SHEAR STRENGTH, CREEP AND STABILITY

OF FIBER-REINFORCED SOIL SLOPES

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NOMENCLATURE

ASTM = American Society for Testing and Materials

$c =$ cohesion of raw soil

$\Delta c_{frs} =$ corrected apparent increase in cohesive strength due to fiber

$c_{frs} =$ the cohesion value of FRS

$d =$ diameter of fiber, or equivalent diameter for non-circular fiber

FRS = fiber-reinforced soil

FS = factor of safety

$f_c =$ interaction coefficient related to the cohesive component of the shear strength

$f_\phi =$ interaction coefficient related to the frictional component of the shear strength (sometimes referred to as $f_\phi \tan\phi =$ tan δ)

$G_s =$ Specific Gravity of fiber material

$K_0 =$ At-rest earth pressure coefficient

$l =$ length of fiber

L_e = effective length of an individual fiber

$n_f =$ average number of fibers per unit volume

NP = non plastic

$N_f =$ number of fibers intersecting the shear plane

pcf = pounds per cubic foot

Nomenclature (continued)

τ_{frsc} = uncorrected apparent increase in cohesive shear strength due to fiber

$\tau_{frs\phi}$ = apparent increase in frictional shear strength due to fiber

$\tan\phi_{frs}$ = tangent of the friction angle for FRS

V_r = fiber volume ratio (ratio of fiber volume to total volume of a unit mass of FRS)

W_f = weight of fibers in a unit volume of FRS

z = depth below ground surface

γ = soil unit weight

γ_w = unit weight of water

ϕ = angle of shearing resistance of raw soil

σ_h or σ_h = horizontal stress

σ_v or σ_v = vertical stress

CHAPTER I

INTRODUCTION

Background

The concept of fiber-reinforced soil (FRS) dates to ancient times when clay bricks were reinforced with straw fibers. This concept is also similar to natural root reinforcement of soil where micro root structure increases the apparent shear strength of the root reinforced zone compared to similar soil with no root structure. The use of geosynthetics (synthetic plastic materials) for reinforcement of soil structures has become well established in the past 20 years. The geosynthetic reinforcement materials initially consisted mostly of geotextiles and geogrids, often referred to as planar reinforcement. Techniques for design and analysis of earth structures reinforced with planar geosynthetics are well developed, and have been presented extensively in the literature.

The rapid increase in the use of planar geosynthetics led to the concept and development of synthetic fibers for soil reinforcement. The concept of using short synthetic fibers for soil reinforcement was the subject of several early research studies and was discussed in the literature (Andersland and Khattak, 1979; Hoare, 1979; Gray and Ohashi, 1983). However, short synthetic fibers for soil reinforcement were not commercially available until about 1990 when a pilot program of fiber research, production, and full-scale test projects was undertaken by a major geosynthetics manufacturer in the United States (Synthetic Industries, 1990). The author became involved in numerous projects consisting of fiber-reinforced embankments and related laboratory testing in 1994. Fiber-reinforced soil (FRS) has been used successfully on more than 50 embankment slopes in the United States in the past 15 years (Gregory and Chill, 1998, Gregory, 1999b, Chill 2006). The author has been involved in more that 15 of the FRS projects. The geosynthetic fiber reinforcement has consisted of 1-inch to 2.75-inch (25- to 70-mm) length polypropylene fibers. These fibers, when mixed into the soil, significantly increase the apparent shear strength of the entire soil mass. An FRS mixture is illustrated in Figure 1.

Although a significant number of FRS projects have been completed and numerous research papers have been presented and published, the reinforcement mechanisms of the fibers have not been well understood and a widely accepted design methodology has not been developed.

Scope of Research Study

The current research study for this dissertation consisted of review of available related literature, an extensive laboratory testing program of FRS including tests on a fat clay soil and a non-plastic silty sand, refinement and extension of an FRS design model previously proposed by the author, and presentation of two recent case histories of actual large projects utilizing FRS. The laboratory testing program included both shear strength testing and creep testing of FRS, as more fully described in Chapter IV. A theoretical model is presented which can be used to mathematically calculate the improved shear strength of the raw soil when

Figure 1. FRS Mixture with Sand

reinforced with fibers, referred to as the FRS (fiber-reinforced soil) shear strength. The model includes a unique effective normal stress formulation based upon 3-dimensional random orientation of the fibers under geostatic stress conditions in a half-space continuum (soil mass). The model utilizes a mathematically derived “effective aspect ratio,” a_{re}, which is different than the conventional aspect ratio based upon the actual fiber length-equivalent diameter ratio. The input to the model includes the fiber volume ratio (ratio of fiber volume to total volume of a unit mass of FRS), unique effective stress variable, effective aspect ratio, frictional and adhesion interaction coefficients, and the non-reinforced soil shear-strength parameters ϕ and c. The model was calibrated and confirmed based upon comparison of calculated results and actual laboratory shear strength test results performed during this study.

FRS specimens can be tested for shear-strength properties, using conventional geotechnical-laboratory triaxial shear and modified direct shear testing equipment. The triaxial test is a higher quality test and is preferred over the direct shear test in most cases. The apparent increase in shear strength can be determined by comparing test results from both non-reinforced and fiber-reinforced specimens. However, it is often not practical to perform triaxial tests on FRS materials for smaller, non-critical projects, or for preliminary design or analysis of larger or more critical projects. Often, the shear strength parameters of non-reinforced ("raw") soil are known, or can be estimated with reasonable accuracy from previous testing and experience with similar soils in the project area. An analytical model previously proposed in preliminary form by the author that can predict the increase in shear strength resulting from fiber reinforcement, based upon the raw soil and fiber properties, was extended and refined during this study. The model is described and discussed in detail in Chapter III.

Format of Dissertation

The dissertation is presented in eight chapters. Chapter I (current chapter) contains the introduction to the dissertation. Brief descriptions of Chapters II through VIII are provided below.

Chapter II – Literature Research – Chapter II includes a discussion of related literature reviewed during the current research study, including 21 journal papers and professional reports. Four of the journal papers that are directly relevant to the current research are summarized in Chapter II. Two previous related studies consisting of laboratory testing of FRS are also discussed in the chapter.

Chapter III – Conceptual Model – The purpose of the model is to mathematically calculate the shear strength of FRS without having to perform laboratory tests on FRS specimens. Chapter III contains documentation of the development of the conceptual model, including the final form of the equations for calculating the shear strength of FRS.

Chapter IV – Laboratory Testing Program – Chapter IV describes the laboratory testing program, and the laboratory test reports are included in Appendix A. The testing program included a clay soil and a silty sand soil. The tests performed included moisture-density relationship tests, Atterberg Limits tests, sieve tests, triaxial shear tests, direct shear tests, and constant load direct shear creep tests. The test series included non-reinforced specimens and fiber-reinforced specimens. Interface shear tests were also performed to determine the interaction coefficients between the soil and the plastic material from which the fibers are made.

Chapter V – Correlation and Analysis of Data – Chapter V includes summaries of the laboratory test results, analysis of the test data, and correlation with the conceptual model. The actual laboratory test results are compared with predictions from the model by performing statistical analysis of the data to obtain correlation coefficient (R^2) values for both frictional and cohesive components of the FRS shear strength values. The results indicate that the model predicts the FRS shear strength within an acceptable and practical range of accuracy compared to actual laboratory test results.

The shear strength test results indicate “decay” in the increase of shear strength with fiber contents greater than about 0.5 pcf (8 kg/m^3). The test results were used to develop a decay function to reduce the interaction coefficients at higher fiber contents to account for a larger percentage of fiber-to-fiber contact rather than fiber-to-soil contact. Any significant decay in shear strength gain was found to occur at fiber contents well above any practical mixture rate.

The creep test results are plotted as deformation versus time in semi-log and arithmetic form in Chapter V. The creep tests results indicate that the FRS specimens are more resistant to creep deformation and failure than the non-reinforced specimens.

Chapter VI – Application of FRS in Slope Stability – This chapter presents information on the application of FRS for stabilizing new slopes and for repair of failed slopes. The types of slopes where FRS is most applicable are discussed. Methods for including the model in slope stability analyses of FRS are presented.

Chapter VII – Case History Projects – Case histories are presented on two actual FRS projects. These two projects include the largest and second largest use of FRS, based upon the total weight of fibers used on each project. The PGBT Turnpike project in Dallas, Texas included an FRS zone in the clay embankment slopes constructed for the new turnpike. The FRS zone was designed to reduce the potential for creep failures in the surfaces of the embankment slopes. The Lake Ridge Parkway project included FRS for repair of failed slopes on a major roadway in Grand Prairie, Texas. Details of these projects are presented in this chapter, and slope stability analyses of the non-reinforced conditions and FRS conditions are presented for comparative purposes. The computer output from the slope stability analyses are included in Appendix C.

Chapter VIII – Conclusions and Recommendations – This chapter includes a summary of conclusions concerning the laboratory test results, conceptual model development, application of the model, and FRS in project applications. A summary of the final form of the equations developed in the conceptual model is also presented. The chapter includes recommendations for future research on FRS.

Unit System Used in the Dissertation - The primary unit system used in this dissertation is the English system. The approximate metric (SI) unit equivalents are given in parenthesis immediately following the English units in the text. Only English units are used in tables, figures, test reports, and computer output.

CHAPTER II

LITERATURE RESEARCH

Related Published Literature

Research of existing published literature related to FRS included 21 journal papers and professional reports. This literature and other literature sources used during this study are listed in the Bibliography and selected pertinent publications are discussed individually in this chapter.

<u>"Mechanics of Fiber Reinforcement in Sand</u>," Gray, D. H., and H. Ohashi (1983). This is one of the earliest studies of fiber-reinforced soil that includes a mathematical model for predicting the increase in shear strength due to fiber reinforcement. The study included a series of direct shear tests in a conventional apparatus with both non-reinforced and fiber-reinforced sand. A variety of fibers were used including plastic, plant roots, and copper wire. The plastic fibers are particularly applicable to the author's current study. An interesting and important conclusion of the Gray and Ohashi work is that fiber orientation has very little effect on shear strength results. The study included tests with various orientations of fibers with respect to the shear plane and also tests with fibers randomly oriented. Although the study showed that an orientation of 60 degrees

to the shear plane was most efficient, the difference in test results for the randomly oriented fibers was small and well within test result variables. This paper also discusses the concept of critical confining pressure below which the fiber failure mode is pullout of the fibers and above which the failure mode is yield or rupture of the fibers.

"Static Response of Sand Reinforced with Randomly Distributed Fibers," Maher, M. H. and D. H. Gray (1990). This study also included a series of direct shear tests with non-reinforced and fiber-reinforced sand. Some of the fibers used in this study are very similar to the fibers used in the author's current research study. The Maher and Gray study includes a probabilistic model of fiber distribution within a spherical soil mass and number of fibers crossing a shear plane within the mass. This probabilistic model of fiber distribution was integrated into the overall model developed by the author in the current study. The Maher and Gray study concluded that shear strength is not affected by fiber orientation. Their study also showed that the shear strength increase due to fiber reinforcement is directly related to the fiber aspect ratio. This conclusion is also strongly supported by the author's current work.

"Reinforcing Sand with Strips of Reclaimed High-Density Polyethylene," Benson, C. H. and M. Khire (1994). This research also included a series of direct shear tests with sand reinforced with plastic strips (fibers) cut from recycled milk jugs. This study showed that the increase in shear strength is directly proportional to fiber aspect ratio up to the critical confining pressure. The direct shear tests on fiber-reinforced sand showed a continuous increase in shear strength well beyond the strain value where the non-reinforced sand reached peak strength. The study also determined the interface friction coefficient between the plastic fibers and sand, which was approximately 0.34 (tangent of 19 degrees).

"Probabilistic Analysis of Randomly Distributed Fiber-Reinforced Soil," Ranjan, G., R. M. Vassan, and H. D. Charan (1996). This research study included triaxial compression tests on sand and sand-fibers mixture. The fibers included plastic fibers and natural fibers. The study includes a model for prediction of shear strength with a logarithmic function based upon regression analysis of the test data. The researchers concluded that the failure mechanism is pullout of the fibers below the critical confining stress and that the strength increase is related to fiber content and aspect ratio. They also found that the gain in shear strength due to fiber reinforcement is essentially linear up to a mixture rate of approximately 2 percent of fibers by dry weight of soil, beyond which the improvement rate decreases.

The previous studies listed above, and most of the studies listed in the Bibliography (except the author's studies) deal with cohesionless granular soils and do not address clay (cohesive) soils. While improvement of sandy soils with fiber reinforcement is of significant interest, the most practical use of FRS is for clay soils since many slopes are constructed of clays and the clay soils usually provide lower long term (effective stress) shear strength than sands. Accordingly, the increase in shear strength in clays with addition of fiber reinforcement has a high potential for widespread practical use.

Related Studies

Fugro-McClelland (now known as Fugro South) performed an extensive research and project-related laboratory testing program on FRS from 1995 to 1998 in the Fort Worth, Texas office. The author was a vice president and manager of the Fort Worth office for Fugro South during the testing program. The laboratory testing program included both triaxial shear and direct shear tests and involved mostly clay soils. The results of these tests were consistent with previous related research and established the first major data base of the shear strength of fiber-reinforced clay soils.

AGT Laboratory of Chattanooga, Tennessee performed an extensive research testing program consisting of laboratory testing of fat clay, lean clay, and sand type soils with various fiber types and sizes. The study was conducted from 1998 until about 2001 and consisted of approximately 110 triaxial compression tests and related index testing. Each triaxial test consisted of a 3-specimen series for a total of approximately 330 specimens. The author was involved in several specific projects related to this testing program and also consulted with AGT Laboratory on various testing procedures and data reduction. These test results were provided to the author by the current owner of the test data and are discussed in Chapter V.

CHAPTER III

CONCEPTUAL MODEL

Utilization of Existing Data

Significant research and information related to fiber-reinforced soil and other pertinent geosynthetics data developed by others including the author, as previously discussed in Chapter II, were reviewed and utilized during the development of the proposed model. These sources are referenced in the text and in the Bibliography section following the text.

Theory

Planar materials, such as geotextiles and geogrids, provide reinforcement in the form of a tensile force at each discrete layer, as a result of tensile strength of the material and pullout resistance developed by friction and adhesion between the geosynthetic and adjacent soil (Koerner, 1994). The pullout resistance is typically calculated as the product of the overburden pressure (vertical stress), tangent ϕ (angle of shearing resistance of the soil), and a coefficient of interaction, usually between 0.6 and 0.9 for planar geosynthetics. The value obtained is doubled since the frictional component acts on both the top and bottom of the planar material. The pullout resistance is controlled by the anchorage-length behind the critical failure surface. The ultimate strength, creep, and durability properties of the planar geosynthetic must be reduced by appropriate “partial” factors of safety. The allowable tensile strength is determined based upon the allowable material properties and pullout resistance.

The reinforcement properties of the fibers are similar to those of planar geosynthetics in some aspects, but are significantly different in others. The mechanisms involved in the increased shear strength of fiber-reinforced soil are believed to include: (1) pullout resistance due to friction between individual fibers and the surrounding soil; (2) adhesion between individual fibers and the surrounding soil (in cohesive-type soil); (3) micro-bearing capacity of the soil, mobilized during pullout resistance of individual fibers looped across the shear plane; and (4) increased localized normal stress in the soil across the shear surface resulting from pullout resistance of the fibers during shearing of the soil (Gregory and Chill, 1998). The individual interaction and contribution of these mechanisms is difficult to determine. However, the combined effects can be easily determined by shear strength testing of both reinforced and non-reinforced specimens in a geotechnical engineering laboratory.

Stress Conditions

The normal stress conditions acting on an individual fiber in a soil mass due to overburden soil are significantly different than those acting on a layer of planar reinforcement, such as a geotextile. Since the planar reinforcement is placed in the embankment in an essentially horizontal orientation, the stress component from the overburden soil is the vertical stress, as expressed by Equation (1).

$$\sigma_v = \gamma z \tag{1}$$

where:

σ_v = vertical stress

γ = soil unit weight

z = depth below ground surface

The vertical stress acts on both the top and bottom of the planar geosynthetic, as illustrated in Figure 2.

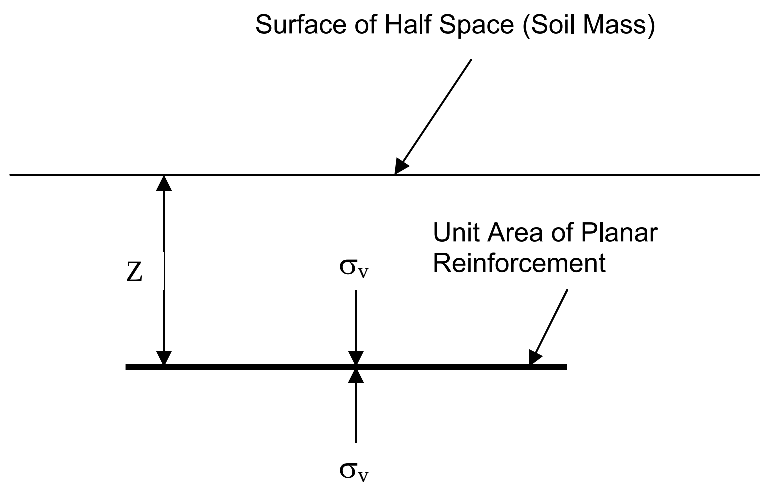

Figure 2. Normal Stress on Planar Reinforcement

In the case of FRS, an individual fiber will be randomly oriented in the soil mass, with respect to the longitudinal axis, as illustrated in Figure 2 (Gregory, 1999a).

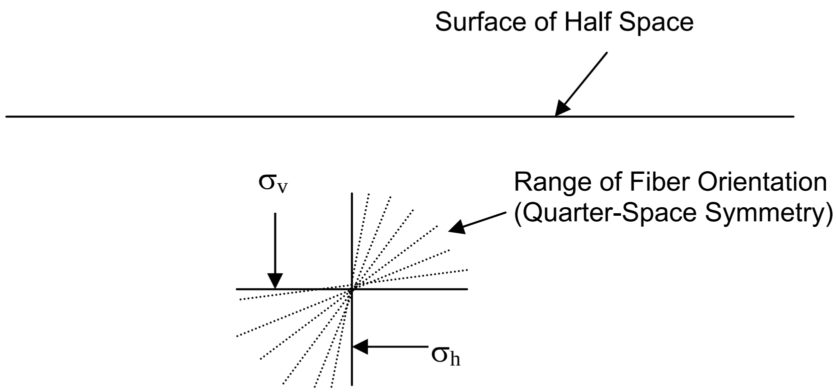

Figure 3. Range of Potential Orientation About Fiber Longitudinal Axis

This random orientation was verified experimentally by Maher and Gray (1990). If we consider the fibers to be under geostatic stress conditions in a half-space continuum (soil mass), then the average normal stress with respect to the longitudinal axis is not the vertical stress, but a combination of the vertical and horizontal stresses. As illustrated in Figure 3, vertical stress (σ_v) applies to fibers oriented horizontally, and horizontal stress (σ_h) applies to those fibers oriented vertically. If an individual fiber has essentially equal probability of being oriented vertically, horizontally, or in between (random distribution), the effective normal stress, with respect to the longitudinal axis, will be the average of the vertical and horizontal stresses. Moreover, an individual fiber of rectangular cross section should have equal probability of any orientation between vertical and horizontal with respect to the cross-sectional axis (Gregory, 1999a). Consequently, a rectangular cross-section fiber that is oriented horizontally with respect to the longitudinal axis, will be under normal stress conditions that are an average of the vertical and horizontal stresses. Square or circular fibers will also be under normal stress conditions with respect to the cross-sectional axis, which are averages of the horizontal and vertical stresses ($\frac{\sigma_h + \sigma_v}{2}$). This stress condition is illustrated in Figure 4 (Gregory, 1999a).

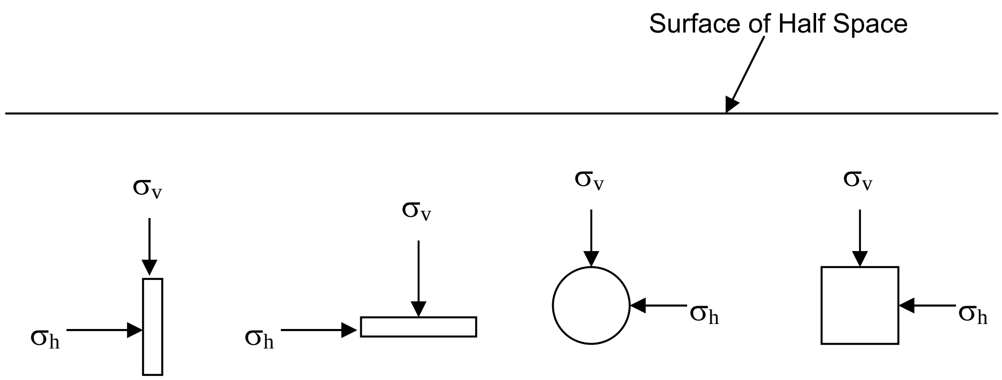

Figure 4. Stress Distribution on Fiber Cross-Sectional Axis

Therefore, the average normal stress on the fibers is an average of the horizontal stress (σ_h) for a vertical fiber and the horizontal and vertical stress ($\frac{\sigma_h + \sigma_v}{2}$) for a horizontal fiber. The combined expression for the average stress conditions on an individual fiber, with respect to both the longitudinal and cross-sectional axes, is presented in Equations (2), (3) and (4).

$$\sigma_{ave} = \frac{\sigma_h + \frac{\sigma_h + \sigma_v}{2}}{2} = \frac{3\sigma_h + \sigma_v}{4} \tag{2}$$

For geostatic stress conditions:

$$\sigma_h = K_0 \sigma_v \tag{3}$$

where:

$K_0 =$ At-rest earth pressure coefficient

Substituting (3) into (2):

$$\sigma_{ave} = \frac{3K_0\sigma_v + \sigma_v}{4} = \frac{\sigma_v(3K_0 + 1)}{4} = \sigma_v(0.75K_0 + 0.25) = \sigma_v K_e \qquad (4)$$

where: $K_e = 0.75K_0 + 0.25$, the stress variable for fibers

Below the threshold confining stress, or "critical confining stress" (Maher and Gray, 1990), the fibers slip during deformation. Above the critical confining stress, the fibers yield or break. In consideration of practical fiber lengths, cross-sectional area, and ultimate tensile strength, an extremely tall embankment would be required to reach the critical confining stress. Therefore, the failure mechanism of FRS, under virtually all practical conditions, will be pullout of the fibers. Consequently, only confining stresses below the critical confining stress are considered in the remainder of this study.

Effective Fiber Length

The effective length of an individual fiber (L_e) across a potential shear plane varies between zero and one-half the fiber length, as illustrated in Figure 5.

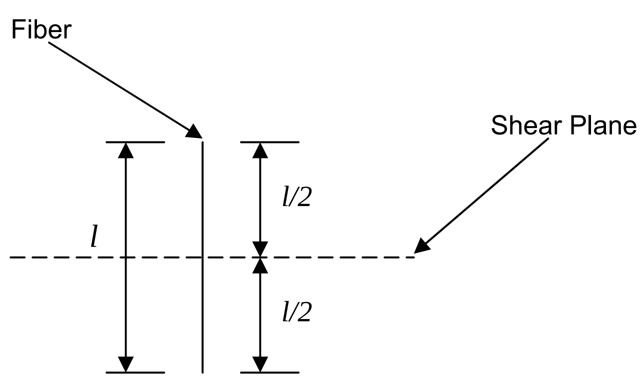

Figure 5. Effective Fiber Length Across Shear Plane

The effective fiber length is defined by Equations (5) and (6).

$$\frac{l}{2} \geq L_e \geq 0 \tag{5}$$

Therefore:

$$L_{e(ave)} = \frac{\frac{l}{2} + 0}{2} = \frac{l}{4} \tag{6}$$

FRS Shear-Strength Formulation

The average number of fibers per unit volume of FRS can be determined using Equation (7).

$$n_f = \frac{4V_r}{\pi d^2 l} \tag{7}$$

where:

n_f = Average number of fibers per unit volume

V_r = Fiber volume ratio (ratio of fiber volume to total volume of a unit mass of FRS)

d = Diameter of fiber, or equivalent diameter for non-circular fiber

l = Length of fiber

If a fiber is rotated in all directions about its centroid, it will trace out a sphere. Consider a single fiber, randomly distributed within the sphere, with respect to a reference plane, such as a shear plane, as illustrated in Figure 6 (after Maher and Gray, 1990).

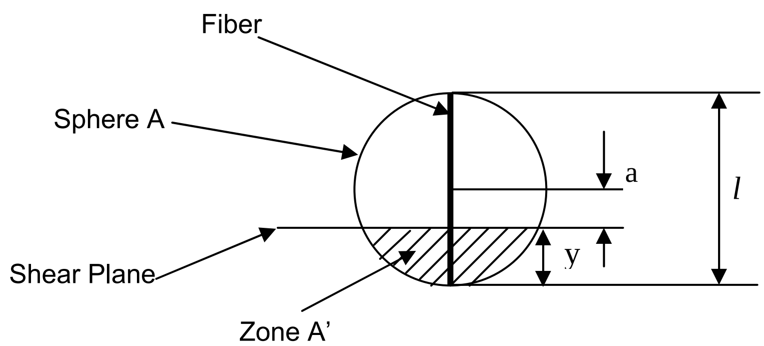

Figure 6. Geometry of Fiber Distribution in Sphere Space

The probability that a fiber will intersect the shear plane, with its center at distance “a” from the plane is given by Equation (8).

$$P(i) = \frac{\frac{l}{2} - a}{\frac{l}{2}} \tag{8}$$

The probability that a fiber will intersect the shear plane is related to the surface area ratio of the portion of the sphere designated as Zone A’ (which is proportional to height “y”) in Figure 6, to the surface area ratio of the entire sphere. The probability is equal to $1 - \frac{2a}{l}$ for “a” less than or equal to $\frac{l}{2}$, and equal to zero for “a” greater than $\frac{l}{2}$, with the distance “a” being uniformly distributed between zero and $\frac{l}{2}$. Considering a unit volume of the FRS on one side of the shear plane, the number of fibers intersecting on a unit area A = 1, is given by (Maher and Gray, 1990):

$$\int_0^{l/2} \left(1 - \frac{2a}{l}\right) n_f A da = n_f \frac{l}{4} \tag{9}$$

Since the fibers on both sides of the shear plane must be considered, the total number of fibers intersecting the plane is:

$$2n_f \frac{l}{4} = n_f \frac{l}{2} \tag{10}$$

Substituting (7) into (10), we obtain (Ranjan, et al, 1996; Maher and Gray, 1990):

$$N_f = \frac{2V_r}{\pi d^2} \tag{11}$$

and:

$$V_r = \frac{W_f}{G_s \gamma_w} \tag{12}$$

where:

N_f = Number of fibers intersecting the shear plane

V_r = Fiber volume ratio

W_f = Weight of fibers in a unit volume of FRS

G_s = Specific Gravity of fiber material

γ_w = Unit weight of water

The pullout resistance of a single fiber due to friction, and thus its contribution to apparent frictional shear strength, with stress conditions below the critical confining stress, may be calculated using Equation (13) (Gregory, 1999a):

$$\tau_{frs\phi} = L_e \pi d \sigma_v K_e f_\phi \tan\phi \tag{13}$$

where:

$\tau_{frs\phi} =$ Apparent increase in frictional shear strength due to fiber

$f_\phi =$ Interaction coefficient related to the frictional component of the shear strength (sometimes referred to as $f_\phi \tan\phi = \tan\delta$)

$\phi =$ Angle of shearing resistance of raw soil

Other symbols as previously defined

The apparent increase in frictional shear strength due to any application rate of fibers can be calculated by inserting N_f into Equation (13), to obtain:

$$\tau_{frs\phi} = L_e \pi d \sigma_v K_e f_\phi N_f \tan\phi \tag{14a}$$

Substituting the full expressions for L_e and N_f into Equation (14a):

$$\tau_{frs\phi} = \frac{l}{4} \pi d \sigma_v K_e f\phi \frac{2V_r}{\pi d^2} \tan\phi \tag{14b}$$

Which reduces to:

$$\tau_{frs\phi} = \frac{l}{2d} \sigma_v K_e f_\phi V_r \tan\phi \tag{14c}$$

Now, since $\tan\phi = \frac{\tau}{\sigma_v}$, and setting $\frac{l}{2d} = a_{re} =$ the "effective aspect ratio," we have:

$$\frac{\tau_{frs\phi}}{\sigma_v} = a_{re} K_e f_\phi V_r \tan\phi \tag{14d}$$

and:

$$\Delta\phi_{frs} = \tan^{-1}\left[a_{re} K_e f_\phi V_r \tan\phi\right] \tag{14e}$$

where:

$\Delta\phi_{frs} =$ Increase in ϕ due to fiber reinforcement

and:

$$\tan\phi_{frs} = \tan\phi + \Delta\phi_{frs} \tag{14f}$$

The apparent increase in the cohesive shear strength component due to fiber reinforcement can be developed in a similar manner, resulting in Equation (15):

$$\tau_{frsc} = a_{re} f_c V_r c \tag{15}$$

where:

$\tau_{frsc} =$ Apparent increase in cohesive shear strength due to fiber when $\Delta\phi_{frs} = 0$

$f_c =$ Interaction coefficient related to the cohesive component of the shear strength

$c =$ Cohesion of raw soil

However, Equation (15) represents the increase in cohesion assuming there is no increase in ϕ ($\Delta\phi_{frs} = 0$). This assumption would hold true if the linear strength envelope for the FRS specimens increased by moving upward parallel to the original strength envelope so that only the cohesion increased. Based upon the vast majority of shear test results, this is not the case and $\Delta\phi_{frs}$ will be greater than zero for virtually all cases of effective stress tests. Consequently, for a linear interpretation of the strength envelope, the increase in cohesion calculated by Equation (15) must be reduced by the magnitude implied by the increase in ϕ for the fiber reinforced case. The increase in cohesion calculated by Equation (15) will be referred to as the "uncorrected" cohesion increase. The required reduction in the uncorrected cohesion to achieve the actual increase in cohesion (Δc_{frs}) is related to the difference in slope of the two strength envelopes projected back to the axis from the point of "rotation" of the FRS strength envelope. The point of rotation will occur at a normal stress (σ_r) as calculated by Equation (16) and illustrated in Figure 7. If there was no increase in ϕ then the value calculated by Equation (15) would be the total increase in the shear strength (τ). If a line is constructed parallel to the non-reinforced strength envelope and at a vertical distance above equal to τ_{frsc} calculated by Equation (15), the rotation point will be located at this point on the parallel line as shown in Figure 7. If the axis of the strength plot is temporarily shifted along the non-reinforced strength line a horizontal distance equal to σ_r immediately below the rotation point and the vertical intercept is set equal to ($\Delta\tau$ in Figure 7) the value calculated by Equation (15), then the increase in ϕ at that point will be zero. Accordingly, the increase in total shear strength due to fibers will be greater than the value calculated by Equation (15) for all normal stress values greater than σ_r (right of the rotation point) and less than this value for normal stress values less than σ_r (left of the rotation point). Based upon this formulation, the corrected increase in cohesion due to fibers may be calculated by Equation (17a). Based upon the test results presented in Chapter IV, a good fit of the data is achieved using Equation (17a), derived from the formulation discussed above.

$$\sigma_r = \frac{c}{\tan\phi} \tag{16}$$

$$\Delta c_{frs} = \tau_{frsc} - \sigma_r(\tan\phi_{frs} - \tan\phi) \tag{17a}$$

$$c_{frs} = c + \Delta c_{frs} \tag{17b}$$

where:

σ_r = Normal stress value at which the cohesion correction factor is calculated

$\tan\phi$ = tangent of the non-reinforced ϕ value

c = non-reinforced cohesion value

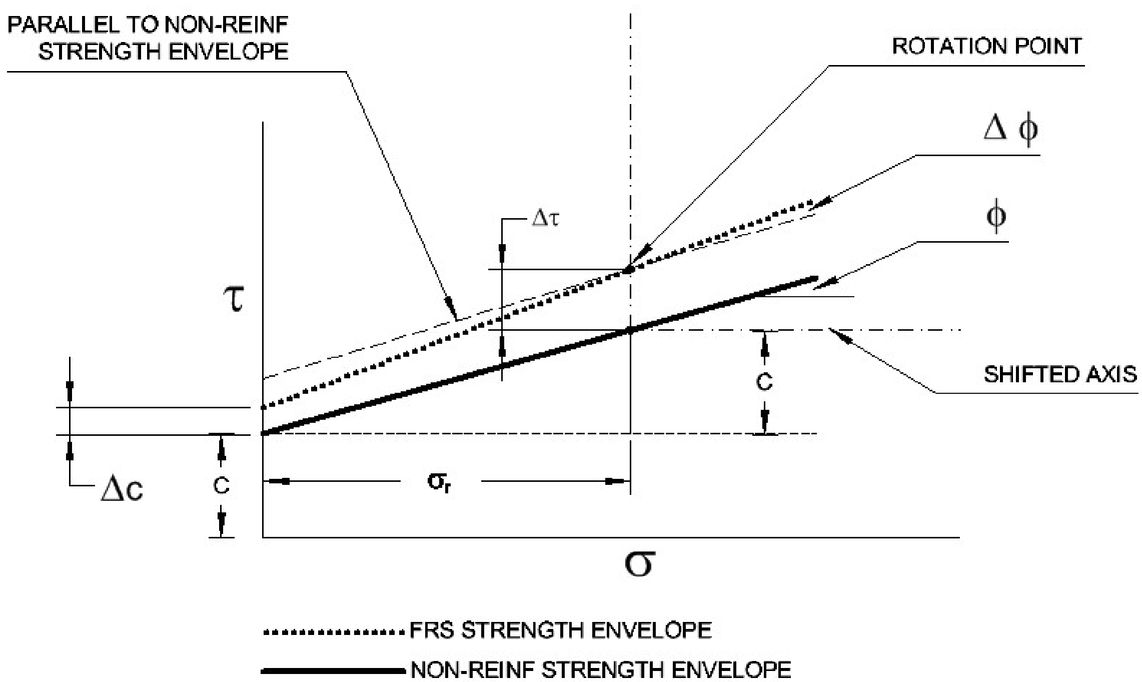

Figure 7. Rotation Point of FRS Strength Envelope

Adjustment of Interaction Coefficients Based On Fiber Content - The conceptual model requires a “decay function” to reduce the interaction coefficients f_ϕ and f_c as the fiber content increases to the point that fiber-to-fiber content is dominate rather than fiber-to-soil contact. This is discussed further in Chapters IV and V.

This concludes the formulation of the conceptual model. The model is discussed further when correlated with laboratory test results in Chapter V.

CHAPTER IV

LABORATORY TESTING PROGRAM

Material Properties

Soil Materials - Two soil types consisting of a fat clay soil and a silty sand were selected for the laboratory testing program. The soil properties are summarized in Table 1. The clay soil consists of residual clays of the Eagle Ford geologic formation of North Central Texas. The silty sand consists of a non-plastic natural soil commercially available in the Stillwater, Oklahoma area. The two soil types represent the upper and lower limits with respect to plasticity of soils generally used for embankment construction.

Table 1. Soil Properties

Soil Description	Liquid Limit	Plastic Limit	% < No. 200 Sieve
Fat Clay (CH) grayish brown	59	20	94
Silty Sand (SM) reddish tan	NP	NP	13

Fiber Material – The fiber material used in the laboratory testing program consisted of commercially produced polypropylene fibers. The nominal dimensions of the individual fibers are 2 inches (50 mm) long by 0.047 inches (1.2 mm) wide by 0.00149 inches (0.038 mm) thick. The material used for the interface tests was a sheet of the same polypropylene material from which the fibers are cut during the manufacturing process. The fibers and sheet material were obtained from the fiber manufacturer.

Laboratory Test Series

Sample and Specimen Terminology – In this study the term "sample" refers to the large bulk sample of the soil and the term "specimen" refers to an individual test portion taken from the bulk sample.

Quantities and Types - The laboratory test series included index and classification testing, shear strength testing consisting of direct shear and triaxial shear tests, and constant load direct shear creep tests, as described in detail in this chapter. The tests were conducted by or under the direct supervision and observation of the author and were performed in the OSU Civil Engineering Soils Laboratory and in the geotechnical engineering laboratory of the author's firm, Gregory Geotechnical, in Stillwater, Oklahoma. The laboratory testing program was conducted during the period from July 2005 through March 2006. The routine laboratory test types and quantities are presented in Table 2.

The laboratory shear and creep test types and quantities are presented in Table 3. The index and classification tests listed in Table 2 were conducted on raw soil only. Previous research and testing programs have established that there is no perceptible difference in test results between FRS and raw soil for the types of tests listed in Table 2 (Al Wahab and Al-Qurna, 1995; AGT Laboratories, 1999; Gregory, 1999a). Accordingly, the index and classification tests were performed on raw soil only.

Table 2. Routine Laboratory Testing Program

Test Type	Description	Quantity	No. Specimens	Remarks
Standard Proctor	Clay	2	10	ASTM D 698
Liquid & Plastic Limits	Clay	2	4	ASTM D 4318
Percent < No. 200 Sieve	Clay	2	2	ASTM D 1140
Maximum and Minimum Index Density	Sand	2	8	ASTM D 4253 & 4254
Sieve Analysis	Sand	2	2	ASTM D 422
Totals	**Clay and Sand**	**10**	**26**	

Table 3. Shear Strength and Creep - Laboratory Testing Program

Test Type	Description	Quantity	No. Specimens
CU Triaxial w/Pore Pressure Measurements	Clay – No Fibers	2	6
CU Triaxial w/Pore Pressure Measurements	Clay – 0.17 pcf Fibers	2	6
CU Triaxial w/Pore Pressure Measurements	Clay – 0.25 pcf Fibers	2	6
CU Triaxial w/Pore Pressure Measurements	Clay – 1, 1.5, 2 pcf Fibers	1 each	3
CU Triaxial w/Pore Pressure Measurements	Clay – Field Samples	2	6
CD Triaxial	Sand – No Fibers	2	6
CD Triaxial	Sand – 0.17 pcf Fibers	2	6
CD Triaxial	Sand – 0.25 pcf Fibers	2	6
CD Triaxial	Sand – 0.50 pcf Fibers	2	6
CD Direct Shear	Clay – No Fibers	2	6
CD Direct Shear	Clay – 0.17 pcf Fibers	2	6
CD Direct Shear	Clay – 0.25 pcf Fibers	2	6
CD Direct Shear	Sand – No Fibers	2	6
CD Direct Shear	Sand – 0.17 pcf Fibers	2	6
CD Direct Shear	Sand – 0.25 pcf Fibers	2	6
CD Direct Shear	Sand – 0.50 pcf Fibers	2	6
Creep Test	Clay – No Fibers	2	2
Creep Test	Clay – 0.25 pcf Fibers	4	4
Interface Test	Sand and Clay	2	2
Totals	**Clay and Sand**	**41**	**101**

The various tests listed in Tables 2 and 3 are discussed in detail in this Chapter and the results are summarized in various tables in Chapter V. The actual laboratory test reports are included in Appendix A.

Test Durations – The approximate test durations for the laboratory test program are summarized in Table 4. The test durations include only the actual clock time required to prepare specimens (including hydration time where required) and perform the test in the laboratory, including set up and breakdown of the test apparatus, and do not include the time required to reduce the test data and prepare test reports. Numerous test activities allowed concurrent preparation of specimens, but the times listed are for individual tests. The triaxial tests were performed in two different triaxial cells for the clay tests and therefore allowed two specimens to be saturating and consolidating concurrently, but the times listed are for individual tests.

Bulk Sample Preparation

Clay Soil Sample – The bulk sample of the clay soil was prepared from thin-wall tube (Shelby Tube) samples obtained from soil borings previously performed by the author's firm on one of the case history projects discussed in Chapter VII. The soil was very uniform from boring to boring based upon classification tests performed during the geotechnical study for the project. The Shelby Tube samples were initially processed by chopping into approximate 1-inch pieces and allowed to air dry for approximately one week. The samples were then processed through an electrically driven mechanical soil processor as shown in Figure 8 to produce a large bulk sample. The bulk sample was thoroughly mixed by hand by repeatedly using the “quartering” method in a large mixing box. The sample was then stored in labeled 5-gallon (19 liter) buckets with sealed lids. The total bulk sample consisted of approximately 400 pounds (181.5 kilograms) of soil.

Figure 8. Processing of Clay Sample

<u>Silty Sand Sample</u> – The silty sand sample was obtained from a local commercial source that provides fill sand for construction projects. The sample was obtained by shoveling from a large stockpile into individual 5-gallon (19 liter) buckets. The buckets were then hauled to the laboratory and mixed into a large bulk sample by the quartering method as described for the clay sample. The silty sand sample

was then stored in individual 5-gallon (19 liter) buckets with sealed lids. The total bulk sample consisted of approximately 300 pounds (136 kilograms) of soil.

Table 4. Approximate Test Durations

Activity	No. Specimens	Unit Time-Hr	Total Time - Hours
Clay			
Standard Proctor	10	2	20
Percent < No. 200 Sieve	2	2	4
Liquid-Plastic Limits	4	2	8
Triaxial Spec. Prep	27	26	702
Triaxial Saturation	27	30	810
Triaxial Consol.	27	24	648
Triaxial Shear	27	29	783
Direct Shear Prep	18	26	468
Direct Shear Consol	18	24	432
Direct Shear-Shear	18	23	414
Creep Test Prep	6	26	156
Creep-Shear	6	504	3024
Sand			
Relative Density	8	2	16
Sieve Analysis	2	2	4
Triaxial Prep	27	3	81
Triaxial Shear	27	1	27
Direct Shear Prep	24	3	72
Direct Shear-Shear	24	1	24
Totals	**Clay and Sand**		**7693**

Index and Classification Tests

Liquid and Plastic Limits Tests – Liquid and Plastic limits (Atterberg Limits) tests were performed on the clay soil in general accordance with ASTM D 4318 using the one-point method. This method requires the test to be repeated two times for each point and the average of the two points are taken as the result, if the two test values are within the acceptance criteria. The Atterberg limits are used in the classification of the soil. The silty sand soil was determined to be non-plastic by visual-manual procedures and it was therefore not necessary to perform Atterberg limits tests on the silty sand. Results of the Atterberg limit tests are presented on the Standard Proctor test reports in Appendix A.

Percent Passing No. 200 Sieve Tests – Percent passing the No. 200 (0.075 mm) sieve tests were performed for the clay soil in general accordance with ASTM D 1140, using the wet sieve method. The percent passing the No. 200 sieve is used in the classification of the soil. Results of the percent passing No. 200 sieve tests are presented on the Standard Proctor test reports in Appendix A.

Standard Proctor Tests – Standard Proctor (moisture-density relationship) tests were performed on the clay soil in general accordance with ASTM D 698, Method A. Each of the two tests consisted of a 5-specimen series to establish the moisture-density relationship curve. The Standard Proctor test establishes the optimum moisture content and maximum dry density relationship required to establish target moisture and density parameters for the laboratory compacted specimens of the clay soil. Results of these tests are presented on the Standard Proctor test reports in Appendix A.

Sieve Analysis Tests – Sieve Analysis (grain-size distribution) tests were performed on the silty sand in general accordance with ASTM D 422. The sieve analysis tests are used in the classification of the soil. The results of these tests are presented on the Grain-Size Distribution test reports in Appendix A.

Maximum and Minimum Index Density Tests – Maximum and Minimum Index Density tests were performed on the silty sand in general accordance with ASTM D 4253 and D 4254. Each of the two test series consisted of 4 specimens. The Relative Density value can be calculated from the maximum and minimum index densities and the actual compacted density of a soil specimen. However, in this study 95 percent of the Maximum Index Density was used as a target density for the specimens rather than a relative density. This is in line with current practice for controlling field density of granular (non-plastic) soils. The results of these tests are presented on the Maximum and Minimum Index Density test reports in Appendix A.

Specimen Preparation Prior to Compaction

General Methodology - Previous research and project testing of FRS has consisted of specimen preparation by mixing "batches" of soil from the bulk sample in sufficient quantity to produce 4 to 6 individual specimens (AGT

Laboratory, 1999; Fugro McClelland, 1997a, 1997b). The fiber content was added to the batch based upon the weight of the entire batch and then mixed in a large mixer. Individual specimens were then hand grabbed from the batch. This procedure, although carefully controlled, was later found to produce considerable variation in the amount of fibers actually contained in each individual specimen and some extent variability in the actual moisture content of each specimen taken from the batch. Based upon the past variability of the batch method, a different method of specimen preparation was developed for this study as described below.

<u>Moisture and Weight Preparation</u> - Each specimen was prepared individually rather than by the batch method. Each specimen was weighed to provide an amount for moisture content specimens and a small amount of waste over the exact required weight. The specimens were placed in individual sealed bags and the moisture content was determined from specimens taken from each bag by obtaining a composite mixture from three places in the bag. Typical specimens are shown in Figure 9.

Moisture contents were determined with a lab oven in general accordance with ASTM D 2216. Once moisture contents were determined for each bag, the specimens were individually mixed with the exact amount of water required to bring the specimen to the target moisture content (optimum per ASTM D 698). The specimens were hydrated in the sealed bags for a minimum of 36 hours to allow uniform distribution of the moisture. Following hydration, the final specimen quantity was obtained by carefully weighing the exact amount of moist soil required for the compacted specimen size. For non-reinforced specimens, the soil was sealed in a new plastic bag and labeled with the specimen number. For FRS specimens, the fibers were mixed into the specimen prior to placing in the new bag as described in the next paragraph.

Figure 9. Clay Specimens Prior to Hydration

FRS Mixing – The fibers were weighed to the exact amount for each specimen and placed in labeled plastic bags for each specimen prior to the mixing stage. The fibers were mixed into each individual specimen by hand. The small quantity involved in mixing individual specimens makes it impractical to use a mixer. The soil was spread into a flat mixing pan and the fibers were evenly spread over the soil and thoroughly mixed into the soil by hand as illustrated in Figures 10 through 13.

Figure 10. Spreading Fibers over Hydrated Clay Soil Specimen

The fibers had been weighed to the exact required amount and placed in labeled plastic zip-lock bags prior to the mixing operation. The soil specimen was spread out over the bottom of the pan to a thickness of approximately 0.75 inches. The fibers were then spread uniformly over the soil based on visual observation. The fibers were then blended into the soil by hand by repeatedly kneading the soil and fibers as illustrated in Figures 11 and 12. A fine water mist was applied one or two times during mixing to facilitate bonding of the fibers into the mix. The final

FRS mixture is illustrated in Figure 13. Immediately following mixing, the specimen was carefully placed into a labeled zip-lock bag with the air being pushed out by hand prior to zipping the bag. The specimen was then placed in storage until the compaction process. Due to the cohesion (stickiness) of the clay soil, segregation of the fibers from the soil was not a problem during subsequent handling.

Figure 11. Initial Hand Mixing of FRS Specimen

Compaction of Clay Specimens

Triaxial Shear Specimens – The clay specimens for the triaxial shear tests were compacted in a steel mold that produces a 2.875-inch (73 mm) diameter by 5.8-

inch (147 mm) tall specimen. This specimen size is one of the standard sizes for triaxial testing and was selected so that the specimen would be greater in all dimensions than the fiber length of 2 inches (50 mm). The mold and compaction process are illustrated in Figures 14 through 17. The mold was fitted with a temporary plastic collar mounted on top of the steel collar. The entire loose specimen was then placed in the mold with a small scoop prior to compaction as illustrated in Figure 14. The same procedure was used for raw soil and FRS.

Figure 12. Final Hand Mixing of FRS Specimen

The plastic collar was removed and the specimen was then compacted with multiple strokes of a 0.5-inch (13 mm) diameter metal rod with a rounded tip as illustrated in Figures 15 and 16. The rod was used as a miniature simulation of a tamping-foot (sheep-foot) compaction roller typically used for embankment

Figure 13. Mixed FRS Specimen Ready for Storage or Compaction

construction. The rod also caused the fibers to be randomly oriented in the compacted specimen, rather than being horizontally oriented as would occur if a flat piston or hammer had been used for compaction. The rod was initially plunged numerous times to a depth almost to the bottom of the mold. This was initially possible in the loose specimen. As the specimen became partially compacted, the depth of plunge of the rod became less. This process was repeated until all the soil was well consolidated and was below the top of the steel collar. The process was completed by compacting and smoothing the top of the specimen with a steel piston just slightly smaller in diameter than the mold.

