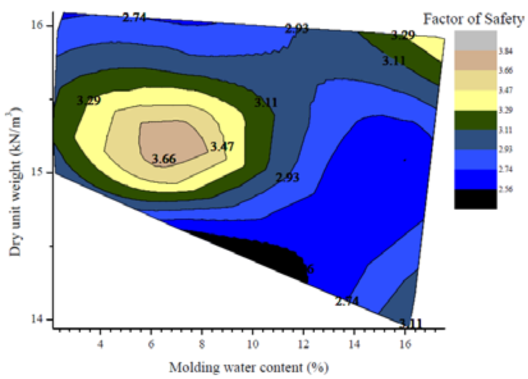
2H:1V Slope



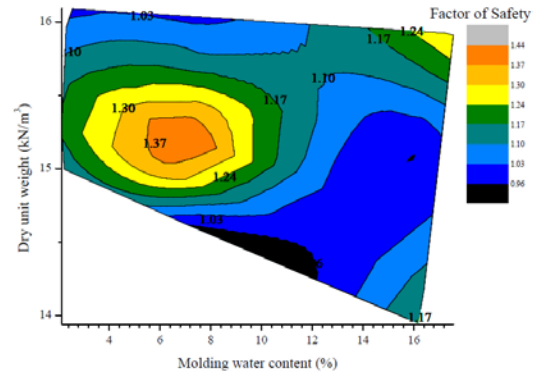


3H:1V Slope

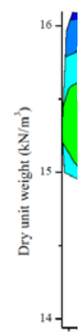
4H:1V Slope



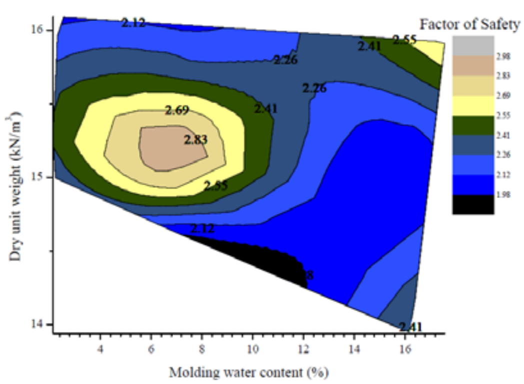
Brunswick Soil: Embankment Height = 40 feet

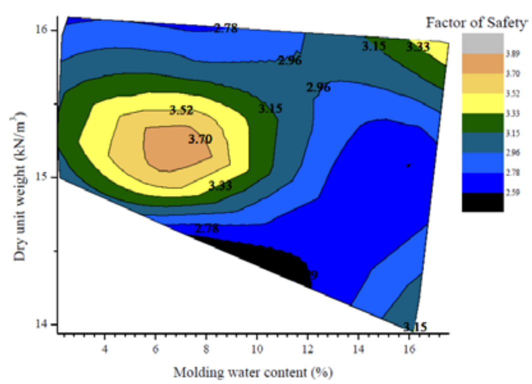


1H:1V Slope



2H:1V Slope





3H:1V Slope

4H:1V Slope

APPENDIX D: STATISITCAL ANALYSIS

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	1:1	10	20.45	11.89	41	10	2.053
Test 2	1:1	10	20.38	14.80	46	13	2.506
Test 3	1:1	10	20.22	16.67	47	21	3.041
Test 4	1:1	10	20.35	9.42	34	21	2.321
Test 5	1:1	10	18.51	3.90	40	44	3.969
Test 6	1:1	10	18.93	5.57	38	43	3.787
Test 7	1:1	10	19.94	5.80	35	85	5.894
Test 9	1:1	10	21.08	9.90	46	17	2.717
Test 10	1:1	10	21.03	10.70	47	21	2.998
Test 11	1:1	10	21.57	8.95	35	40	3.342
Test 14	1:1	10	20.17	4.50	32	142	8.841
Test 15	1:1	10	20.58	6.30	36	70	5.058
Test 1	1:1	20	20.45	11.89	41	10	1.726
Test 2	1:1	20	20.38	14.80	46	13	2.100
Test 3	1:1	20	20.22	16.67	47	21	2.484
Test 4	1:1	20	20.35	9.42	34	21	1.874
Test 5	1:1	20	18.51	3.90	40	44	3.169
Test 6	1:1	20	18.93	5.57	38	43	3.010
Test 7	1:1	20	19.94	5.80	35	85	4.533
Test 9	1:1	20	21.08	9.90	46	17	2.249
Test 10	1:1	20	21.03	10.70	47	21	2.464
Test 11	1:1	20	21.57	8.95	35	40	2.611
Test 14	1:1	20	20.17	4.50	32	142	6.693

