Technical Report Documentation Page

1. Report No. FHWA/TX-06/5-1874-01-1	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Evaluation of Soil Shear Strengths for Slope and Retaining Wall Stability Analyses with Emphasis on High Plasticity Clays		5. Report Date August 2005
		6. Performing Organization Code
7. Author(s) Stephen G. Wright		8. Performing Organization Report No. 5-1874-01-1
9. Performing Organization Name and Address Center for Transportation Research The University of Texas at Austin 3208 Red River, Suite 200 Austin, TX 78705-2650		10. Work Unit No. (TRAIS)
		11. Contract or Grant No. 5-1874-01
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Implementation Office P.O. Box 5080 Austin, TX 78763-5080		13. Type of Report and Period Covered Technical Report, 04/18/05-08/31/05
		14. Sponsoring Agency Code

15. Supplementary Notes

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration; Project Title: Implementation of Research Results Regarding Shear Strength and Stability of Embankments

16. Abstract

A number of TxDOT-sponsored studies and research projects have been conducted over the years regarding shear strength and slope stability issues of embankments. These projects span approximately 15 years, and each developed relationships and theories for soil strength relationships in different areas of the state. In addition, some findings from earlier projects have been refined or disputed in later studies. Data from these studies are spread throughout numerous reports, and in some cases unpublished, making the data are difficult to utilize. This implementation project was undertaken to review the data and develop a single, unified data set and guidelines that can be utilized in refining the Geotechnical Manual and presented to the geotechnical community in other publications.

In this report important fundamentals pertaining to the shear strength of soils are reviewed and guidelines for determining appropriate values of soil shear strength parameters are presented for both undrained (short-term) and drained (long-term) stability conditions. Particular attention is given to the long-term strength properties of compacted high PI clay fills used for embankment construction.

17. Key Words	18. Distribution Statement
Soil Shear Strengths, Retaining Wall Stability, High Plasticity Clays	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161; www.ntis.gov.

19. Security Classif. (of report)	20. Security Classif. (of this page)	21. No. of pages	22. Price
Unclassified	Unclassified	100	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

Evaluation of Soil Shear Strengths for Slope and Retaining Wall Stability Analyses with Emphasis on High Plasticity Clays

Stephen G. Wright

CTR Technical Report:	5-1874-01-1
Report Date:	August 2005
Project:	5-1874-01
Project Title:	Implementation of Research Results Regarding Shear Strength and Stability of Embankments
Sponsoring Agency:	Texas Department of Transportation
Performing Agency:	Center for Transportation Research at The University of Texas at Austin

Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.

Center for Transportation Research
The University of Texas at Austin
3208 Red River
Austin, TX 78705

www.utexas.edu/research/ctr

Disclaimers

Author's Disclaimer: The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Federal Highway Administration or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation.

Patent Disclaimer: There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine manufacture, design or composition of matter, or any new useful improvement thereof, or any variety of plant, which is or may be patentable under the patent laws of the United States of America or any foreign country.

Engineering Disclaimer

NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES.

Project Engineer: Stephen G. Wright
Professional Engineer License State and Number: 49007
P. E. Designation: Stephen G. Wright

Acknowledgments

The writer expresses his appreciation to TxDOT for their support of this implementation study. He is particularly grateful for the vision and encouragement of Mr. Mark McClelland of TxDOT whose efforts made this work possible. Mr. McClelland has been involved with many of the prior research studies that this work is derived from. He continues to show leadership and support for advancing the practice of geotechnical engineering within TxDOT as well as within the transportation industry nationwide.

The writer also wishes top acknowledge the work of many excellent students who participated in various TxDOT research projects on which this current study was based. This includes, but is not limited to Mr. Tim G. Abrams, Mr. Alexander W. Gourlay, Mr. Roger Green, Dr. Mohamad Kayyal, Mr. Ed O'Malley, Ms. Laura E. Rogers, Dr. Adam A. Saleh and Mr. Peter A. Stauffer.

Products

Portions of Product 1 related to previous research studies for TxDOT are contained in Chapter 3. Product 1, including supplemental data from the literature, is also contained in and the basis for Chapters 4 and 5 of this report.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter 1 – Introduction

Introduction

During the past approximately 30 years, a number of research projects have been conducted for the Texas Department of Transportation (TxDOT) by the Center for Transportation Research at The University of Texas at Austin (CTR) to address problems of slope stability. An important part of this research has been devoted to characterizing the shear strength of Texas' soils as it pertains to slope stability. Most of the slope failures involved soils with high plasticity indices—generally classified as CH materials by the Unified Soil Classification System. Most of the slope failures also involved relatively shallow slides, typically extending to depths of ten feet or less and, thus, the stresses were relatively low. A significant understanding of these materials and their shear strength values, including particularly the shear strengths at low stresses comparable to those along observed slip surfaces, has been developed by the research. This information is contained in numerous reports and while the information exists, it is sometimes difficult for a design engineer to locate and synthesize the necessary details. In some cases conclusions and recommendations from earlier work were revised and updated as additional data and information became available. Other research reported in the technical literature can also be used to supplement the research performed for TxDOT and to establish guidelines for design of new and repaired slopes. The purpose of this report is to review the previous research conducted for TxDOT and combine pertinent data with results from the technical literature to develop guidelines for selection of shear strengths for slope stability.

The primary emphasis of the work described in this report is on the shear strength of clays with high plasticity. The soils are generally classified as CH by the Unified Soil Classification System and have liquid limits in excess of fifty.

In Chapter 2 important fundamentals of shear strength with particular emphasis on shear strength for slope stability and retaining structure design are reviewed. This coverage should be helpful to designers and includes important details that are either not included or receive only minimal coverage in TxDOT's current Geotechnical Manual (Texas Department of Transportation, 2000).

Various research projects related to slope stability and soil shear strength that have been conducted for TxDOT by the University of Texas' Center for Transportation Research (CTR) are reviewed in Chapter 3. Important findings and results from each of these projects are summarized and discussed.

Different tests and characterizations of shear strength are required for short-term stability, where clays do not have ample time to drain, and for long-term stability, where it is assumed that any pore water pressures in excess[1] of long-term, steady-state seepage values have dissipated. Appropriate shear strengths for short-term stability are discussed in Chapter 4 along with guidelines for estimating and measuring shear strength. Corresponding guidelines and a discussion of long-term shear strengths are presented in Chapter 5.

A brief summary of this report along with recommendations for future work is presented in Chapter 6; however, most of the important guidelines for shear strength are presented earlier in Chapters 4 and 5.

---

[1] In this case "excess" means pore water pressures that are either greater than or less than the long-term, steady-state seepage values.

Chapter 2 – Background and Fundamentals

Introduction

Determination of the shear strength to be used in an analysis of stability of walls and slopes requires definition of the appropriate loading conditions followed by determination of the appropriate shear strength parameters. Loading conditions include the temporary or permanent nature of the structure. Loading conditions also must take into account if either the soil will drain freely during loading (construction) or a number of years will be required for the soil to expand or compress to its final equilibrium state. Once the appropriate loading conditions have been established, an appropriate technique should be selected for determining the relevant shear strength properties. This report will focus primarily on the shear strengths determined in laboratory tests, supplemented with various correlations between shear strength and soil index properties. Various in-situ field measurement techniques may also be used to supplement the values determined in the laboratory.

Short-Term and Long-Term Stability

Proper evaluation of the stability of many earth structures requires consideration of both *short-term* and *long-term* stability. Short-term stability applies to conditions during and immediately after construction and is associated with conditions where one or more of the soils involved are of sufficiently low permeability that no significant movement of moisture into or out of the soil occurs during construction. In practice no drainage is assumed to occur for the short-term condition, i.e., the loading is said to be *undrained*. The short-term stability condition exists for almost all clays and some silts, and is seldom applicable for coarse-grained soils except for dynamic and sudden impact loading. Long-term stability is used in reference to conditions where the soil has had sufficient time to fully consolidate or swell and reach a final equilibrated state. The long-term condition may be reached almost immediately, i.e., during construction, in most coarse-grained soils, but may require years to be attained in fine-grained soils. Eventually all soils will reach the long-term state. The long-term loading condition is also termed the *drained* loading condition.

The shear strength of soils can be significantly different depending on whether the soil is allowed to expand and/or compress under the applied loads. If the soil tends to expand (swell), the shear strength after expansion will be less than the shear strength before the soil has had an opportunity to do so. For soils that may experience both the short-term and long-term conditions, i.e., clays which may not drain initially, but will do so over time, both the short-term and long-term conditions must be evaluated and appropriate strengths determined for each. If the soil compresses over time as it passes from the short-term to long-term condition, the short-term strength will most likely be the most critical (lowest) and will govern the design. On the other hand, if the soil expands over time, the long-term condition will often govern and the shear strength for the long-term condition is of greatest interest. In many instances, such as an embankment constructed of highly plastic[2] clay, the soil near the surface may expand and get weaker with time while the soil at greater depths may consolidate and become stronger. In such cases it may not be immediately obvious whether the short-term or long-term condition is the critical condition and both short-term and long-term conditions must be evaluated.

Total and Effective Stress Representations of Shear Strength

The shear strength of soils is usually expressed on a Mohr-Coulomb diagram similar to the ones shown in Figure 2.1. The shear strength may be plotted and expressed on such diagrams in terms of either the total normal stress, σ (Figure 2.1a), or the effective normal stress, σ' (Figure 2.1b). The effective normal stress is defined as follows:

$$\sigma' = \sigma - u \tag{2.1}$$

where u is the pore water pressure. When the shear strength is expressed in terms of total stresses it is expressed by an equation of the form,

$$s = c + \sigma \tan \phi \tag{2.2}$$

where c and ϕ are the cohesion and friction angle, respectively, for strengths expressed using total stresses (Figure 2.1a). When the shear strength is expressed in terms of effective stresses, it is expressed by an equation of the form,

$$s = c' + \sigma' \tan \phi' \tag{2.3}$$

where c' and ϕ' are the cohesion and friction angle, respectively, for strengths expressed using effective stresses (Figure 2.1b). The decision to express the shear strength in terms of either total or effective normal stresses is determined by the loading conditions and type of test used to

[2] Highly plastic clay is used in this report and geotechnical practice in general to indicate clay soils that have a liquid limit of 50 or greater.

define the shear strength; the decision is not an arbitrary one. Loading conditions and the types of shear strength tests are discussed in the sections that follow.

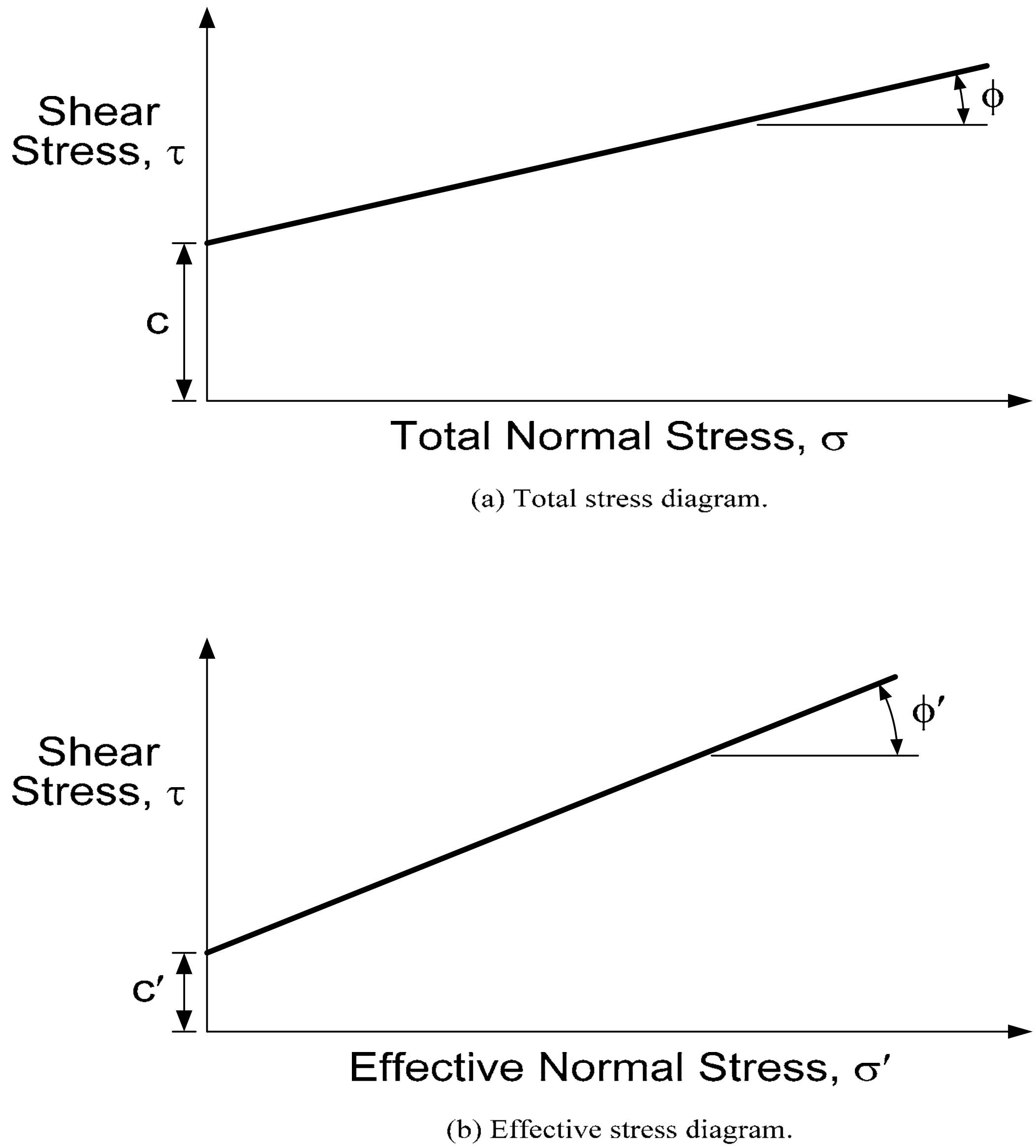


Figure 2.1 Mohr-Coulomb failure envelopes for total and effective stresses.

The distinction between shear strengths expressed in terms of total stresses and effective stresses is an important one. The strength parameters, c and ϕ, are usually very different from the strength parameters, c' and ϕ', and the two sets of strength parameters generally cannot be related to one another in any simple way.

When shear strengths are used in stability calculations for slopes and walls, it is important to distinguish between the two types of strength parameters—those expressed in terms of total stresses and those expressed in terms of effective stresses. When the shear strengths are expressed in terms of effective stresses, the stability calculations must be performed using effective stresses, i.e., the pore water pressures must be determined and included in the computations. Conversely, when the shear strengths are expressed in terms of total stresses, pore water pressures must not be used in the stability calculations. Use of pore water pressures in stability calculations when the shear strength is expressed in terms of total stresses will produce erroneous and meaningless results.

Laboratory Tests

Several types of laboratory tests exist for measuring the shear strength of soil. They differ in the type of apparatus and the procedures used for applying loads to specimens. Each type of apparatus and each type of loading condition are discussed in the sections that follow.

Apparatus

Three different types of laboratory test apparatus are commonly used to measure the shear strength for soils, and clays in particular: (1) direct shear, (2) unconfined compression, and (3) triaxial compression.

Direct Shear Test

In the direct shear test, specimens are sheared in a metal box that is split into two halves (Figure 2.2). There is a small gap between the upper and lower halves of the box and a horizontal shear plane forms through this gap. Vertical loads are applied to a plate placed on the top of the specimen and fitted loosely inside the shear box. There is a small gap between the top loading plate and the sides of the shear box. Because of the gaps between the two halves of the box and between the top (and bottom) loading plates and the sides of the metal shear box, it is impossible to prevent drainage of water into or out of a specimen in the direct shear tests. The only meaningful test that can be performed in the direct shear device is one where the specimen is allowed to fully drain. Thus, the direct shear test is only applicable to measuring shear strength under *drained* (long-term) conditions. The direct shear apparatus should not be used to measure undrained shear strengths.

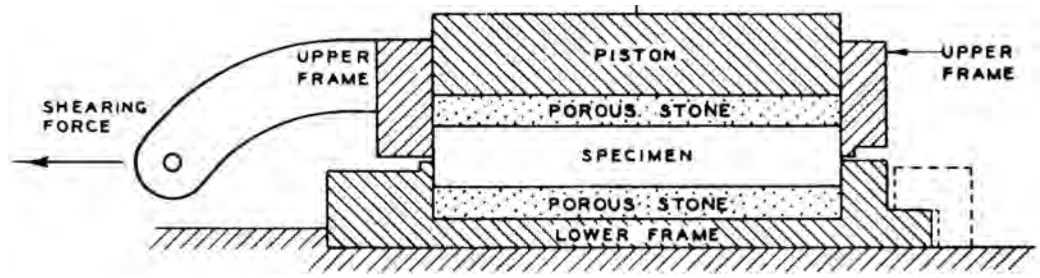


Figure 2.2 Schematic diagram of direct shear box (from Corps of Engineers, 1970).

In order to measure the drained shear strength properly in the direct shear apparatus, the rates of loading must be chosen so that they are slow enough to allow the specimen to fully drain. After the vertical (normal) load is applied, a sufficient period of time must be allowed for the soil to fully consolidate before the specimen is sheared. Similarly, when the specimen is sheared the shear load must be applied at a slow enough rate to allow any excess pore water pressures generated during shear to fully dissipate. If the shear load is applied too fast and the soil tends to expand (dilate) during shear, the shear strength will be incorrectly overestimated. Procedures for determining the proper loading rates are described by ASTM (2003).

Because specimens are sheared in the direct shear test at a rate that allows the soil to drain fully, there are no excess pore water pressures. That is, the pore water pressures equal those in the water that surrounds the specimen. Thus, the effective stress can be calculated and strengths can be plotted in terms of effective stresses as shown in Figure 2.3. The corresponding shear strength parameters determined from the direct shear tests are the effective stress cohesion and friction angle, c' and ϕ', respectively.

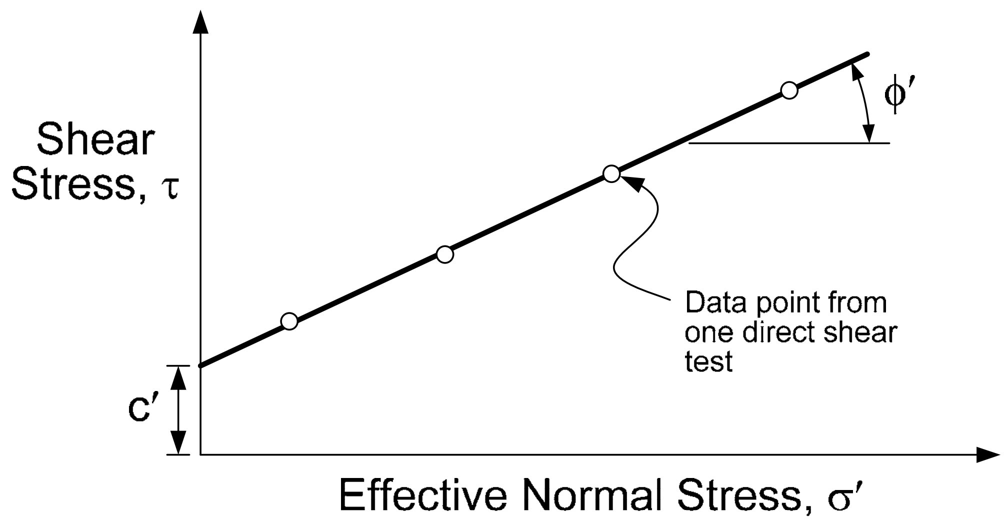


Figure 2.3 Mohr-Coulomb shear strength envelope for direct shear test plotted in terms of effective stresses.

Unconfined Compression Test

The unconfined compression test is performed on a cylindrical specimen by increasing the axial load until the specimen fails by either reaching a maximum load or attaining some maximum level of axial strain, e.g., 15 percent. Specimens are sheared at a relatively fast rate that generally produces failure in less than 15 minutes. The test is restricted to soils that have a low enough permeability to prevent the expulsion or taking up of water by the specimen during the relatively short time of loading. Accordingly, the unconfined compression test is appropriate for measuring the *undrained* shear strength of soils for short-term stability problems.

