QUALITY CONTROL TESTING FOR GRANULAR BACKFILL MATERIALS

By

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CHAPTER 1

INTRODUCTION

Quality control in any engineering project is essential to proper placement of materials and satisfactory function of the structure being built. In the case of granular backfill material for bridge approach embankments and abutments, quality control procedures have fallen short. This has lead to unequal settlement between the bridge approach embankment and the bridge structure, causing a bump at the end of the bridge. These unequal settlements cause unsafe driving conditions as well as expensive repairs. There are many different reasons for deformations that cause such settlement, including lateral earth pressure on abutment walls, settlement of the embankment soil, and lateral movement of embankment soil as settlement occurs.

Description of Research Project

This research project addressed one of these issues. Settlement of the embankment soil is a large contributor to the bump at the end of bridges. Settlement is more likely to occur when the granular fill material used for embankments is not compacted properly. To evaluate this aspect of the settlement issue, six different granular soil samples were tested using current Oklahoma Department of Transportation (ODOT) quality control specifications, as well as an alternative approach. Physical properties were determined for each sample for classification purposes, including grain size distribution, fine aggregate angularity, and estimated values of specific gravity. Engineering properties for each sample were used to evaluate the effectiveness of compaction using current Standard Proctor compaction methods, which is currently used by ODOT, and an alternative approach using relative density values. Appropriate density values taken from Standard Proctor compaction and relative density results were used for placement conditions in direct shear tests. The modulus results from direct shear testing will be used to estimate settlement in an arbitrary approach embankment. The two different methods of quality control were compared based upon direct shear data, settlement calculations, and correlations between Standard Proctor compaction and relative density values.

Purpose of Thesis

The purpose of this thesis is to compare current ODOT specifications for quality control of granular backfill material with an alternative approach using relative density. ODOT currently specifies the use of Standard Proctor compaction techniques for placement conditions of granular materials in bridge approach embankments. The research project used for this thesis compared these two methods and provided information to demonstrate the need for a change in the specifications. A series of steps were taken to compare the usefulness of each method for quality assurance. First, a review of literature was performed to get information pertaining to the subject. This included basic information regarding each method, information about correlations between the two methods, as well as specifications for individual tests used for classification and testing purposes. Next, six soil samples were gathered and tested for grain size distribution. The sieve analysis data was compared to grain size distribution requirements for soils classified as granular backfill (ODOT 703.05) and select borrow (ODOT 703.01). Standard Proctor compaction tests were run on each sample to determine the optimum moisture content and maximum dry density. Then, maximum and minimum index densities were determined for each sample. These parameters, combined with a percentage of the maximum dry density determined from Standard Proctor compaction tests, were used to calculate the relative density of each sample. Direct shear tests were conducted based on the densities from the relative density testing and Standard Proctor compaction testing. Each sample was placed in a direct shear apparatus at 95% Standard Proctor density and 75% relative density (85% for sample 2) and results were compared. Also, the initial tangent modulus determined from direct shear testing was used for each placement condition to estimate settlement in an assumed embankment case study. The results from maximum densities, shear strength tests, and settlement estimates were used to compare each method of quality control for granular backfill placement.

CHAPTER 2

BACKGROUND INFORMATION

At the end of many bridges a bump caused by differential settlement between the bridge and approach embankment typically occurs. This is because the bridge is placed on deep foundations, while the backfill material for the approach embankment is placed on existing ground. The backfill material in the approach embankment settles, but the bridge deck does not, creating the bump. Since the bump can be hazardous to road users and cause wear on the bridge, the problem must be repaired, typically on an ongoing basis, which can be costly. If the backfill is compacted to a proper density, the settlement can be minimized which would reduce such problems.

Settlement of Backfill Material

Compaction of fill material is a fundamental aspect of Geotechnical Engineering. Soils used for fills must be compacted to a firm state in order to carry the loads that will be applied by the structures placed upon them. The primary purpose of compaction is to prevent densification of the soils resulting in a decreased volume under loading (Rollings). Soils are compacted to a specified density to prevent excessive settlement in the future. Densification due to loading after the structure is in place leads to defects, such as the bump-at-the-end-of-bridge. Inadequately compacted backfill material is a major cause of structural problems in many bridges and highways (Schwidder). Another reason soils used for backfill are compacted is to ensure that the soils have the required engineering properties to serve their particular purpose. The properties of a compacted fill that are most important to design engineers are compressibility, shear strength, and permeability (Lacroix and Horn). Embankments are designed to carry specified loads based on the strength of the backfill material at a specific density. The embankments are also supposed to drain freely to prevent a build up of pore water pressure. Properties such as shear strength, permeability, and compressibility are affected by density and affect the design of the structure. Therefore, if the soil is improperly placed at a density less than the specified density, problems can and do occur relating to densification after the structure is in place and as repetitive loading from traffic is applied to the structure. A noticeable problem to road users is the bump that results from approach embankment settlement due to improperly compacted backfill material, while the bridge structure remains in place because it is resting on deep foundations. This is not only destructive to the vehicles crossing the bridge, but to the bridge superstructure itself. Impact loads caused by the bump can cause wear on the bridge deck and the rest of the structure that is typically unaccounted for in design. The current procedures for quality control of granular backfill materials used by ODOT are not appropriate. ODOT specifies Standard Proctor densities for granular materials, which generally results in density values lower than required for minimizing settlement.

Backfill Materials

The two types of soil used to define "backfill materials" in this project were classified by ODOT Standard Specifications for Highway Construction as "Granular Backfill" and "Select Borrow". Each soil type has required maximum and minimum values of grain size distributions. The properties of Granular Backfill are specified in ODOT 703.05, and the properties for Select Borrow are specified in ODOT 705.01. According to ODOT specifications, granular backfill shall be free from organic material and shale or other soft, poor durability particles, and conform to gradation requirements shown in Appendix A. Any material specified as select borrow shall pass a 3 inch sieve and be classified using maximum and minimum sieve analysis values for AASHTO Soil Classification System group classifications of A-1, A-2-4, and A-3. AASHTO group A-1 was used to represent Select Borrow for this research project and has gradation requirements shown in Appendix A.

Standard Proctor Compaction of Granular Materials

The moisture density test, also known as the Standard Proctor Compaction test, AASHTO T99, has worked well for quality control of fine-grained soils and granular soils with a substantial amount of fines. It is based on impact compaction methods, where compaction results from applying an impact force to the soil. This method is appropriate for soils that have cohesion between particles which provides confinement, but does not work well for cohesionless soils such as granular materials used for backfill in bridge approach embankments (Felt). The Proctor test employs a form of dynamic compaction, using a hammer that is smaller in size than the confining mold. This poses a problem with cohesionless sands because the soil particles displace when impacted by the hammer. As the soil directly under the hammer is compacted, the surround soil is actually forced into a loose state due to lack of confinement around the hammer (Felt). Standard Proctor Compaction testing also does not work for coarse-graded crushed stone and gravels having angular stability because of the lack of horizontal movement in the confining mold. Insufficient movement of particles does not allow the soil to fill voids. Another reason Standard Proctor Compaction has limited use for acceptance criteria for granular materials is the resulting moisture density curves. Compaction curves for granular materials do not always yield unique curves which poses a problem when determining the maximum dry density and optimum moisture content. This results from the aforementioned limitations as well as the variation in water content throughout the sample during compaction. Since the materials are free draining, the water added to the sample during testing drains to the bottom of the mold, resulting in layers with different water contents. In fact, if the mold is not sealed at the bottom, the water will drain out of the mold. This results in inconsistent data. One can not specify quality control criteria from data that is not well-defined. In addition, the maximum density recorded in the laboratory is not as great as that achieved in the field using vibratory compaction methods.

In the 1958 Symposium on Application of Soil Testing In Highway Design and Construction, it was noted that for a given sample of sand the Standard Proctor Compaction test yielded a moisture density curve without a well-defined peak with a maximum dry density of 111 pcf, but the same sand when compacted by vibration yielded a maximum density of 122 pcf. Similarly, a sample of crushed limestone yielded a Standard Proctor Compaction dry density of 125 pcf, and a maximum dry density of 132 pcf when compacted by vibratory methods (Felt). A typical moisture density curve for sand is shown in Figure 1. Due to the limitations of Standard Proctor Compaction testing for granular materials, this method should not be used for quality control of these materials. If this method is used to specify acceptance criteria for in-place dry density and moisture content for granular backfill material, problems such as excessive settlement could occur.

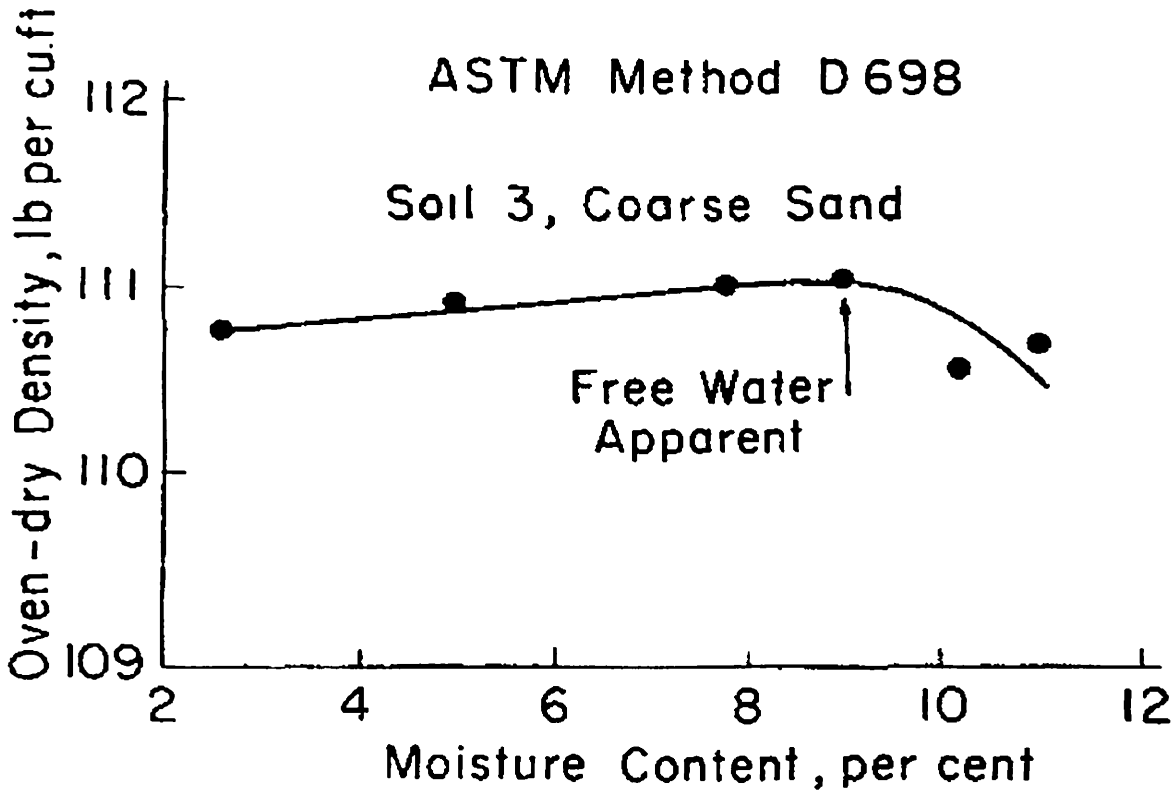

Figure 1. Example of Poorly Defined Moisture Density

Curve for Granular Soil (Felt).

Alternative to Standard Proctor Compaction

There are two common methods for testing compaction for soils, the aforementioned Standard Proctor compaction test and the relative density test. The relative density test is based on vibratory compaction methods rather than impact compaction. Vibratory techniques are proven to work better for granular material than compaction methods using impact forces. Generally, the Standard Proctor compaction test is used for cohesive soils and the relative density test is used for cohesionless soils

(Townsend). "Relative density establishes the significant state of the grain structure of granular soils" that controls the behavior of the soil in construction applications including soil embankments and natural deposits (Burmister). According to the Corps of Engineers, the relative density test is more applicable for soils having less than about 5% by weight passing the No. 200 sieve (Townsend). ASTM Standard Test Method for Relative Density of Cohesionless Soils (D-4253 & 4254) suggests that soils having more than 12% fines use Standard Proctor compaction methods, and soils with less than this amount of fines should be tested based on relative density (Townsend). Also, the U.S. Bureau of Reclamation separates soils suitable for vibratory compaction into two different categories: (a) those suitable for vibratory compaction and (b) those borderline for vibratory compaction. The borderline soils contain more than 12% fines (Townsend).

Relative density can be used to correlate important properties such as shear strength and settlement characteristics of granular materials. It is also a useful parameter for liquefaction studies and seismic studies for sand and gravel embankments in earthquake and other vibrational conditions (Tavenas). One of the criticisms of relative density as a control parameter is the repeatability of the test. However, studies have shown that "variations associated with minimum and maximum density tests are about the same or less than those associated with the impact type compaction test (Holtz)." Another reason relative density is useful as acceptance criteria for granular material is the correlations between relative density and field testing techniques. Relative density is the "most commonly measured *in situ*" soil parameter and "controls a majority of designs involving cohesionless soils (Tavenas)." It can be obtained directly and indirectly from in place determinations of dry unit weights and empirical relationships between relative density and measurements reflecting compactness of the soil (Leary and Woodward). Some of the field test methods used to evaluate relative density include nuclear moisture-density gauge, Dynamic Cone Penetrometer, Standard Penetration Test, Static Cone Penetration Test, and a new test called the PANDA Cone Penetration Test.

CHAPTER 3

MATERIALS AND TEST PROCEDURES

In order to compare Proctor compaction density values and Relative Density values for granular materials, six soil samples were tested. Physical and engineering properties were determined for each sample so that they could be classified and compared. Basic properties such as grain size distribution, percent passing U.S. No. 200 sieve, and fine aggregate angularity were determined for each sample. Engineering properties measured for comparison were maximum dry density and optimum water content using the Standard Proctor compaction test procedure, relative density which included maximum and minimum index densities, and angle of internal friction using direct shear test procedures. Test specimens were obtained from larger samples using sample size preparation in accordance with AASHTO T-248, which specifies using a mechanical separator or quartering method to reduce soil samples to a workable size.

Soil Samples

Physical properties of soil samples were measured in order to classify them properly. The physical properties used to classify the soil samples in this project were specific gravity, grain size distribution, %-200, and uncompacted void content. The specific gravity of soils were accurately estimated based on soil type. The grain size distribution was determined in accordance with AASHTO T-27, %-200 was determined according to AASHTO T-11, and the uncompacted void content was determined according to AASHTO T-304, Method C. Method C specifies an as-received gradation of the sample passed through the No. 4 sieve.

Sample 1 was a light brown, coarse, concrete sand, classified as A-1-b using the AASHTO classification and SP (poorly graded sand) using Unified Soil Classification System (USCS) parameters. The sample had 0.6 % passing the # 200 sieve, while 96.5 % of the sample was sand and 2.9 % was gravel. The particles in this sample were rounded, having a maximum particle size of 4.8 mm. The sample had an uncompacted void content of 36.4 % using an assumed specific gravity of 2.65. The results of the sieve analysis are shown in Appendix A.

Sample 2 was a well graded sand with silt and classified as A-2-4 and SW-SM using AASHTO and USCS respectively. The common name for the material was screenings. The sample had angular coarse particles with a maximum size of 4.8 mm, and 20.8% of the material passed the #200 sieve. The uncompacted void content was 37.8 % using an assumed specific gravity of 2.70. Sieve analysis results for Sample 2 are shown in Appendix A.

Sample 3 was a poorly graded fine gravel, which was classified as A-1-a and GP using AASHTO and USCS methods, respectively. This sample was different shades of brown, containing small amounts of organic matter such as tree bark and small crustacean shells. Particles were smooth with rounded edges. The largest particles were 3/8", or approximately 10 mm, and 0.4 % of the material passed the #200 sieve. AASHTO T-56, Method C was used for uncompacted void content instead of AASHTO T-304, because T-304 is designated for fine aggregates. T-56 is used for coarser aggregates like Sample

3. Using an assumed specific gravity of 2.50, sample number 3 had an uncompacted void content of 45.3 %. Results of the sieve analysis are shown in Appendix A.

Sample 4 was a mix of the concrete sand and the fine river rock. It contained 70% of Sample 1 and 30% of Sample 3 by weight. The resulting mixed sample was a poorly graded coarse sand with gravel and classified as A-3 and SP using AASHTO and USCS methods respectively. This sample had a maximum particle size of 3/8", or approximately 10 mm, and 1.2 % passing the #200 sieve. Uncompacted void content of this sample was determined to be 31.8 % using an assumed specific gravity of 2.65 and specifications from AASHTO T-56, Method C. AASHTO T-304 was not used because it is meant for fine aggregate, whereas AASHTO T-56 is designated for coarse aggregates. Results of the sieve analysis are shown in Appendix A.

Sample 5 was also a mix of two previous samples, 1 and 2. Sample 5 contained 80% of Sample 1 and 20% of Sample 2 by weight. The sample was classified as poorly graded sand (SP) by USCS methods and A-3 by AASHTO criteria. Sample 5 had a maximum particle size of 4.8 mm and 5.0% passing the #200 sieve. The uncompacted void content was 35.3% using an assumed specific gravity of 2.68. Sieve analysis results for Sample 5 are shown in Appendix A.

Sample 6 was coarse sand provided by the University of Oklahoma (OU). It was similar to Sample 1, and classified as poorly graded sand (SP) and A-1-b using USCS and AASHTO procedures, respectively. This sample had 0.4% of the particles passing the #200 sieve, 98.5% of the sample was sand, and 1.1% was gravel. Using an assumed specific gravity of 2.65, the uncompacted void content was determined to be 39.2%. The results of sieve analysis for Sample 6 are shown in Appendix A.

Other parameters were determined from the sieve analysis results for each specimen. Properties such as D60, D30, and D10 were taken from the grain size distribution of each sample to determine uniformity coefficients and coefficient of curvature. D60 represents the particle size corresponding to 60% of the sample being smaller than that particle size. Similarly, D30 represents 30% of the sample being smaller than that particle size and D10 represents 10% of the sample being smaller than that particle size. These parameters along with USCS classification can be used to determine whether or not the soil is well-graded or poorly graded. Properties for each soil sample are summarized in Table 1.

Soil Properties

	Gs	D10 (mm)	D30 (mm)	D60 (mm)	Cu	Cc	-200%	Ur *
Sample 1 (concrete sand)	2.65	0.212	0.392	0.913	4.3	0.8	0.6%	36.4%
Sample 2 (screenings)	2.7	0.06	0.266	1.18	19.7	1.0	20.8%	37.8%
Sample 3 (fine river rock)	2.5	5.11	6.1	7.5	1.5	1.0	0.4%	45.3%
Sample 4 (70%1/30%3)	2.65	0.285	0.727	3.53	0.5	12.4	1.2%	31.8%
Sample 5 (80%1/20%2)	2.68	0.195	0.433	1.07	5.5	0.9	5.0%	35.3%
Sample 6 (OU sand)	2.65	0.215	0.384	0.778	3.6	0.9	1.2%	39.2%

Table 1. Soil Properties Used for Soil Classification

* See Appendix B for calculations

Compaction and Strength Properties

Once a soil has been classified and determined suitable for use as a backfill material, the engineering properties were determined for proper design of the foundation or embankment. Properties such as dry density and moisture content influence the strength of the soil. Therefore, soil samples were tested to determine the placement conditions used for construction. As stated previously, data from the Standard Proctor Compaction test typically were not considered adequate for use as quality control for granular backfill materials. The appropriate method for establishing acceptance criteria is Relative Density. Results from Standard Proctor Compaction tests, Relative Density tests, and Direct Shear tests for each soil sample were measured to compare the effectiveness of Proctor density and relative density as quality control criteria for granular materials.

Standard Proctor Compaction Test

The Standard Proctor Compaction test method was used to determine the relationship between moisture content and dry density for a each soil specimen. Testing procedures followed AASHTO T-99, Method A specifications. Method A used a 5.5 lb. hammer dropped 25 times from a height of 12 inches in three equal lifts. A 4 inch diameter mold was used for this method. The mold and compacted soil was weighed to determine the moist unit weight of the soil, and a sample of the compacted soil was weighed and then dried and reweighed to determine the water content of the sample.

Knowing water content, ω, and the moist unit weight of the sample, the dry unit weight of the sample was determined using the equation:

$$\gamma(dry) = \gamma(wet) \div (1+\omega) \qquad \text{Eq. 1}$$

A plot of dry unit weight versus moisture content produced a specific curve for each type of soil, and the peak of the curve defined the maximum dry density and optimum water content of that particular soil sample. Generally, some percentage of the maximum dry density is specified for field compaction criteria, and a corresponding range of acceptable water contents is allowed. This technique works well for fine-grained soils, but granular materials don't always produce distinct curves and the nature of these types of soils creates low values of maximum density. The maximum dry density and optimum water content were determined for each soil sample and are listed in Table 2. The highest maximum dry density was produced by Sample 2, screenings, which had the highest percentage of fines. The fine river rock in Sample 3 produced the lowest maximum dry density and had the lowest amount of fines present. Moisture density curves for each sample are shown in Appendix C.

Standard Proctor Compaction Data

	γ dmax	ω opt	γ @ ω=0%	% - # 200
Sample 1 (concrete sand)	117.0	9.7%	118.2	0.6%
Sample 2 (screenings)	137.9	8.4%	137.9	20.8%
Sample 3 (fine river rock)	104.8	13.5%	99.4	0.4%
Sample 4 (70%1/30%3)	123.5	8.2%	123.9	1.2%
Sample 5 (80%1/20%2)	122.0	8.9%	121.6	5.0%
Sample 6 (OU sand)	111.1	11.4%	113.0	1.2%

Table 2. Standard Proctor Compaction Test Results

Relative Density test

The Relative Density test differed from the Standard Proctor Compaction test because it is based on vibratory compaction techniques rather than impact compaction. Instead of using a falling hammer to exert energy, a sample was placed in a mold and compacted using a vibratory table. Relative density of a soil specimen is a comparison of the in-place density of the sample to the maximum and minimum index densities of the sample. Minimum and maximum index densities were calculated in accordance with ASTM D-4254 method A and ASTM D-4253 method 1A, respectively. This comparison, expressed as a percentage, was calculated by using densities or void ratios as follows:

$$Dr = \frac{\gamma\ max\ (\gamma\ field - \gamma\ min)}{\gamma\ field\ (\gamma\ max - \gamma min)} \cdot 100 \qquad \text{Eq. 2}$$

or

$$Dr = \frac{e\ max - e\ field}{e\ max - e\ min} \cdot 100 \qquad \text{Eq. 3}$$

where γ max is equal to the maximum index dry density of the soil corresponding to the minimum void ratio, e min, and γ min is the minimum index dry density of the soil corresponding to the maximum void ratio of the soil, e max. Relative density is measured from a scale of 0-100%; 0% pertaining to a very loose state of compaction, and 100% pertaining to a very high state of compaction. Table 3 shows the general level of density corresponding to percentages of relative density.

Relative Density (%)	Description of Soil Deposit
0-15	Very loose
15-50	Loose
50-70	Medium
70-85	Dense
85-100	Very dense

Table 3. Description of Soils Based On Relative Density (Das)

For this project, values of "field" density were actually different percentages of Standard Proctor Compaction maximum dry density results for each sample. Relative density values were calculated for each sample at 100% of maximum Proctor density, 95% maximum Proctor density, and at Proctor densities corresponding to 0% water content. Resulting relative density values are shown in Table 4. In general, 95% and even 100% Standard Proctor maximum densities yielded low to mid-ranged levels of relative density, corresponding to loosely compacted soils (Das).