Test 15	1:1	20	20.58	6.30	36	70	3.897
Test 1	1:1	30	20.45	11.89	41	10	1.625
Test 2	1:1	30	20.38	14.80	46	13	1.972
Test 3	1:1	30	20.22	16.67	47	21	2.289
Test 4	1:1	30	20.35	9.42	34	21	1.688
Test 5	1:1	30	18.51	3.90	40	44	2.787
Test 6	1:1	30	18.93	5.57	38	43	2.639
Test 7	1:1	30	19.94	5.80	35	85	3.835
Test 9	1:1	30	21.08	9.90	46	17	2.088
Test 10	1:1	30	21.03	10.70	47	21	2.270
Test 11	1:1	30	21.57	8.95	35	40	2.278
Test 14	1:1	30	20.17	4.50	32	142	5.557
Test 15	1:1	30	20.58	6.30	36	70	3.322

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	1:1	40	20.45	11.89	41	10	1.568
Test 2	1:1	40	20.38	14.80	46	13	1.897
Test 3	1:1	40	20.22	16.67	47	21	2.167
Test 4	1:1	40	20.35	9.42	34	21	1.567
Test 5	1:1	40	18.51	3.90	40	44	2.522
Test 6	1:1	40	18.93	5.57	38	43	2.383
Test 7	1:1	40	19.94	5.80	35	85	3.345
Test 9	1:1	40	21.08	9.90	46	17	1.991
Test 10	1:1	40	21.03	10.70	47	21	2.150
Test 11	1:1	40	21.57	8.95	35	40	2.055
Test 14	1:1	40	20.17	4.50	32	142	4.750
Test 15	1:1	40	20.58	6.30	36	70	2.925

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	2:1	10	20.45	11.89	41	10	2.897
Test 2	2:1	10	20.38	14.80	46	13	3.549
Test 3	2:1	10	20.22	16.67	47	21	4.354
Test 4	2:1	10	20.35	9.42	34	21	3.428
Test 5	2:1	10	18.51	3.90	40	44	6.031
Test 6	2:1	10	18.93	5.57	38	43	5.762
Test 7	2:1	10	19.94	5.80	35	85	9.220
Test 9	2:1	10	21.08	9.90	46	17	3.888
Test 10	2:1	10	21.03	10.70	47	21	4.325
Test 11	2:1	10	21.57	8.95	35	40	5.067
Test 14	2:1	10	20.17	4.50	32	142	14.041
Test 15	2:1	10	20.58	6.30	36	70	7.843
Test 1	2:1	20	20.45	11.89	41	10	2.561
Test 2	2:1	20	20.38	14.80	46	13	3.124
Test 3	2:1	20	20.22	16.67	47	21	3.740
Test 4	2:1	20	20.35	9.42	34	21	2.863
Test 5	2:1	20	18.51	3.90	40	44	4.938
Test 6	2:1	20	18.93	5.57	38	43	4.693
Test 7	2:1	20	19.94	5.80	35	85	7.198
Test 9	2:1	20	21.08	9.90	46	17	3.366
Test 10	2:1	20	21.03	10.70	47	21	3.706
Test 11	2:1	20	21.57	8.95	35	40	4.058
Test 14	2:1	20	20.17	4.50	32	142	10.735

Test 15	2:1	20	20.58	6.30	36	70	6.152
Test 1	2:1	30	20.45	11.89	41	10	2.424
Test 2	2:1	30	20.38	14.80	46	13	2.947
Test 3	2:1	30	20.22	16.67	47	21	3.454
Test 4	2:1	30	20.35	9.42	34	21	2.578
Test 5	2:1	30	18.51	3.90	40	44	4.331
Test 6	2:1	30	18.93	5.57	38	43	4.103
Test 7	2:1	30	19.94	5.80	35	85	6.056
Test 9	2:1	30	21.08	9.90	46	17	3.135
Test 10	2:1	30	21.03	10.70	47	21	3.422
Test 11	2:1	30	21.57	8.95	35	40	3.531
Test 14	2:1	30	20.17	4.50	32	142	8.848
Test 15	2:1	30	20.58	6.30	36	70	5.222

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	2:1	40	20.45	11.89	4:01	10	2.455
Test 2	2:1	40	20.38	14.80	46	13	2.978
Test 3	2:1	40	20.22	16.67	47	21	3.260
Test 4	2:1	40	20.35	9.42	34	21	2.381
Test 5	2:1	40	18.51	3.90	40	44	3.890
Test 6	2:1	40	18.93	5.57	38	43	3.678
Test 7	2:1	40	19.94	5.80	35	85	5.240
Test 9	2:1	40	21.08	9.90	46	17	2.983
Test 10	2:1	40	21.03	10.70	47	21	3.405
Test 11	2:1	40	21.57	8.95	35	40	3.366
Test 14	2:1	40	20.17	4.50	32	142	7.489
Test 15	2:1	40	20.58	6.30	36	70	4.561