Unconfined compression tests are only applicable to saturated soils where the Mohr-Coulomb failure envelope can be represented by a horizontal line when plotted in terms of total stresses (Figure 2.4). When a soil is saturated and there is no drainage, the shear strength is independent of the total confining pressure because the applied confining pressure is carried entirely by the pore water. Thus, there is no increase in effective stress with an increase in confining pressure and no increase in shear strength. In this case the shear strength is expressed by a value of cohesion (c) and ϕ is assumed to be zero. The undrained shear strength in this case ($\phi = 0$) is also commonly expressed by the symbol, s_u. Analyses using this representation of shear strength ($s = c = s_u$, $\phi = 0$) are performed using total stresses; effective stresses are neither known nor used.

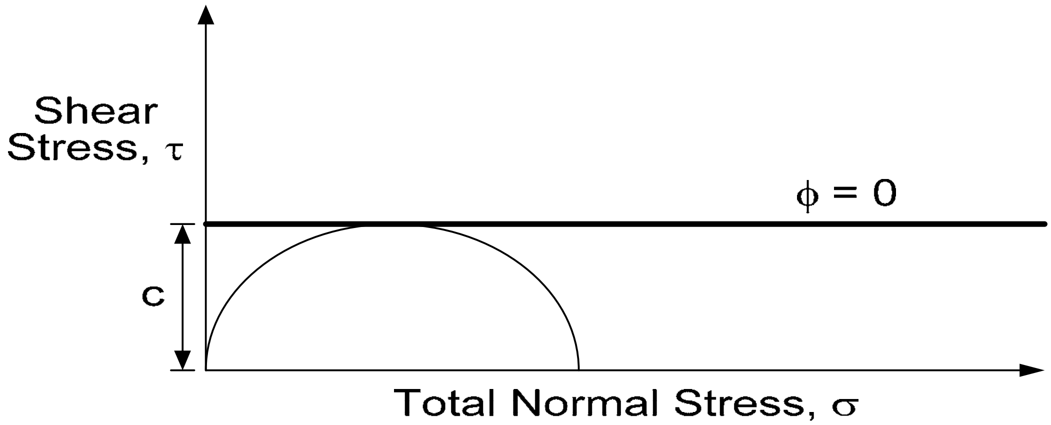


Figure 2.4 Horizontal Mohr-Coulomb failure envelope implied for an unconfined compression test on a saturated soil.

Theoretically any confining pressure, including zero, should be acceptable for determining the undrained shear strength (c) of a saturated soil when ϕ is zero. However, in actual practice the shear strength of even saturated soils will probably vary with confining pressure, particularly for natural soils where the confining pressures in the laboratory are less than in the field. The actual strengths are lower at low confining pressures because when a saturated soil is sampled the stress is reduced and air that is originally dissolved in the pore water comes out of solution and allows the soil to expand. The stress release caused by sampling may also allow joints and fissures that are closed in the field to open. Expansion and the opening of joints and fissures will result in a reduction of shear strength. Although the lower strengths measured in unconfined compression tests may be considered "conservative," they may also impose an unnecessary penalty to the design caused by use of unreasonably low values for shear strength. The additional cost of triaxial (confined) compression tests, which are described next, should be weighed against the additional cost of designs based on strengths from unconfined compression tests, which may be too low.

Triaxial Compression Tests

Triaxial tests are performed on cylindrical specimens that are surrounded by a rubber membrane. The specimen is subjected to an all-around confining pressure applied through fluid (normally water) in the triaxial cell. The specimen is sheared by increasing the axial load through a piston extending out through the top of the triaxial chamber. During the application of both the confining pressure and axial load the specimen may or may not be allowed to drain. If the specimen is allowed to drain the volume of water flowing into or out of the specimen may be measured. If the specimen is not allowed to drain during the application of the axial load the pore water pressures in the specimen may be measured. A schematic of a typical triaxial test setup to allow measurement of both volume and pore water pressure changes is shown in Figure 2.5.

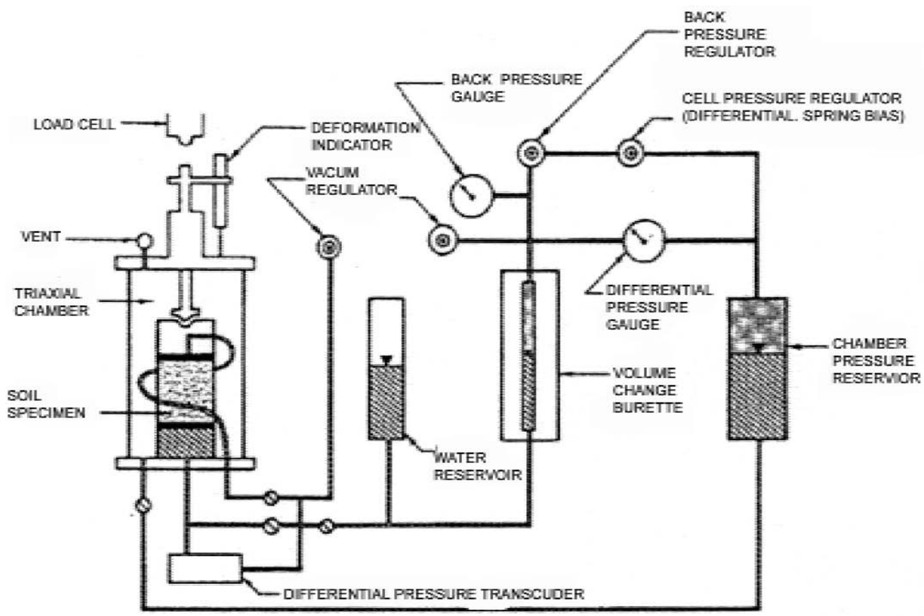


Figure 2.5 Schematic diagram of triaxial test setup allowing measurement of both volume change and pore water pressures (from ASTM, 2002).

Several different types of loading may be applied in triaxial tests depending on whether the specimen is allowed to drain during (1) the application of the confining pressure and (2) the application of the shear load. Depending on the drainage allowed, different shear strengths are measured. The various loading (drainage) conditions and shear strengths are discussed in the next section.

Loading Conditions

The triaxial test allows for loading under either drained or undrained conditions. In contrast, the direct shear apparatus requires that loads be applied slowly enough for the soil to freely drain (expand or compress), while in the unconfined compression test the loads are applied quickly and the soil has no time to drain. Depending on the drainage allowed in the triaxial test, three different loading possibilities and test types are possible as described below.

Unconsolidated-Undrained (UU) Test

In the unconsolidated-undrained (UU) triaxial compression test no drainage is allowed during application of either the confining pressure or axial (shear) load. Pore water pressures are not measured and, thus, the pore pressures and effective stresses are not known. Any attempt to measure the pore water pressures would most likely cause water to move into or out of the specimen to or from the measuring system, thus causing the test no longer to be an undrained test.

Results of unconsolidated-undrained tests are always plotted on a Mohr diagram using total normal stresses because only the total stresses are known. If the soil is *saturated*, the shear strength envelope will be approximately a horizontal line (Figure 2.6). The pore water carries any increase in total confining pressure because the soil cannot compress without some drainage. Thus, there will be no change in the effective stress carried by the soil solids and no increase in shear strength. For saturated soils the undrained shear strength is expressed by the cohesion intercept (c) and $\phi = 0$.

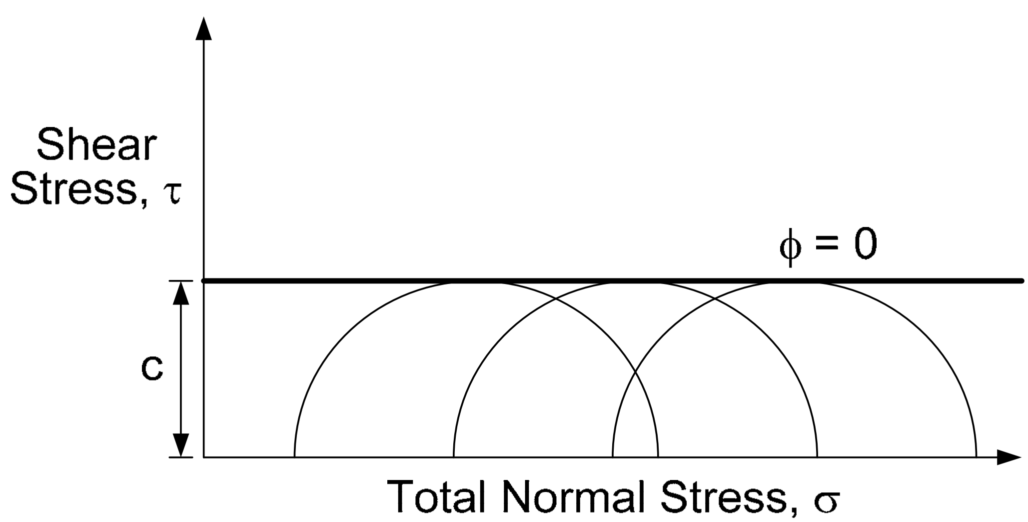


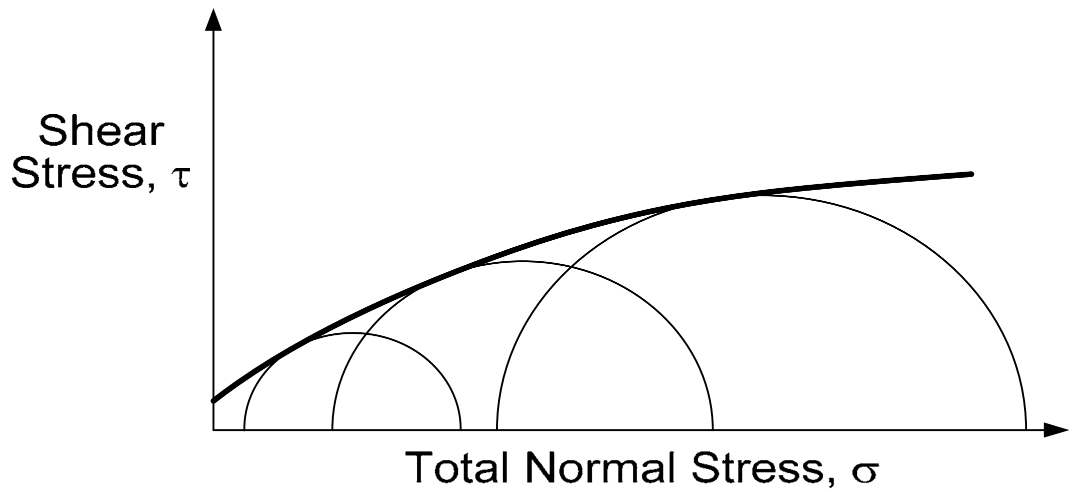
Figure 2.6 Horizontal Mohr-Coulomb failure envelope for unconsolidated-undrained (UU) triaxial compression tests on a saturated soil.

If the soil is *unsaturated*, the soil will be able to compress some in an unconsolidated-undrained test due to the compression of air in the void space as the confining pressure is increased. In this case the strength will increase some as the confining pressure is increased. When tested over a wide range of stresses, unsaturated soil will often exhibit a failure envelope that is curved as shown in Figure 2.7a. In an analysis for an unsaturated soil either the curved envelope can be used directly or the envelope may be approximated by a straight line as shown in Figure 2.7b. Care must be used in approximating a curved failure envelope by a straight line to not extrapolate to stresses beyond those where the linear envelope applies.

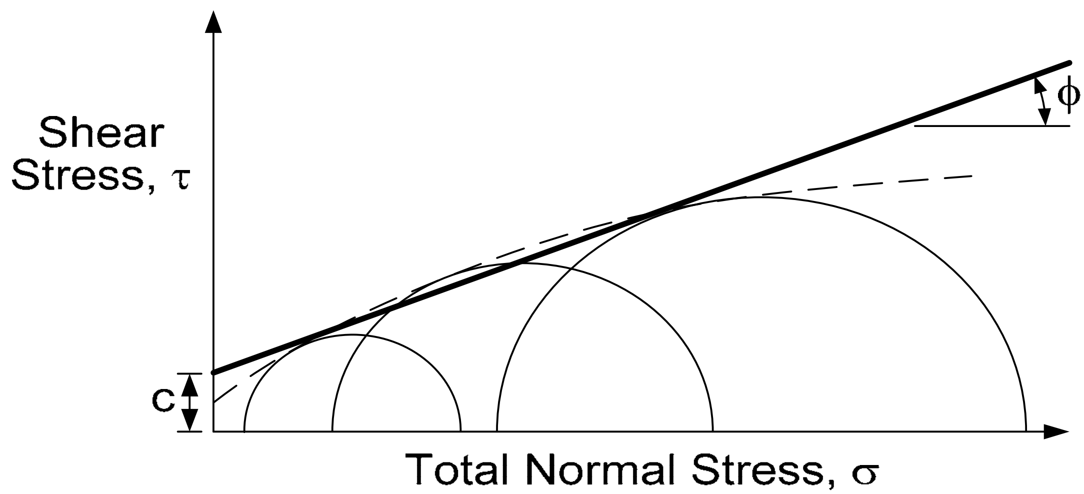
The unconsolidated-undrained (UU) test provides a measure of the shear strength for conditions where there will be little or no drainage during the application of loads in the field. For saturated soils theoretically the unconfined compression test should also give the same value of shear strength (c) as the unconsolidated-undrained (UU) triaxial test. However, as discussed earlier, in practice the unconfined compression test may give too low a strength due to effects of stress release and the possible opening of fissures and joints in specimens. In such cases UU tests performed using total confining pressures comparable to those in the field are recommended in favor of unconfined compression tests.

Consolidated-Drained (CD) Test

In consolidated-drained (CD) triaxial compression tests the soil is allowed to drain (consolidate or swell) fully under both the applied confining pressure and the shear (axial) load. As is the case with direct shear tests it is important that the loads be applied slowly enough to ensure that the specimen drains freely and that there are no excess pore water pressures in the specimen during loading. It is also necessary to fully saturate the specimen to avoid suction due to capillary stresses that might increase the effective stresses and shear strength of the specimen.



(a) Curved Mohr failure envelope.



(b) Equivalent linear representation of curved Mohr failure envelope.

Figure 2.7 Curved Mohr failure envelope for undrained shear strength of an unsaturated soil and equivalent linear representation.

Because specimens are allowed to drain and the pore water pressures remain constant during shear, the pore water pressure during the test is equal to the known value at the start of the test. Thus, it is possible to plot the results of consolidated-drained tests in terms of effective stresses as illustrated in Figure 2.8. In Figure 2.8 a linear Mohr-Coulomb envelope is suggested;

however, in some cases and as discussed later in this report, the envelope may actually be curved. For analyses when the envelope is curved, either the curved envelope may be used directly or an equivalent linear envelope may be fitted and used (as suggested earlier for the total stress failure envelope from UU tests). Because the failure envelope from consolidated-drained tests is plotted in terms of effective stresses, analyses that use the strength envelope must be performed using effective and not total stresses.

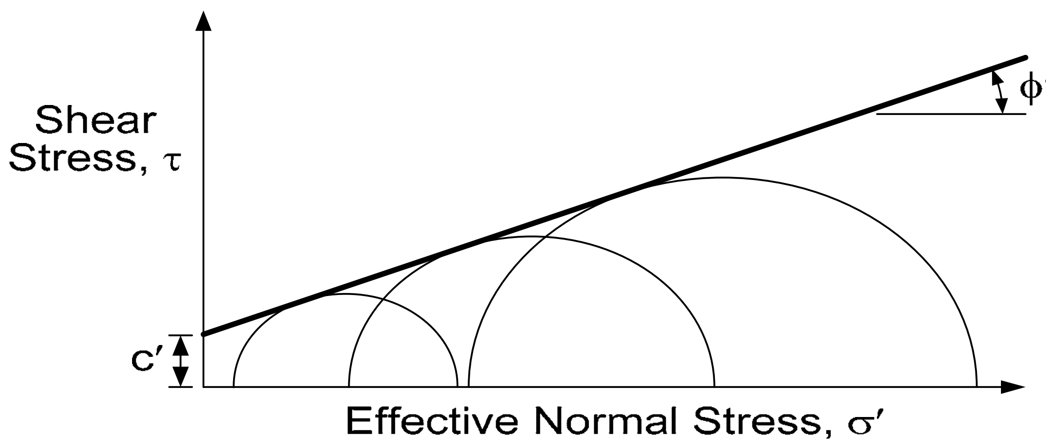


Figure 2.8 Mohr-Coulomb failure envelope for consolidated-drained (CD) triaxial compression tests plotted in terms of effective stresses.

Consolidated-Undrained (CU) Test

In the consolidated-undrained test, specimens are allowed to fully consolidate or swell, i.e., "drain" under the application of the all-around confining pressure, but are not allowed to drain when sheared. During shear the pore water pressures that develop in specimens may or may not be measured depending on the eventual use of the test results. However, for the applications of interest in this report pore water pressures should always be measured to enable the test results to be plotted using effective stresses.

Although there is no drainage during shear in the consolidated-undrained test, the test should still be performed at a slow enough rate to achieve good measurement of pore water pressures. Because the strains are not uniform over the height of the specimen, the pore water pressures generated during shear will vary. In most cases the pore water pressures are measured at a point outside the specimen through a measuring system connected to one or both ends of the specimen. In order to ensure that the pore water pressures measured at the ends of the specimen are representative of those in the rest of the specimen, the loading rate should be relatively slow. Details on the loading rates for proper measurement of pore water pressures in consolidated-undrained triaxial tests and how the loading rates should be calculated are given in ASTM (2002).

Results of consolidated-undrained triaxial compression tests may be plotted and used in several ways. The most common use of consolidated-undrained tests is to measure the shear strength of the soil as a function of effective stress for use in analyses of problems where the loading is actually drained. The effective stress envelope for fully drained conditions has already been discussed and illustrated earlier for consolidated-drained loading (Figure 2.8). The principal limitation of consolidated-drained (CD) tests for determining the strength for drained problems is that for clays the CD test requires relatively long times to perform because of the low permeability of the clay and the slow rates of drainage. Consolidated-undrained tests can be performed faster than CD tests, and results show that both tests (CD and CU) yield essentially the same shear strength envelope when plotted in terms of effective stress. Thus, by performing consolidated-undrained tests and measuring the pore water pressures, strengths can be expressed as a function of effective stress and then applied to problems where there is drainage.

There are also several cases where consolidated-undrained tests are performed to measure the shear strength for problems where the soil may be consolidated and then subjected to *undrained* loading. The first case is stage construction where only a portion of the final fill is placed and the soil is allowed to consolidate before the next level of fill is added. The second case is rapid drawdown, where water adjacent to a slope is suddenly removed after the slope has been in place for some time and consolidation or swell has occurred. Both of these cases (stage construction and rapid drawdown) involve relatively complex testing and analyses and are not addressed further in this report. For further discussion of these special loading conditions the reader is referred to Corps of Engineers (2003) or Duncan and Wright (2005).

A third case where consolidated-undrained (CU) tests are used for problems involving undrained loading is for reducing the effects of sampling disturbance on the undrained shear strength. Procedures such as the SHANSEP procedure developed by Ladd and Foott (1974) can be used for this purpose. In this approach special procedures are used to reconsolidate soil specimens before shearing them in an effort to reduce the effects of sampling disturbance. However, care must be exercised to avoid testing specimens that are too strong due to reconsolidation to lower void ratios (higher densities) than in-situ and, thus, overestimation of the strength in the field. It is unlikely that procedures such as SHANSEP will be used for many TxDOT projects.

Residual and Fully Softened Shear Strengths

Normal practice for determining the drained shear strength of soils in the laboratory is to use the stresses corresponding to peak load in terms of effective stresses. However, experience indicates that in a number of cases, particularly cases involving natural and excavated slopes in highly plastic clays, the shear strength may be less than the values corresponding to peak stresses. Instead, and depending on the particular slope and its history, the appropriate strengths may be either the *residual* strength or the *fully-softened* strength, which are both lower than the peak strength.

Residual Shear Strength

The term *residual strength* was apparently first used by Skempton (1964) to describe the shear strength that is ultimately developed after soil has experienced large strains under drained conditions (Figure 2.9a). For many highly plastic clays the residual shear strength is significantly less than the peak shear strength, with a lower friction angle, ϕ' (ϕ'_r), and a small or negligible cohesion, c' (c'_r), as suggested in Figure 2.9b.

Skempton (1964) measured the residual shear strengths for London Clay, a heavily overconsolidated, stiff-fissured clay, and compared the strength to the strength that was apparently developed over time in the field. It was suggested that over time the residual shear strength would eventually develop and govern the design. However, subsequent studies over time (e.g., Skempton, 1977) eventually led to the conclusion that residual shear strengths probably only develop in slides that are a recurrence of a previous slide and/or similar large strains have been experienced in the past. Residual shear strengths are probably not applicable to slopes in general.