	Standard Proctor		Relative Density @ %Std Proctor *			Index Densities	
	γ dmax	γd @ ω=0%	Dr @ 95%	Dr @ 100%	Dr @ ω=0%	γ dmax	γ dmin
	(pcf)			(%)		(pcf)	
Sample 1 (concrete sand)	117.0	118.2	8.1%	51.4%	59.6%	125.3	110.0
Sample 2 (screenings)	137.9	136.5	73.0%	91.7%	88.0%	141.2	109.6
Sample 3 (fine river rock)	104.8	99.4	43.7%	100.0%	41.6%	104.1	96.3
Sample 4 (70%1/30%3)	123.5	123.9	4.6%	55.7%	58.9%	129.4	116.8
Sample 5 (80%1/20%2)	122.0	121.6	17.6%	48.7%	46.8%	133.6	112.7
Sample 6 (OU sand)	111.1	113.0	8.5%	45.5%	57.3%	120.5	104.3

Table 4. Relative Density Values Corresponding to Standard Proctor Data

* See Appendix D for calculations

Direct Shear test

Direct shear test data were used to measure the angle of internal friction and initial tangent modulus of each soil sample, Φ' and Es respectively. This parameter defined the shear strength of each soil. In addition, results from direct shear data were used to calculate settlements in compacted fills. Direct shear testing procedures were in accordance with AASHTO T-236 specifications. Samples used were placed at specific densities corresponding to 95% Standard Proctor maximum dry density and 75% relative density values, with the exception of sample 2. The screenings, sample 2, were tested at 95% Standard Proctor maximum dry density and 85% relative density, due to the higher values of relative density calculated from Standard Proctor results. In addition, the relatively large grain sizes of the fine river rock, sample 3, resulted in incomplete direct shear data. Instead of failing the soil in shear, the apparatus crushed individual particles creating inaccurate data. Therefore, the direct shear data for the fine river rock sample was not included for evaluation. Each sample was tested under 15, 30, and 60 psi normal stress conditions. For each condition, plots were made of horizontal displacement versus shear stress. The peak of each curve corresponded to the shear stress at failure, and was then plotted versus the normal stress for that particular condition. The angle of internal friction was determined from the normal stress versus shear stress plots. The initial tangent modulus used in settlement calculations was equal to the slope of a line tangent to the linear portion of the 15 psi stress/strain curve for each placement condition. Plots of horizontal displacement versus shear stress and plots of normal stress versus shear stress are given for each sample at 75% relative density (85% for sample 2) and 95% Standard Proctor Density in Appendix E. Table 5 is a summary of the direct shear testing data.

Soil Parameters for Direct Shear Test at 95 % Standard Proctor Density

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
τ @ 15 psi	18 psi	32 psi	N/A	25 psi	17.8 psi	13 psi
τ @ 30 psi	32 psi	48 psi	N/A	42 psi	35.8 psi	23.5 psi
τ @ 60 psi	59 psi	83 psi	N/A	64 psi	65 psi	46.5 psi
Φ'	45.5	55.7°	N/A	49°	48.1°	38.1°
Es (psi)	1,000	1,050	N/A	2,200	1,400	1,000

Soil Parameters for Direct Shear Test at Dr = 75% (85% for Sample 2)

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
τ @ 15 psi	32.5 psi	34 psi	N/A	32 psi	25 psi	19.5 psi
τ @ 30 psi	42.5 psi	53 psi	N/A	55 psi	45 psi	31.2 psi
τ @ 60 psi	73 psi	88 psi	N/A	79 psi	75.6 psi	55 psi
Φ'	52.4°	57.3°	N/A	55.7°	52.9°	44.0°
Es (psi)	3,567	1,150	N/A	2,300	1,600	1,000

Table 5. Direct Shear Test Data

Chapter 4

Evaluation of Data

Each soil sample was classified using sieve analysis results, and then tested for maximum dry density and optimum water content using Standard Proctor Compaction methods. Then maximum and minimum index densities were determined using AASHTO specifications, and these parameters were used to calculate the relative density of each sample at 95% Standard Proctor compaction. Also, the density corresponding to a relative density of 75% was calculated for samples 1, 3, 4, 5, and 6. A relative density of 85% was used for Sample 2 because the relative density corresponding to 95% Proctor compaction was already 74%. Direct shear tests were run for each sample at 95% Proctor density and the densities corresponding to 75% or 85% relative density for each sample. The initial tangent modulus was determined from stress-strain relationships, which were then used for a case study comparing the difference in settlement between soils placed at 95% Proctor density and soils placed at densities corresponding to higher relative density. The following sections are evaluations of the data obtained.

Summary of Testing Data

Soil Classification Tests:

The first tests on each sample were used for classification purposes. These tests included sieve analysis, percent minus U.S. No. 200 sieve, and uncompacted void content. Results from the sieve analysis and percent minus U.S. No. 200 sieve showed that all of the samples except Sample 2 had less than 5 percent material passing the No. 200 sieve. Sample 2 had almost 21 percent fines, which affected the outcomes of Standard Proctor Compaction test results, relative density calculations, and the amount of settlement calculated in the case study. The second highest percentage of fines was found in Sample 5, which was a mix of Sample 2 and Sample 1. This sample had 5 percent material passing the No. 200 sieve; however it was not enough fines to affect Standard Proctor Compaction test results as in Sample 2. Table 1 shows important soil properties obtained from sieve analysis, percent minus U.S. No. 200 sieve, and uncompacted void content procedures. Sieve analysis results for each sample are shown in Appendix A, and uncompacted void content calculations are shown in Appendix B.

Standard Proctor Results:

Standard Proctor Compaction test maximum dry density results ranged from 111.1 pcf to 137.9 pcf and yielded water contents ranging from 8.2% to 13.5%. The higher dry densities corresponded to samples with higher percent fines. The highest dry density of 137.9 pcf came from Sample 2, which had 20.8 percent fines. Samples 4 and 5 had maximum dry densities of 123.5 and 122.0 pcf respectively, and 1.2% and 5.0% passing the U.S. No. 200 sieve, respectively. Sample 3 had the least amount of fine material as well as the lowest maximum dry density of all the samples. Table 2 shows the results of Standard Proctor Compaction tests as well as the percent passing the U.S. No. 200 sieve for each sample. Appendix C contains moisture density curves from Standard Proctor Compaction tests for each sample.

Relative Density Results:

For each sample, the maximum index density and minimum index density were determined using ASTM procedures. The maximum index density was higher than the maximum dry density determined using Standard Proctor Compaction data in all cases except Sample 3. Sample 3 had a maximum dry density of 104.8 pcf, while it yielded a maximum index density of 104.1 pcf. This may be due to the larger particle sizes present in the sample. The relative density of each sample was calculated using "field density" values corresponding to 95% of the maximum dry density calculated from Standard Proctor Compaction test results and maximum and minimum index density values. The values of relative density were low for all samples except Sample 2, which had a relative density of 74%. This value of relative density corresponds to that of dense sand, but the rest of the samples had relative density values corresponding to that of very loose to loose sands (Das). In fact, Samples 1, 4, and 6 had generated relative density values of 8.1%, 4.6%, and 8.5% respectively. All of these values fall in the range of a very loose soil deposit. After relative densities were calculated using 95% of Standard Proctor densities, density values were calculated corresponding to a relative density of 75%. The exception was Sample 2 because it had a relative density equal to 74% corresponding to 95% Proctor compaction. Therefore, a relative density of 85% was used to calculate a corresponding density. Table 6 contains relative density values for each sample corresponding to 95% Standard Proctor Compaction results, as well as maximum and minimum index densities and densities corresponding to relative densities equal to 75% (85% for Sample 2). Figure 2 contains a comparison of 95% Standard Proctor density and density values corresponding to relative density values equal to 75% (85% for Sample 2) for each sample. Calculations of maximum and minimum index densities for each sample are shown in Appendix D.

	Standard Proctor	Relative Density @ 95% Std Proctor	γ @ Dr = 75% (85% for Sample 2)	Index Densities	
	γ dmax	Dr @ 95%	γ @ Dr = 75%	γ dmax	γ dmin
	(pcf)	(%)	(pcf)	(pcf)	
Sample 1 (concrete sand)	117.0	8.1%	120.5	125.3	110
Sample 2 (screenings)	137.9	73.0%	135.3	141.2	109.6
Sample 3 (fine river rock)	104.8	43.7%	102.0	104.1	96.3
Sample 4 (70%1/30%3)	123.5	4.6%	126.0	129.4	116.8
Sample 5 (80%1/20%2)	122.0	17.6%	127.7	133.6	112.7
Sample 6 (OU sand)	111.1	8.5%	116.0	120.5	104.3

Table 6. Relative Density Data.

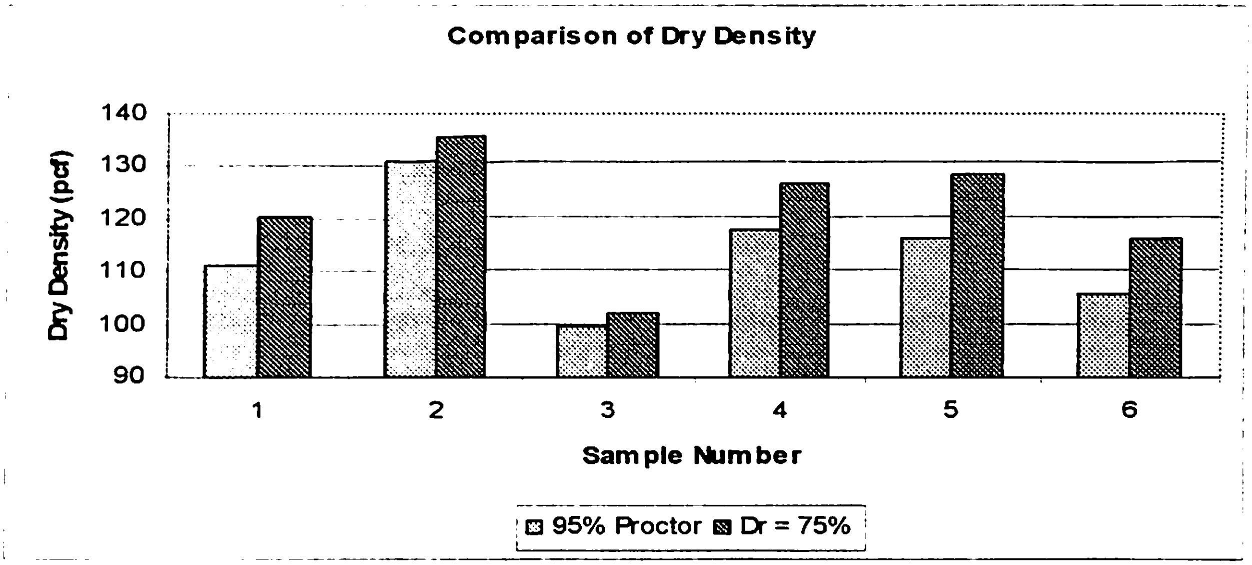

Figure 2. Comparison of Dry Densities Based on Standard Proctor Data and Relative Density Data.

Direct Shear Data:

Direct shear tests were run to determine shear strength parameters for each sample. The angle of internal friction and initial tangent modulus were estimated for all of the samples except Sample 3. Sample 3 did not produce usable data when tested using direct shear methods. This is most likely due to the large particle sizes in contrast with the relatively small volume of the testing mold. Instead of shearing the soil, the test crushed the soil particles, which produced inaccurate results. Therefore, Sample 3 was not used for shear data evaluation. Each sample was tested at densities corresponding to 75% relative density (85% for Sample 2) and 95% Standard Proctor density. Each sample was tested under 15 psi, 30 psi, and 60 psi normal stress conditions for each density. The initial tangent modulus was calculated by determining the slope of the line tangent to the linear portion of the stress versus horizontal displacement curve for the 15 psi normal stress condition. The initial tangent modulus is equal to the slope of that line. Prior to calculating the slope, the horizontal displacement values were divided by the 2 inch length of the direct shear box, converting horizontal displacement to strain. Table 7 and Table 8 show the angle of internal friction and initial tangent modulus for each sample at values of density corresponding to 95% Standard Proctor density and a relative density equal to 75% (85% for Sample 2), respectively.

Soil Parameters for Direct Shear Test at 95 % Standard Proctor Density

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
τ @ 15 psi	18 psi	32 psi	N/A	25 psi	17.8 psi	13 psi
τ @ 30 psi	32 psi	48 psi	N/A	42 psi	35.8 psi	23.5 psi
τ @ 60 psi	59 psi	83 psi	N/A	64 psi	65 psi	46.5 psi
Φ'	45.5°	55.7	N/A	49°	48.1°	38.1°
Es (psi)	1,000	1,050	N/A	2,200	1,400	1,000

Table 7. Angle of Internal Friction and Initial Tangent Modulus

at 95% Standard Proctor Density

Soil Parameters for Direct Shear Test at Dr = 75% (85% for Sample 2)

	Sample 1	Sample 2	Sample 3	Sample 4	Sample 5	Sample 6
τ @ 15 psi	32.5 psi	34 psi	N/A	32 psi	25 psi	19.5 psi
τ @ 30 psi	42.5 psi	53 psi	N/A	55 psi	45 psi	31.2 psi
τ @ 60 psi	73 psi	88 psi	N/A	79 psi	75.6 psi	55 psi
Φ'	52.4°	57.3	N/A	55.7°	52.9°	44.0
Es (psi)	3,567	1,150	N/A	2,300	1,600	1,333

Table 8. Angle of Internal Friction and Initial Tangent Modulus

at Dr = 75% (85% for Sample2)

Sample 2 had the lowest modulus and the least amount of difference between it internal angle of friction for each condition. This is most likely related to the high percentage of fines in this sample. Sample 2 had a high maximum dry density using Standard Proctor Compaction test methods, so the difference between strength parameters calculated using 95% Standard Proctor density and the density calculated using a relative density of 85% are not that different. This is the reason the resulting strength parameters are very similar. Stress versus strain curves and failure envelopes for each sample are shown in Appendix E. However, due to incomplete data, results for Sample 3 are not included.

Comparison of Strength Parameters for Proctor Density versus Relative Density

Direct shear testing showed that the strength parameters for each sample corresponding to relative density equal to 75% (85% for Sample 2) were higher than those produced by the samples when placed at 95% Standard Proctor density. The difference between internal angles of friction (Φ') produced for each scenario ranged from about 2 degrees to almost 7 degrees. Sample 2 had a difference of 1.6 degrees, most likely due to the high percent of fines and resulting high values of density using Standard Proctor Compaction test procedures. Samples 1 and 4 had the highest differences in Φ' at 6.9 degrees and 6.7 degrees, respectively. Sample 6 had a difference of 5.9 degrees and Sample 5 yielded a difference of 4.8 degrees. The samples with the largest variation in angle of internal friction between the two placement conditions also had the lowest percentages of fines. From these results it can be concluded that the amount of fines present in a soil sample affect the reliability of the Standard Proctor Compaction test. The Standard Proctor Compaction test yielded low values of maximum dry density for all of the samples with low percentages of fines, which influenced the strength of the soil during the direct shear tests. In contrast, the values of dry density calculated from relative densities corresponding to dense soil yielded higher values of internal angle of friction. Also, the average initial tangent modulus for each sample was higher using the density corresponding to Dr = 75% than those corresponding to 95% Standard Proctor density. Figure 3 shows a comparison of the angle of internal friction for each shear test condition, and Figure 4 shows a comparison of the initial tangent modulus for each test condition.

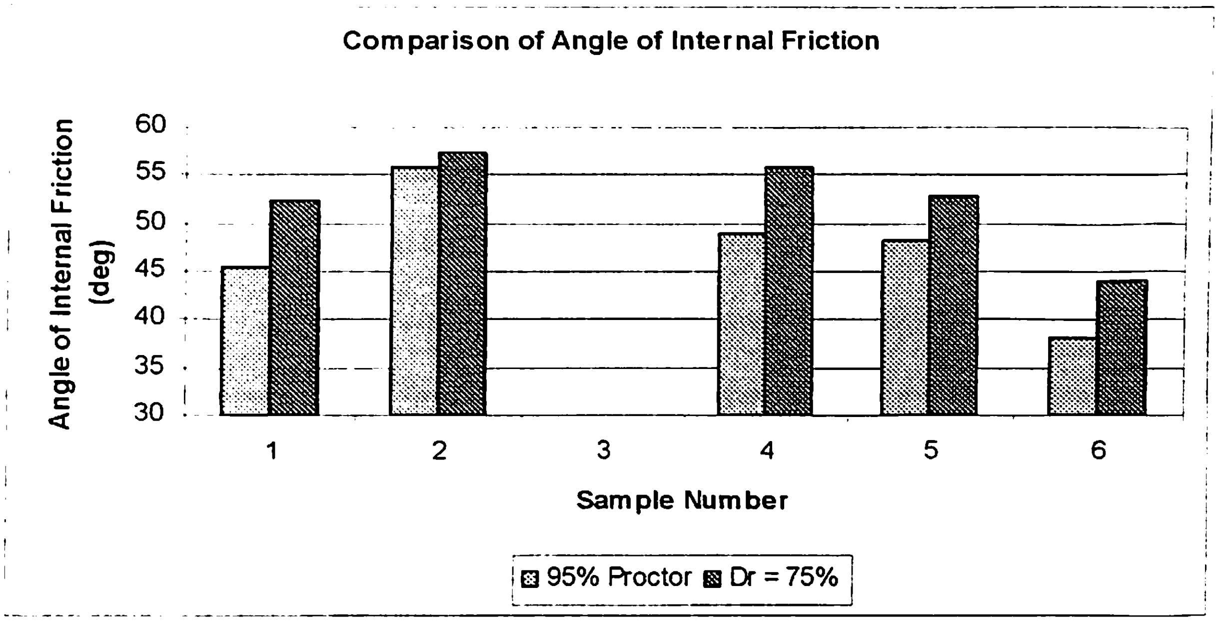

Figure 3. Angle of Internal Friction for Different Placement Conditions

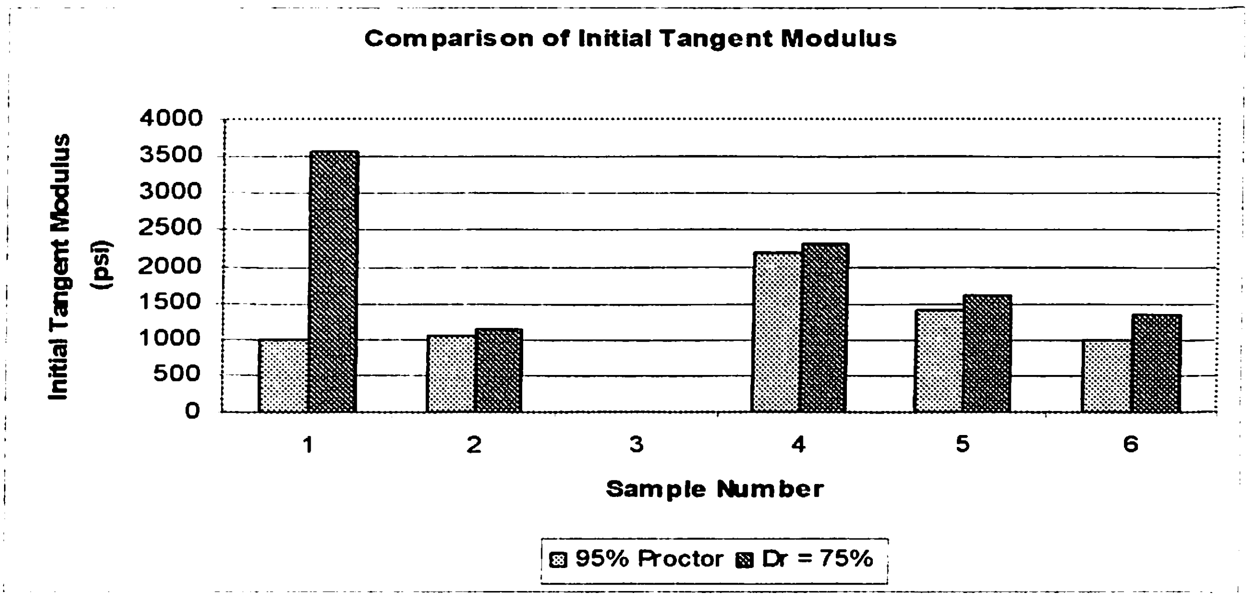

Figure 4. Initial Tangent Modulus for Different Placement Conditions

Settlement Analysis for Case Studies

A case study was conducted to further evaluate the differences between the Standard Proctor Compaction test and relative density test as quality control for granular backfill material. The study consisted of settlement analysis using the initial tangent modulus and Poisson's ratio for each soil sample under the different placement conditions. Figure 5 is a basic cross –section of an approach embankment for a bridge (Schwidder).

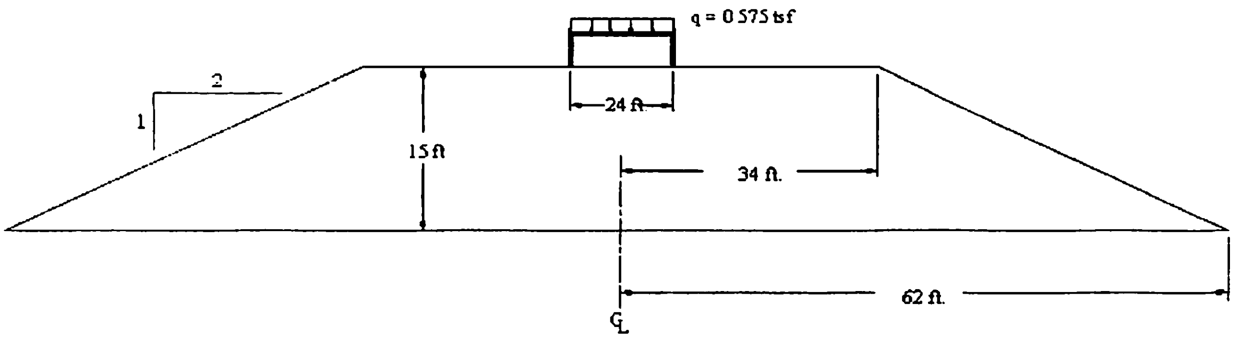

Figure 5. Cross-Section of Bridge Approach Embankment (Schwidder)

Two cases were evaluated for the settlement analysis. An assumed footing size of 24 ft. by 24 ft. was used under a normal load of 0.2 tons per square foot and 0.575 tons per square foot. The first loading scenario was to simulate a pavement load acting alone and the second case was used to simulate the pavement load in addition to an 18 kip single axle load. The footing was chosen as 24 feet wide to represent a common roadway width. The settlement was estimated for each sample using strength parameters determined from the direct shear tests and the following equation (Das):

$$S = (q)(B)(Ip)\frac{(1 - \mu^2)}{Es} \qquad \text{Eq. 4}$$

where

S = elastic settlement

q = net pressure applied

B = width of foundation

μ = Poisson's ratio

Es = modulus of elasticity for soil

Ip = nondimentional influence factor

Poisson's ratio for each soil sample was determined using Table 9. If the soil had a relative density corresponding to loose sand, the midpoint of the values of μ in Table 6 corresponding to loose sand were used. Likewise, if the soil had a relative density corresponding to medium or dense sand, the midpoint of either medium or dense sand in Table 9 was used. The influence factor, Ip, was found from Table 10, where m is equal to the length of the foundation divided by the width of the foundation.