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	3:1	10	20.45	11.89	41	10	3.973
Test 2	3:1	10	20.38	14.80	46	13	4.881
Test 3	3:1	10	20.22	16.67	47	21	6.074
Test 4	3:1	10	20.35	9.42	34	21	4.858
Test 5	3:1	10	18.51	3.90	40	44	8.725
Test 6	3:1	10	18.93	5.57	38	43	8.340
Test 7	3:1	10	19.94	5.80	35	85	13.591
Test 9	3:1	10	21.08	9.90	46	17	5.385
Test 10	3:1	10	21.03	10.70	47	21	6.026
Test 11	3:1	10	21.57	8.95	35	40	7.305
Test 14	3:1	10	20.17	4.50	32	142	20.908
Test 15	3:1	10	20.58	6.30	36	70	11.488
Test 1	3:1	20	20.45	11.89	41	10	3.618
Test 2	3:1	20	20.38	14.80	46	13	4.420
Test 3	3:1	20	20.22	16.67	47	21	5.336
Test 4	3:1	20	20.35	9.42	34	21	4.122
Test 5	3:1	20	18.51	3.90	40	44	7.216
Test 6	3:1	20	18.93	5.57	38	43	6.859
Test 7	3:1	20	19.94	5.80	35	85	10.655
Test 9	3:1	20	21.08	9.90	46	17	4.780
Test 10	3:1	20	21.03	10.70	47	21	5.280
Test 11	3:1	20	21.57	8.95	35	40	5.906
Test 14	3:1	20	20.17	4.50	32	142	16.012
Test 15	3:1	20	20.58	6.30	36	70	9.060
Test 1	3:1	30	20.45	11.89	41	10	3.435
Test 2	3:1	30	20.38	14.80	46	13	4.178
Test 3	3:1	30	20.22	16.67	47	21	4.921
Test 4	3:1	30	20.35	9.42	34	21	3.694
Test 5	3:1	30	18.51	3.90	40	44	6.264
Test 6	3:1	30	18.93	5.57	38	43	5.935
Test 7	3:1	30	19.94	5.80	35	85	8.840
Test 9	3:1	30	21.08	9.90	46	17	4.454
Test 10	3:1	30	21.03	10.70	47	21	4.871
Test 11	3:1	30	21.57	8.95	35	40	5.093
Test 14	3:1	30	20.17	4.50	32	142	12.981
Test 15	3:1	30	20.58	6.30	36	70	7.591

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	3:1	40	20.45	11.89	0:00	10	3.308
Test 2	3:1	40	20.38	14.80	46	13	4.011
Test 3	3:1	40	20.22	16.67	47	21	4.635
Test 4	3:1	40	20.35	9.42	34	21	3.399
Test 5	3:1	40	18.51	3.90	40	44	5.592
Test 6	3:1	40	18.93	5.57	38	43	5.287
Test 7	3:1	40	19.94	5.80	35	85	7.584
Test 9	3:1	40	21.08	9.90	46	17	4.232
Test 10	3:1	40	21.03	10.70	47	21	4.591
Test 11	3:1	40	21.57	8.95	35	40	4.540
Test 14	3:1	40	20.17	4.50	32	142	10.888
Test 15	3:1	40	20.58	6.30	36	70	6.582

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	4:1	10	20.45	11.89	41	10	5.186
Test 2	4:1	10	20.38	14.80	46	13	6.386
Test 3	4:1	10	20.22	16.67	47	21	8.047
Test 4	4:1	10	20.35	9.42	34	21	6.520
Test 5	4:1	10	18.51	3.90	40	44	11.918
Test 6	4:1	10	18.93	5.57	38	43	11.396
Test 7	4:1	10	19.94	5.80	35	85	18.826
Test 9	4:1	10	21.08	9.90	46	17	7.089
Test 10	4:1	10	21.03	10.70	47	21	7.972
Test 11	4:1	10	21.57	8.95	35	40	9.940
Test 14	4:1	10	20.17	4.50	32	142	29.181
Test 15	4:1	10	20.58	6.30	36	70	15.833
Test 1	4:1	20	20.45	11.89	41	10	4.775
Test 2	4:1	20	20.38	14.80	46	13	5.841
Test 3	4:1	20	20.22	16.67	47	21	7.101
Test 4	4:1	20	20.35	9.42	34	21	5.529
Test 5	4:1	20	18.51	3.90	40	44	9.798
Test 6	4:1	20	18.93	5.57	38	43	9.314
Test 7	4:1	20	19.94	5.80	35	85	14.605
Test 9	4:1	20	21.08	9.90	46	17	6.336
Test 10	4:1	20	21.03	10.70	47	21	7.020
Test 11	4:1	20	21.57	8.95	35	40	7.990
Test 14	4:1	20	20.17	4.50	32	142	22.074
Test 15	4:1	20	20.58	6.30	36	70	12.371
Test 1	4:1	30	20.45	11.89	41	10	4.516
Test 2	4:1	30	20.38	14.80	46	13	5.497
Test 3	4:1	30	20.22	16.67	47	21	6.498
Test 4	4:1	30	20.35	9.42	34	21	4.899
Test 5	4:1	30	18.51	3.90	40	44	4.726
Test 6	4:1	30	18.93	5.57	38	43	7.929
Test 7	4:1	30	19.94	5.80	35	85	11.881
Test 9	4:1	30	21.08	9.90	46	17	5.869
Test 10	4:1	30	21.03	10.70	47	21	6.428
Test 11	4:1	30	21.57	8.95	35	40	6.788
Test 14	4:1	30	20.17	4.50	32	142	17.515
Test 15	4:1	30	20.58	6.30	36	70	10.177