"Fully-Softened" Shear Strength

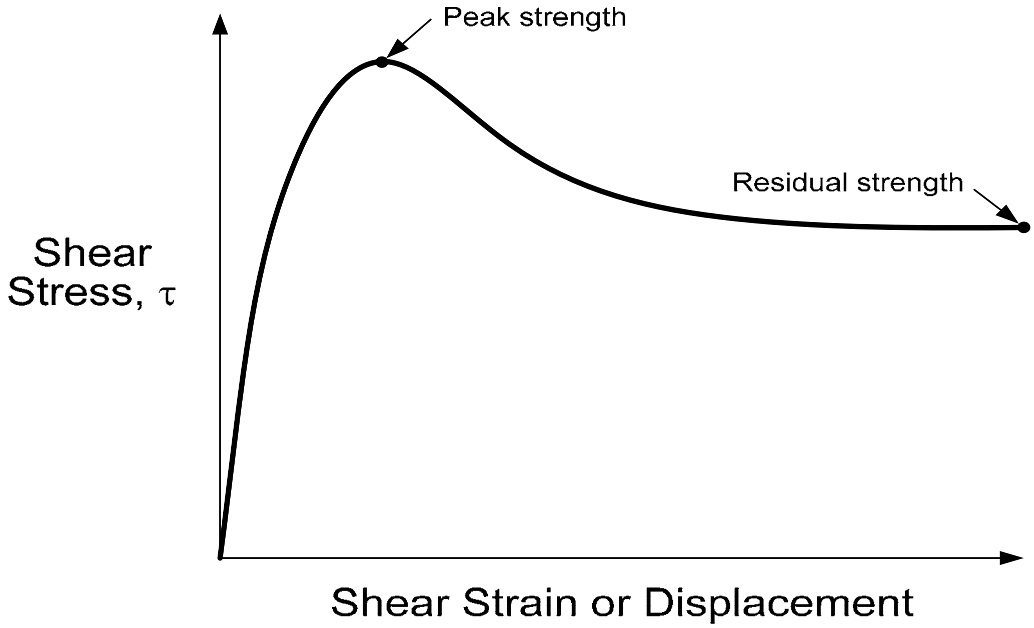
Further studies by Skempton and his co-workers revealed that the shear strength in many slopes was lower than the peak strength, but higher than the residual value discussed in the previous section. This lower strength has been termed the *fully-softened* strength. Skempton (1977) noted that the fully-softened strength corresponded to the strength of the soil in a "normally consolidated state." The fully-softened strength can be measured in the laboratory by preparing samples of normally consolidated clay and then testing them. Usually samples are prepared by mixing the soil with water to form a slurry and then consolidating the slurry to various pressures for testing.

The term "fully-softened" strength is used to describe a drained shear strength, expressed in terms of effective stress shear strength parameters, c' and ϕ'. Although there is also a softening and reduction in strength that occurs over time simply due to wetting and reduction in the effective stress (σ'), the term fully-softened is generally used in reference to the effective stress shear strength parameters, c' and ϕ', rather than the reduction in effective stress, σ'.

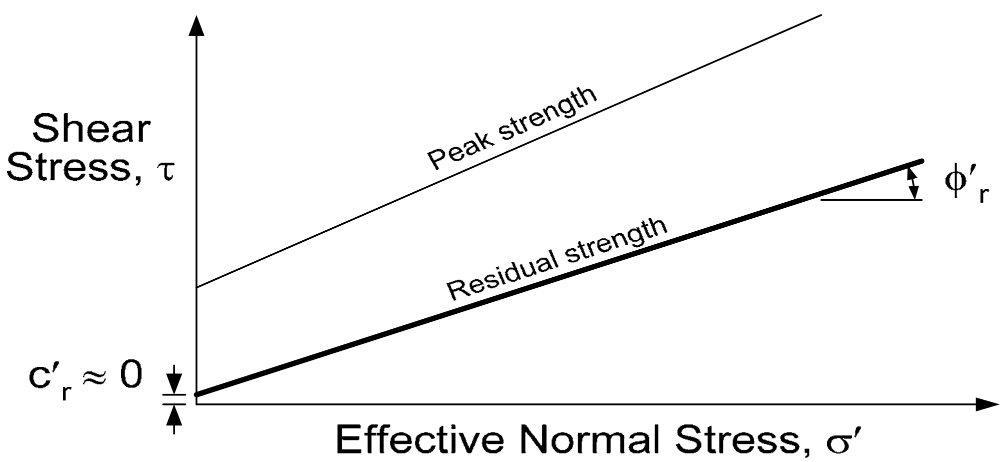
Time Effects

Time influences the strength of many soils, and clays in particular. Firstly, undrained strengths may vary due to creep effects. Studies have shown that the undrained strength generally decreases as the rate of loading is decreased (made slower). Also, laboratory creep tests in which specimens are subjected to a sustained load with no drainage show that the soil may fail under a lower sustained load than the one corresponding to short-term loading and the conventional loading rates used in the laboratory. However, such creep effects observed in purely undrained loading are probably offset to some extent in the field by partial drainage. Consequently, creep effects are generally ignored for most applications. Only when relatively low factors of safety (less than about 1.3) are used for design is it probably necessary to consider creep effects.

The most important effects of time leading to reductions in shear strength are those that occur as the soil "drains" and approaches a long-term condition. There are two effects: The first effect is due to a reduction in effective stress that occurs when soil expands (swells) over time. Expansion will occur in most excavated (cut) slopes and at shallow depths in many fill slopes constructed of highly plastic or so-called "expansive" soils. In the case of highly plastic fill materials the pore water pressures at shallow depths are typically negative (suction) after construction and may gradually increase toward atmospheric or positive pressures with time. The second long-term time effect is the reduction in the shear strength expressed in terms of effective stresses, i.e., reduction in c' and ϕ' over time due to repeated wetting and drying, cracking, and possibly other factors. Both effects—reduction in effective stress (σ') and



(a) Stress-strain curve.



(b) Effective stress failure envelopes.

Figure 2.9 Peak and residual shear strengths.

reduction in effective stress shear strength parameters (c' and ϕ')—should be taken into account in assessing the long-term strength of clays. This necessitates use of drained shear strengths with allowance for residual or fully-softened values of the shear strength parameters (c' and ϕ') as discussed in the previous section.

Ground Water and Pore Water Pressures

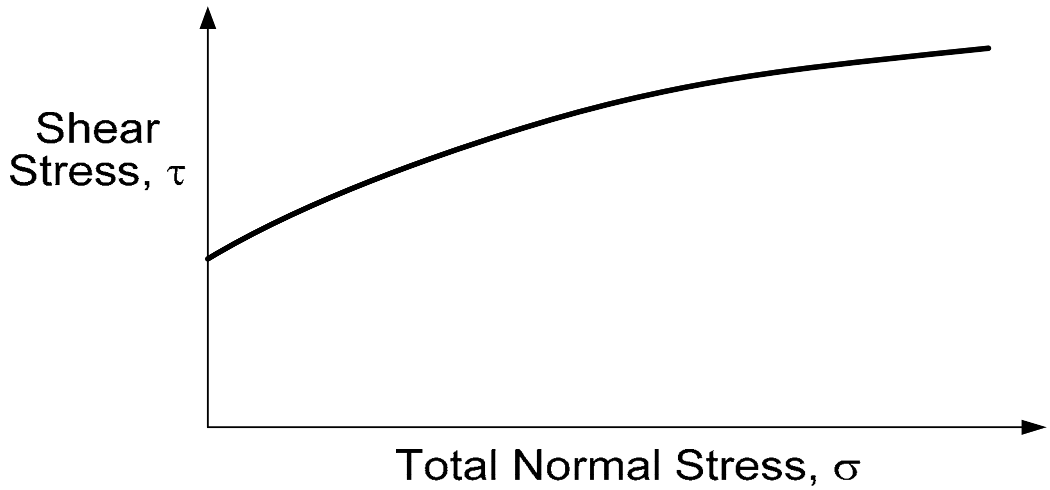
Long-term, “drained” shear strengths are always expressed in terms of effective stresses (Eq 2.3). Accordingly, for design the pore water pressures must be known in order to determine the appropriate shear strength. If seepage is present, an appropriate seepage analysis should be performed (Cedergren, 1989; U. S. Army Corps of Engineers, 1986). If there is no seepage, the location of the water table, if present, should be determined by appropriate field investigations and monitoring. Further details of seepage analysis and groundwater studies are beyond the scope of this report but can be found in the references cited.

Above a water table the pore water pressures will be negative: By definition the pore water pressures are zero at the water table. Any negative water pressures that exist will contribute to increasing the effective stresses in the soil and, thus, will contribute positively to the shear strength. However, for most design applications negative pore water pressures will not be considered to contribute to the long-term, drained strength, because of the likelihood that rainfall or other sources of water may greatly reduce or eliminate negative pore water pressures (soil suction).

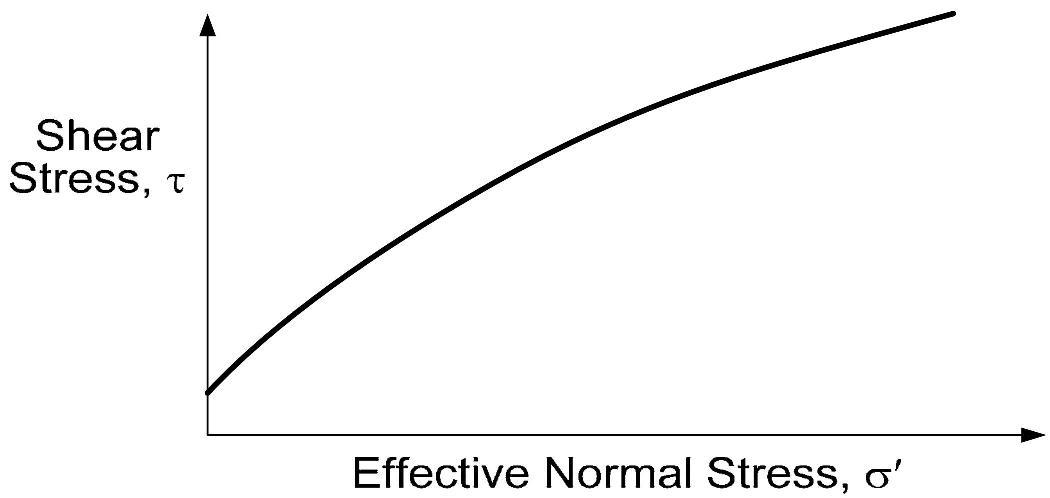
Negative pore water pressures do play a significant role in the undrained strength of fine-grained soils and are reflected in the strengths measured in unconfined compression and unconsolidated-undrained tests, especially at low stresses. However, for undrained shear strengths the effect of negative pore water pressures is only being counted on for the short-term, not long-term, strength, which is reasonable.

Curved Mohr Failure Envelopes

The Mohr failure envelopes used to describe the shear strength of many soils are not linear, but rather curved. Curved envelopes may exist for strengths expressed in terms of both total stresses (Figure 2.10a) and effective stresses (Figure 2.10b). There are at least three ways in which shear strengths are reported and used when the failure envelope is curved: (1) by an equivalent linear Mohr-Coulomb envelope, (2) as the actual curved strength envelope, and (3) by a series of “secant” friction angles that vary with the normal stress (σ).



(a) Total stresses



(b) Effective stresses

Figure 2.10 Curved Mohr failure envelopes for total and effective stresses.

Equivalent Linear Envelopes

As discussed earlier and illustrated in Figure 2.7, a curved failure envelope may be represented by an equivalent linear envelope and set of strength parameters (c and ϕ, or c′ and ϕ′, depending on the type of test and strength envelope). This approach may be necessary when the particular method of slope stability or retaining wall analysis being used requires that shear strengths be represented by a cohesion and friction angle value. Many of the equations used to compute earth pressures and bearing capacity are based on soil strengths defined by a linear Mohr-Coulomb strength envelope and values for c and ϕ. However, care must be exercised to ensure that strengths are not extrapolated beyond the range of stresses where the equivalent linear envelope is valid. An estimate of the maximum and minimum stresses involved in an analysis should be made before fitting a linear envelope.

Actual Curved Envelope

If the failure envelope is curved, it can be approximated by defining the coordinates (τ and σ or τ and σ') of a series of points on the envelope which are connected by straight lines, to define a piecewise linear envelope. Such an approach is recommended when using software such as the UTEXAS slope stability software that allows a curved shear strength envelope to be defined in this manner (Wright, 1999).

"Secant" Friction Angles

A common way of representing a curved failure envelope is to compute a series of secant friction angles, ϕ_{secant}, (Figure 2.11) for various stresses, σ, and plot the secant friction angles as a function of stress. This is commonly done for soils that exhibit no cohesion intercept. Depending on the type of test, the secant friction angles may be plotted versus the normal stress on the failure plane, σ_f, or versus the confining pressure, σ_3. For direct shear and ring shear tests the secant friction angles are usually plotted versus the normal stress on the failure plane (σ_f), while for triaxial tests it is common practice to plot the secant friction angles versus the confining pressure (σ_3) used in the test. Duncan et al. (1989) have shown that a linear equation can be used to approximate the relationship between secant friction angle and the logarithm of the effective confining pressure, (σ_3'). This is discussed further in Chapter 5.

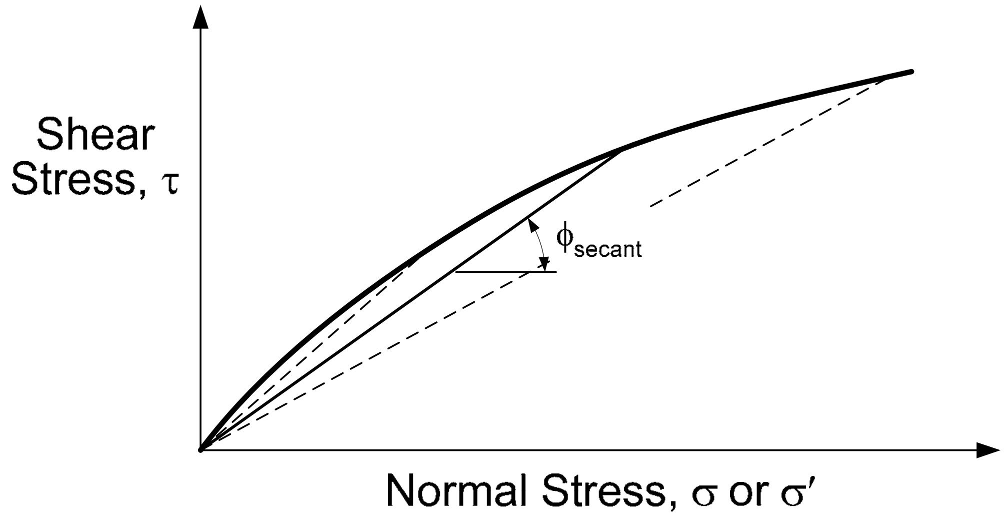


Figure 2.11 Secant friction angles used to represent shear strengths when a failure envelope is curved.

Back-Analyses

A useful way of confirming measured or estimated shear strength values for slopes that have failed is to perform a *back-analysis* of the slope. If the slope failed, the factor of safety at the time of failure should be unity. Thus, if the shear strengths and other conditions used in an analysis are correct representations of the conditions at the time of failure, the computed factor of safety should be unity. By varying the assumed conditions for an analysis, a set of conditions applicable to the slope at failure can be established. Although these will depend not only on the shear strength parameters, but also on the slope geometry, unit weights, subsurface stratigraphy, external water and/or surcharge loads, and in the case of effective stress analyses, the pore water pressures, back-analyses can be helpful in establishing strength values. With little extra effort and cost, such analyses can be preformed for a slope that has failed and provide useful information for redesign and remedial measures.

Although back-analyses are useful they must also be used cautiously. Back-analyses are useful in establishing the conditions in the slope at the time of failure, but these may not be the ultimate, worst conditions that the slope will experience. For example, pore water pressures may rise further due to increases in water levels and the soil may continue to swell if failure occurred before final equilibrium was reached. Also, for soils exhibiting low residual shear strengths, the shear strength at the time of failure, e.g., peak or fully-softened strength, may be higher than the strength that exists once a slide has occurred. Once a slide occurs, residual shear strengths may be applicable, although higher strengths controlled the initial slide and may be calculated by back-analysis.

Unsaturated Soils and Unsaturated Soil Mechanics

In recent years considerable progress has been made in understanding the behavior of unsaturated soils and developing a fundamental basis for representation of shear strengths, particularly for drained conditions and an effective stress framework. Most notable among these efforts is the work of Fredlund and his coworkers (Fredlund and Rahardjo, 1993; Fredlund, 2000). This work has shown that the expression of effective stress given by Eq 2.1 does not strictly apply for unsaturated soils: The pore water pressure and pore air pressure in the voids of the soil are different due to capillary effects and both pore pressures (air and water) need to be considered as separate, independent contributors to the state of stress. Although significant advances have been made, it does not seem likely that these will become a part of routine TxDOT practice in the near future except for unusual circumstances. The present practice of expressing undrained shear strengths in terms of total stresses (Figure 2.4 and 2.6) and only considering effective stresses for long-term conditions and saturated soils is adequate for most problems. For long-term conditions where the soil is unsaturated, the pore water pressures will typically be negative, but can be assumed to be zero because of uncertainty. A well-established geotechnical practice is to consider the most critical conditions for design, which is consistent with the practice of neglecting negative pore water pressures in slope stability analyses. For unsaturated soils Eq 2.1 is used with zero pore water pressures to compute effective stresses.

Chapter 3 – Previous TxDOT Research Studies

Introduction

During the past approximately thirty years several research projects have been conducted for TxDOT by the Center for Transportation Research (CTR) related to issues of slope stability. Many of these studies have addressed, at least to some extent, the shear strength of clay soils. The various research studies and findings related to shear strength are summarized and reviewed in this chapter. The studies are covered in chronological order, except for a study by O'Malley and Wright (1987), which dealt primarily with short-term, rather than long-term strengths. The study by O'Malley and Wright is discussed last.

Project 161 – Abrams and Wright (1972)

Project 161 focused primarily on design of measures for repairing earth slopes, including both cut and fill slopes. As part of that effort a method was developed for back-calculating the shear strength of the soil from slopes that had failed. A chart was developed for back-calculating cohesion and friction angle values given the slope and slide geometry. The chart was based on total—rather than effective—stresses.

Project 161 examined a number of slope failures along Texas highways. Most of the observed failures involved cut slopes and occurred a number of years after construction of the slope. Significant groundwater and surface water were observed at many of the failures. These observations showed the need to consider long-term stability and the importance of water. However, no soil shear strength data were reported for any of the slope failures examined.

Project IAC 2187 – Gourlay and Wright (1984)

This project was initiated as an Interagency Contract with the Houston District of TxDOT to address a number of then recent (1982-1984) slope failures that had occurred in embankments of highly plastic clays in the area of Houston, Texas. Prior to this time there had been little or no laboratory tests performed by or for TxDOT to measure the long-term (drained) strength properties of the clays involved.

Soil tested for this project consisted of soil taken from the site of an embankment failure at the intersection of Scott Street and I. H. 610 in Houston. Two apparently different clays, designated as “red” and “grey” clay, were identified in the embankment. Index properties and compaction information are summarized for the two clays in Table 3.1.

Table 3.1 Summary of Index and Compaction Properties for Clay from Scott Street and I.H. 610 Site from Gourlay and Wright (1984).

Soil Property	**Red Clay**	**Grey Clay**
Plastic Limit	19.7 – 21.1	14.6 – 18.0
Liquid Limit	71.4 – 72.7	53.8 – 55.2
Plasticity Index	51.6 – 51.7	37.2 – 39.2
ASTM D698 (Standard Proctor) Maximum Dry Unit Weight	100 pcf	105 pcf
ASTM D698 (Standard Proctor) Optimum Water Content	22.5 %	19 %

Laboratory strength testing consisted of several series of consolidated-undrained (CU) triaxial compression tests and a limited number of unconfined compression tests. Unconfined compression tests were performed on specimens of the red clay compacted at moisture contents generally within the range of 23–24 percent. The compactive effort was varied to produce dry unit weights ranging from approximately 81 pcf to 98 pcf. The corresponding unconfined compressive strengths (q_u) ranged from 2000 psf to 5000 psf, depending on the compaction unit weight. At a dry unit weight of approximately 95 pcf, which corresponds to about 95 percent of the ASTM D698 (Standard Proctor) maximum dry unit weight, the unconfined compressive strength was approximately 4000 psf. Assuming a friction angle of essentially zero—an approximation—this strength (q_u = 4000 psf) corresponds to a cohesion value of 2000 psf. A cohesion of 2000 psf with $\phi = 0$ produces a factor of safety of approximately 2.0 for a vertical slope 30 feet high, and much higher factors of safety for flatter slopes and slopes of lesser height. For example, a 3(horizontal):1(vertical) 20 feet high slope would have a factor of safety of approximately 8.0! This clearly indicates that slopes constructed of the compacted clay that was tested would be very stable during and immediately following construction.