Type of soil	Poisson's ratio, μ_s
Loose sand	0.2–0.4
Medium sand	0.25–0.4
Dense sand	0.3–0.45
Silty sand	0.2–0.4
Soft clay	0.15–0.25
Medium clay	0.2–0.5

Table 9. Poisson's Ratio for Different Soil Types (Das)

		Ip		
		Flexible		
Shape	m	Center	Corner	Rigid
Circle	N/A	1.00	0.64	0.79
Rectangle	1	1.12	0.56	0.88
	1.5	1.36	0.68	1.07
	2	1.53	0.77	1.21
	3	1.78	0.89	1.42
	5	2.10	1.05	1.70
	10	2.54	1.27	2.10
	20	2.99	1.49	2.46
	50	3.57	1.80	3.00
	100	4.01	2.00	3.43

Table 10. Table for Influence Factor for Elastic Settlement (after Das, 2002)

The calculated values of settlement for each sample were higher using 95% Standard Proctor Compaction results than those using a relative density of 75% (85% for Sample 2). For case 1, samples had differences in settlement from around one tenth of an inch to six tenths of an inch. Sample 1 produced a difference of six tenths of an inch, and the others were all less than one quarter of an inch. When the 18 kip axle load was added, the settlements increased by a significant amount. Sample 1 still had the largest difference of 1.7 inches. The differences in settlements ranged from one tenth of an inch to 1.7 inches. Sample 6 had a difference of almost seven tenths of an inch and the rest of the samples were between one tenth of an inch and three tenths of an inch. The calculations of settlement for each sample are in Appendix F. Figure 6 shows a comparison of the settlements for each sample.

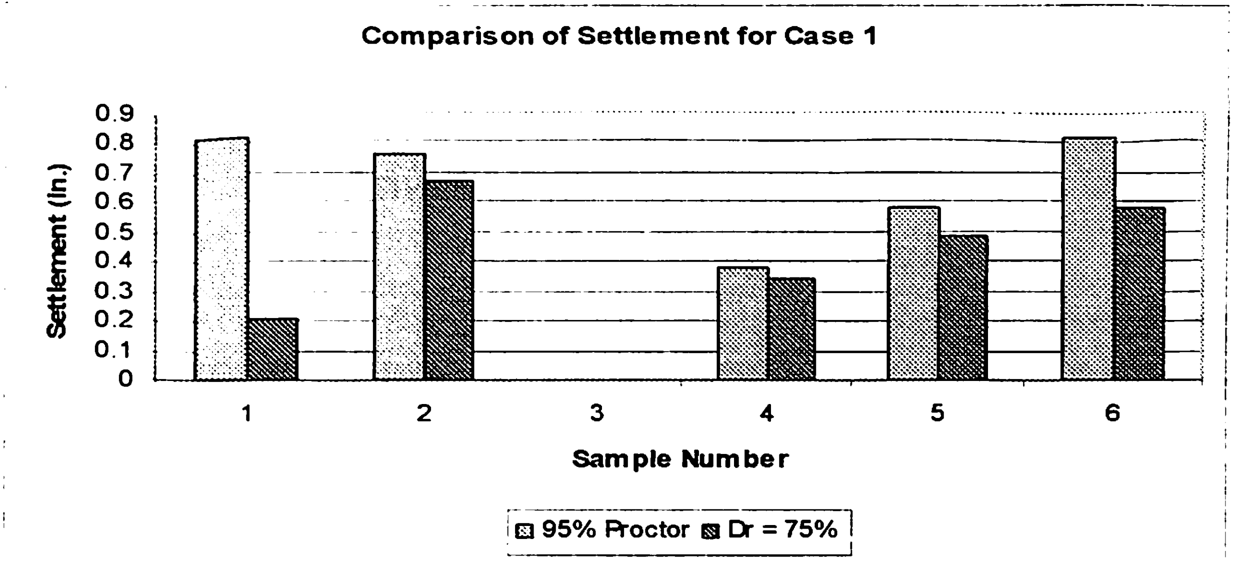

Figure 6. Settlement Under Pavement Load

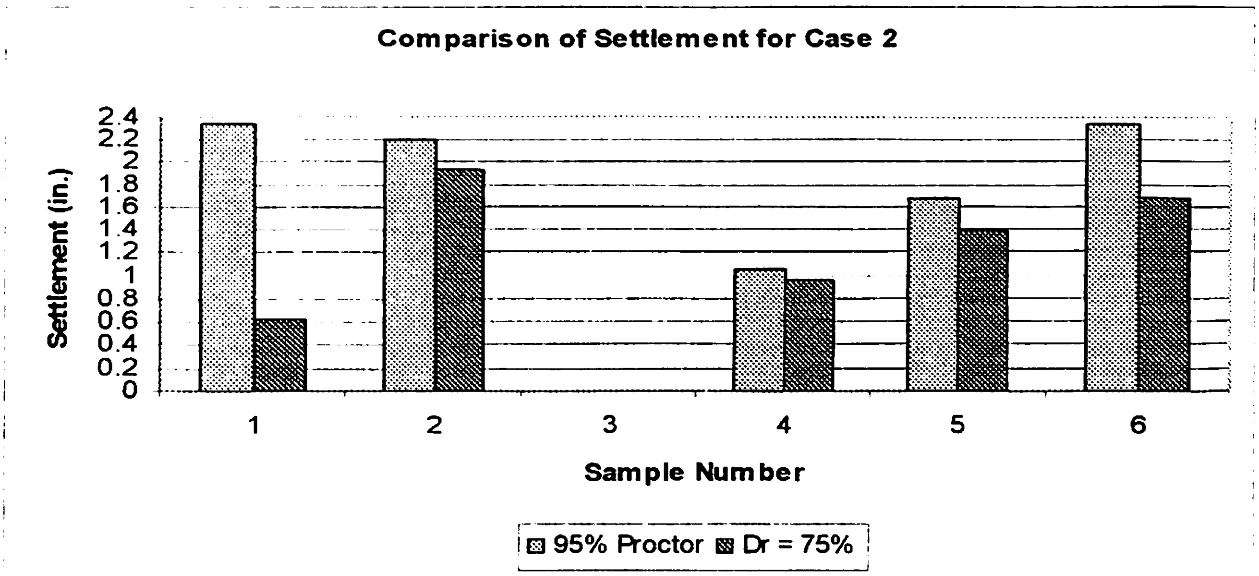

Figure 7. Settlement Under Pavement and 18 Kip Axle Load

CHAPTER 5

DISCUSSION OF RESULTS

The purpose of this paper was to compare Standard Proctor compaction testing, which is the current ODOT acceptance criteria procedure for quality control of granular backfill material, with an alternative approach using relative density. The first step to achieve this was to conduct a thorough literature review pertaining to each method. The next step was to compare each method based on laboratory results of six soil samples. The final step was to use the laboratory results to conduct a case study which estimated settlement of soils placed using the two different methods as quality control criteria.

Discussion of Standard Proctor Compaction and Relative Density Results

The results of Standard Proctor Compaction tests produced lower values of maximum density than relative density testing techniques. Even Sample 2, with a high percentage of fines, had a lower value from Proctor tests. When Standard Proctor Compaction maximum dry density results were used in the relative density equation along with maximum and minimum index densities, only sample 2 produced a value for relative density corresponding to medium density. The rest of the samples fell in the ranges of loose to very loose densities. Also, the results from direct shear tests showed that soils placed at relative density values of 75% (85% for sample 2) had higher shear strength than those placed at 95% of the maximum dry density produced from Standard

Proctor Compaction tests. Results from the settlement case study showed that the values of settlement for each method were less when the samples were placed at 75% relative density. For the first case, without the 18 kip axle load, the settlements ranged from two tenths of an inch to eight tenths of an inch, and the differences between the two placement conditions ranged from one tenth of an inch to six tenths of an inch. This is with no load other than the pavement. When the18 kip axle load was added the differences in settlement between the two placement conditions went up. The actual settlements ranged from one inch to over two inches. The difference ranged from one tenth of an inch to 1.7 inches. Only one sample had a settlement of one tenth of an inch, the rest ranged from about three tenths to seven tenths, and sample 1 had a settlement of 1.7 inches. It is clear that the relative density test produces higher values of maximum density and those common recommended percentages of maximum dry densities based on Standard Proctor Compaction results yield low relative density percentages. This research also shows that the angle of internal friction and initial tangent modulus are higher when a soil is compacted to 75% relative density instead of 95% of the maximum dry density produced from the Standard Proctor Compaction test. When these values are entered into an equation for settlement, the difference between the two test methods is in favor of relative density. Sample had higher ranges for settlement than the other samples had, but all samples had higher settlement when placed at 95% Standard Proctor density than they did when placed at 75% relative density. More research is recommended for settlement analysis due to such high ranges of values for Sample 1 and the lack of information for Sample 3. More samples should be tested to be sure of the results since only four samples produced results that appeared to be accurate for settlement analysis.

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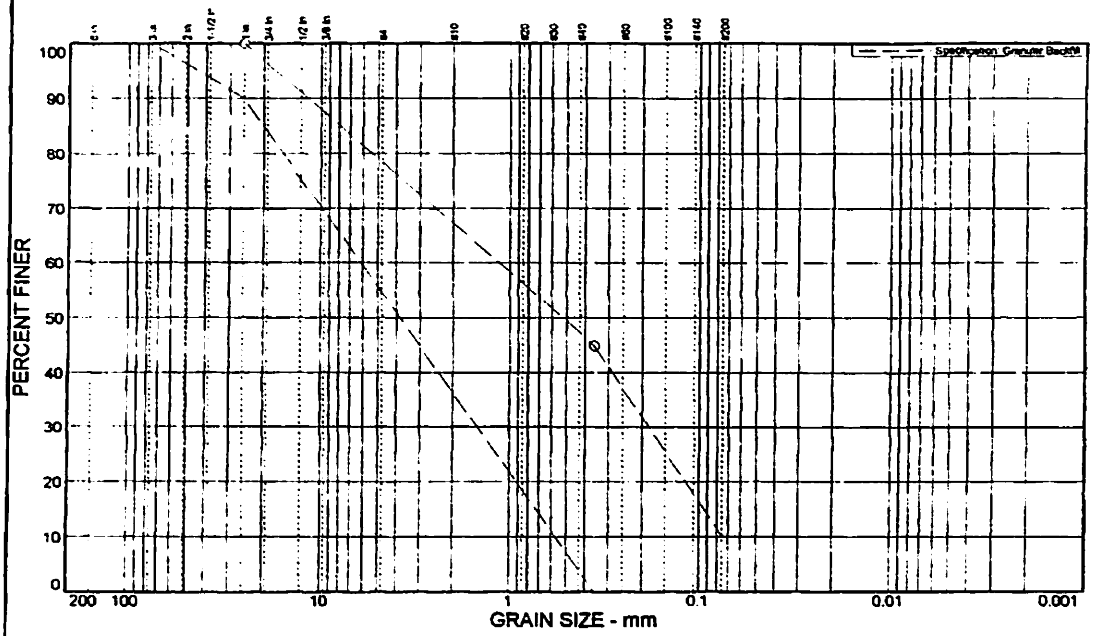
Appendix A
Sieve Analysis Data

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
	21.6							

SIEVE inches size	PERCENT FINER O		
1	100.0		

	GRAIN SIZE		
D_{60}	1.14		
D_{30}			
D_{10}			

	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER O		
#45	45.0		

SOIL DESCRIPTION

Granular Backfill band

REMARKS:

O

O Source

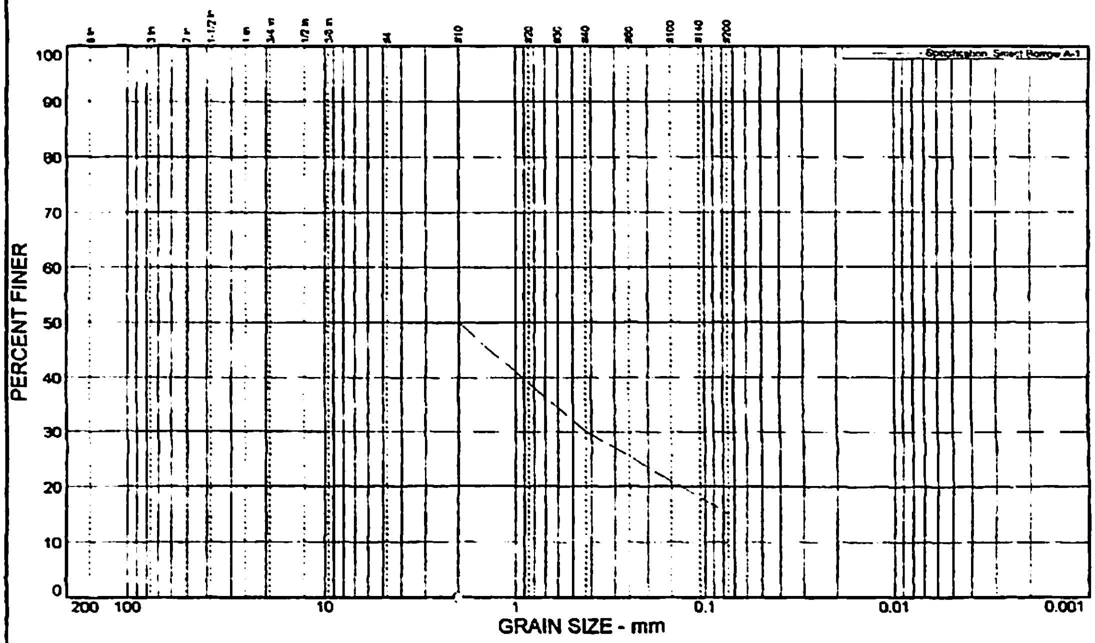
Sample No.: Granular Backfill Band

Oklahoma State University	Client: Project: Quality Control Testing for Granular Materials Project No.: Plate

Gradation Requirements for Granular Backfill

(ODOT 703.05)

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL

SIEVE inches size	PERCENT FINER O		
	GRAIN SIZE		
D_{60}			
D_{30}			
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER O		
#10	0.0		

SOIL DESCRIPTION

O Select Borrow A-1 Band

REMARKS:

O

O Source:

Sample No.: Select Borrow A-1 Band

Oklahoma State University

Client:

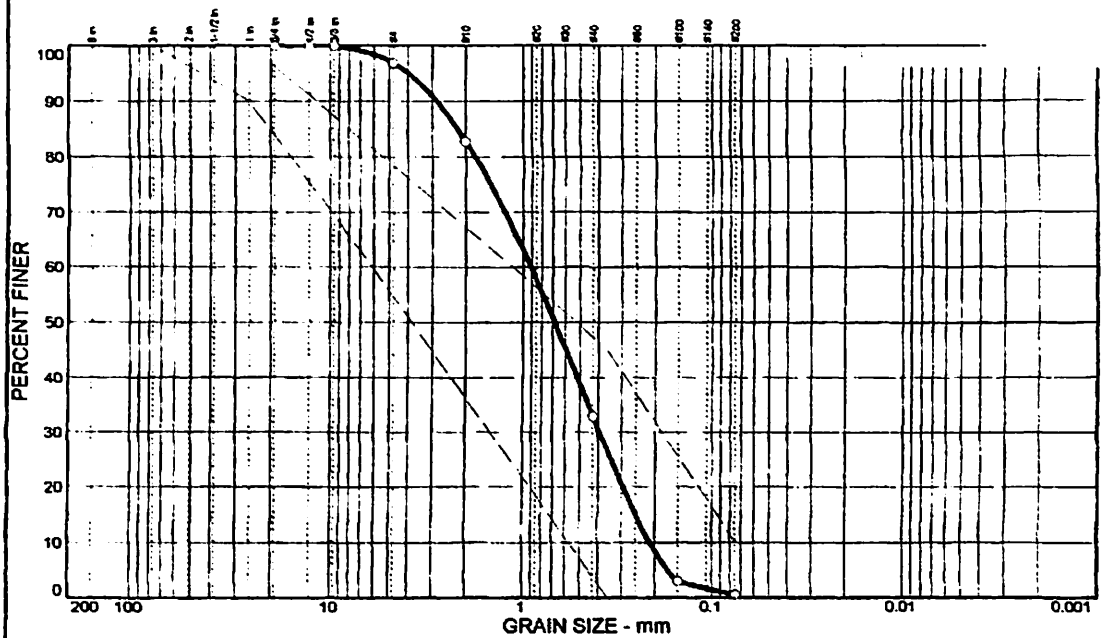
Project: Quality Control Testing for Granular Materials

Project No.: Plate

Gradation Requirements for Select Borrow (AASHTO group A-1)

(ODOT 705.01)

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
	3.1	96.5	0.4					

SIEVE inches size	PERCENT FINER O		
3/4	100.0		
1/2	100.0		
3/8	100.0		

	GRAIN SIZE		
D_{60}	0.913		
D_{30}	0.392		
D_{10}	0.212		
	COEFFICIENTS		
C_c	0.79		
C_u	4.30		

SIEVE number size	PERCENT FINER O		
#4	96.9		
#10	82.8		
#40	32.9		
#100	2.9		
#200	0.4		

SOIL DESCRIPTION
O Concrete Sand

REMARKS
O

O Source:

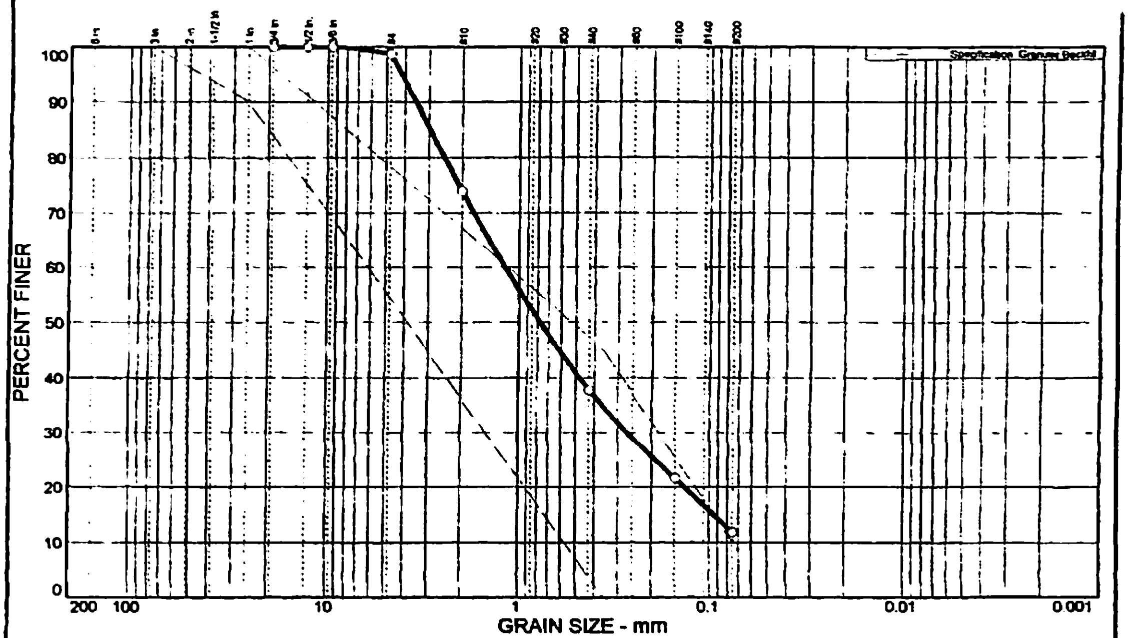
Sample No.: 1 (CC) *

Oklahoma State University

Client:
Project: Quality Control Testing for Granular Materials
Project No.
Plate

Sieve Analysis for Sample 1

Particle Size Distribution Report



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
o		1.3	86.9	11.8					

SIEVE inches size	PERCENT FINER o		
3/4	100.0		
1/2	100.0		
3/8	100.0		
	GRAIN SIZE		
D_{60}	1.18		
D_{30}	0.266		
D_{10}			
	COEFFICIENTS		
C_c			
C_u			

SIEVE number size	PERCENT FINER o		
#4	98.7		
#10	73.9		
#40	37.8		
#100	21.6		
#200	11.8		

SOIL DESCRIPTION

o Screenings

REMARKS:

o

o Source:

Sample No.: 2 (Scr) *

Oklahoma State University

Client:

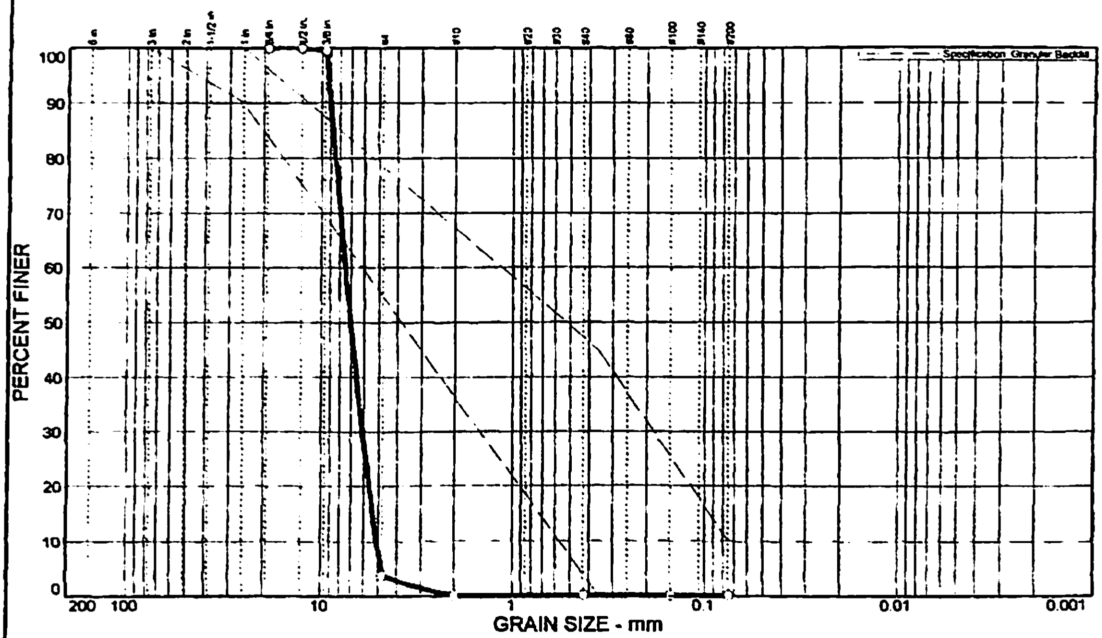
Project: Quality Control Testing for Granular Materials

Project No.:

Plate

Sieve Analysis for Sample 2

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
	96.2	3.7	0.1					

SIEVE inches size	PERCENT FINER o		
3/4	100.0		
1/2	100.0		
3/8	99.5		

	GRAIN SIZE		
D_{60}	7.50		
D_{30}	6.10		
D_{10}	5.11		

	COEFFICIENTS		
C_c	0.97		
C_u	1.47		

SIEVE number size	PERCENT FINER o		
#4	3.8		
#10	0.1		
#40	0.1		
#100	0.1		
#200	0.1		

SOIL DESCRIPTION

o Fine River Rock

REMARKS:

o

o Source.