The piston was tapped or pressed into the mold until it bottomed out on a guide ring on the piston as illustrated in Figure 17. The piston extension below the guide ring was set to result in a finished specimen height of 5.8 inches (147 mm).

Figure 14. Placement of Loose Specimen into Mold

Immediately following compaction, the specimen was carefully extruded from the mold with an electrically-operated hydraulic extruder. The dimensional integrity of each specimen was checked following extrusion with a caliper. None of the specimens were shortened or otherwise distorted by the extrusion process.

The weight of each specimen had been prepared so that exactly 95 percent of maximum dry density as determined in the Proctor test (ASTM D 698) would be achieved when the all the soil in the specimen was compacted into the mold to the dimensions discussed above. Accordingly, all specimens were at exactly the same moisture content and dry density. The maximum variation from the target weight in the compacted specimens was plus or minus 3 grams as determined by weighing the completed specimens immediately following extrusion.

Figure 15. Compaction with Metal Rod

<u>Direct Shear Specimens</u> – The clay specimens for the direct shear tests were prepared in a very similar manner to the triaxial specimens. A similar, but smaller mold was used for the direct shear specimens as shown in Figure 18. The mold was configured to produce a final specimen size of 2.5-inches (64 mm) in diameter by 2.25-inches (57 mm) in height. This specimen size was also selected

so that all dimensions of the specimen would be greater than the fiber length. The 2.5-inch (64 mm) diameter is one of the standard sizes for a direct shear box, but most available shear boxes will accommodate a maximum specimen height of about 1.5 inches (38 mm). The shear box used in this study is a custom fabricated shear box available in the laboratory of the author's firm.

Figure 16. Rod Plunged to Near Bottom of Mold During Initial Compaction

Placement in the mold and compaction of the direct shear specimens were performed in exactly the same manner as for the triaxial specimens except that a smaller diameter piston was required for final compaction and smoothing of the specimens top. The process is shown in Figures 18 and 19. Following compaction, the specimens were extruded as previously described for the triaxial specimens and dimensional integrity was verified with a caliper.

Figure 17. Finishing Compaction with Piston and Guide Ring

Creep Specimens – The clay specimens for the creep tests were prepared in exactly the same manner and with the same equipment as described for the direct shear specimens. No modifications were required in the procedure since the direct shear specimens and creep specimens are the same size.

Storage of Specimens – Following compaction and extrusion, each clay specimen was double wrapped in plastic cling wrap. Each specimen was then labeled and placed in a portable cooler to maintain uniform moisture. The specimens were covered with heavy duty paper lab towels and the towels and inside of the cooler were sprayed with a water mist sprayer each day. The storage cooler is illustrated in Figure 20.

Figure 18. Preparation for Compaction of Direct Shear Specimen

Moisture Content Stability During Storage – Moisture content stability of the specimens during storage was periodically verified by weighing selected specimens. The specimens that had been in storage the longest period of time were selected for moisture checking each time the verifications were performed. The verification specimens were removed from the storage cooler, temporarily unwrapped and weighed. The specimen was sprayed with a light mist of water, rewrapped and immediately placed back in the storage cooler. All specimens checked were very stable with respect to moisture content. All specimens were individually checked for moisture content stability by weighing the unwrapped specimen just prior to testing.

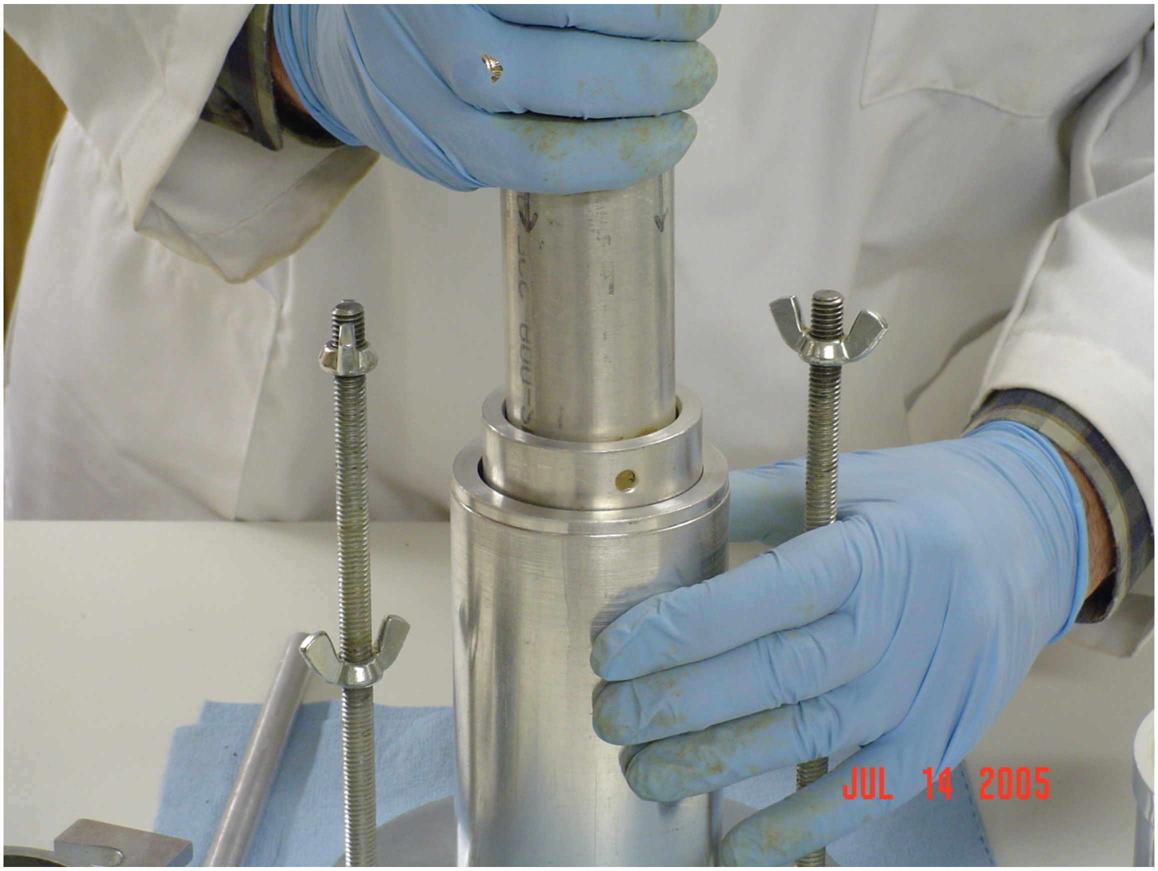

Figure 19. Completing Compaction of Direct Shear Specimen

Compaction of Sand Specimens

Triaxial Shear Specimens – It was necessary to prepare the sand specimens for the triaxial tests inside the triaxial test membrane just prior to shear testing since the sand will not mold into a specimen that will hold together after compaction without confinement. Therefore, the sand specimens were compacted inside the membrane in a split mold that also serves as a membrane stretcher. The mold and the compaction operation are shown in Figure 21.

Figure 20. Clay Specimen Storage Cooler

For the FRS specimens, the fibers were added along with the sand during the compaction stage. The entire specimen was placed in the mold by inserting the fibers as the sand was placed as shown in Figure 22. After the entire loose specimen (and fibers for FRS specimens) was placed in the mold, the specimen was compacted with the metal rod as described for the clay specimens. The mold was periodically tapped on the sides to help in consolidating the sand by vibration. The top of each specimen was smoothed and final compaction performed with the steel piston as previously described for the clay. A typical compacted specimen after removal of the split mold is shown in Figure 23.

Figure 21. Preparation of Sand Specimen in Split Mold

As illustrated in Figure 23, the specimens were prepared directly on the base of the triaxial cell, with the split mold being fitted around the bottom platen of the cell. This procedure eliminated the need to handle the specimen following compaction and allowed the triaxial cell to be assembled around the prepared specimen.

<u>Direct Shear Specimens</u> – The sand specimens for the direct shear tests were prepared directly in the assembled shear box as illustrated in Figure 24. The fibers were added as the sand was placed for the FRS specimens as described for the triaxial specimen preparation. This procedure allowed the sand specimens

to be prepared without subsequent handling outside the shear box.

Figure 22. Addition of Fibers to Sand Specimen During Compaction

Triaxial Shear Tests – Clay

Test Type - The triaxial tests on the clay specimens were performed as Isotropically Consolidated Undrained (ICU) tests with pore pressure measurements during the tests. The tests were all performed as three-specimen series, unless stated otherwise in the text. The tests were performed at a shear rate of 0.00049 inches (0.0125 mm) per minute. This required approximately 29 hours during the shear stage to achieve 15 percent strain, which was equivalent to a deformation of approximately 0.87-inches (22 mm).

Figure 23. Compacted Sand Specimen After Removal of Split Mold

Mounting in Triaxial Cell - For each test, the clay specimen was removed from the storage cooler and the cling wrap was removed prior to mounting the specimen. Filter papers were placed between the specimen and the bronze porous stones on each end of the specimen to prevent intrusion of the clay soil into the porous stones. A filter paper “skirt” was provided on the perimeter of the specimen to facilitate saturation. The membrane was placed over the specimen with a membrane stretcher by applying vacuum to hold the membrane to the stretcher tube during placement. An FRS specimen prior to placement of the membrane is shown in Figure 25, and a specimen with the membrane and top cap in place is shown in Figure 26. Note that two different models of triaxial cells were used in the testing. However, both cells function basically the same.

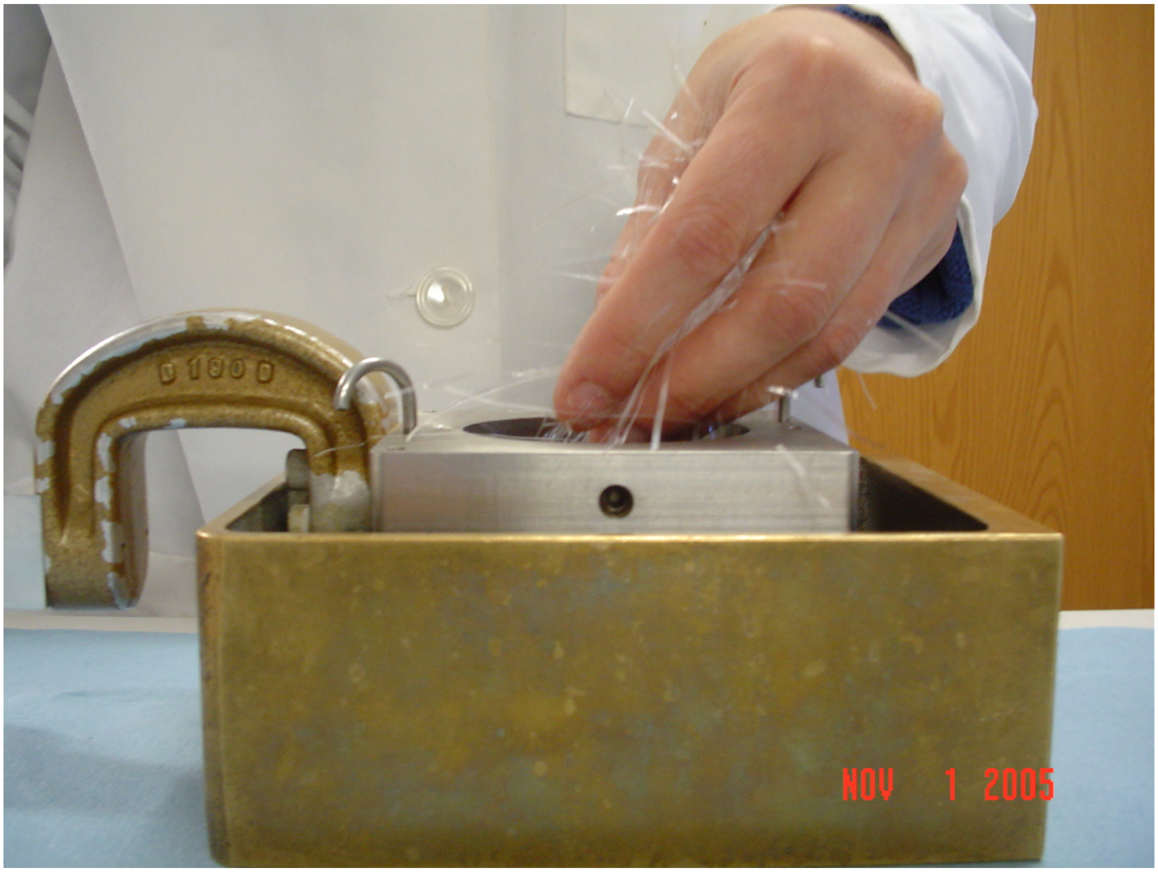

Figure 24. Preparation of FRS Sand Specimen in Direct Shear Box

Following placement of the membrane and top cap, the remainder of the cell was mounted around the specimen and the cell was filled with water and the back pressure lines were purged of air.

<u>Saturation and Consolidation</u> - The specimen was saturated under a cell pressure of 65 psi (448 kPa) and a back pressure of 60 psi (414 kPa). Saturation of the clay specimens typically required approximately 48 hours with the back pressure of 60 psi (414 kPa). Saturation was verified by checking the “B” parameter in general accordance with ASTM standards.

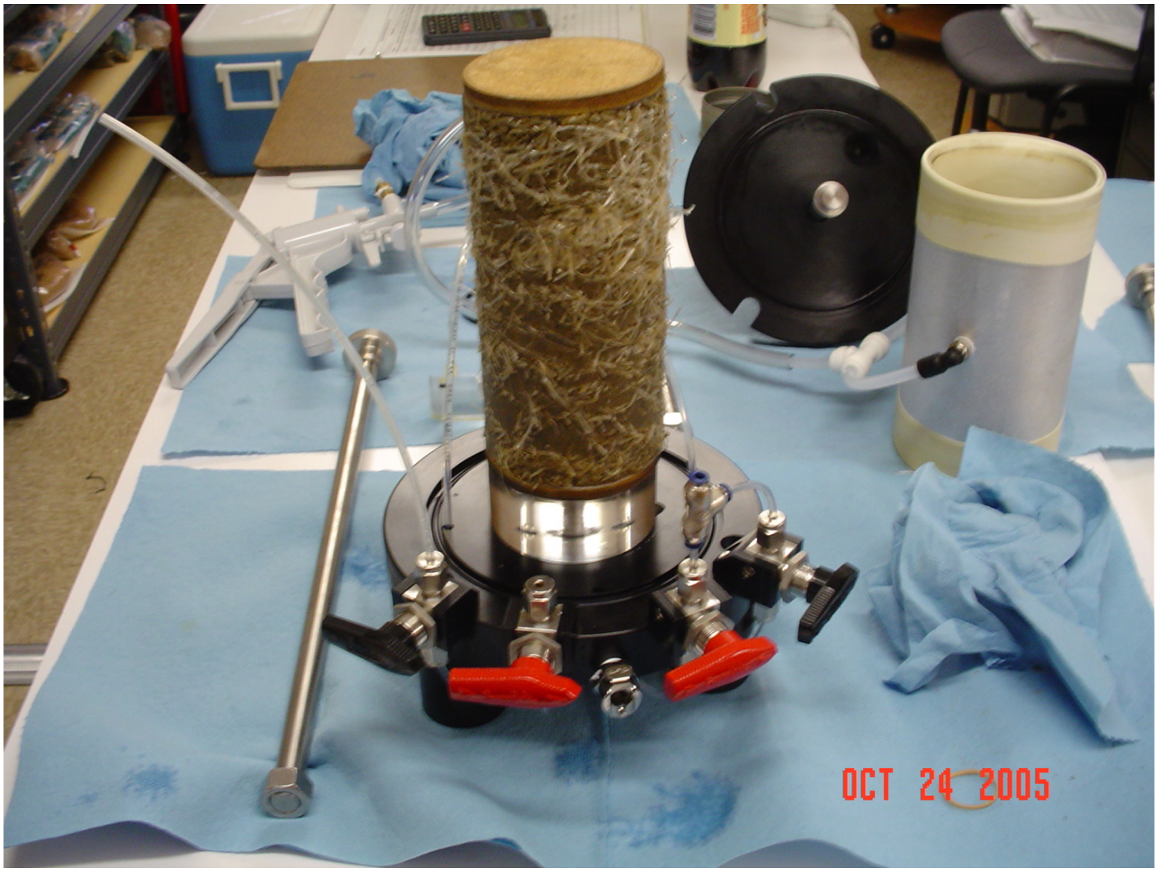

Figure 25. FRS Specimen Mounted on Base of Triaxial Cell

Following saturation the specimen was consolidated by increasing the cell pressure to 70, 80, or 100 psi (483, 552, or 690 kPa) for specimen number 1, 2, or 3, respectively for each test series. This produced an effective stress for the three-specimen series of 10, 20, and 40 psi (69, 138, and 276 kPa), respectively. Consolidation of each clay specimen required approximately 48 hours. The end of primary consolidation was verified by monitoring specimen height and change in the panel burette water height until both were stabilized with no additional change. Two triaxial pressure panels were used for saturation and consolidation. This allowed the two triaxial cells to be in the various test stages simultaneously. An illustration of the saturation/consolidation stage is presented in Figure 27.

Figure 26. Specimen With Membrane and Top Cap in Place

Shear Stage – The specimens were sheared in a triaxial compression machine that is capable of a very slow shear rate. As previously stated the specimens were sheared at a rate of 0.00049 inches (.0125 mm) per minute. A triaxial test on clay during the shear stage is illustrated in Figure 28.

Electronic Data Acquisition – All test parameters during the shear stage were recorded electronically to a computer file. The initial stage of the test was recorded each time a load change occurred until approximately 6 to 10 readings had occurred and at 5-minute intervals thereafter. During the test the test data were also displayed in real time on the computer screen, as shown in Figure 29.

Figure 27. Saturation/Consolidation Stage

Inspection and Dissection of Specimens Following Test – Upon completion of the shear stage, each specimen was removed from the cell and membrane and visually examined for failure mode and was then dissected to visually observe the interior of the specimen. Typical post-test specimens are shown in Figures 30 and 31. Final moisture contents were obtained on cuttings from each specimen following completion of the test.

Direct Shear Tests – Clay

Test Type – The direct shear tests on clay were performed as Consolidated Drained (CD) tests. The tests were all performed as three-specimen series. The specimens were sheared at a rate of 0.0003 inches (0.0076 mm) per minute to a total deformation of approximately 0.4 inches (10 mm), resulting in a total strain of 16 percent for the 2.5-inch (64 mm) diameter shear box.

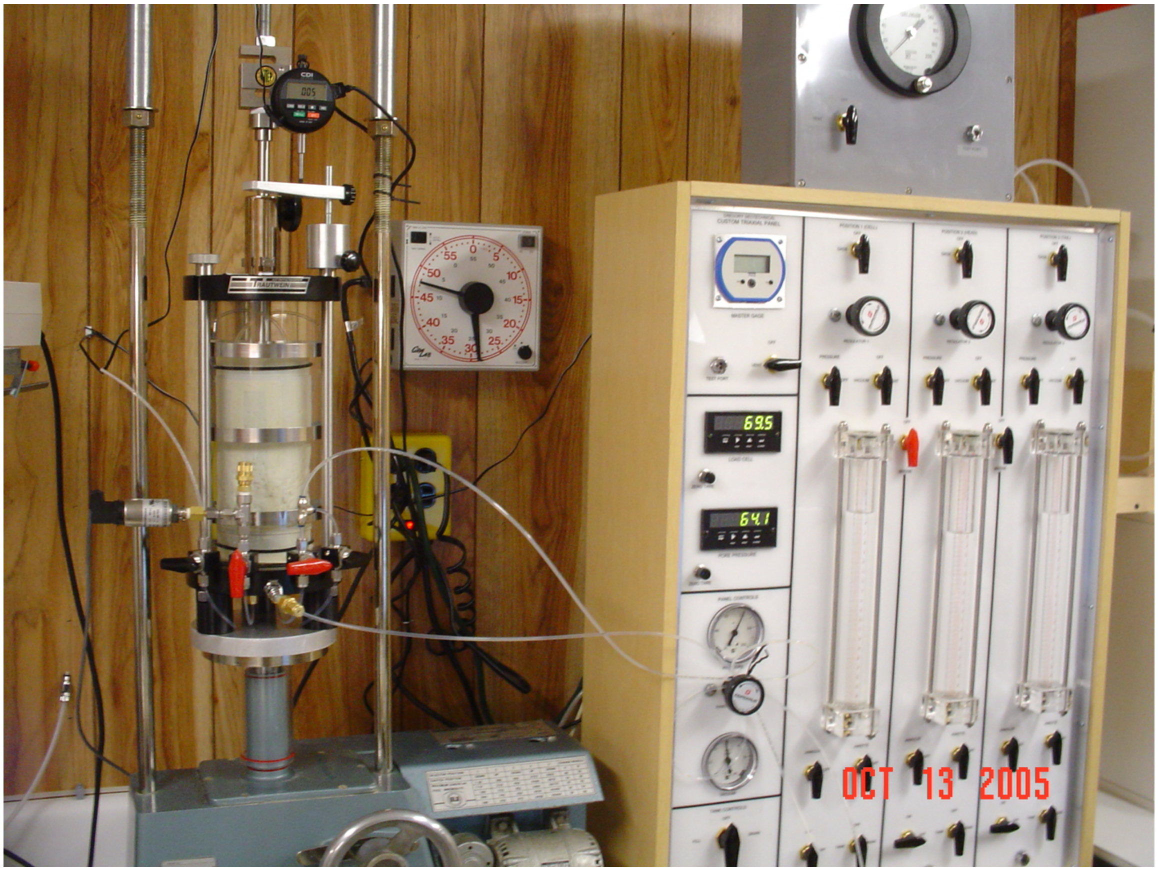

Figure 28. Shear Stage of Triaxial Test on Clay Specimen

<u>Mounting in Direct Shear Box</u> – Each clay specimen was taken from the storage cooler and the cling wrap was removed prior to mounting. The specimen was fitted with a filter paper on each end to separate the clay soil from the bronze porous stones. The specimen was carefully pushed into the shear box with a metal piston with an end cap slightly smaller than the inside diameter of the shear box. The bottom porous stone and filter paper had already been placed in the bottom of the box. The top filter paper was in place during placement of the

specimen in the box, but the top porous stone was not placed until the specimen had been pushed into final place. An illustration of mounting a clay specimen in the shear box is presented in Figure 32.

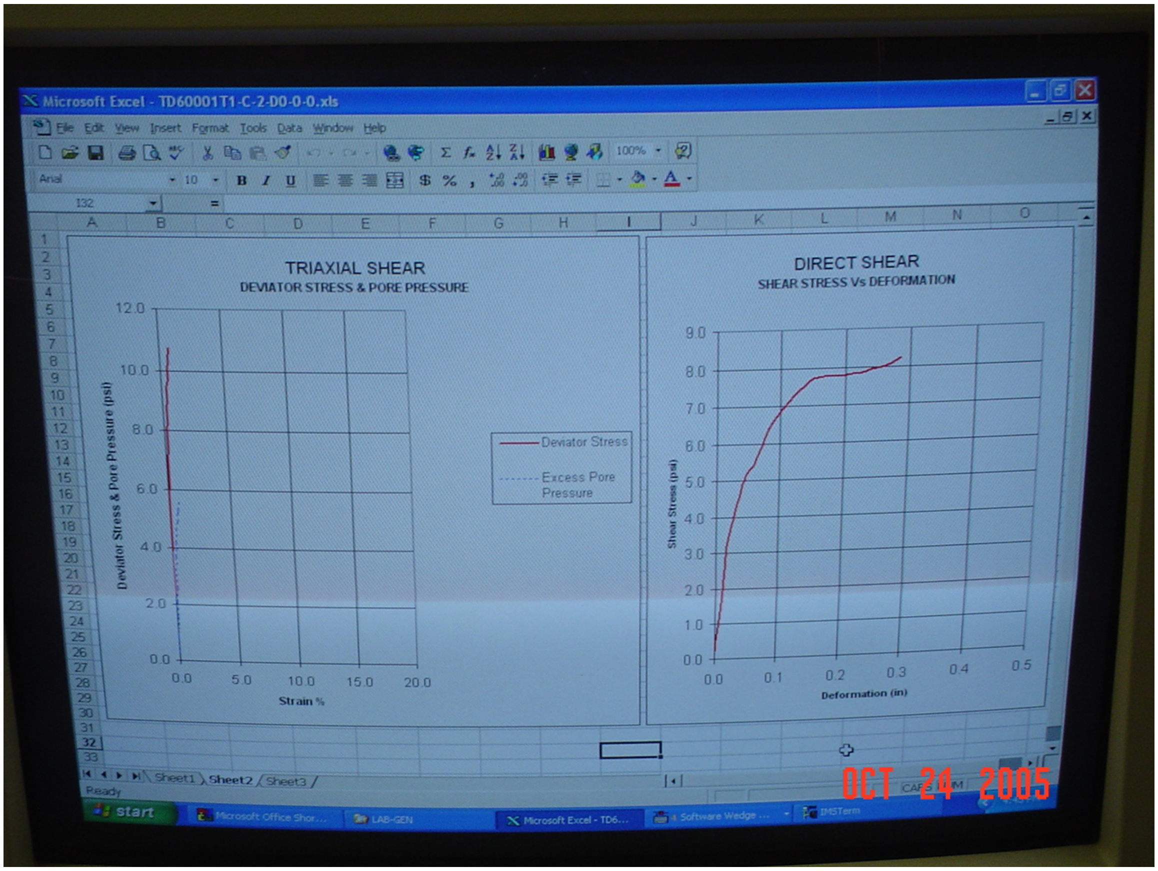

Figure 29. Test Data Display in Real Time on Computer Screen

After placement of the specimen in the direct shear box, the box was mounted into the direct shear machine and a seating load was applied to the specimen with a dead weight hanger. Distilled water was then added to the water reservoir around the shear box.

<u>Saturation and Consolidation</u> – The specimen was saturated and consolidated at the same time by applying the required normal load while maintaining the water

level in the reservoir by adding water several times a day. During a direct shear test full saturation of the specimen cannot be verified because it is not possible to measure pore pressures in the device. Saturation is assumed to have occurred along the shear surface between the top and bottom halves of the shear box by the time the specimen has reached the end of primary consolidation. The end of primary consolidation was verified by recording readings of the vertical dial indicator until the deformation essentially leveled out and became stable. The consolidation stage typically required approximately 24 to 36 hours for the clay specimens. The three-specimens for each test series were consolidated under normal stresses of 10, 20, and 40 psi (69, 138, and 276 kPa), respectively.

Normal stress is applied with a dead weight hanger. This method provides a constant and positive normal loading arrangement and does not have the potential variability or “drift” of an air-applied normal load system. A full set of uniform weights are available that will allow precise loading in 5 psi (34.5 kPa) increments for each weight placed on the hanger.

<u>Shear Stage</u> – The specimens were sheared in a computer-controlled direct shear machine. The shear rate and total deformation values are entered into the computer interface program that controls the shear machine. The shear rate can be set over a large range of values from very fast to extremely slow. As previously stated, the shear rate was set at 0.0003 inches (0.0076 mm) per minute. The shear machine was programmed to shear the specimen to a

deformation value of 0.4 inches (10 mm), hold the shear load at that location for 30 seconds, then release the load and return to the zero position at a faster rate.

Figure 30. Clay Triaxial Specimen Following Test

Electronic Data Acquisition – The shear load and displacement were recorded electronically to a computer file during the test. The shear load was recorded by a load cell and the displacement was recorded as a time-displacement rate by the computer. These readings are very precise in the apparatus used for the direct shear testing. During the shear stage, the data were also displayed in real time on the computer screen as previously described for the triaxial shear data and shown in Figure 29. The direct shear machine is illustrated in Figure 33.

Figure 31. Dissected Triaxial Clay Specimen With Exposed Fibers

Inspection and Dissection of Specimens Following Test – Upon completion of the shear stage, each specimen was removed from the shear box and the shear plane was visually examined. The specimen was then dissected to visually observe the interior of the specimen. Typical post-test specimens are shown in Figure 34. Final moisture contents were obtained on cuttings from each specimen following completion of the test.

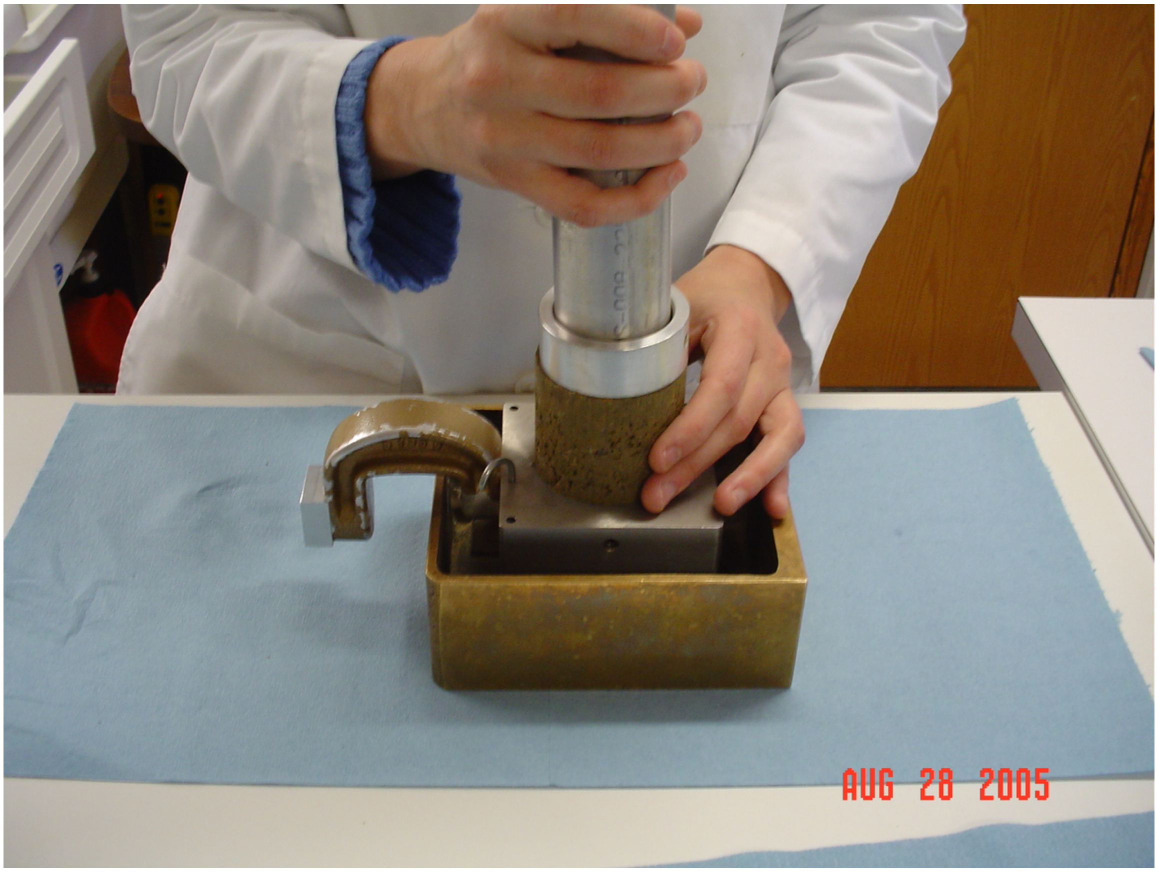

Figure 32. Mounting of Clay Specimen in Direct Shear Box

Creep Tests – Clay

Test Type – The creep tests on clay specimens were performed as constant-load direct shear creep tests. In this test, a constant shear load is applied with dead load weights and a lever-advantage hanger system. This differs from the standard direct shear test in which a constant rate of shear is applied. The normal load was applied in the creep tests with a dead load hanger. A special test device was designed for the creep tests. The laboratory research program included six creep tests to be performed simultaneously. This required fabrication of six creep devices. The creep devices were designed by the author and were

Figure 33. Computer-Controlled Direct Shear Machine

fabricated in the Civil Engineering Machine Shop facility at OSU. Schematic Drawings of the Direct Shear Creep devices are included in Appendix B. The direct shear creep devices are shown in Figure 35.

<u>Mounting in Creep Device</u> – The specimens were mounted in the creep devices in the same manner as described previously for the standard direct shear tests. This procedure is illustrated in Figure 36 and one of the devices with the specimen fully in place is shown in Figure 37.

Figure 34. Dissected Direct Shear Clay Specimen With Exposed Fibers

Saturation and Consolidation – The creep specimens were saturated and consolidated in the same manner as previously described for the standard direct shear tests. The specimens were consolidated with a normal stress of 5.65 psi (39 kPa), which is equivalent to approximately 6.5 feet (2 m) of overburden pressure. Five to eight feet (1.5 to 2.4 m) is a common depth range for shallow slope failure surfaces in clay slopes. This will be discussed further in Chapter VII on case history projects. The specimens reached the end of primary consolidation under the relatively light normal load in about 24 hours.

Figure 35. Direct Shear Creep Devices

<u>Creep Shear Stage</u> – All specimens were initially loaded to produce a shear stress of approximately 70 percent of the peak shear strength of the raw soil as determined in the standard direct shear tests. The load was applied by hanging the appropriate weights on the lever arm of each device. The lever arm has a maximum lever ratio of 17.5 to 1.0. The lever arms were adjusted to a lever ratio of approximately 14.9 to 1.0 in order to apply the desired stress with the available weights. The 70-percent stress ratio is in the range known to likely cause creep failure in clay slopes if sustained over the long term (Sowers, 1979, 1984). The creep tests were performed for approximately 23,000 minutes (16 days) to obtain an indication of the creep behavior of the raw soil compared to the FRS

specimens. Four of the individual specimens were incrementally loaded to failure in small load increments to determine the shear stress required to fail each specimen. The other two specimens have not failed to date under the higher loading. The creep test results are discussed in Chapter V.

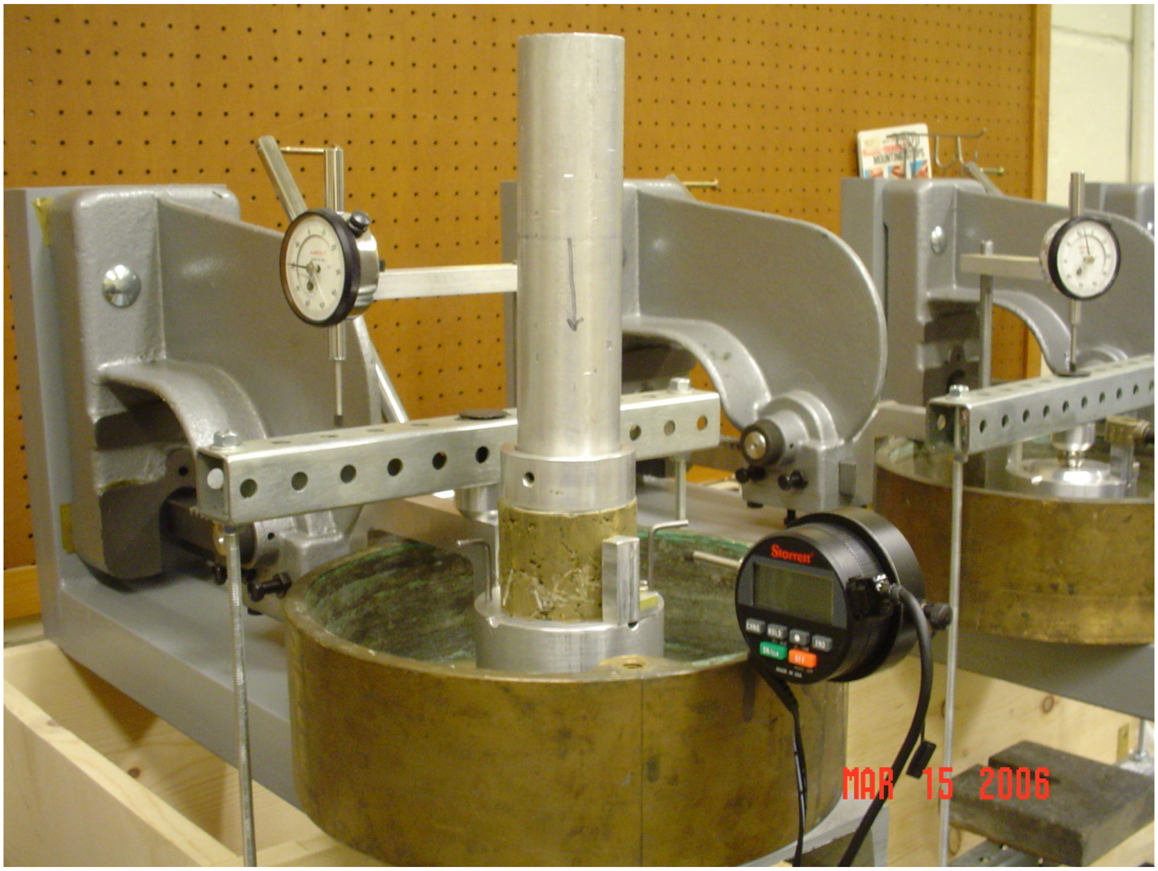

Figure 36. Mounting Clay Specimen in Creep Device

Electronic Data Acquisition – Deformation of four of the six creep specimens was monitored and recorded electronically to a computer file with electronic digital dial indicators, as shown in Figure 37. The data was also displayed on the computer screen in real time during the tests. When the shear load was applied, a series of data readings were taken with the electronic dial indicators, followed by automatic readings at 30-minute intervals thereafter. Displacements of the other two specimens were monitored by mechanical dial indicators with manual readings. Vertical displacements (consolidations) of all six specimens were monitored manually with mechanical dial indicators.

Figure 37. Fully-Mounted Creep Specimen With Water in Reservoir

Triaxial Shear Tests – Sand

Test Type – The triaxial test on the sand material were performed as Consolidated Drained (CD) tests. Since the silty sand is free draining, this allowed the test durations to be short compared to the clay tests previously described. The tests were performed at a shear rate of 0.03 inches (0.76 mm) per minute, which resulted in test durations of approximately 30 minutes for the shear stage. The tests were conducted with the drain lines open to the atmosphere. A special triaxial cell that uses air for the cell fluid instead of water was utilized for the tests. The tests were performed as three-specimen series except as otherwise discussed.

<u>Consolidation and Saturation</u> – The triaxial sand specimens were compacted and mounted in the triaxial cell as previously described under "Compaction of Sand Specimens." Saturation was accomplished by connecting a distilled water tank to the bottom drain line of the triaxial cell and applying a vacuum to the top drain line. Saturation was accomplished while maintaining a cell pressure of 10 psi (67 kPa). Saturation was confirmed by visual observation, achieving flow of water out the top of the specimen, and by monitoring the volume of water transferred into the specimen. Following saturation, the specimens were consolidated under cell pressures of 10, 20, and 40 psi (69, 138, and 276 kPa), respectively for each series. Consolidation of the specimens was achieved almost immediately upon applying the cell pressure.

<u>Shear Stage</u> – The sand specimens were sheared in a multi-purpose compression machine with digital indicators. The readings were recorded manually from the digital indicators, which was practical and efficient for the short duration tests. A triaxial test on the sand is illustrated in Figure 38.

Figure 38. Triaxial Test on Sand Specimen

Inspection and Dissection of Specimens Following Test – Upon completion of the shear stage, each sand specimen was removed from the cell and membrane and dissected to visually observe the interior of the specimen. Final moisture contents were obtained on each specimen following completion of the test.

Direct Shear Tests – Sand

Test Type and Shear Stage - The direct shear tests on sand were performed as Consolidated Drained (CD) tests in the direct shear machine described for the clay specimens and shown in Figure 33. The tests were sheared at a rate of 0.03 inches (0.76 mm) per minute. The test data were recorded electronically in a computer file and displayed in real time on the computer screen as previously described.

Inspection and Dissection of Specimens Following Test – Upon completion of the shear stage, each sand specimen was removed from the shear box and dissected to visually observe the interior of the specimen. Final moisture contents were obtained on each specimen following completion of the test.

Interface Shear Tests

Test Description - The author retained the services of TRI Environmental in Austin, Texas to perform two interface tests for the clay and sand. The large scale equipment necessary for the interface test is not available in the OSU laboratory or in the author' laboratory, and only a few firms in the US have the necessary equipment. The interface tests were performed with each soil type shearing against a sheet of the polypropylene material from which the fibers are made. The sheet material was from the same production run as the fibers used in this study and was from the sheet goods prior to being cut into fibers.

The interface tests were performed in large-scale direct shear machines. The machines have a 12-inch (300 mm) bottom shear box and a 16-inch (400 mm) top shear box. The soil specimen is 2-inches (50 mm) thick after compaction into the shear box. The sheet material is anchored to the bottom shear box with an Emory-board backing to limit slippage. The soil specimen is compacted into the upper shear box and protrudes slightly from the bottom of the box. This arrangement allows the top box to move horizontally and shear the soil across the sheet material on the bottom box. This test measures the interaction coefficient (interface friction-adhesion coefficient) between the polypropylene sheet material and the particular soil being tested. This interaction coefficient is a necessary input into the conceptual model presented in Chapter III.

Specimen Preparation and Test Observation – The bulk soil specimens for the interface tests were taken from the bulk clay and sand samples previously described and were hydrated to the target moisture content, sealed in plastic bags, placed inside sealed plastic buckets along with the sheet material, and shipped to TRI Environmental with instructions for setting up the tests. TRI Environmental prepared the specimens in two different shear machines and placed them in a water bath under 20 psi (138 kPa) normal stress and allowed the specimens to consolidate for 24 hours. The author traveled to Austin to observe the shear stage of the tests and to take photographs. The interface shear tests were performed at a shear rate of 0.04 inches (1 mm) per minute. The test data were recorded automatically to a computer file and the real time data were displayed on the computer screen during the shear stage of the tests. The test results are presented in Appendix A and discussed further in Chapter V. Photographs of the interface tests are presented in Figures 39, 40, and 41.

Figure 39. Large Scale Direct Shear Machines Used in Interface Tests

Figure 40. Real Time Data From Interface Shear Tests (Green = Sand, Purple = Clay)

Figure 41. Sheet Material on Bottom Shear Box After Interface Test

CHAPTER V

CORRELATION AND ANALYSIS OF DATA

Triaxial Shear Test Data

Available Test Data – The triaxial test data utilized in this study consists of the results of the laboratory tests performed during the current study as described in Chapter IV, and triaxial test results from the previous AGT Laboratory testing program described in Chapter II. The author was involved indirectly in a portion of the AGT Laboratory testing program and the test results were made available to the author by the current owner of the test data. It should be noted that only effective stress triaxial test data and effective stress test data from the direct shear tests are used to calibrate and validate the conceptual model. The comparison of effective stress ϕ and c values for non-reinforced and fiber-reinforced soil is more straight forward, whereas picking strength values for comparison between total stress triaxial tests is more ambiguous. The model should be equally accurate for either comparison since both effective and total stress parameters are obtained from the same test, but the stress levels at which to compare the results are less clear and are not as ideal for validating the model. However, the total stress test data developed during this study are included in Appendix A for informational purposes.

Summary of Current Triaxial Test Data – The triaxial shear test data for the three specimen series from the current test program are summarized in Table 5. These tests were performed on both non-reinforced soil (raw soil) specimens and FRS specimens to establish criteria to evaluate the accuracy of the conceptual model.

The triaxial test data summarized in Table 6 are from the single-specimen tests on clay and sand FRS performed at 1, 1.5, and 2 pcf (16, 24, and 32 kg/m^3) fiber content. These tests were performed for comparative purposes to help establish a relationship for the "decay" in strength improvement at larger fiber contents.

If the fiber content of FRS was increased without limit, there would be a continually increasing number of fibers with fiber-to-fiber contact rather than fiber-to-soil contact. The absolute upper limit of this trend would be the case where all the soil was eventually replaced with fibers. In this hypothetical and extreme case, there would no longer be any fiber-to-soil contact and the interface friction coefficient would be reduced to that of the polypropylene fiber material. Although the fiber content at which a significant reduction in strength improvement would occur is well beyond practical application limits, the conceptual model should address this upper limiting value. Otherwise, the model would predict an infinite linear increase in shear strength with ever increasing fiber content, without regard to reduction of the effective interface coefficient.

Table 5. Summary of Triaxial Test Results (3-Specimen Series)

Test No.	Fiber Rate - pcf	Ø' Deg	C' psi	$Ø_u$ Deg	C_u psi
Fat Clay (CH), Grayish Brown					
TX-1-1	0 (Raw Soil)	22.3	2.3	13.7	2.4
TX-1-2	0 (Raw Soil)	22.0	2.6	15.1	1.5
TX-1-3	0.17	24.6	2.7	16.3	1.6
TX-1-4	0.17	25.0	2.5	14.7	2.4
TX-1-5	0.25	25.6	2.7	16.8	1.5
TX-1-6	0.25	25.5	2.4	16.5	2.0
Silty Sand (SM), Reddish Tan					
TX-2-1	0 (Raw Soil)	32.6	NA	NA	NA
TX-2-2	0 (Raw Soil)	33.7	NA	NA	NA
TX-2-3	0.17	37.5	NA	NA	NA
TX-2-4	0.17	35.9	NA	NA	NA
TX-2-5	0.25	38.2	NA	NA	NA
TX-2-6	0.25	38.8	NA	NA	NA
TX-2-7	0.50	40.0	NA	NA	NA
TX-2-8	0.50	39.8	NA	NA	NA

The triaxial tests performed to establish an inference of the decay function were numbered with a “C” to indicate that the tests were to help establish the decay curve. The function developed from these tests is discussed later in this chapter.

Table 6. Summary of Triaxial Test Results (1-Specimen Tests)					
Fat Clay (CH), Grayish Brown					
Test No.	**Fiber Rate - pcf**	**Ø' Deg**	**C' psi**	**$Ø_u$ Deg**	**τ psi**
TX-1-C-1	1.0	NA	NA	NA	6.2
TX-1-C-2	1.5	NA	NA	NA	8.7
TX-1-C-3	2.0	NA	NA	NA	14.0
Silty Sand (SM), Reddish Tan					
TX-2-C-1	1.0	51.5	NA	NA	NA
TX-2-C-2	1.5	54.8	NA	NA	NA
TX-2-C-3	2.0	53.6	NA	NA	NA

<u>Summary of Previous AGT Laboratory Triaxial Test Data</u> – The AGT Laboratory (AGT) testing program included three soils, Fat Clay (CH), Sandy Lean Clay (CL), and Poorly Graded Sand (SP). The soil properties are presented in Table 7.

The specimens in the AGT testing program were prepared by the "batch" method discussed in Chapter III. The clay specimens were compacted to approximately 95 percent of Standard Proctor density, near optimum moisture content (ASTM D 698). The sand specimens were compacted to approximately 95 percent of Maximum Index Density (ASTM D 4253). Various fiber lengths and widths were used for the testing program, as shown in the summary of test results in Table 8. However, all the fibers had the same thickness as the fibers used by the author in the current study. Only effective stress test results are listed in Table 8.

Table 7. AGT Soil Properties

Soil Description	Liquid Limit	Plastic Limit	% < No. 200 Sieve
Fat Clay (CH)	68	28	96
Sandy Lean Clay (CL)	27	12	55
Poorly Graded Sand (SM)	NP	NP	< 2

A total of 59 tests, consisting of three specimens each for a total of 177 specimens, are listed in Table 8. The AGT program included other tests, but the details of those tests are not known to the author. Only those tests on which the author is familiar with the test details are listed in the current study.

There was a relatively wide variation in the AGT test results within each soil and fiber type compared to the author's current study. One factor that likely contributed to the wider variation in test results in the AGT program was use of the batch method for preparing and mixing the specimens, rather than preparing and mixing each specimen individually as was done in the current study. The batch method has been found to result in significant variation in fiber content among individual specimens pulled from the batch.