Consolidated-undrained triaxial compression tests with pore water pressure measurements were performed on specimens of both the red and grey clays. Specimens of the red clay were prepared for laboratory testing by compacting the soil to a target dry unit weight of 96.3 pcf. This dry unit weight corresponds to approximately 96 percent of the ASTM D698 maximum dry unit weight. The corresponding target moisture content was 24 percent, which is approximately the optimum moisture content for the target unit weight. Specimens of the grey clay were compacted to a target dry unit weight and moisture content of 102.0 pcf and 21 percent respectively. The target unit weight corresponds to approximately 97 percent of ASTM D698 maximum dry unit weight and the moisture content of 21 percent is the approximate optimum moisture content for this unit weight. Results of the consolidated-undrained triaxial tests with pore water pressure measurements yielded the strength parameters expressed in terms of effective stresses that are summarized in Table 3.2.

Table 3.2 Summary of effective stress (drained) shear strength parameters from Gourlay and Wright (1984)

Soil	Cohesion, c′ (psf)	Friction Angle, φ′
Red clay	270 psf	20.0º
Grey clay	390 psf	19.7º

Project 353 – Stauffer and Wright (1984)

Project 353 was initiated to conduct a detailed study of slope failures in Texas, including the cause of failures and potential remedial measures. Early in the study it became evident that the major type of slope failure occurring on Texas' highways at the time involved sliding in embankments constructed of highly plastic clays. Accordingly, most of the study was focused on this problem.

Numerous failures of embankments in highly plastic clays were identified, and detailed measurements of the slope and slide geometry were taken. Most of the observed slides were shallow with estimated depths ranging from 2 to 10 feet below the ground surface. In each case disturbed samples of the soils believed to be involved in the failure were also taken and used to determine soil index properties. Finally, the age of the slope was determined based on TxDOT construction records when available. The age of the slope at the time of failure is plotted versus the slope angle in Figure 3.1. It can be seen that all of the slopes that failed were at least 10 years old, with the average age at the time of failure being nearly 20 years.

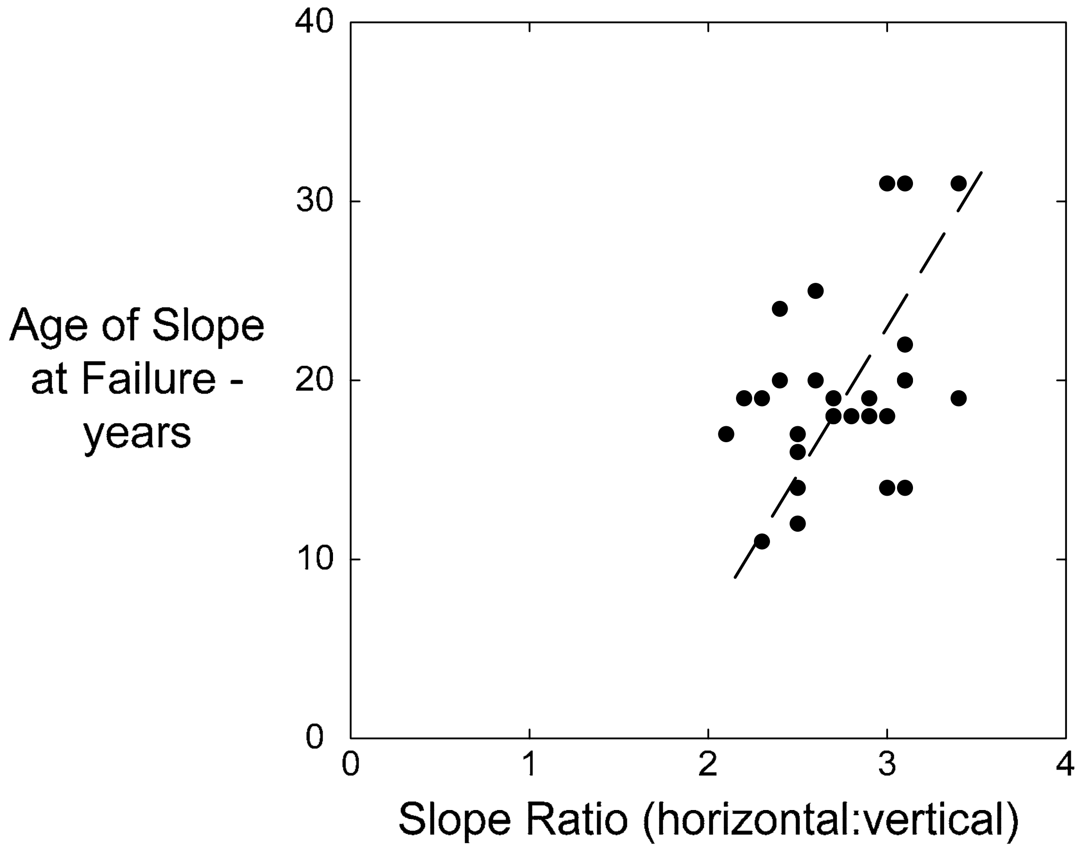


Figure 3.1 Variation in the age of TxDOT embankment slopes at failure with the slope ratio (from Stauffer and Wright, 1984).

It was recognized that useful information about the strength properties of the soils involved in the observed slope failures could be obtained by back-analysis to calculate shear strength parameters. To facilitate this effort the charts first developed by Abrams and Wright (1972) for back-calculating shear strengths were extended to effective stress analyses. These charts were then used to back-calculate shear strength parameters for 26 of the slides that were documented. The pore water pressures at the time of failure were not known, but at the same time it seemed reasonable to assume that, because the slopes were all embankments above natural ground and did not impound water, there would not be any positive pore water pressure. Accordingly, the pore water pressures were assumed to be zero for back-calculating the shear strength parameters. Back calculated friction angles (ϕ') ranged from 10.5° to 23.2°; back-calculated cohesion (c') values ranged from approximately 3 to 59 psf. The back-calculated friction angles for at least some of the slides examined were in agreement with the value of approximately 20° reported by

Gourlay and Wright (1984). However, the back-calculated cohesion values were almost an order of magnitude (factor of 10) lower than the measured values.

Stauffer and Wright (1984) also performed slope stability analyses for the slope at Scott and I.H. 610 that had failed, using the shear strength values that Gourlay and Wright (1984) had reported. Using the lower of the shear strength values reported ($c' = 270$ psf and $\phi' = 20^{\circ}$), they computed a factor of safety of approximately 2.4 assuming zero pore water pressures. This clearly indicated that the slope should not have failed. Stauffer and Wright repeated the calculations assuming a pore water pressure equal to 80 percent of the *total* overburden pressure. For a slide depth of 6 feet, which is roughly the approximate average depth of the critical slip surface found in the slope stability analyses, a pore pressure equal to 80 percent of the total overburden pressure corresponds to an artesian pressure with a piezometric level that is approximately 3.5 feet above the ground surface! These pore water pressures produced a factor of safety of approximately 1.3, which still indicates that the slope should have been stable.

The studies by Stauffer and Wright (1984), along with the laboratory tests by Gourlay and Wright (1984), showed that a significant discrepancy existed between the shear strengths measured in the laboratory and those apparently developed in the field. One of the possible explanations for this discrepancy is a change that takes place over time and produces a gradual weakening of the soil. As noted above, the average age of the slopes at the time of failure was approximately 20 years, while the laboratory strengths were based on freshly compacted specimens.

Stauffer and Wright (1984) also reported one case of an embankment failure caused by a weak clay foundation. The failure occurred during construction of an embankment adjacent to Oso Bay near Corpus Christi, Texas. Field vane shear tests had been performed on the foundation soils to measure the undrained shear strength. Analyses were performed to back-calculate shear strengths from the failure and the strengths were then compared to the values measured in the field vane shear tests. The back-calculated strengths were found to agree with the field vane shear test values; however, there was a large amount of scatter in the strengths from the field vane making it difficult to establish any measure of accuracy.

Project 436 – Green and Wright (1986)

Research Project 436 was undertaken to investigate the reasons for the discrepancy reported by Stauffer and Wright (1984) between the laboratory-measured shear strengths and those apparently developed in the field. Several series of consolidated-undrained (CU) tests with pore water pressure measurements were performed for this study to determine the drained shear strengths expressed in terms of effective stresses. Most of the tests were directed toward determining what appeared to be a loss in shear strength with time in the field. Except for some of the tests on undisturbed specimens all of the tests were performed on the red clay tested earlier by Gourlay and Wright.

Normally Consolidated and "Packed" Specimens

The first two series of tests were performed to measure what is commonly termed the *fully-softened* strength of the clay. Previous to this study Skempton (1977) had reported that over time the strength of slopes in the highly plastic London Clay lost strength, eventually reaching what Skempton termed a "fully-softened" strength. Skempton (1977) indicated that the fully-softened strength is comparable to the shear strength of the soil in a normally consolidated state. In order to determine if a similar fully-softened strength might develop in the compacted highly plastic clay slopes of interest in Texas a series of consolidated-undrained triaxial compression tests was performed on specimens of the normally consolidated red clay. Specimens were prepared by consolidating a soil-water slurry one-dimensionally in specially fabricated acrylic consolidation tubes. Specimens were consolidated in the tubes to a maximum vertical effective stress of approximately 10 psi.

Preparation of normally consolidated soil specimens by consolidating a slurry one-dimensionally required a long period of time, taking up to several weeks to prepare a single specimen for testing. In an effort to prepare specimens more quickly, but having a strength comparable to a normally consolidated clay, an alternative method was developed in which specimens were prepared by "packing" soil at a relatively high moisture content into a special acrylic forming tube (cylinder). Specimens were prepared by mixing the soil at a water content of 50–60 percent, which is approximately 10 percent less than the Liquid Limit. Once mixed, the soil was carefully placed by packing it into the forming tube. Specimens prepared in this manner were subsequently referred to as *packed* specimens.

Consolidated-undrained triaxial compression tests with pore water pressure measurements were performed on both the normally consolidated and packed specimens prepared as described above. Tests were performed at effective consolidation pressures (σ'_{3c}) in the triaxial apparatus ranging from approximately 1 to 20 psi. A linear Mohr-Coulomb failure envelope was fit to the test data for each series of specimens (consolidated from a slurry and "packed"). The resulting shear strength parameters (c' and ϕ') are summarized in Table 3.3.

Table 3.3 Summary of effective stress ("drained") shear strength parameters from Green and Wright (1986) on specimens consolidated from a slurry mixture and specimens prepared by "packing."

Specimen Type	**Cohesion, c′**	**Friction Angle, φ′**
Consolidated from a slurry mixture	110 psf	24°
Packed	80 psf	22°

Both series of test specimens produced comparable values of strengths. The tests show slightly larger friction angles than those reported by Gourlay and Wright for compacted specimens, and significantly smaller cohesion intercepts. The cohesion intercepts are only approximately 20–30 percent of the values reported by Gourlay and Wright. This indicates that a fully softened strength, as suggested by Skempton, exists and is lower than the strength of freshly compacted specimens of the clay. The lower strength is consistent with lower strengths previously back-calculated by Stauffer and Wright.

While Skempton suggested that the fully softened strength corresponded to specimens that were normally consolidated, not all of the specimens for which data are summarized in Table 3.3 were normally consolidated. The specimens that were prepared by consolidation from a slurry and tested at confining pressures less than 10 psi were actually overconsolidated. Similarly, the specimens prepared by packing may have behaved as overconsolidated soils at the lower stresses used in testing, but this cannot be confirmed. Fitting linear Mohr-Coulomb failure envelopes to test data for overconsolidated soil frequently results in a cohesion intercept that does not exist for the same soil in a normally consolidated state. If for the specimens that were consolidated from a slurry only the data for effective consolidation pressures greater than 10 psi are considered, the data indicate a friction angle of approximately 28.5 degrees with very little or no cohesion.

Drained Direct Shear Tests – Compacted Specimens

Green and Wright (1986) also performed a series of drained direct shear tests on laboratory compacted specimens of the red clay from the Scott Street and I. H. 610 site. Specimens were compacted to moisture and density conditions comparable to those used for the CU triaxial tests by Gourlay and Wright. Green and Wright reported shear strength parameters from their direct shear tests for both the *peak* strength and the *residual* strength measured at large displacements. The values are summarized below in Table 3.4.

Table 3.4 Summary of shear strength parameters from direct shear tests on compacted specimens.

Condition	**Cohesion, c′**	**Friction Angle, ϕ′**
Peak strength	260 psf	21º
Residual strength	≈ 0	14º

The peak strengths from the direct shear tests shown in Table 3.4 agree very well with the values reported by Gourlay and Wright (1984) for their CU triaxial tests. As shown previously in Table 3.2, Gourlay and Wright reported $c' = 270$ psf and $\phi' = 20^{\circ}$.

The residual shear strengths reported in Table 3.4 agree favorably with the values reported by Focht and Sullivan (1969). Focht and Sullivan reported a range in values for ϕ' of from 13 to 16 degrees and suggest an average value of 14 degrees. The residual shear strengths are all significantly lower than the values of shear strength back-calculated from actual slope failures. Using the residual shear strength parameters ($c' = 0$, $\phi' = 14^{\circ}$) a 4:1(horizontal:vertical) slope would have a factor of safety of only 1.0 with no pore water pressures; any pore water pressure would reduce the factor of safety further. The test results by Green and Wright seem to confirm that the shear strengths developed in the field are higher than residual values, which is in agreement with the work of others as well as the discussion and recommendations made later in Chapter 5 of this report.

Tests on “Undisturbed” Samples

Green and Wright (1986) also performed several series of consolidated-undrained triaxial compression tests with pore water pressure measurements on samples taken in the vicinity of the slope failure at Scott Street and I. H. 610. Specimens were separated based on their color (red vs. grey clay) and location (inside vs. outside the slide area). The effective stress shear strength parameters determined from these tests are summarized in Table 3.5.

Table 3.5 Summary of effective stress (drained) shear strength parameters from tests on “undisturbed” samples.

Specimen Type	**Cohesion, c′**	**Friction Angle, ϕ'**
Grey clay from outside the slide area.	300 psf	23º
Red clay from outside the slide area.	150 psf	23º
Red clay from inside the slide area.	130 psf	23º

The shear strength parameters obtained from all the undisturbed specimens of the red clay agree favorably with those from the tests on the “normally consolidated” and packed specimens shown previously in Table 3.3. All of the cohesion values shown in Tables 3.3 and 3.5 for the red clay are significantly less (by a factor of almost 2) than those reported by Gourlay and Wright (1984) for freshly compacted specimens.

Shear strength parameters obtained from undisturbed specimens of the grey clay are comparable to those for the freshly compacted specimens reported by Gourlay and Wright (1984). This suggests that there is a lesser effect of softening in the grey clay, but the data are more limited than those for the red clay.

Project 436 – Rogers and Wright (1986)

During the course of Project 436 and the work reported above by Green and Wright, it became apparent that a softening process occurred over time in the field. Due to the high plasticity of the clays involved and the attendant shrink-swell characteristics, it seemed probable that the repeated wetting and drying in the field might be one source of the softening. Accordingly, as part of the research for Project 436, Rogers and Wright performed additional tests on specimens that were subjected to repeated cycles of wetting and drying. Both drained direct shear tests and consolidated-undrained triaxial compression tests with pore water pressure measurements were performed. These tests were all performed on the clay from the Scott Street and I. H. 610 site identified as the red clay.

Direct Shear Tests

Four series of drained direct shear tests were performed on specimens subjected to 1, 3, 9 and 30 cycles of wetting and drying. Shear strength parameters obtained from these tests are summarized below in Table 3.6.

Table 3.6 Summary of shear strength parameters from drained direct shear tests on specimens subjected to wetting and drying cycles.

Number of Wet-Dry Cycles	**Cohesion, c′**	**Friction Angle, φ′**
1	29 psf	23º
3	77 psf	26º
9	33 psf	25º
30	0	27º

The shear strength parameters summarized in Table 3.6 reveal a significantly lower cohesion value than any of the previous test series. This suggests that cyclic wetting and drying of the soil produces a significant shear strength loss, particularly in terms of the effective cohesion intercept, c′. The low cohesion values (0–77 psf) are in good agreement with the values (3–59 psf) back-calculated from actual slope failures by Stauffer and Wright (1984). However, the measured friction angles (23º–27º) are generally near or exceed the upper limit of values (10.5º -–23.2º) that were back-calculated.

The direct shear tests also indicated that the loss in cohesion occurs within a relatively few numbers of cycles of wetting and drying. In fact most of the strength loss occurred on the first cycle. However, the wetting and drying that specimens were subjected to in the laboratory was much more severe than what would be expected to occur during any cycle of wetting and drying in the field. The laboratory sequence of wetting and drying should be viewed as an *accelerated* test procedure rather than anything representing a true time sequence. Nevertheless, the eventual effects of wetting and drying in the laboratory and field are believed to be similar.

Consolidated-Undrained Triaxial Compression Tests

Rogers and Wright performed three series of consolidated-undrained triaxial compression tests on specimens subjected to 6, 10 and 30 cycles of wetting and drying. Shear strength parameters obtained from these tests are summarized in Table 3.7.

Table 3.7 Summary of effective stress shear strength parameters from consolidated-undrained triaxial compression tests on specimens subjected to wetting and drying cycles.

Number of Wet-Dry Cycles	**Cohesion, c′**	**Friction Angle, φ′**
6	242 psf	20º
10	253 psf	19º
30	213 psf	23º

The triaxial shear tests on specimens subjected to wetting and drying showed essentially the same values for the shear strength parameters as the values ($c' = 270$ psf, $\phi' = 20^\circ$) reported by Gourlay and Wright (1984) for freshly compacted specimens. Thus, the effects of wetting and drying were negligible for the triaxial specimens and did not agree with the results from the direct shear tests discussed earlier and summarized in Table 3.6.

The primary reason why the triaxial tests showed only negligible effects of wetting and drying compared to the direct shear tests is believed to be the way in which the triaxial specimens were subjected to wetting and drying. During wetting and drying the triaxial specimens were contained in an acrylic cylinder with a number of holes drilled in the cylinder to allow water to enter and exit the specimen. This apparently restricted the amount of wetting and drying and confined the specimen against lateral expansion during wetting.

Project 1195 – Kayyal and Wright (1991)

Project 1195 was undertaken to extend the work of Projects 353 and 436 and develop a reliable procedure for determining the appropriate long-term (drained) shear strengths of highly plastic clays in embankments. One of the primary objectives was to develop procedures for measuring and then establishing values for the fully-softened shear strengths that developed over time, presumably due to effects of repeated wetting and drying. Additional failures of embankments were also examined and data were collected from these failures.

Shear Strength Testing

Kayyal and Wright (1991) developed a new procedure for subjecting triaxial specimens to repeated cycles of wetting and drying. The procedure allowed the specimens much greater access to moisture and exposure for drying than the procedure used by Rogers and Wright (1986), which employed an acrylic tube for confinement. The procedure also allowed substantial lateral expansion and volume change to occur in the soil during drying.

Kayyal and Wright tested two soils. The first soil was the red clay taken from the site at Scott Street and I. H. 610 in Houston that was investigated previously. This clay is also referred to locally as "Beaumont" clay. The second soil was a highly plastic clay soil from Paris, Texas. Index properties and compaction data for the two soils are summarized in Table 3.8.

Table 3.8 Index properties of the two clays tested by Kayyal and Wright (1991).