Sample No.: 3 (FRR) *

Oklahoma State University

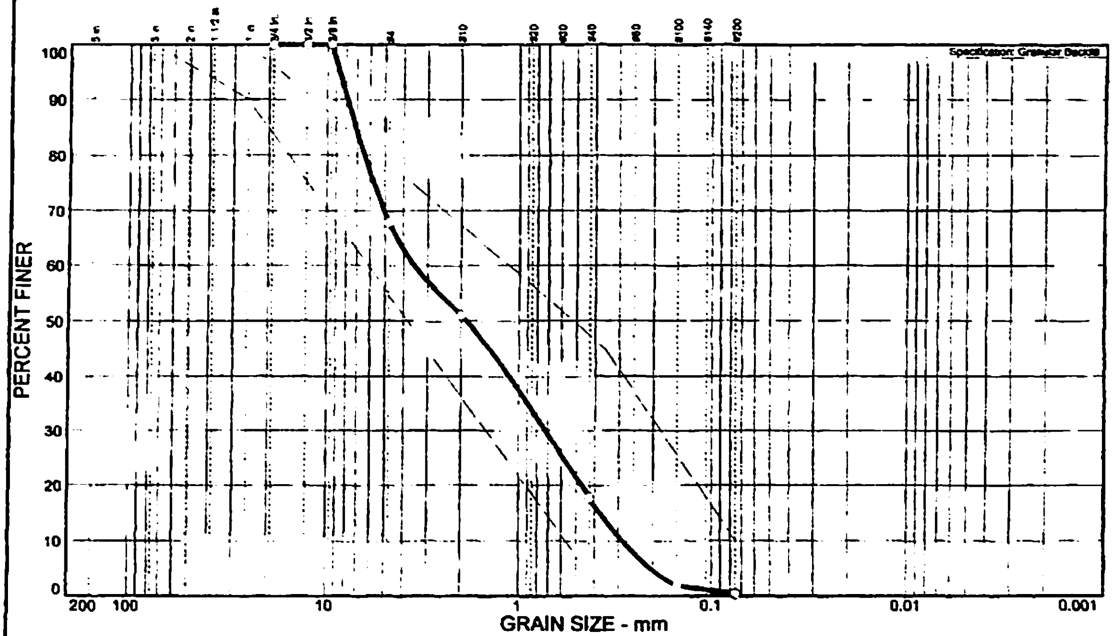
Client:

Project: Quality Control Testing for Granular Materials

Project No.: Plate

Sieve Analysis for Sample 3

Particle Size Distribution Report



	% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
○		32.2	67.4	0.4					

SIEVE inches size	PERCENT FINER ○		
3/4	100.0		
1/2	100.0		
3/8	100.0		
	GRAIN SIZE		
D_{60}	3.53		
D_{30}	0.727		
D_{10}	0.285		
	COEFFICIENTS		
C_c	0.52		
C_u	12.37		

SIEVE number size	PERCENT FINER ○		
#4	67.8		
#10	51.0		
#40	17.8		
#100	1.9		
#200	0.4		

SOIL DESCRIPTION

70% sand 30% fine river rock

REMARKS

○

○ Source:

Sample No.: 4 70cc/30f.:*

Oklahoma State University

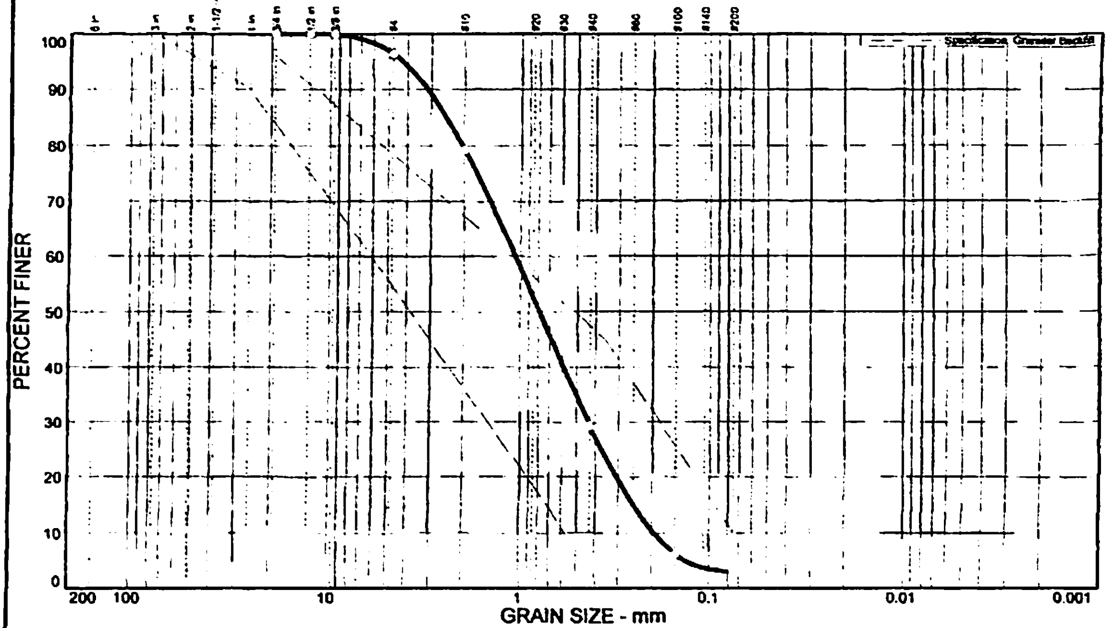
Client

Project Quality Control Testing for Granular Materials

Project No. Plate

Sieve Analysis for Sample 4

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
	3.6	93.5	2.9					

SIEVE inches size	PERCENT FINER ○		
3/4	100.0		
1/2	100.0		
3/8	100.0		
	GRAIN SIZE		
D_{60}	1.07		
D_{30}	0.433		
D_{10}	0.195		
	COEFFICIENTS		
C_c	0.90		
C_u	5.50		

SIEVE number size	PERCENT FINER ○		
#4	96.4		
#10	79.2		
#40	29.4		
#100	6.2		
#200	2.9		

SOIL DESCRIPTION

○ 80% Concrete Sand 20% Screenings

REMARKS:

○

○ Source:

Sample No.: 5 80cc20scr *

Oklahoma State University

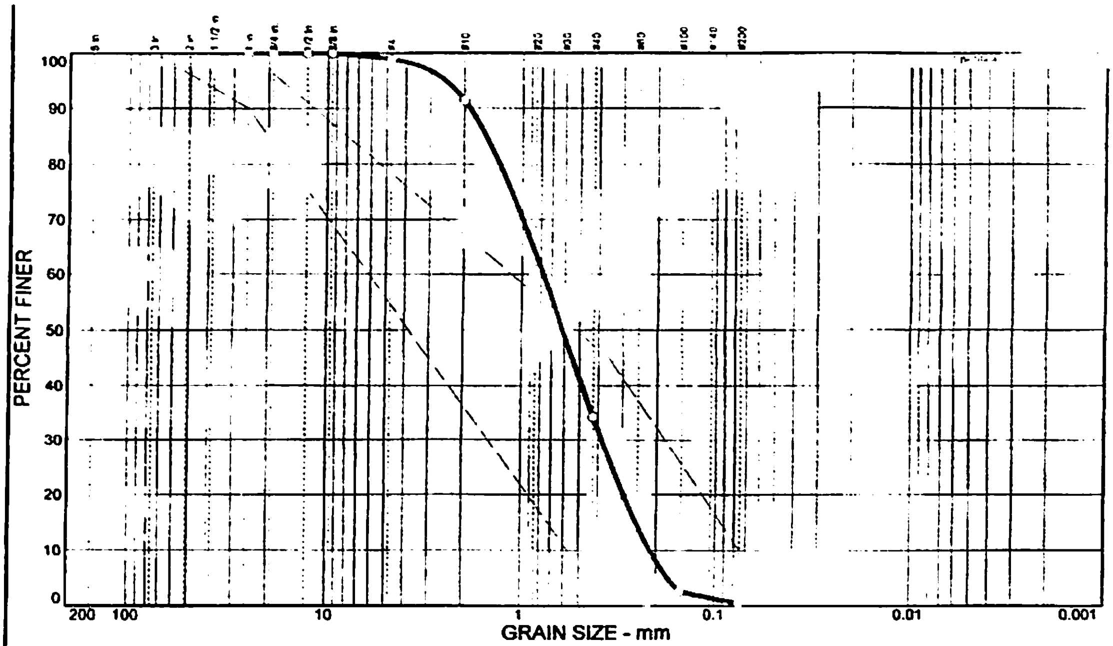
Client

Project Quality Control Testing for Granular Materials

Project No.: Plate

Sieve Analysis for Sample 5

Particle Size Distribution Report



% + 3"	% GRAVEL	% SAND	% SILT	% CLAY	USCS	AASHTO	PL	LL
	1.1	98.5	0.4					

SIEVE inches size	PERCENT FINER O		
1	100.0		
3/4	100.0		
1/2	100.0		
3/8	100.0		

	GRAIN SIZE		
D_{60}	0.778		
D_{30}	0.384		
D_{10}	0.215		

	COEFFICIENTS		
C_c	0.88		
C_u	3.62		

SIEVE number size	PERCENT FINER O		
#4	98.9		
#10	91.4		
#40	34.2		
#100	2.6		
#200	0.4		

SOIL DESCRIPTION

O OU Sand

REMARKS:

O

O Source:

Sample No.: 6 OUsand

Oklahoma State University

Client

Project Quality Control Testing for Granular Materials

Project No. Plate

Sieve Analysis for Sample 6

Appendix B
Uncompacted Void Content
Calculations

Uncompacted Void Content Calculations Sample 1

Assume Gs = 2.65

V = 99.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot =	353.1	g
M cyl =	184.7	g
M soil =	168.4	g

$$U = \frac{99.9\ mL - (168.4\ g/2.65)}{99.9\ mL}$$

U = 36.4%

Trial 2:

M tot =	352.8	g
M cyl =	184.7	g
M soil =	168.1	g

$$U = \frac{99.9\ mL - (168.1\ g/2.65)}{99.9\ mL}$$

U = 36.5%

$$\mathbf{Ur} = \frac{36.4 + 36.5}{2} \quad \boxed{\mathbf{36.4\%}}$$

Uncompacted Void Content Calculations Sample 2

Assume Gs = 2.70

V = 99.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot = 350.6 g

M cyl = 184.7 g

M soil = 165.9 g

$$U = \frac{99.9\ mL - (165.9\ g/2.70)}{99.9\ mL}$$

U = 38.5%

Trial 2:

M tot = 352.5 g

M cyl = 184.7 g

M soil = 167.8 g

$$U = \frac{99.9\ mL - (167.8\ g/2.70)}{99.9\ mL}$$

U = 37.8%

$$\mathbf{Ur} = \frac{38.5 + 37.8}{2} \quad \boxed{\mathbf{38.2\%}}$$

Uncompacted Void Content Calculations Sample 3

ssume Gs = 2.50

V = 2910.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot =	6355.8	g
M cyl =	2395.2	g
M soil =	3860.6	g

$$U = \frac{2910.9 \text{ mL} - (3860.6 \text{ g}/2.50)}{2910.9 \text{ mL}}$$

U = 46.9%

Trial 2:

M tot =	6447.3	g
M cyl =	2395.2	g
M soil =	4052.1	g

$$U = \frac{2910.9 \text{ mL} - (4052.1 \text{ g}/2.50)}{2910.9 \text{ mL}}$$

U = 44.3%

$$\mathbf{Ur} = \frac{46.9 + 44.3}{2} \quad \boxed{\mathbf{45.6\%}}$$

Uncompacted Void Content Calculations Sample 4

Assume Gs = 2.65

V = 2910.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot =	7650.3	g
M cyl =	2395.2	g
M soil =	5255.1	g

$$U = \frac{2910.9\ mL - (5255.1\ g/2.65)}{2910.9\ mL}$$

U = 31.9%

Trial 2:

M tot =	7674.4	g
M cyl =	2395.2	g
M soil =	5279.2	g

$$U = \frac{2910.9\ mL - (5279.2\ g/2.65)}{2910.9\ mL}$$

U = 31.6%

$$\mathbf{Ur} = \frac{31.9 + 31.6}{2} \quad \boxed{\mathbf{31.8\%}}$$

Uncompacted Void Content Calculations Sample 5

Assume Gs = 2.68

V = 99.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot =	356.3	g
M cyl =	184.7	g
M soil =	171.6	g

$$U = \frac{99.9 \text{ mL} - (171.6 \text{ g}/2.68)}{99.9 \text{ mL}}$$

U = 35.9%

Trial 2:

M tot =	359.3	g
M cyl =	184.7	g
M soil =	174.6	g

$$U = \frac{99.9 \text{ mL} - (174.6 \text{ g}/2.68)}{99.9 \text{ mL}}$$

U = 34.8%

$$\mathbf{Ur} = \frac{35.9 + 34.8}{2} \quad \boxed{\mathbf{35.3\%}}$$

Uncompacted Void Content Calculations Sample 6

Assume Gs = 2.65

V = 99.9 mL

$$U = \frac{V - (Msoil / Gs) * 100}{V}$$

Trial 1:

M tot =	346	g
M cyl =	184.7	g
M soil =	161.3	g

$$U = \frac{99.9\ mL - (161.3\ g/2.65)}{99.9\ mL}$$

U = 39.1%

Trial 2:

M tot =	345.1	g
M cyl =	184.7	g
M soil =	160.4	g

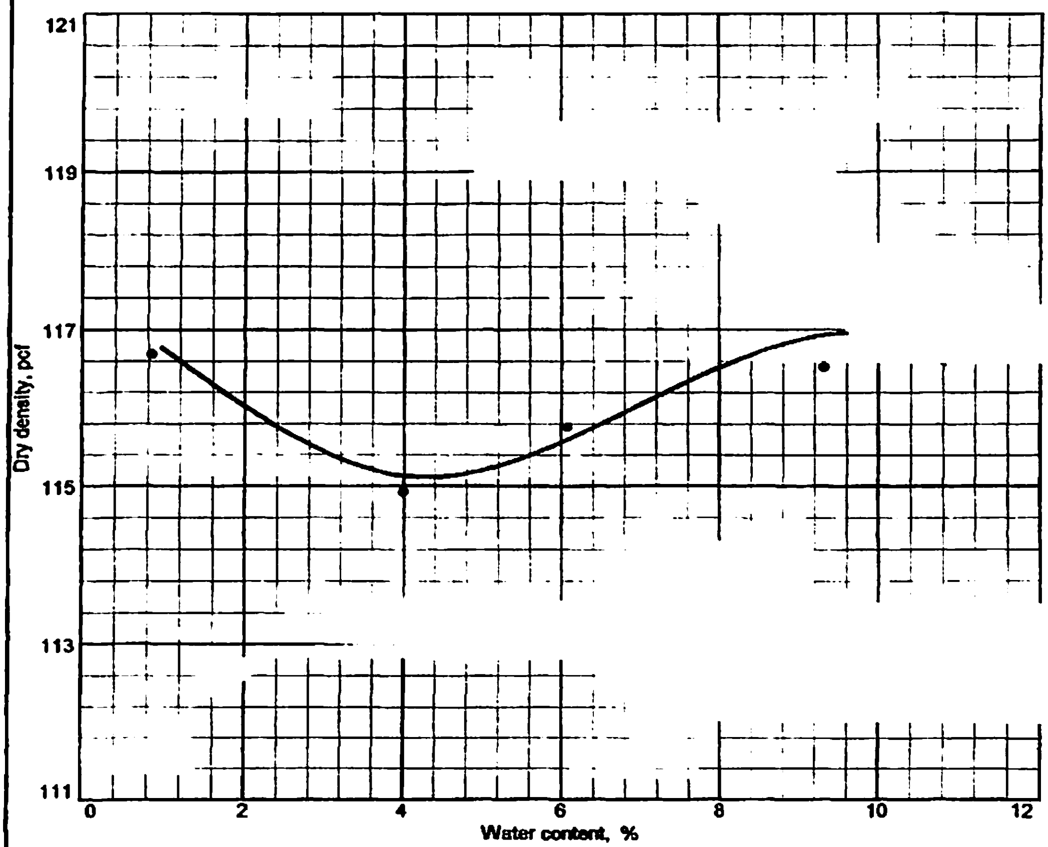
$$U = \frac{99.9\ mL - (160.4\ g/2.65)}{99.9\ mL}$$

U = 39.4%

$$\mathbf{Ur} = \frac{39.1 + 39.4}{2} \quad \boxed{\mathbf{39.2\%}}$$

Appendix C
Standard Proctor Compaction Data
(Moisture Density Curves)

COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
	SP poorly graded sand	A-1-b					0.0	0.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 117.0 pcf Optimum moisture = 9.7 %	Concrete Sand

Project No. **Client:**
Project: Quality Control Testing for Granular Materials

• **Source:** **Sample No.:** 1

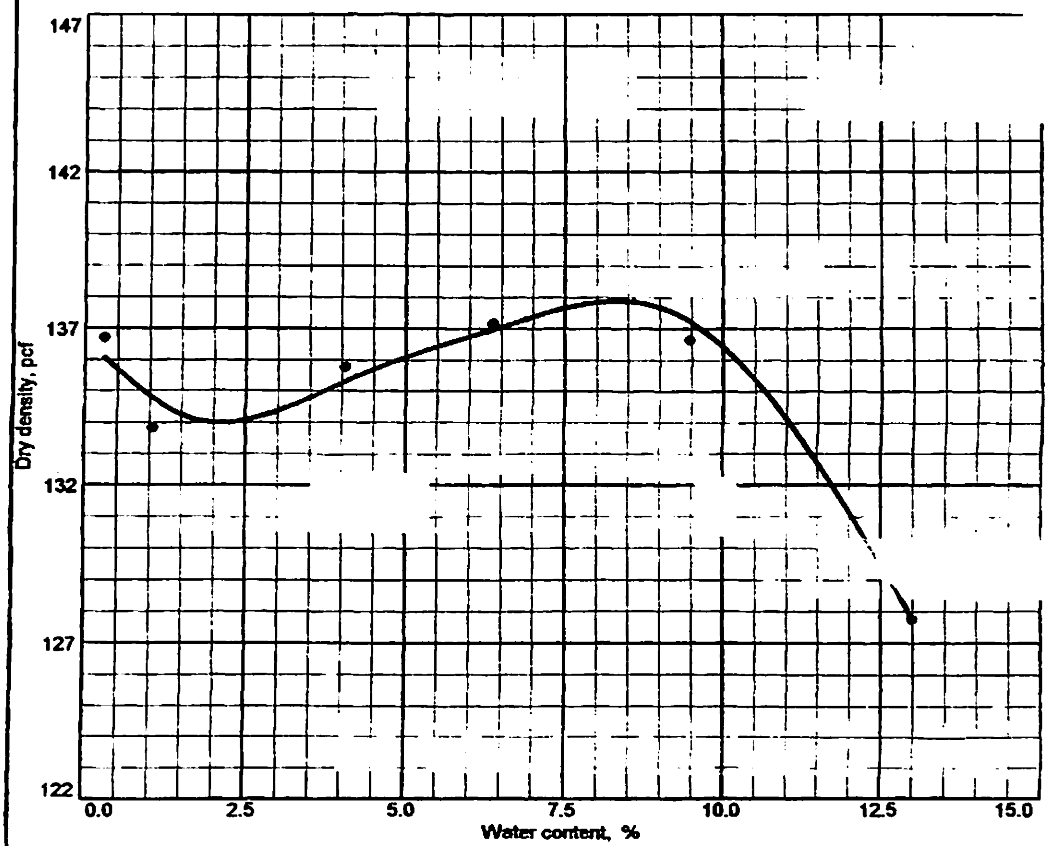
Remarks:

COMPACTION TEST REPORT

Oklahoma State University

Plate

COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	SW-SM	A-2-4					0.0	11.8

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 137.9 pcf Optimum moisture = 8.3 %	Screenings

Project No. **Client:**

Project: Quality Control Testing for Granular Materials

• **Source:** **Sample No.:** 2

Remarks:

COMPACTION TEST REPORT

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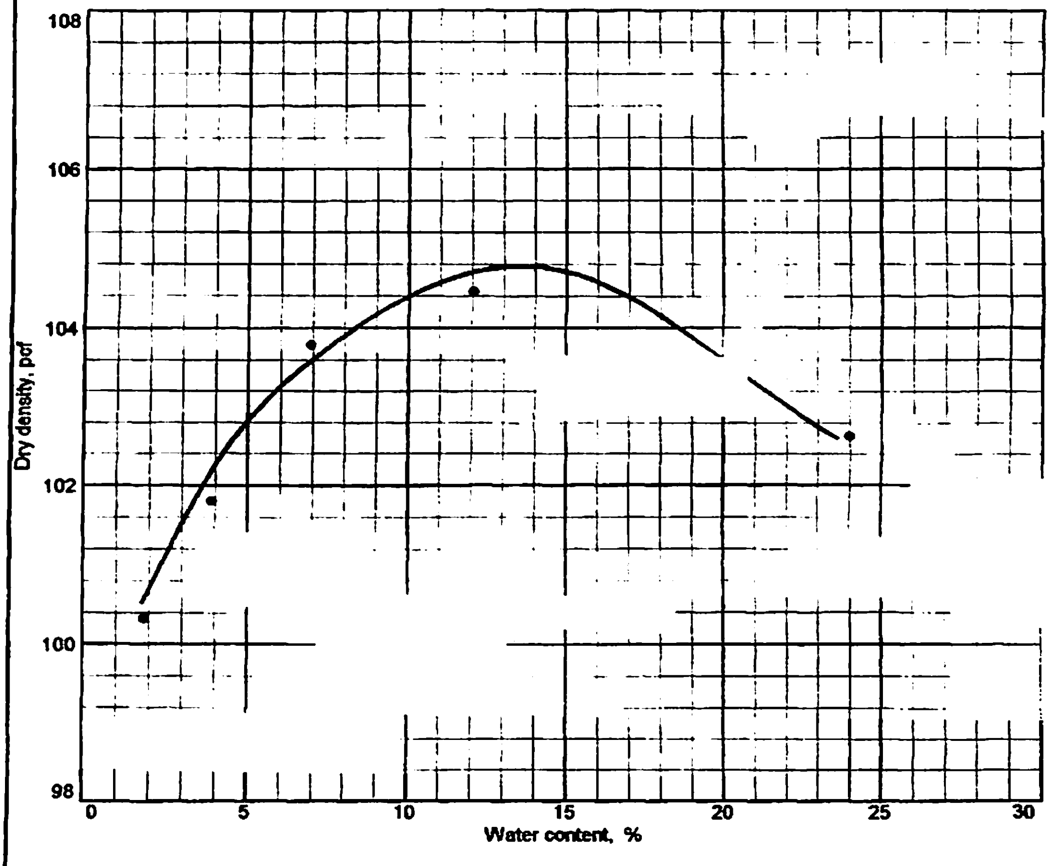
COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
	GP poorly graded grav.	A-1-a					0.0	0.1

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 104.8 pcf Optimum moisture = 13.6 %	Fine River Rock