Wayne Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Friction	Cohesion	Factor of Safety
Test 1	4:1	40	20.45	11.89	41	10	4.342
Test 2	4:1	40	20.38	14.80	46	13	5.267
Test 3	4:1	40	20.22	16.67	47	21	6.101
Test 4	4:1	40	20.35	9.42	34	21	4.487
Test 5	4:1	40	18.51	3.90	40	44	7.421
Test 6	4:1	40	18.93	5.57	38	43	7.017
Test 7	4:1	40	19.94	5.80	35	85	10.112
Test 9	4:1	40	21.08	9.90	46	17	5.563
Test 10	4:1	40	21.03	10.70	47	21	6.041
Test 11	4:1	40	21.57	8.95	35	40	6.015
Test 14	4:1	40	20.17	4.50	32	142	14.565
Test 15	4:1	40	20.58	6.30	36	70	8.759

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	1:1	10	21.09	12.2	56	30	4.003
Test 2	1:1	10	20.37	10.1	89	37	6.156
Test 3	1:1	10	18.24	7.3	70	37	5.333
Test 5	1:1	10	19.83	7.2	68	47	5.623
Test 6	1:1	10	20.41	8	163	32	9.931
Test 7	1:1	10	21.74	9.8	145	29	8.576
Test 8	1:1	10	20.87	12.8	10	44	2.218
Test 10	1:1	10	20.99	11.9	16	34	2.037
Test 11	1:1	10	21.99	10.3	79	40	5.602
Test 12	1:1	10	21.65	11.1	56	35	4.180
Test 13	1:1	10	19.60	6.5	239	28	14.152
Test 14	1:1	10	21.04	7.9	114	43	7.727
Test 1	1:1	20	21.09	12.2	56	30	3.072
Test 2	1:1	20	20.37	10.1	89	37	4.722
Test 3	1:1	20	18.24	7.3	70	37	4.191
Test 5	1:1	20	19.83	7.2	68	47	4.420
Test 6	1:1	20	20.41	8	163	32	7.485
Test 7	1:1	20	21.74	9.8	145	29	6.388
Test 8	1:1	20	20.87	12.8	10	44	1.869
Test 10	1:1	20	20.99	11.9	16	34	1.660
Test 11	1:1	20	21.99	10.3	79	40	4.279
Test 12	1:1	20	21.65	11.1	56	35	3.220
Test 13	1:1	20	19.60	6.5	239	28	10.683
Test 14	1:1	20	21.04	7.9	114	43	5.890
Test 1	1:1	30	21.09	12.2	56	30	2.616
Test 2	1:1	30	20.37	10.1	89	37	3.995
Test 3	1:1	30	18.24	7.3	70	37	3.597
Test 5	1:1	30	19.83	7.2	68	47	3.844
Test 6	1:1	30	20.41	8	163	32	6.191
Test 7	1:1	30	21.74	9.8	145	29	5.260
Test 8	1:1	30	20.87	12.8	10	44	1.766
Test 10	1:1	30	20.99	11.9	16	34	1.517
Test 11	1:1	30	21.99	10.3	79	40	3.642
Test 12	1:1	30	21.65	11.1	56	35	2.762
Test 13	1:1	30	19.60	6.5	239	28	8.788
Test 14	1:1	30	21.04	7.9	114	43	4.970

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	1:1	40	21.09	12.2	56	30	2.302
Test 2	1:1	40	20.37	10.1	89	37	3.487
Test 3	1:1	40	18.24	7.3	70	37	3.173
Test 5	1:1	40	19.83	7.2	68	47	3.449
Test 6	1:1	40	20.41	8	163	32	5.272
Test 7	1:1	40	21.74	9.8	145	29	4.473
Test 8	1:1	40	20.87	12.8	10	44	1.709
Test 10	1:1	40	20.99	11.9	16	34	1.425
Test 11	1:1	40	21.99	10.3	79	40	3.210
Test 12	1:1	40	21.65	11.1	56	35	2.452
Test 13	1:1	40	19.60	6.5	239	28	7.422
Test 14	1:1	40	21.04	7.9	114	43	4.332