Property	**Red Beaumont Clay**	**Paris Clay**
Liquid Limit–percent	73	80
Plastic Limit–percent	21	22
Plasticity Index–percent	52	58
Percent passing #200 sieve	12	8
Percent clay sizes (finer than 0.002 mm)	46	59
ASTM D698 (Standard Proctor) Maximum Dry Density–pcf	101	93
ASTM D698 (Standard Proctor) Optimum Moisture Content–percent	23	27

Several series of consolidated-undrained triaxial compression tests were performed with pore water pressure measurements. Tests were performed on specimens subjected to repeated wetting and drying and on specimens that were consolidated from a soil-water slurry to form normally consolidated soil. Results of the tests showed that the shear strengths of specimens subjected to repeated cycles of wetting and drying were essentially identical to the shear strengths of the normally consolidated specimens. The strength is believed to represent the fully-softened shear strength of the soil that is attained after a number of years of exposure in the field.

Tests were also performed on freshly compacted specimens. All tests were performed on specimens that were fully saturated prior to shear to obtain reliable measurements of pore water pressures and, thus, effective stresses.

The shear strength envelopes for specimens of Beaumont clay tested in the as-compacted condition and after wetting and drying are shown in Figure 3.2. Both envelopes are distinctly curved (nonlinear). The strength envelope for the specimens subjected to wetting and drying lies significantly below the envelope for the specimens tested in the as-compacted condition, especially at lower values of normal stress. The envelope for the specimens subjected to wetting and drying shows negligible intercept, which is consistent with the findings from earlier studies that suggested the cohesion value eventually became very small or zero.

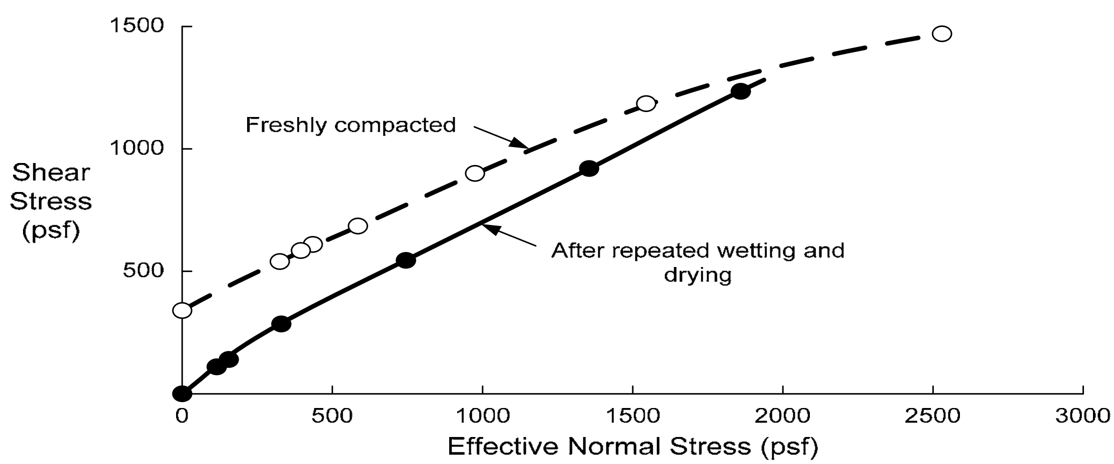


Figure 3.2 Shear strength envelopes expressed in terms of effective stresses for specimens of red Beaumont clay in the as-compacted condition and after wetting and drying.

The shear strength envelopes for specimens of the Paris clay tested in the as-compacted condition and after wetting and drying are shown in Figure 3.3. These envelopes again show a pattern similar to what is shown for the Beaumont clay: (1) Both strength envelopes are curved,

(2) the strength for the specimens subjected to wetting and drying is less than the as-compacted strength, especially at low stresses, and (3) the intercept of the strength envelope for specimens subjected to wetting and drying is small and could be considered negligible.

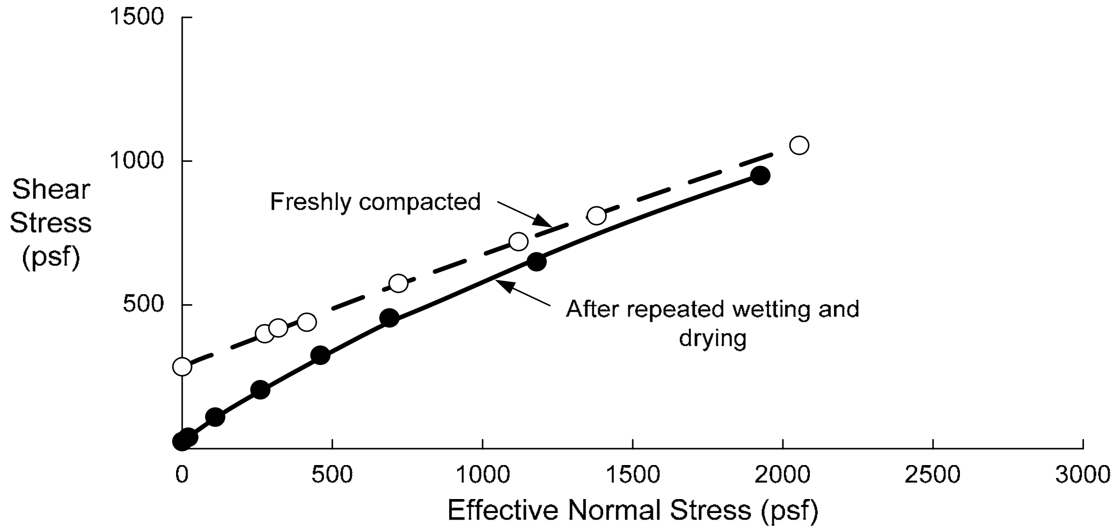


Figure 3.3 Shear strength envelopes expressed in terms of effective stresses for specimens of Paris clay in the as-compacted condition and after wetting and drying.

It is difficult to compare the strength envelopes reported by Kayyal and Wright with those from earlier studies. Kayyal and Wright clearly showed that the envelopes are curved, while in the previous studies it was assumed that the envelopes were linear and straight lines were fit to the data. Thus, the values of cohesion and friction angle that were reported earlier represent the intercept and slope of a best-fit straight line approximation of a curve. The reported cohesion and friction angle will depend on the particular range in stresses over which the straight line was fit. Some of the reported values of cohesion and friction angle also suggest that the higher friction angles were accompanied by lower values of cohesion, which further is an indication of a curved envelope and the effect of the range in stress over which the envelope was fit. In spite of this discrepancy, the earlier data seem to be in agreement with the data by Kayyal and Wright in the sense that the shear strength, and particularly the cohesion value, decreases with softening over time.

Slope Stability Analyses

Kayyal and Wright performed slope stability analyses for a number of slopes that had failed in the Beaumont and Paris clays. The shear strength envelopes shown in Figure 3.2 and 3.3 for specimens that had been subjected to wetting and drying were used for the analyses. Analyses with no (zero) pore water pressures generally revealed factors of safety considerably greater than 1.0, indicating that the slopes should have been stable. In fact, relatively high pore water pressures had to be assumed before factors of safety close to 1.0 were calculated. Based on these analyses Kayyal and Wright concluded that shear strengths were not only reduced by the effects of wetting and drying, but that significant pore water pressures must exist at the time of failure. It was concluded that pore water pressures represented by a piezometric line nearly coincident with the face of the slope and ground surface should be assumed for design.

Project 1435 – Saleh and Wright (1997)

Research project 1435 was undertaken with the recognition that TxDOT already had constructed a number of compacted clay embankments in highly plastic clays that were expected to eventually fail as the soil softened and the shear strength decreased. Emphasis was placed primarily on remedial measures. It was also recognized that the type of testing needed to define the long-term, fully-softened shear strength parameters was too costly and time-consuming to be performed for many projects. Accordingly an emphasis was placed on correlations between the applicable shear strength parameters and simple soil index properties. The correlations are the primary product of interest in this report.

Saleh and Wright recognized that the fully-softened shear strength was probably the governing strength of most highly plastic clay embankments and that the shear strength envelope, expressed in terms of effective stresses is significantly curved over the range of stresses of interest. Utilizing the results of a correlation developed by Stark and Eid (1997) for fully-softened shear strengths, and supplemented by data from the previous studies of TxDOT embankments reported above, Saleh and Wright developed the chart shown in Figure 3.4. This chart expresses the secant friction angle (Figure 2.11) as a function of the liquid limit of the soil and the effective normal stress. The secant friction angle decreases as the liquid limit and effective normal stress increases. Although the correlation expressed by the chart in Figure 3.4

has apparently not been used to a significant extent by TxDOT, it is believed to be reasonable and the basis for further discussion and recommendation in Chapter 5.

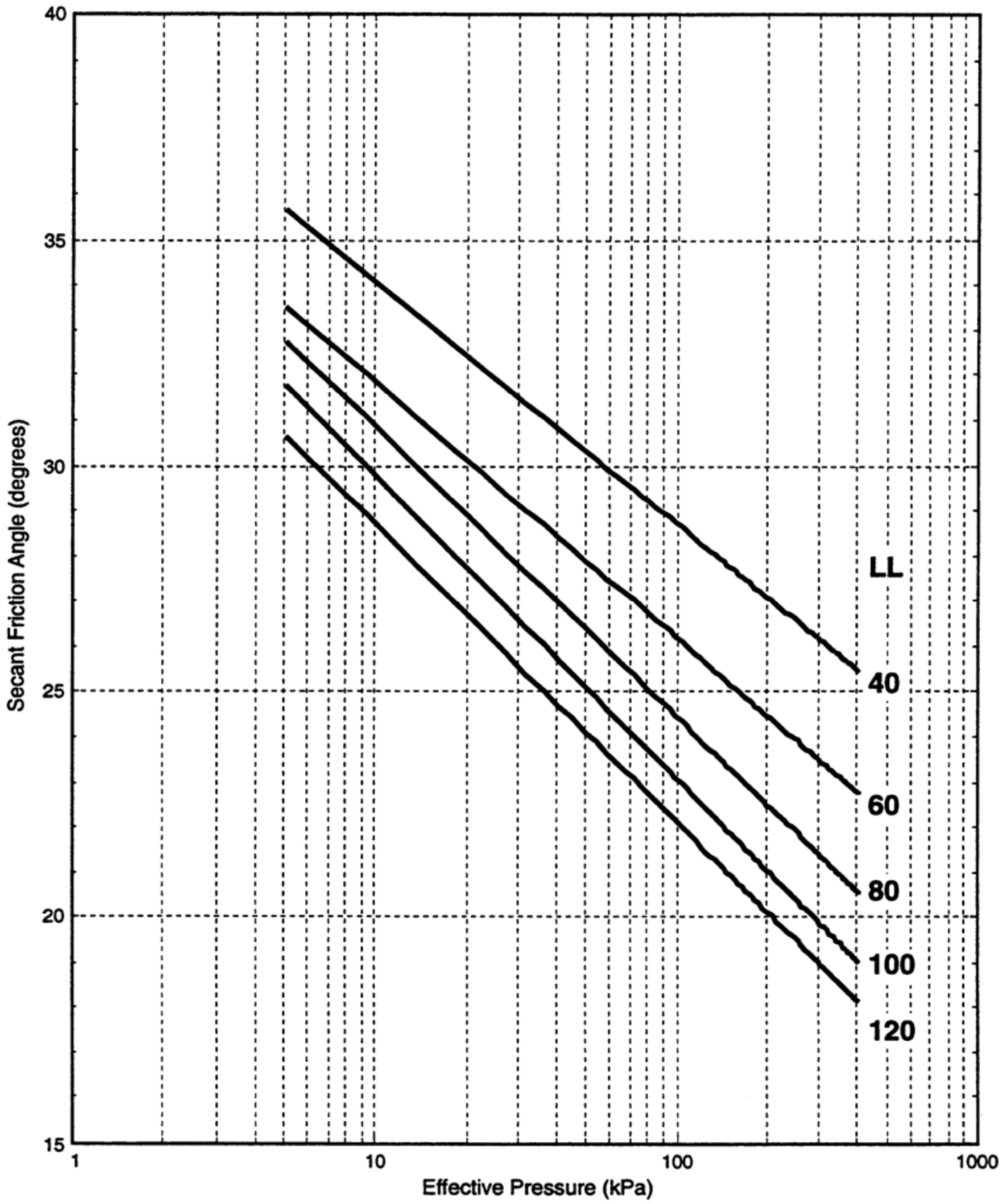


Figure 3.4 Variation in secant friction angle for fully-softened conditions with the effective normal stress for various liquid limits (from Saleh & Wright, 1997).

Project 446 – O'Malley and Wright (1987)

Project 446 was devoted exclusively to the undrained (short-term) shear strength of soft clays such as those sometimes encountered beneath embankments. The Oso Bay embankment cited earlier in the study by Stauffer and Wright (1984) is one such example where the undrained shear strength controlled the stability. In the case of the Oso Bay embankment, the embankment failed during construction, rather than many years later as was the case for most of the slopes described earlier in this chapter.

The following five types of test and apparatus were evaluated in the study by O'Malley and Wright (1987):

1. Texas Triaxial
2. Texas Transmatic
3. ASTM Unconsolidated-Undrained (UU)
4. Laboratory Vane
5. Torvane

Two types of specimens of clay were prepared for testing: The first type was prepared using a vacuum extrusion process to form the specimens. The second type was prepared by placing a mixture of soil and water at a high water content into a special acrylic tube. This process was essentially identical to the process used by Green and Wright (1986) to prepare their "packed" specimens. Both types of specimens had degrees of saturation ranging from 94 to 98 percent. Unconsolidated-undrained triaxial compression tests were performed at various confining pressures and showed very little effect of the confining pressure. This further indicates that the specimens were nearly saturated. For interpretation purposes all specimens in the study were assumed to be fully saturated such that the friction angle for undrained loading was assumed to be zero ($\phi = 0$).

Tests with the laboratory vane shear apparatus generally gave higher strengths than UU tests. Strengths from the vane shear tests ranged from 40 to 60 percent higher than the UU strength values. The higher strengths from the vane tests are consistent with the findings of others and support the need to reduce vane shear strengths before using them in slope stability analyses, as first suggested by Bjerrum (1972). This issue will be noted again in the discussion of undrained shear strengths in Chapter 4.

Torvane tests produced results comparable to those from the laboratory vane. It should also be noted that Torvane tests are often performed on soils in the ends of sampling tubes, which is often disturbed and, thus, in practice will yield less reliable values of strength.

It is important to recognize that all vane tests are only applicable for measuring the *undrained* shear strength of *saturated* soils where the friction angle (ϕ) is expressed as zero. This restriction is due to the fact that the normal stress on the failure plane is not known for the vane shear tests and the normal stress will also change during the test. Thus, for the vane shear test to be applicable, the shear strength must not vary with the applied normal stress.

The Texas Triaxial test was found to give shear strengths that were appreciably higher than those measured in UU tests. O'Malley and Wright reported that an earlier correction factor by Hamoudi (1974) appeared to be conservative for shear strengths greater than about 1000 psf, but that no reasonable correction could be found for lower strengths. Due to a dominant effect of the apparatus and rubber membrane used to confine the specimen, the Texas Triaxial apparatus was judged unsuitable for measuring the shear strength of soft clay.

The Texas Transmatic test employed a cumbersome piece of apparatus that offered no apparent advantage over the more conventional apparatus used for unconsolidated-undrained triaxial tests. Numerous deficiencies in the apparatus were noted and the apparatus was not recommended for use.

Based on this study it was recommended that the ASTM unconsolidated-undrained (UU) test be used to measure the undrained shear strength of clays in the laboratory.

Summary

Most of the past research on slope stability conducted by the Center for Transportation Research has focused on the stability of embankments constructed of highly plastic clays. Most of these embankments have been founded on firm ground and failures have been restricted to relatively shallow depths (10feet or less) in the compacted fill. The compacted highly plastic fills are generally very strong immediately after construction. The factors of safety at the end of construction probably exceed 2, but the soil tends to soften and weaken over time and the factor of safety decreases to values that approach 1, i.e., failure. The softening is probably enhanced by the repeated expansion and shrinkage that accompany seasonal wetting and drying of the soil, respectively. Studies of the long-term, fully-softened strength of the soil show that it is best characterized by a Mohr failure envelope that is curved, rather than linear. The failure envelope for fully-softened conditions lies below the failure envelope for the soil immediately after compaction.

A relatively small number of slope failures have been examined where the failure occurred during or shortly after construction. In these cases the failure was attributed to a weaker foundation with relatively low undrained shear strength. The strength of the foundation can be determined from unconsolidated-undrained (UU) triaxial compression tests performed in the laboratory on undisturbed specimens. Vane shear tests may also be used, but they tend to overestimate the shear strengths and require that a correction factor be applied before the shear strength is used in slope stability analyses.

Specific guidelines and recommendations for determining the undrained and drained shear strengths are presented in the following two chapters, respectively.

Chapter 4 – Guidelines for Determining Undrained (Short-Term) Shear Strengths

Introduction

Undrained shear strengths generally control stability when an embankment is founded on a relatively weak clay foundation that will consolidate and become stronger with time. In the case of the embankment itself, and particularly many embankments constructed of highly plastic clays, the embankment can become weaker with time and the eventual drained—rather than undrained—shear strength will control the stability. Similarly for excavated slopes, the removal of load by excavation may cause the soil to become weaker with time as the soil expands and the long-term, drained strength will control. Undrained shear strengths are primarily of interest for the foundations of compacted fills and some excavated slopes in very soft, normally consolidated or slightly overconsolidated clays.

Undrained shear strengths are usually measured in triaxial compression tests employing unconsolidated-undrained (UU) test procedures. In the following discussion this approach is assumed except where specifically noted otherwise.

For most cases where the undrained shear strength controls the stability of an embankment the foundation is saturated or nearly saturated. In this case the friction angle is considered to be zero and the undrained shear strength is expressed by a cohesion value (c) with $\phi = 0$. For unsaturated soils the strength will depend to varying degrees on the total confining pressure. The friction angle will be some finite, but usually small, value for the range of stresses applicable to typical TxDOT embankments, i.e., for embankments usually no more than 30 feet in height.

Factors Influencing Undrained Shear Strengths

Several factors influence the undrained shear strength of clays and may need to be taken into account when performing tests, interpreting data and selecting design values. These include the following:

- Sample disturbance
- Specimen size
- Anisotropy
- Creep

- Testing Procedure

Each of these is discussed further below.

Sample Disturbance

Removal of samples from the ground and handling them while transporting to the laboratory and setting then up for testing generally reduces the shear strength of clays and clayey soils. This effect can be particularly pronounced on the undrained shear strength and generally has a much lesser effect on the drained shear strength of clays. Undrained shear strengths measured on samples of Chicago clay obtained from block samples and from Shelby tube samples are compared in Figure 4.1. The data suggest that the smaller Shelby tube samples yielded strengths that were approximately 70 percent of the strengths measured for specimens carefully trimmed from larger block samples, presumably due to effects of sample disturbance.

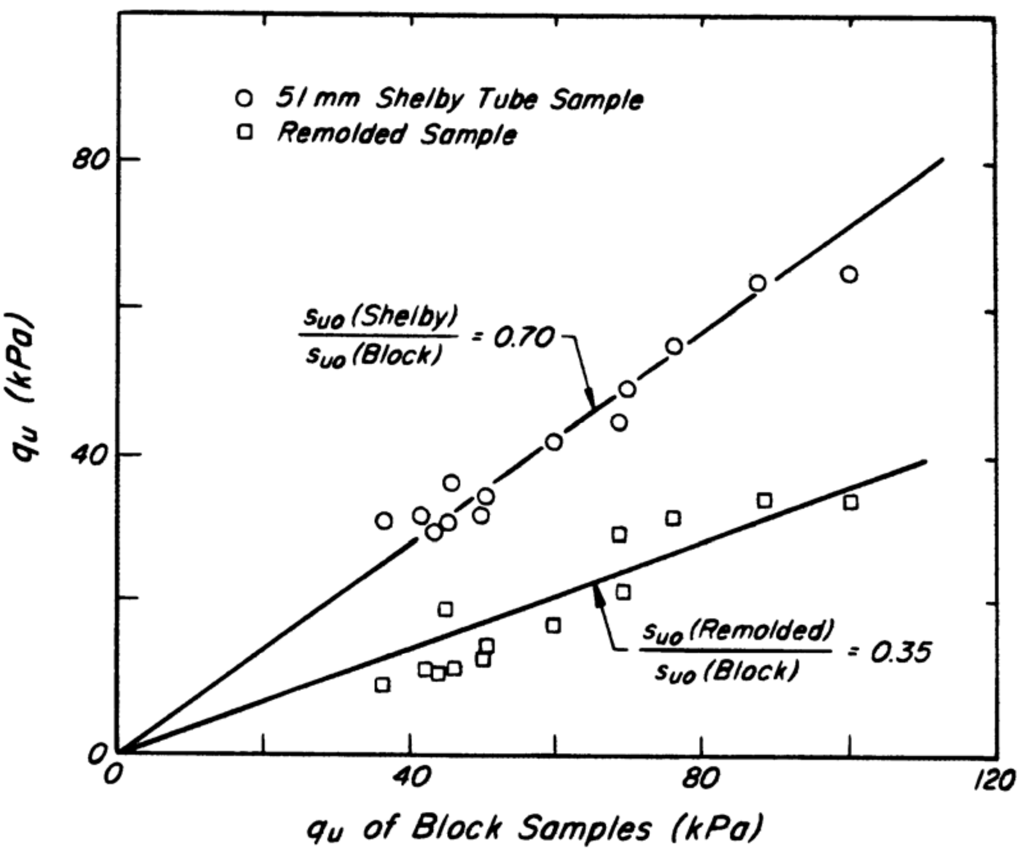


Figure 4.1 Effect of sample disturbance on undrained shear strength of Chicago clay (from Terzaghi, Peck and Mesri, 1996).