Project No. **Client:**

Project: Quality Control Testing for Granular Materials

● **Source:** **Sample No.:** 3

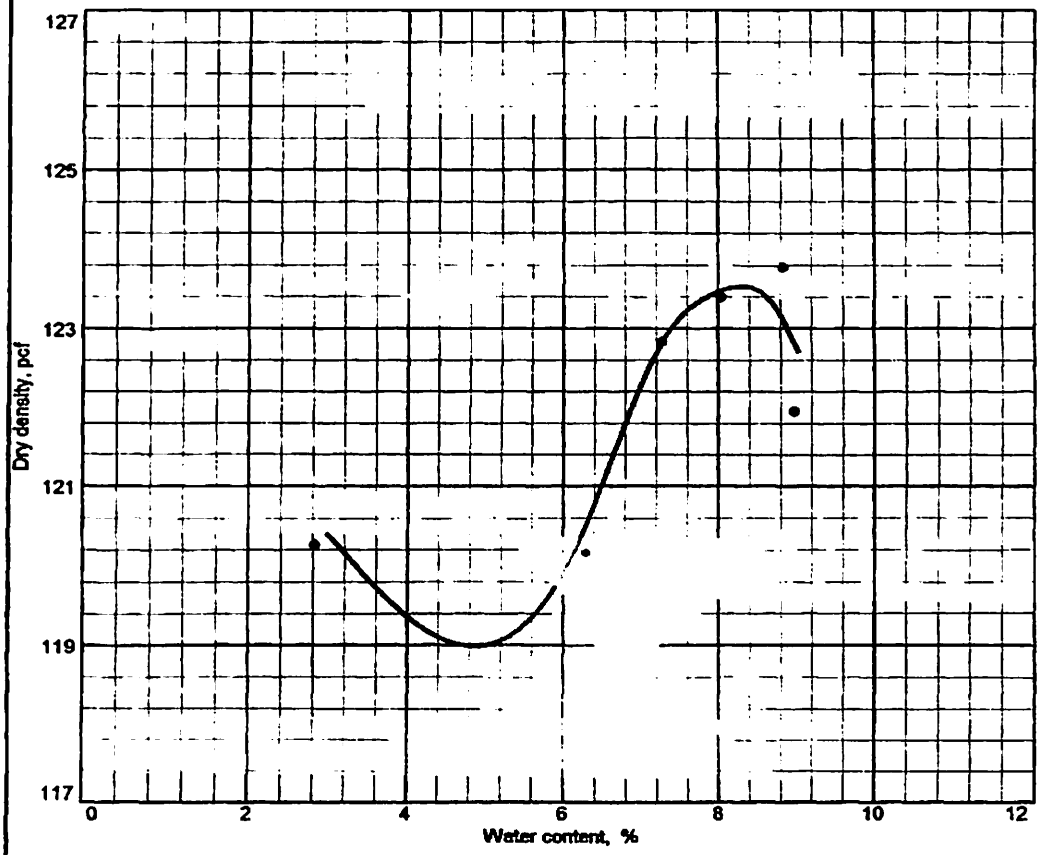
COMPACTION TEST REPORT

Oklahoma State University

Remarks:

Plate

COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	SP sand with gravel	A-3					0.0	0.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 123.5 pcf Optimum moisture = 8.3 %	70% sand 30% fine river rock

Project No. Client:

Project: Quality Control Testing for Granular Materials

• Source: Sample No.: 4

Remarks:

COMPACTION TEST REPORT

Oklahoma State University

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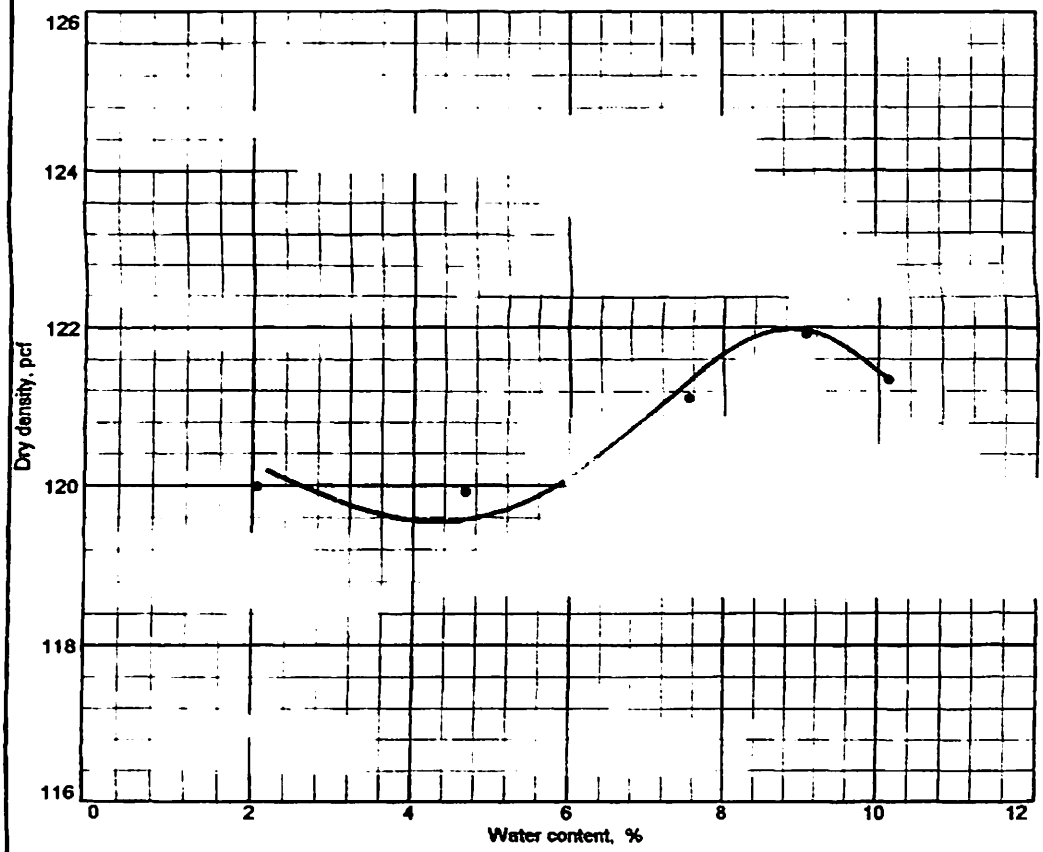
COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	USCS	AASHTO						
	SP poorly graded sand	A-3					0.0	2.9

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 122.0 pcf Optimum moisture = 8.9 %	80% Concrete Sand 20% Screenings

Project No. **Client:**

Project: Quality Control Testing for Granular Materials

● **Source:** **Sample No.:** 5

Remarks:

COMPACTION TEST REPORT

Oklahoma State University

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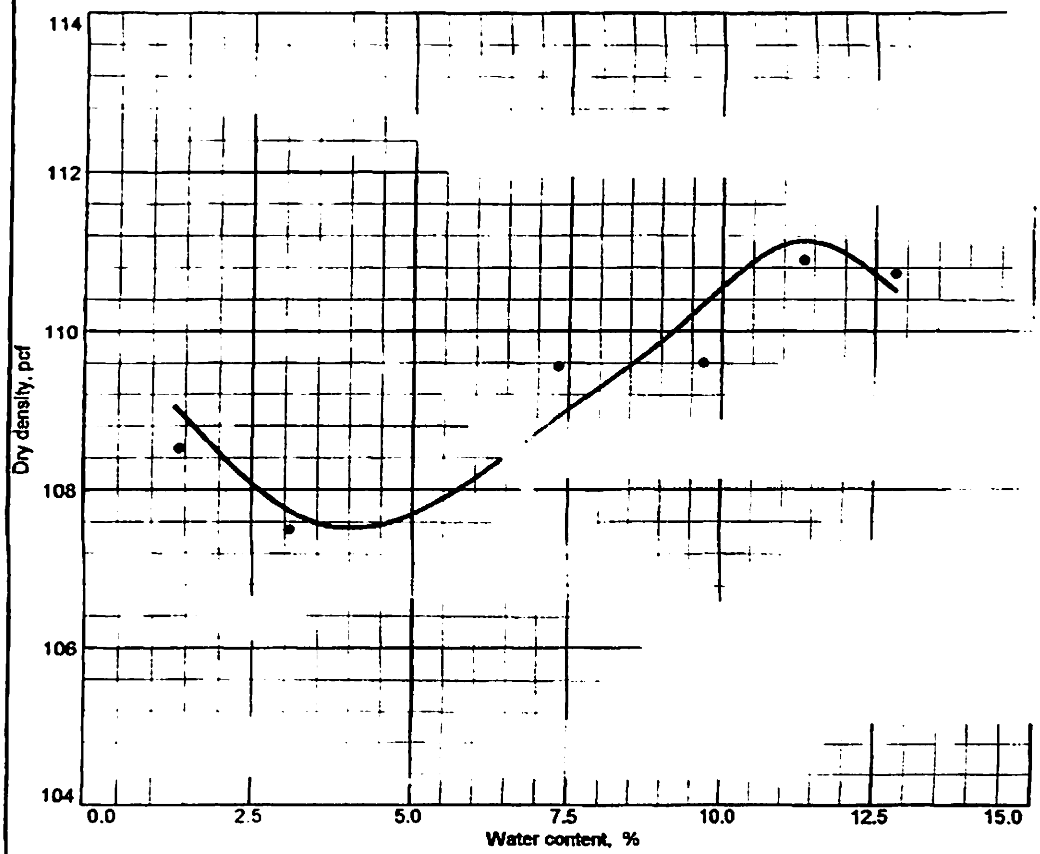
COMPACTION TEST REPORT



Test specification: AASHTO T 99 Method C Standard

Elev/ Depth	Classification USCS	Classification AASHTO	Nat. Moist.	Sp.G.	LL	PI	% > 3/4 in.	% < No.200
	SP Poorly graded sand	A-1-b					0.0	0.4

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 111.1 pcf Optimum moisture = 11.4 %	OU Sand

Project No. **Client:**

Project: Quality Control Testing for Granular Materials

● **Source:** **Sample No.:** 6

Remarks:

COMPACTION TEST REPORT

Oklahoma State University

Plate

Appendix D
Relative Density Calculations

Minimum Index Density Calculations for Sample 1

Trial 1:

V = 0.09991 ft^3

Mtot = 10014.8 g

Mmold = 5052.8 g

Msoil = 4962.0 g

$$\rho\ dmin = \frac{(4962\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{109.5 lb/ft^3}$$

Trial 2:

V = 0.09991 ft^3

Mtot = 10027.6 g

Mmold = 5052.8 g

Msoil = 4974.8 g

$$\rho\ dmin = \frac{(4974.8\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{109.8 lb/ft^3}$$

Trial 3:

V = 0.09991 ft^3

Mtot = 10029.0 g

Mmold = 5052.8 g

Msoil = 4976.2 g

$$\rho\ dmin = \frac{(4976.2\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{109.8 lb/ft^3}$$

Trial 4:

V = 0.09991 ft^3

Mtot = 10079.2 g

Mmold = 5052.8 g

Msoil = 5026.4 g

$$\rho\ dmin = \frac{(5026.4\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{110.9 lb/ft^3}$$

$$\boldsymbol{\rho}\ \textbf{dmin} = \frac{109.5 + 109.8 + 109.8 + 110.9}{4} = \boxed{\textbf{110.0 lb/ft^3}}$$

Maximum Index Density Calculations for Sample 1

Trial 1:

Vc = 0.09991 ft^3   Rf = 1.123 in.

Mtot = 10014.8 g

Mmold = 5052.8 g

Msoil = 4962.0 g

H = Ri - Rf + Tp = 1.325" - 1.123" + 0.505" = 0.707 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.707 in.)(1/144) = 0.08835 ft^3

$$\rho\ dmax = \frac{(4982.0\ g) * (2.205\ lb/kg)}{(0.08835\ ft^3) * (1000\ g/kg)} = \textbf{124.3 lb/ft}^3$$

Trial 2:

Vc = 0.09991 ft^3   Rf = 1.138 in.

Mtot = 9984.3 g

Mmold = 5052.8 g

Msoil = 4931.5 g

H = Ri - Rf + Tp = 1.325" - 1.138" + 0.505" = 0.692 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.692 in.)(1/144) = 0.08859 ft^3

$$\rho\ dmax = \frac{(4931.5\ g) * (2.205\ lb/kg)}{(0.08859\ ft^3) * (1000\ g/kg)} = \textbf{122.7 lb/ft}^3$$

Maximum Index Density Calculations for Sample 1

Trial 1: Vc = *0.09991 ft^3* *Rf =* *1.123 in.*

Mtot = 10014.8 g

Mmold = 5052.8 g

H = Ri - Rf + Tp = 1.325" - 1.123" + 0.505" = ...

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * ...

ρ dmax = (4982.0 g) * (2.205 lb/kg) / (0.08835 ft^3) * (1000 g/kg) ...

Trial 2:

Vc = 0.09991 ft^3 Rf = 1.138 in.

Mtot

Mmold

Msoil

H = Ri - Rf + Tp = 1.325" - 1.138" + 0.505" = 0.692 in

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.692 in^3)(1/144) = 0.08859 ft^3

ρ dmax = (4931.5 g) * (2.205 lb/kg) / (0.08859 ft^3) * (1000 g/kg) = 122.7 ...

Trial 3: Vc = 0.09991 ft^3 Rf = 1.162 in.

Mtot = 10111.7 g

Mmold = 5052.8 g

Msoil = 5058.9 g

H = Ri - Rf + Tp = 1.325" - 1.162" + 0.505" = 0.668 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.668 in.)(1/144) = 0.08899 ft^3

$$\rho\ dmax = \frac{(5058.9\ g) * (2.205\ lb/kg)}{(0.08899\ ft^3) * (1000\ g/kg)} = \mathbf{125.3\ lb/ft^3}$$

Trial 4: Vc = 0.09991 ft^3 Rf = 1.152 in.

Mtot = 10120.1 g

Mmold = 5052.8 g

Msoil = 5067.3 g

H = Ri - Rf + Tp = 1.325" - 1.152" + 0.505" = 0.678 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.678 in.)(1/144) = 0.08882 ft^3

$$\rho\ dmax = \frac{(5067.3\ g) * (2.205\ lb/kg)}{(0.08882\ ft^3) * (1000\ g/kg)} = \mathbf{125.8\ lb/ft^3}$$

$$\boldsymbol{\rho\ dmax} = \frac{124.3 + 122.7 + 125.3 + 125.8}{4} = \boxed{\mathbf{124.5\ lb/ft^3}}$$

Minimum Index Density Calculations for Sample 2

Trial 1:

V = 0.09991 ft^3

Mtot = 10002.8 g

Mmold = 5051.0 g

Msoil = 4951.8 g

$$\rho\ dmin = \frac{(4951.8\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{109.3\ lb/ft^3}$$

Trial 2:

V = 0.09991 ft^3

Mtot = 10014.3 g

Mmold = 5051.0 g

Msoil = 4963.3 g

$$\rho\ dmin = \frac{(4963.3\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{109.5\ lb/ft^3}$$

Trial 3:

V = 0.09991 ft^3

Mtot = 10035.3 g

Mmold = 5051.0 g

Msoil = 4984.3 g

$$\rho\ dmin = \frac{(4984.3\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{110.0\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \mathbf{dmin} = \frac{109.3 + 109.5 + 110.0}{3} = \boxed{\mathbf{109.6\ lb/ft^3}}$$

Maximum Index Density Calculations for Sample 2

Trial 1: Vc = 0.09991 ft^3   Rf = 0.482 in.

Mtot = 10061.6 g

Mmold = 5051.0 g

Msoil = 5010.6 g

$H = Ri - Rf + Tp$ = 1.311" - 0.482" + 0.505" = 1.334 in.

$V = Vc - (Ac * H)$ = (0.09991 ft^3)-(28.255 in^2 * 1.334 in.)(1/144) = 0.0781 ft^3

$$\rho\ dmax = \frac{(5010.6\ g) * (2.205\ lb/kg)}{(0.0781\ ft^3) * (1000\ g/kg)} = \mathbf{141.5\ lb/ft^3}$$

Trial 2: Vc = 0.09991 ft^3   Rf = 0.491 in.

Mtot = 10065.4 g

Mmold = 5051.0 g

Msoil = 5014.4 g

$H = Ri - Rf + Tp$ = 1.311" - 0.491" + 0.505" = 1.325 in.

$V = Vc - (Ac * H)$ = (0.09991 ft^3)-(28.255 in^2 * 1.325 in.)(1/144) = 0.07824 ft^3

$$\rho\ dmax = \frac{(5014.4\ g) * (2.205\ lb/kg)}{(0.07824\ ft^3) * (1000\ g/kg)} = \mathbf{141.3\ lb/ft^3}$$

Trial 3: Vc = 0.09991 ft^3 Rf = 0.502 in.

Mtot = 10058.2 g

Mmold = 5051.0 g

Msoil = 5007.2 g

H = Ri - Rf + Tp = 1.311" - 0.502" + 0.505" = 1.314 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 1.314 in.)(1/144) = 0.07842 ft^3

$$\rho \text{ dmax} = \frac{(5007.2 \text{ g}) * (2.205 \text{ lb/kg})}{(0.07842 \text{ ft}^3) * (1000 \text{ g/kg})} = \mathbf{140.8 \text{ lb/ft}^3}$$

$$\rho \text{ dmax} = \frac{141.5 + 141.3 + 140.8}{3} = \boxed{\mathbf{124.5 \text{ lb/ft}^3}}$$

Minimum Index Density Calculations for Sample 3

Trial 1:

$V = 0.09991\ ft^3$

$M_{tot} = 9378.7\ g$

$M_{mold} = 5051.0\ g$

$M_{soil} = 4327.7\ g$

$$\rho_{dmin} = \frac{(4327.7\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{95.5\ lb/ft^3}$$

Trial 2:

$V = 0.09991\ ft^3$

$M_{tot} = 9445.3\ g$

$M_{mold} = 5051.0\ g$

$M_{soil} = 4394.3\ g$

$$\rho_{dmin} = \frac{(4394.3\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{97.0\ lb/ft^3}$$

Trial 3:

$V = 0.09991\ ft^3$

$M_{tot} = 9413.2\ g$

$M_{mold} = 5051.0\ g$

$M_{soil} = 4362.2\ g$

$$\rho_{dmin} = \frac{(4362.2\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \mathbf{96.3\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \mathbf{dmin} = \frac{95.5 + 97.0 + 96.3}{3} = \boxed{\mathbf{96.3\ lb/ft^3}}$$

Maximum Index Density Calculations for Sample 3

Trial 1: Vc = 0.09991 ft^3 Rf = 1.426 in.

Mtot = 9402.5 g

Mmold = 5051.0 g

Msoil = 4351.5 g

H = Ri - Rf + Tp = 1.415" - 1.426" + 0.505" = 0.494 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.494 in.)(1/144) = 0.09183 ft^3

$$\rho\ dmax = \frac{(4351.5\ g) * (2.205\ lb/kg)}{(0.09183\ ft^3) * (1000\ g/kg)} = \mathbf{104.5\ lb/ft^3}$$

Trial 2: Vc = 0.09991 ft^3 Rf = 1.353 in.

Mtot = 9332.2 g

Mmold = 5051.0 g

Msoil = 4281.2 g

H = Ri - Rf + Tp = 1.415" - 1.353" + 0.505" = 0.567 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.567 in.)(1/144) = 0.09064 ft^3

$$\rho\ dmax = \frac{(4281.2\ g) * (2.205\ lb/kg)}{(0.09064\ ft^3) * (1000\ g/kg)} = \mathbf{104.2\ lb/ft^3}$$

Trial 3: Vc = 0.09991 ft^3 Rf = 1.353 in.

Mtot = 9313.7 g

Mmold = 5051.0 g

Msoil = 4262.7 g

H = Ri - Rf + Tp = 1.415" - 1.353" + 0.505" = 0.567 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.567 in.)(1/144) = 0.09064 ft^3

$$\rho\ dmax = \frac{(4262.7\ g) * (2.205\ lb/kg)}{(0.09064\ ft^3) * (1000\ g/kg)} = \mathbf{103.7\ lb/ft^3}$$

$$\rho\ dmax = \frac{104.5 + 104.2 + 103.7}{3} = \boxed{\mathbf{104.1\ lb/ft^3}}$$

Minimum Index Density Calculations for Sample 4

Trial 1:

V = 0.09991 ft^3

Mtot = 10349.4 g

Mmold = 5051.9 g

Msoil = 5297.5 g

$$\rho\ \text{dmin} = \frac{(5297.5\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{116.9\ lb/ft^3}$$

Trial 2:

V = 0.09991 ft^3

Mtot = 10369.7 g

Mmold = 5051.9 g

Msoil = 5317.8 g

$$\rho\ \text{dmin} = \frac{(5317.8\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{117.4\ lb/ft^3}$$

Trial 3:

V = 0.09991 ft^3

Mtot = 10332.1 g

Mmold = 5051.9 g

Msoil = 5280.2 g

$$\rho\ \text{dmin} = \frac{(5280.2\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{116.5\ lb/ft^3}$$

Trial 4:

V = 0.09991 ft^3

Mtot = 10324.6 g

Mmold = 5051.9 g

Msoil = 5272.7 g

$$\rho\ \text{dmin} = \frac{(5272.7\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{116.4\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \textbf{dmin} = \frac{116.9 + 117.4 + 116.5 + 116.4}{4} = \boxed{\mathbf{116.8\ lb/ft^3}}$$

Maximum Index Density Calculations for Sample 4

Trial 1: Vc = 0.09991 ft^3 Rf = 1.191 in.

Mtot = 10315.0 g

Mmold = 5051.9 g

Msoil = 5263.1 g

H = Ri - Rf + Tp = 1.302" - 1.191" + 0.505" = 0.616 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.616 in.)(1/144) = 0.08984 ft^3

$$\rho\ dmax = \frac{(5263.1\ g) * (2.205\ lb/kg)}{(0.08984\ ft^3) * (1000\ g/kg)} = \mathbf{129.2\ lb/ft^3}$$

Trial 2: Vc = 0.09991 ft^3 Rf = 1.180 in.

Mtot = 10332.7 g

Mmold = 5051.9 g

Msoil = 5280.8 g

H = Ri - Rf + Tp = 1.302" - 1.180" + 0.505" = 0.627 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.627 in.)(1/144) = 0.08966 ft^3

$$\rho\ dmax = \frac{(5280.8\ g) * (2.205\ lb/kg)}{(0.08966\ ft^3) * (1000\ g/kg)} = \mathbf{129.9\ lb/ft^3}$$

Trial 3:

$V_c = 0.09991 \text{ ft}^3$ $R_f = 1.274 \text{ in.}$

$M_{tot} = 10342.0 \text{ g}$

$M_{mold} = 5051.9 \text{ g}$

$M_{soil} = 5290.1 \text{ g}$

$H = R_i - R_f + T_p = 1.302" - 1.274" + 0.505" = 0.533 \text{ in.}$

$V = V_c - (A_c * H) = (0.09991 \text{ ft}^3)-(28.255 \text{ in}^2 * 0.533 \text{ in.})(1/144) = 0.09119 \text{ ft}^3$

$$\rho_{dmax} = \frac{(5290.1 \text{ g}) * (2.205 \text{ lb/kg})}{(0.09119 \text{ ft}^3) * (1000 \text{ g/kg})} = \mathbf{127.9 \text{ lb/ft}^3}$$

Trial 4:

$V_c = 0.09991 \text{ ft}^3$ $R_f = 1.186 \text{ in.}$

$M_{tot} = 10367.8 \text{ g}$

$M_{mold} = 5051.9 \text{ g}$

$M_{soil} = 5315.9 \text{ g}$

$H = R_i - R_f + T_p = 1.302" - 1.186" + 0.505" = 0.621 \text{ in.}$

$V = V_c - (A_c * H) = (0.09991 \text{ ft}^3)-(28.255 \text{ in}^2 * 0.621 \text{ in.})(1/144) = 0.08976 \text{ ft}^3$

$$\rho_{dmax} = \frac{(5315.9 \text{ g}) * (2.205 \text{ lb/kg})}{(0.08976 \text{ ft}^3) * (1000 \text{ g/kg})} = \mathbf{130.6 \text{ lb/ft}^3}$$

$$\boldsymbol{\rho_{dmax}} = \frac{129.2 + 129.9 + 127.9 + 130.6}{4} = \boxed{\mathbf{129.4 \text{ lb/ft}^3}}$$

Minimum Index Density Calculations for Sample 5

Trial 1:

V = 0.09991 ft^3

Mtot = 1008..0 g

Mmold = 5051.7 g

Msoil = 5056.0 g

$$\rho\ dmin = \frac{(5056.0\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{111.6 lb/ft^3}$$

Trial 2:

V = 0.09991 ft^3

Mtot = 10149.1 g

Mmold = 5051.7 g

Msoil = 5097.4 g

$$\rho\ dmin = \frac{(5097.4\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{112.5 lb/ft^3}$$

Trial 3:

V = 0.09991 ft^3

Mtot = 10183.8 g

Mmold = 5051.7 g

Msoil = 5132.1 g

$$\rho\ dmin = \frac{(5132.1\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{113.3 lb/ft^3}$$

Trial 4:

V = 0.09991 ft^3

Mtot = 10188.3 g

Mmold = 5051.7 g

Msoil = 5136.6 g

$$\rho\ dmin = \frac{(5136.6\ g) * (2.205\ lb/kg)}{(0.09991\ ft^3) * (1000\ g/kg)} = \textbf{113.4 lb/ft^3}$$

$$\boldsymbol{\rho}\ \textbf{dmin} = \frac{111.6 + 112.5 + 113.3 + 113.4}{4} = \boxed{\textbf{112.7 lb/ft^3}}$$

Maximum Index Density Calculations for Sample 5

Trial 1: Vc = 0.09991 ft^3 Rf = 0.905 in.