Table 8. Summary of AGT Triaxial Test Results (3-Specimen Series)

Soil Type	Fiber Rate (pcf)	Fiber Size (in)	No. of Tests	Ø' Deg	C' psi
Fat Clay (CH)	0 (raw)	NA	3	18.0	2.1
Fat Clay (CH)	0.20	0.047 x 2.0	7	22.6	1.8
Fat Clay (CH)	0.20	0.1306 x 2.0	3	18.2	2.1
Fat Clay (CH)	0.20	0.047 x 1.0	3	20.9	1.9
Fat Clay (CH)	0.40	0.047 x 1.0	3	24.5	2.0
Lean Clay (CL)	0 (raw)	NA	4	31.5	0.7
Lean Clay (CL)	0.20	0.047 x 2.0	8	34.7	1.1
Lean Clay (CL)	0.40	0.047 x 2.0	5	46.6	1.0
Lean Clay (CL)	0.40	0.1306 x 2.0	4	35.6	0.9
Sand (SP)	0 (raw)	NA	4	34.5	NA
Sand (SP)	0.20	0.047 x 2.0	6	41.6	NA
Sand (SP)	0.40	0.047 x 2.0	5	49.9	NA
Sand (SP)	0.20	0.1306 x 1.0	4	37.7	NA

Direct Shear Test Data

Summary of Direct Shear Test Data – The results of the direct shear tests from the current study are summarized in Table 9. The AGT testing program did not involve direct shear testing. The triaxial shear test is considered to be a significantly superior test for determining the shear strength of soil compared to the direct shear test for most conditions. The primary reasons for the superiority of the triaxial test are that saturation prior to testing can be verified and the pore

Table 9. Summary of Direct Shear Test Results (3-Specimen Series)

Fat Clay (CH), Grayish Brown					
Test No.	**Fiber Rate - pcf**	**Ø' Deg**	**C' psi**	**$Ø_r$ Deg**	**C_r psi**
DS-1-1	0 (Raw Soil)	20.3	3.7	20.0	2.4
DS-1-2	0 (Raw Soil)	19.8	4.2	16.3	2.2
DS-1-3	0.17	22.3	3.1	22.9	2.3
DS-1-4	0.17	22.2	3.8	22.7	2.2
DS-1-5	0.25	23.3	3.5	22.8	3.4
DS-1-6	0.25	24.1	3.6	26.8	1.1
Silty Sand (SM), Reddish Tan					
DS-2-1	0 (Raw Soil)	39.0	NA	NA	NA
DS-2-2	0 (Raw Soil)	38.5	NA	NA	NA
DS-2-3	0.17	42.6	NA	NA	NA
DS-2-4	0.17	43.5	NA	NA	NA
DS-2-5	0.25	46.6	NA	NA	NA
DS-2-6	0.25	47.0	NA	NA	NA
DS-2-7	0.50	48.0	NA	NA	NA
DS-2-8	0.50	49.0	NA	NA	NA

pressure response during shear can be monitored. The stress conditions on the specimen are also better controlled due to the cell pressure. However, the direct shear test is still a common test method in many geotechnical laboratories and is sometimes preferred if shear along a predetermined plane is desired in the test. For this reason, and for the fact that the creep tests performed for this study were configured in the same manner as the direct shear test (common for creep tests), the author elected to perform direct shear tests as part of the current study. The direct shear test results on the clay soil were used as a guide in establishing loading conditions for the creep tests, as described later.

It should be noted that the two direct shear three-specimen test series on sand with a fiber content of 0.5 pcf (8 kg/m^3) showed cohesion values of 5 psi (34 kPa) and 4 psi (28 kPa), respectively (Pages 190 and 191). These relatively large cohesion values were not evident in the triaxial tests on sand with the 0.5 pcf (8 kg/m^3) fiber content. The author believes that the cohesion values in these direct shears tests were caused by scale effects in the smaller direct shear box with the larger fiber content. These cohesion values from the direct shear tests on sand should be considered as a phenomenon of the shear box and should not be considered as valid cohesion values for the fiber-reinforced sand.

Creep Test Data

A plot of the creep test data is presented in semi-log form in Figure 42. The test data are plotted to 20,000 minutes on the time scale, since the time scale is in log form and the test did not run to 100,000 minutes. A plot of the creep test data in arithmetic form is presented in Figure 43. The creep tests were initially loaded in shear to approximately 70 percent of the peak failure stress as determined in the standard direct shear tests for non-reinforced soil, as previously stated. The load was incrementally increased to 90 percent of peak stress but this time based on the non-reinforced soil peak strength for the non-reinforced specimens and based on the peak stress in the direct shear tests for FRS at 0.25 pounds of fibers per cubic foot (4 kg/m^3) of soil. One of the FRS specimens failed when the 90 percent peak stress load was applied at about 16,000 minutes, as shown in Figures 42 and 43. This specimen had been slightly damaged during mounting

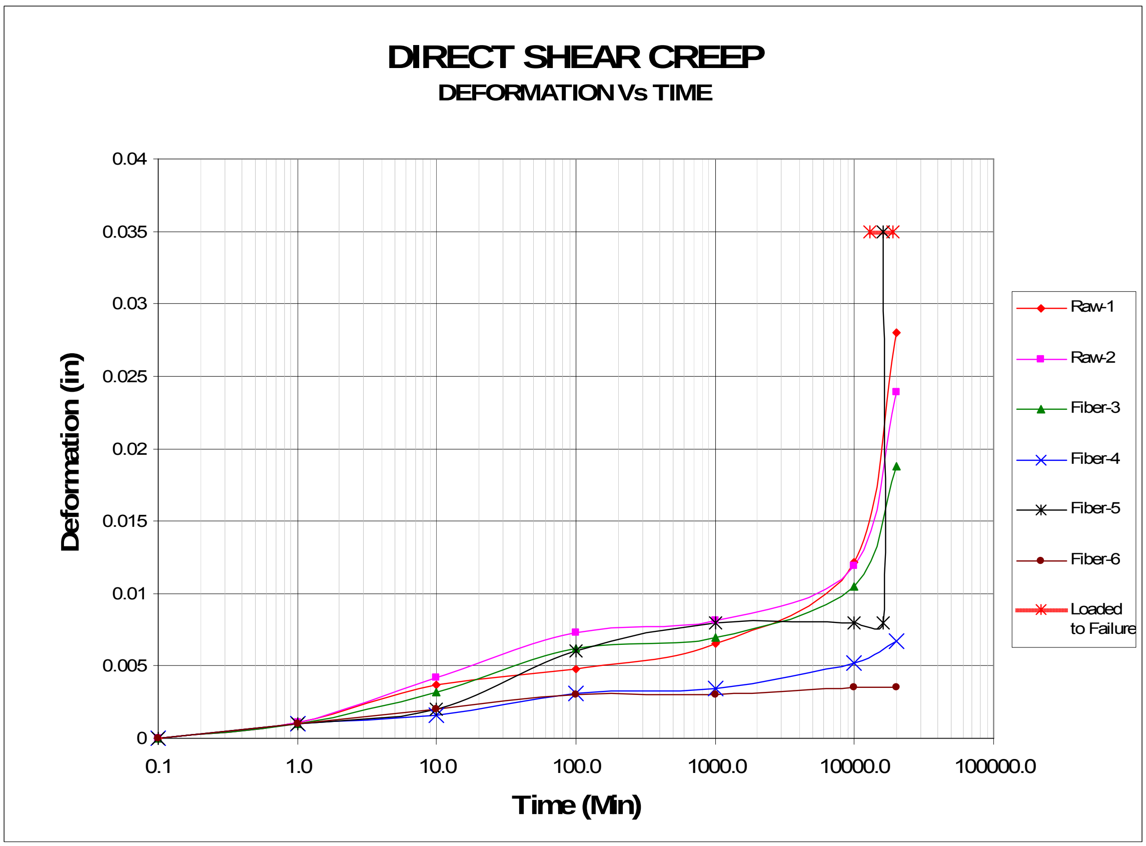

Figure 42. Plot of Creep Test Data in Semi-Log Form

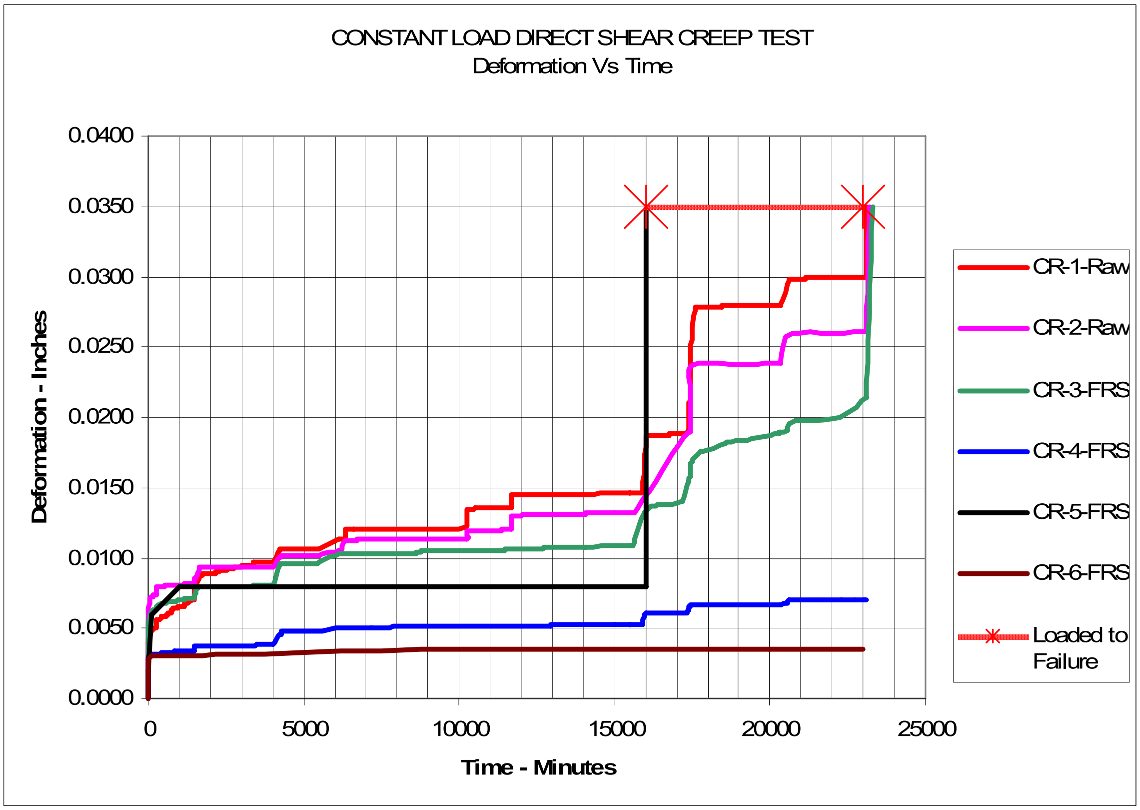

Figure 43. Plot of Creep Test Data in Arithmetic Form

into the creep test device and this likely contributed to the failure. The remaining 5 specimens were loaded to 100 percent of peak shear stress at about 23,000 minutes, at the different stress levels respectively for non-reinforced and FRS specimens. Within 15 minutes the two non-reinforced specimens failed and a short time later one of the FRS specimens failed, as shown in Figure 43. The other two FRS specimens have sustained the 100 percent shear stress loading without any significant additional displacement to date.

The creep response of the raw soil and the FRS indicates that the FRS did not experience as much deformation during the tests as the non-reinforced soil and a much higher stress level was required to fail the specimens. This indicates that

FRS could be used to help prevent long-term creep failure in marginal slope cases where the factor of safety (FS) related to a sliding failure might be marginally acceptable, but would be too low related to creep. Additional study will be required to more fully define the creep characteristics of FRS, but this initial test series indicates that the creep resistance of FRS is significantly greater than the same soil without fiber reinforcement.

Interface Test Data

Interface shear tests were performed to evaluate the interface shear coefficients between the fiber material and the soil, as discussed in Chapter IV. The stress strain curves from the interface tests are shown in Figure 44. Depending on the strain level where the interface value is taken and the shear strength value to which it is compared, the interface coefficient values ranged from about 0.4 to 0.5 rounded to one decimal place.

Considering all the other variables involved and the limited number of interface test results available, the interface coefficient should be taken to only one decimal place. A reasonable value for both sand and clay would appear to be about 0.5. The correlations between actual test results and model predictions fit well using interface coefficients of 0.5. It should be noted that the interface coefficients were determined by dividing the shear stress from the interface tests by the corresponding shear stress from the triaxial tests on raw soil. This is equivalent to the decimal percent efficiency of the fiber material in interface shear.

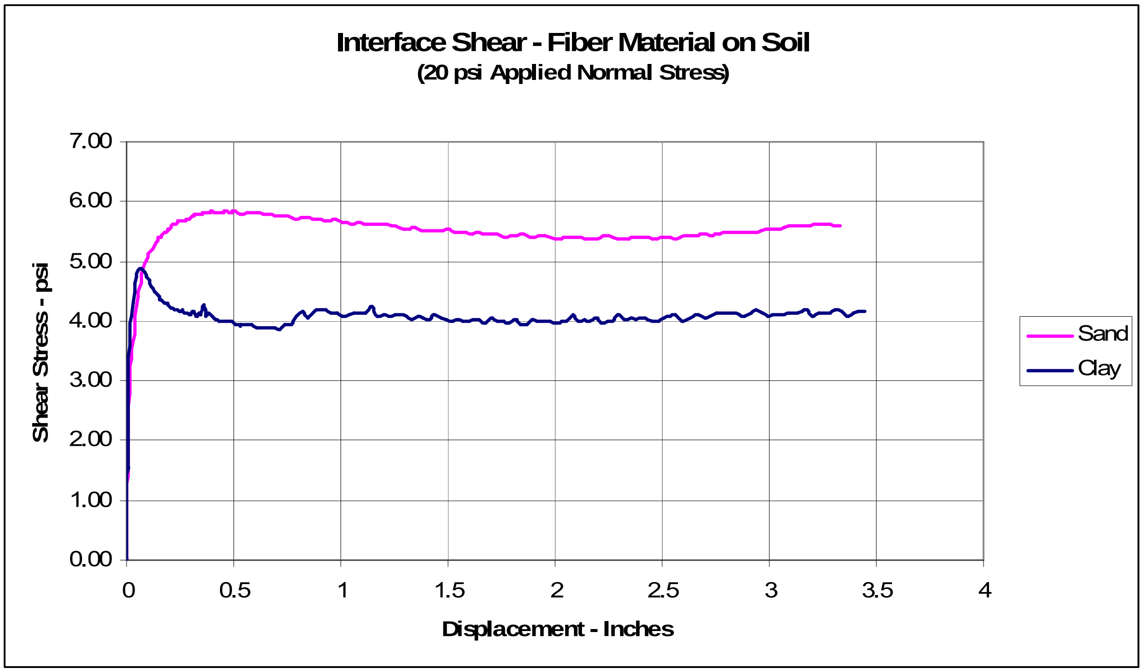

Figure 44. Interface Shear Test Results – Fiber Material on Soils

Correlation of Shear Strength with Conceptual Model

Conceptual Model Calculations – A computer spread sheet was developed to perform calculations of predicted FRS shear strength using appropriate equations from the conceptual model discussed in Chapter III. Calculations were performed to predict the FRS shear strength parameters ϕ and c, with input of the raw soil ϕ and c, the fiber application rate, and the fiber properties. Only effective stress parameters were considered as previously discussed in this study. The correlations of frictional shear strength and cohesive shear strength are discussed separately.

<u>Frictional Strength Correlations for Current Test Results</u> - The calculated values of FRS ϕ using the model were plotted versus the results of the FRS tests performed during the current study, and the data were analyzed statistically with respect to correlation coefficient (R^2) and slope of the linear regression line. The results of both the clay and sand specimens with regard to frictional strength increase and both triaxial and direct shear test results are included in the plot. To make a comparison of predicted versus test data, the two triaxial test results for raw soil for the clay were averaged to obtain a single result for input into the model to predict FRS strength values to compare to the actual triaxial test results. The two direct shear test results for raw soil for the clay were also averaged to obtain a single result for input into the model to predict FRS strength values to compare to the actual direct shear tests on the clay soil. The same procedure was used for the sand. The goal of the analysis described here is to evaluate the accuracy of the model with respect to predicting FRS frictional strength values, and not to evaluate the consistency of the test data. The plot of FRS frictional strength predicted by the model versus the actual corresponding test results is presented in Figure 45. The plotted values are actually tangent ϕ rather than ϕ as shown on the graph. A linear regression trend line of the points is plotted on the graph, along with the equation of the line and the R^2 value. The slope of the line is reasonably close to 1 as shown by the equation of the line, and the R^2 value is above 0.94. These values indicate a very good fit of the model to the test results for the available data.

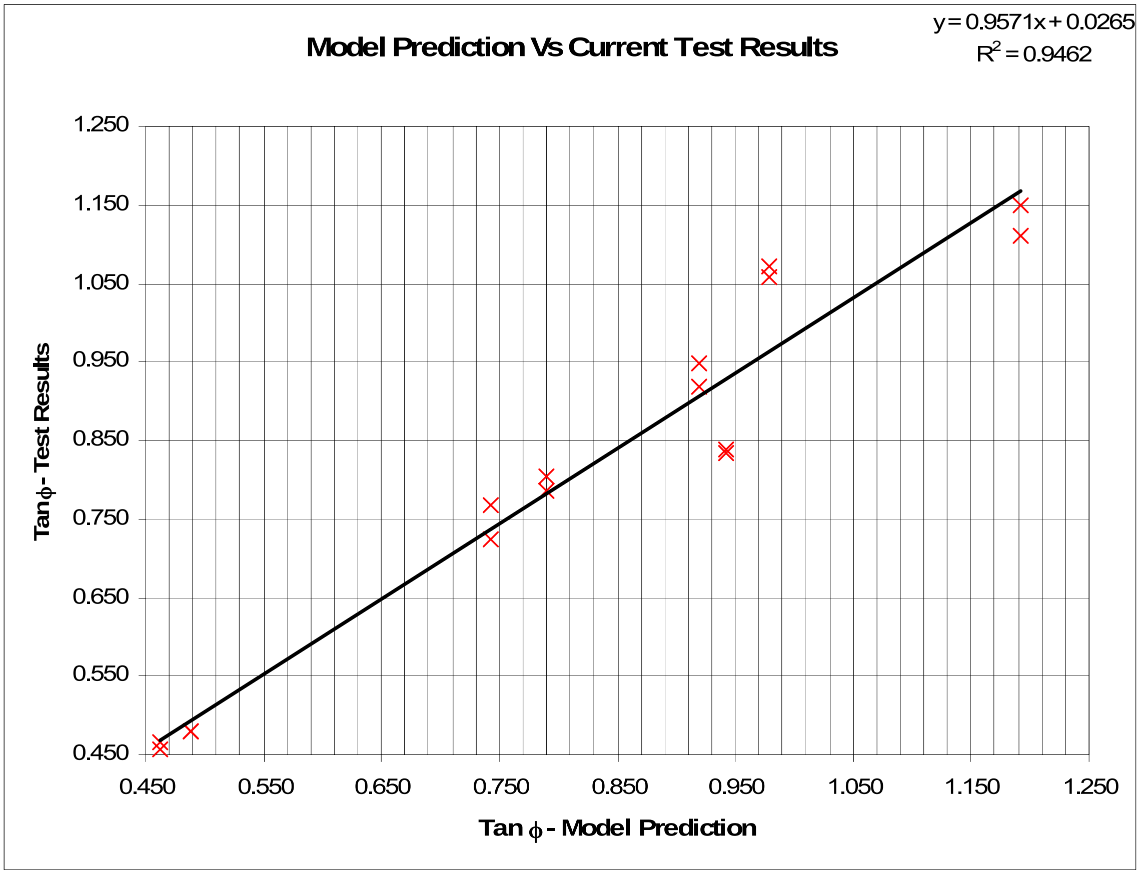

Figure 45. Model Versus Current Test Results for Tan Ø

<u>Frictional Strength Correlations for Previous AGT Test Results</u> - Model predictions were made of the FRS strength parameters for the soil materials tested in the AGT program and the results were plotted as described for the test results for the current study. The plot of this data is presented in Figure 46. The slope of the linear regression line in Figure 46 is not as close to 1 as previously discussed for Figure 45, and the R^2 value is a little lower. However, the slope is still reasonably close to 1 and the R^2 value is greater than 0.92. These values also indicate a very good correlation between the model predictions and the AGT test data.

Cohesive Strength Correlations for Current Test Results – Increase in cohesive strength predicted by the model was compared with actual test results on the clay soil from the current study. The results are presented in the plot in Figure 47, as described previously for the frictional shear strength parameters.

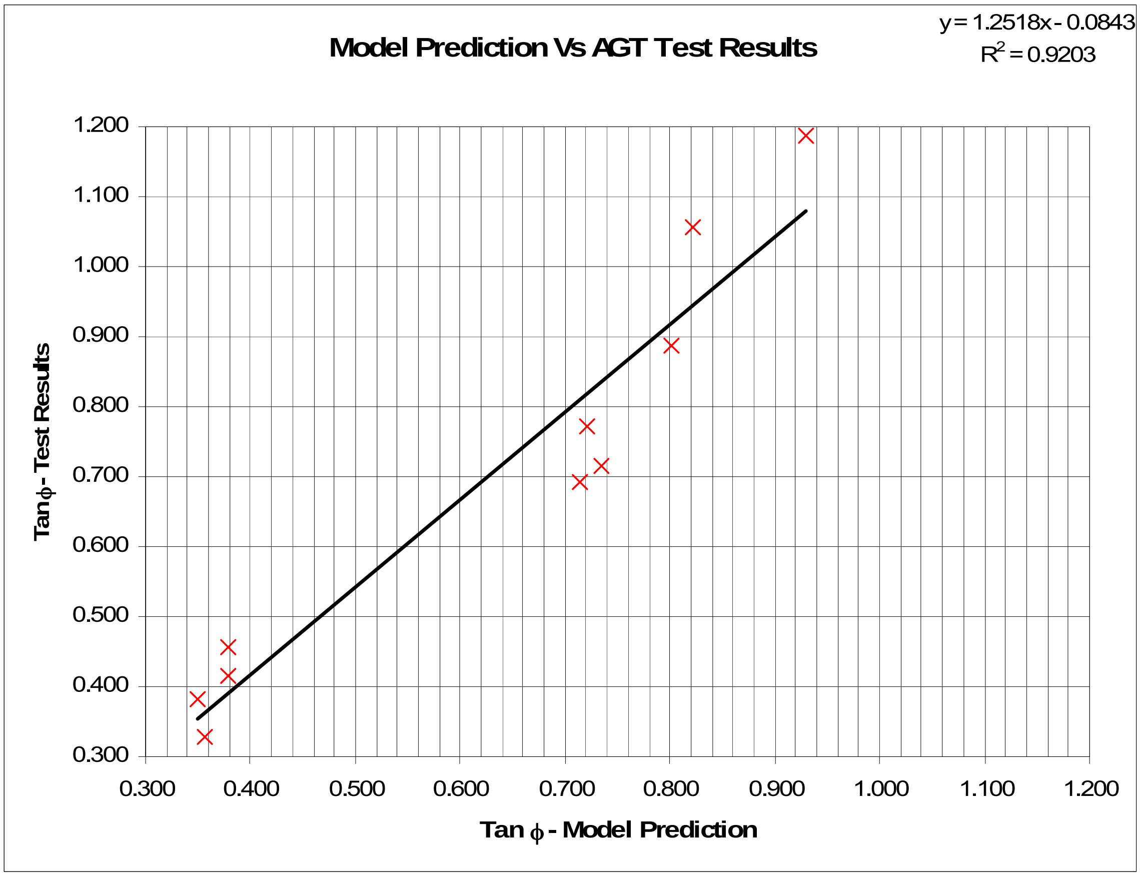

Figure 46. Model Versus AGT Test Results for Tan Ø

The cohesive strength (c) values in Figure 47 have been normalized to make them non-dimensional by dividing the actual values by atmospheric pressure (Pa), as shown in the figure. The regression line is not as good a fit for the cohesive strength comparisons as for the frictional strength comparisons. However, the slope of the regression line of approximately 0.6 and the R^2 value of approximately 0.84 indicate a good fit of the data. This is especially true since the cohesive strength values tend to be significantly more variable in laboratory test results than frictional strength values.

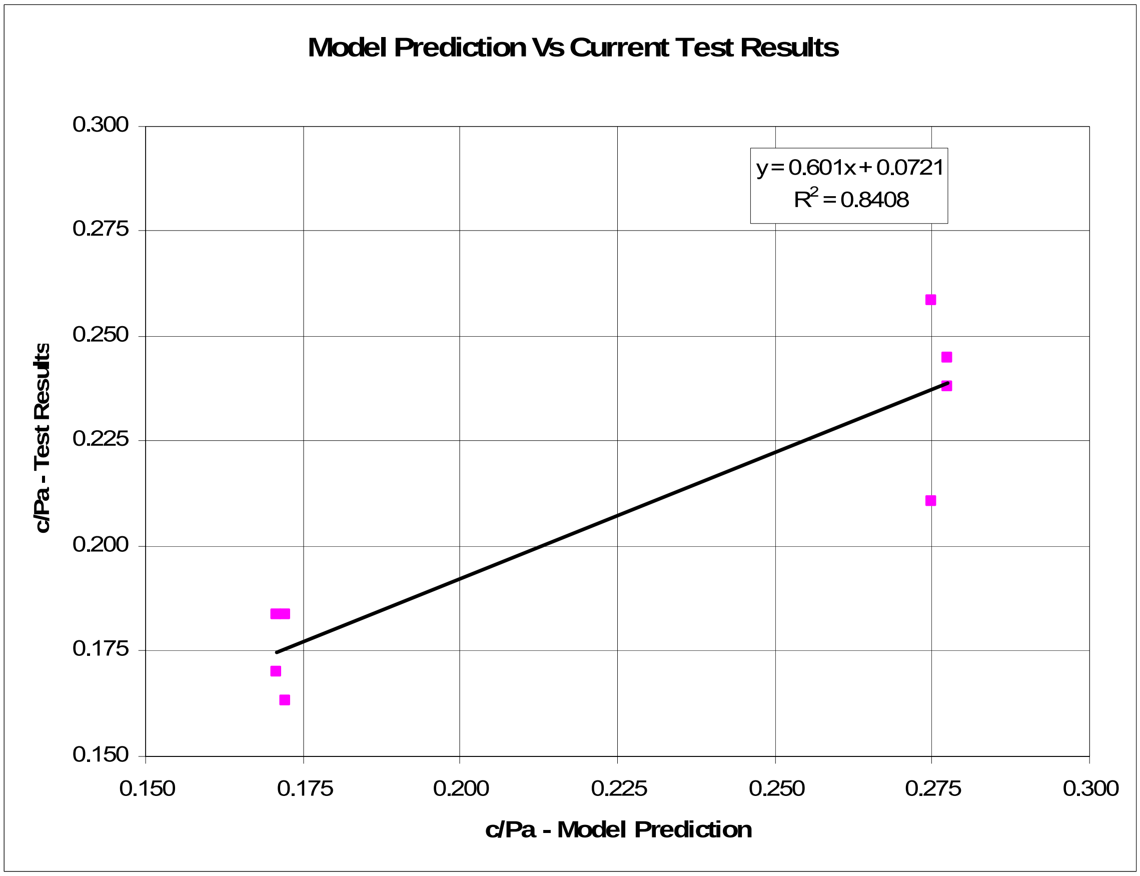

Figure 47. Model Prediction Versus Current Test Results for c

<u>Cohesive Strength Correlations for Previous AGT Test Results</u> – Cohesive strength test results from the previous AGT testing program were compared to model predictions as previously described for the frictional strength. The plot of the data is shown in Figure 48. The actual cohesion values were divided by atmospheric pressure to normalize them to a non-dimensional form as previously stated for the current test comparisons. The regression line for these data still has a reasonably good slope of approximately 0.7. The R^2 value of approximately 0.78 is not as good as the 0.84 value from the current study, but

still indicates a reasonably good fit for cohesion values.

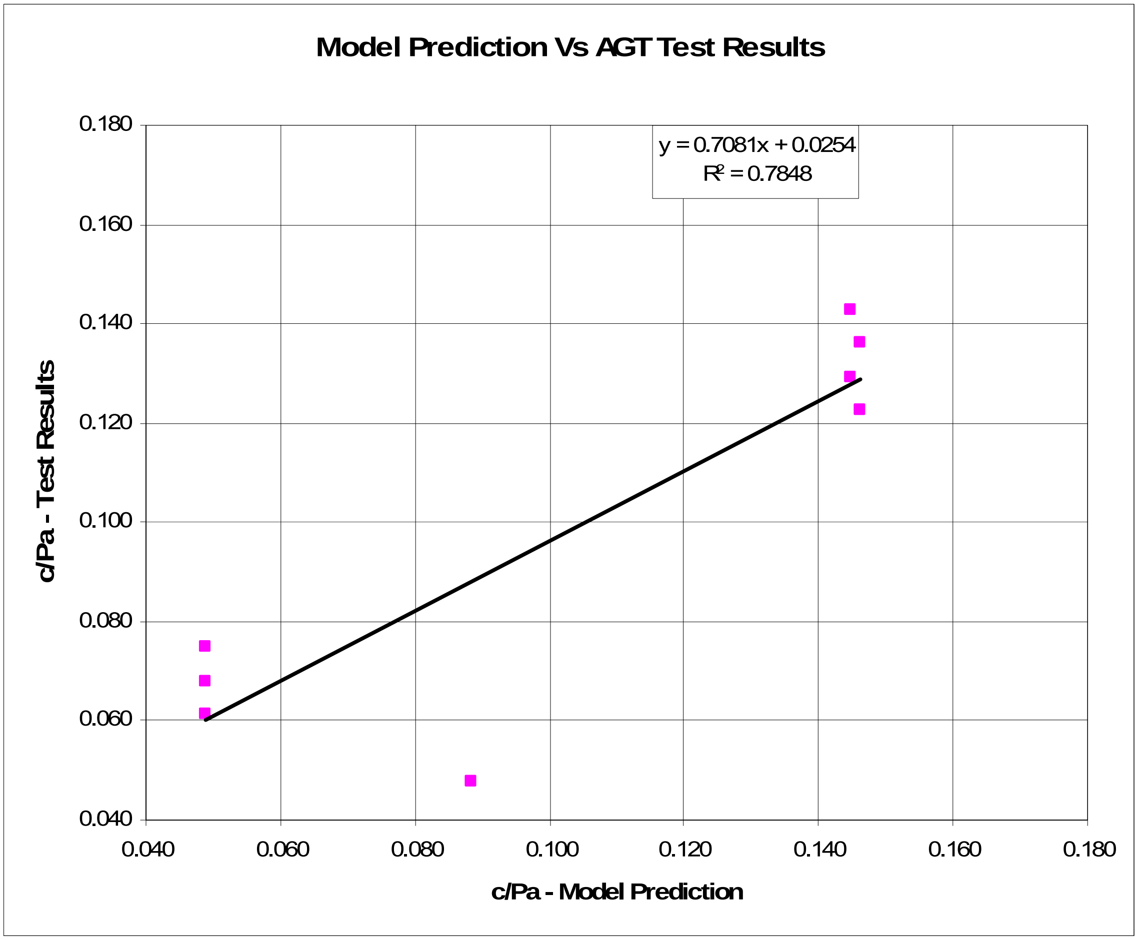

Figure 48. Model Prediction Versus AGT Test Results for c

Calibration of Conceptual Model

Calibration of the conceptual model includes establishing appropriate input variables and verification that the model can predict FRS shear strength parameters with reasonable accuracy. In developing a model, especially one for soil response under loading, it is sometimes necessary to include one or more scaling factors to be used to bring the theoretical model into more close agreement with actual test data or experience with soil response. Based upon the reasonably good agreement between predicted FRS properties and actual test results considered in this study, it appears that a scaling factor is not needed for the model. This is especially true considering the inherent variability of laboratory test results on soils, even when a very high level of care is exercised in conducting the tests.

Input parameters for the conceptual model include the fiber properties, non-reinforced soil properties, interaction coefficients, and the coefficient of earth pressure at rest (K_o) as illustrated in Chapter III. The fiber properties are well known for the manufactured fibers. The raw soil properties can be easily tested in the geotechnical laboratory or estimated with reasonable accuracy from local experience based upon a data base of previous testing. The interaction coefficients need more consideration and more interface testing to fully establish values for a range of soils. However, based on the interface tests performed for this study and the reasonably good fit of the data, a value of approximately 0.5 for both the silty sand and fat clay seems reasonable. Several methods of calculating K_o were evaluated in the model, including the typical expression of $K_o = 1\text{-}\sin\phi$ for normally consolidated clay. None of the more elaborate methods of calculating K_o appeared to improve the model prediction.

<u>Decay Function for Large Fiber Content</u> – As previously discussed earlier in this chapter, a decay function is desirable in the conceptual model to account for the increase in fiber-to-fiber contact and the decrease in fiber to soil contact as the

fiber mixture rate becomes increasingly large. Results from the triaxial shear tests on clay and sand were used to develop the decay function. The full range of the fiber content tested for the clay and sand specimens was used in developing the curve, including the single specimen tests at 1, 1.5, and 2.0 pcf (16, 24, and 32 kg/m^3). The curve was developed by back-calculating the value of the interaction coefficient that was required to match the test results for each of the fiber contents, then dividing by the standard interaction coefficient of 0.5 (discussed previously in this chapter) to obtain the reduction factor for each fiber content. There was essentially no reduction required for fiber contents of 0.17 and 0.25 pcf (2.7 and 4 kg/m^3). The reduction factors were then used to obtain exponential functions to fit the data points for the clay and sand, and for the average of the clay and sand. The curves and equations are shown in Figure 49.

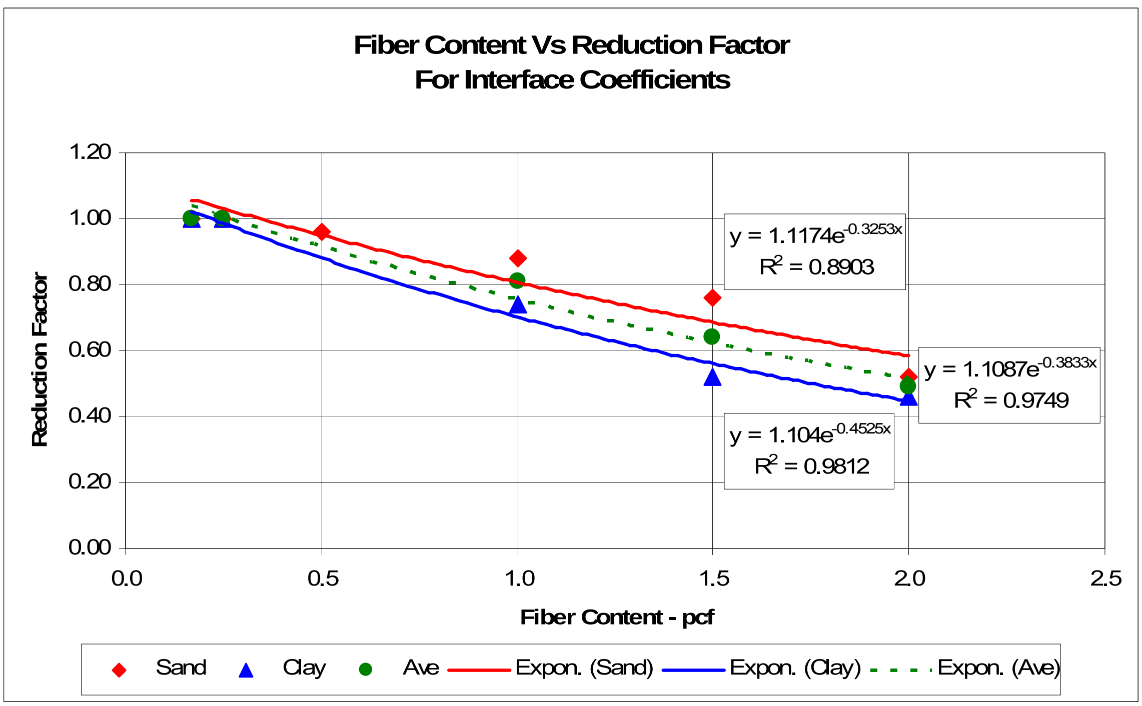

Figure 49. Fiber Content Vs Reduction Factor for Interface Coefficients

Both the sand and clay test data show that there is no significant decay in strength improvement until well above any practical mixture rate for fibers in FRS.

Based upon the above information and considerations, calibration of the model should be sufficiently complete for use in practical applications. Use of the model in slope stability analysis and application of FRS for slope stability are discussed in the next chapter.

CHAPTER VI

APPLICATION OF FRS IN SLOPE STABILITY

Slope Applications

FRS has a significant potential for use in a wide range of slope applications (Gregory and Chill, 1998; Gregory, 1999b). These applications include repair of existing slope failures and efficient construction of new slopes. FRS has proven most efficient for application in shallow slope failures where the failure surface is approximately 10 to 12 feet (3 to 4 m) in depth or less, and as secondary reinforcement between layers of planar reinforcement (i.e. geogrids). For deeper slope reinforcement, geogrids are typically more cost effective. However, in these cases the FRS can be used very efficiently for secondary reinforcement (Gregory, 1998d).

Since the fibers are essentially a soil additive, they reinforce the entire soil mass with the same capacity throughout. For example, consider a 2-inch (50 mm) long fiber, which requires only 1-inch (25 mm) of anchorage zone to develop the full design capacity of the fiber. This is in contrast to planar reinforcement such as geogrids or high-strength geotextiles that may require one to three feet (0.3 to 1 m) or more of anchorage zone to develop the full capacity of the planar reinforcement. This is an important aspect in numerous project applications such as slope repairs on highways where the shoulder or part of the outside lane may require removal to provide an anchorage zone for planar reinforcement whereas the shoulder or pavement edge may be left in place if FRS is used for the repairs (Gregory and Chill, 1998; Gregory, 1999b).

Unlike geogrids or geotextiles, the fibers are not damaged by normal earthwork construction operations such as processing of the soil with rotomixers, disc plows, or compaction equipment. This feature of FRS facilitates slope construction or slope repairs in constricted areas. The fibers are easily mixed into a wide range of soil types with a rotomixer or pulverizer mixer of the same type used for lime-soil mixing and can be compacted with conventional equipment such as tamping foot rollers.

Slope Stability Analysis of FRS Slopes

<u>Analysis Using Existing Computer Programs</u> - Analysis of FRS slopes can be accomplished using existing slope stability computer software for limit equilibrium analysis. A relatively simple spread sheet can be developed for calculating the FRS ϕ and c using the equations presented in Chapter III. The FRS ϕ and c can then be input into the slope stability program for the FRS zones and the analysis can proceed as usual. Since the soil parameters have to be determined by laboratory testing or estimating from previous experience with the same soils, there is no difference in requirements for the slope analysis, except for

calculating the FRS strength parameters using the conceptual model. Several different fiber contents can be evaluated until the required FS is achieved in the analyses.

Analysis Using Modified Computer Programs – More efficient analyses could be conducted if the conceptual model for FRS was integrated into a limit equilibrium slope program. This would eliminate the need for developing a spread sheet for calculating the FRS properties and would make multiple runs to find the required fiber content more efficient. This approach has been accomplished by the author as part of the research program for this dissertation. The author's existing slope stability analysis computer program "*GEOSTASE*" has been modified to include the conceptual model. The program includes a very user friendly GUI (graphical user interface) that allows all the input values to be entered in an interactive manner with dialog menus for all input items. The fiber properties and non-reinforced properties for the various zones in the slope are input and the program internally calculates the FRS properties and uses those properties in the slope analysis. The FRS properties are included in the output. Examples of the program output are included in Appendix C for the case history projects discussed in Chapter VII.

CHAPTER VII

CASE HISTORY PROJECTS

PGBT Turnpike Project

Project Description - The PGBT (President George Bush Turnpike), is named after the former President Bush and is located in the Dallas, Texas area. It is a multi-segment 6-lane toll road that has been constructed over approximately the past five years to help relieve some of the ever-increasing vehicle traffic in the Dallas area. The portion of the project that the author was involved in is a 6-mile long north-south segment that is located in the Farmers-Branch and Carrollton, Texas areas. The project involved a large element of subsurface stabilization of problematic soils areas on which the author performed the geotechnical design. The project also involved a large amount of soil embankment construction with embankment heights ranging from about 15 feet to over 35 feet. The project is located within the Eagle Ford Shale geologic formation, with residual soils consisting largely of highly expansive fat clays. These clay soils are essentially the only earth fill material available at affordable cost for construction of the embankments. These soils are known to experience widespread shallow slope failures within a few years after embankment construction for slopes about 15 feet (4.6 m) or more in height and that have slope ratios of 4 (4 horizontal to 1 vertical) or steeper. Once these shallow failures begin they are very expensive and inconvenient to repair on an active highway. If not repaired in a timely manner, the failures become progressive and soon impact the shoulder and roadway pavements.

<u>FRS Application in Project</u> – The author recommended the use of FRS in the top 6 feet (1.8 m) of the side slopes as a preventive maintenance measure to significantly reduce the potential for the shallow slope failures. The recommendation included all slopes that were 15.5 feet (4.7 m) in height or taller and that had slope ratios of 4 or steeper. A portion of the project also included geogrid reinforcement of an embankment area that had to be constructed with a slope ratio of approximately 2 to prevent encroachment onto an adjacent closed landfill site. The author also recommended FRS as secondary reinforcement between the geogrids layers in this area. The recommendations were accepted. The author performed slope stability analyses to determine the fiber application rate based upon an earlier less complete conceptual model. However, since the author was aware that the model was not fully developed, a conservative approach was taken in the design. The slope stability was re-evaluated as part of the current study using the new model as discussed in the next section. The FRS volume on this project is the largest ever used to date on an earthwork project. Approximately 520,000 pounds (236,000 kilograms) were used on the project at an application rate of 6 pounds per cubic yard (3.6 kilograms per m^3). Photographs of the FRS construction are included in Figures 50 and 51.

Figure 50. Spreading Fibers for FRS on PGBT Project

Slope Stability Analyses – As previously discussed in Chapter VI, the author of this dissertation is also the author of a comprehensive slope stability analysis computer program that is distributed commercially and is in widespread use. As part of the current study, the author modified the program to incorporate the new model for FRS. The slope stability analysis output for the PGBT project is included in Appendix C. The output includes graphics of the slope profile and the text output. The refined analyses show that the slopes as designed have the intended FS values. The analyses were performed for the new slope profile without any reinforcement and with FRS in the appropriate zones to illustrate how the use of FRS significantly increased the FS. In the actual analyses originally

Figure 51. Mixing FRS on PGBT Project

performed for the project numerous other computer runs were performed for various conditions. However, since the analyses performed in the current study were to illustrate the use of FRS, only the two comparative analyses with and without FRS are included. The analyses are for the shallow slope zone (veneer) as shown on Plates C.1 and C.2 in Appendix C. The calculated FS for the veneer without the FRS is approximately 1.29 (Plate C.1), which equates to a stress ratio of approximately 0.78 (reciprocal of the FS). This is well above the potential creep failure threshold of about 0.7 for clay slopes (Sowers, 1979, 1984). The FRS veneer has a calculated FS of approximately 1.52 (Plate C.2), which equates to a stress ratio of approximately 0.66, well below the 0.7 threshold.

Project Related Testing – An extensive laboratory testing program had been conducted during the design phase of the project to establish the standard properties of site soils. This information was used during design of the FRS portion of the project. During construction of the FRS, the author's firm performed periodic testing of FRS as a means to help verify compliance with respect to fiber application rate. The test procedure involved processing the FRS specimens obtained from the field through a sieve to determine the fiber content of each specimen. This process is discussed further in the second case history project.

Project Performance - Embankment construction in the FRS areas was completed in late 2004. The embankments have performed well to date, however a number of years will be required to fully evaluate the performance.

Lake Ridge Parkway Slope Repair Project

Project Description - This project is located along Joe Pool Lake in the city of Grand Prairie, Texas. The existing embankment slopes had been constructed by the US Army Corps of Engineers in about 1980 to raise the roadway level above the proposed normal pool level of Joe Pool Lake, which was under construction. This project is also located within residual soils of the Eagle Ford Shale geologic formation. The slopes were constructed of fat clay soil with a side slope ratio of 3 and heights ranging from about 10 to 25 feet (3 to 7.6 m). Within about 5 to 8 years after construction, the embankment slopes began to experience shallow slope failures.

The City of Grand Prairie (owner of the roadway) began performing minor slope repair maintenance on the roadway slopes. By 2003 the slope failures had become progressive and had slightly damaged a portion of the roadway pavement. Approximately 2,000 linear feet (600 m) of one lane adjacent to the slope had to be shut down and barricaded to traffic. The author was retained to perform a geotechnical study and work with the project design team to develop a repair method for the slopes. The total length of distressed slope was in excess of 6,700 linear feet (2,000 m).

FRS Application in Project - After evaluating numerous alternatives, FRS was selected as the repair method for the slopes. Eight soil borings were performed and 4 inclinometers were installed to help locate the depth to the failure surface. Numerous borings were sampled continuously, and all soil samples were retained in the author's laboratory following laboratory testing for the geotechnical study. The City elected to repair about 3,700 linear feet of the most distressed slopes in the first phase of the repairs and to follow with another phase within one or two years. An application rate of 6.75 pounds per cubic yard (4 kg/m^3) was used on the project. Approximately 365,000 pounds (166,000 kilograms) of fibers were used on the project, making it the second largest volume of FRS used on an earthwork project.

Obtaining Soil Samples for Research Testing – The clay soil for the research testing for this dissertation was taken from the unused soil from the borings performed by the author's firm for the Lake Ridge Parkway project, as previously described in Chapter IV. Six Shelby tube samples were also obtained of the FRS during construction for additional research testing, as described in the next section.

Project Related Testing – Geotechnical laboratory testing was performed during the design phase of the project to establish shear strength and index properties of the project soils. During construction, fiber content testing was performed as described for the PGBT project.

As part of the current research study, six Shelby tube samples of the FRS was obtained from the site during construction. The samples were obtained from FRS after compaction in the embankment. The samples were returned to the author's laboratory and six specimens were trimmed from the samples for triaxial shear testing. These tests were performed as a means to illustrate that FRS can be tested for shear strength during construction in the same general manner that the other triaxial tests were performed for this study. The test results on the field specimens are included in Appendix A. One of the dissected field specimens following testing is shown in Figure 52. Photographs of the mixer (custom fabricated from a drill press) used to process the FRS field specimens into slurry prior to sieving, and of the sieving process to determine fiber content are presented in Figures 53 and 54. Note that the fibers in the field samples are black in color, depicting the carbon black content included in fibers to be used in actual construction to limit ultra violet damage from sunlight. The fibers used in the laboratory research testing were opaque without the carbon black additive. The carbon black does not change the fiber strength properties as demonstrated by fiber material properties tests performed by the manufacturers on both carbon- black treated and non-treated polypropylene material.