Specimen Size

The size of the test specimen can have a significant effect on the shear strength, particularly in stiff-fissured clays. For example, Peterson et al. (1960) reported the data shown in Table 4.1 for the Bearpaw Shale, a heavily overconsolidated, stiff-fissured clay. They performed a relatively large number of unconfined compression tests on 1.4-inch and 6-inch diameter specimens. Average strengths determined for the two different sizes of specimens are summarized in the table for both a "medium" and "hard" zone of the shale. Depending on the specimen size the strengths differed by almost as much as a factor of 6 (e.g., 300 ÷ 53)!

Table 4.1 Summary of unconfined compressive strengths for Bearpaw Shale

Description	Unconfined Compressive Strength, q_u (psi)	
	1.4-inch diameter specimens	6-inch diameter specimens
Medium zone	53 (22*)	20 (16*)
Hard zone	300 (34*)	50 (23*)

* Numbers in parentheses represent the number of specimens tested.

In many cases the effect of specimen size can be anticipated by a close examination of the soil involved. The Bearpaw shale is fissured and the writer has examined samples of the shale which revealed a spacing between fissures of several inches—typically of the order of 2 inches. Consequently it is not difficult to understand why a specimen of less than two inches in diameter (e.g., 1.4 inches in diameter) might yield a higher strength than a specimen that is several times larger (e.g., 6 inches in diameter) and may include fissures that the smaller specimen does not.

Anisotropy

Many clays exhibit some degree of anisotropy in their undrained shear strength such that the strength differs depending on the orientation of the shear plane. Most laboratory compression tests are performed on cylindrical specimens with the longitudinal axis of the specimens oriented vertically with respect to the field and, thus, the laboratory tests measure a shear strength for a failure plane inclined at approximately 60 degrees from the horizontal, e.g., $45^o + \phi'/2 = 45^o + 30^o/2 = 60^o$. If, instead of testing vertical specimens, specimens are tested with their axes inclined relative to the field such that failure occurs along different planes, the undrained shear strength may be different.

Typical undrained shear strength data from specimens with their axes inclined at different directions, ranging from vertical to horizontal, are shown in Figure 4.2. The strengths in this figure are plotted as normalized values by dividing the shear strength for specimens at various orientations by the shear strength measured on specimens with axes oriented in the vertical direction, i.e., with the conventional orientation. It can be seen that the shear strengths for specimens that are inclined at angles such that failure occurs along horizontal planes—generally corresponding to bedding planes—are less than the shear strengths measured on conventional vertical specimens.

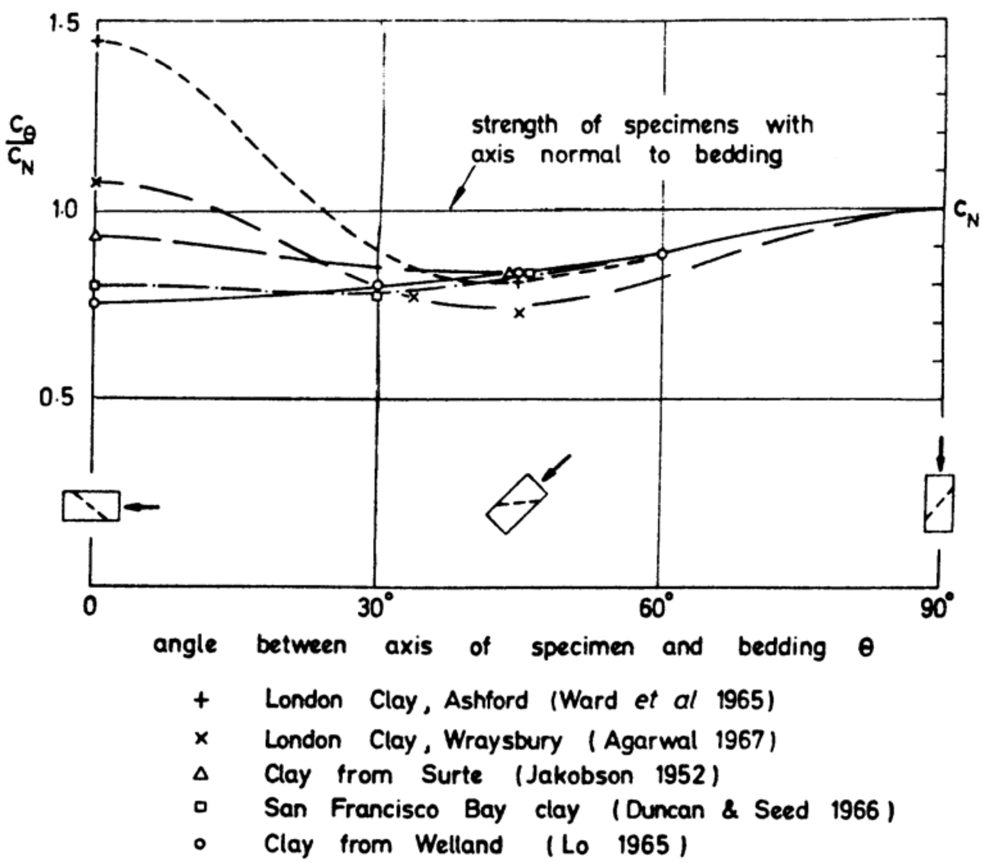


Figure 4.2 Effect of sample orientation on undrained shear strength (from Skempton and Hutchinson, 1969).

Data like that shown in Figure 4.2 is shown in non-normalized form in Figure 4.3 for Pepper Shale, a heavily overconsolidated, stiff-fissured clay from the foundation of Waco Dam near

Waco, Texas. These data may be of special interest because they represent clay from Texas. The data show that the shear strength of the Pepper Shale along a nearly horizontal failure plane is approximately 40 percent of the shear strength that was measured on vertical specimens. The Waco Dam was designed with a factor of safety of approximately 1.5 based on the strengths measured on vertical specimens, and the dam failed during construction by sliding in the foundation.

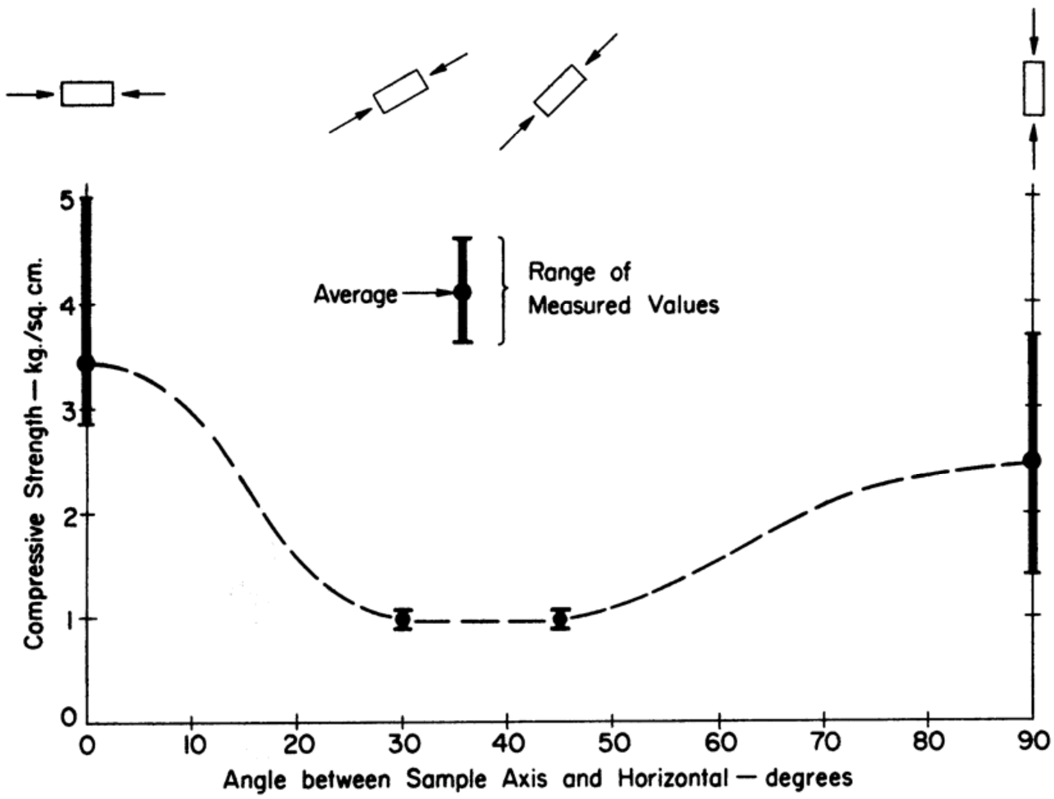


Figure 4.3 Effect of sample orientation on undrained shear strength of Pepper Shale from Waco Dam (from Wright and Duncan, 1972).

For many soils the potential for a high degree of anisotropy can be detected by simple inspection of specimens of the soil involved. For example, the Pepper Shale shows pronounced lamination along the horizontal bedding planes and could be anticipated to be anisotropic. In such cases the reduced strength along bedding planes due to the anisotropy should be accounted for when determining shear strengths.

Creep

Specimens subjected to sustained loads or loaded very slowly without drainage typically show lower strengths than those measured in conventional tests with a time to failure of generally much less than one hour. The lower strengths are attributed to creep, including the buildup of pore water pressures under the longer, sustained loads. However, such lower strengths measured under undrained conditions in the laboratory may not apply to the field where some drainage can occur. Consequently, creep can generally be ignored in determining undrained strengths, particularly if conventional factors of safety of at least 1.3 are used for design.

Testing Procedure

As mentioned at the beginning of this chapter the primary focus is on undrained shear strengths as measured in unconsolidated-undrained triaxial compression tests. However, vane shear tests are also often used to measure the undrained shear strength of soft clays. Such a practice has been successfully used in many places of the world and often represents the best way of determining the shear strength where sampling is difficult and significant disturbance can be expected. Experience with shear strengths measured with vane shear tests suggests that in many cases the vane shear test overestimates the undrained shear strengths, probably due to a number of factors. Based on this experience correction factors have been proposed and are commonly used to reduce the shear strength measured in the vane test before using it to compute slope stability. The most widely used correction factor is the one proposed by Bjerrum (1972). The correction factor depends on the plasticity index of the soil and is shown in Figure 4.4 along with additional data that were compiled after the original correction factor was proposed. Although some of the data show considerable variation from the correction proposed by Bjerrum, the correction on the average still appears to be reasonable. The shear strength measured in the vane shear tests is multiplied by the correction factor (μ) to obtain the corrected strength. Bjerrum's correction is recommended for application to all vane shear strength data.

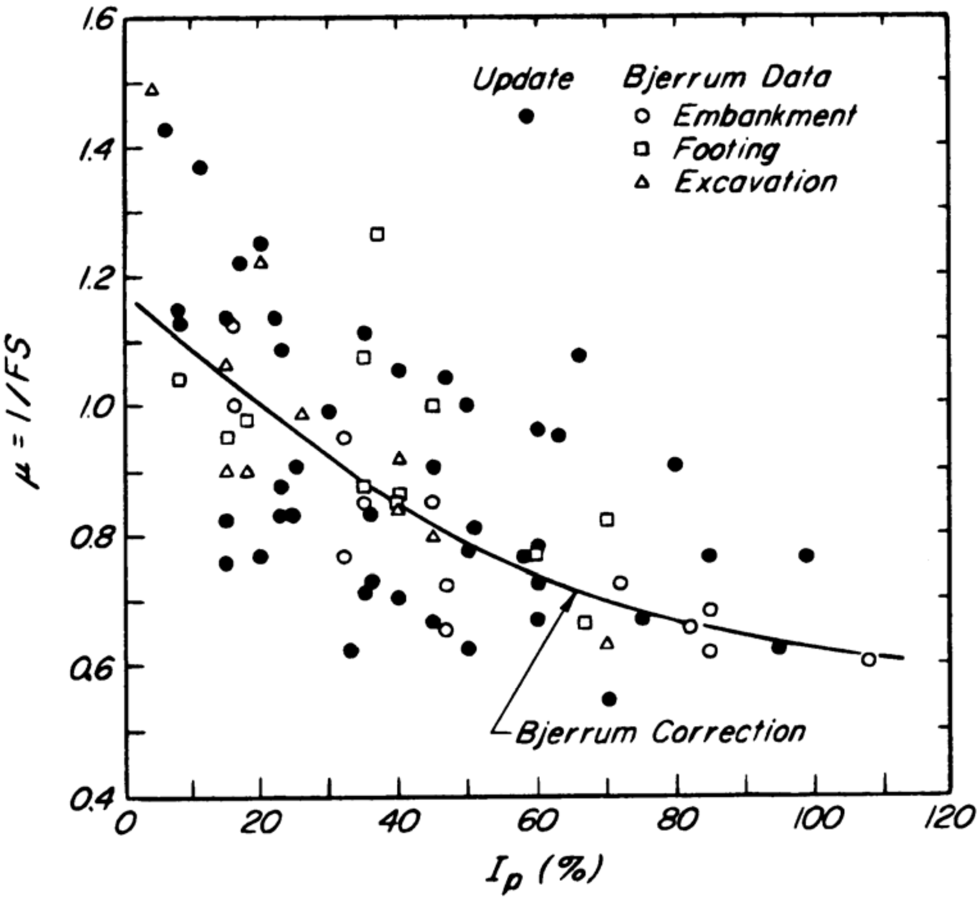


Figure 4.4 Bjerrum's correction factor for undrained shear strengths measured in field vane shear tests (from Terzaghi, Peck and Mesri, 1996).

Correlations for Estimating Undrained Strengths

Unconsolidated-undrained triaxial compression tests or vane shear tests with an appropriate correction factor applied are the preferred ways for determining the undrained shear strength of clays for slope stability and retaining wall foundation analyses. However, in many cases such testing is beyond the financial resources for a project or there is need to make a preliminary estimate of undrained shear strength before proceeding further. Accordingly, correlations between undrained shear strength and soil index properties that permit shear strengths to be estimated are attractive. Various correlations that have been proposed are thoroughly reviewed and presented in publications by Duncan et al. (1989) and Kulhawy and Mayne (1990). All of the correlations that the writer is aware of are for soils that are saturated, i.e., where $\phi = 0$ and the shear strength is expressed in terms of a cohesion value. Several of the more widely used and accepted correlations for estimating undrained shear strengths are reviewed and discussed in this section.

<u>Relationships to Effective Consolidation Pressure</u>

For many years it has been recognized that the undrained shear strength of a saturated, normally consolidated clay increases approximately linearly with depth (Figure 4.5) and with the effective consolidation pressure (e.g., Skempton, 1948). The increase in strength with effective stress is commonly expressed by a *c/p ratio*, defined as the ratio of undrained shear strength (s_u = c) to effective vertical stress (p, σ'_v). The ratio, c/p, provides a useful basis for characterizing the undrained shear strength of clays.

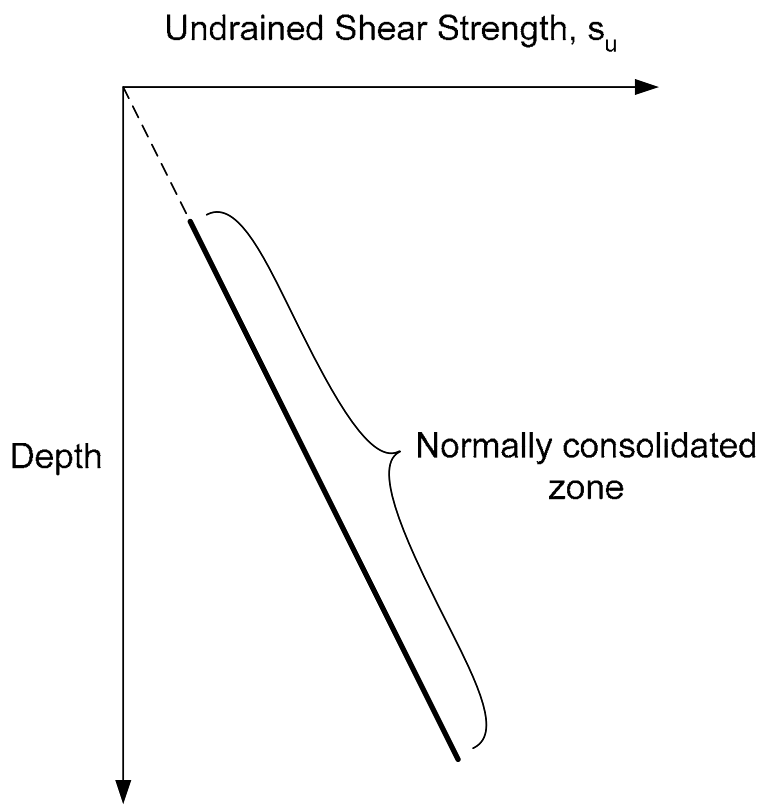


Figure 4.5 Typical variation in undrained shear strength with depth for a normally consolidated clay.

Various correlations have been suggested between the c/p ratio for normally consolidated clays and soil index properties. One of the first such correlations is the one suggested by Skempton (1948) between the c/p ratio and plasticity index illustrated in Figure 4.6. A more recent and complete correlation is the one shown in Figure 4.7 from Terzaghi, Peck and Mesri

(1996) and based on vane shear tests. Although there is considerable scatter in the data, Figure 4.7 can be used to estimate a c/p ratio for a normally consolidated soil. Given a c/p ratio, the shear strength at any depth can then be calculated by multiplying the c/p ratio by the present effective vertical stress.

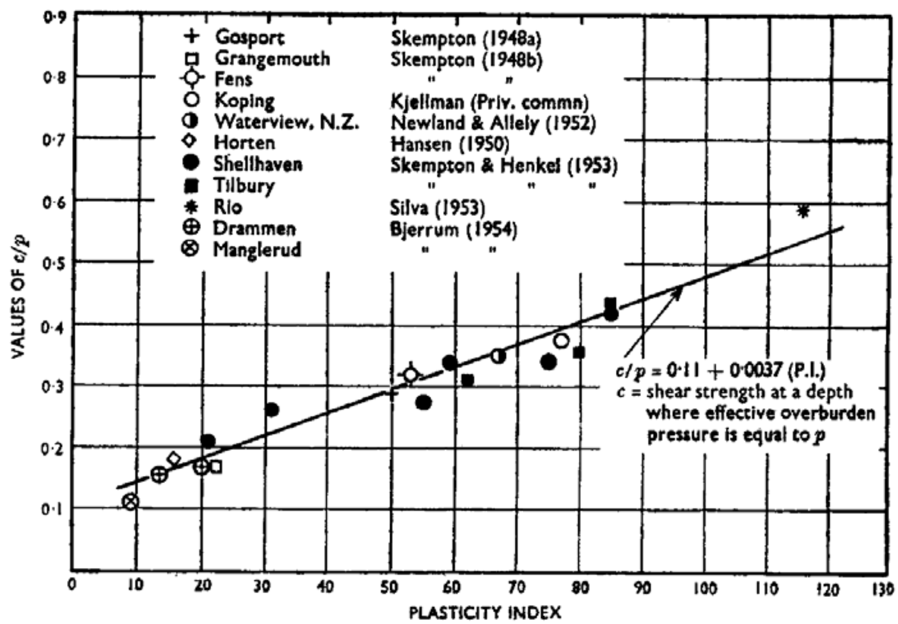


Figure 4.6 Relationship between c/p ratio and plasticity index suggested by Skempton (1948).