Mtot = 10101.1 g

Mmold = 5051.7 g

Msoil = 5049.4 g

H = Ri - Rf + Tp = 1.434" - 0.905" + 0.505" = 1.034 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 1.034 in.)(1/144) = 0.08300 ft^3

$$\rho\ dmax = \frac{(5049.4\ g) * (2.205\ lb/kg)}{(0.08300\ ft^3) * (1000\ g/kg)} = \mathbf{134.1\ lb/ft^3}$$

Trial 2: Vc = 0.09991 ft^3 Rf = 0.921 in.

Mtot = 10145.0 g

Mmold = 5051.7 g

Msoil = 5093.3 g

H = Ri - Rf + Tp = 1.434" - 0.921" + 0.505" = 1.018 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 1.018 in.)(1/144) = 0.08415 ft^3

$$\rho\ dmax = \frac{(5093.3\ g) * (2.205\ lb/kg)}{(0.08415\ ft^3) * (1000\ g/kg)} = \mathbf{133.5\ lb/ft^3}$$

Trial 3: Vc = 0.09991 ft^3 Rf = 1.025 in.

Mtot = 10175.9 g

Mmold = 5051.7 g

Msoil = 5124.2 g

H = Ri - Rf + Tp = 1.434" - 1.025" + 0.505" = 0.914 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.914 in.)(1/144) = 0.08496 ft^3

$$\rho\ dmax = \frac{(5124.2\ g) * (2.205\ lb/kg)}{(0.08496\ ft^3) * (1000\ g/kg)} = \mathbf{133.0\ lb/ft^3}$$

Trial 4: Vc = 0.09991 ft^3 Rf = 1.010 in.

Mtot = 10184.4 g

Mmold = 5051.7 g

Msoil = 5132.7 g

H = Ri - Rf + Tp = 1.434" - 1.010" + 0.505" = 0.929 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.929 in.)(1/144) = 0.08472 ft^3

$$\rho\ dmax = \frac{(5132.7\ g) * (2.205\ lb/kg)}{(0.08472\ ft^3) * (1000\ g/kg)} = \mathbf{133.6\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \mathbf{dmax} = \frac{134.1 + 133.5 + 133.0 + 133.6}{4} = \boxed{\mathbf{133.6\ lb/ft^3}}$$

Minimum Index Density Calculations for Sample 6

Trial 1:

V = 0.09991 ft^3

Mtot = 9739.5 g

Mmold = 5052.0 g

Msoil = 4687.5 g

$$\rho\ \text{dmin} = \frac{(4687.5\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{103.4\ lb/ft^3}$$

Trial 2:

V = 0.09991 ft^3

Mtot = 9761.0 g

Mmold = 5052.0 g

Msoil = 4709.0 g

$$\rho\ \text{dmin} = \frac{(4709.0\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{103.9\ lb/ft^3}$$

Trial 3:

V = 0.09991 ft^3

Mtot = 9800.6 g

Mmold = 5052.0 g

Msoil = 4748.6 g

$$\rho\ \text{dmin} = \frac{(4748.6\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{104.8\ lb/ft^3}$$

Trial 4:

V = 0.09991 ft^3

Mtot = 9821.4 g

Mmold = 5052.0 g

Msoil = 4769.4 g

$$\rho\ \text{dmin} = \frac{(4769.4\ \text{g}) * (2.205\ \text{lb/kg})}{(0.09991\ \text{ft}^3) * (1000\ \text{g/kg})} = \mathbf{105.3\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \textbf{dmin} = \frac{103.4 + 103.9 + 104.8 + 105.3}{4} = \boxed{\mathbf{104.3\ lb/ft^3}}$$

Maximum Index Density Calculations for Sample 6

Trial 1:

Vc = 0.09991 ft^3 Rf = 0.947 in.

Mtot = 9737.8 g

Mmold = 5052.0 g

Msoil = 4685.8 g

H = Ri - Rf + Tp = 1.273" - 0.947" + 0.505" = 0.831 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.831 in.)(1/144) = 0.08632 ft^3

$$\rho\ dmax = \frac{(4685.8\ g) * (2.205\ lb/kg)}{(0.08632\ ft^3) * (1000\ g/kg)} = \mathbf{119.7\ lb/ft^3}$$

Trial 2:

Vc = 0.09991 ft^3 Rf = 0.925 in.

Mtot = 9759.3 g

Mmold = 5052.0 g

Msoil = 4707.3 g

H = Ri - Rf + Tp = 1.273" - 0.925" + 0.505" = 0.853 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.853 in.)(1/144) = 0.08596 ft^3

$$\rho\ dmax = \frac{(4707.3\ g) * (2.205\ lb/kg)}{(0.08596\ ft^3) * (1000\ g/kg)} = \mathbf{120.7\ lb/ft^3}$$

Trial 3: Vc = 0.09991 ft^3 Rf = 0.971 in.

Mtot = 9798.7 g

Mmold = 5052.0 g

Msoil = 4746.7 g

H = Ri - Rf + Tp = 1.273" - 0.971" + 0.505" = 0.807 in.

V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.807 in.)(1/144) = 0.08671 ft^3

$$\rho\ dmax = \frac{(4746.7\ g) * (2.205\ lb/kg)}{(0.08671\ ft^3) * (1000\ g/kg)} = \mathbf{120.7\ lb/ft^3}$$

Trial 4: Vc = 0.09991 ft^3 Rf = 0.980 in.

Mtot = 9819.7 g

Mmold = 5052.0 g

Msoil = 4767.7 g

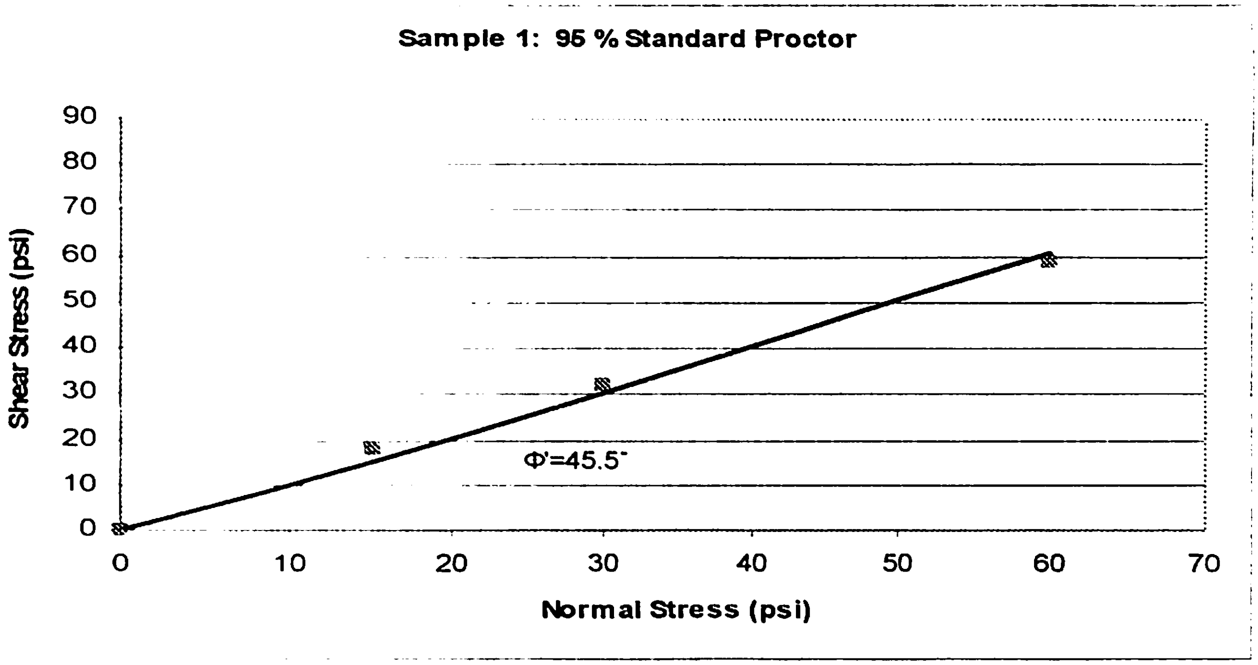
H = Ri - Rf + Tp = 1.273" - 0.980" + 0.505" = 0.798 in.

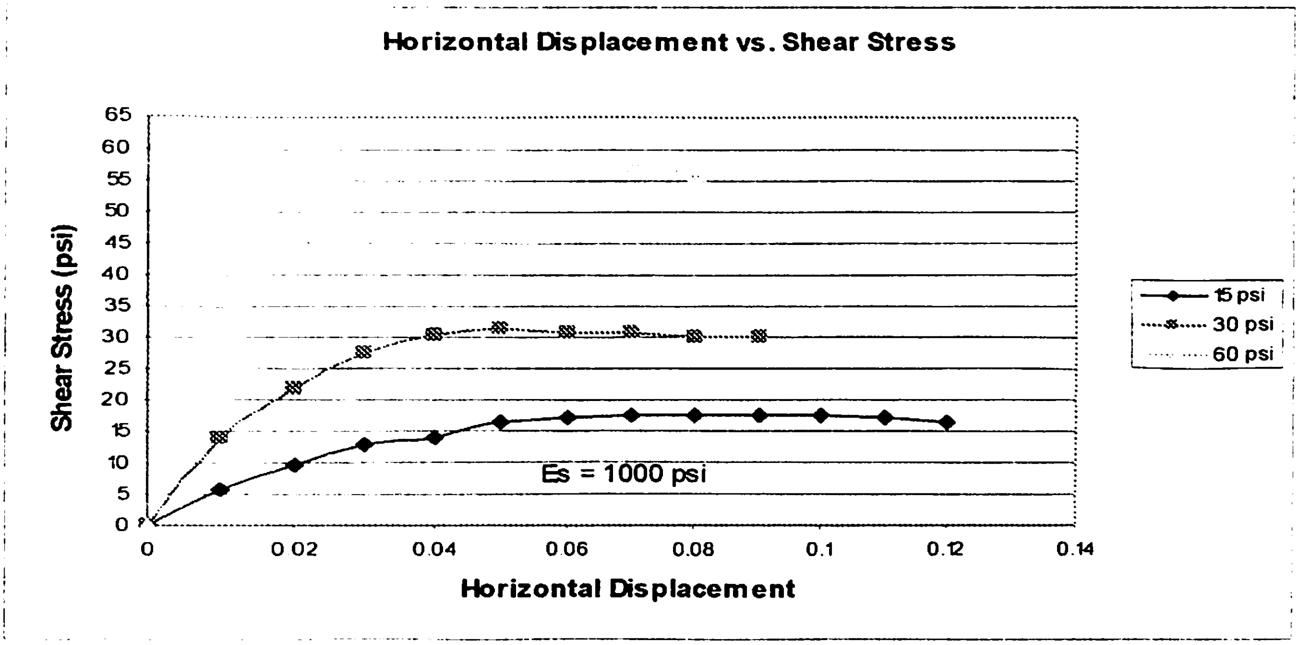
V = Vc - (Ac * H) = (0.09991 ft^3)-(28.255 in^2 * 0.798 in.)(1/144) = 0.08686 ft^3

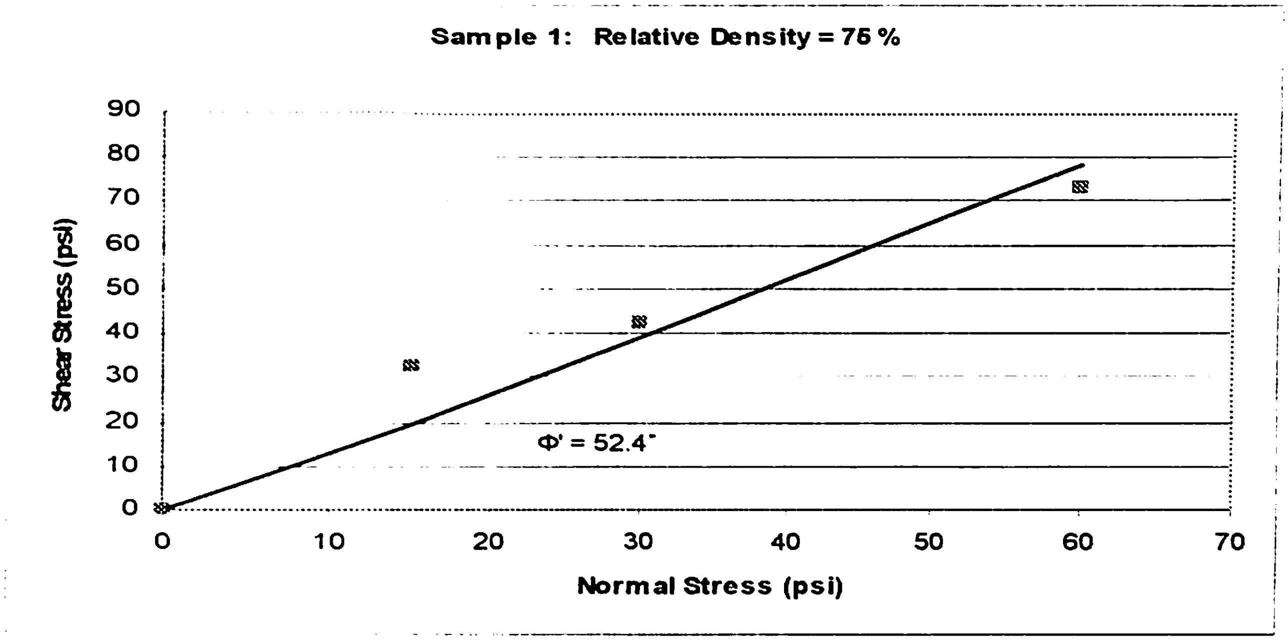
$$\rho\ dmax = \frac{(4767.7\ g) * (2.205\ lb/kg)}{(0.08686\ ft^3) * (1000\ g/kg)} = \mathbf{121.0\ lb/ft^3}$$

$$\boldsymbol{\rho}\ \mathbf{dmax} = \frac{119.7 + 120.7 + 120.7 + 121.0}{4} = \boxed{120.5\ lb/ft^3}$$

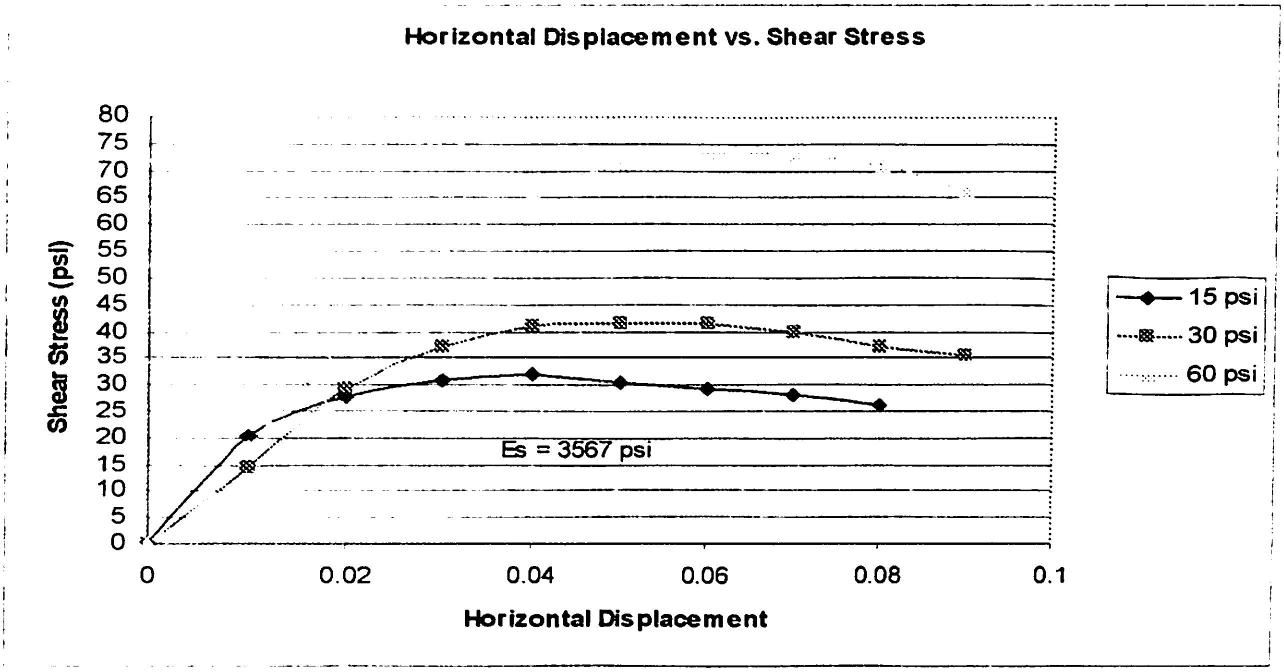
Appendix E
Direct Shear Data


Sample 1: 95 % Standard Proctor
Shear Stress (psi)
Normal Stress (psi)
Φ'=45.5˚
0
10
20
30
40
50
60
70
80
90

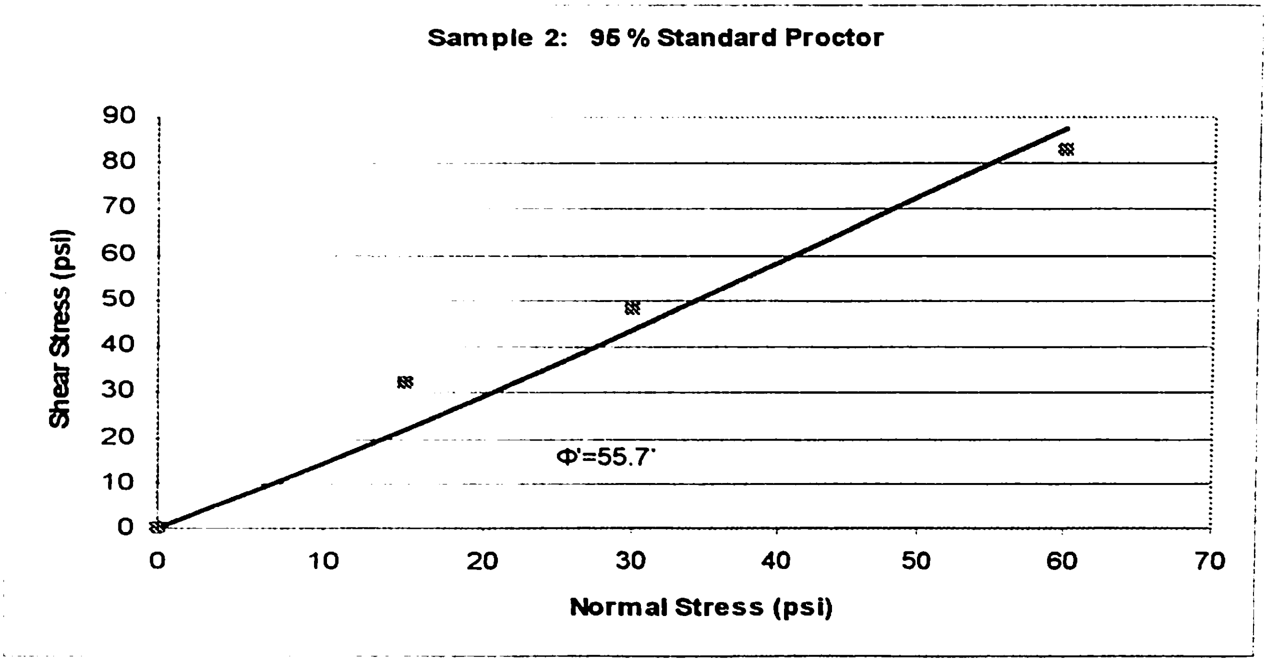

Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement
Es = 1000 psi
15 psi
30 psi
60 psi
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14



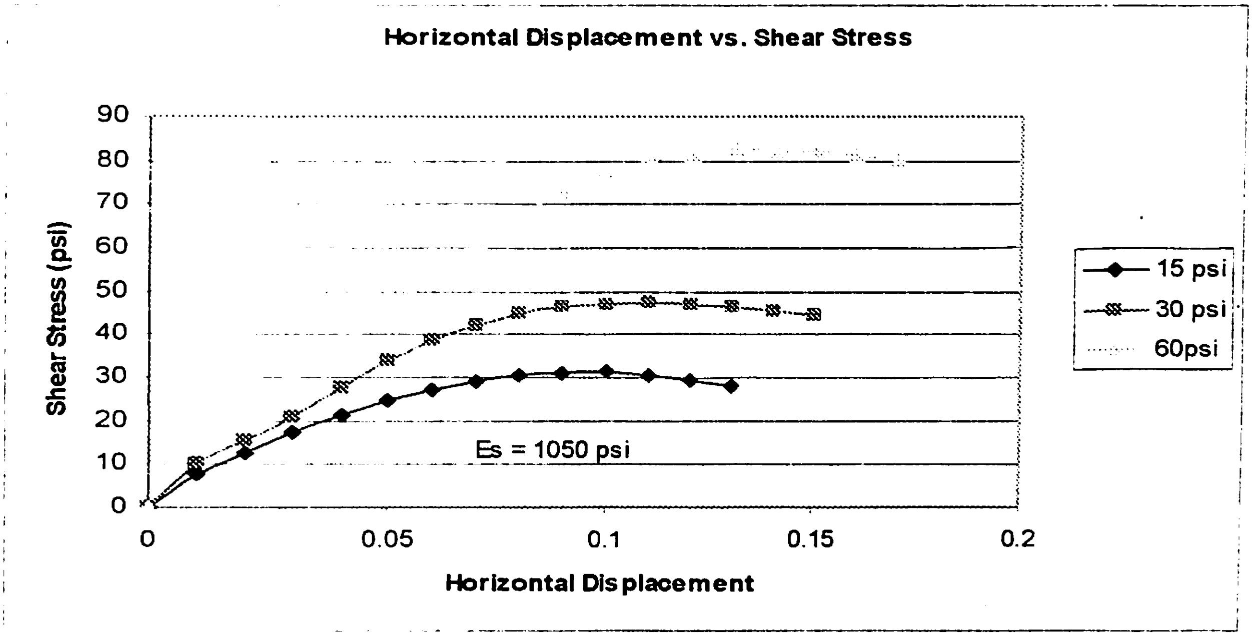
Sample 1: Relative Density = 75 %
Φ' = 52.4˚
Shear Stress (psi)
Normal Stress (psi)
0
10
20
30
40
50
60
70
80
90