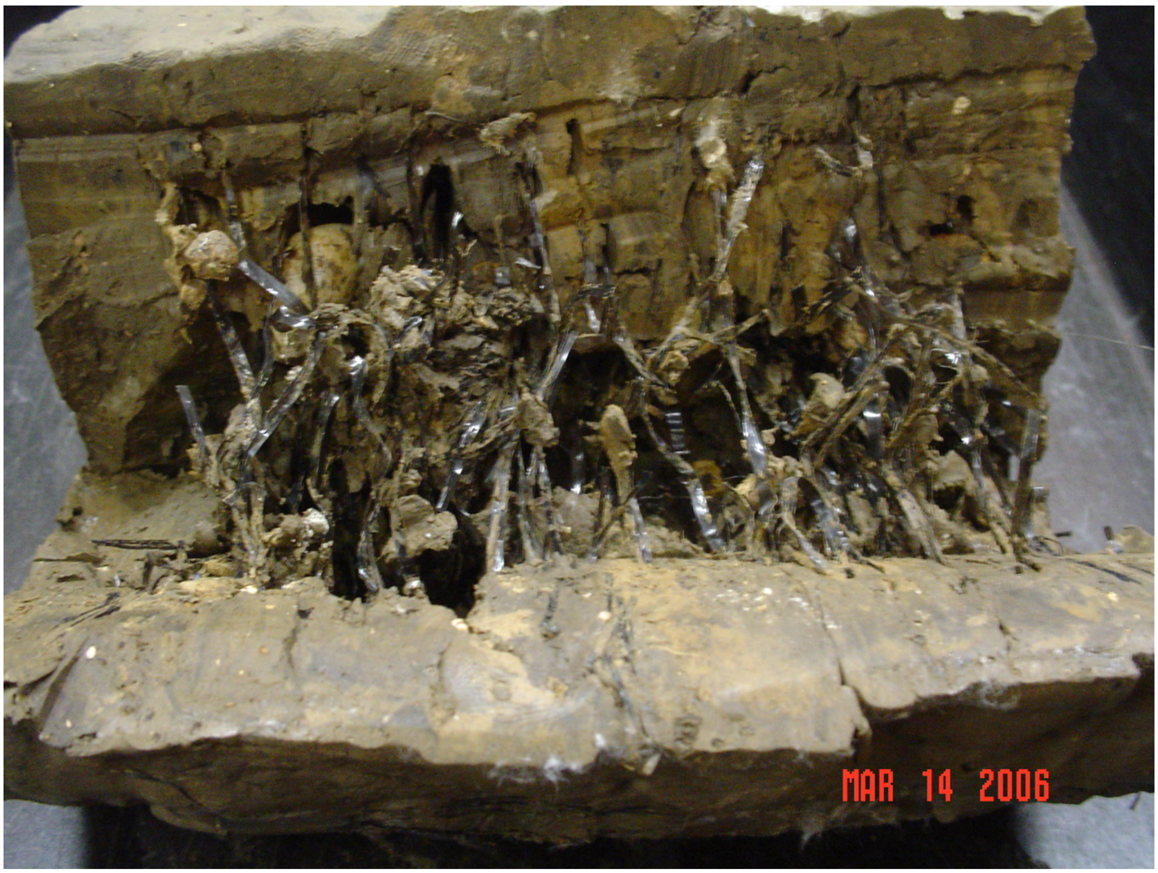

Figure 52. Dissected Field Specimen Following Triaxial Test

Figure 53. Mixer for Processing Fiber-Soil Specimen Into Slurry

<u>Slope Stability Analyses</u> – Slope stability analyses were originally performed by the author during the design phase of the project. These analyses had been performed with a preliminary model as previously described for the PGBT project, and conservative assumptions were made regarding the shear strength of the FRS. As part of the current research study, the author re-analyzed the slopes for the Lake Ridge Parkway project using the new model incorporated into the slope stability program. The required FS values had been achieved originally due to the conservative assumptions. The results of the current slope stability analyses are included in Appendix C and are included on Plates C.3 and C.4.

Figure 54. Sieving of Slurry to Extract Fibers

The analyses were performed for the reconstructed slope profile without any reinforcement and with FRS in the appropriate zones to illustrate how the use of FRS significantly increased the FS. In the actual analyses originally performed for the project numerous other computer runs were performed for various conditions including the initial failure condition, rapid drawdown, and end of construction. However, since the analyses performed in the current study were to illustrate the use of FRS, only the two comparative analyses with and without FRS were included. The slope without FRS has a calculated FS of approximately 1.32 (Plate C.3) or a stress ratio of about 0.76, above the threshold for potential creep failures. The calculated FS with FRS in the slope is approximately 1.51, which equates to a stress ratio of approximately 0.66, well below the creep failure threshold.

Project Performance – Construction of the slope repairs with FRS was completed in September 2005. The slopes have performed well to date, however a number of years will be required to fully evaluate the performance. The author is currently involved in the geotechnical design of the second phase of slope repairs on the next section of the roadway for the City of Grand Prairie.

Photographs of the initial slope failure along the roadway are presented in Figures 55 through 57. Photographs of the FRS construction and the completed embankment are presented in Figures 58 through 62.

Figure 55. Slope Failure on Lake Ridge Parkway

Figure 56. Slope Failure Scarp at Roadway Edge – Lake Ridge Pkwy

Figure 57. Slope Failure at Roadway Edge – Lake Ridge Pkwy

Figure 58. Initial Excavation for FRS Slope Repair – Lake Ridge Pkwy

Figure 59. Partially-Used Fiber Supply Bag – Lake Ridge Pkwy

Figure 60. FRS Embankment Construction- Lake Ridge Pkwy

Figure 61. Down-Slope View of Completed FRS Embankment Prior to Grass Establishment - Lake Ridge Pkwy

Figure 62. Up-Slope View of Completed FRS Slope Prior to Grass Establishment (Existing Soil-Cement in Foreground) – Lake Ridge Pkwy

CHAPTER VIII

CONCLUSIONS AND RECOMMENDATIONS

Summary

Shear strength, creep, and stability of fiber-reinforced soil (FRS) slopes have been the subject areas of this study. The main focus of the research was to perform a comprehensive laboratory testing program of non-reinforced (raw) soil and FRS to provide data to complete and validate a conceptual model previously proposed by the author for calculating the increase in soil shear strength by addition of fibers to the soil. The primary goal was to develop the model to the extent that it can be used with confidence to predict the FRS shear strength based upon knowledge of the raw soil properties and the fiber properties without requiring extensive laboratory testing of FRS for specific project use. Secondary goals were to perform and analyze a limited program of laboratory creep testing of raw soil and FRS to get an initial indication of the potential for improved creep resistance of FRS compared to non-reinforced soil, and to incorporate the conceptual model into a limit equilibrium slope stability analysis computer program to facilitate analysis of FRS slopes. These goals were accomplished in this research study and are discussed further under Conclusions.

Conclusions Regarding Laboratory Test Results

The laboratory test results obtained during this study are very consistent for soil materials. The results varied for the same soil between triaxial shear tests and direct shear tests. This is a common occurrence and is well known in the geotechnical engineering profession since the shearing mechanism is very different in the two tests. The test results in each group (triaxial and direct shear) were very consistent and reasonable within each group. The results show progressive increase in shear strength of FRS with additional fiber content up to the point where decay of the strength improvement begins due to very large fiber content. This decay limit appears to be about 1.5 pounds of fibers per cubic foot (24 kg/m^3) of soil. This value is well above any practical application rate. However, it was desirable to define a decay function for the model so that it would not predict a linear gain in FRS shear strength without limit as the fiber content is increased above the decay limit. The results of the laboratory tests are much more consistent than tests performed using the batch method of mixing multiple FRS specimens at the same time rather than mixing each specimen individually as was done in this research study. This is likely the primary reason for the higher level of consistency of laboratory test results in the current study.

Conclusions Regarding Conceptual Model Development

The conceptual model was refined and extended from the preliminary model formerly proposed by the author. Predictions of FRS shear strength using the new conceptual model fit well with the laboratory tests on both clay and sand from the current study, and also fit well with a substantial body of test results performed by AGT Laboratory. A decay function was added to the model as previously described. The author believes that the conceptual model is sufficiently complete and accurate to be used in practice for the general soil types and conditions considered in the research study. However, considerable engineering judgment and experience must be prudently applied in all cases of FRS applications in slope stability.

Conclusions Regarding Application of Model

The model can be applied to slope stability projects by using a simple spread sheet to predict the FRS shear strength of soils for which the non-reinforced (raw) soil strength parameters and fiber properties are available. The required fiber properties are readily available from the fiber manufacturers. The predicted FRS shear strength can then be input into a conventional slope stability computer program and the slope analyzed in the usual manner. A significant improvement to the application of the model is to incorporate it directly into a slope stability computer program so the FRS properties are calculated and applied internally in the program. This has been done in the author's slope stability computer program. The program was verified by comparing the output of FRS strength values with the spread sheet analysis and with hand-worked examples.

The final form of the equations developed in the model for calculation of FRS shear Strength parameters are repeated here.

$$\Delta\phi_{frs} = \tan^{-1}\left[a_{re}K_e f_\phi V_r \tan\phi\right] \tag{14e}$$

$$\tan\phi_{frs} = \tan\phi + \Delta\phi_{frs} \tag{14f}$$

$$\Delta c_{frs} = \tau_{frsc} - \sigma_r(\tan\phi_{frs} - \tan\phi) \tag{17a}$$

$$c_{frs} = c + \Delta c_{frs} \tag{17b}$$

(Symbols as previously described in Chapter III and in Nomenclature on pages xiii and xiv).

It should be noted that the model will work well for predicting the shear strength of soils naturally reinforced with plant roots. The key element in this case will be estimating the root properties. The strength properties of roots have been studied by others for this purpose (Shields and Gray, 1992).

Recommendations Regarding Project Applications

The most obvious applications of FRS are for shallow slope failure conditions where the failure surface zone is about 12 feet (3.7 m) or less in depth and for use as secondary reinforcement in conjunction with geogrids used as primary reinforcement for deeper slope failure conditions. FRS should be considered for general use as veneer reinforcement in all new slopes that have the potential for developing shallow slides and that will be difficult to repair or maintain such as highway embankment slopes. FRS has a good potential for use in landfill soil cover stabilization, for use as reinforcement in soil veneer over lightweight geofoam fill, and as key-trench fill (Gregory, 1999b).

Recommendations for Future Research

Much additional research is desirable for FRS. While this study has been comprehensive with respect to shear strength of FRS consisting of one type of synthetic fibers and two different soil types (clay and sand), much useful information could be obtained from research involving other fiber types, especially fibers with different surface texture or roughness compared to the fibers currently commercially available and used in this study.

The current study included only a nominal program of creep testing of clay soils reinforced with fibers. Although this program is the first one conducted for the specific purpose to the author’s knowledge, it was of necessity limited in scope. A future comprehensive creep testing program involving many specimens and many different stress levels with a broader range of fiber contents would be necessary and desirable for more fully defining the creep characteristics of FRS compared to non-reinforced soil.

A much larger data base of interface test results is needed. The interface testing should include a range of soil types and multiple stress levels in the interface tests to more fully establish the range of interface friction and adhesion coefficients for use in the conceptual model.

Closure

The information contained in this dissertation is based upon an academic research study. Future research could change some of the conclusions contained in this study. The information may not be applicable to some conditions and may not be suitable for some applications. The applicability and appropriateness of this information for project use must be evaluated in detail by the engineer of record for the particular project. Any use of the information contained in this dissertation for actual project application is at the sole risk and responsibility of the user.

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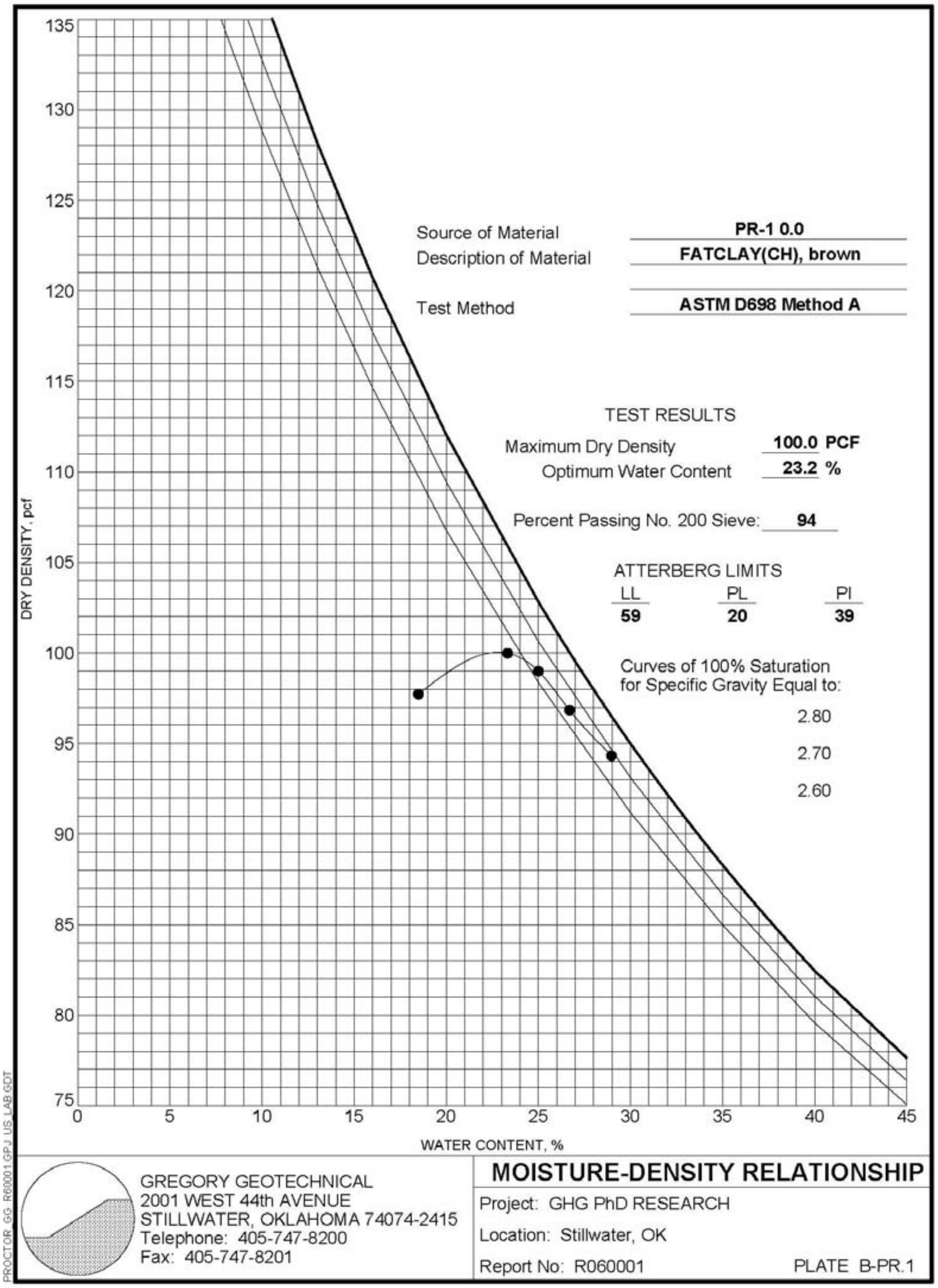
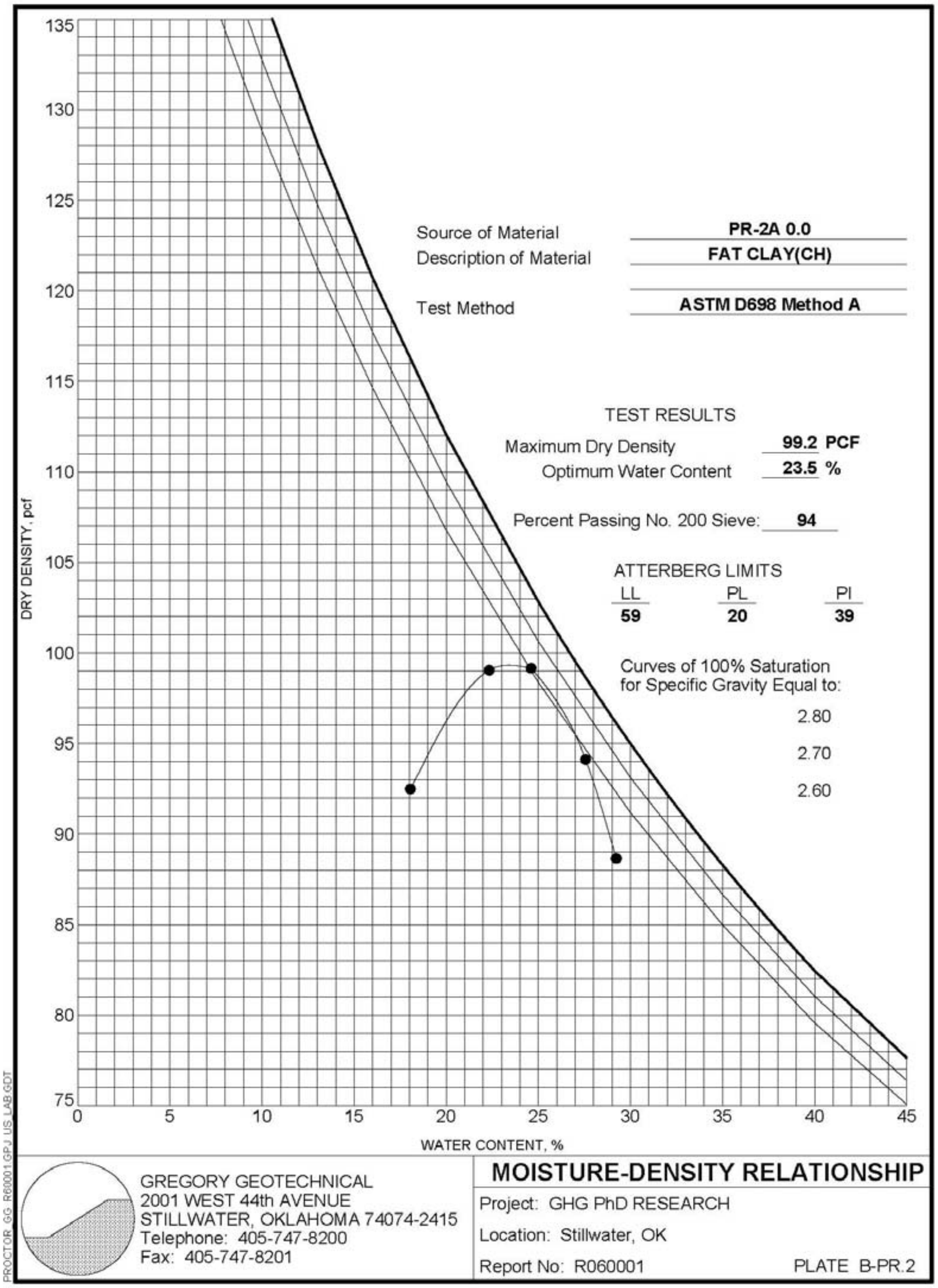
APPENDIX A

LABORATORY TEST REPORTS


DRY DENSITY, pcf
135
130
125
120
115
110
105
100
95
90
85
80
75
0
5
10
15
20
25
30
35
40
45
WATER CONTENT, %
Source of Material PR-1 0.0
Description of Material FATCLAY(CH), brown
Test Method ASTM D698 Method A
TEST RESULTS
Maximum Dry Density 100.0 PCF
Optimum Water Content 23.2 %
Percent Passing No. 200 Sieve: 94
ATTERBERG LIMITS
LL PL PI
59 20 39
Curves of 100% Saturation
for Specific Gravity Equal to:
2.80
2.70
2.60
GREGORY GEOTECHNICAL
2001 WEST 44th AVENUE
STILLWATER, OKLAHOMA 74074-2415
Telephone: 405-747-8200
Fax: 405-747-8201
MOISTURE-DENSITY RELATIONSHIP
Project: GHG PhD RESEARCH
Location: Stillwater, OK
Report No: R060001
PLATE B-PR.1


Source of Material
PR-2A 0.0
Description of Material
FAT CLAY(CH)
Test Method
ASTM D698 Method A
TEST RESULTS
Maximum Dry Density 99.2 PCF
Optimum Water Content 23.5 %
Percent Passing No. 200 Sieve: 94
ATTERBERG LIMITS
LL 59
PL 20
PI 39
Curves of 100% Saturation for Specific Gravity Equal to:
2.80
2.70
2.60
DRY DENSITY, pcf
135
130
125
120
115
110
105
100
95
90
85
80
75
0
5
10
15
20
25
30
35
40
45
WATER CONTENT, %
PROCTOR GG R60001.GPJ US LAB.GDT
GREGORY GEOTECHNICAL
2001 WEST 44th AVENUE
STILLWATER, OKLAHOMA 74074-2415
Telephone: 405-747-8200
Fax: 405-747-8201
MOISTURE-DENSITY RELATIONSHIP
Project: GHG PhD RESEARCH
Location: Stillwater, OK
Report No: R060001
PLATE B-PR.2

GREGORY GEOTECHNICAL

2001 West 44th Avenue
Stillwater, Oklahoma 74074-2415
Phone: 405-747-8200

MAXIMUM/MINIMUM INDEX DENSITY TEST

(ASTM D 4253/4254) or (ASTM D 4253-Manual Method)

Project No.: GHG6001 **Test No.:** MID-1

Project Name : GHG PhD Research

Client : GHG-OSU-Stillwater, OK **Client Reference No.:** NA

Sample No. : L60001-2 **Location :** Bulk Sample (Fill Sand)

Sample Description : SILTY SAND(SM), reddish tan

Diameter of Mold: 2.8670 **inches**

Mold No.: ____ **Volume of Mold :** 0.022328 ft^3

Moisture Content During Test (%):	0.00	0.00	0.00	0.00
Plate Thickness (in.):	0.00	0.00	0.00	0.00

Height Reading

	Test 1	Test 2	Test 3	Test 4
Reading # 1 (in)	0.000	0.000	0.000	0.000
Reading # 2 (in)	0.000	0.000	0.000	0.000
Reading # 3 (in)	0.000	0.000	0.000	0.000
Reading # 4 (in)	0.000	0.000	0.000	0.000
Average Ht.	0.000	0.000	0.000	0.000
Corrected mold Vol :	0.0223	0.0223	0.0223	0.0223

	Test			
	1	2	3	4
Weight (grams)				
Mold :	3531.65	3531.65	3531.65	3531.65
Mold + Material :	4405.27	4389.84	4394.38	4390.75
Weight of Material :	873.62	858.19	862.73	859.10
Wet Density (lb/cu. ft.):	NA	NA	NA	NA
Dry Density (lb/cu. ft.):	86.3	84.7	85.2	84.8

Density Standard Dev. : 0.70 (ASTM D 4253: 0.8-fine to med. sand, 1.4-gravelly sand; single operator)

Density Variation % of Mean: 1.8 (ASTM D 4253: <= 2.7%-fine to med. sand; 4.1%-gravelly sand; single operator)

Minimum Wet Density (Ave.) = **NA** **lb/ft^3**

Minimum Dry Density (Ave.) = **85.3** lb/ft^3

MID-60001-3.xls

PLATE MID.1B

GREGORY GEOTECHNICAL

2001 West 44th Avenue
Stillwater, Oklahoma 74074-2415
Phone: 405-747-8200

MAXIMUM/MINIMUM INDEX DENSITY TEST

(ASTM D 4253/4254) or (ASTM D 4253-Manual Method)

Project No.: GHG60001 **Test No.:** MID-1

Project Name : GHG PhD Research

Client : GHG-OSU - Stillwater, OK **Client Reference No.:** NA

Sample No. : L60001-2 **Location :** Bulk Sample (Fill Sand)

Sample Description : SILTY SAND (SM), reddish tan

Diameter of Mold: 2.8670 **inches**

Mold No.: MP-1 **Volume of Mold :** 0.022328 **ft^3**

Moisture Content During Test (%):	0.00	0.00	0.00	0.00
Plate Thickness (in.):	4.17	4.17	4.17	4.17

Height Reading

	Test 1	Test 2	Test 3	Test 4
Reading # 1 (in)	-3.299	-3.279	-3.274	-3.246
Reading # 2 (in)	-3.298	-3.277	-3.268	-3.258
Reading # 3 (in)	-3.300	-3.279	-3.273	-3.247
Reading # 4 (in)	-3.294	-3.272	-3.273	-3.255
Average Ht.	-3.298	-3.277	-3.272	-3.252
Corrected mold Vol :	0.0191	0.0190	0.0190	0.0189

	Test			
	1	2	3	4
Weight (grams)				
Mold :	3531.65	3531.65	3531.65	3531.65
Mold + Material :	4439.33	4423.40	4428.02	4424.25
Weight of Material :	907.68	891.75	896.37	892.60
Wet Density (lb/cu. ft.):	NA	NA	NA	NA
Dry Density (lb/cu. ft.):	105.0	103.6	104.2	104.2

Density Standard Dev. : 0.58 (ASTM D 4253: 0.8-fine to med. sand, 1.4-gravelly sand; single operator)

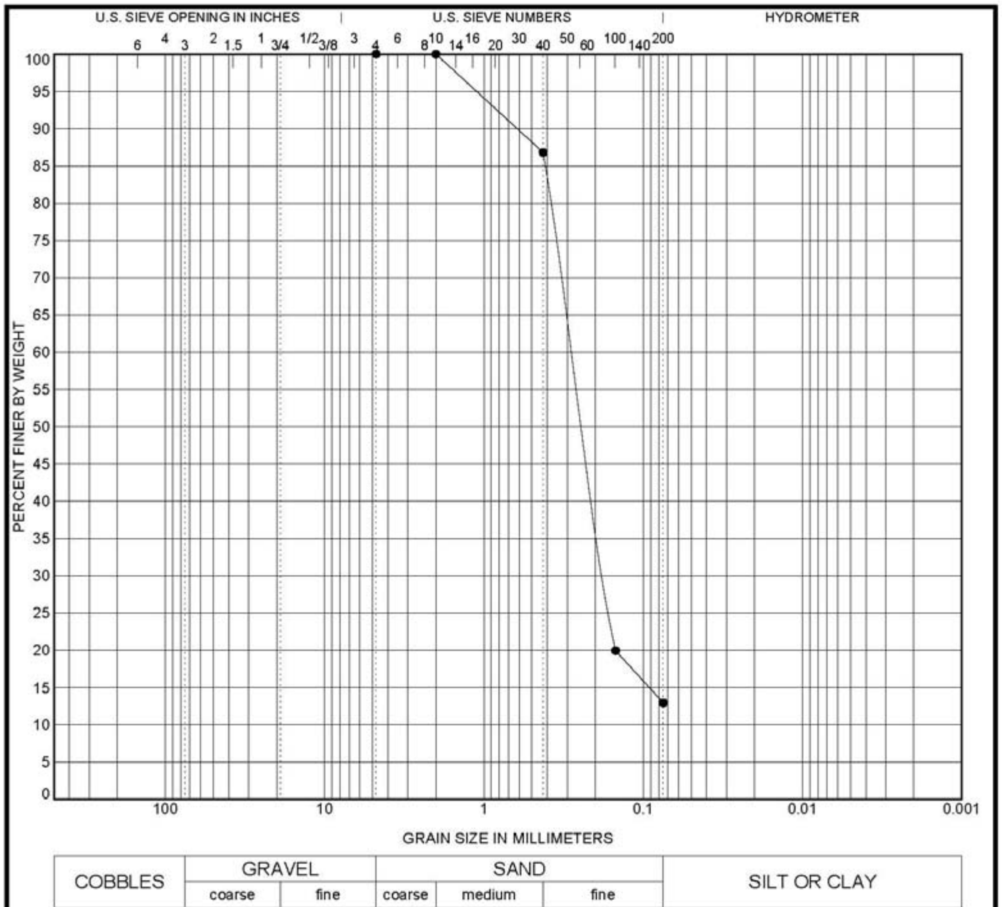
Density Variation % of Mean: 1.4 (ASTM D 4253:<= 2.7%-fine to med. sand; 4.1%-gravelly sand; single operator)

Manual-Maximum Wet Density (Ave.) = **NA** **lb/ft^3**

Manual-Maximum Dry Density (Ave.) = **104.2** lb/ft^3

MID-60001-4.xls

PLATE MID.1A



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● **SA-2**	**0.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

Specimen Identification		D100	D85	D50	D15	%Gravel	%Sand	%Silt	%Clay
● **SA-2**	**0.0**	**4.75**	**0.413**	**0.239**	**0.092**	**0.0**	**87.1**	**12.9**	

GG_SIEVE_SIZE R60001.GPJ US_LAB.GDT

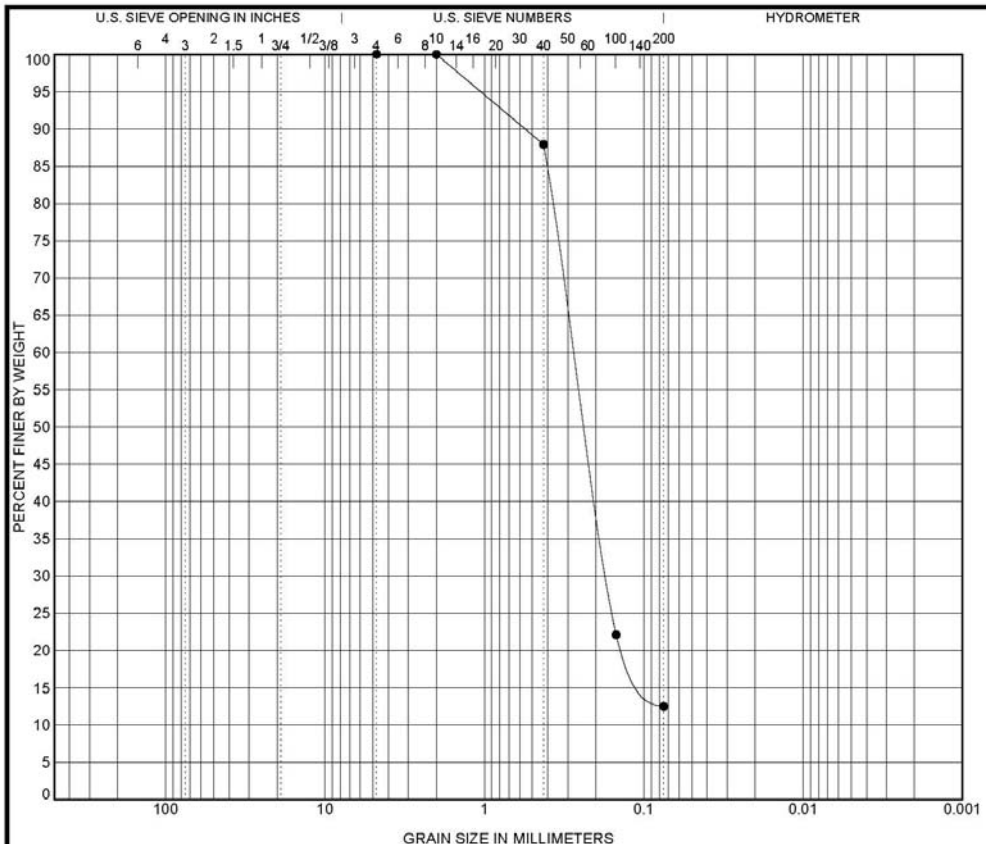
GREGORY GEOTECHNICAL
2001 WEST 44th AVENUE
STILLWATER, OKLAHOMA 74074-2415
Telephone: 405-747-8200
Fax: 405-747-8201

GRAIN SIZE DISTRIBUTION

Project: GHG PhD RESEARCH

Location: Stillwater, OK

Report No: R060001 PLATE B-GS.1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**SA-2a**	**0.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**	**1.67**	**4.34**

	Specimen Identification		D100	D85	D50	D15	%Gravel	%Sand	%Silt	%Clay
●	**SA-2a**	**0.0**	**4.75**	**0.406**	**0.232**	**0.09**	**0.0**	**87.5**	**12.5**	

GREGORY GEOTECHNICAL
2001 WEST 44th AVENUE
STILLWATER, OKLAHOMA 74074-2415
Telephone: 405-747-8200
Fax: 405-747-8201

GRAIN SIZE DISTRIBUTION

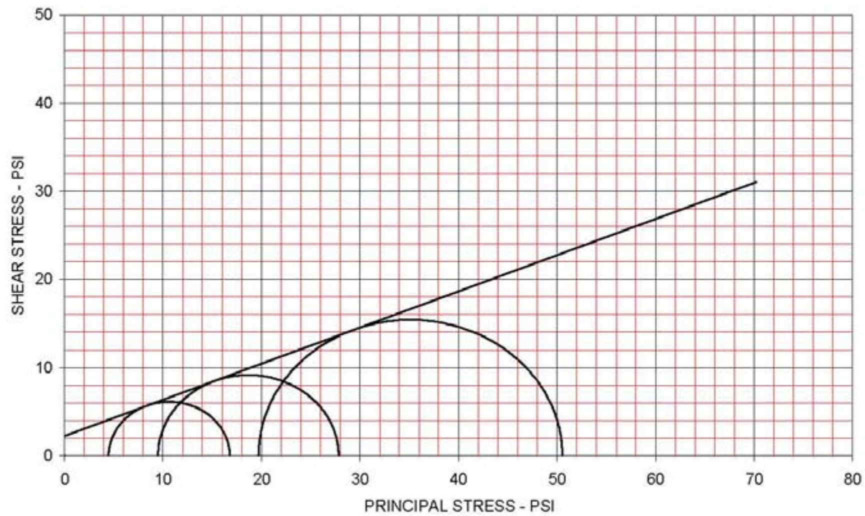
Project: GHG PhD RESEARCH

Location: Stillwater, OK

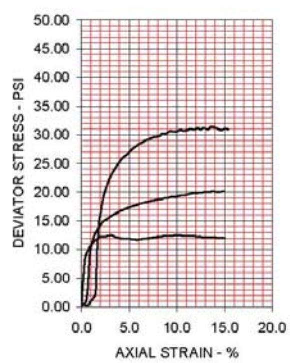
Report No: R060001 PLATE B-GS.2

GG_SIEVE_SIZE R60001.GPJ US_LAB.GDT

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ' =	22.3 deg	c' =	2.3 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.9	26.2	26.2	
Dry Density - pcf	95.5	96.6	102.3	
Calculated Diameter (in.)	2.87	2.87	2.86	
Height - inches	5.79	5.78	5.77	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.28	18.34	30.85	
Total Pore Pressure - psi	65.5	70.5	80.3	
Strain Rate - inches/min.	0.00056	0.00050	0.00050	
Failure Strain - %	2.4	7.0	10.4	
σ_1' Failure - psi	16.78	27.84	50.55	
σ_3' Failure - psi	4.50	9.50	19.70	

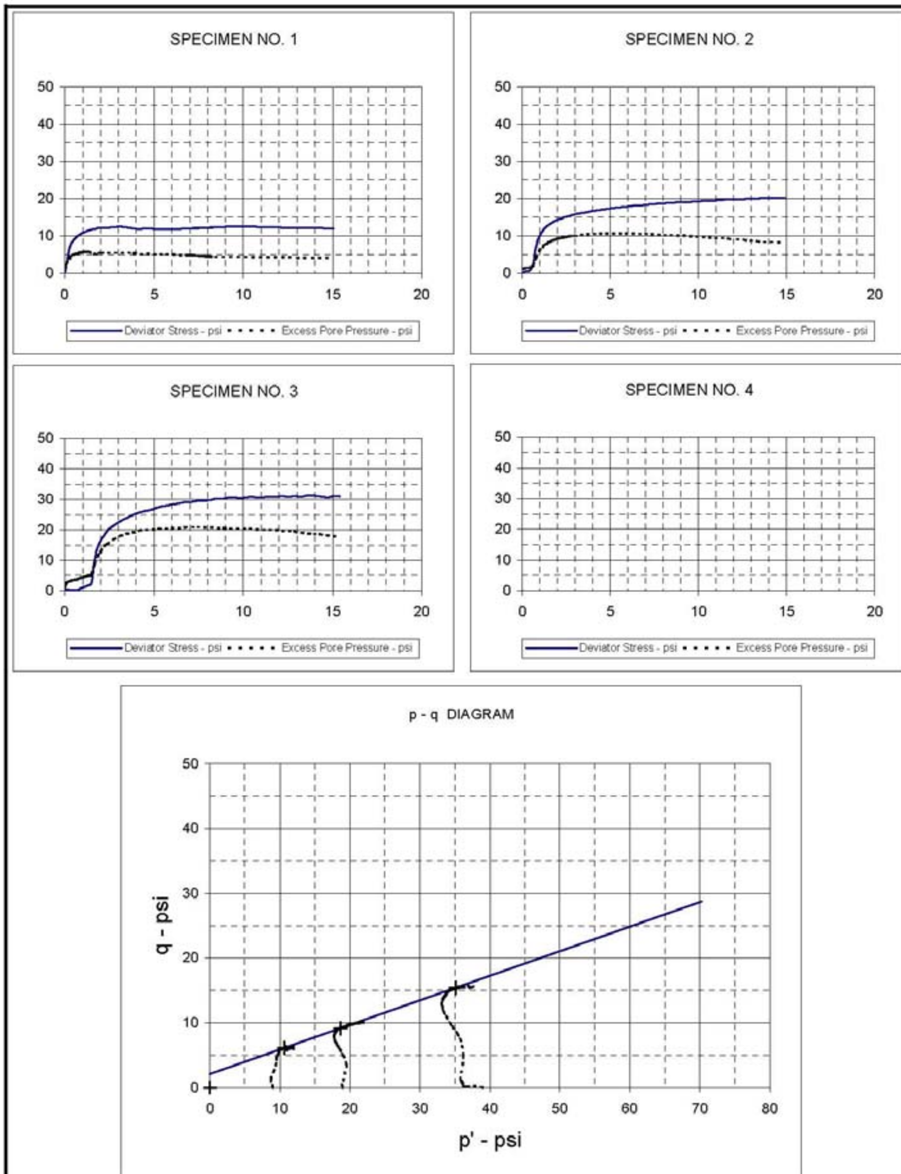
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-1
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers Cell No. 1-1-1

PROJECT INFORMATION

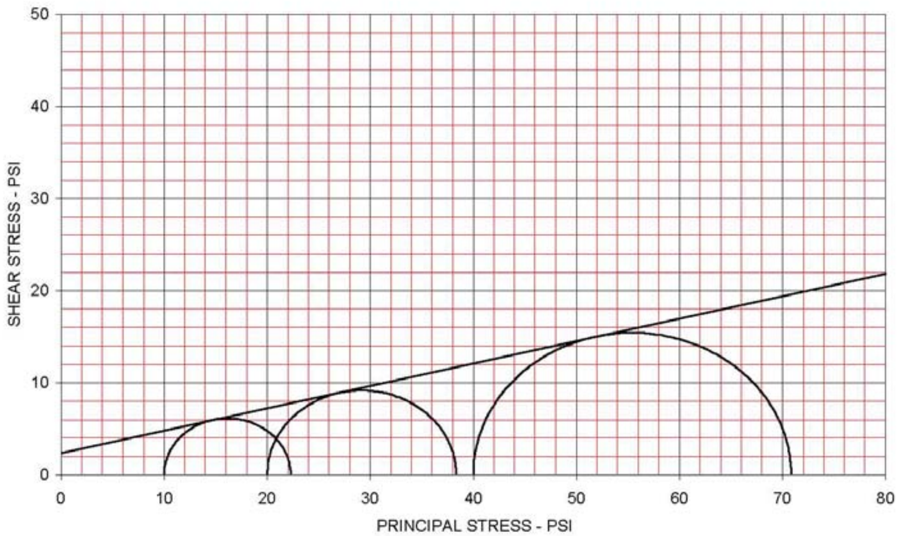
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.1A

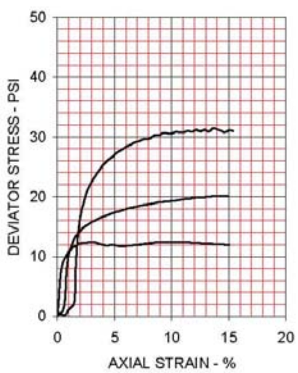


EFFECTIVE STRESS PARAMETERS	R^2 = 1.000	α (deg) = 20.8	a (psi) = 2.1
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-1	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.1B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 13.7 deg	c = 2.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.9	26.2	26.2	
Dry Density - pcf	95.5	96.6	102.3	
Calculated Diameter (in.)	2.87	2.87	2.86	
Height - inches	5.79	5.78	5.77	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.28	18.34	30.85	
Total Pore Pressure - psi	65.5	70.5	80.3	
Strain Rate - inches/min.	0.00056	0.00050	0.00050	
Failure Strain - %	2.4	7.0	10.4	
σ_1 Failure - psi	22.28	38.34	70.85	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

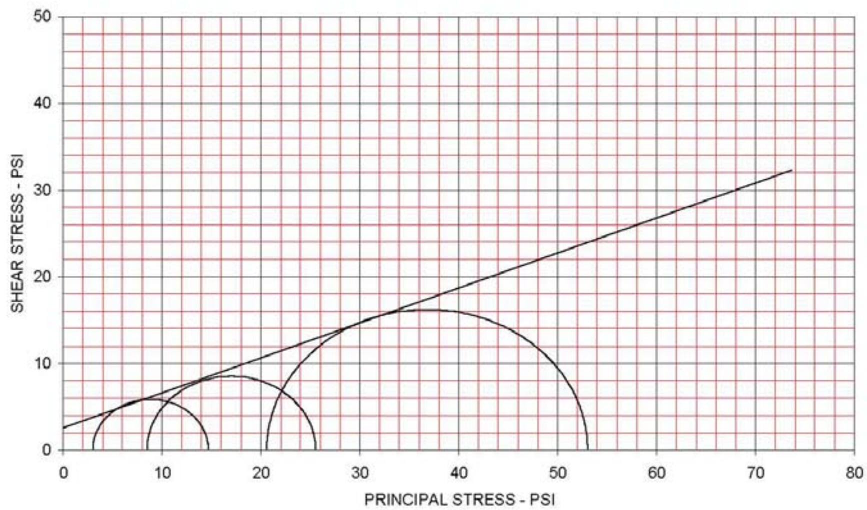
TYPE OF TEST & NO: CU' Triaxial - TX-1-1
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers Cell No. 1-1-1

PROJECT INFORMATION

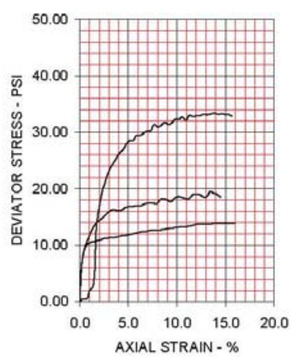
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.1C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	22.0 deg	c' =	2.6 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.7	28.6	25.4	
Dry Density - pcf	93.8	97.2	101.2	
Calculated Diameter (in.)	2.91	2.87	2.81	
Height - inches	5.90	5.78	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	11.69	17.04	32.44	
Total Pore Pressure - psi	67.0	71.5	79.4	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	4.3	5.8	9.8	
σ_1' Failure - psi	14.69	25.54	53.04	
σ_3' Failure - psi	3.00	8.50	20.60	

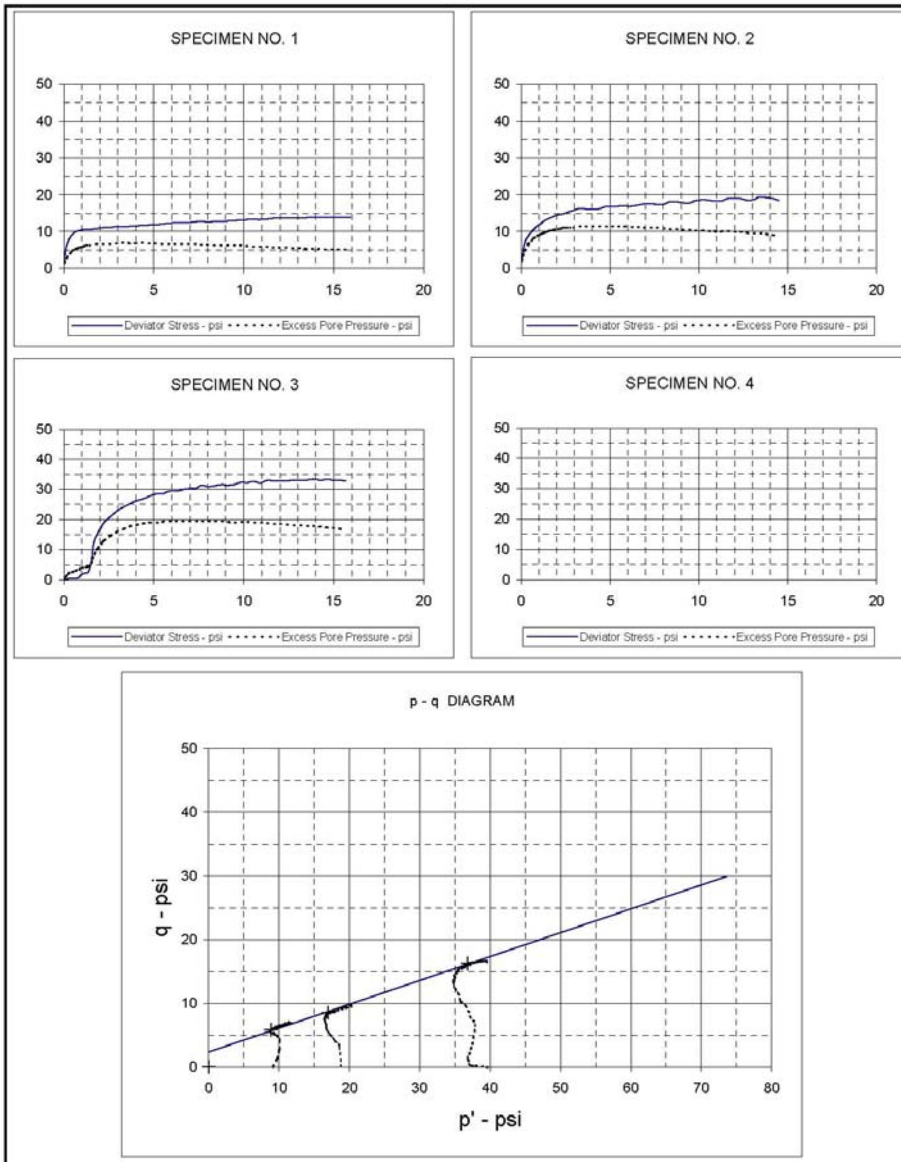
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers Cell No. 2-1-1

PROJECT INFORMATION

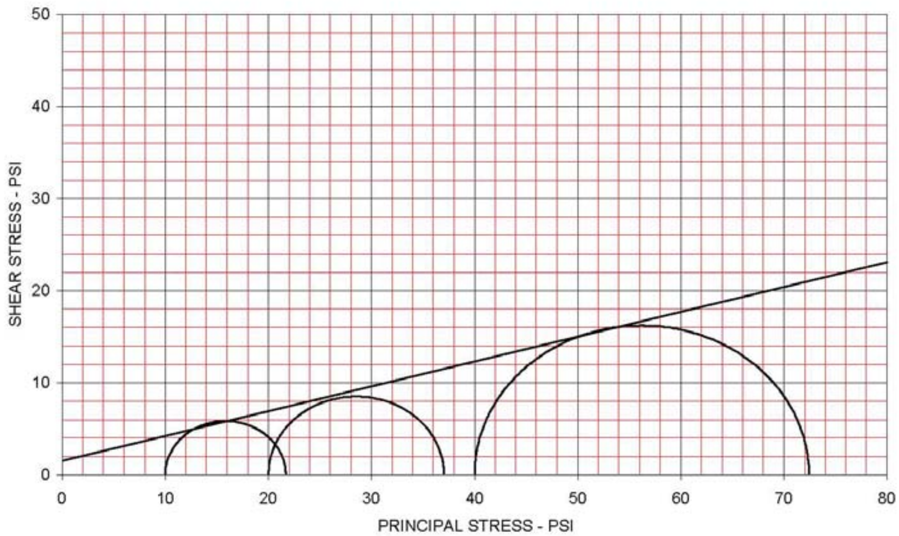
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.2A

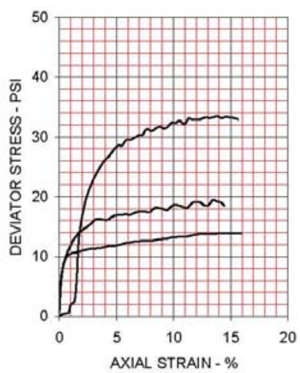


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 20.5	a (psi) = 2.4
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-2	
PROJECT NO: GHG60001		**GREGORY GEOTECHNICAL**	PLATE: TX.2B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	φ = 15.1 deg	c = 1.5 psi		
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.7	28.6	25.4	
Dry Density - pcf	93.8	97.2	101.2	
Calculated Diameter (in.)	2.91	2.87	2.81	
Height - inches	5.90	5.78	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	11.69	17.04	32.44	
Total Pore Pressure - psi	67.0	71.5	79.4	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	4.3	5.8	9.8	
σ_1 Failure - psi	21.69	37.04	72.44	
σ_3 Failure - psi	10.00	20.00	40.00	



TEST DESCRIPTION

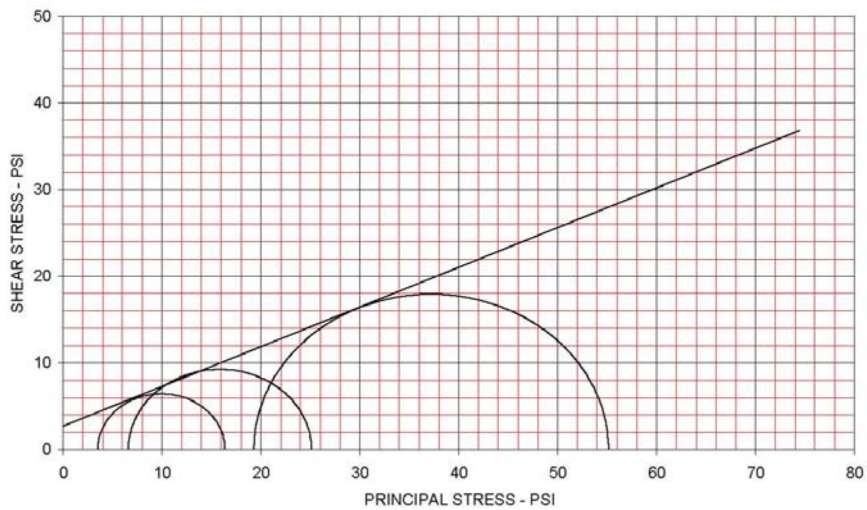
TYPE OF TEST & NO: CU' Triaxial - TX-1-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers Cell No. 2-1-1