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	2:1	10	21.09	12.2	56	30	6.202
Test 2	2:1	10	20.37	10.1	89	37	9.613
Test 3	2:1	10	18.24	7.3	70	37	8.296
Test 5	2:1	10	19.83	7.2	68	47	8.592
Test 6	2:1	10	20.41	8	163	32	15.807
Test 7	2:1	10	21.74	9.8	145	29	13.623
Test 8	2:1	10	20.87	12.8	10	44	3.114
Test 10	2:1	10	20.99	11.9	16	34	2.965
Test 11	2:1	10	21.99	10.3	79	40	8.656
Test 12	2:1	10	21.65	11.1	56	35	6.425
Test 13	2:1	10	19.60	6.5	239	28	22.730
Test 14	2:1	10	21.04	7.9	114	43	12.065
Test 1	2:1	20	21.09	12.2	56	30	4.847
Test 2	2:1	20	20.37	10.1	89	37	7.489
Test 3	2:1	20	18.24	7.3	70	37	6.635
Test 5	2:1	20	19.83	7.2	68	47	6.908
Test 6	2:1	20	20.41	8	163	32	12.020
Test 7	2:1	20	21.74	9.8	145	29	10.241
Test 8	2:1	20	20.87	12.8	10	44	2.765
Test 10	2:1	20	20.99	11.9	16	34	2.512
Test 11	2:1	20	21.99	10.3	79	40	6.736
Test 12	2:1	20	21.65	11.1	56	35	5.050
Test 13	2:1	20	19.60	6.5	239	28	17.255
Test 14	2:1	20	21.04	7.9	114	43	9.337
Test 1	2:1	30	21.09	12.2	56	30	4.109
Test 2	2:1	30	20.37	10.1	89	37	6.305
Test 3	2:1	30	18.24	7.3	70	37	5.668
Test 5	2:1	30	19.83	7.2	68	47	5.990
Test 6	2:1	30	20.41	8	163	32	9.866
Test 7	2:1	30	21.74	9.8	145	29	8.373
Test 8	2:1	30	20.87	12.8	10	44	2.628
Test 10	2:1	30	20.99	11.9	16	34	2.296
Test 11	2:1	30	21.99	10.3	79	40	5.712
Test 12	2:1	30	21.65	11.1	56	35	4.318
Test 13	2:1	30	19.60	6.5	239	28	14.056
Test 14	2:1	30	21.04	7.9	114	43	7.842

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	2:1	40	21.09	12.2	56	30	3.846
Test 2	2:1	40	20.37	10.1	89	37	5.879
Test 3	2:1	40	18.24	7.3	70	37	4.956
Test 5	2:1	40	19.83	7.2	68	47	5.334
Test 6	2:1	40	20.41	8	163	32	8.318
Test 7	2:1	40	21.74	9.8	145	29	7.051
Test 8	2:1	40	20.87	12.8	10	44	2.541
Test 10	2:1	40	20.99	11.9	16	34	2.150
Test 11	2:1	40	21.99	10.3	79	40	5.337
Test 12	2:1	40	21.65	11.1	56	35	4.062
Test 13	2:1	40	19.60	6.5	239	28	11.727
Test 14	2:1	40	21.04	7.9	114	43	6.780

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	3:1	10	21.09	12.2	56	30	9.078
Test 2	3:1	10	20.37	10.1	89	37	14.151
Test 3	3:1	10	18.24	7.3	70	37	12.198
Test 5	3:1	10	19.83	7.2	68	47	12.466
Test 6	3:1	10	20.41	8	163	32	23.569
Test 7	3:1	10	21.74	9.8	145	29	20.270
Test 8	3:1	10	20.87	12.8	10	44	4.257
Test 10	3:1	10	20.99	11.9	16	34	4.159
Test 11	3:1	10	21.99	10.3	79	40	12.642
Test 12	3:1	10	21.65	11.1	56	35	9.349
Test 13	3:1	10	19.60	6.5	239	28	34.116
Test 14	3:1	10	21.04	7.9	114	43	17.751
Test 1	3:1	20	21.09	12.2	56	30	7.133
Test 2	3:1	20	20.37	10.1	89	37	11.071
Test 3	3:1	20	18.24	7.3	70	37	9.809
Test 5	3:1	20	19.83	7.2	68	47	10.109
Test 6	3:1	20	20.41	8	163	32	17.947
Test 7	3:1	20	21.74	9.8	145	29	15.260
Test 8	3:1	20	20.87	12.8	10	44	3.898
Test 10	3:1	20	20.99	11.9	16	34	3.593
Test 11	3:1	20	21.99	10.3	79	40	9.893
Test 12	3:1	20	21.65	11.1	56	35	7.399
Test 13	3:1	20	19.60	6.5	239	28	25.907
Test 14	3:1	20	21.04	7.9	114	43	13.799
Test 1	3:1	30	21.09	12.2	56	30	6.349
Test 2	3:1	30	20.37	10.1	89	37	9.190
Test 3	3:1	30	18.24	7.3	70	37	8.262
Test 5	3:1	30	19.83	7.2	68	47	8.669
Test 6	3:1	30	20.41	8	163	32	14.485
Test 7	3:1	30	21.74	9.8	145	29	12.275
Test 8	3:1	30	20.87	12.8	10	44	3.719
Test 10	3:1	30	20.99	11.9	16	34	3.276
Test 11	3:1	30	21.99	10.3	79	40	8.287
Test 12	3:1	30	21.65	11.1	56	35	6.255
Test 13	3:1	30	19.60	6.5	239	28	20.722
Test 14	3:1	30	21.04	7.9	114	43	11.425