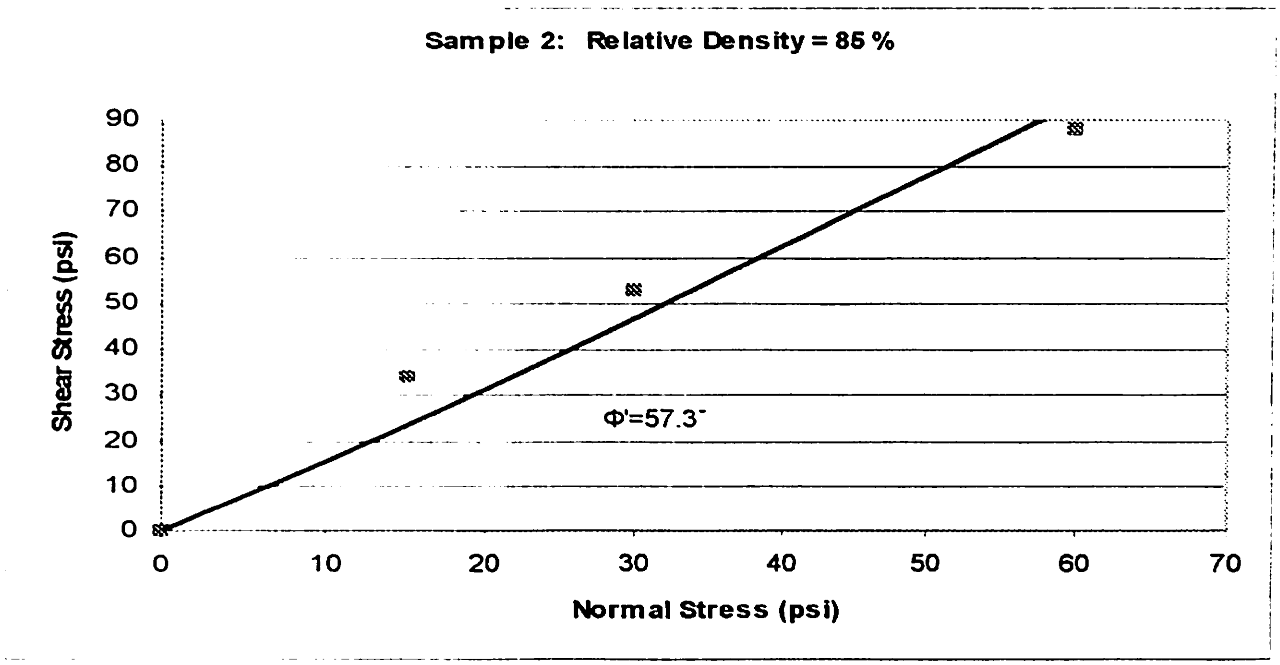
Horizontal Displacement vs. Shear Stress
Es = 3567 psi
Shear Stress (psi)
Horizontal Displacement
0
0.02
0.04
0.06
0.08
0.1
15 psi
30 psi
60 psi

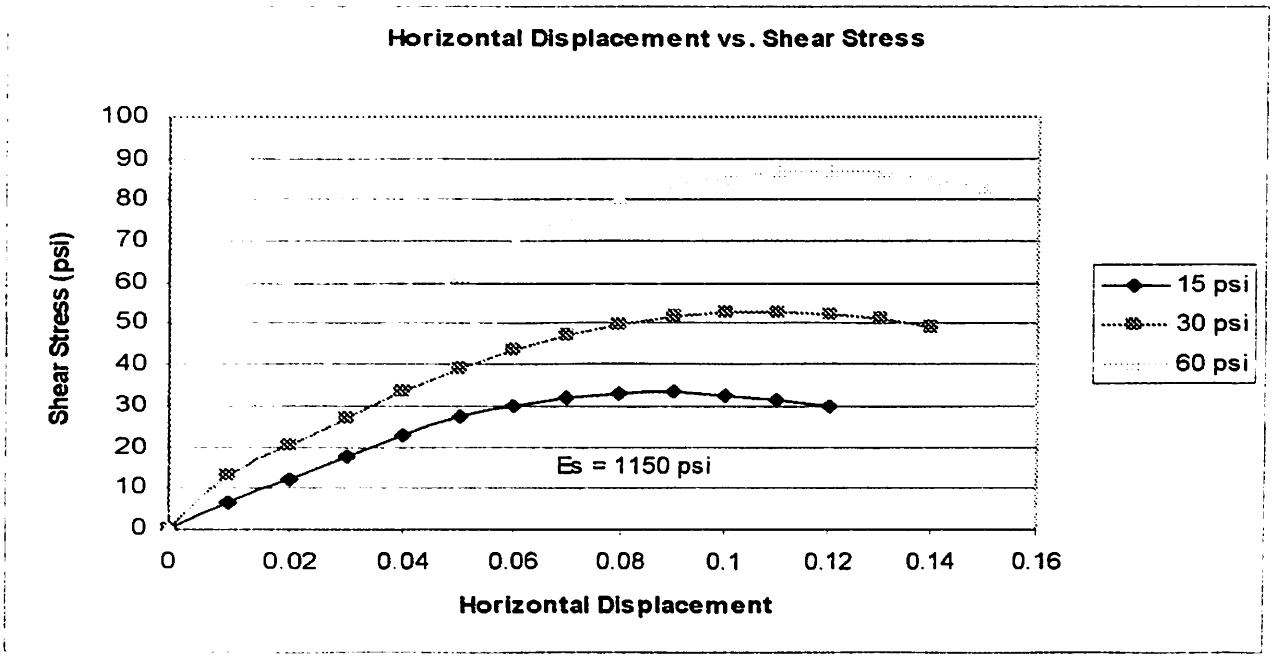


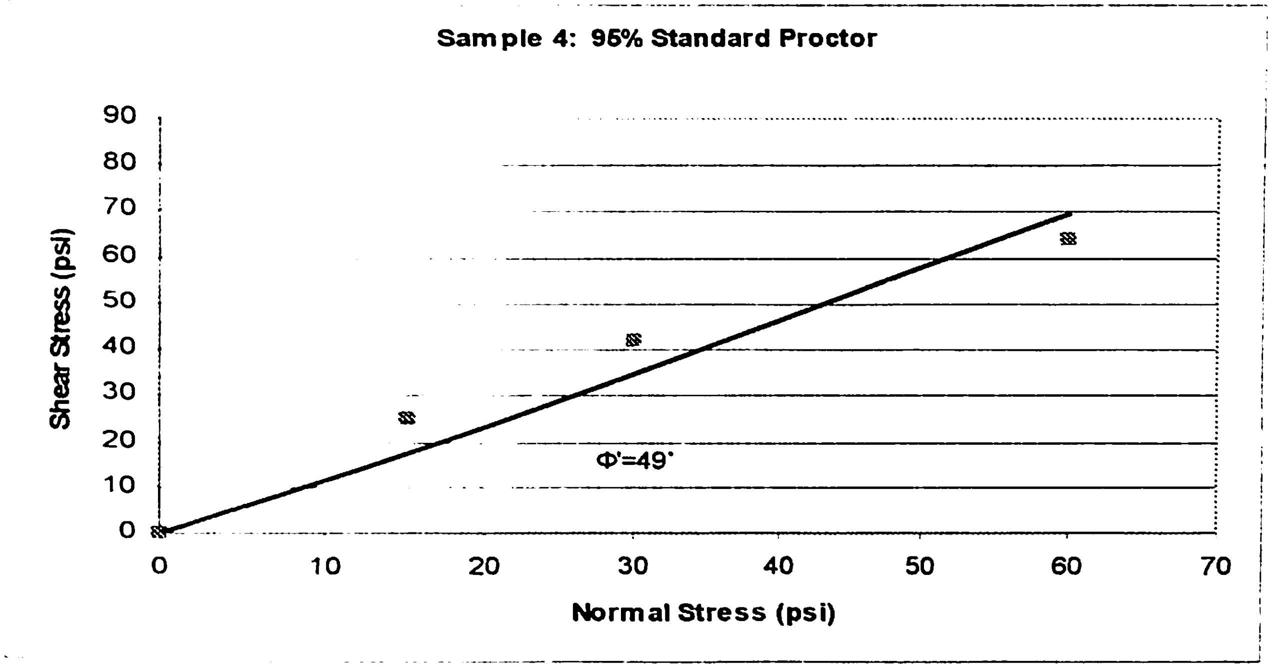
Sample 2: 95 % Standard Proctor
Shear Stress (psi)
Normal Stress (psi)
Φ'=55.7°
0
10
20
30
40
50
60
70
80
90

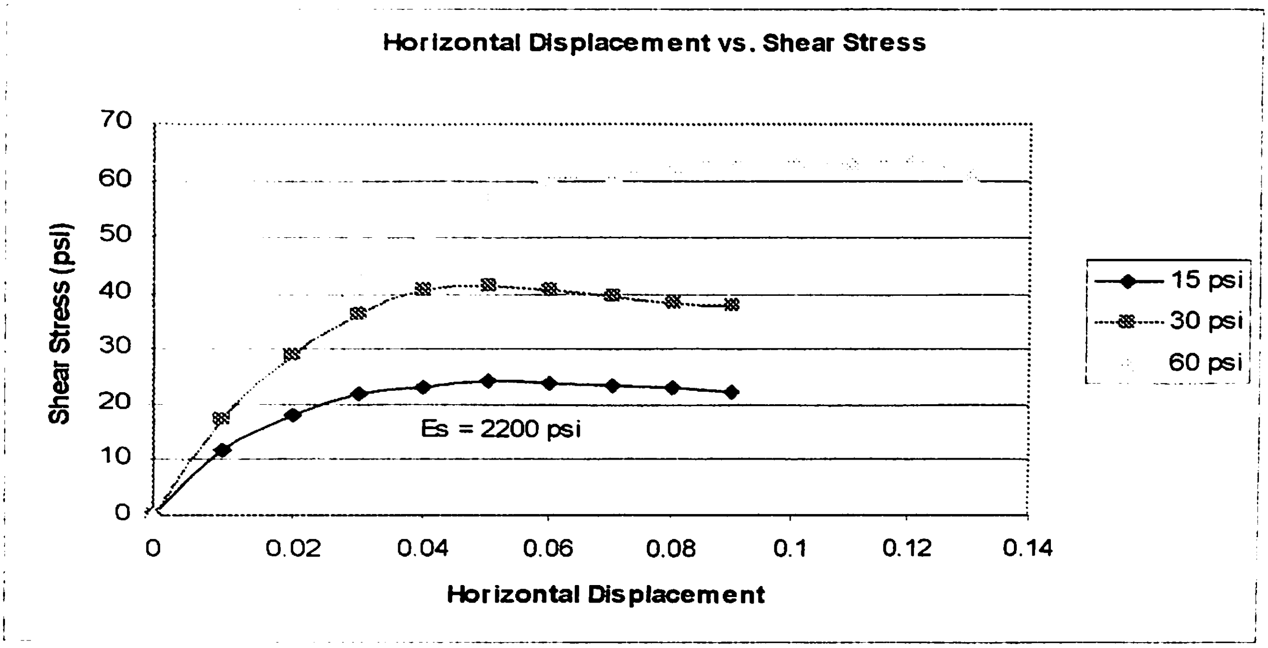


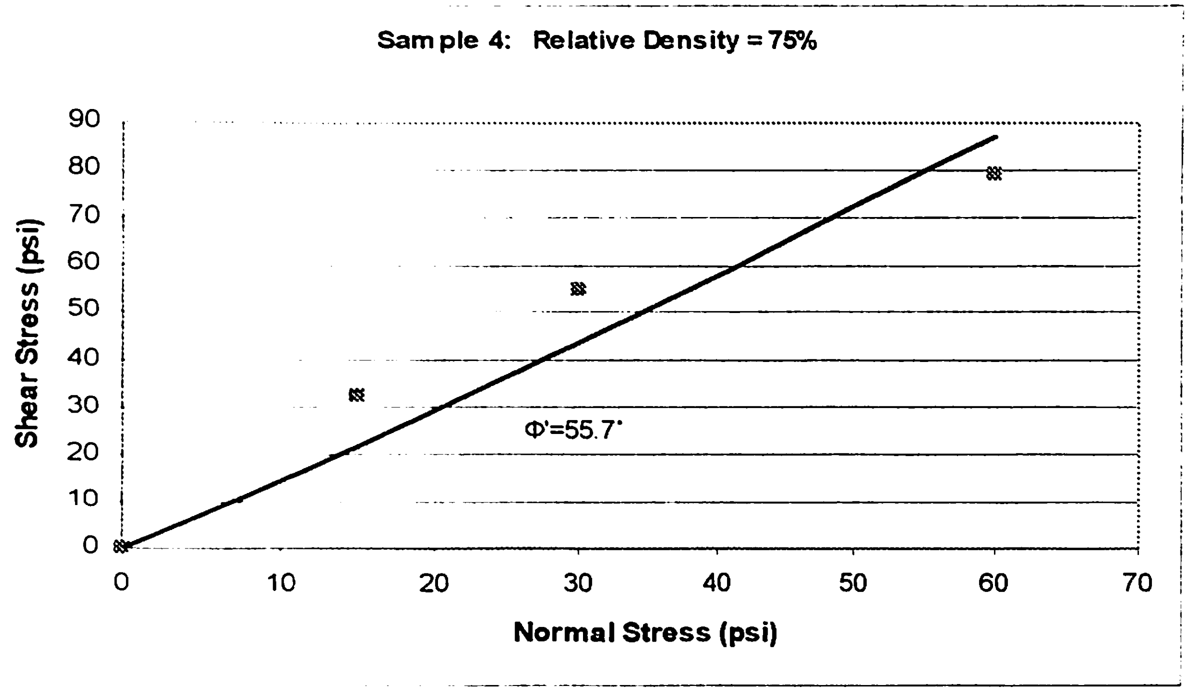
Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement
Es = 1050 psi
15 psi
30 psi
60psi
0
0.05
0.1
0.15
0.2


Sample 2: Relative Density = 85 %
Shear Stress (psi)
Normal Stress (psi)
Φ'=57.3˚
0
10
20
30
40
50
60
70
80
90

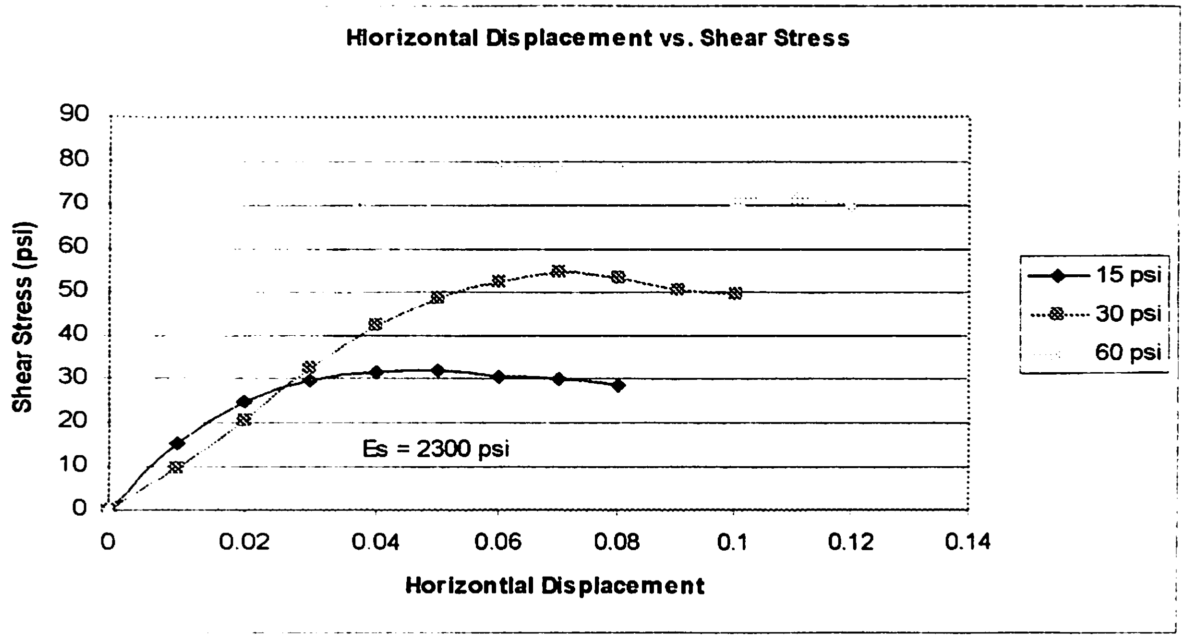

Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement
Es = 1150 psi
15 psi
30 psi
60 psi
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
0.16
10
20
30
40
50
60
70
80
90
100


Sample 4: 95% Standard Proctor
Φ'=49°
Shear Stress (psi)
Normal Stress (psi)
0
10
20
30
40
50
60
70
80
90

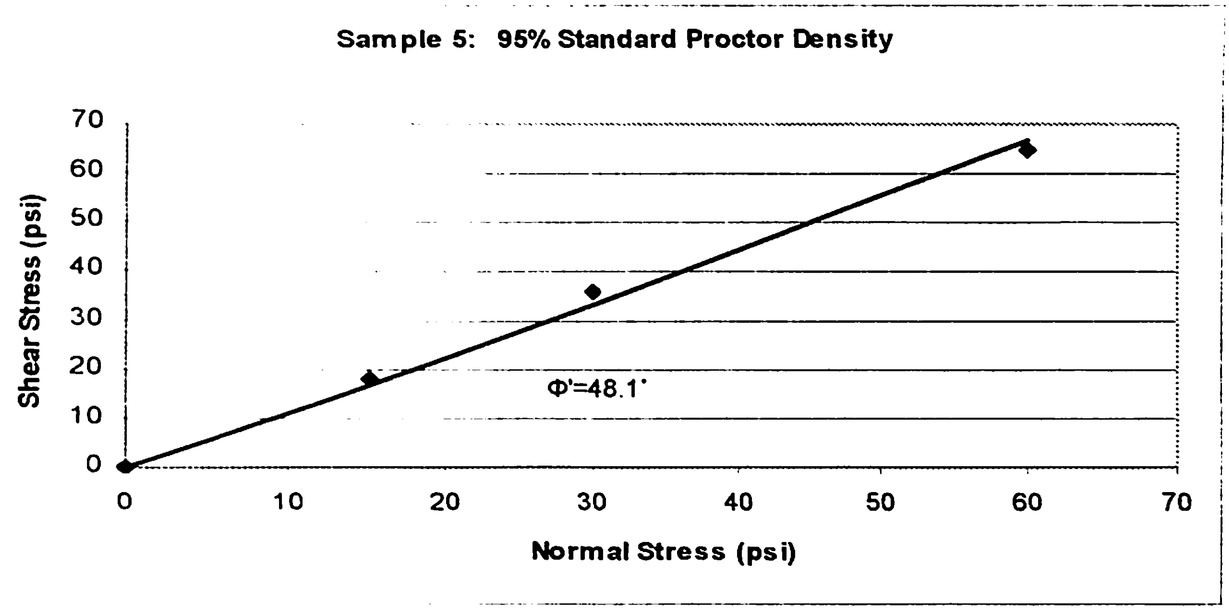

Horizontal Displacement vs. Shear Stress
Es = 2200 psi
Shear Stress (psi)
Horizontal Displacement
15 psi
30 psi
60 psi
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14



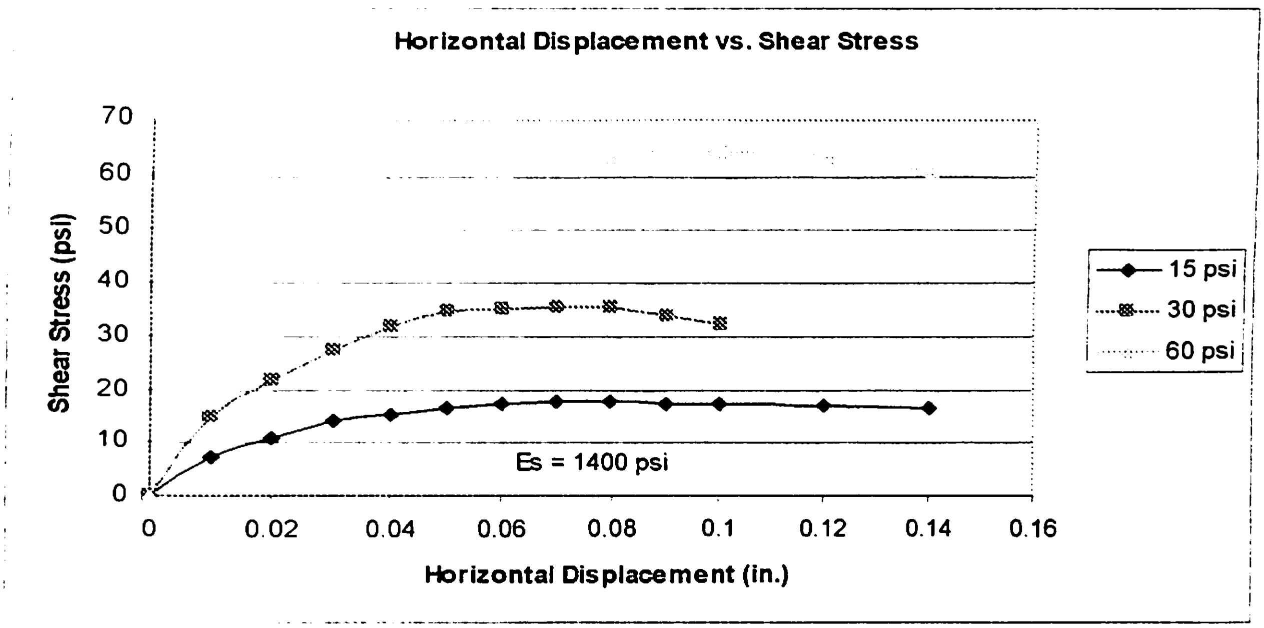
Sample 4: Relative Density = 75%
Shear Stress (psi)
Normal Stress (psi)
Φ'=55.7°



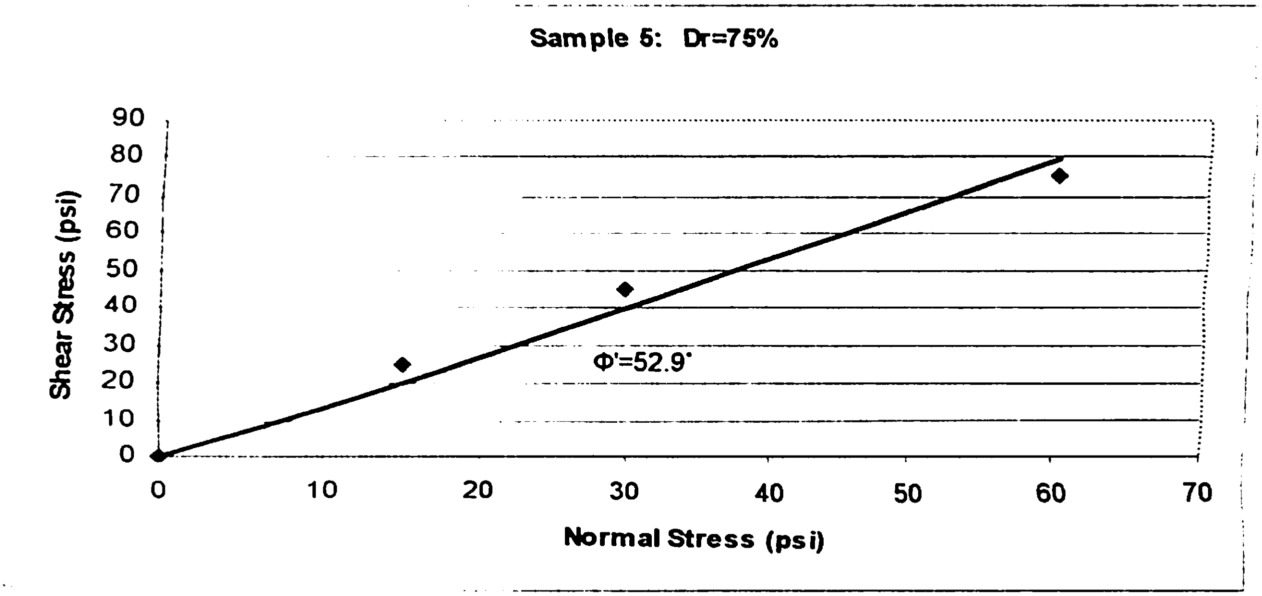
Hiorizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontial Displacement
Es = 2300 psi
15 psi
30 psi
60 psi