PROJECT INFORMATION

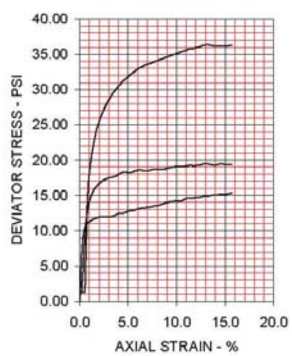
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL | PLATE: TX.2C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ' =	24.6 deg	c' =	2.7 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	30.0	28.6	26.4	
Dry Density - pcf	94.8	97.7	102.6	
Calculated Diameter (in.)	2.86	2.85	2.83	
Height - inches	5.76	5.73	5.67	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.88	18.53	35.84	
Total Pore Pressure - psi	66.5	73.4	80.7	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	6.6	11.7	
σ_1' Failure - psi	16.38	25.13	55.14	
σ_3' Failure - psi	3.50	6.60	19.30	

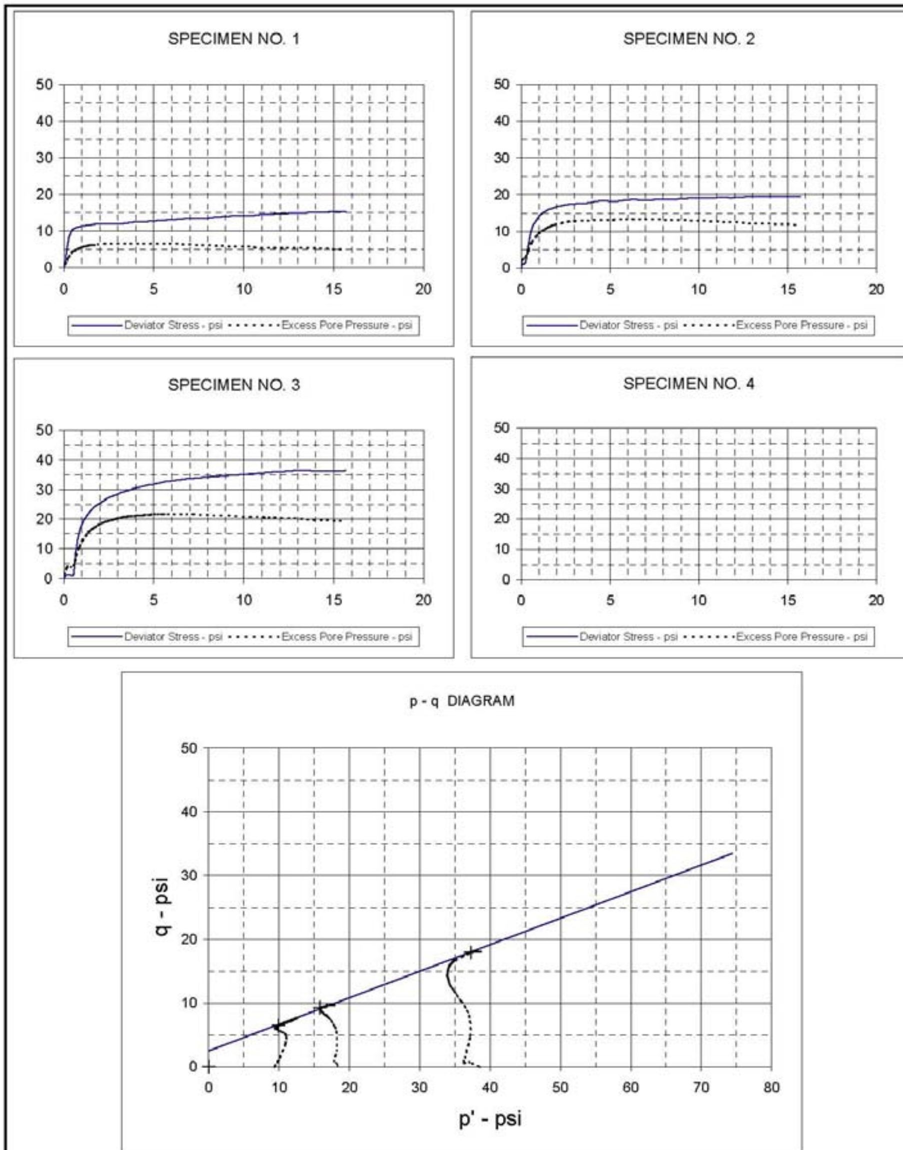
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-3
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf Cell No. 2-1-2

PROJECT INFORMATION

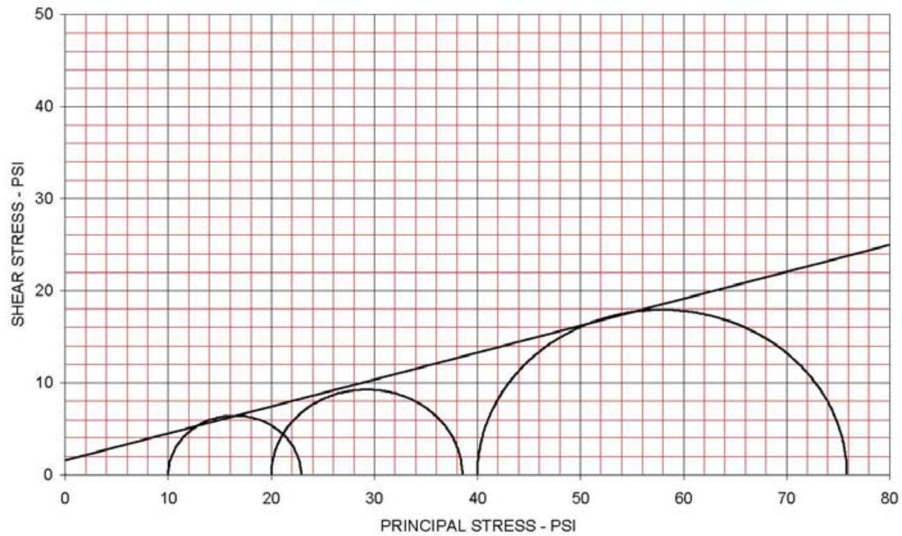
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.3A

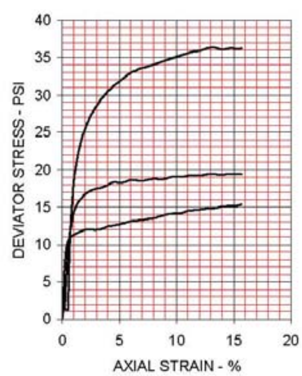


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 22.6	a (psi) = 2.5
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-3	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.3B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 16.3 deg	c = 1.6 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	30.0	28.6	26.4	
Dry Density - pcf	94.8	97.7	102.6	
Calculated Diameter (in.)	2.86	2.85	2.83	
Height - inches	5.76	5.73	5.67	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.88	18.53	35.84	
Total Pore Pressure - psi	66.5	73.4	80.7	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	6.6	11.7	
σ_1 Failure - psi	22.88	38.53	75.84	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

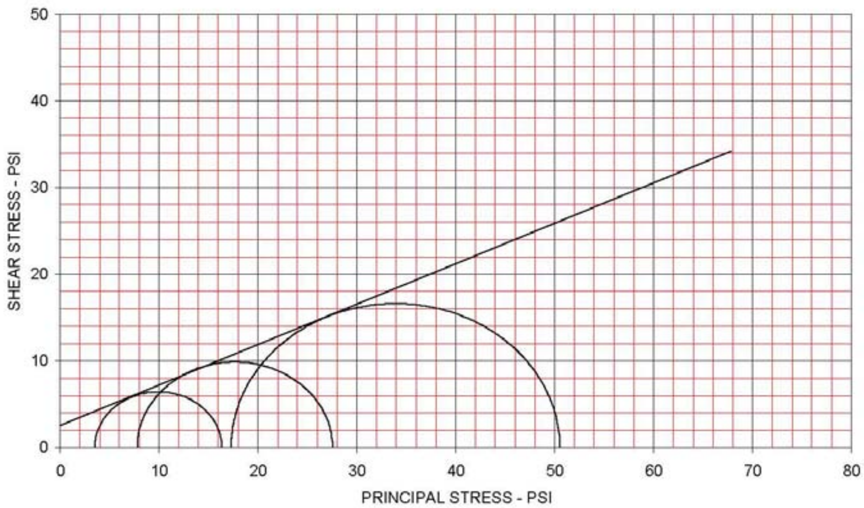
TYPE OF TEST & NO: CU' Triaxial - TX-1-3
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf Cell No. 2-1-2

PROJECT INFORMATION

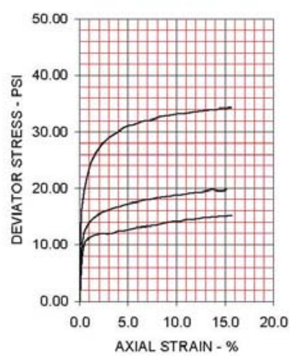
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.3C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' =	25.0 deg	c' =	2.5 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.8	28.9	27.1	
Dry Density - pcf	93.6	96.4	99.0	
Calculated Diameter (in.)	2.91	2.87	2.87	
Height - inches	5.90	5.79	5.78	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.86	19.75	33.23	
Total Pore Pressure - psi	66.5	72.2	82.7	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	15.1	10.5	
σ_1' Failure - psi	16.36	27.55	50.53	
σ_3' Failure - psi	3.50	7.80	17.30	

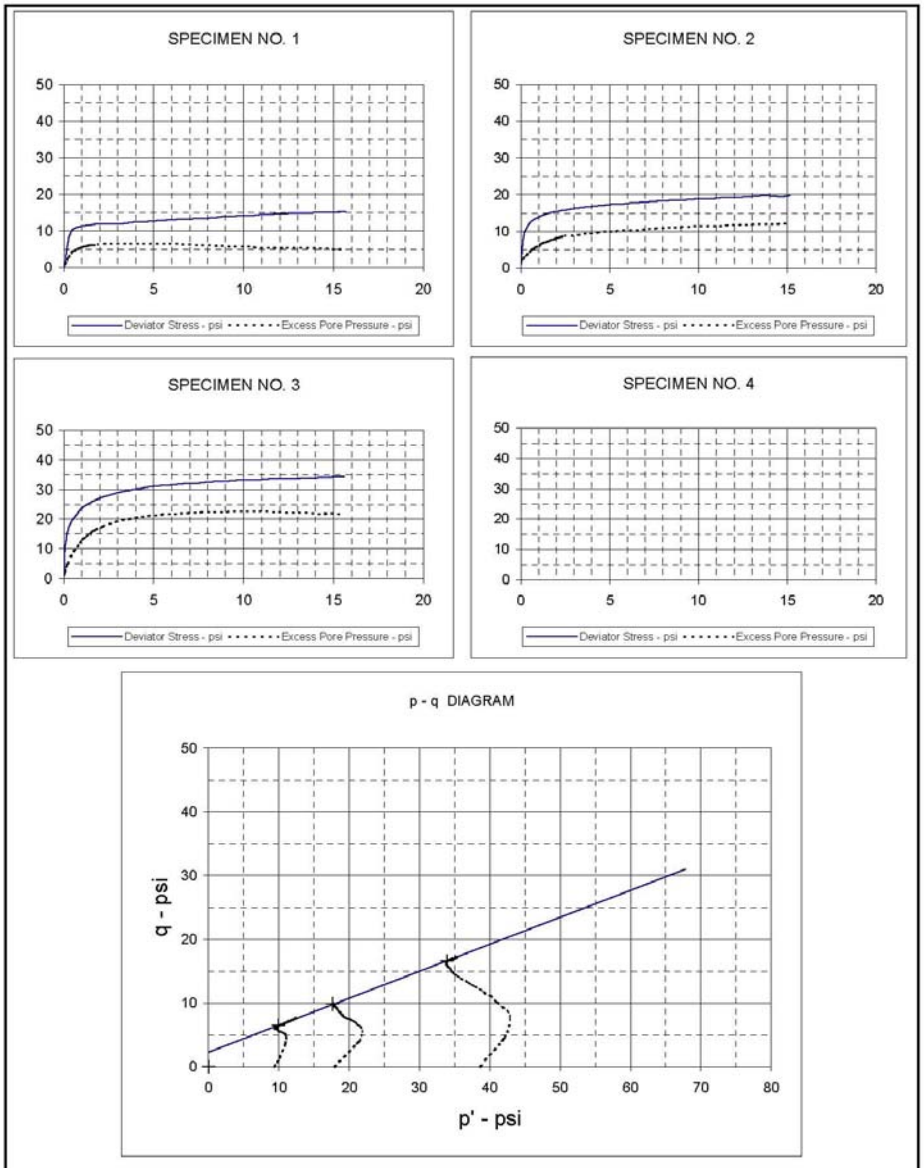
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-4
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf Cell No. 2-2-1

PROJECT INFORMATION

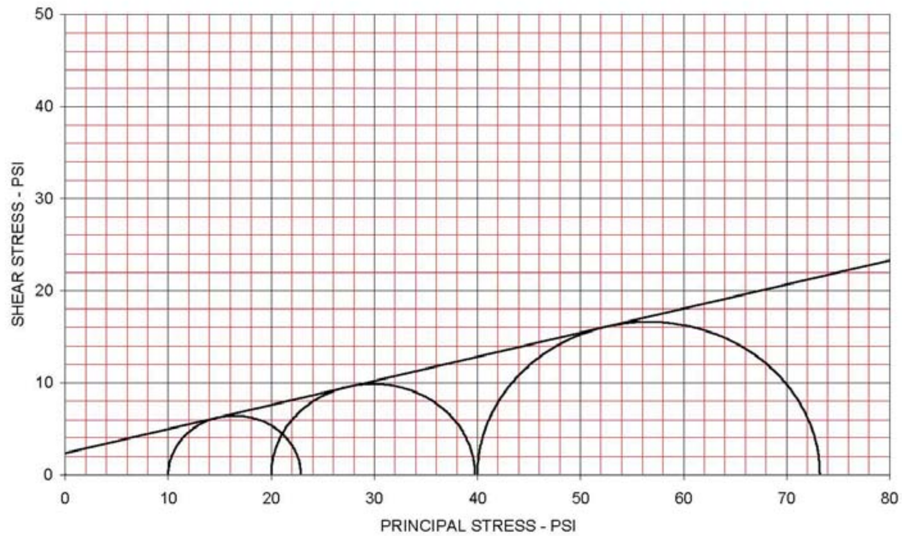
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.4A

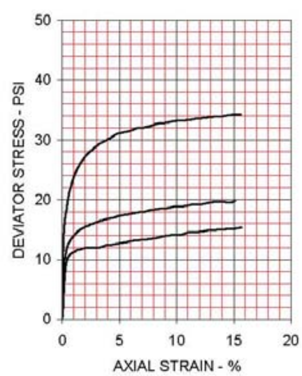


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 22.9	a (psi) = 2.3
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-4	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.4B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ =	14.7 deg	c =	2.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.8	28.9	27.1	
Dry Density - pcf	93.6	96.4	99.0	
Calculated Diameter (in.)	2.91	2.87	2.87	
Height - inches	5.90	5.79	5.78	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.86	19.75	33.23	
Total Pore Pressure - psi	66.5	72.2	82.7	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	15.1	10.5	
σ_1 Failure - psi	22.86	39.75	73.23	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

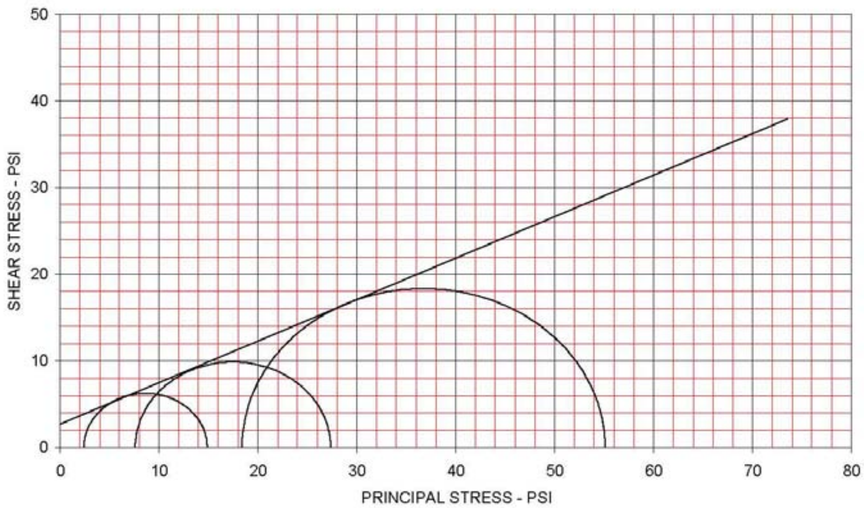
TYPE OF TEST & NO: CU' Triaxial - TX-1-4
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf Cell No. 2-2-1

PROJECT INFORMATION

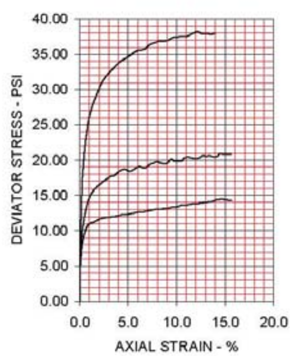
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.4C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	25.6 deg	c' =	2.7 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.8	28.9	31.1	
Dry Density - pcf	95.7	96.7	96.9	
Calculated Diameter (in.)	2.87	2.86	2.86	
Height - inches	5.79	5.77	5.76	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.48	19.77	36.70	
Total Pore Pressure - psi	67.6	72.4	81.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	8.0	7.8	
σ_1' Failure - psi	14.88	27.37	55.10	
σ_3' Failure - psi	2.40	7.60	18.40	

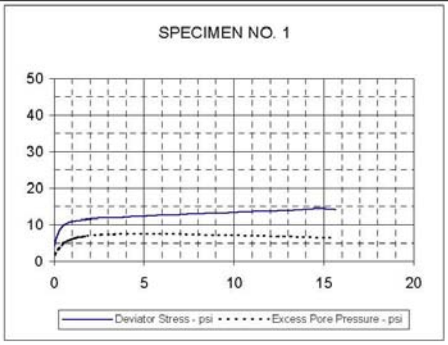
TEST DESCRIPTION

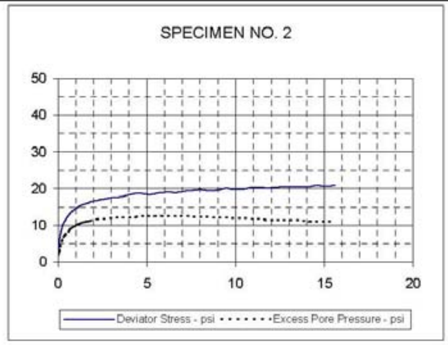
TYPE OF TEST & NO: CU' Triaxial - TX-1-5
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf Cell No. 2-1-2

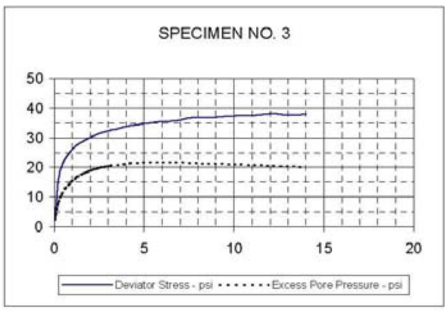
PROJECT INFORMATION

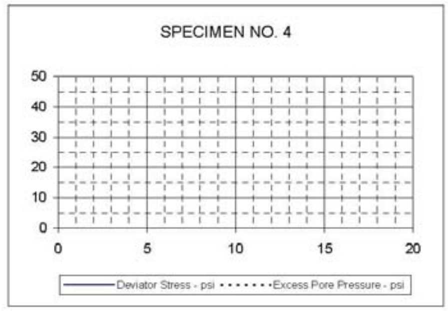
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

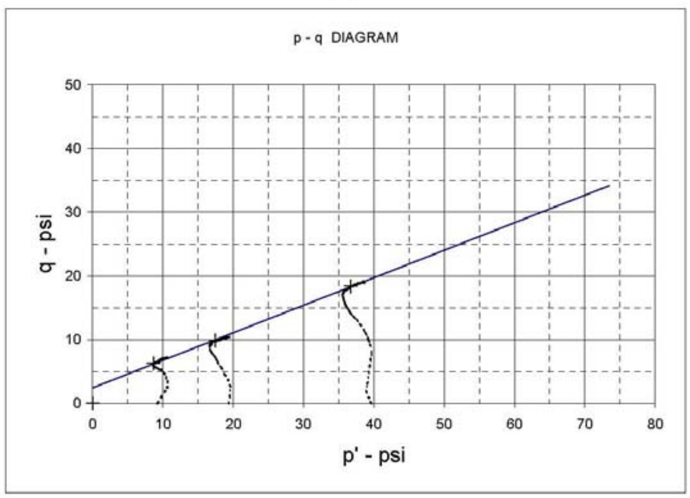
GREGORY GEOTECHNICAL PLATE: TX.5A





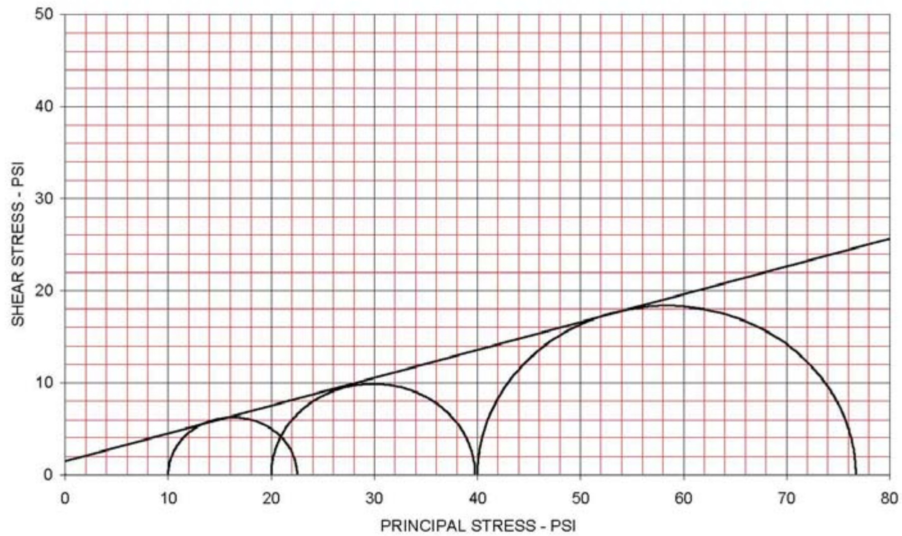




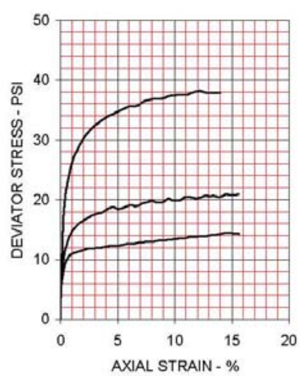


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 23.4	a (psi) = 2.4
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-5	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.5B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ =	16.8 deg	c =	1.5 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	29.8	28.9	31.1	
Dry Density - pcf	95.7	96.7	96.9	
Calculated Diameter (in.)	2.87	2.86	2.86	
Height - inches	5.79	5.77	5.76	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	12.48	19.77	36.70	
Total Pore Pressure - psi	67.6	72.4	81.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.5	8.0	7.8	
σ_1 Failure - psi	22.48	39.77	76.70	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

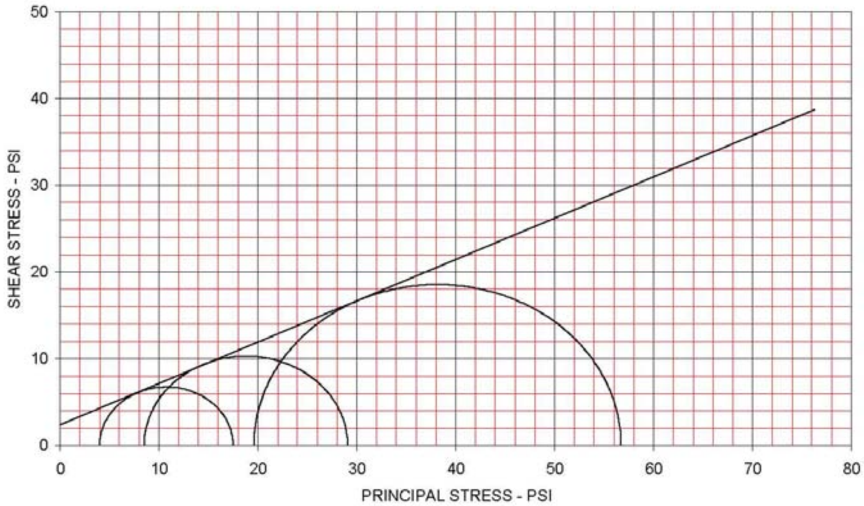
TYPE OF TEST & NO: CU' Triaxial - TX-1-5
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf Cell No. 2-1-2

PROJECT INFORMATION

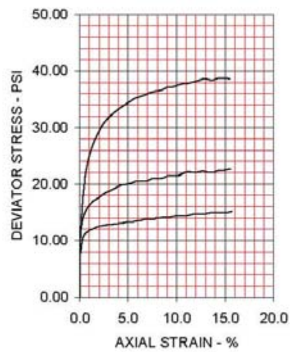
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.5C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' =	25.5 deg	c' =	2.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	31.1	28.5	31.1	
Dry Density - pcf	95.6	99.0	99.2	
Calculated Diameter (in.)	2.87	2.87	2.86	
Height - inches	5.79	5.78	5.75	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	13.50	20.58	37.11	
Total Pore Pressure - psi	66.0	71.5	80.4	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.9	6.1	9.0	
σ_1' Failure - psi	17.50	29.08	56.71	
σ_3' Failure - psi	4.00	8.50	19.60	

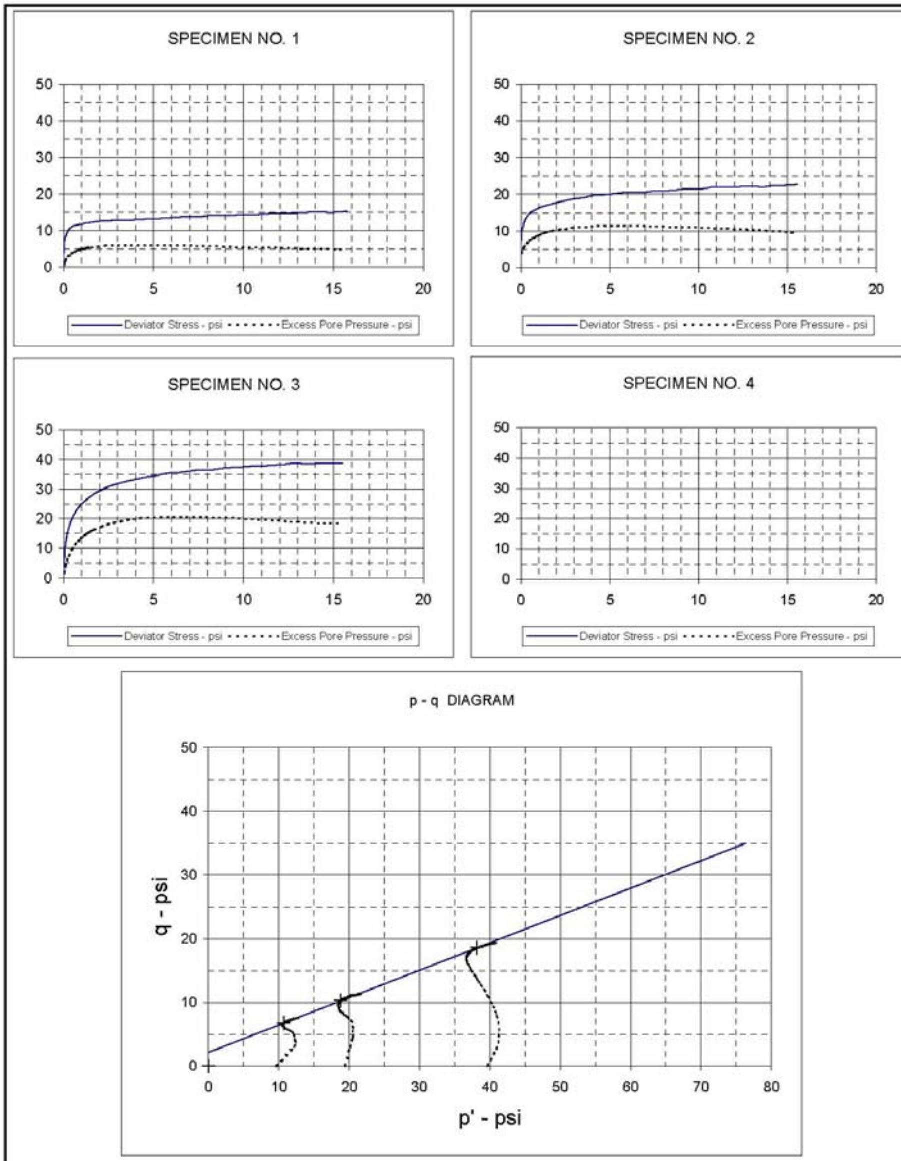
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-6
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf Cell No. 2-1-2

PROJECT INFORMATION

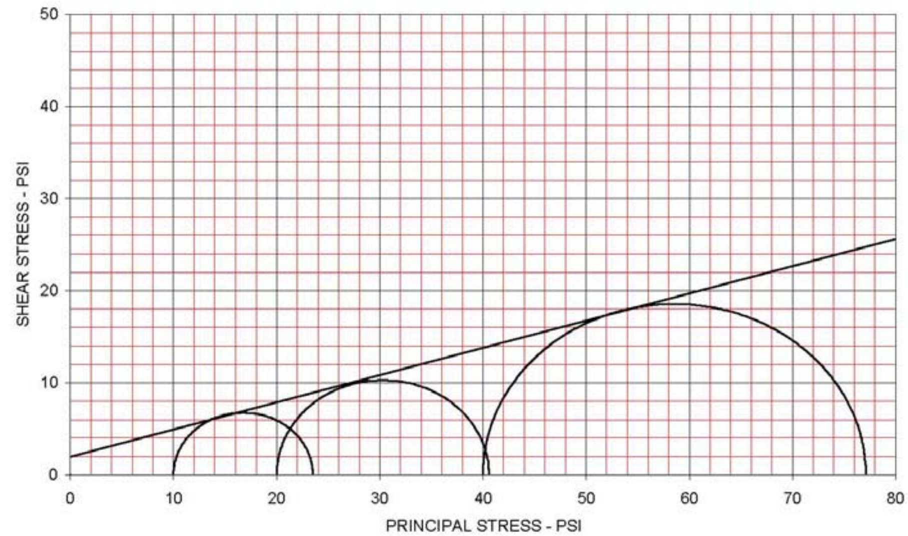
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.6A

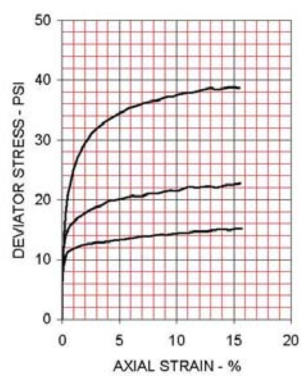


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 23.3	a (psi) = 2.2
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-6	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.6B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 16.5 deg	c = 2.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.88	2.88	2.88	
Height - inches	5.80	5.80	5.80	
AT TEST				
Final Moisture - %	31.1	28.5	31.1	
Dry Density - pcf	95.6	99.0	99.2	
Calculated Diameter (in.)	2.87	2.87	2.86	
Height - inches	5.79	5.78	5.75	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	13.50	20.58	37.11	
Total Pore Pressure - psi	66.0	71.5	80.4	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	5.9	6.1	9.0	
σ_1 Failure - psi	23.50	40.58	77.11	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

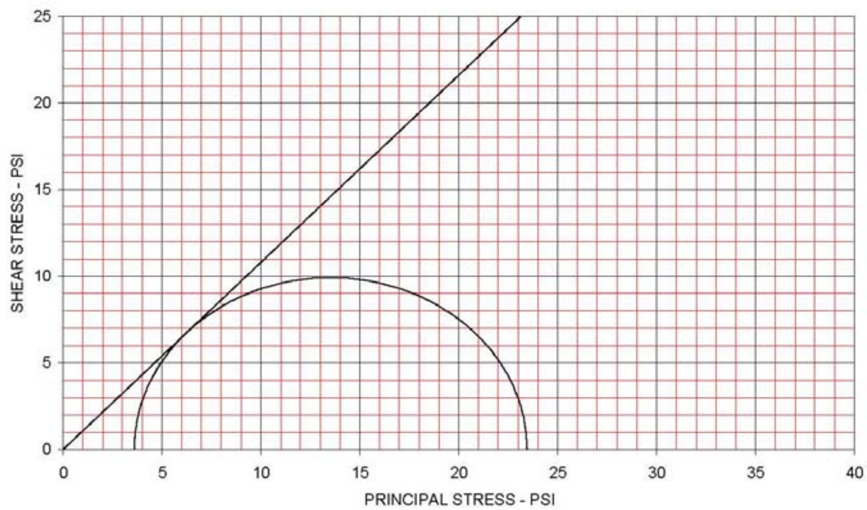
TYPE OF TEST & NO: CU' Triaxial - TX-1-6
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf Cell No. 2-1-2

PROJECT INFORMATION

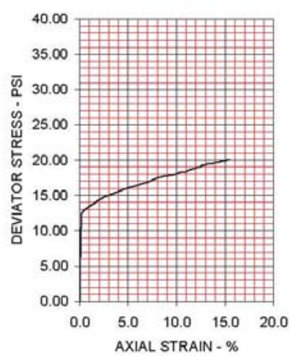
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.6C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	47.2 deg	c' =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	29.5			
Dry Density - pcf	95.2			
Calculated Diameter (in.)	2.87			
Height - inches	5.78			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	19.85			
Total Pore Pressure - psi	66.4			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	14.7			
σ_1' Failure - psi	23.45			
σ_3' Failure - psi	3.60			

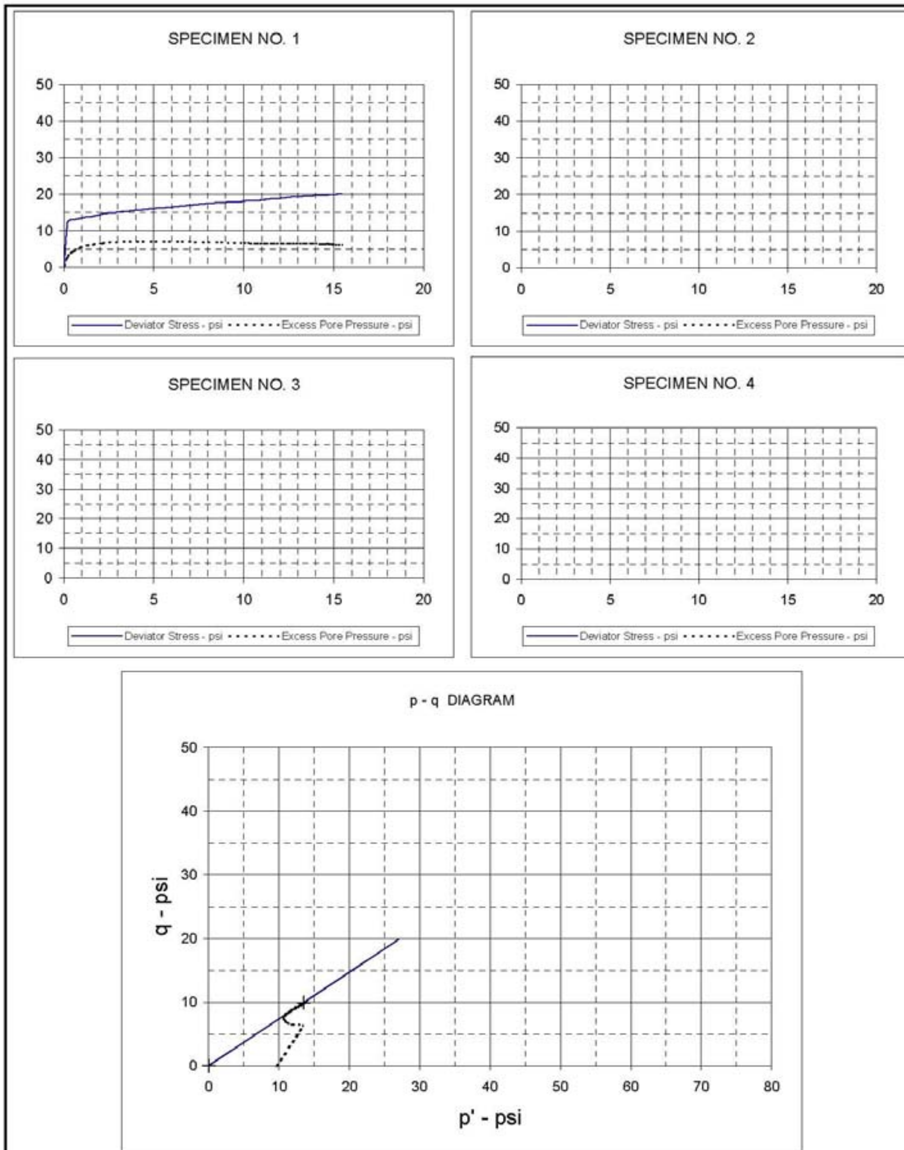
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-C-1
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 1.0 pcf Cell No. 2

PROJECT INFORMATION

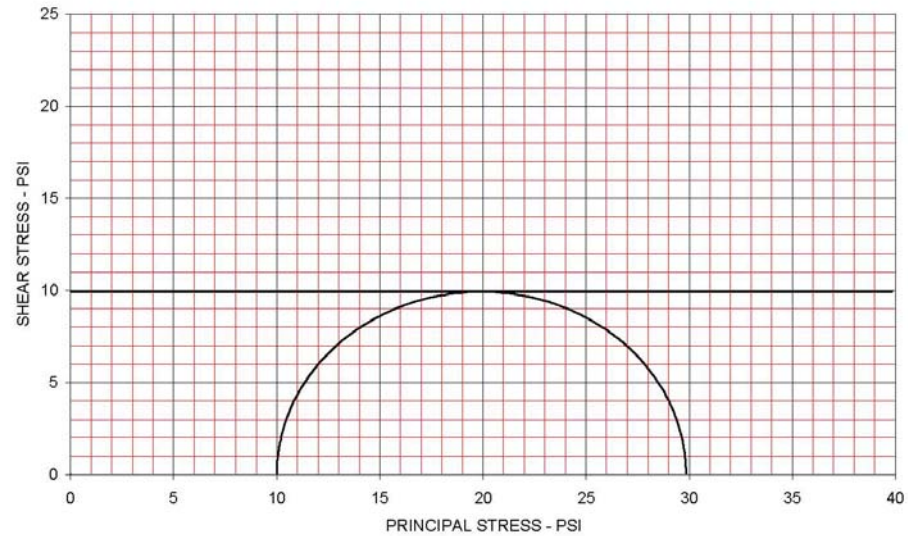
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/23/05

GREGORY GEOTECHNICAL | PLATE: TX.C1A

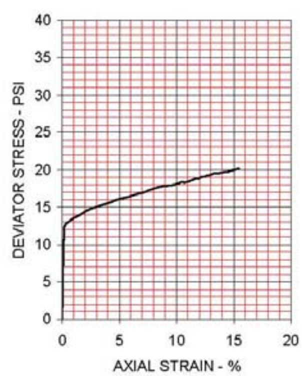


EFFECTIVE STRESS PARAMETERS	R^2 =	α (deg) = 36.3	a (psi) = 0.0
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-C-1	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.C1B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ =	0.0 deg	c =	9.9 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	29.5			
Dry Density - pcf	95.2			
Calculated Diameter (in.)	2.87			
Height - inches	5.78			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	19.85			
Total Pore Pressure - psi	66.4			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	14.7			
σ_1 Failure - psi	29.85			
σ_3 Failure - psi	10.00			

TEST DESCRIPTION

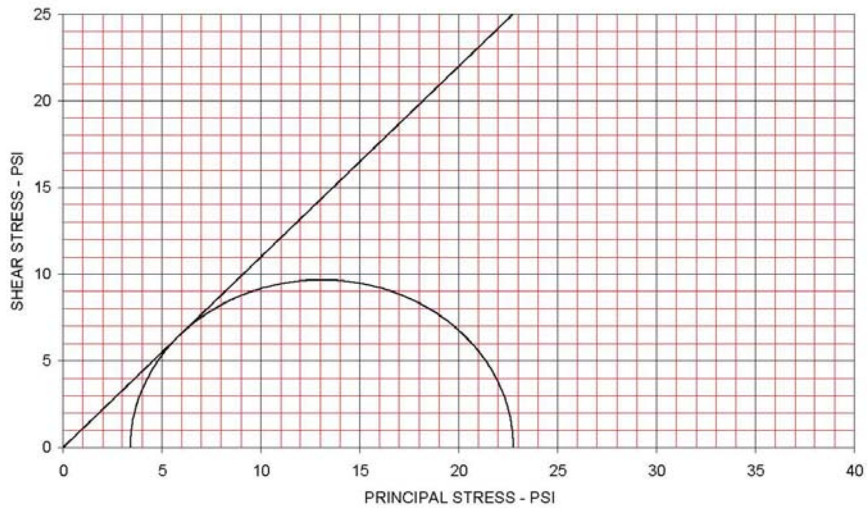
TYPE OF TEST & NO: CU' Triaxial - TX-1-C-1
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 1.0 pcf Cell No. 2

PROJECT INFORMATION

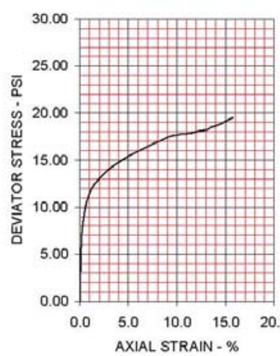
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/23/05

GREGORY GEOTECHNICAL PLATE: TX.C1C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	47.7 deg	c' =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	27.2			
Dry Density - pcf	95.6			
Calculated Diameter (in.)	2.87			
Height - inches	5.79			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	19.36			
Total Pore Pressure - psi	66.6			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	15.4			
σ_1' Failure - psi	22.76			
σ_3' Failure - psi	3.40			

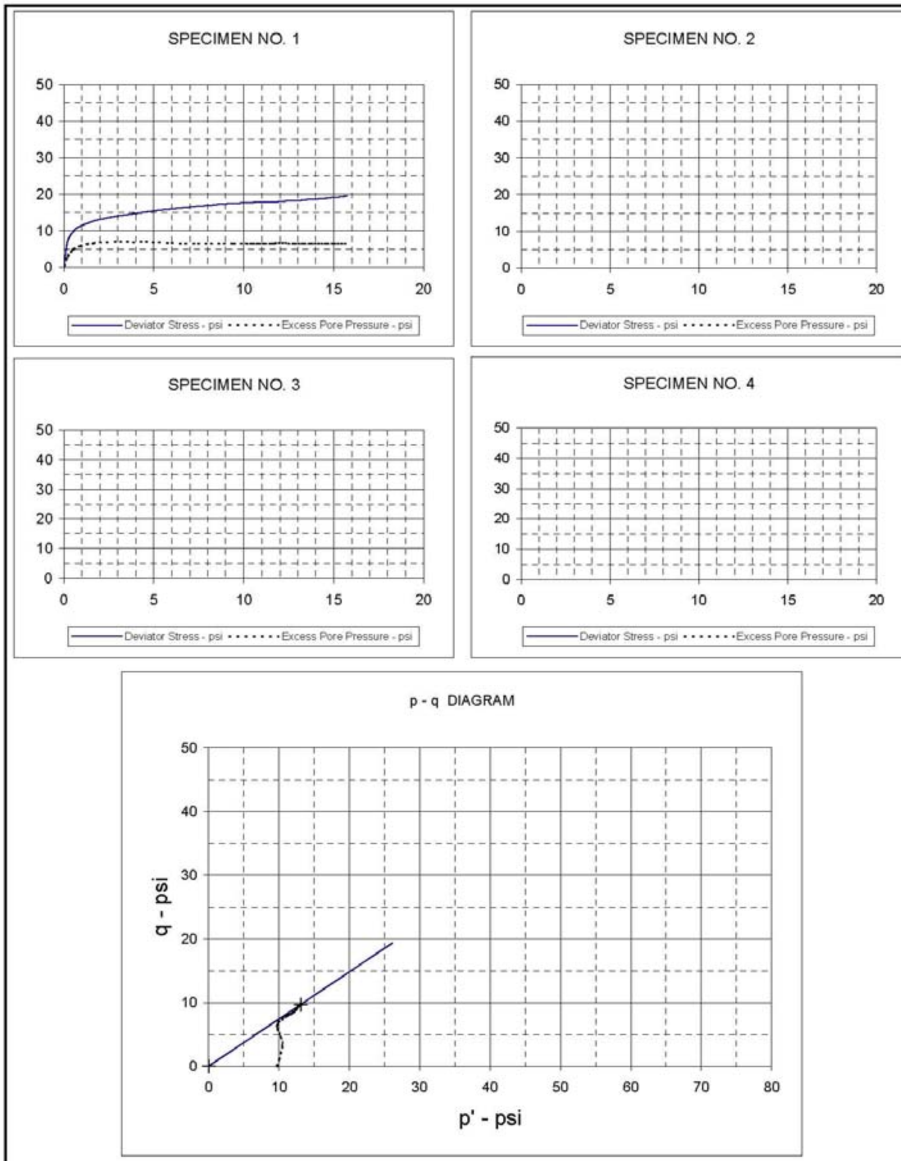
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-C-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 1.5 pcf Cell No. 1-1-2

PROJECT INFORMATION

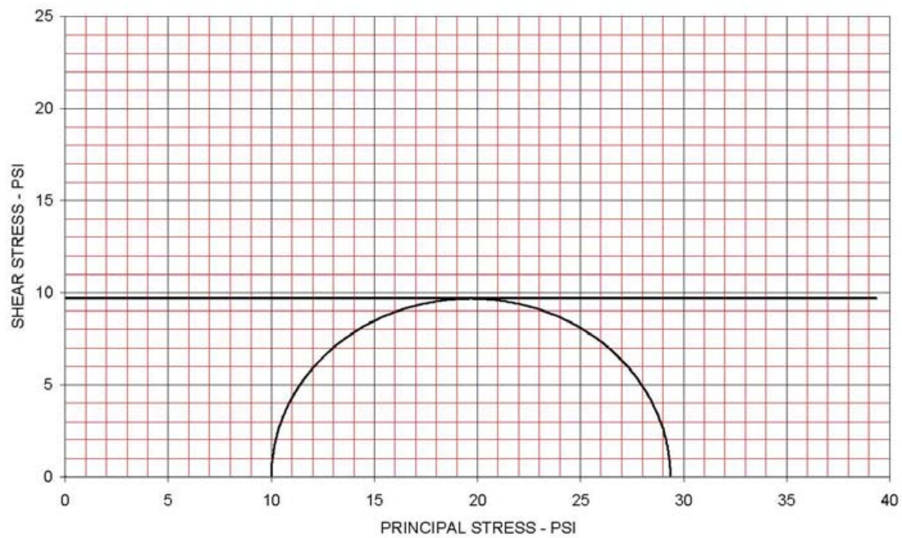
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/24/05

GREGORY GEOTECHNICAL PLATE: TX.C2.A

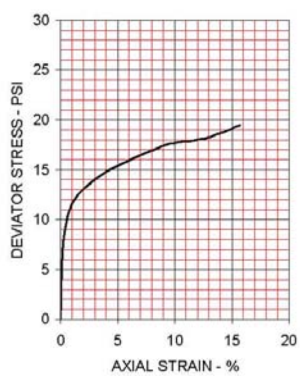


EFFECTIVE STRESS PARAMETERS	R^2 =	α (deg) = 36.5	a (psi) = 0.0
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-C-2	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.C2B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ =	0.0 deg	c =	9.7 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	27.2			
Dry Density - pcf	95.6			
Calculated Diameter (in.)	2.87			
Height - inches	5.79			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	19.36			
Total Pore Pressure - psi	66.6			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	15.4			
σ_1 Failure - psi	29.36			
σ_3 Failure - psi	10.00			