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	3:1	40	21.09	12.2	56	30	5.177
Test 2	3:1	40	20.37	10.1	89	37	7.894
Test 3	3:1	40	18.24	7.3	70	37	7.169
Test 5	3:1	40	19.83	7.2	68	47	7.672
Test 6	3:1	40	20.41	8	163	32	12.101
Test 7	3:1	40	21.74	9.8	145	29	10.247
Test 8	3:1	40	20.87	12.8	10	44	3.596
Test 10	3:1	40	20.99	11.9	16	34	3.060
Test 11	3:1	40	21.99	10.3	79	40	7.200
Test 12	3:1	40	21.65	11.1	56	35	5.478
Test 13	3:1	40	19.60	6.5	239	28	17.119
Test 14	3:1	40	21.04	7.9	114	43	9.802

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	4:1	10	21.09	12.2	56	30	12.501
Test 2	4:1	10	20.37	10.1	89	37	19.579
Test 3	4:1	10	18.24	7.3	70	37	16.876
Test 5	4:1	10	19.83	7.2	68	47	17.058
Test 6	4:1	10	20.41	8	163	32	32.924
Test 7	4:1	10	21.74	9.8	145	29	28.255
Test 8	4:1	10	20.87	12.8	10	44	3.352
Test 10	4:1	10	20.99	11.9	16	34	5.532
Test 11	4:1	10	21.99	10.3	79	40	17.372
Test 12	4:1	10	21.65	11.1	56	35	12.815
Test 13	4:1	10	19.60	6.5	239	28	47.902
Test 14	4:1	10	21.04	7.9	114	43	24.545
Test 1	4:1	20	21.09	12.2	56	30	9.732
Test 2	4:1	20	20.37	10.1	89	37	15.160
Test 3	4:1	20	18.24	7.3	70	37	13.442
Test 5	4:1	20	19.83	7.2	68	47	13.736
Test 6	4:1	20	20.41	8	163	32	24.758
Test 7	4:1	20	21.74	9.8	145	29	21.012
Test 8	4:1	20	20.87	12.8	10	44	5.135
Test 10	4:1	20	20.99	11.9	16	34	4.792
Test 11	4:1	20	21.99	10.3	79	40	13.474
Test 12	4:1	20	21.65	11.1	56	35	10.059
Test 13	4:1	20	19.60	6.5	239	28	35.891
Test 14	4:1	20	21.04	7.9	114	43	18.884
Test 1	4:1	30	21.09	12.2	56	30	8.001
Test 2	4:1	30	20.37	10.1	89	37	12.343
Test 3	4:1	30	18.24	7.3	70	37	11.102
Test 5	4:1	30	19.83	7.2	68	47	11.586
Test 6	4:1	30	20.41	8	163	32	19.554
Test 7	4:1	30	21.74	9.8	145	29	16.757
Test 8	4:1	30	20.87	12.8	10	44	4.885
Test 10	4:1	30	20.99	11.9	16	34	4.331
Test 11	4:1	30	21.99	10.3	79	40	11.093
Test 12	4:1	30	21.65	11.1	56	35	8.363
Test 13	4:1	30	19.60	6.5	239	28	28.057
Test 14	4:1	30	21.04	7.9	114	43	15.340

Kinston Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	4:1	40	21.09	12.2	56	30	6.887
Test 2	4:1	40	20.37	10.1	89	37	10.521
Test 3	4:1	40	18.24	7.3	70	37	9.555
Test 5	4:1	40	19.83	7.2	68	47	10.185
Test 6	4:1	40	20.41	8	163	32	16.194
Test 7	4:1	40	21.74	9.8	145	29	13.701
Test 8	4:1	40	20.87	12.8	10	44	4.717
Test 10	4:1	40	20.99	11.9	16	34	4.030
Test 11	4:1	40	21.99	10.3	79	40	9.570
Test 12	4:1	40	21.65	11.1	56	35	7.274
Test 13	4:1	40	19.60	6.5	239	28	22.966
Test 14	4:1	40	21.04	7.9	114	43	13.060