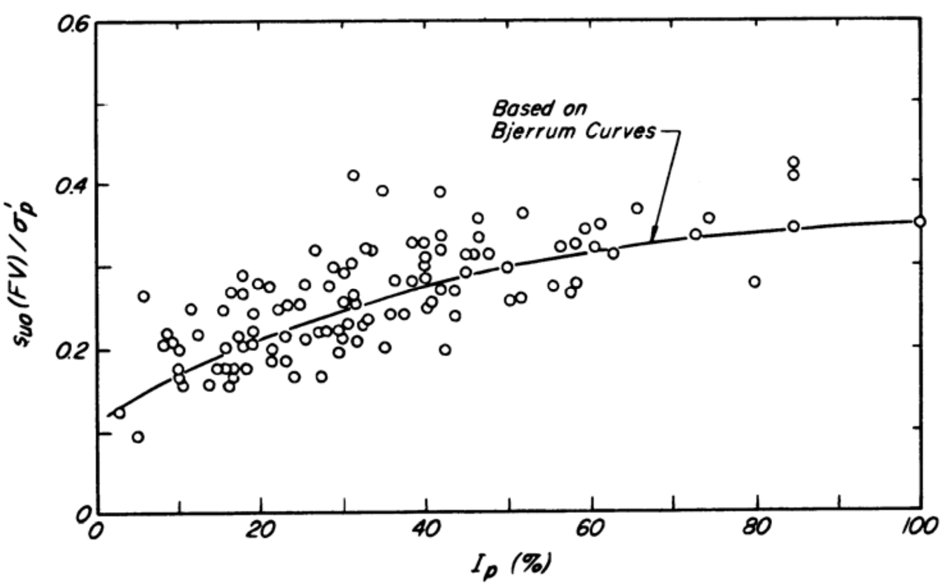


Figure 4.7 Relationship between c/p ratio and plasticity index

(from Terzaghi, Peck and Mesri, 1996).

The concept of a c/p ratio for normally consolidated clays can be extended to overconsolidated clays as well. Studies by Ladd and others (e.g., Ladd and Foott, 1974; Ladd et al. 1977) have led to the following empirical equation for the c/p ratio for overconsolidated clays:

$$\left(\frac{c}{p}\right)_{O_r} = \left(\frac{c}{p}\right)_{O_r=1} (O_r)^{0.8} \tag{4.1}$$

where O_r is the overconsolidation ratio, $(c/p)_{O_r=1}$ is the c/p ratio for a normally consolidated clay, and $(c/p)_{O_r}$ is the c/p ratio for clay with a given overconsolidation ratio (O_r). The overconsolidation ratio is defined as the maximum past vertical effective stress (σ'_{max})—sometimes called the "preconsolidation pressure"—divided by the present effective vertical stress (σ'_v), i.e.,

$$O_r = \frac{\sigma'_{max}}{\sigma'_v} \tag{4.2}$$

Equation 4.1 can also be written to express the undrained shear strength, s_u, of an overconsolidated clay in the form,

$$s_u = c = \left(\frac{c}{p}\right)_{O_r=1} (O_r)^{0.8} \sigma'_v \tag{4.3}$$

where σ'_v is the effective vertical stress in the field (actually the same as "p").

Additional studies have also shown that for clays with a plasticity index of less than 60 percent, Equation 4.3 can be further simplified to the following (Jamiolkowski et al. 1985):

$$s_u = 0.23\sigma'_{max} \tag{4.4}$$

Jamiolkowski et al. (1985) suggest that the "constant" (0.23) in Eq 4.3 will likely vary by ± 0.04. Independent studies and evaluation of the undrained shear strength of both normally consolidated and overconsolidated clays led Mesri (1989) to suggest the following equation:

$$s_u = 0.22\sigma'_{max} \tag{4.5}$$

Equations 4.4 and 4.5 are essentially identical, especially considering the variation in c/p ratio shown in Figure 4.7 for different soils and the empirical nature of these equations.

The above equations should produce comparable values for the undrained strength within the accuracy that can be expected of such empirical equations. To use any of Eqs. 4.1–4.5 the maximum past effective vertical stress (σ'_{max}) that the soil has been subjected to must be estimated. This is normally done using the results of one-dimensional consolidation tests. Estimates can also sometimes be made based on a knowledge of the prior stress and geologic history for a site.

Correlations with Standard Penetration Test Blow Count

Correlations have also been developed between undrained shear strength and various *in-situ* tests. Probably the most widely used among these are correlations between the undrained shear strength and the Standard Penetration Resistance, expressed by the blow count, N_{60}, representing the resistance for a Standard Penetration Test delivering the specified energy at an efficiency of 60 percent. Terzaghi, Peck and Mesri (1996) presented the following Table 4.2 of compressive strengths and Standard Penetration Resistance, N_{60}.

Table 4.2 Relation of Number of Blows (N_{60}) and Unconfined Compressive Strength (q_u) [from Terzaghi, Peck and Mesri, 1996]

Blow Count, N_{60}	**Unconfined Compressive Strength, q_u (kPa)**
<2	25
2–4	25–50
4–8	50–100
8–15	100–200
15–30	200–400
> 30	> 400

The values shown in this table correspond approximately to the following equation:

$$s_u = 0.063\ N_{60} \tag{4.6}$$

where s_u is the undrained shear strength (= $q_u/2$) in kPa. Equation 4.6 can also be written as

$$s_u = 130\ N_{60} \tag{4.7}$$

where s_u is in pounds per square foot (psf).

Equations 4.6 and 4.7 provide a simple and useful means of making estimates of the undrained shear strength for saturated clays. The estimates are very approximate. Other, different correlations have been suggested as well. For example Kulhawy and Mayne (1990) present a summary of various correlations between undrained shear strength and Standard Penetration Resistance that was originally presented by Djoenaidi (1985). This summary is shown below in Figure 4.8. The undrained shear strengths (s_u) shown in this figure are expressed as normalized values by dividing them by atmospheric pressure, p_a. Equations 4.6 and 4.7 correspond to a normalized undrained shear strength (s_u/p_a) of approximately 0.062 per blow, N. Thus, for an SPT N value of 25 the value of s_u/p_a according to Eqs. 4.6 and 4.7 is approximately 1.55 (= 0.062 x 25).Comparison of this value (1.55) with the values suggested by the various correlations shown in Figure 4.8 for N = 25 suggests that Eqs. 4.6 and 4.7 yield undrained shear strengths that are on the lower side of those from the various other correlations and, thus, are on the conservative side. Due to the very approximate nature of relationships between undrained shear strength and Standard Penetration Resistance N value, the correlations expressed by Eqs. 4.6 and 4.7 are recommended.

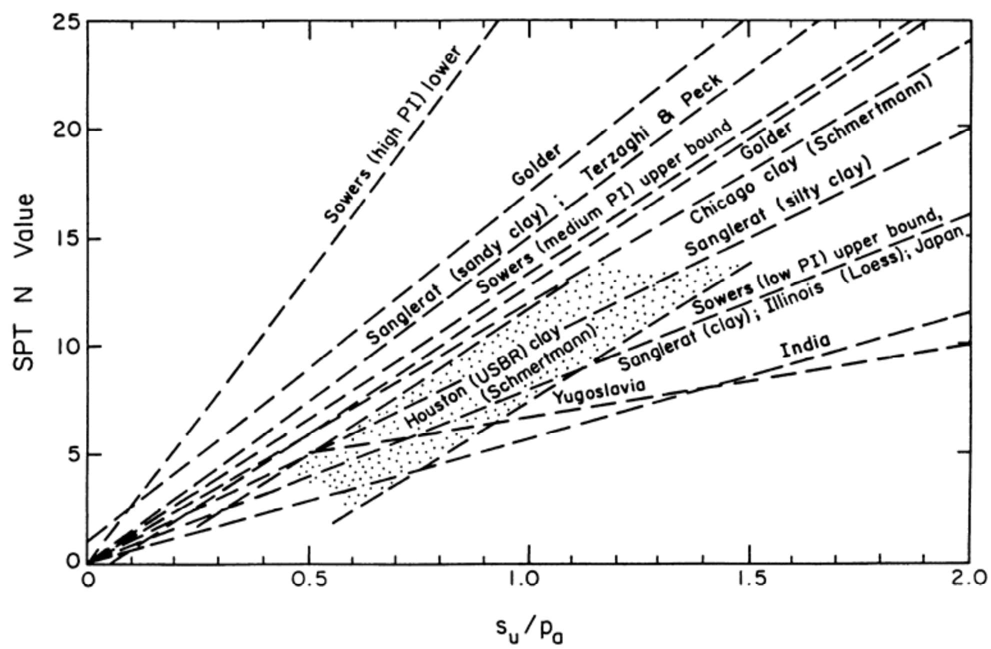


Figure 4.8 Relationships between undrained shear strength and Standard Penetration Resistance N values (from Kulhawy and Mayne, 1990; based on Djoenaidi, 1985).

Summary

Several different approaches have been discussed for determining the undrained shear strength of saturated clays. These are listed in order of preference based on accuracy and reliability as follows:

- Unconsolidated-undrained (UU) triaxial compression tests on undisturbed samples.
- Vane shear tests with an appropriate correction factor applied.
- Estimates based on empirical equations (4.1 through 4.5) that relate undrained shear strength to effective consolidation pressures.
- Correlations with Standard Penetration Resistance blow counts (Eqs. 4.6 and 4.7).

Effects of sample disturbance, sample size, and anisotropy should be considered when basing shear strengths on the results of unconsolidated-undrained triaxial compression tests. Unconfined compression tests may also be used in place of UU triaxial tests; however, they may significantly underestimate the undrained shear strength, especially for heavily overconsolidated, stiff-fissured clays.

There are other correlations for undrained shear strength that relate the undrained shear strength to measurements made with various in-situ tests such as the static cone penetrometer. However, these in-situ tests do not appear to have received widespread use by TxDOT and the correlations are not considered in this report. The reader is referred to Duncan et al. (1989) and Kulhawy and Mayne (1990) for further details on such correlations.

Chapter 5 – Guidelines for Determining Drained (Long-Term) Shear Strengths

Introduction

The stability of most excavated slopes and many low (less than 50 foot high) compacted fill slopes decreases with time following construction. These slopes reach their least stable condition some time after construction when the soil has expanded to its final equilibrium condition. The governing shear strength for stability of such slopes is the drained shear strength, which is represented by a Mohr failure envelope expressing shear strength as a function of effective normal stress.

Drained shear strengths can be measured in triaxial compression tests employing either consolidated-drained (CD) testing procedures or consolidated-undrained (CU[3]) test procedures with pore water pressure measurements. Strength envelopes are plotted as a function of effective normal stress and both types of tests (CD, CU) yield essentially identical shear strength envelopes. In triaxial tests on clay soils consolidated-undrained test procedures are usually preferred over consolidated-drained procedures because the tests can be performed faster — time is not required for drainage during shear in CU tests.

Direct shear tests can also be used to measure the drained shear strength. As with all drained tests, the direct shear tests must be performed slowly enough to allow the soil to drain completely, i.e., for water to freely flow into or out of the specimen as it is sheared. Failure to perform drained tests slowly enough can result in the measured strength being significantly higher and, thus, unrealistically large values may result. This is particularly a problem with highly plastic clays that are either densely compacted or heavily overconsolidated.

Three different shear strength envelopes can be defined and used to represent the drained (long-term) shear strength of clay soils. Each envelope expresses the relationship between shear strength and effective normal stress on a Mohr diagram. The three envelopes represent (1) the *peak* strength, (2) the *fully-softened* strength and (3) the *residual* shear strength. These envelopes are discussed separately in the following sections.

---

[3] The designation $\overline{\mathrm{CU}}$ is also sometimes used for these tests to distinguish tests where pore water pressures are measured from those where no pore water pressure measurements are made.

Peak Shear Strength

The peak shear strength represents the peak load that the soil can carry under drained conditions. Various correlations have been proposed between the peak drained shear strength of clays and soil index properties, often the plasticity index. Virtually all these correlations express the strength in terms of a friction angle (ϕ') assuming zero cohesion ($c' = 0$). Although many soils exhibit either some cohesion intercept or the failure envelope is curved suggesting that the values c' and/or ϕ' must vary with stress, the correlations for peak strength ignore this effect and consider only a single value of the friction angle to represent the drained shear strength. A typical relationship between the peak friction angle (ϕ') and plasticity index for normally consolidated clays is shown in Figure 5.1.

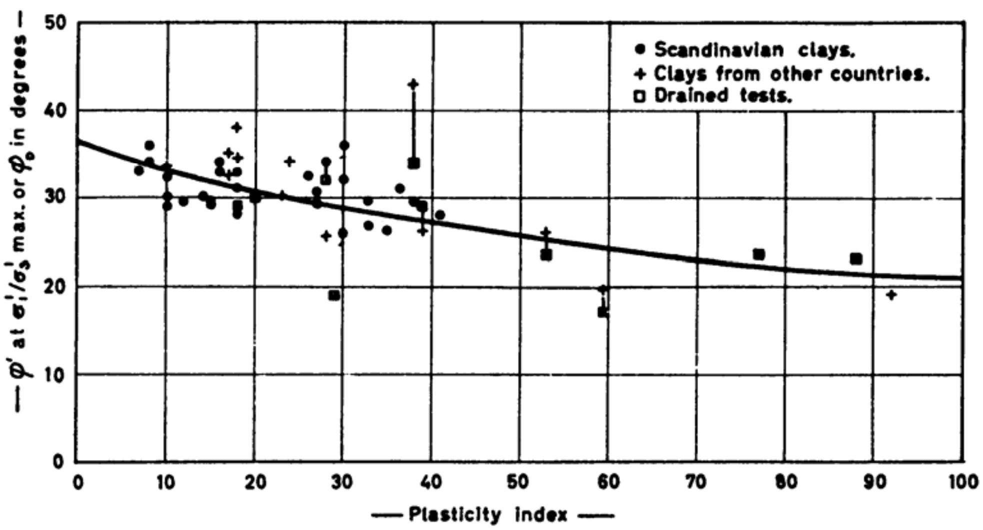


Figure 5.1 Relationship between peak effective friction angle (ϕ') and Plasticity Index for normally consolidated clays (from Bjerrum and Simons, 1960).

Kanji compiled correlations between friction angle (ϕ') and plasticity index from a number of different sources. Kanji's compilation is summarized in Figure 5.2. A relatively wide range in values can be seen for the various relationships with a range in values for a given plasticity index being at least 10 degrees.

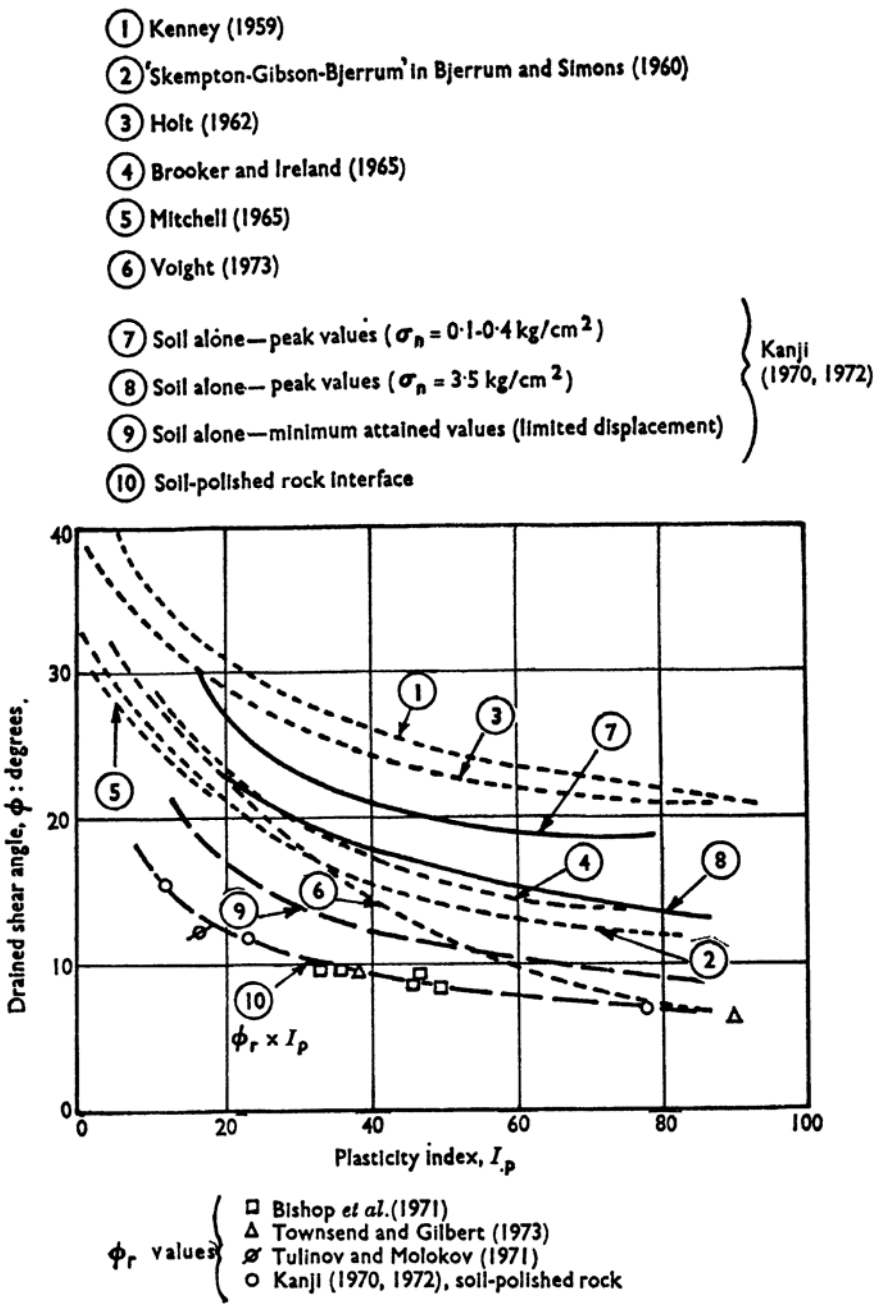


Figure 5.2 Relationships between peak effective friction angle (ϕ′) and Plasticity Index compiled from various sources (from Kanjii, 1974).

It does not appear that many of the slope failures experienced in Texas were governed by peak strengths and, thus, correlations like those shown in Figures 5.1 and 5.2 are probably of relatively little interest. Peak strengths may be applicable to stability of slopes in clays of lower plasticity, e.g., with Liquid Limits less than 50; however, there is not sufficient data to verify this for the soils in Texas. In addition to clays of low PI, peak strengths may be of some interest for highly plastic soils because they serve to establish probable upper bounds on the drained shear strength.

Fully-Softened Shear Strength

The fully-softened shear strength corresponds to the shear strength that many soils, especially those with a high PI, seem to develop over time. The concept of a fully-softened strength appears to have been first suggested for natural and excavated slopes in London Clay by Skempton and his co-workers. Skempton (1977) suggested that the fully-softened strength was equivalent to the strength of the soil in a normally consolidated state.

Previous research performed for TxDOT and reviewed in Chapter 3 suggests that the fully-softened strength may also be applicable to compacted fills constructed of highly plastic clays. It has been shown that the fully-softened strength as measured for normally consolidated clay is essentially identical to the strength that the soil develops after repeated cycles of wetting and drying. Wetting and drying therefore produces the same type of softening in compacted fills as has been observed in slopes of natural deposits of London clay.

It is not clear that the softening observed in highly plastic compacted clays is due exclusively to repeated wetting and drying. There may be other environmental factors that cause softening even in the absence of repeated wetting and drying. Even if there are other factors that contribute to the softening of the soil, they are expected to require longer times to reach the fully-softened strength than when soils are subjected to repeated wetting and drying. This is based on the writer’s observation that soils in highly plastic clay embankments in Texas that are protected from repeated wetting and drying, e.g., by concrete rip-rap, do not seem to lose strength to the same extent as soils that are subjected to wetting and drying. Further research is required to verify this, but for now this knowledge can be used in an empirical way to mitigate strength losses, i.e., by reducing the extent of repeated wetting and drying, it is believed that the softening process can be delayed or reduced, but probably not eliminated.