TEST DESCRIPTION

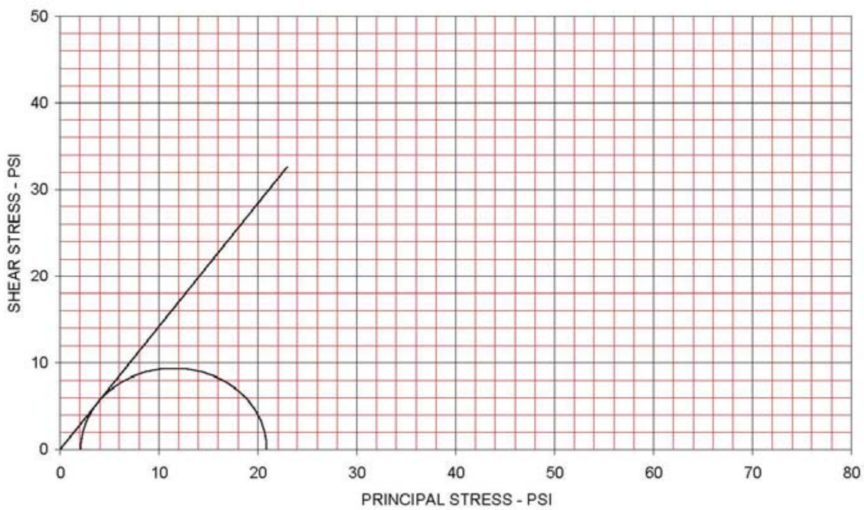
TYPE OF TEST & NO: CU' Triaxial - TX-1-C-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 1.5 pcf Cell No. 1-1-2

PROJECT INFORMATION

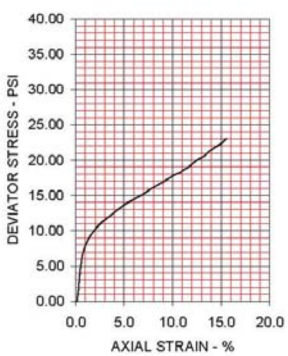
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/24/05

GREGORY GEOTECHNICAL PLATE: TX.C2C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' =	54.8 deg	c' =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	27.8			
Dry Density - pcf	96.8			
Calculated Diameter (in.)	2.86			
Height - inches	5.75			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	18.80			
Total Pore Pressure - psi	68.2			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	15.4			
σ_1' Failure - psi	20.90			
σ_3' Failure - psi	2.10			

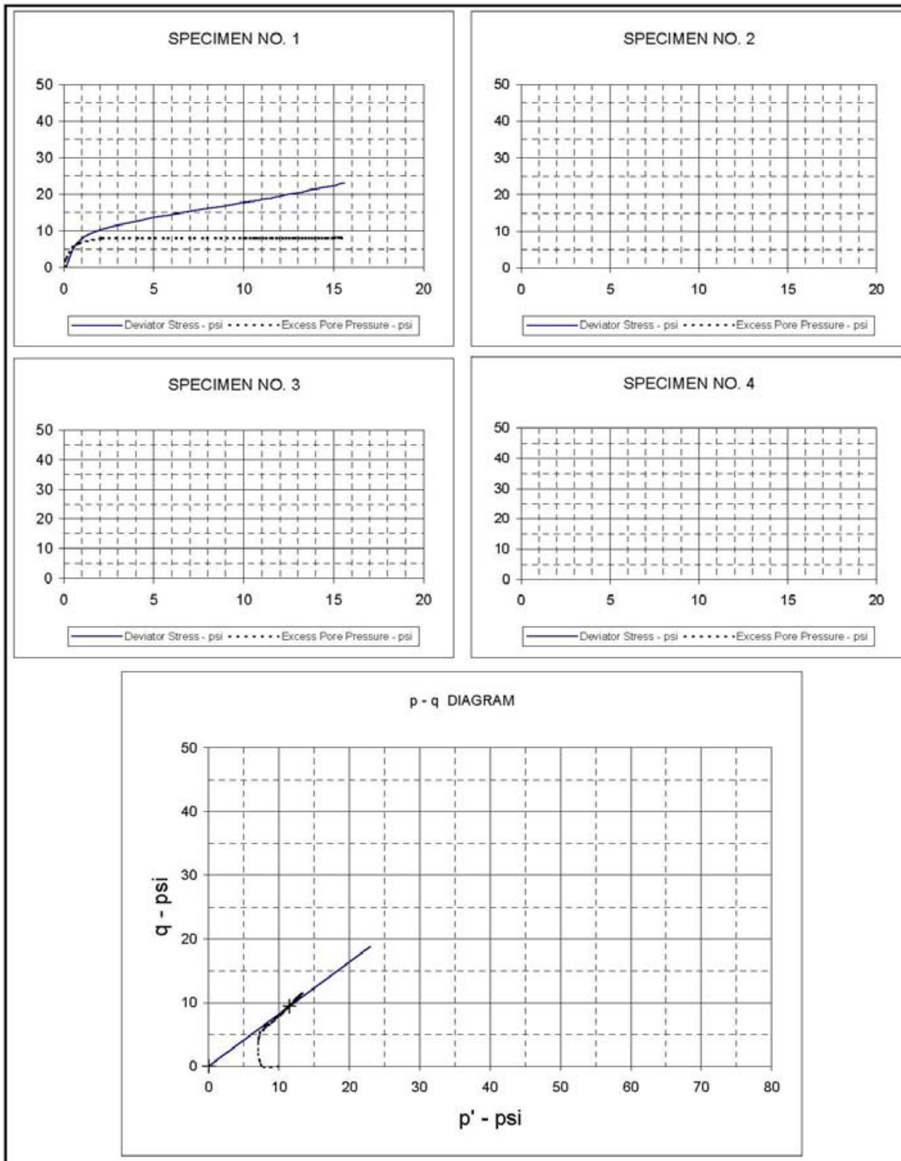
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-C-3
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 2.0 pcf Cell No. 2-1-2

PROJECT INFORMATION

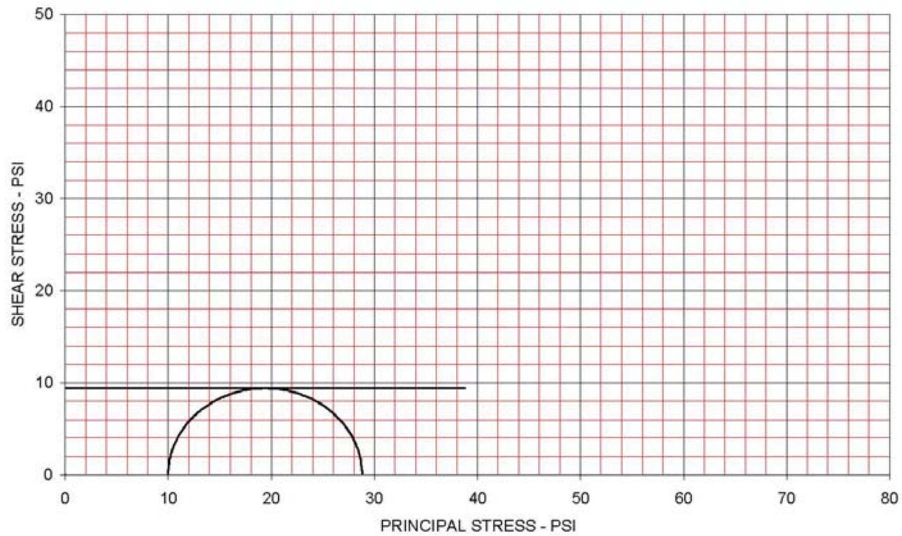
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:11/3/05

GREGORY GEOTECHNICAL PLATE: TX.C3A

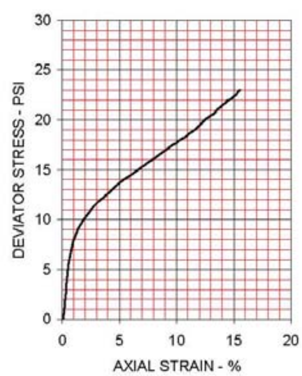


EFFECTIVE STRESS PARAMETERS	R^2 =	α (deg) = 39.3	a (psi) = 0.0
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-1-C-3	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.C3B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 0.0 deg	c = 9.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4			
Dry Density - pcf	94.6			
Diameter - inches	2.88			
Height - inches	5.80			
AT TEST				
Final Moisture - %	27.8			
Dry Density - pcf	96.8			
Calculated Diameter (in.)	2.86			
Height - inches	5.75			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	18.80			
Total Pore Pressure - psi	68.2			
Strain Rate - inches/min.	0.00050			
Failure Strain - %	15.4			
σ_1 Failure - psi	28.80			
σ_3 Failure - psi	10.00			

TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-1-C-3
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 2.0 pcf Cell No. 2-1-2

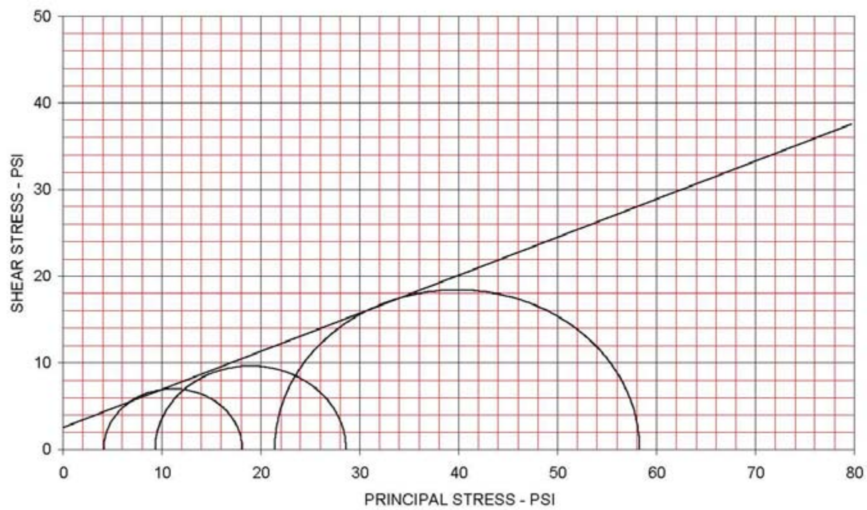
PROJECT INFORMATION

PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:11/3/05

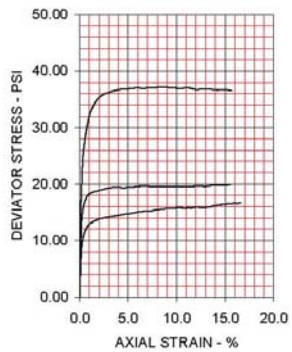
GREGORY GEOTECHNICAL PLATE: TX.C3C

9-TRX60001-C-3.xls

TRIAXIAL SHEAR TEST REPORT

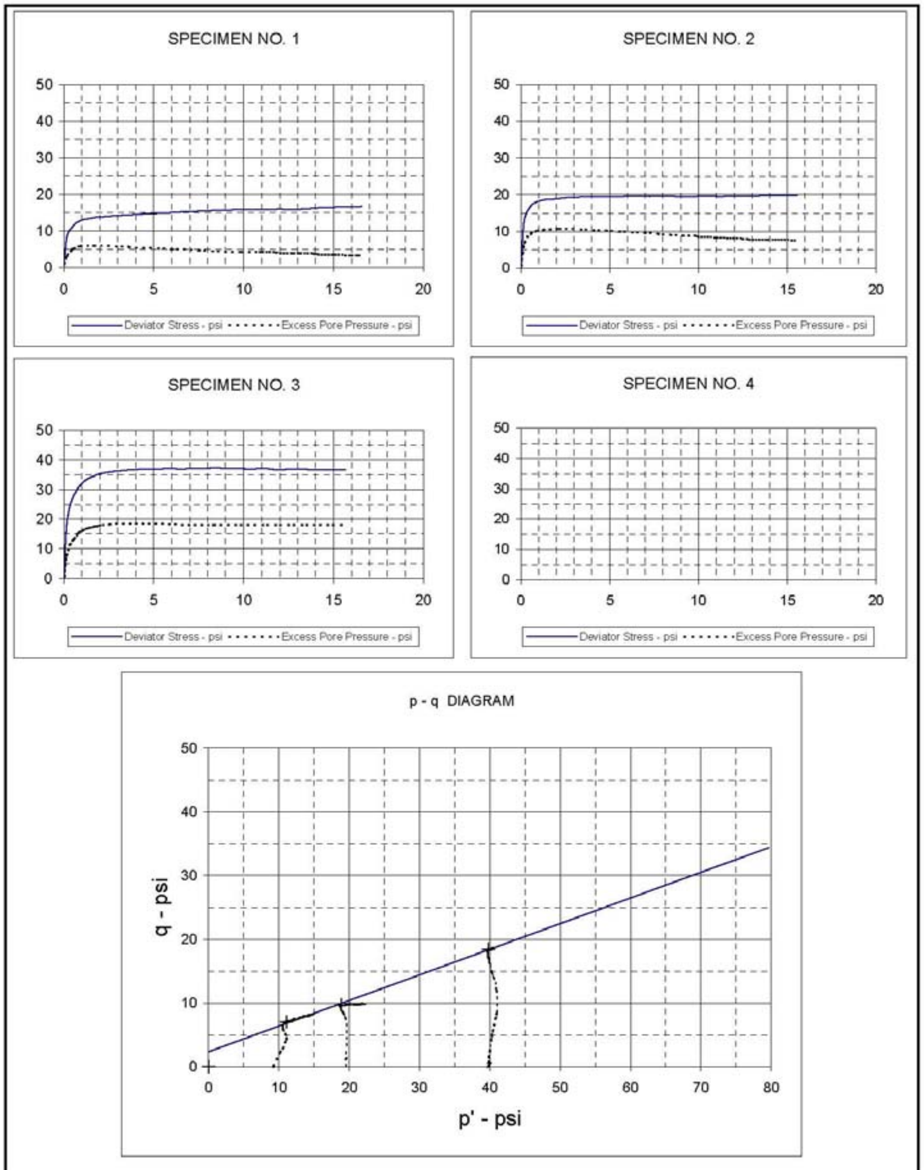


EFFECTIVE STRESS PARAMETERS	ϕ ' =	23.7 deg	c' =	2.5 psi



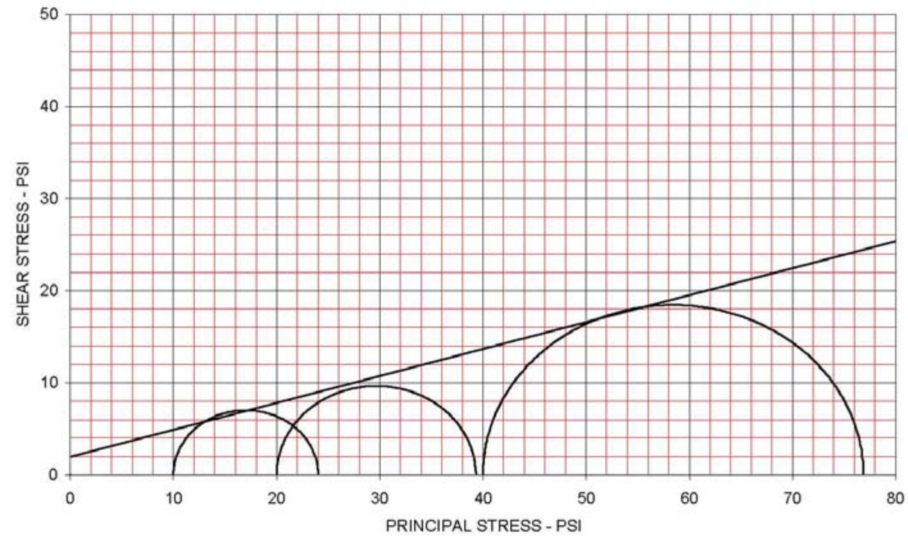
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.6	25.2	22.2	
Dry Density - pcf	96.0	96.5	100.0	
Diameter - inches	2.77	2.80	2.82	
Height - inches	5.51	5.82	5.75	
AT TEST				
Final Moisture - %	25.8	27.1	22.6	
Dry Density - pcf	98.1	99.3	102.4	
Calculated Diameter (in.)	2.76	2.79	2.81	
Height - inches	5.48	5.79	5.71	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	14.03	19.31	36.87	
Total Pore Pressure - psi	65.9	70.7	78.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	2.4	2.9	5.1	
σ_1' Failure - psi	18.13	28.61	58.27	
σ_3' Failure - psi	4.10	9.30	21.40	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CU' Triaxial - TX-F1-1	PROJECT:GHG-PhD Research
SAMPLE TYPE: Field Compacted-Shelby Tube Sample	LOCATION: Stillwater, Oklahoma
DESCRIPTION: FAT CLAY (CH), grayish brown	PROJECT NO: GHG60001
SAMPLE LOCATION: Field Samples from FRS on Jobsite	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.7	DATE:3/15/06
LL: 59 PL: 20 PI: 39 Percent -200: 94	***GREGORY GEOTECHNICAL*** PLATE: TX.F1A
REMARKS: Fiber Rate = 0.22 (Field) Index prop. From Bulk	

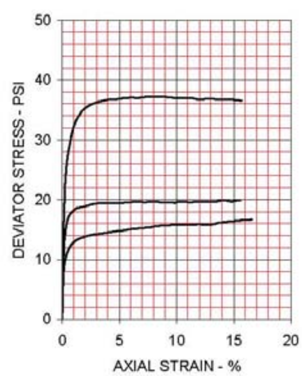


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 21.9	a (psi) = 2.3
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-F1-1	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.F1B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ =	16.3 deg	c =	1.9 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.6	25.2	22.2	
Dry Density - pcf	96.0	96.5	100.0	
Diameter - inches	2.77	2.80	2.82	
Height - inches	5.51	5.82	5.75	
AT TEST				
Final Moisture - %	25.8	27.1	22.6	
Dry Density - pcf	98.1	99.3	102.4	
Calculated Diameter (in.)	2.76	2.79	2.81	
Height - inches	5.48	5.79	5.71	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	14.03	19.31	36.87	
Total Pore Pressure - psi	65.9	70.7	78.6	
Strain Rate - inches/min.	0.00050	0.00050	0.00050	
Failure Strain - %	2.4	2.9	5.1	
σ_1 Failure - psi	24.03	39.31	76.87	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

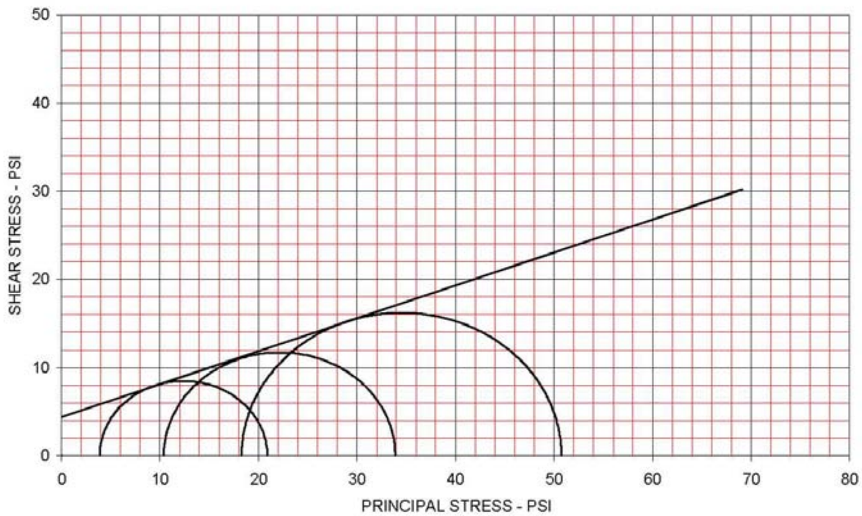
TYPE OF TEST & NO: CU' Triaxial - TX-F1-1
SAMPLE TYPE: Field Compacted-Shelby Tube Sample
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: Field Samples from FRS on Jobsite
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.22 (Field) Index prop. From Bulk

PROJECT INFORMATION

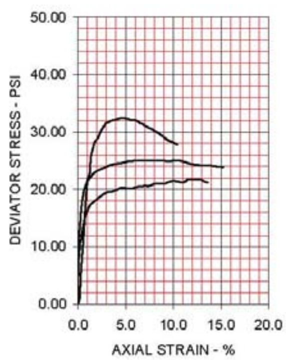
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:3/15/06

GREGORY GEOTECHNICAL PLATE: TX.F1C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	20.5 deg	c' =	4.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	22.8	24.5	24.4	
Dry Density - pcf	102.3	95.8	100.3	
Diameter - inches	2.79	2.83	2.80	
Height - inches	5.81	5.69	5.76	
AT TEST				
Final Moisture - %	26.9	27.0	24.4	
Dry Density - pcf	103.4	96.9	101.5	
Calculated Diameter (in.)	2.77	2.81	2.78	
Height - inches	5.76	5.63	5.69	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	17.00	23.49	32.46	
Total Pore Pressure - psi	66.1	69.6	81.7	
Strain Rate - inches/min.	0.00050	0.00050	0.05000	
Failure Strain - %	1.2	2.3	4.7	
σ_1' Failure - psi	20.90	33.89	50.76	
σ_3' Failure - psi	3.90	10.40	18.30	

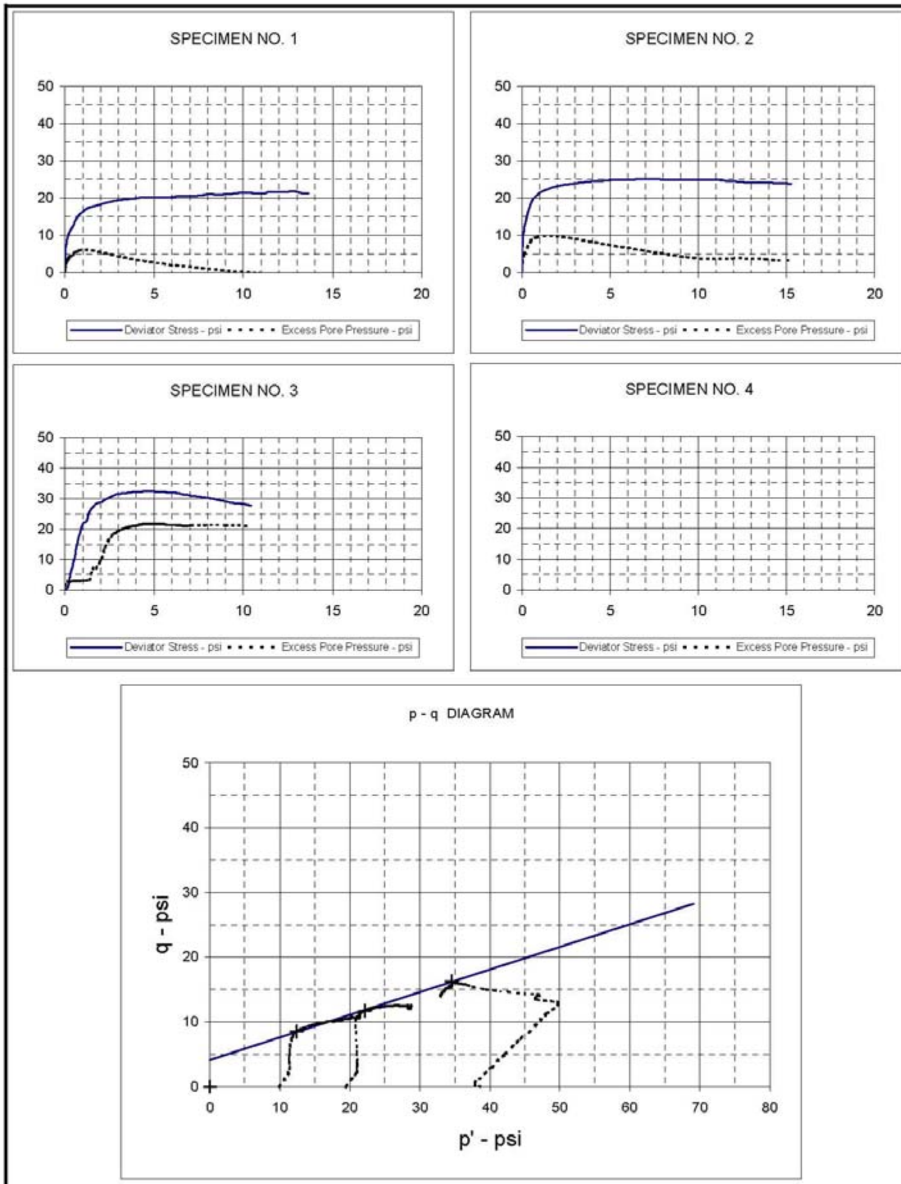
TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-F1-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: Fiber-reinforced Field Specimens
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.19 pcf (field) Index Prop. From Bulk

PROJECT INFORMATION

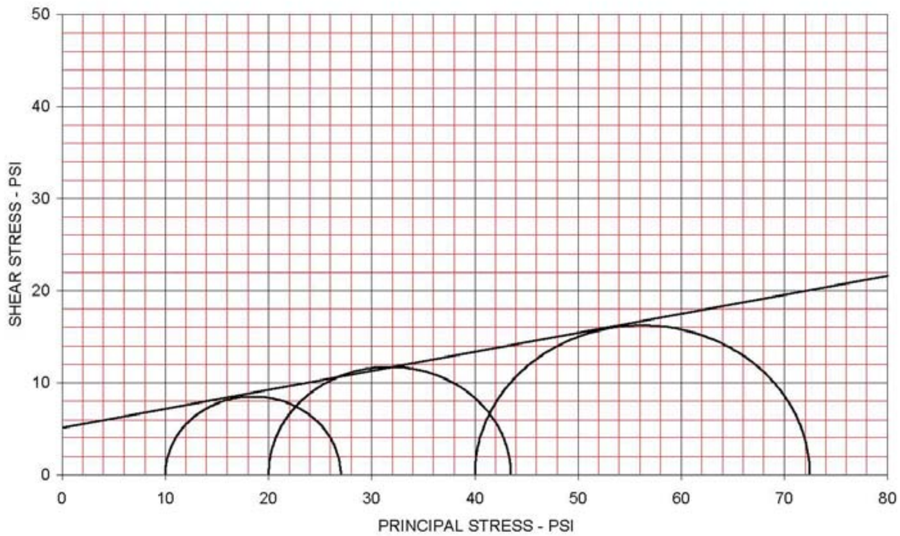
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL | PLATE: TX.F2A

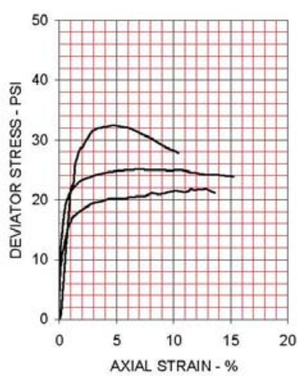


EFFECTIVE STRESS PARAMETERS	R^2 = 1.00	α (deg) = 19.3	a (psi) = 4.1
PROJECT:GHG-PhD Research		TYPE OF TEST & NO: CU' Triaxial - TX-F1-2	
PROJECT NO: GHG60001		***GREGORY GEOTECHNICAL***	PLATE: TX.F2B
DESCRIPTION: FAT CLAY (CH), grayish brown			

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	φ =	11.7 deg	c =	5.1 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	22.8	24.5	24.4	
Dry Density - pcf	102.3	95.8	100.3	
Diameter - inches	2.79	2.83	2.80	
Height - inches	5.81	5.69	5.76	
AT TEST				
Final Moisture - %	26.9	27.0	24.4	
Dry Density - pcf	103.4	96.9	101.5	
Calculated Diameter (in.)	2.77	2.81	2.78	
Height - inches	5.76	5.63	5.69	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	17.00	23.49	32.46	
Total Pore Pressure - psi	66.1	69.6	81.7	
Strain Rate - inches/min.	0.00050	0.00050	0.05000	
Failure Strain - %	1.2	2.3	4.7	
σ_1 Failure - psi	27.00	43.49	72.46	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

TYPE OF TEST & NO: CU' Triaxial - TX-F1-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: FAT CLAY (CH), grayish brown
SAMPLE LOCATION: Fiber-reinforced Field Specimens
ASSUMED SPECIFIC GRAVITY: 2.7
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.19 pcf (field) Index Prop. From Bulk

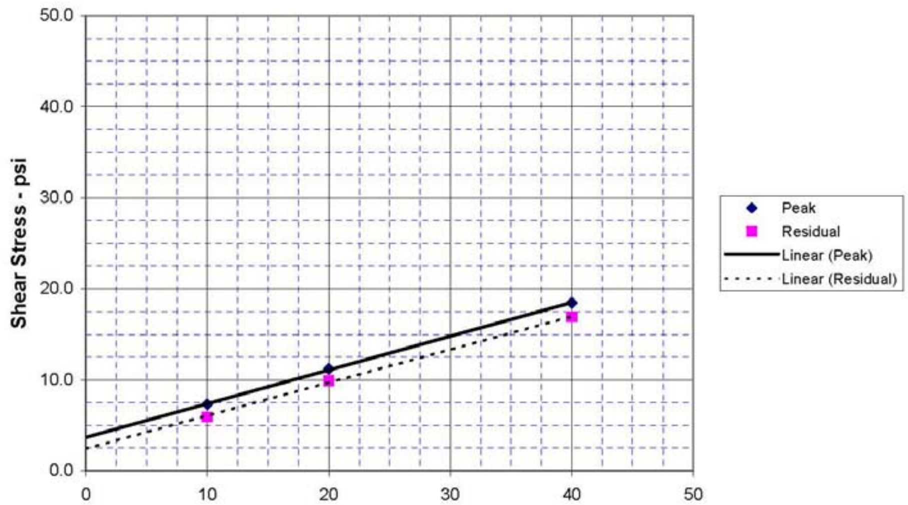
PROJECT INFORMATION

PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:7/25/05

GREGORY GEOTECHNICAL PLATE: TX.F2C

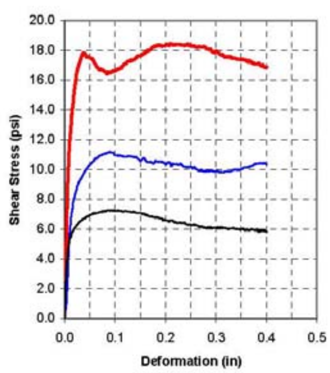
$y = 0.3707x + 3.6696$
$R^2 = 0.9996$

DIRECT SHEAR TEST REPORT



$y = 0.3643x + 2.4$
$R^2 = 0.9988$

PEAK STRENGTH PARAMETERS	φ =	20.3 deg	c =	3.7 psi
RESIDUAL STRENGTH PARAMETERS	φ =	20.0 deg	c =	2.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	30.6	30.2	26.5	
Dry Density - pcf	94.8	97.2	103.3	
Height-End of Consol. (in.)	2.25	2.19	2.06	
Height-End of Shear (in.)	2.23	2.18	2.01	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.3	11.2	18.5	
Residual Failure Stress-psi	5.9	9.9	16.9	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	3.7	3.6	8.4	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

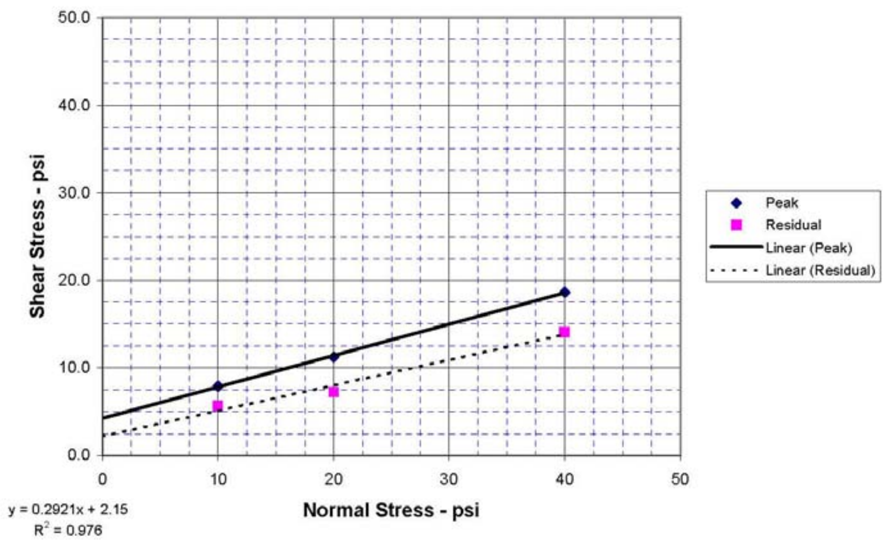
TYPE OF TEST & NO: CD, DS-1-1
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers added.

PROJECT INFORMATION

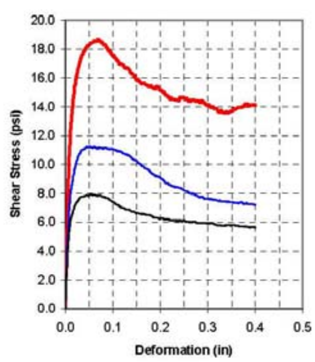
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 8/14/05

GREGORY GEOTECHNICAL PLATE: DS.1

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	19.8 deg	c =	4.2 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	16.3 deg	c =	2.2 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	29.8	28.1	25.4	
Dry Density - pcf	94.8	95.2	95.5	
Height-End of Consol. (in.)	2.24	2.24	2.23	
Height-End of Shear (in.)	2.24	2.18	2.23	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.5	9.1	12.5	
Residual Failure Stress-psi	3.8	6.0	9.6	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	1.8	1.9	2.8	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

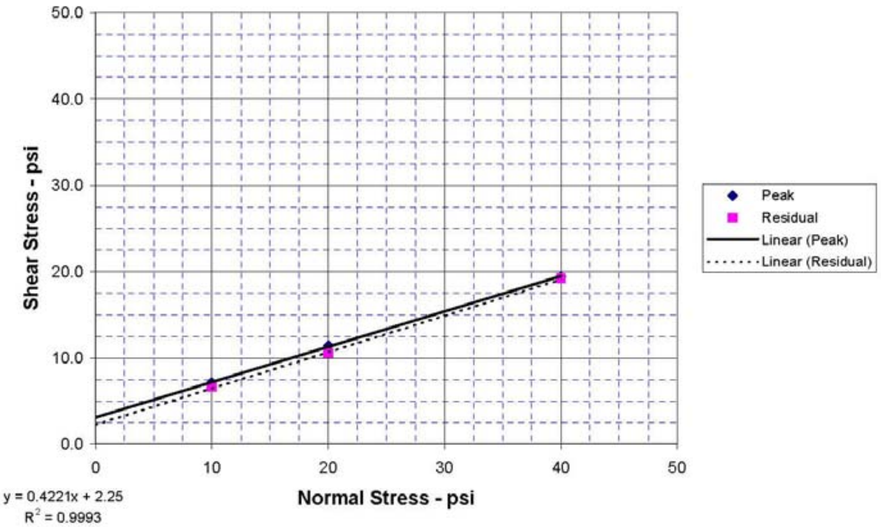
TYPE OF TEST & NO: CD, DS-1-2
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Raw Soil - No fibers added.

PROJECT INFORMATION

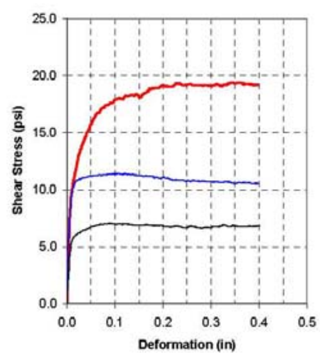
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 8/1/05

GREGORY GEOTECHNICAL PLATE: DS.2

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	22.3 deg	c =	3.1 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	22.9 deg	c =	2.3 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	27.6	28.1	25.4	
Dry Density - pcf	95.1	94.8	100.2	
Height-End of Consol. (in.)	2.24	2.25	2.13	
Height-End of Shear (in.)	2.23	2.24	2.11	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.1	11.4	19.4	
Residual Failure Stress-psi	6.6	10.5	19.2	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	3.4	3.6	13.6	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

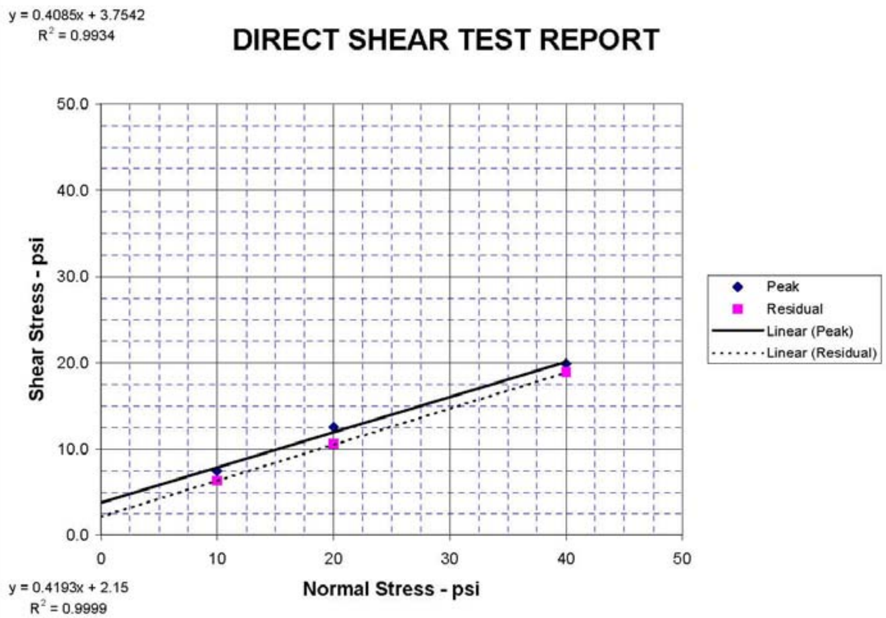
TYPE OF TEST & NO: CD, DS-1-3
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf

PROJECT INFORMATION

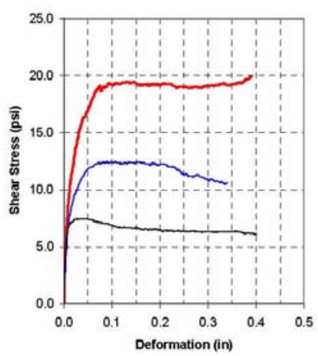
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 8/28/05

GREGORY GEOTECHNICAL PLATE: DS.3

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	22.2 deg	c =	3.8 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	22.7 deg	c =	2.2 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	29.6	28.1	26.0	
Dry Density - pcf	94.7	97.4	99.2	
Height-End of Consol. (in.)	2.25	2.19	2.15	
Height-End of Shear (in.)	2.24	2.18	2.12	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.5	12.5	19.9	
Residual Failure Stress-psi	6.3	10.6	18.9	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	1.3	3.9	5.8	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

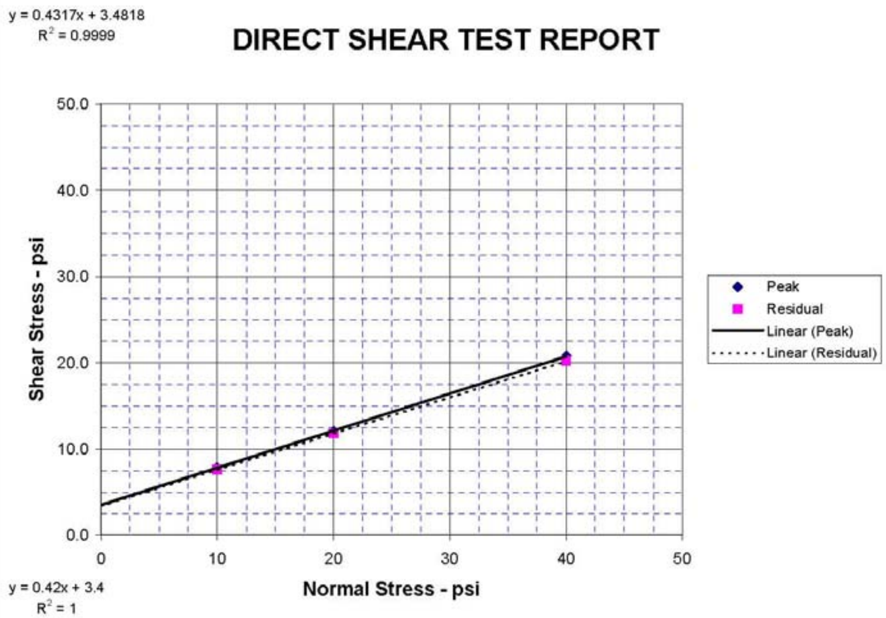
TYPE OF TEST & NO: CD, DS-1-4
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.17 pcf

PROJECT INFORMATION

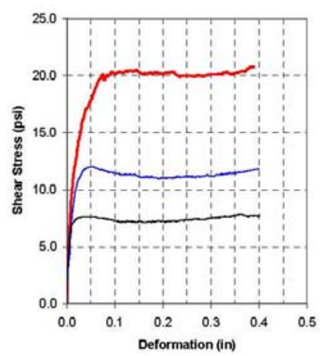
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE: 9/15/05

GREGORY GEOTECHNICAL PLATE: DS.4

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	23.3 deg	c =	3.5 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	22.8 deg	c =	3.4 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	29.3	27.5	26.7	
Dry Density - pcf	97.1	96.6	98.5	
Height-End of Consol. (in.)	2.19	2.20	2.16	
Height-End of Shear (in.)	2.20	2.20	2.15	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.9	12.0	20.8	
Residual Failure Stress-psi	7.6	11.8	20.2	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	1.7	1.8	5.9	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-1-5
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf

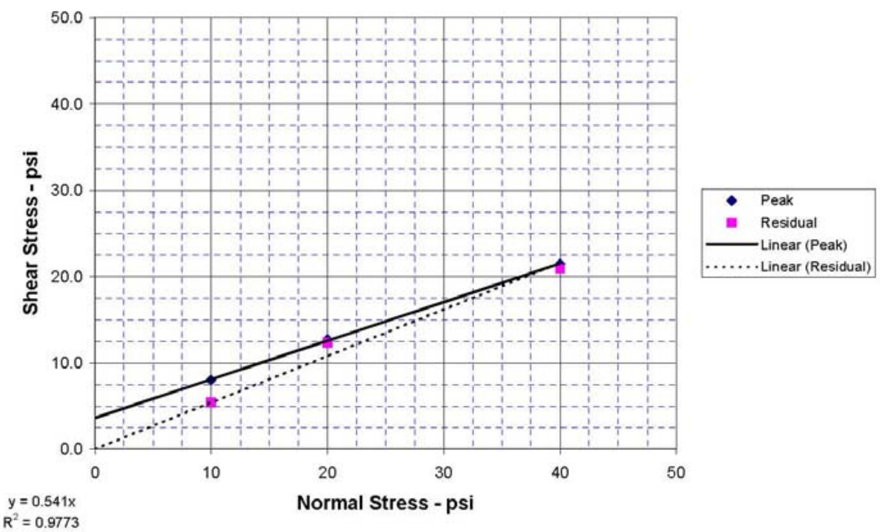
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: Stillwater,OK
DATE: 10/6/05

GREGORY GEOTECHNICAL PLATE: DS.5

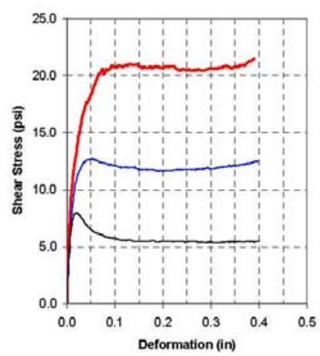
y = 0.447x + 3.6238
R^2 = 0.9997

DIRECT SHEAR TEST REPORT



y = 0.541x
R^2 = 0.9773

PEAK STRENGTH PARAMETERS	ϕ =	24.1 deg	c =	3.6 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	26.8 deg	c =	1.1 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	23.4	23.4	23.4	
Dry Density - pcf	94.6	94.6	94.6	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	29.3	28.8	26.7	
Dry Density - pcf	96.2	96.8	99.9	
Height-End of Consol. (in.)	2.21	2.20	2.13	
Height-End of Shear (in.)	2.22	2.18	2.12	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	8.0	12.7	21.5	
Residual Failure Stress-psi	5.4	12.3	20.9	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	0.1	2.1	5.9	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

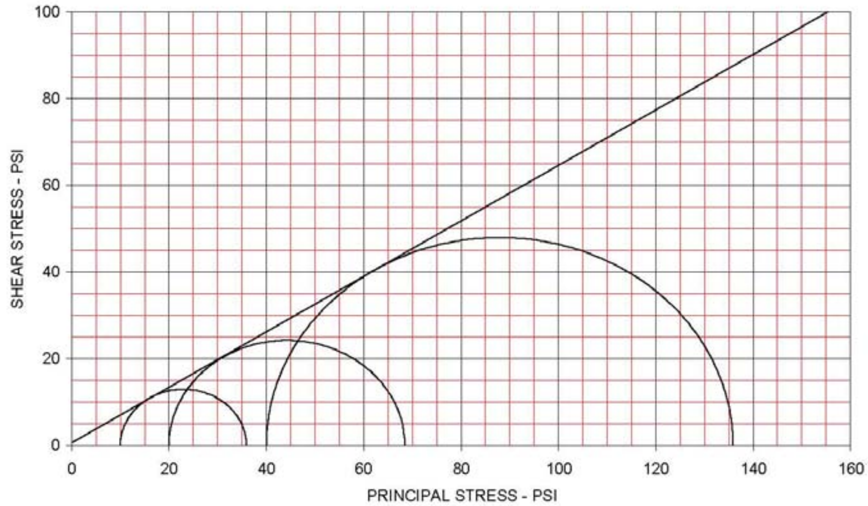
TYPE OF TEST & NO: CD, DS-1-6
SAMPLE TYPE: Re-Compacted
DESCRIPTION: Fat Clay (CH), grayish brown
SAMPLE LOCATION: L60001-1
ASSUMED SPECIFIC GRAVITY: 2.70
LL: 59 PL: 20 PI: 39 Percent -200: 94
REMARKS: Fiber Rate = 0.25 pcf

PROJECT INFORMATION

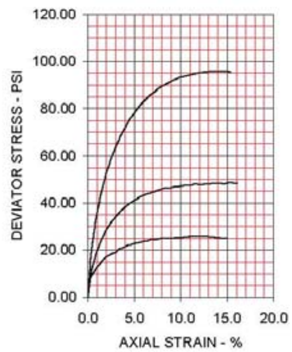
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater, Oklahoms
PROJECT NO: GHG6001
CLIENT: Stillwater,OK
DATE: 9/16/05

GREGORY GEOTECHNICAL PLATE: DS.6

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' =	32.6 deg	c' =	0.6 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	17.3	16.4	15.2	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	25.91	48.48	95.89	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	12.5	15.4	12.9	
σ_1' Failure - psi	35.91	68.48	135.89	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

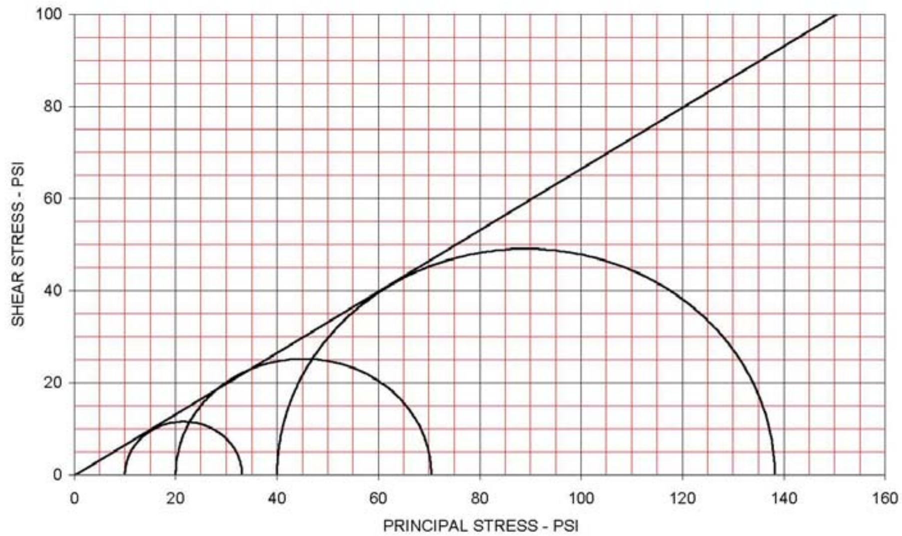
TYPE OF TEST & NO: CD Triaxial - TX-2-1
SAMPLE TYPE: Bulk-Lab Compacted
DESCRIPTION: SITLY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Saturated during test. Raw Soil - No fibers added.