Brunswick Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	1:1	10	15.61	2.14	0.2	38.5	1.336
Test 2	1:1	10	15.32	2.16	0.01	37	1.252
Test 3	1:1	10	17.55	15.83	0.16	32.7	1.094
Test 4	1:1	10	16.21	16.25	0.01	38.4	1.326
Test 5	1:1	10	16.13	6.89	0.22	44	1.627
Test 6	1:1	10	15.65	7.05	0.26	32.8	1.089
Test 7	1:1	10	17.02	7.65	0.47	35.4	1.226
Test 8	1:1	10	18.72	17.57	0.24	41	1.497
Test 9	1:1	10	16.47	2.34	0.33	34	1.151
Test 1	1:1	20	15.61	2.14	0.2	38.5	1.185
Test 2	1:1	20	15.32	2.16	0.01	37	1.110
Test 3	1:1	20	17.55	15.83	0.16	32.7	0.960
Test 4	1:1	20	16.21	16.25	0.01	38.4	1.176
Test 5	1:1	20	16.13	6.89	0.22	44	1.441
Test 6	1:1	20	15.65	7.05	0.26	32.8	0.964
Test 7	1:1	20	17.02	7.65	0.47	35.4	1.081
Test 8	1:1	20	18.72	17.57	0.24	41	1.320
Test 9	1:1	20	16.47	2.34	0.33	34	1.015
Test 1	1:1	30	15.61	2.14	0.2	38.5	1.180
Test 2	1:1	30	15.32	2.16	0.01	37	1.108
Test 3	1:1	30	17.55	15.83	0.16	32.7	0.959
Test 4	1:1	30	16.21	16.25	0.01	38.4	1.171
Test 5	1:1	30	16.13	6.89	0.22	44	1.433
Test 6	1:1	30	15.65	7.05	0.26	32.8	0.959
Test 7	1:1	30	17.02	7.65	0.47	35.4	1.070
Test 8	1:1	30	18.72	17.57	0.24	41	1.306
Test 9	1:1	30	16.47	2.34	0.33	34	1.008
Test 1	1:1	40	15.61	2.14	0.2	38.5	1.187
Test 2	1:1	40	15.32	2.16	0.01	37	1.116
Test 3	1:1	40	17.55	15.83	0.16	32.7	0.962
Test 4	1:1	40	16.21	16.25	0.01	38.4	1.178
Test 5	1:1	40	16.13	6.89	0.22	44	1.440
Test 6	1:1	40	15.65	7.05	0.26	32.8	0.964
Test 7	1:1	40	17.02	7.65	0.47	35.4	1.072
Test 8	1:1	40	18.72	17.57	0.24	41	1.308
Test 9	1:1	40	16.47	2.34	0.33	34	1.011

Brunswick Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	2:1	10	15.61	2.14	0.2	38.5	1.783
Test 2	2:1	10	15.32	2.16	0.01	37	1.666
Test 3	2:1	10	17.55	15.83	0.16	32.7	1.461
Test 4	2:1	10	16.21	16.25	0.01	38.4	1.766
Test 5	2:1	10	16.13	6.89	0.22	44	2.171
Test 6	2:1	10	15.65	7.05	0.26	32.8	1.455
Test 7	2:1	10	17.02	7.65	0.47	35.4	1.640
Test 8	2:1	10	18.72	17.57	0.24	41	1.996
Test 9	2:1	10	16.47	2.34	0.33	34	1.539
Test 1	2:1	20	15.61	2.14	0.2	38.5	1.707
Test 2	2:1	20	15.32	2.16	0.01	37	1.599
Test 3	2:1	20	17.55	15.83	0.16	32.7	1.393
Test 4	2:1	20	16.21	16.25	0.01	38.4	1.692
Test 5	2:1	20	16.13	6.89	0.22	44	2.077
Test 6	2:1	20	15.65	7.05	0.26	32.8	1.391
Test 7	2:1	20	17.02	7.65	0.47	35.4	1.560
Test 8	2:1	20	18.72	17.57	0.24	41	1.899
Test 9	2:1	20	16.47	2.34	0.33	34	1.468
Test 1	2:1	30	15.61	2.14	0.2	38.5	1.720
Test 2	2:1	30	15.32	2.16	0.01	37	1.615
Test 3	2:1	30	17.55	15.83	0.16	32.7	1.398
Test 4	2:1	30	16.21	16.25	0.01	38.4	1.706
Test 5	2:1	30	16.13	6.89	0.22	44	2.090
Test 6	2:1	30	15.65	7.05	0.26	32.8	1.400
Test 7	2:1	30	17.02	7.65	0.47	35.4	1.563
Test 8	2:1	30	18.72	17.57	0.24	41	1.901
Test 9	2:1	30	16.47	2.34	0.33	34	1.473
Test 1	2:1	40	15.61	2.14	0.2	38.5	1.736
Test 2	2:1	40	15.32	2.16	0.01	37	1.633
Test 3	2:1	40	17.55	15.83	0.16	32.7	1.407
Test 4	2:1	40	16.21	16.25	0.01	38.4	1.722
Test 5	2:1	40	16.13	6.89	0.22	44	2.108
Test 6	2:1	40	15.65	7.05	0.26	32.8	1.411
Test 7	2:1	40	17.02	7.65	0.47	35.4	1.571
Test 8	2:1	40	18.72	17.57	0.24	41	1.911
Test 9	2:1	40	16.47	2.34	0.33	34	1.483