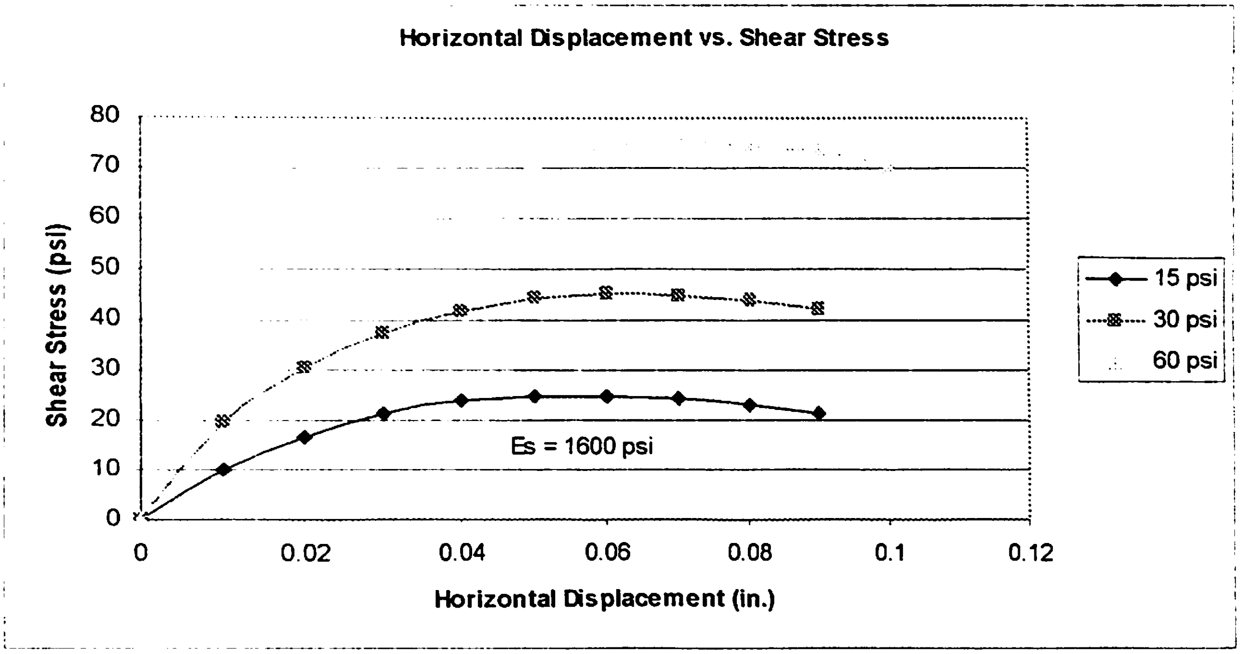
Sample 5: 95% Standard Proctor Density
Shear Stress (psi)
Normal Stress (psi)
Φ'=48.1˚
0
10
20
30
40
50
60
70



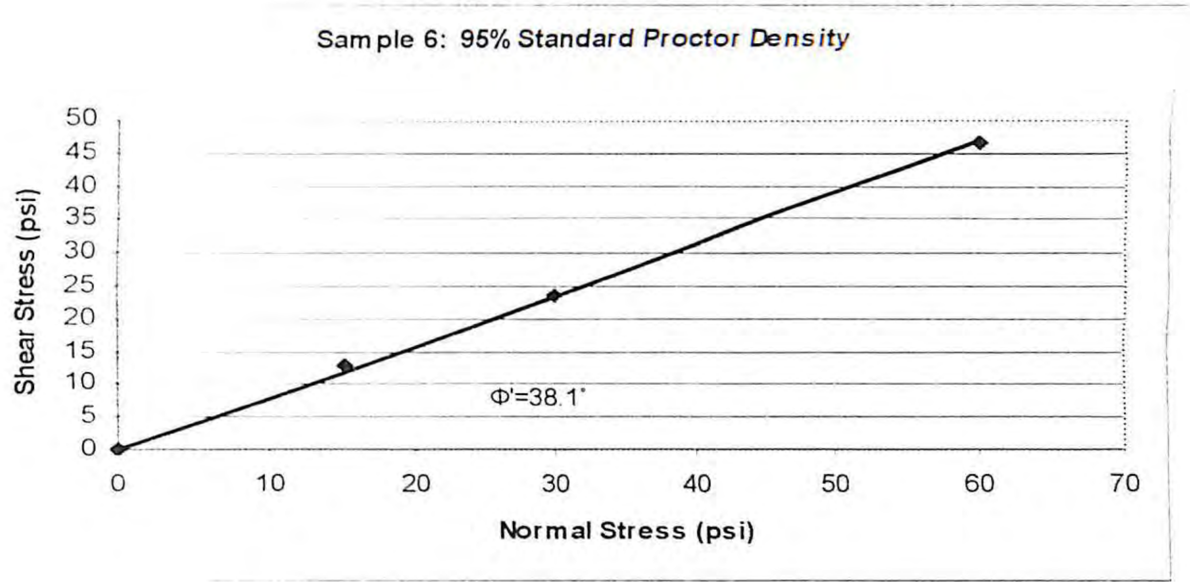
Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement (in.)
Es = 1400 psi
15 psi
30 psi
60 psi
0
0.02
0.04
0.06
0.08
0.1
0.12
0.14
0.16

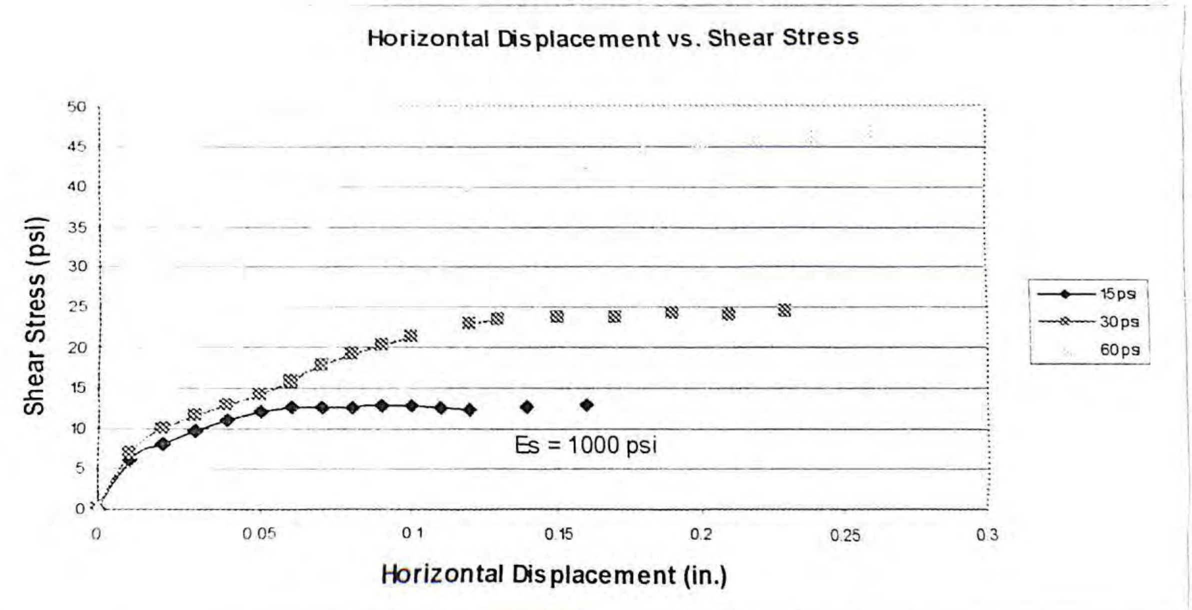


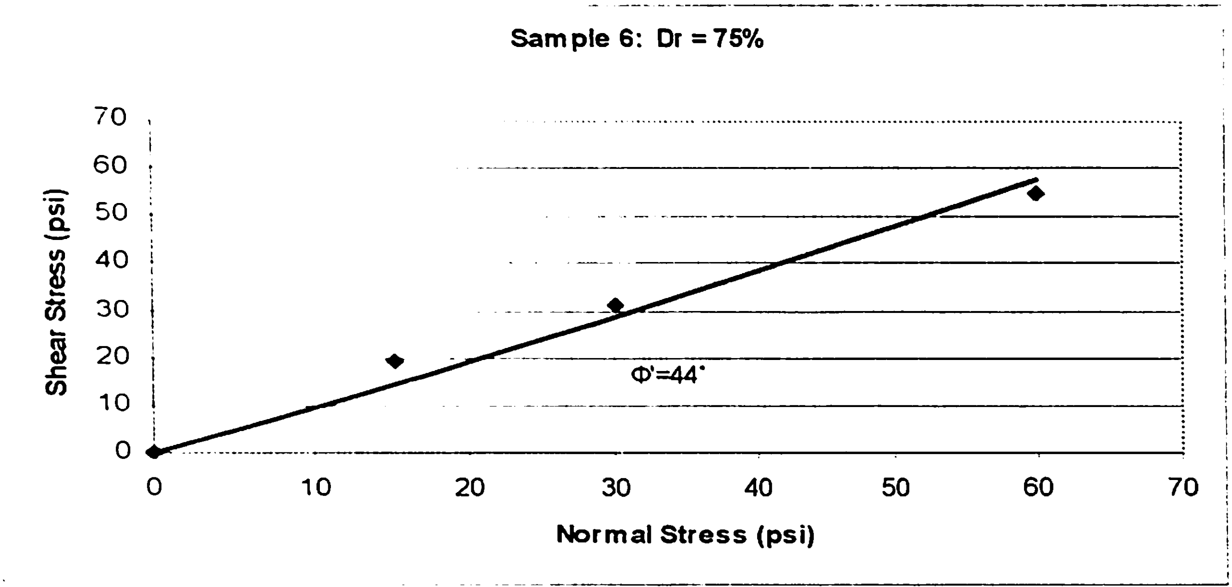
Sample 5: Dr=75%
Shear Stress (psi)
90
80
70
60
50
40
30
20
10
0
Φ'=52.9°
0
10
20
30
40
50
60
70
Normal Stress (psi)

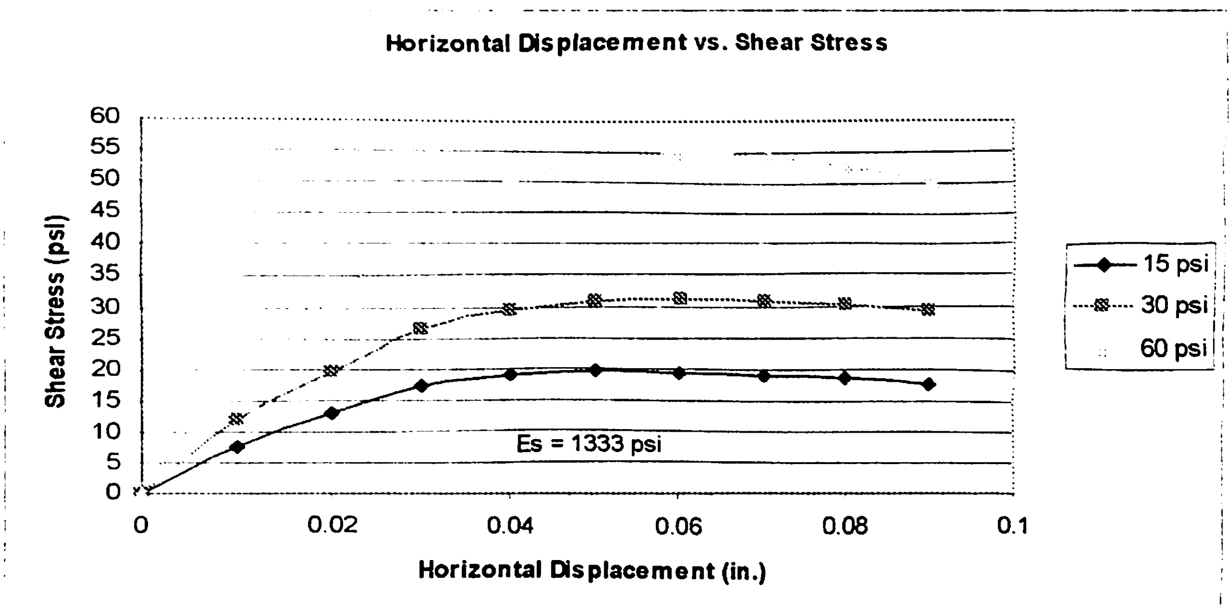


Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
80
70
60
50
40
30
20
10
0
Es = 1600 psi
0
0.02
0.04
0.06
0.08
0.1
0.12
Horizontal Displacement (in.)
15 psi
30 psi
60 psi

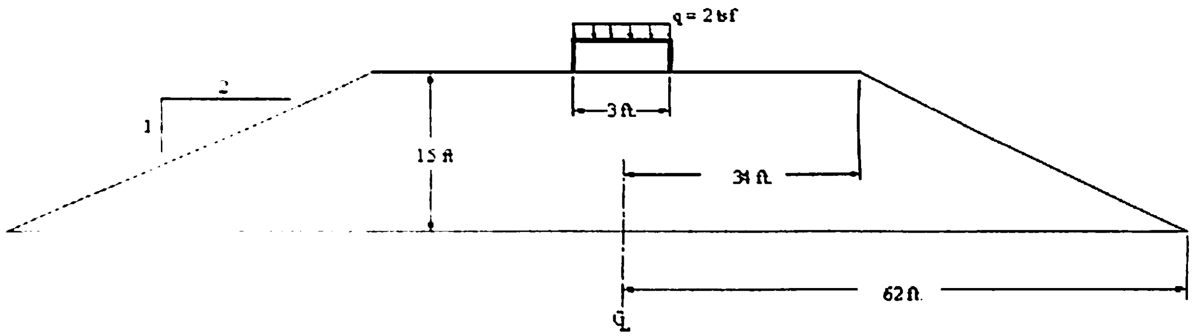

Sample 6: 95% Standard Proctor Density
Shear Stress (psi)
Normal Stress (psi)
Φ'=38.1°
0
5
10
15
20
25
30
35
40
45
50
0
10
20
30
40
50
60
70


Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement (in.)
Es = 1000 psi
15psi
30psi
60psi


Sample 6: Dr = 75%
Shear Stress (psi)
Normal Stress (psi)
Φ'=44°
0
10
20
30
40
50
60
70


Horizontal Displacement vs. Shear Stress
Shear Stress (psi)
Horizontal Displacement (in.)
Es = 1333 psi
15 psi
30 psi
60 psi
0
0.02
0.04
0.06
0.08
0.1

Appendix F
Case Study: Settlement Analysis



Cross-Section of Bridge Approach Embankment

$$S = (q)(B)(Ip)\frac{(1 - \mu^2)}{Es} \qquad \text{Eq. 4}$$

where

S = elastic settlement

q = net pressure applied

B = width of foundation

μ = Poisson's ratio

Es = modulus of elasticity for soil

Ip = nondimentional influence factor

Case 1: Pavement Load

Settlement Calculations for Sample 1

95% $\gamma dmax$:

$q = 400$ psf $\mu = 0.30$ $Es = 144{,}000$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.3)^{\wedge}2)}{144{,}000 \text{ psf}} * (1.12) = \boxed{\mathbf{0.068 \text{ ft.} = 0.82 \text{ in.}}}$$

Dr = 75% :

$q = 400$ psf $\mu = 0.37$ $Es = 513{,}648$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.37)^{\wedge}2)}{513{,}648 \text{ psf}} * (1.12) = \boxed{\mathbf{0.017 \text{ ft.} = 0.21 \text{ in.}}}$$

Settlement Calculations for Sample 2

95% $\gamma dmax$:

$q = 400$ psf $\mu = 0.32$ $Es = 151{,}200$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.32)^{\wedge}2)}{151{,}200 \text{ psf}} * (1.12) = \boxed{\mathbf{0.063 \text{ ft.} = 0.76 \text{ in.}}}$$

Dr = 85% :

$q = 400$ psf $\mu = 0.37$ $Es = 165{,}600$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.37)^{\wedge}2)}{165{,}600 \text{ psf}} * (1.12) = \boxed{\mathbf{0.056 \text{ ft.} = 0.67 \text{ in.}}}$$

Settlement Calculations for Sample 4

95% $\gamma dmax$:

q = 400 psf $\mu = 0.30$ Es = 316,800 psf

B = 24 ft.   Ip = 1.12

$$S = (400\ psf)(24\ ft)*\frac{(1 - (0.3)^2)}{316{,}800\ psf} * (1.12) = \boxed{0.032\ ft. = 0.38\ in.}$$

Dr = 75% :

q = 400 psf $\mu = 0.37$ Es = 331,200 psf

B = 24 ft.   Ip = 1.12

$$S = (400\ psf)(24\ ft)*\frac{(1 - (0.37)^2)}{331{,}200\ psf} * (1.12) = \boxed{0.028\ ft. = 0.34\ in.}$$

Settlement Calculations for Sample 5

95% $\gamma dmax$:

q = 400 psf $\mu = 0.30$ Es = 201,600 psf

B = 24 ft.   Ip = 1.12

$$S = (400\ psf)(24\ ft)*\frac{(1 - (0.30)^2)}{201{,}600\ psf} * (1.12) = \boxed{0.048\ ft. = 0.58\ in.}$$

Dr = 75% :

q = 400 psf $\mu = 0.37$ Es = 230,400 psf

B = 24 ft.   Ip = 1.12

$$S = (400\ psf)(24\ ft)*\frac{(1 - (0.37)^2)}{230{,}400\ psf} * (1.12) = \boxed{0.040\ ft. = 0.48\ in.}$$

Settlement Calculations for Sample 6

95% γ_{dmax}:

q = 400 psf $\qquad$ $\mu = 0.30$ $\qquad$ Es = 144,000 psf

B = 24 ft. $\qquad$ Ip = 1.12

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1-(0.30)^{\wedge}2)}{144{,}000 \text{ psf}} * (1.12) = \boxed{0.068 \text{ ft.} = 0.82 \text{ in.}}$$

Dr = 75% :

q = 400 psf $\qquad$ $\mu = 0.37$ $\qquad$ Es = 191,952 psf

B = 24 ft. $\qquad$ Ip = 1.12

$$S = (400 \text{ psf})(24 \text{ ft})*\frac{(1-(0.37)^{\wedge}2)}{191{,}952 \text{ psf}} * (1.12) = \boxed{0.048 \text{ ft.} = 0.58 \text{ in.}}$$

Case 2: Pavement Load + 18 Kip Single Axle Load

Settlement Calculations for Sample 1

95% γdmax:

$q = 1150$ psf $\mu = 0.30$ Es = 144,000 psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})* \frac{(1-(0.3)\text{^}2)}{144{,}000\text{ psf}} * (1.12) = \boxed{0.196\text{ ft.} = 2.35\text{ in.}}$$

Dr = 75% :

$q = 1150$ psf $\mu = 0.37$ Es = 513,648 psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})* \frac{(1-(0.37)\text{^}2)}{513{,}648\text{ psf}} * (1.12) = \boxed{0.052\text{ ft.} = 0.63\text{ in.}}$$

Settlement Calculations for Sample 2

95% γdmax:

$q = 1150$ psf $\mu = 0.32$ Es = 151,200 psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})* \frac{(1-(0.32)\text{^}2)}{151{,}200\text{ psf}} * (1.12) = \boxed{0.184\text{ ft.} = 2.21\text{ in.}}$$

Dr = 85% :

$q = 1150$ psf $\mu = 0.37$ Es = 165,600 psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})* \frac{(1-(0.37)\text{^}2)}{165{,}600\text{ psf}} * (1.12) = \boxed{0.162\text{ ft.} = 1.94\text{ in.}}$$

Settlement Calculations for Sample 4

95% γdmax:

$q = 1150$ psf $\mu = 0.30$ $Es = 316{,}800$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})*\frac{(1-(0.3)^{\wedge}2)}{316{,}800\text{ psf}} * (1.12) = \boxed{0.088\text{ ft.} = 1.06\text{ in.}}$$

Dr = 75% :

$q = 1150$ psf $\mu = 0.37$ $Es = 331{,}200$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})*\frac{(1-(0.37)^{\wedge}2)}{331{,}200\text{ psf}} * (1.12) = \boxed{0.080\text{ ft.} = 0.96\text{ in.}}$$

Settlement Calculations for Sample 5

95% γdmax:

$q = 1150$ psf $\mu = 0.30$ $Es = 201{,}600$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})*\frac{(1-(0.30)^{\wedge}2)}{201{,}600\text{ psf}} * (1.12) = \boxed{0.140\text{ ft.} = 1.68\text{ in.}}$$

Dr = 75% :

$q = 1150$ psf $\mu = 0.37$ $Es = 230{,}400$ psf

$B = 24$ ft. $Ip = 1.12$

$$S = (1150\text{ psf})(24\text{ ft})*\frac{(1-(0.37)^{\wedge}2)}{230{,}400\text{ psf}} * (1.12) = \boxed{0.116\text{ ft.} = 1.39\text{ in.}}$$

Settlement Calculations for Sample 6

95% $\gamma dmax$:

q = 1150 psf $\mu = 0.30$ Es = 144,000 psf

B = 24 ft.   Ip = 1.12

$$S = (1150 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.30)^2)}{144{,}000 \text{ psf}} * (1.12) = \boxed{0.195 \text{ ft.} = 2.34 \text{ in.}}$$

Dr = 75% :

q = 1150 psf $\mu = 0.37$ Es = 191,952 psf

B = 24 ft.   Ip = 1.12

$$S = (1150 \text{ psf})(24 \text{ ft})*\frac{(1 - (0.37)^2)}{191{,}952 \text{ psf}} * (1.12) = \boxed{0.139 \text{ ft.} = 1.67 \text{ in.}}$$

VITA

William Micah Siemers

Candidate for the Degree of

Master of Science

Thesis: QUALITY CONTROL TESTING FOR GRANULAR BACKFILL MATERIALS

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Albuquerque, New Mexico, on November 24, 1978, son of Terry and Linda Siemers. Married to Katie Burden on July 14, 2000.

Education: Graduated from Bartlesville High School, Bartlesville, Oklahoma in May 1997; received Bachelor of Science degree in Civil Engineering from Oklahoma State University, Stillwater, Oklahoma in May 2003. Completed the requirements for the Master of Science Degree with a major in Civil Engineering at Oklahoma State University, December 2004.

Experience: Employed by Oklahoma State University as a teaching assistant for Geotechnical Engineering, 2003; employed by Stillwater Engineering & Consulting Inc, as an assistant surveyor, 2002-2004, employed by Oklahoma State University as a research assistant, 2003-present.

Professional Memberships: American Society of Civil Engineers