PROJECT INFORMATION

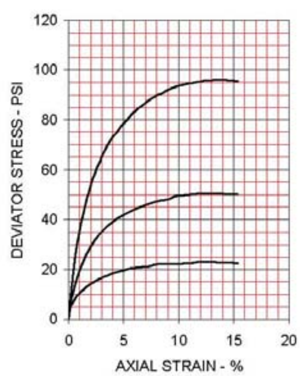
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:9/27/05

GREGORY GEOTECHNICAL PLATE: TX.S1A

TRIAXIAL SHEAR TEST REPORT



TOTAL STRESS PARAMETERS	ϕ = 33.7 deg	c = -0.2 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	16.8	15.5	14.8	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	23.08	50.43	98.20	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	11.8	13.2	12.0	
σ_1 Failure - psi	33.08	70.43	138.20	
σ_3 Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

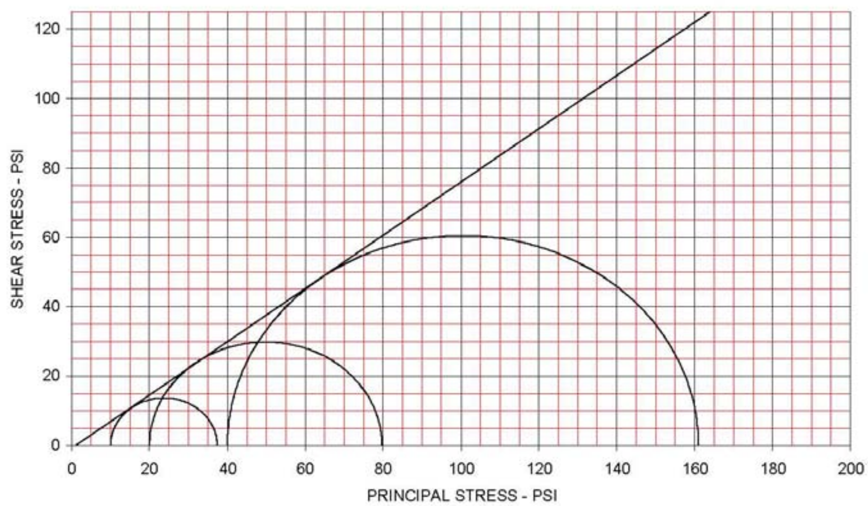
TYPE OF TEST & NO: CD Triaxial - TX-2-2
SAMPLE TYPE: Bulk-Lab Compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Saturated during test. Raw soil - No fibers added.

PROJECT INFORMATION

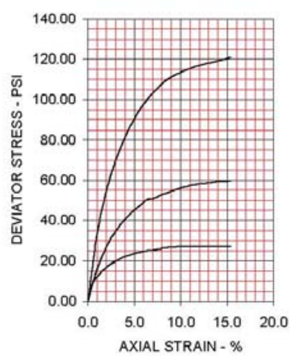
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/3/05

GREGORY GEOTECHNICAL PLATE: TXS.2C

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	φ ' =	37.5 deg	c' =	-0.8 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	16.0	16.7	15.5	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	27.40	59.70	121.01	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	15.4	15.4	15.4	
σ_1' Failure - psi	37.40	79.70	161.01	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION

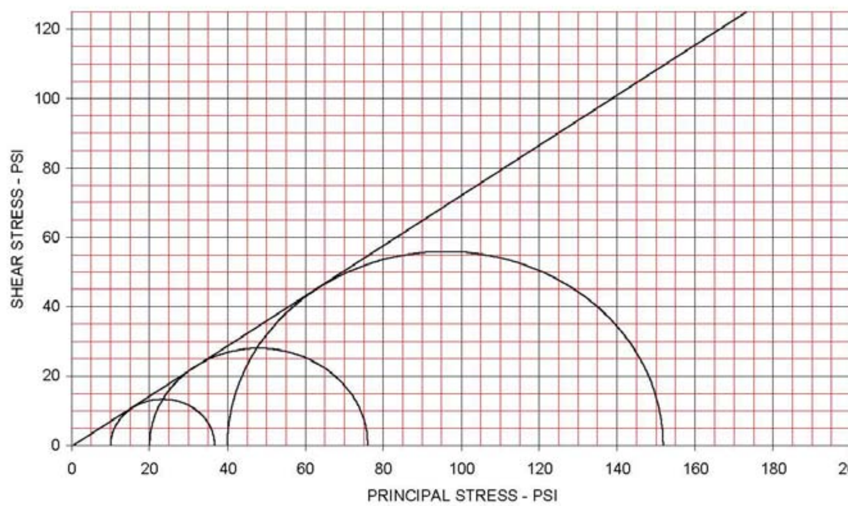
TYPE OF TEST & NO: CD Triaxial - TX-2-3
SAMPLE TYPE: Bulk-Lab Compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Saturated during test. Fiber rate = 0.17 lbs/ft3

PROJECT INFORMATION

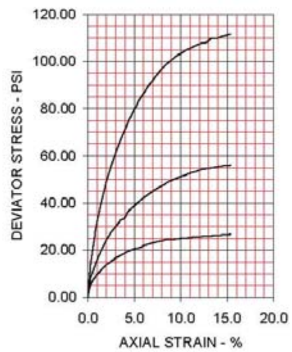
PROJECT:GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE:10/4/05

GREGORY GEOTECHNICAL PLATE: TXS.3A

TRIAXIAL SHEAR TEST REPORT



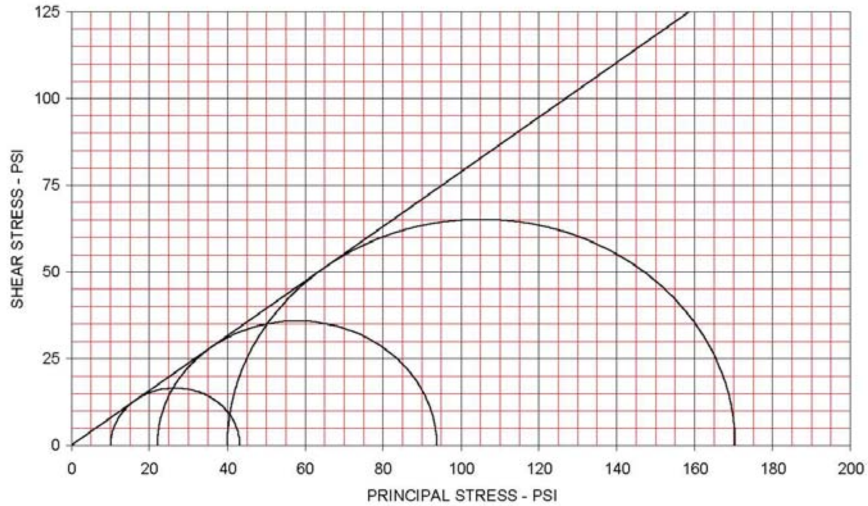
EFFECTIVE STRESS PARAMETERS	φ ' =	35.9 deg	c' =	-0.3 psi



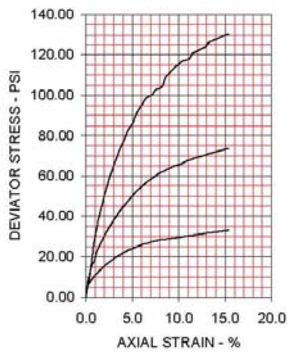
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	16.5	16.8	15.8	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	26.72	56.03	111.80	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	15.4	15.4	15.4	
σ_1' Failure - psi	36.72	76.03	151.80	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-4	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:10/5/05
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.4A
REMARKS: Saturated during test. Fiber rate = 0.17 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



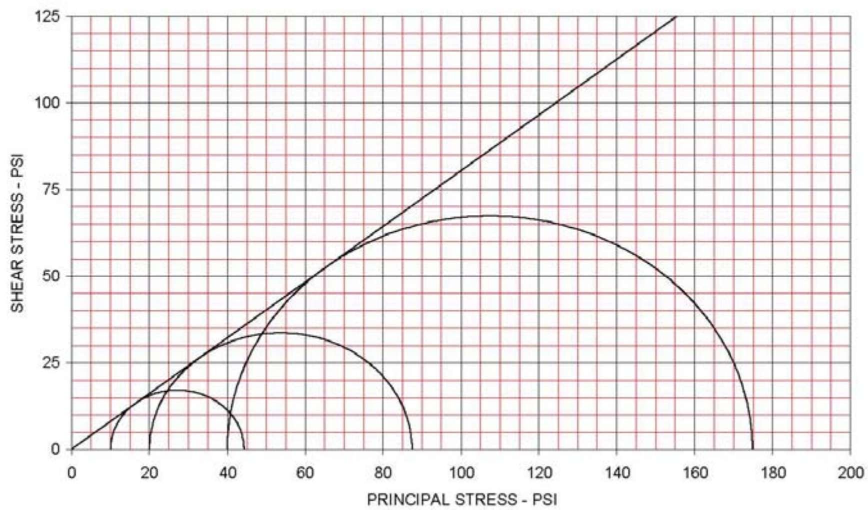
EFFECTIVE STRESS PARAMETERS	φ ' =	38.2 deg	c' =	0.1 psi



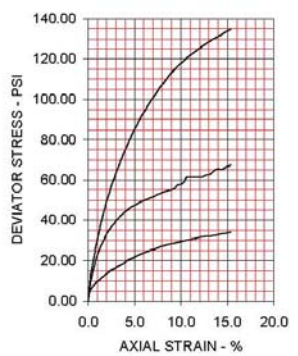
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	16.5	17.3	15.6	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	33.18	71.76	130.44	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	15.4	15.0	15.4	
σ_1' Failure - psi	43.18	93.76	170.44	
σ_3' Failure - psi	10.00	22.00	40.00	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-5	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:10/6/05
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.5A
REMARKS: Saturated during test. Fiber rate = 0.25 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



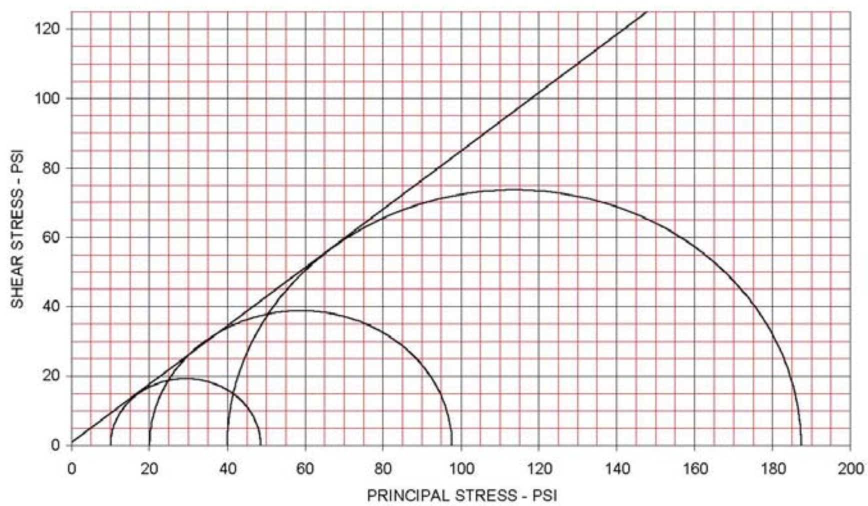
EFFECTIVE STRESS PARAMETERS	φ ' =	38.8 deg	c' =	0.1 psi



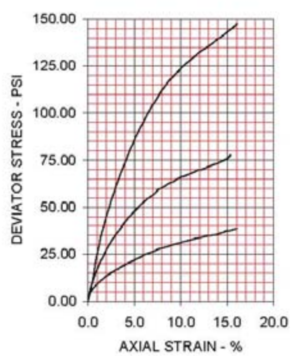
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	16.4	16.8	16.4	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	34.30	67.41	134.84	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	15.4	15.4	15.4	
σ_1' Failure - psi	44.30	87.41	174.84	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-6	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:10/7/05
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.6A
REMARKS: Saturated during test. Fiber rate = 0.25 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



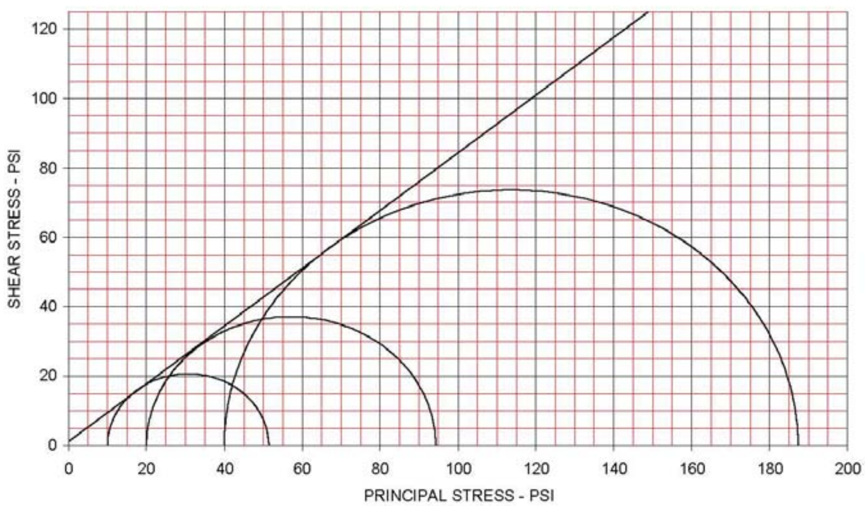
EFFECTIVE STRESS PARAMETERS	φ ' =	40.0 deg	c' =	0.9 psi



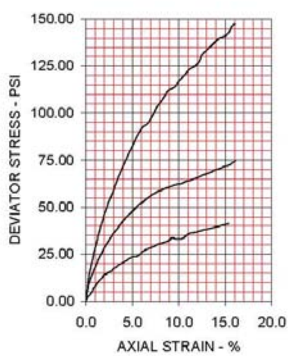
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	18.6	17.2	16.4	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	38.57	77.67	147.34	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	16.1	15.4	16.1	
σ_1' Failure - psi	48.57	97.67	187.34	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-7	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:10/7/05
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.7A
REMARKS: Saturated during test. Fiber rate = 0.5 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



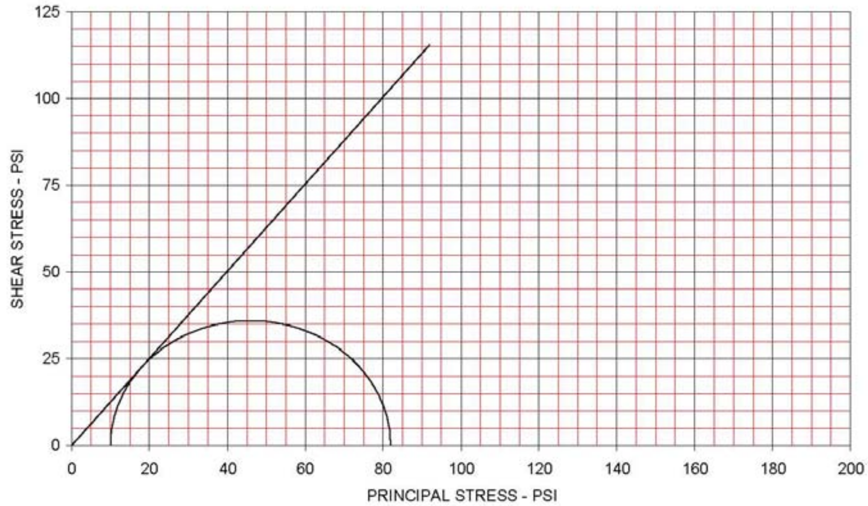
EFFECTIVE STRESS PARAMETERS	φ ' =	39.8 deg	c' =	1.1 psi



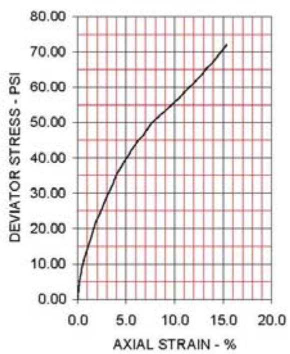
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
AT TEST				
Final Moisture - %	18.6	16.4	16.2	
Dry Density - pcf	99.0	99.0	99.0	
Calculated Diameter (in.)	2.80	2.80	2.80	
Height - inches	5.60	5.60	5.60	
Effect. Cell Pressure - psi	10.0	20.0	40.0	
Failure Stress - psi	41.38	74.28	147.34	
Total Pore Pressure - psi	0.0	0.0	0.0	
Strain Rate - inches/min.	0.03000	0.03000	0.03000	
Failure Strain - %	15.4	16.1	16.1	
σ_1' Failure - psi	51.38	94.28	187.34	
σ_3' Failure - psi	10.00	20.00	40.00	

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-8	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:10/7/05
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.8A
REMARKS: Saturated during test. Fiber rate = 0.5 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



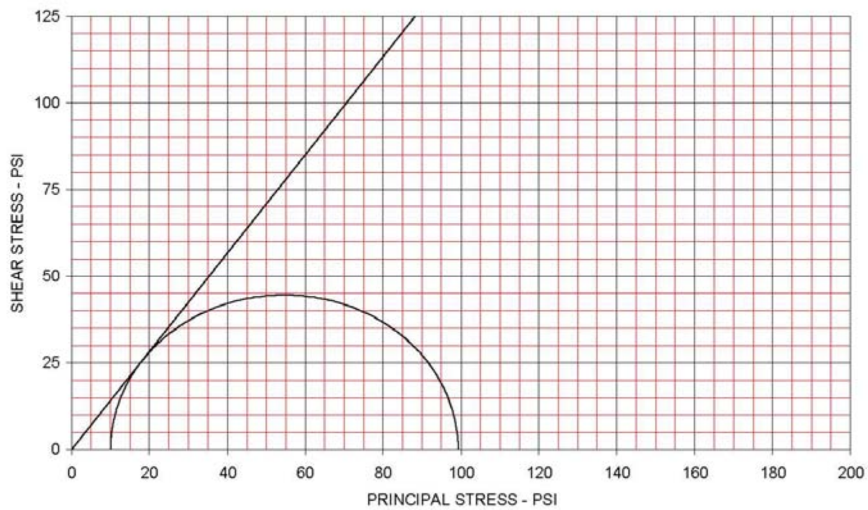
EFFECTIVE STRESS PARAMETERS	ϕ ' =	51.5 deg	c' =	0.0 psi



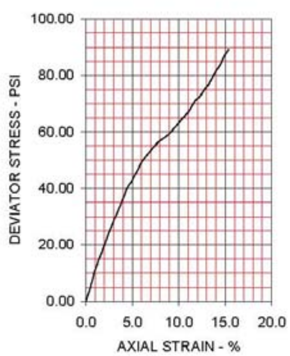
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0			
Dry Density - pcf	99.0			
Diameter - inches	2.80			
Height - inches	5.60			
AT TEST				
Final Moisture - %	16.8			
Dry Density - pcf	99.0			
Calculated Diameter (in.)	2.80			
Height - inches	5.60			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	71.93			
Total Pore Pressure - psi	0.0			
Strain Rate - inches/min.	0.03000			
Failure Strain - %	15.4			
σ_1' Failure - psi	81.93			
σ_3' Failure - psi	10.00			

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-C-1	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:3/15/06
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.C1
REMARKS: Saturated during test. Fiber rate = 1.0 lbs/ft3	

TRIAXIAL SHEAR TEST REPORT



EFFECTIVE STRESS PARAMETERS	ϕ ' =	54.8 deg	c' =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0			
Dry Density - pcf	99.0			
Diameter - inches	2.80			
Height - inches	5.60			
AT TEST				
Final Moisture - %	17.9			
Dry Density - pcf	99.0			
Calculated Diameter (in.)	2.80			
Height - inches	5.60			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	89.27			
Total Pore Pressure - psi	0.0			
Strain Rate - inches/min.	0.03000			
Failure Strain - %	15.4			
σ_1' Failure - psi	99.27			
σ_3' Failure - psi	10.00			

TEST DESCRIPTION

TYPE OF TEST & NO: CD Triaxial - TX-2-C-2
SAMPLE TYPE: Bulk-Lab Compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Saturated during test. Fiber rate = 1.5 lbs/ft3

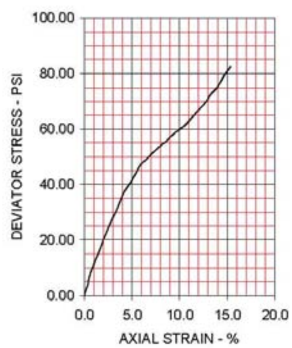
PROJECT INFORMATION

PROJECT: GHG-PhD Research
LOCATION: Stillwater, Oklahoma
PROJECT NO: GHG60001
CLIENT: GHG-OSU
DATE: 3/15/06

GREGORY GEOTECHNICAL PLATE: TXS.C2

TRIAXIAL SHEAR TEST REPORT

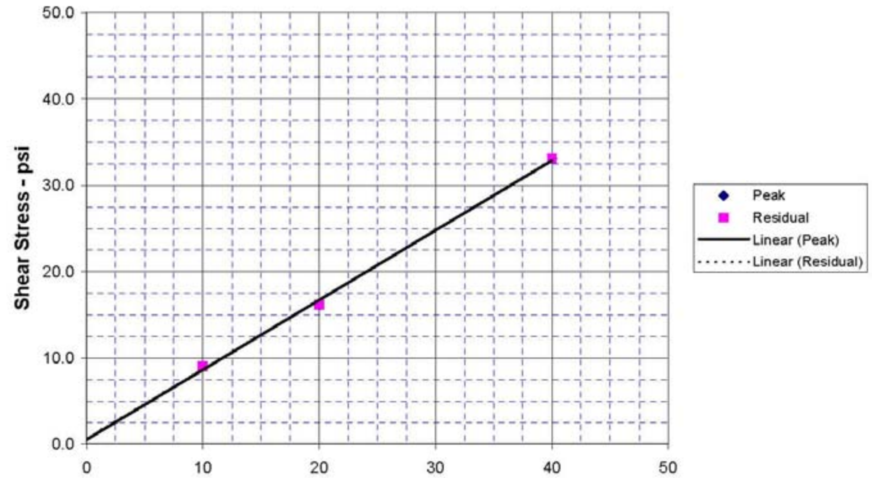
EFFECTIVE STRESS PARAMETERS	φ ' = 53.6 deg	c' = 0.0 psi



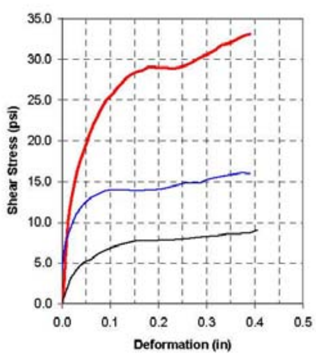
SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0			
Dry Density - pcf	99.0			
Diameter - inches	2.80			
Height - inches	5.60			
AT TEST				
Final Moisture - %	19.8			
Dry Density - pcf	99.0			
Calculated Diameter (in.)	2.80			
Height - inches	5.60			
Effect. Cell Pressure - psi	10.0			
Failure Stress - psi	82.56			
Total Pore Pressure - psi	0.0			
Strain Rate - inches/min.	0.03000			
Failure Strain - %	15.4			
σ_1' Failure - psi	92.56			
σ_3' Failure - psi	10.00			

TEST DESCRIPTION	PROJECT INFORMATION
TYPE OF TEST & NO: CD Triaxial - TX-2-C-3	PROJECT:GHG-PhD Research
SAMPLE TYPE: Bulk-Lab Compacted	LOCATION: Stillwater, Oklahoma
DESCRIPTION: SILTY SAND (SM), reddish tan	PROJECT NO: GHG60001
SAMPLE LOCATION: L60001-2	CLIENT: GHG-OSU
ASSUMED SPECIFIC GRAVITY: 2.68	DATE:3/15/06
LL: NP PL: NP PI: NP Percent -200: 12.7	***GREGORY GEOTECHNICAL*** PLATE: TXS.C3
REMARKS: Saturated during test. Fiber rate = 2.0 lbs/ft3	

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	39.0 deg	c =	0.5 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	39.0 deg	c =	0.5 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	19.4	20.1	19.5	
Dry Density - pcf	99.3	106.9	103.0	
Height-End of Consol. (in.)	2.24	2.08	2.16	
Height-End of Shear (in.)	2.24	2.09	2.17	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	9.0	16.1	33.1	
Residual Failure Stress-psi	9.0	16.1	33.1	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.6	15.6	15.6	
Residual Failure Strain %	15.6	15.6	15.6	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-1
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Raw Soil - No fibers added.

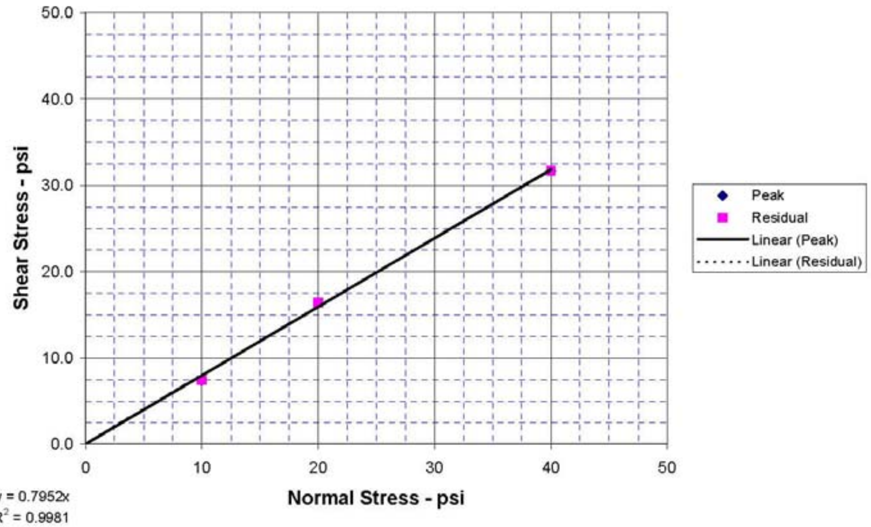
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

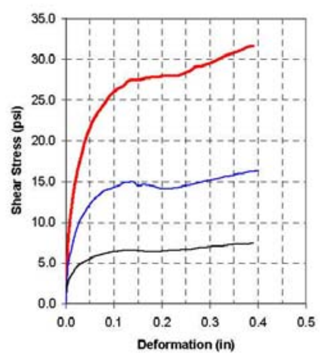
GREGORY GEOTECHNICAL PLATE: DS.7

y = 0.7952x
R^2 = 0.9981

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	38.5 deg	c =	0.0 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	38.5 deg	c =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	18.5	18.3	18.3	
Dry Density - pcf	102.4	103.1	103.1	
Height-End of Consol. (in.)	2.18	2.16	2.16	
Height-End of Shear (in.)	2.17	2.18	2.18	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	7.4	16.4	31.7	
Residual Failure Stress-psi	7.4	16.4	31.7	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	1.4	1.0	1.5	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-2
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Raw Soil - No fibers added.

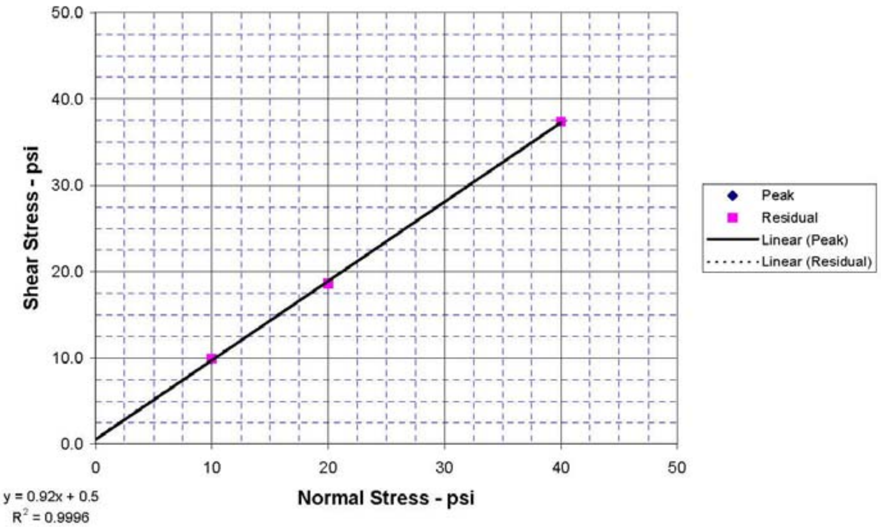
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

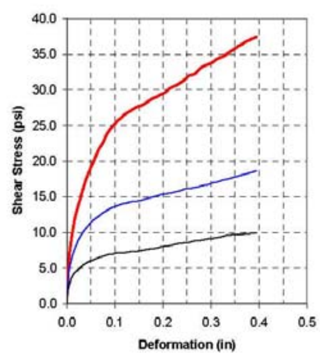
GREGORY GEOTECHNICAL PLATE: DS.8

y = 0.92x + 0.5
R^2 = 0.9996

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	42.6 deg	c =	0.5 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	42.6 deg	c =	0.5 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	19.4	19.2	19.5	
Dry Density - pcf	102.9	103.3	103.5	
Height-End of Consol. (in.)	2.16	2.16	2.15	
Height-End of Shear (in.)	2.14	2.14	2.13	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	9.9	18.6	37.4	
Residual Failure Stress-psi	9.9	18.6	37.4	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	1.4	1.0	1.5	
Residual Failure Strain %	16.00	16.00	16.00	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-3
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.17 pcf

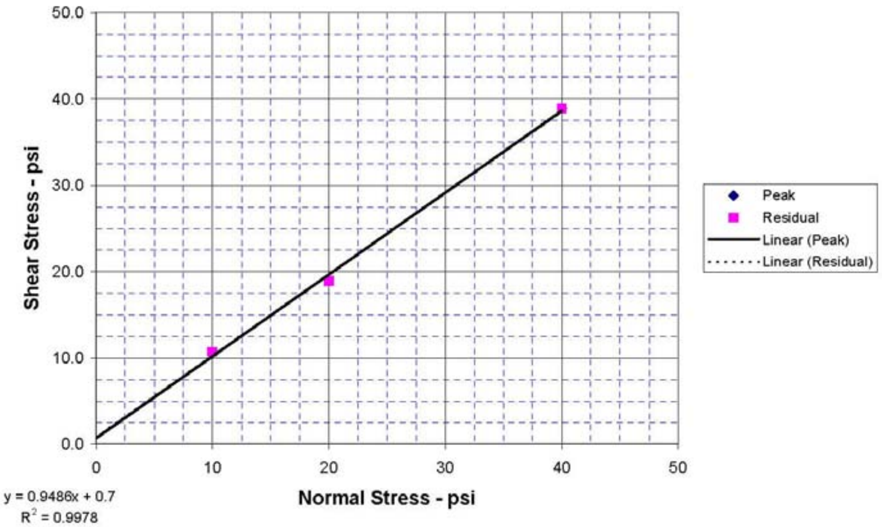
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

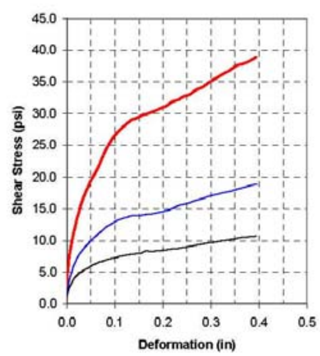
GREGORY GEOTECHNICAL PLATE: DSS.3

y = 0.9486x + 0.7
R^2 = 0.9978

DIRECT SHEAR TEST REPORT



PEAK STRENGTH PARAMETERS	ϕ =	43.5 deg	c =	0.7 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	43.5 deg	c =	0.7 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	19.1	17.9	19.2	
Dry Density - pcf	102.7	103.6	103.5	
Height-End of Consol. (in.)	2.17	2.15	2.15	
Height-End of Shear (in.)	2.15	2.12	2.14	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	10.7	18.9	38.9	
Residual Failure Stress-psi	10.7	18.9	38.9	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.6	15.6	15.6	
Residual Failure Strain %	15.6	15.6	15.6	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-4
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.17 pcf

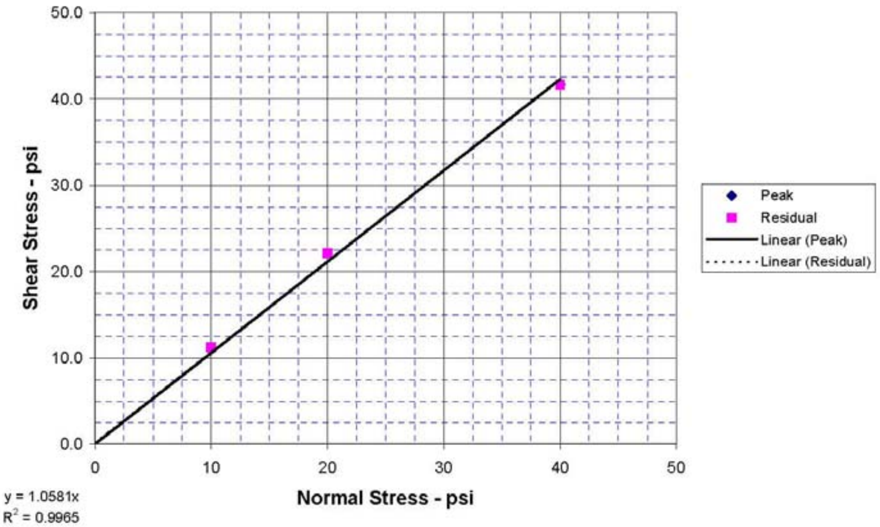
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

GREGORY GEOTECHNICAL PLATE: DSS.4

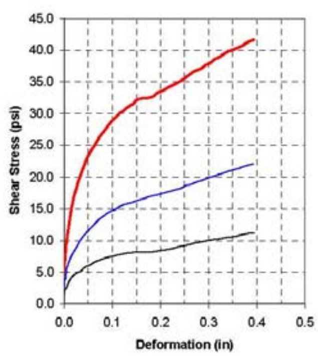
y = 1.0581x
R^2 = 0.9965

DIRECT SHEAR TEST REPORT



y = 1.0581x
R^2 = 0.9965

PEAK STRENGTH PARAMETERS	ϕ =	46.6 deg	c =	0.0 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	46.6 deg	c =	0.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	18.9	18.6	18.0	
Dry Density - pcf	103.2	102.9	104.3	
Height-End of Consol. (in.)	2.16	2.16	2.14	
Height-End of Shear (in.)	2.14	2.15	2.12	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	11.2	22.1	41.7	
Residual Failure Stress-psi	11.2	22.1	41.7	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.7	15.7	15.7	
Residual Failure Strain %	15.7	15.7	15.7	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-5
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.25 pcf

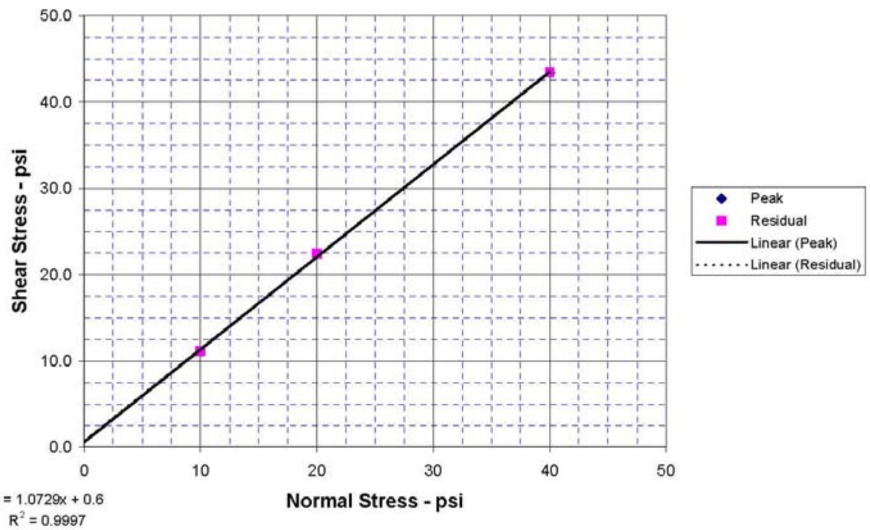
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

GREGORY GEOTECHNICAL PLATE: DSS.5

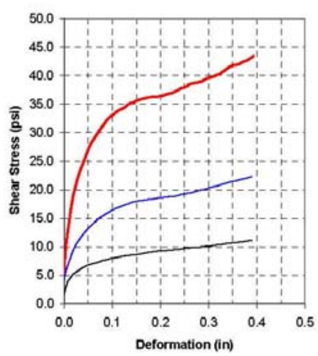
y = 1.0729x + 0.6
R^2 = 0.9997

DIRECT SHEAR TEST REPORT



y = 1.0729x + 0.6
R^2 = 0.9997

PEAK STRENGTH PARAMETERS	ϕ =	47.0 deg	c =	0.6 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	47.0 deg	c =	0.6 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	18.1	19.0	18.6	
Dry Density - pcf	103.1	103.2	104.3	
Height-End of Consol. (in.)	2.16	2.16	2.14	
Height-End of Shear (in.)	2.14	2.14	2.11	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	11.1	22.4	43.4	
Residual Failure Stress-psi	11.1	22.4	43.4	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.7	15.7	15.7	
Residual Failure Strain %	15.7	15.7	15.7	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-6
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.25 pcf

PROJECT INFORMATION

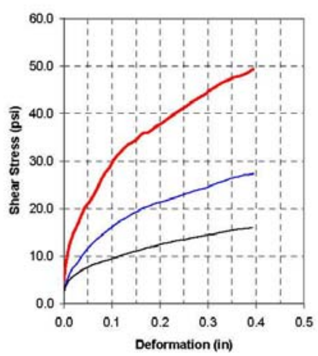
PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

GREGORY GEOTECHNICAL PLATE: DSS.6

y = 1.1114x + 5
R^2 = 0.9999

DIRECT SHEAR TEST REPORT

PEAK STRENGTH PARAMETERS	ϕ =	48.0 deg	c =	5.0 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	48.0 deg	c =	5.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	20.3	20.3	20.1	
Dry Density - pcf	103.1	102.2	103.9	
Height-End of Consol. (in.)	2.16	2.18	2.14	
Height-End of Shear (in.)	2.14	2.18	2.12	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	16.0	27.4	49.4	
Residual Failure Stress-psi	16.0	27.4	49.4	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.7	15.7	15.7	
Residual Failure Strain %	15.7	15.7	15.7	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-7
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.5 pcf

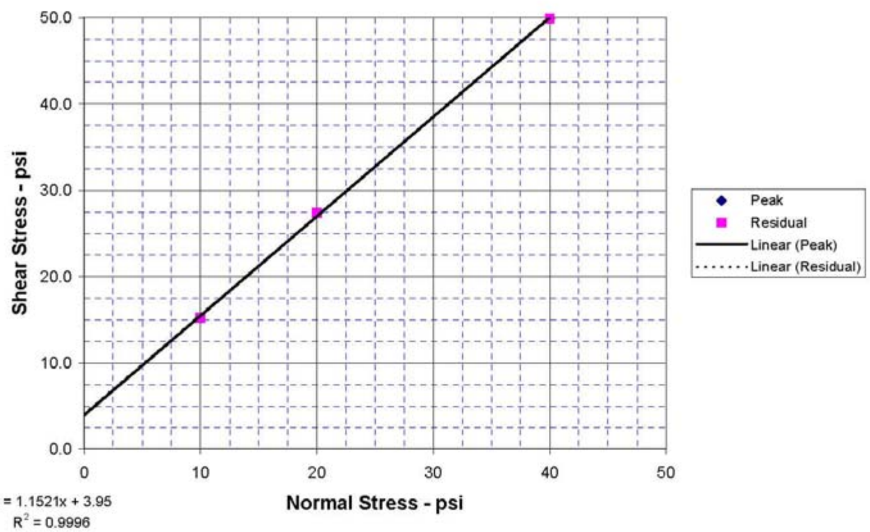
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

GREGORY GEOTECHNICAL PLATE: DSS.7

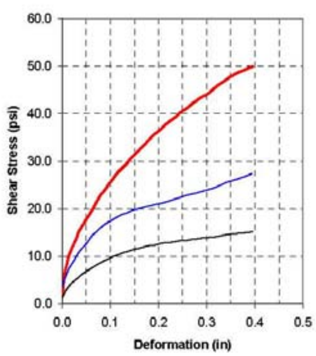
y = 1.1521x + 3.95
R^2 = 0.9996

DIRECT SHEAR TEST REPORT



y = 1.1521x + 3.95
R^2 = 0.9996

PEAK STRENGTH PARAMETERS	ϕ =	49.0 deg	c =	4.0 psi
RESIDUAL STRENGTH PARAMETERS	ϕ =	49.0 deg	c =	4.0 psi



SPECIMEN NO.	1	2	3	4
INITIAL				
Moisture Content - %	0.0	0.0	0.0	
Dry Density - pcf	99.0	99.0	99.0	
Diameter - inches	2.50	2.50	2.50	
Height - inches	2.25	2.25	2.25	
AT TEST				
Final Moisture - %	20.2	24.2	26.1	
Dry Density - pcf	104.1	102.8	104.1	
Height-End of Consol. (in.)	2.14	2.17	2.14	
Height-End of Shear (in.)	2.14	2.16	2.14	
Normal Stress - psi	10.0	20.0	40.0	
Peak Failure Stress-psi	15.2	27.4	49.9	
Residual Failure Stress-psi	15.2	27.4	49.9	
Strain Rate - inches/min.	0.00030	0.00030	0.00030	
Peak Failure Strain - %	15.8	15.8	15.8	
Residual Failure Strain %	15.8	15.8	15.8	

Dry Density at test based on initial moisture and height at end of consolidation.