Brunswick Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	3:1	10	15.61	2.14	0.2	38.5	2.351
Test 2	3:1	10	15.32	2.16	0.01	37	2.193
Test 3	3:1	10	17.55	15.83	0.16	32.7	1.925
Test 4	3:1	10	16.21	16.25	0.01	38.4	2.325
Test 5	3:1	10	16.13	6.89	0.22	44	2.862
Test 6	3:1	10	15.65	7.05	0.26	32.8	1.921
Test 7	3:1	10	17.02	7.65	0.47	35.4	2.168
Test 8	3:1	10	18.72	17.57	0.24	41	2.631
Test 9	3:1	10	16.47	2.34	0.33	34	2.032
Test 1	3:1	20	15.61	2.14	0.2	38.5	2.371
Test 2	3:1	20	15.32	2.16	0.01	37	2.220
Test 3	3:1	20	17.55	15.83	0.16	32.7	1.932
Test 4	3:1	20	16.21	16.25	0.01	38.4	2.347
Test 5	3:1	20	16.13	6.89	0.22	44	2.883
Test 6	3:1	20	15.65	7.05	0.26	32.8	1.933
Test 7	3:1	20	17.02	7.65	0.47	35.4	2.167
Test 8	3:1	20	18.72	17.57	0.24	41	2.632
Test 9	3:1	20	16.47	2.34	0.33	34	2.038
Test 1	3:1	30	15.61	2.14	0.2	38.5	2.415
Test 2	3:1	30	15.32	2.16	0.01	37	2.270
Test 3	3:1	30	17.55	15.83	0.16	32.7	1.963
Test 4	3:1	30	16.21	16.25	0.01	38.4	2.396
Test 5	3:1	30	16.13	6.89	0.22	44	2.937
Test 6	3:1	30	15.65	7.05	0.26	32.8	1.968
Test 7	3:1	30	17.02	7.65	0.47	35.4	2.196
Test 8	3:1	30	18.72	17.57	0.24	41	2.668
Test 9	3:1	30	16.47	2.34	0.33	34	2.071
Test 1	3:1	40	15.61	2.14	0.2	38.5	2.447
Test 2	3:1	40	15.32	2.16	0.01	37	2.301
Test 3	3:1	40	17.55	15.83	0.16	32.7	1.982
Test 4	3:1	40	16.21	16.25	0.01	38.4	2.426
Test 5	3:1	40	16.13	6.89	0.22	44	2.971
Test 6	3:1	40	15.65	7.05	0.26	32.8	1.990
Test 7	3:1	40	17.02	7.65	0.47	35.4	2.214
Test 8	3:1	40	18.72	17.57	0.24	41	2.691
Test 9	3:1	40	16.47	2.34	0.33	34	2.091

Brunswick Test Soil Factor of Safety

Brunswick Test Soil Factor of Safety

Test	Slope	Height	Unit Weight	Water Content	Cohesion	Friction	Factor of Safety
Test 1	4:1	10	15.61	2.14	0.2	38.5	2.959
Test 2	4:1	10	15.32	2.16	0.01	37	2.756
Test 3	4:1	10	17.55	15.83	0.16	32.7	2.422
Test 4	4:1	10	16.21	16.25	0.01	38.4	2.921
Test 5	4:1	10	16.13	6.89	0.22	44	3.601
Test 6	4:1	10	15.65	7.05	0.26	32.8	2.420
Test 7	4:1	10	17.02	7.65	0.47	35.4	2.733
Test 8	4:1	10	18.72	17.57	0.24	41	3.310
Test 9	4:1	10	16.47	2.34	0.33	34	2.560
Test 1	4:1	20	15.61	2.14	0.2	38.5	3.081
Test 2	4:1	20	15.32	2.16	0.01	37	2.884
Test 3	4:1	20	17.55	15.83	0.16	32.7	2.508
Test 4	4:1	20	16.21	16.25	0.01	38.4	3.048
Test 5	4:1	20	16.13	6.89	0.22	44	3.746
Test 6	4:1	20	15.65	7.05	0.26	32.8	2.514
Test 7	4:1	20	17.02	7.65	0.47	35.4	2.817
Test 8	4:1	20	18.72	17.57	0.24	41	3.415
Test 9	4:1	20	16.47	2.34	0.33	34	2.650
Test 1	4:1	30	15.61	2.14	0.2	38.5	3.158
Test 2	4:1	30	15.32	2.16	0.01	37	2.965
Test 3	4:1	30	17.55	15.83	0.16	32.7	2.562
Test 4	4:1	30	16.21	16.25	0.01	38.4	3.128
Test 5	4:1	30	16.13	6.89	0.22	44	3.837
Test 6	4:1	30	15.65	7.05	0.26	32.8	2.572
Test 7	4:1	30	17.02	7.65	0.47	35.4	2.869
Test 8	4:1	30	18.72	17.57	0.24	41	3.481
Test 9	4:1	30	16.47	2.34	0.33	34	2.705
Test 1	4:1	40	15.61	2.14	0.2	38.5	3.200
Test 2	4:1	40	15.32	2.16	0.01	37	3.010
Test 3	4:1	40	17.55	15.83	0.16	32.7	2.591
Test 4	4:1	40	16.21	16.25	0.01	38.4	3.172
Test 5	4:1	40	16.13	6.89	0.22	44	3.886
Test 6	4:1	40	15.65	7.05	0.26	32.8	2.603
Test 7	4:1	40	17.02	7.65	0.47	35.4	2.895
Test 8	4:1	40	18.72	17.57	0.24	41	3.516
Test 9	4:1	40	16.47	2.34	0.33	34	2.734