TEST DESCRIPTION

TYPE OF TEST & NO: CD, DS-2-8
SAMPLE TYPE: Re-compacted
DESCRIPTION: SILTY SAND (SM), reddish tan
SAMPLE LOCATION: L60001-2
ASSUMED SPECIFIC GRAVITY: 2.68
LL: NP PL: NP PI: NP Percent -200: 12.7
REMARKS: Fiber rate = 0.5 pcf

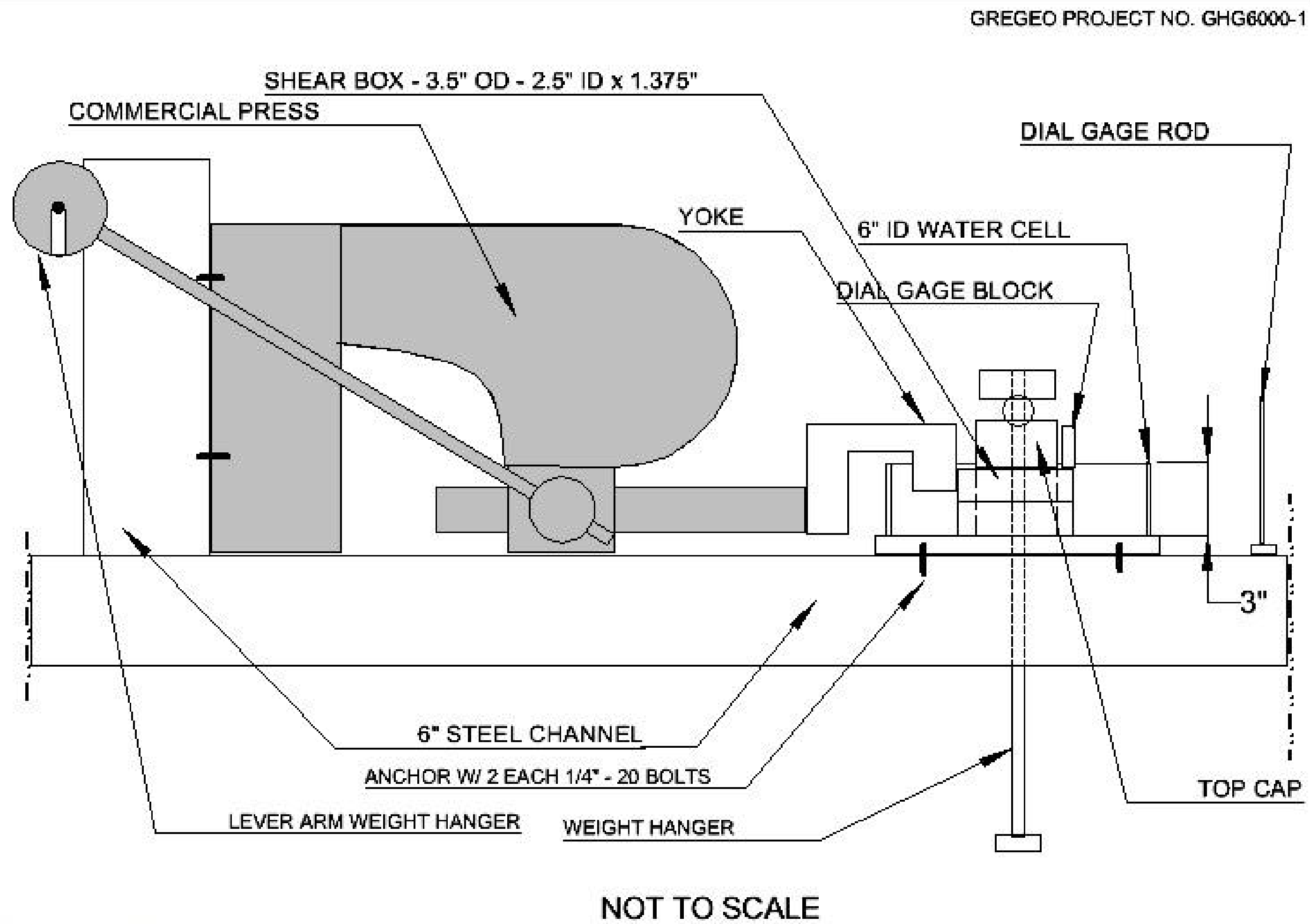
PROJECT INFORMATION

PROJECT: GHG-PhD Research Project
LOCATION: Stillwater,OK
PROJECT NO: GHG6001
CLIENT: GHG-OSU
DATE: 10/25/05

GREGORY GEOTECHNICAL PLATE: DSS.8

APPENDIX B

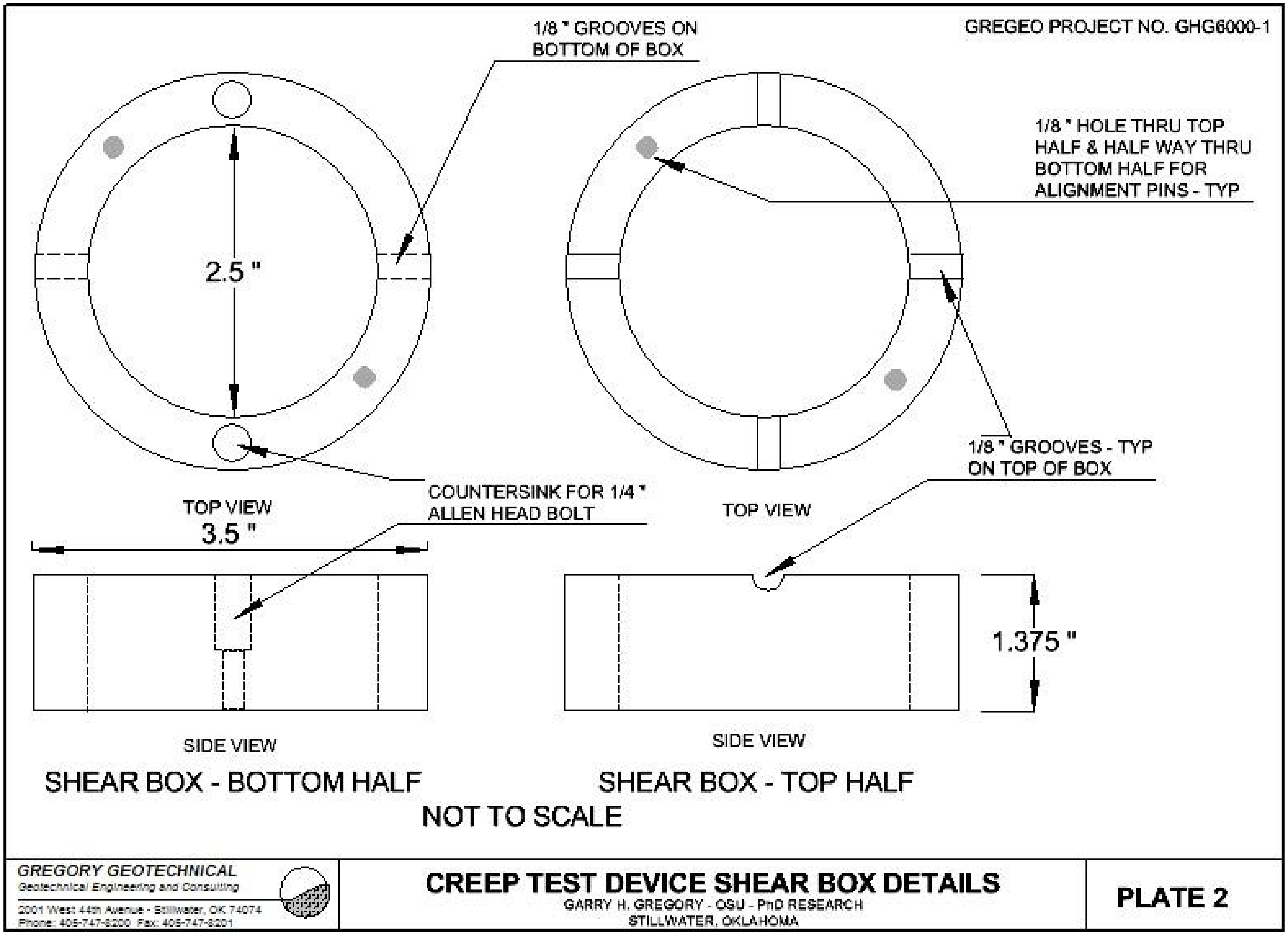
SCHEMATIC DRAWINGS – DIRECT SHEAR CREEP DEVICE



GREGORY GEOTECHNICAL
Geotechnical Engineering and Consulting
2001 West 44th Avenue - Stillwater, OK 74074
Phone: 405-747-8200 Fax: 405-747-8201

DIRECT SHEAR CREEP TEST DEVICE
GARRY H. GREGORY - OSU - PhD RESEARCH
STILLWATER, OKLAHOMA

PLATE 1


GREGEO PROJECT NO. GHG6000-1
1/8 " GROOVES ON BOTTOM OF BOX
1/8 " HOLE THRU TOP HALF & HALF WAY THRU BOTTOM HALF FOR ALIGNMENT PINS - TYP
2.5 "
1/8 " GROOVES - TYP ON TOP OF BOX
COUNTERSINK FOR 1/4 " ALLEN HEAD BOLT
TOP VIEW
TOP VIEW
3.5 "
1.375 "
SIDE VIEW
SIDE VIEW
SHEAR BOX - BOTTOM HALF
SHEAR BOX - TOP HALF
NOT TO SCALE
GREGORY GEOTECHNICAL
Geotechnical Engineering and Consulting
2001 West 44th Avenue - Stillwater, OK 74074
Phone: 405-747-8200 Fax: 405-747-8201
CREEP TEST DEVICE SHEAR BOX DETAILS
GARRY H. GREGORY - OSU - PhD RESEARCH
STILLWATER, OKLAHOMA
PLATE 2

APPENDIX C

SLOPE STABILITY ANALYSIS - COMPUTER OUTPUT

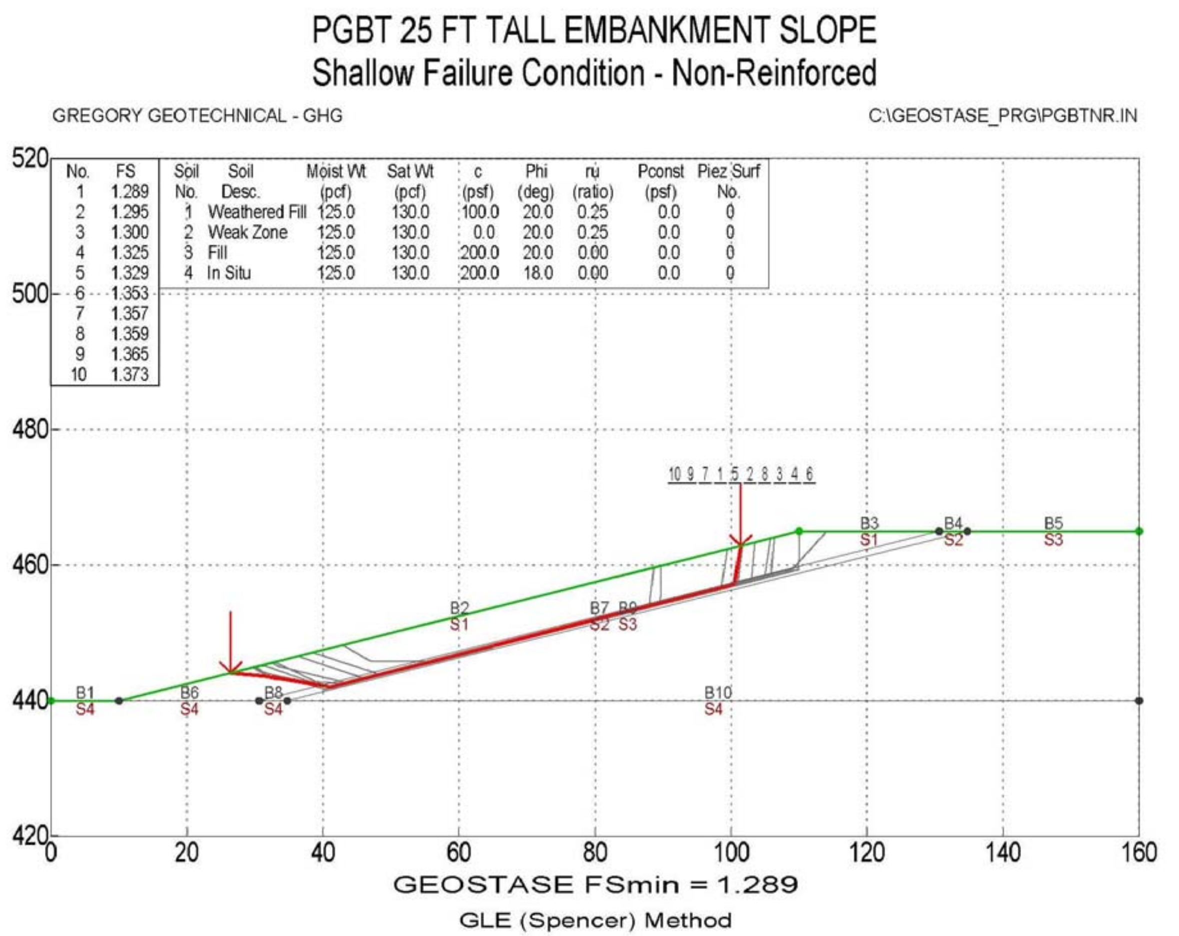

PGBT 25 FT TALL EMBANKMENT SLOPE
Shallow Failure Condition - Non-Reinforced
GREGORY GEOTECHNICAL - GHG
C:\GEOSTASE_PRG\PGBTNR.IN
No. FS
1 1.289
2 1.295
3 1.300
4 1.325
5 1.329
6 1.353
7 1.357
8 1.359
9 1.365
10 1.373
Soil No. Soil Desc. Moist Wt (pcf) Sat Wt (pcf) c (psf) Phi (deg) ru (ratio) Pconst (psf) Piez Surf No.
1 Weathered Fill 125.0 130.0 100.0 20.0 0.25 0.0 0
2 Weak Zone 125.0 130.0 0.0 20.0 0.25 0.0 0
3 Fill 125.0 130.0 200.0 20.0 0.00 0.0 0
4 In Situ 125.0 130.0 200.0 18.0 0.00 0.0 0
10 9 7 1 5 2 8 3 4 6
B1 S4
B6 S4
B8 S4
B2 S1
B7 B9 S2 S3
B10 S4
B3 S1
B4 S2
B5 S3
520
500
480
460
440
420
0
20
40
60
80
100
120
140
160
GEOSTASE FSmin = 1.289
GLE (Spencer) Method

PLATE C.1

*** GEOSTASE ***

** GEOSTASE by Garry H. Gregory, P.E. **

** Current Version 3.10.0000, July 2005 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic , Fiber-Reinforced , Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces .

****

Analysis Date:
Analysis Time:
Analysis By: GREGORY GEOTECHNICAL - GHG
Input Filename: C:\GEOSTASE_PRG\PGBTNR.IN
Output Filename: C:\GEOSTASE_PRG\PGBTNR.OUT
Unit System: English

Plot Filename: C:\GEOSTASE_PRG\PGBTNR.PLT

PROJECT:PGBT 25 FT TALL EMBANKMENT SLOPE

DESCRIPTION:Shallow Failure Condition - Non-Reinforced

BOUNDARY COORDINATES

5 Top Boundaries
10 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.00	440.00	10.00	440.00	4
2	10.00	440.00	110.00	465.00	1
3	110.00	465.00	130.61	465.00	1
4	130.61	465.00	134.73	465.00	2
5	134.73	465.00	160.00	465.00	3
6	10.00	440.00	30.61	440.00	4
7	30.61	440.00	130.61	465.00	2
8	30.61	440.00	34.73	440.00	4
9	34.73	440.00	134.73	465.00	3
10	34.73	440.00	160.00	440.00	4

User Specified Y-Origin = 420.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Number and Description	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1 Weathered Fill	125.0	130.0	100.0	20.0	0.25	0.0	0
2 Weak Zone	125.0	130.0	0.0	20.0	0.25	0.0	0
3 Fill	125.0	130.0	200.0	20.0	0.00	0.0	0
4 In Situ	125.0	130.0	200.0	18.0	0.00	0.0	0

CURVED PHI PARAMETERS
1 Soil Type(s) Assigned Curved Phi Envelope Properties

Soil Type 1:

Specified Critical Effective Normal Stress = 800.00(psf)
Coefficient a = 4.61 Coefficient b = 0.6645

CURVED PHI STRENGTH DATA HAS BEEN SUPPRESSED

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	-80.0	0.00	0.00
2	80.0	100.00	20.00
3	90.0	0.00	0.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of
Sliding Block Is 10.0

Box No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Height (ft)
1	40.00	441.75	65.00	448.00	0.50
2	80.00	452.00	110.00	459.50	0.50

Following Are Displayed The Ten Most Critical Of The Trial
Failure Surfaces Evaluated. They Are
Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *

Selected ki function = Bi-linear

Selected Lambda Coefficient = 1.00

Forces from Reinforcement,Piers/Piles, Soil Nails, and Applied Forces
(if applicable) have been applied to the slice base(s)
on which they intersect.

Specified Tension Crack Water Force Factor = 0.000

Total Number of Trial Surfaces Attempted = 500

Number of Trial Surfaces With Valid FS = 500

Statistical Data On All Valid FS Values:
FS Max = 2.514 FS Min = 1.289 FS Ave = 1.659
Standard Deviation = 0.190 Coefficient of Variation = 11.45 %

((Simplified Janbu FS for Critical Surface = 1.235))

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	26.437	444.109
2	31.396	443.672
3	41.262	442.037
4	100.484	457.257
5	101.415	462.854

*** FOS = 1.289 Theta (ki=1.0) = 17.20 ***
Lambda = 0.310

Individual data on the 6 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	519.9	0.0	130.5	0.	0.	0.0	0.0	0.0
2	8.4	3568.6	0.0	904.3	0.	0.	0.0	0.0	0.0
3	1.5	1028.6	0.0	260.7	0.	0.	0.0	0.0	0.0
4	59.2	41240.4	0.0	10645.2	0.	0.	0.0	0.0	0.0
5	0.0	24.1	0.0	36.7	0.	0.	0.0	0.0	0.0
6	0.9	288.1	0.0	438.8	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.490	445.622
2	33.480	445.058
3	43.209	442.745
4	102.996	457.988
5	103.511	463.378

*** FOS = 1.295 Theta (ki=1.0) = 17.29 ***
Lambda = 0.311

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.642	445.161
2	31.625	445.109
3	41.067	441.812
4	105.875	458.648
5	106.344	464.086

*** FOS = 1.300 Theta (ki=1.0) = 16.78 ***
Lambda = 0.302

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.911	445.728
2	40.920	441.894
3	109.918	459.441
4	110.062	465.000

*** FOS = 1.325 Theta (ki=1.0) = 16.39 ***
Lambda = 0.294

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.317	446.579
2	36.376	446.550
3	45.864	443.392
4	100.967	457.303
5	101.709	462.927

*** FOS = 1.329 Theta (ki=1.0) = 16.85 ***
Lambda = 0.303

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	29.823	444.956
2	30.776	444.369
3	40.407	441.679
4	109.068	459.466
5	114.071	465.000

*** FOS = 1.353 Theta (ki=1.0) = 13.04 ***
Lambda = 0.232

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.572	447.143
2	38.751	447.019
3	48.263	443.932
4	98.514	456.381
5	99.435	462.359

*** FOS = 1.357 Theta (ki=1.0) = 16.82 ***
Lambda = 0.302

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	42.882	448.220
2	46.863	445.888
3	56.862	445.853
4	104.908	458.056

5	105.832	463.958

*** FOS = 1.359 Theta (ki=1.0) = 16.39 ***
Lambda = 0.294

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	29.698	444.924
2	32.488	443.213
3	42.470	442.614
4	89.630	454.590
5	89.730	459.932

*** FOS = 1.365 Theta (ki=1.0) = 16.83 ***
Lambda = 0.303

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.159	445.040
2	31.242	444.142
3	41.036	442.121
4	87.967	453.834
5	88.667	459.667

*** FOS = 1.373 Theta (ki=1.0) = 16.76 ***
Lambda = 0.301

**** END OF GEOSTASE OUTPUT ****

PGBT 25 FT TALL EMBANKMENT SLOPE
Shallow Failure Condition - FRS Veneer

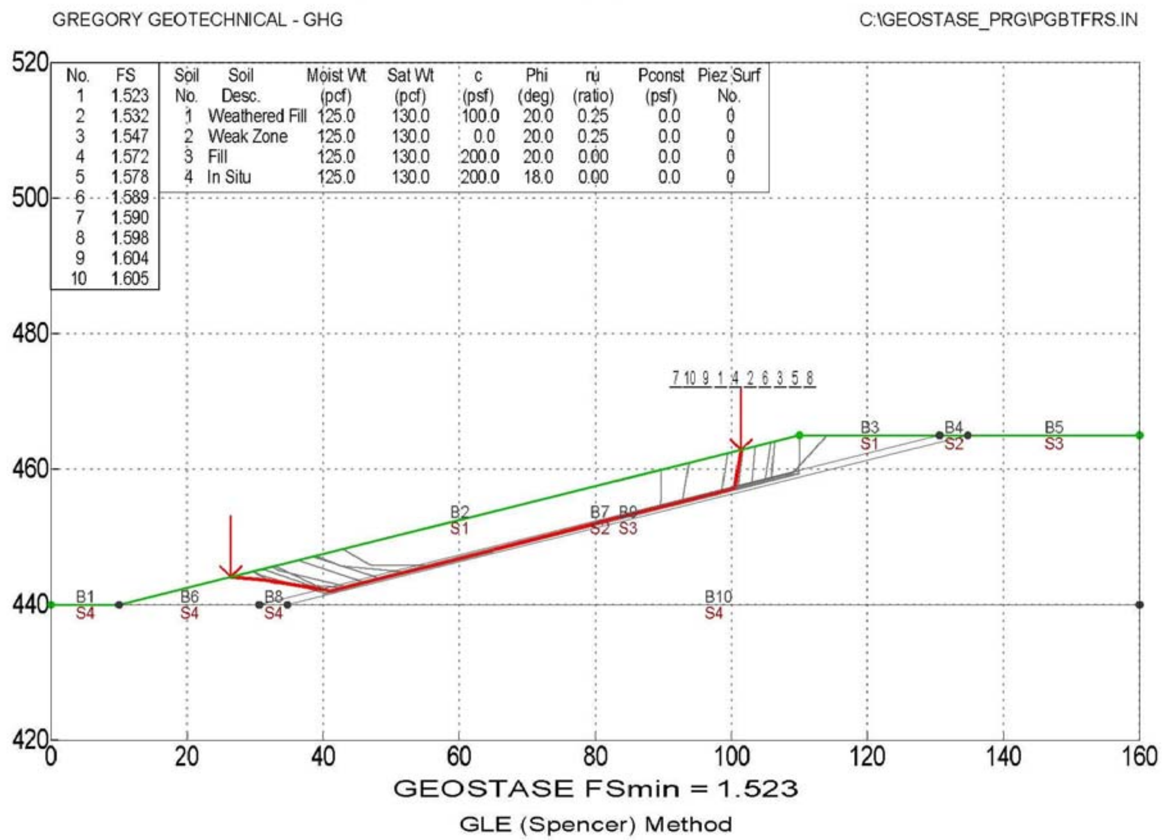


PLATE C.2

*** GEOSTASE ***

** GEOSTASE by Garry H. Gregory, P.E. **

** Current Version 3.10.0000, July 2005 **

****

SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic , Fiber-Reinforced , Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces .

****

Analysis Date:
Analysis Time:
Analysis By: GREGORY GEOTECHNICAL - GHG
Input Filename: C:\GEOSTASE_PRG\PGBTFRS.IN
Output Filename: C:\GEOSTASE_PRG\PGBTFRS.OUT
Unit System: English

Plot Filename: C:\GEOSTASE_PRG\PGBTFRS.PLT

PROJECT:PGBT 25 FT TALL EMBANKMENT SLOPE

DESCRIPTION:Shallow Failure Condition - FRS Veneer

BOUNDARY COORDINATES

5 Top Boundaries
10 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.00	440.00	10.00	440.00	4
2	10.00	440.00	110.00	465.00	1
3	110.00	465.00	130.61	465.00	1
4	130.61	465.00	134.73	465.00	2
5	134.73	465.00	160.00	465.00	3
6	10.00	440.00	30.61	440.00	4
7	30.61	440.00	130.61	465.00	2
8	30.61	440.00	34.73	440.00	4
9	34.73	440.00	134.73	465.00	3
10	34.73	440.00	160.00	440.00	4

User Specified Y-Origin = 420.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Number and Description	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1 Weathered Fill	125.0	130.0	100.0	20.0	0.25	0.0	0
2 Weak Zone	125.0	130.0	0.0	20.0	0.25	0.0	0
3 Fill	125.0	130.0	200.0	20.0	0.00	0.0	0
4 In Situ	125.0	130.0	200.0	18.0	0.00	0.0	0

FIBER-REINFORCED SOIL PROPERTIES
2 Soil Type(s) With Fiber Reinforcement

Soil Type 1:

Fiber Length = 2.65(in) Fiber Width = 0.04700(in)
Fiber Thickness = 0.00149(in) Fiber Equivalent Dia. = 0.00944(in)
Friction Coefficient = 0.50 Cohesion Coefficient = 0.50
Specific Gravity of Fiber = 0.910 Application Rate = 0.222 (pcf)

Soil Type 2:

Fiber Length = 2.65(in) Fiber Width = 0.04700(in)
Fiber Thickness = 0.00149(in) Fiber Equivalent Dia. = 0.00944(in)
Friction Coefficient = 0.50 Cohesion Coefficient = 0.50
Specific Gravity of Fiber = 0.910 Application Rate = 0.222 (pcf)

Fiber-Reinforced Shear-Strength Properties

Soil Type 1: FRS c = 103.69(psf) FRS Phi = 24.25 Deg.

Soil Type 2: FRS c = 0.00(psf) FRS Phi = 24.25 Deg.

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 1 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	-80.0	0.00	0.00
2	80.0	100.00	20.00
3	90.0	0.00	0.00

ANISOTROPIC SOIL NOTES:
(1) An input value of 0.01 for C and/or Phi will cause Aniso C and/or Phi to be ignored in that range.
(2) An input value of 0.02 for Phi will set both Phi and C equal to zero, with no water weight in the tension crack.
(3) An input value of 0.03 for Phi will set both Phi and C equal to zero, with water weight in the tension crack.

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Height (ft)
1	40.00	441.75	65.00	448.00	0.50
2	80.00	452.00	110.00	459.50	0.50

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *

Selected ki function = Bi-linear

Selected Lambda Coefficient = 1.00

Forces from Reinforcement,Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.

Specified Tension Crack Water Force Factor = 0.000

Total Number of Trial Surfaces Attempted = 500

Number of Trial Surfaces With Valid FS = 500

Statistical Data On All Valid FS Values:
FS Max = 2.824 FS Min = 1.523 FS Ave = 1.920
Standard Deviation = 0.206 Coefficient of Variation = 10.75 %

((Simplified Janbu FS for Critical Surface = 1.479))

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	26.437	444.109

2	31.396	443.672
3	41.262	442.037
4	100.484	457.257
5	101.415	462.854

*** FOS = 1.523 Theta (ki=1.0) = 18.03 ***
Lambda = 0.325

Individual data on the 6 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	5.0	519.9	0.0	130.5	0.	0.	0.0	0.0	0.0
2	8.4	3568.6	0.0	904.3	0.	0.	0.0	0.0	0.0
3	1.5	1028.6	0.0	260.7	0.	0.	0.0	0.0	0.0
4	59.2	41240.4	0.0	10645.2	0.	0.	0.0	0.0	0.0
5	0.0	24.1	0.0	36.7	0.	0.	0.0	0.0	0.0
6	0.9	288.1	0.0	438.8	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.490	445.622
2	33.480	445.058
3	43.209	442.745
4	102.996	457.988
5	103.511	463.378

*** FOS = 1.532 Theta (ki=1.0) = 18.13 ***
Lambda = 0.327

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	30.642	445.161
2	31.625	445.109
3	41.067	441.812
4	105.875	458.648
5	106.344	464.086

*** FOS = 1.547 Theta (ki=1.0) = 17.48 ***
Lambda = 0.315

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	36.317	446.579
2	36.376	446.550
3	45.864	443.392
4	100.967	457.303
5	101.709	462.927

*** FOS = 1.572 Theta (ki=1.0) = 17.63 ***
Lambda = 0.318

Failure Surface Specified By 4 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	32.911	445.728
2	40.920	441.894
3	109.918	459.441
4	110.062	465.000

*** FOS = 1.578 Theta (ki=1.0) = 17.06 ***
Lambda = 0.307

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	42.882	448.220
2	46.863	445.888
3	56.862	445.853
4	104.908	458.056
5	105.832	463.958

*** FOS = 1.589 Theta (ki=1.0) = 17.15 ***
Lambda = 0.309

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	29.698	444.924
2	32.488	443.213
3	42.470	442.614
4	89.630	454.590
5	89.730	459.932

*** FOS = 1.590 Theta (ki=1.0) = 17.69 ***

Lambda = 0.319

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	29.823	444.956
2	30.776	444.369
3	40.407	441.679
4	109.068	459.466
5	114.071	465.000

*** FOS = 1.598 Theta (ki=1.0) = 13.79 ***
Lambda = 0.246

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.572	447.143
2	38.751	447.019
3	48.263	443.932
4	98.514	456.381
5	99.435	462.359

*** FOS = 1.604 Theta (ki=1.0) = 17.61 ***
Lambda = 0.317

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	38.912	447.228
2	42.591	445.733
3	52.547	444.797
4	92.757	455.062
5	93.766	460.942

*** FOS = 1.605 Theta (ki=1.0) = 17.46 ***
Lambda = 0.315

**** END OF GEOSTASE OUTPUT ****

Lakeridge Pkwy Slope
Long-Term Repaired Condition - FRS

GREGORY GEOTECHNICAL - GHG

C:\GEOSTASE_PRG\lakeridge-NR.in

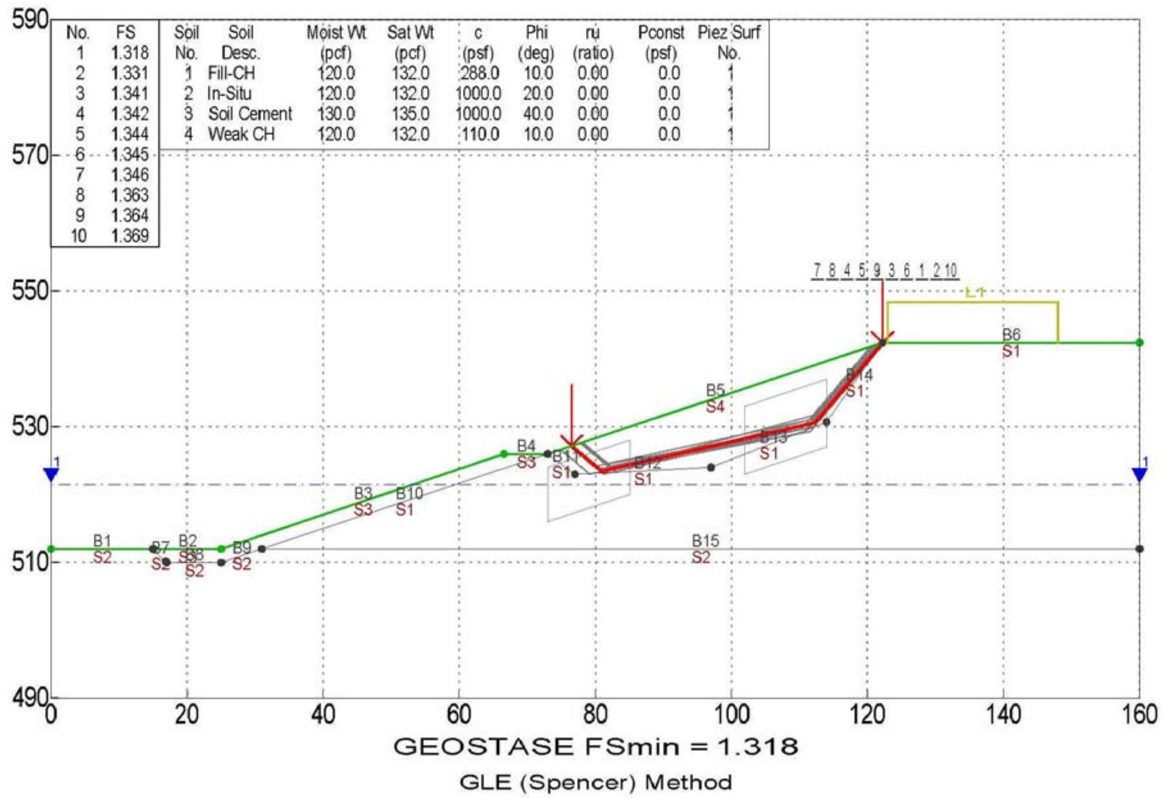


PLATE C.3

*** GEOSTASE ***

** GEOSTASE by Garry H. Gregory, P.E. **

** Current Version 3.10.0000, July 2005 **

****

SLOPE STABILITY ANALYSIS SYSTEM

Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic , Fiber-Reinforced , Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces .

****

Analysis Date:
Analysis Time:
Analysis By: GREGORY GEOTECHNICAL - GHG
Input Filename: C:\GEOSTASE_PRG\lakeridge-NR.in
Output Filename: C:\GEOSTASE_PRG\lakeridge-NR.OUT
Unit System: English

Plot Filename: C:\GEOSTASE_PRG\lakeridge-NR.PLT

PROJECT:Lakeridge Pkwy Slope

DESCRIPTION:Long-Term Repaired Condition - FRS

BOUNDARY COORDINATES

6 Top Boundaries
15 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.00	512.00	15.00	512.00	2
2	15.00	512.00	25.00	512.00	3
3	25.00	512.00	66.60	526.00	3
4	66.60	526.00	73.00	526.00	3
5	73.00	526.00	122.20	542.40	4
6	122.20	542.40	160.00	542.40	1
7	15.00	512.00	17.00	510.00	2
8	17.00	510.00	25.00	510.00	2
9	25.00	510.00	31.00	512.00	2
10	31.00	512.00	73.00	526.00	1
11	73.00	526.00	77.00	523.00	1
12	77.00	523.00	97.00	524.00	1
13	97.00	524.00	114.00	530.70	1
14	114.00	530.70	122.20	542.40	1
15	31.00	512.00	160.00	512.00	2

User Specified Y-Origin = 490.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Number and Description	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1 Fill-CH	120.0	132.0	288.0	10.0	0.00	0.0	1
2 In-Situ	120.0	132.0	1000.0	20.0	0.00	0.0	1
3 Soil Cement	130.0	135.0	1000.0	40.0	0.00	0.0	1
4 Weak CH	120.0	132.0	110.0	10.0	0.00	0.0	1

CURVED PHI PARAMETERS
1 Soil Type(s) Assigned Curved Phi Envelope Properties

Soil Type 4:

Specified Critical Effective Normal Stress = 3000.00(psf)
Coefficient a = 4.70 Coefficient b = 0.6135

CURVED PHI STRENGTH DATA HAS BEEN SUPPRESSED

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	521.50
2	160.00	521.50

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X - 1 (ft)	X - 2 (ft)	Intensity (psf)	Deflection (deg)
1	123.00	148.00	250.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.
Specified Peak Ground Acceleration Coefficient (A) = 0.070(g)

Specified Horizontal Earthquake Coefficient (kh) = 0.030(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

The Active And Passive Portions Of The Sliding Surfaces Are Generated According To The Rankine Theory.

500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Height (ft)
1	73.00	520.00	85.00	524.00	8.00
2	102.00	528.00	114.00	532.00	10.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *

Selected ki function = Bi-linear

Selected Lambda Coefficient = 1.00

Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.

Specified Tension Crack Water Force Factor = 1.000

Total Number of Trial Surfaces Attempted = 500

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 3

Number of Trial Surfaces With Valid FS = 497
Percentage of Trial Surfaces With Non-Valid FS Solutions

of the Total Attempted = 0.6 %

Statistical Data On All Valid FS Values:
FS Max = 3.358 FS Min = 1.318 FS Ave = 1.957
Standard Deviation = 0.441 Coefficient of Variation = 22.52 %

((Simplified Janbu FS for Critical Surface = 1.241))

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.513	527.171
2	80.948	523.449
3	112.332	530.623
4	118.760	538.284
5	122.128	542.297
6	122.214	542.400

*** FOS = 1.318 Theta (ki=1.0) = 13.07 ***
Lambda = 0.232

Individual data on the 6 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.4	1383.6	0.0	0.0	0.	0.	41.5	0.0	0.0
2	31.4	25774.5	0.0	0.0	0.	0.	773.2	0.0	0.0
3	6.4	4418.8	0.0	0.0	0.	0.	132.6	0.0	0.0
4	3.4	616.0	0.0	0.0	0.	0.	18.5	0.0	0.0
5	0.1	0.4	0.0	0.0	0.	0.	0.0	0.0	0.0
6	0.0	0.0	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.388	527.129
2	79.916	524.168
3	112.051	529.983
4	118.479	537.643
5	120.829	540.443
6	122.471	542.400

*** FOS = 1.331 Theta (ki=1.0) = 11.65 ***
Lambda = 0.206

Failure Surface Specified By 6 Coordinate Points

Point X-Surf Y-Surf

No.	(ft)	(ft)
1	76.271	527.090
2	80.233	523.766
3	110.774	529.156
4	111.115	529.563
5	117.543	537.223
6	121.765	542.255

*** FOS = 1.341 Theta (ki=1.0) = 14.02 ***
Lambda = 0.250

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.884	527.628
2	82.026	524.153
3	111.535	530.475
4	117.963	538.135
5	121.286	542.095

*** FOS = 1.342 Theta (ki=1.0) = 13.46 ***
Lambda = 0.239

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.396	527.132
2	81.077	523.204
3	81.264	523.047
4	111.055	529.797
5	117.483	537.457
6	121.409	542.136

*** FOS = 1.344 Theta (ki=1.0) = 13.30 ***
Lambda = 0.236

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	78.001	527.667
2	81.709	524.555
3	112.813	531.301
4	119.241	538.961
5	122.097	542.366

*** FOS = 1.345 Theta (ki=1.0) = 13.79 ***

Lambda = 0.245

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.767	527.589
2	82.342	523.750
3	110.258	529.649
4	116.686	537.309
5	120.475	541.825

*** FOS = 1.346 Theta (ki=1.0) = 13.13 ***
Lambda = 0.233

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	78.009	527.670
2	83.057	523.434
3	111.817	531.115
4	118.245	538.776
5	120.931	541.977

*** FOS = 1.363 Theta (ki=1.0) = 12.78 ***
Lambda = 0.227

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	78.126	527.709
2	82.740	523.836
3	113.094	531.942
4	119.522	539.602
5	121.741	542.247

*** FOS = 1.364 Theta (ki=1.0) = 13.16 ***
Lambda = 0.234

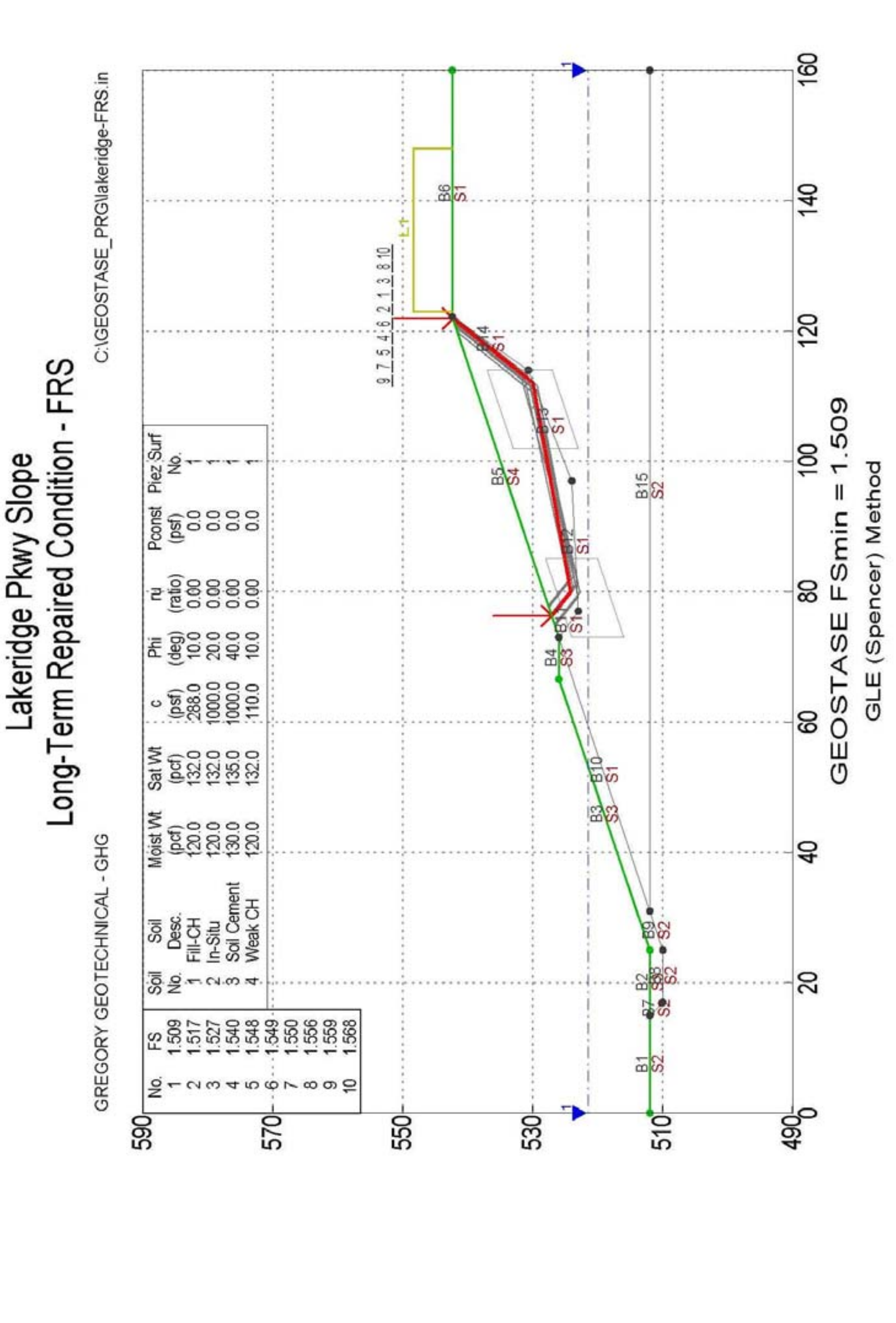
Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	74.899	526.633
2	79.104	523.105
3	79.155	523.063
4	111.571	529.305
5	112.119	529.959
6	118.547	537.619
7	122.559	542.400

*** FOS = 1.369 Theta (ki=1.0) = 11.45 ***
Lambda = 0.203

**** END OF GEOSTASE OUTPUT ****



Lakeridge Pkwy Slope
Long-Term Repaired Condition - FRS
GREGORY GEOTECHNICAL - GHG
C:\GEOSTASE_PRG\lakeridge-FRS.in
No.	FS
1	1.509
2	1.517
3	1.527
4	1.540
5	1.548
6	1.549
7	1.550
8	1.556
9	1.559
10	1.568

Soil No.	Soil Desc.	Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.
1	Fill-CH	120.0	132.0	288.0	10.0	0.00	0.0	1
2	In-Situ	120.0	132.0	1000.0	20.0	0.00	0.0	1
3	Soil Cement	130.0	135.0	1000.0	40.0	0.00	0.0	1
4	Weak CH	120.0	132.0	110.0	10.0	0.00	0.0	1

GEOSTASE FSmin = 1.509
GLE (Spencer) Method

PLATE C.4

*** GEOSTASE ***

** GEOSTASE by Garry H. Gregory, P.E. **

** Current Version 3.10.0000, July 2005 **

****
SLOPE STABILITY ANALYSIS SYSTEM
Modified Bishop, Simplified Janbu, or GLE Method of Slices.
(Includes Spencer & Morgenstern-Price Type Analysis)
Including Pier/Pile, Reinforcement, Nail, Tieback,
Nonlinear Undrained Shear Strength, Curved Phi Envelope,
Anisotropic , Fiber-Reinforced , Boundary Loads, Water
Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces .

****

Analysis Date:
Analysis Time:
Analysis By: GREGORY GEOTECHNICAL - GHG
Input Filename: C:\GEOSTASE_PRG\lakeridge-FRS.in
Output Filename: C:\GEOSTASE_PRG\lakeridge-FRS.OUT
Unit System: English

Plot Filename: C:\GEOSTASE_PRG\lakeridge-FRS.PLT

PROJECT:Lakeridge Pkwy Slope

DESCRIPTION:Long-Term Repaired Condition - FRS

BOUNDARY COORDINATES

6 Top Boundaries
15 Total Boundaries

Boundary No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Soil Type Below Bnd
1	0.00	512.00	15.00	512.00	2
2	15.00	512.00	25.00	512.00	3
3	25.00	512.00	66.60	526.00	3
4	66.60	526.00	73.00	526.00	3
5	73.00	526.00	122.20	542.40	4
6	122.20	542.40	160.00	542.40	1
7	15.00	512.00	17.00	510.00	2
8	17.00	510.00	25.00	510.00	2
9	25.00	510.00	31.00	512.00	2
10	31.00	512.00	73.00	526.00	1
11	73.00	526.00	77.00	523.00	1
12	77.00	523.00	97.00	524.00	1
13	97.00	524.00	114.00	530.70	1
14	114.00	530.70	122.20	542.40	1
15	31.00	512.00	160.00	512.00	2

User Specified Y-Origin = 490.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS

4 Type(s) of Soil

Soil Number and Description	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1 Fill-CH	120.0	132.0	288.0	10.0	0.00	0.0	1
2 In-Situ	120.0	132.0	1000.0	20.0	0.00	0.0	1
3 Soil Cement	130.0	135.0	1000.0	40.0	0.00	0.0	1
4 Weak CH	120.0	132.0	110.0	10.0	0.00	0.0	1

FIBER-REINFORCED SOIL PROPERTIES
1 Soil Type(s) With Fiber Reinforcement

Soil Type 4:

Fiber Length = 2.65(in) Fiber Width = 0.04700(in)
Fiber Thickness = 0.00149(in) Fiber Equivalent Dia. = 0.00944(in)
Friction Coefficient = 0.50 Cohesion Coefficient = 0.50
Specific Gravity of Fiber = 0.910 Application Rate = 0.250 (pcf)

Fiber-Reinforced Shear-Strength Properties

Soil Type 4: FRS c = 113.25(psf) FRS Phi = 12.71 Deg.

CURVED PHI PARAMETERS
1 Soil Type(s) Assigned Curved Phi Envelope Properties

Soil Type 4:

Specified Critical Effective Normal Stress = 3000.00(psf)
Coefficient a = 4.73 Coefficient b = 0.6392

CURVED PHI STRENGTH DATA HAS BEEN SUPPRESSED

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 2 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)

1	0.00	521.50
2	160.00	521.50

BOUNDARY LOAD(S)

1 Load(s) Specified

Load No.	X - 1 (ft)	X - 2 (ft)	Intensity (psf)	Deflection (deg)
1	123.00	148.00	250.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.070(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.030(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

Janbus Empirical Coef is being used for the case of c & phi both > 0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

The Active And Passive Portions Of The Sliding Surfaces Are Generated According To The Rankine Theory.

500 Trial Surfaces Have Been Generated.

2 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X - 1 (ft)	Y - 1 (ft)	X - 2 (ft)	Y - 2 (ft)	Height (ft)
1	73.00	520.00	85.00	524.00	8.00
2	102.00	528.00	114.00	532.00	10.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *

Selected ki function = Bi-linear

Selected Lambda Coefficient = 1.00

Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces (if applicable) have been applied to the slice base(s) on which they intersect.

Specified Tension Crack Water Force Factor =   1.000

Total Number of Trial Surfaces Attempted =   500

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS =   3

Number of Trial Surfaces With Valid FS = 497

Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Attempted =   0.6 %

Statistical Data On All Valid FS Values:
FS Max =   3.455   FS Min =   1.509   FS Ave =   2.097
Standard Deviation =   0.412   Coefficient of Variation =   19.63 %

((Simplified Janbu FS for Critical Surface = 1.428))

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.264	527.088
2	79.916	524.168
3	112.051	529.983
4	118.296	537.793
5	121.900	542.300

*** FOS =   1.509   Theta (ki=1.0) =   14.37 ***
Lambda =   0.256

Individual data on the   4 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	3.7	906.5	0.0	0.0	0.	0.	27.2	0.0	0.0
2	32.1	25396.3	0.0	0.0	0.	0.	761.9	0.0	0.0
3	6.2	4623.9	0.0	0.0	0.	0.	138.7	0.0	0.0
4	3.6	714.8	0.0	0.0	0.	0.	21.4	0.0	0.0

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.358	527.119
2	80.948	523.449
3	112.332	530.623
4	118.577	538.433
5	121.585	542.195

*** FOS = 1.517 Theta (ki=1.0) = 13.67 ***
Lambda = 0.243

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	74.753	526.584
2	79.104	523.105
3	79.155	523.063
4	111.571	529.305
5	112.119	529.959
6	118.364	537.769
7	122.019	542.340

*** FOS = 1.527 Theta (ki=1.0) = 14.12 ***
Lambda = 0.252

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.133	527.044
2	80.233	523.766
3	110.774	529.156
4	111.115	529.563
5	117.360	537.373
6	121.081	542.027

*** FOS = 1.540 Theta (ki=1.0) = 14.15 ***
Lambda = 0.252

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.233	527.078
2	81.077	523.204
3	81.264	523.047
4	111.055	529.797

5	117.300	537.607
6	120.744	541.915

*** FOS = 1.548 Theta (ki=1.0) = 13.48 ***
Lambda = 0.240

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.871	527.624
2	81.709	524.555
3	112.813	531.301
4	119.058	539.111
5	121.501	542.167

*** FOS = 1.549 Theta (ki=1.0) = 14.11 ***
Lambda = 0.251

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.739	527.580
2	82.026	524.153
3	111.535	530.475
4	117.780	538.285
5	120.660	541.887

*** FOS = 1.550 Theta (ki=1.0) = 13.72 ***
Lambda = 0.244

Failure Surface Specified By 6 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	76.587	527.196
2	81.550	523.227
3	81.662	523.133
4	113.891	532.090
5	120.136	539.900
6	122.111	542.370

*** FOS = 1.556 Theta (ki=1.0) = 14.06 ***
Lambda = 0.250

Failure Surface Specified By 5 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	77.608	527.536
2	82.342	523.750
3	110.258	529.649
4	116.503	537.459
5	119.819	541.607

*** FOS = 1.559 Theta (ki=1.0) = 13.37 ***
Lambda = 0.238

Failure Surface Specified By 7 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	74.992	526.664
2	79.423	523.121
3	79.869	522.746
4	113.129	530.772
5	119.374	538.582
6	120.588	540.101
7	122.427	542.400

*** FOS = 1.568 Theta (ki=1.0) = 12.30 ***
Lambda = 0.218

**** END OF GEOSTASE OUTPUT ****

VITA

Garry Haden Gregory, P. E.

Candidate for the Degree of

Doctor of Philosophy

Dissertation: SHEAR STRENGTH, CREEP, AND STABILITY OF FIBER REINFORCED SOIL SLOPES

Major Field: Civil Engineering

Biographical:

Born in Gorman, Texas on September 4, 1944.

Education:

Graduated from Rapid City High School, Rapid City, S. D. in May 1963. Received a Bachelor of Science Degree from Oklahoma City University, Oklahoma City, Oklahoma in December 1991. Received a Master of Science Degree in Civil Engineering from South Dakota School of Mines and Technology in December 1993. Completed the requirements for the Doctor of Philosophy Degree with a Major in Civil Engineering at Oklahoma State University in Stillwater, Oklahoma in May, 2006.

Experience:

More than 35 years of civil engineering and construction management experience on major projects. Licensed as a professional engineer in 11 states, including Oklahoma. Principal and owner of Gregory Geotechnical, a specialty geotechnical engineering firm performing services for clients on a national basis since 1998.

Professional Memberships:

American Society of Civil Engineers (ASCE), member. Member of the ASCE Geo-Institute Embankment, Dams, and Slopes Committee. Member of the U. S. Society on Large Dams, and member of the Materials for Embankment Dams Committee. Affiliate Member of the Association of Engineering Geologists.

Name: Garry Haden Gregory Date of Degree: May, 2006

Institution: Oklahoma State University Location: Stillwater, Oklahoma

Title of Study: SHEAR STRENGTH, CREEP, AND STABILITY OF FIBER REINFORCED SOIL SLOPES

Pages in Study: 225 Candidate for the Degree of Doctor of Philosophy

Major Field: Civil Engineering

Fiber-reinforced soil (FRS) has been used successfully on more than 50 embankment slopes in the United States in recent years. The geosynthetic fiber reinforcement has consisted predominantly of 25 to 50 mm long polypropylene fibers. These fibers, when mixed into the soil, significantly increase the apparent shear strength of the entire soil mass. This study includes an extensive laboratory testing program to characterize shear strength of clay and silty sand soils reinforced with synthetic fibers as compared to non-reinforced soil. A series of creep tests were also performed to obtain an initial indication of the resistance of FRS to creep failure. The creep test results indicate an increased resistance to creep of FRS compared to non-reinforced soil. A theoretical conceptual model is presented which can be used to mathematically calculate the shear strength of the soil when reinforced with fibers, referred to as the FRS shear strength. The model includes a unique effective normal stress formulation based upon 3-dimensional random orientation of the fibers under geostatic stress conditions in a half-space continuum (soil mass). The model utilizes an "effective aspect ratio," $\boldsymbol{a_{re}}$, which is different than the conventional aspect ratio based upon the actual fiber length-equivalent diameter ratio. The input to the model includes the fiber volume ratio (ratio of fiber volume to total volume of a unit mass of FRS), unique effective stress variable, effective aspect ratio, frictional and adhesion interaction coefficients, and the non-reinforced soil shear-strength parameters ϕ and c. The model was calibrated and validated based upon comparison of calculated results and actual shear strength test results performed during this study, and also compared to other available test results. Case histories of two major FRS projects are presented in the study.

Advisor's Approval ______________________________________

Dr. Donald R. Snethen, Ph.D., P. E.