Laboratory Tests

Based on the research described in Chapter 3, the fully-softened shear strength can be measured by testing either specimens that are normally consolidated or compacted specimens that have been subjected to repeated wetting and drying with corresponding shrinkage and expansion. Normally consolidated clay specimens can be prepared by mixing soil and water to form a slurry and then one-dimensionally consolidating the slurry to relatively low stresses, i.e., to avoid overconsolidating specimens. Experience with preparing and testing specimens of highly plastic clay either as normally consolidated specimens or compacted specimens that are subjected to repeated wetting and drying indicate that relatively long times are required. The tests on Beaumont and Paris clays described in Chapter 3 typically took almost a month to prepare specimens and then test them in the triaxial apparatus. Accordingly, laboratory testing will most likely be restricted to research studies or perhaps large projects with both time and financial resources to permit such testing.

Correlations by Stark and Co-Workers

Stark and his co-workers (Stark and Eid, 1997; Stark et al. 2005) examined a large amount of data for the fully-softened shear strength of clays. They developed relationships between the secant friction angle for fully-softened conditions (ϕ'_{secant}) and the liquid limit of the soil. The relationships depend on the effective normal stress on the failure plane (σ') and the clay content. The most recent such relationships are shown below in Figure 5.3.

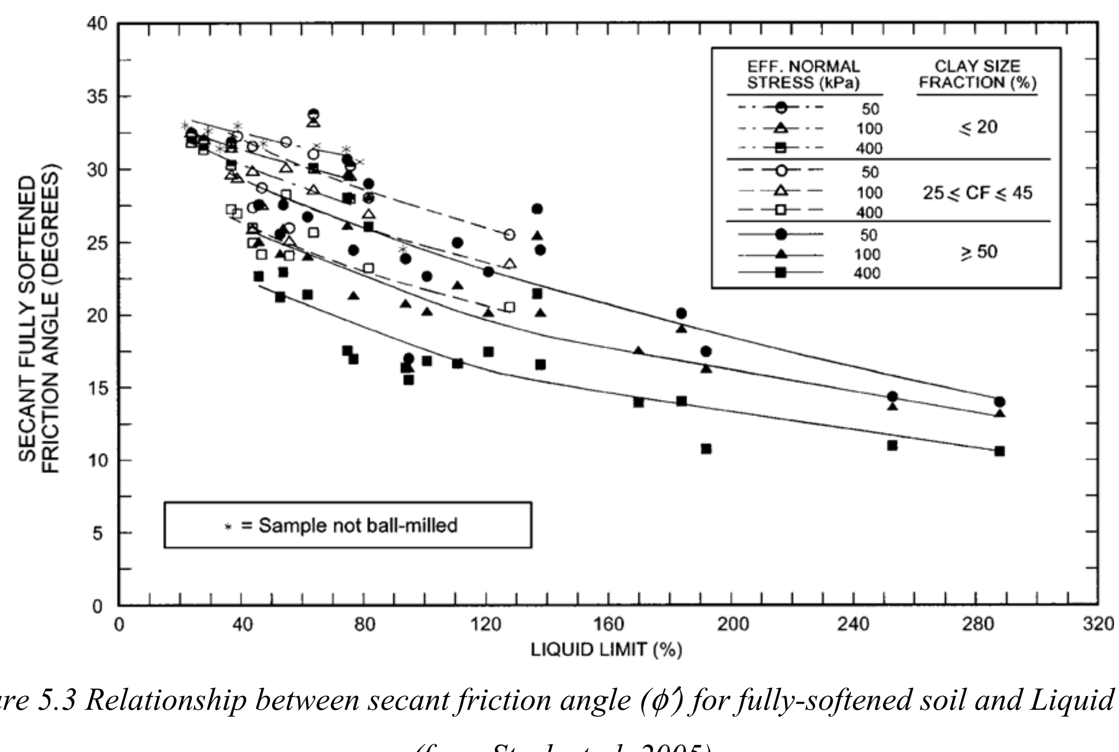


Figure 5.3 Relationship between secant friction angle (ϕ′) for fully-softened soil and Liquid Limit (from Stark et al. 2005).

The relationships developed by Stark et al. (2005) are believed to be the most representative correlations available between the fully-softened shear strength and soil index properties.

Saleh and Wright (1997) used the earlier correlation by Stark and Eid (1997) coupled with data for the Paris and Beaumont clays reported by Kayyal and Wright (1991) to develop the suggested relationships shown earlier in Figure 3.4. The primary difference between Saleh and Wright's relationships and the ones by Stark and Eid is that Saleh and Wright extended the range of the relationships to lower effective stresses corresponding to stresses more typical for the shallow slides experienced by TxDOT. Otherwise, the two sets of relationships are essentially the same. The relationships developed by Saleh and Wright for highly plastic clays are still believed to be reasonable and valid. Such relationships are examined further, including incorporation of data from the recent paper by Stark et al. (2005) in the next section.

Extension and Modification to Correlations by Stark and Co-Workers

The relationships for shear strength presented by Stark et al. (2005) and shown in Figure 5.3 are believed to be reasonable for estimating the fully-softened shear strengths of highly plastic clays. The relationships acknowledge that the Mohr failure envelope is curved and, thus, the (secant) friction angles vary with the effective normal stress on the failure plane. Curved failure envelopes are also consistent with the failure envelopes presented by Kayyal and Wright (1991) for the Beaumont and Paris clays and discussed earlier in Chapter 3.

The relationships developed by Stark et al. can also be simplified further and expressed by an empirical equation based on related work by Duncan et al. (1989). Duncan and his co-workers have shown that for cohesionless soils where the failure envelope is curved, it is convenient to express the shear strength by a secant friction angle, ϕ'_{secant}. The secant friction angle varies with the logarithm of the effective confining pressure, σ'_3[4], which can be expressed by an equation of the form,

$$\phi'_{secant} = \phi'_0 - \Delta\phi' \log_{10}\left(\frac{\sigma'_3}{p_a}\right) \qquad (5.1)$$

where, ϕ'_0, is the secant friction angle at an effective confining pressure (σ'_3) of 1 atmosphere, $\Delta\phi'$ is the change (reduction) in the secant friction angle with each ten-fold increase in confining pressure, and p_a represents atmospheric pressure. Atmospheric pressure (p_a) is used as a

[4] Duncan et al. worked with data from triaxial tests where it was convenient to use the effective confining pressure, σ'_3, rather than the effective normal stress on the failure plane, σ'_f. However, the two stresses are closely related.

convenient reference for stresses, making the value for $\Delta\phi'$ independent of the particular units used. Duncan et al. present typical values of ϕ'_0 and $\Delta\phi'$ for peak strengths of coarse-grained soils and low plasticity (CL) clays.

Equations of the form of Eq 5.1 are convenient for characterizing Mohr failure envelopes when the envelopes are curved because they allow the strength of a given soil to be represented by just two parameters, ϕ'_0 and $\Delta\phi'$. In order to investigate the possibility of such a relationship for the fully-softened strength of the highly plastic clays of current interest, data for the nonlinear, fully softened Mohr failure envelopes from Kayyal and Wright (1991) were replotted in the form shown in Figure 5.4. Except for the data for Paris Clay at the lowest normal stress (approximately 20 psf) the data for each soil show a nearly linear relationship between the friction angle and the logarithm of the effective normal stress. The data for the Paris Clay at the lowest normal stress were obtained for a specimen that was consolidated before shear in the triaxial apparatus to an effective normal stress of approximately 2 psi. This stress (2 psi) is identical to the stress to which the specimens were previously consolidated in the one-dimensional consolidation tubes and the specimen may have behaved as a slightly overconsolidated soil[5]. Thus, the data for Paris Clay at the lowest normal stress (20 psf at failure) were ignored in fitting lines to the data. The lines shown in Figure 5.4 were obtained by a least squares fit. The lines shown can be represented by an equation of the form

$$\phi'_{secant} = \phi'_0 - \Delta\phi' \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \tag{5.2}$$

where σ'_f is the effective normal stress on the failure plane, ϕ'_0 is the secant friction angle at an effective normal stress (σ'_f) of 1 atmosphere, and $\Delta\phi'$ is the change (reduction) in the secant friction angle with each ten-fold increase in effective normal stress. Values of the parameters ϕ'_0 and $\Delta\phi'$ obtained by fitting Eq 5.2 to the data in Figure 5.4 are summarized in Table 5.1. The relatively large values shown in this table for the coefficients of determination (R^2) of 0.96-0.99 suggest that Eq 5.2 provides an excellent representation of the data.

[5] Specimens must generally be reconsolidated to stresses greater than their maximum past pressure before they show normally consolidated behavior (e.g., Ladd and Foott, 1977).

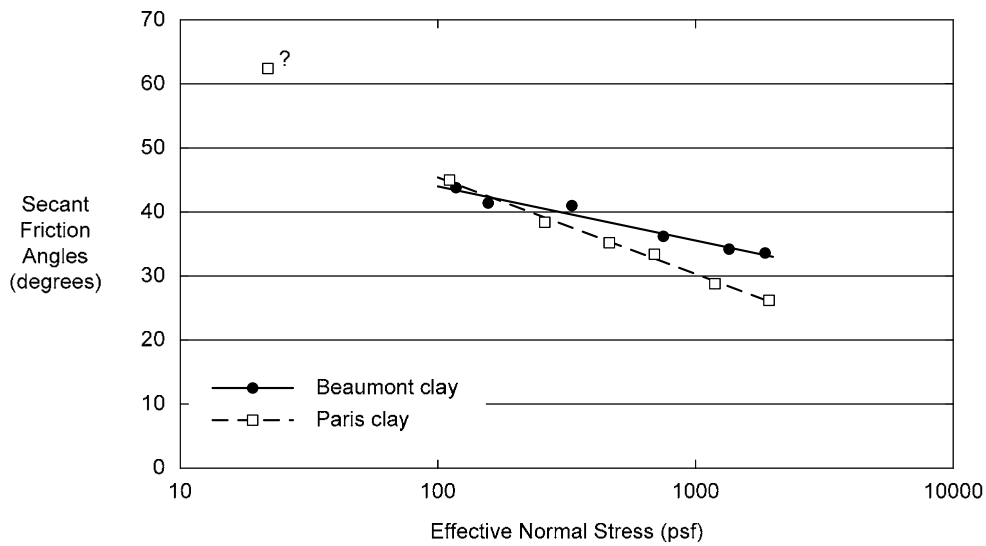


Figure 5.4 Relationship between secant friction angle and effective normal stress on the failure plane for Beaumont and Paris clays.

Table 5.1 Strength parameters ϕ_0 and $\Delta\phi$ obtained by least squares curve fitting to data for Beaumont and Paris Clays.

Parameter	Beaumont Clay	Paris Clay
"Intercept" value, ϕ'_0	32.9 degrees	25.4 degrees
"Slope" value, $\Delta\phi'$	8.4 degrees	15.0 degrees
Coefficient of determination, R^2	0.96	0.99

Given the excellent ability of Eq 5.2 to fit the data for the fully-softened shear strength of the Beaumont and Paris Clays, the data presented by Stark et al. (2005) for fully-softened shear strengths were examined further. First, the actual data presented in Figure 5.3 for soils with a clay fraction of 50 percent or greater were digitized. For all but two of the sets of data (two soils) this provided three values of the secant friction angle corresponding to three different values of effective normal stress (σ' = 50, 100 400 kPa). Equation 5.2 was then fit to the three values for each soil using a least squares approach. Finally, the values of ϕ'_0 and $\Delta\phi'$ obtained for each soil were plotted versus the liquid limit as shown in Figures 5.5 and 5.6, respectively.

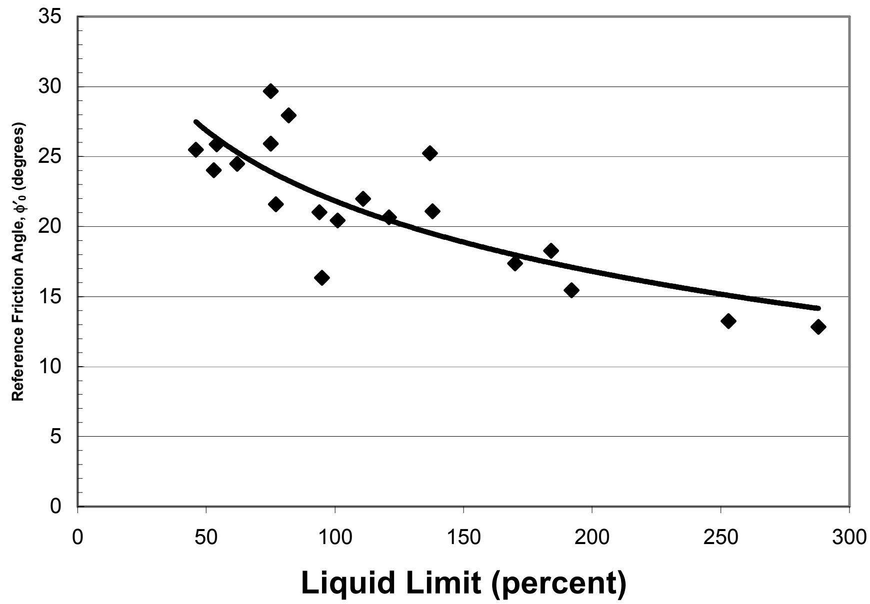


Figure 5.5 Variation in parameter ϕ'_0 with liquid limit calculated from Stark et al. (2005) data set.

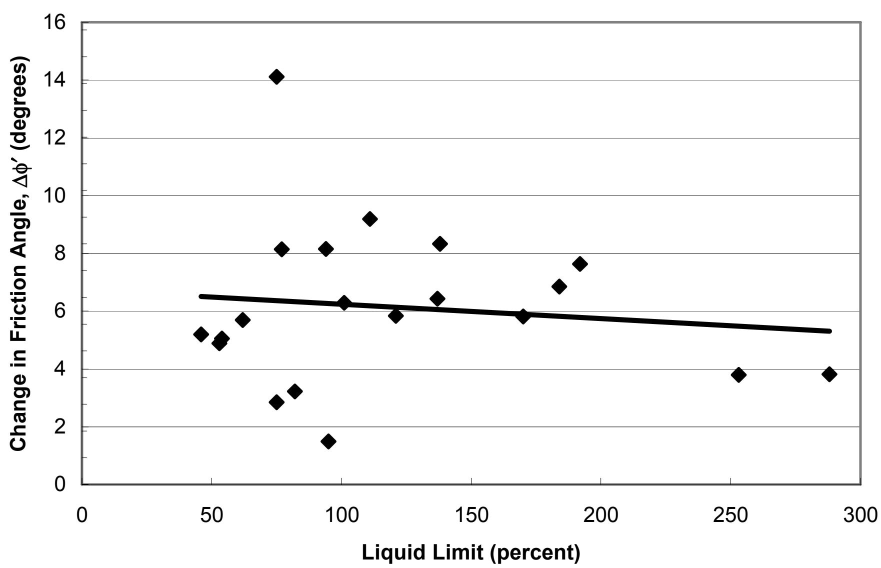


Figure 5.6 Variation in parameter $\Delta\phi'$ with liquid limit calculated from Stark et al. (2005) data set.

Although there is noticeable scatter in the values of ϕ'_0 shown in Figure 5.5, there appears to be a trend toward decreasing values with increasing liquid limit. This is consistent with other correlations that generally show friction angles decrease with increases in either liquid limit or plasticity index. Examination of the data in Figure 5.5 suggests that ϕ'_0 decreases at a decreasing rate as the liquid limit increases. Based on this observation a relationship between ϕ'_0 and the logarithm of liquid limit was assumed and a curve was fit to the data by the method of least squares. The resulting curve is shown in Figure 5.5 and is expressed by the equation:

$$\phi_0' = 55.3° - 16.7° \log_{10}(w_{LL}) \tag{5.3}$$

where w_{LL} represents the liquid limit of the soil in percent.

The values for the parameter $\Delta\phi'$ shown in Fig. 5.6 show considerable scatter. No apparent trend in these values with liquid limit is evident. Accordingly, a single, constant value of 6 degrees for $\Delta\phi'$ was selected as a reasonable, representative value. Inserting this value ($\Delta\phi'$ = 6 degrees) and Eq 5.3 into Eq 5.2 gives the following approximation for the secant friction angle:

$$\phi'_{secant} = 55.3° - 16.7° \log_{10}(w_{LL}) - 6° \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \tag{5.4}$$

To examine the validity of Eq 5.4 values of the secant friction angle were calculated for each of the highly plastic soils and the three normal stresses (50, 100, and 400 kPa) reported by Stark et al. Values of the secant friction angle for the three normal stresses were also estimated from the curves presented by Stark et al. The values of the secant friction angles from the measured data as well as the values from Stark et al.'s curves and Eq 5.3 are summarized in Table 5.2. As a measure of the errors associated with the calculated and estimated values, the sum of the squares of the errors (difference between measured and calculated or estimated values) were calculated for the values estimated from both Stark et al.'s curves and Eq 5.4. These values are summarized in Table 5.3. The values in Table 5.3 indicate that Eq 5.4 gives a slightly poorer estimate of the secant friction angle at low stresses (50 kPa), but a slightly better estimate than Stark et al.'s curves at higher stresses of 100 and 400 kPa.

Table 5.2 Summary of measured values of secant friction angle and values estimated from Stark et al.'s curves and Equation 5.4 developed in this study.

	Measured Data			Stark et al. Curves			Equation 5.4		
Liquid Limit - %	ϕ (50 kPa)	ϕ (100 kPa)	ϕ (400 kPa)	ϕ (50 kPa)	ϕ (100 kPa)	ϕ (400 kPa)	ϕ (50 kPa)	ϕ (100 kPa)	ϕ (400 kPa)
46	27.5	24.9	22.6	28.8	25.5	21.9	29.4	27.6	24.0
53	25.5	24.1	21.1	28.1	24.9	21.3	28.3	26.5	22.9
54	27.5	25.8	22.9	28.0	24.8	21.2	28.2	26.4	22.8
62	26.7	23.9	21.3	27.4	24.1	20.6	27.2	25.4	21.8
75	30.6	29.6	28.0	26.3	23.1	19.5	25.8	24.0	20.4
75	-	26.0	17.5	-	23.1	19.5	-	24.0	20.4
77	24.4	21.2	16.9	26.2	22.9	19.4	25.6	23.8	20.2
82	28.9	28.0	26.0	25.8	22.5	18.9	25.2	23.4	19.8
94	23.8	20.7	16.3	24.9	21.5	18.0	24.2	22.4	18.8
95	16.9	16.2	15.5	24.8	21.4	17.9	24.1	22.3	18.7
101	22.6	20.1	16.8	24.3	20.9	17.5	23.7	21.9	18.2
111	24.9	21.9	16.6	23.6	20.2	16.7	23.0	21.2	17.6
121	22.9	20.0	17.4	23.0	19.5	16.1	22.4	20.6	16.9
137	27.2	25.3	21.4	22.0	18.6	15.4	21.5	19.7	16.0
138	24.4	20.0	16.5	21.9	18.5	15.3	21.4	19.6	16.0
170	-	17.4	13.9	-	17.2	14.2	-	18.1	14.5
184	20.0	18.9	14.0	19.3	16.7	13.8	19.3	17.5	13.9
192	17.4	16.1	10.7	18.9	16.4	13.5	19.0	17.2	13.6
253	14.3	13.5	10.9	15.7	14.2	11.6	17.0	15.2	11.6
288	13.9	13.0	10.5	14.1	12.9	10.5	16.1	14.3	10.7

Table 5.3 Computed sum of squares of errors in secant friction angle estimated using Stark et al.'s curves and Equation 5.4 developed in this study.

Effective Normal Stress (kPa)	Stark et al. Curves	Equation 5.4
50	146	165
100	171	163
400	193	179

To gain further insight into the validity of Eq 5.4, the equation was used to calculate shear strengths for the Beaumont and Paris clays. Using the liquid limits of 73 and 80 for the Beaumont and Paris clays, respectively, secant friction angles were calculated for selected values of effective normal stress encompassing the range of values for the envelopes shown previously in Figures 3.2 and 3.3. The tangents of the secant friction angles were then multiplied by the corresponding values of normal stress to obtain values of shear stress, i.e., $\tau = \sigma' \tan\phi'_{secant}$. The values of shear stress calculated in this manner are plotted in Figures 5.7 and 5.8 for the Beaumont and Paris clays, respectively. The measured failure envelopes shown previously in Figures 3.2 and 3.3 are also plotted in these figures. Considering the approximations involved in correlating shear strengths with index properties (liquid limit), the agreement between the estimated and measured shear strength envelopes is considered good. In addition, the estimated strength envelopes for both soils lie below the measured envelopes, indicating that Eq 5.3 errs on the safe side for the two TxDOT soils, i.e., Eq 5.4 provides conservative estimates of the strength for these two Texas soils.

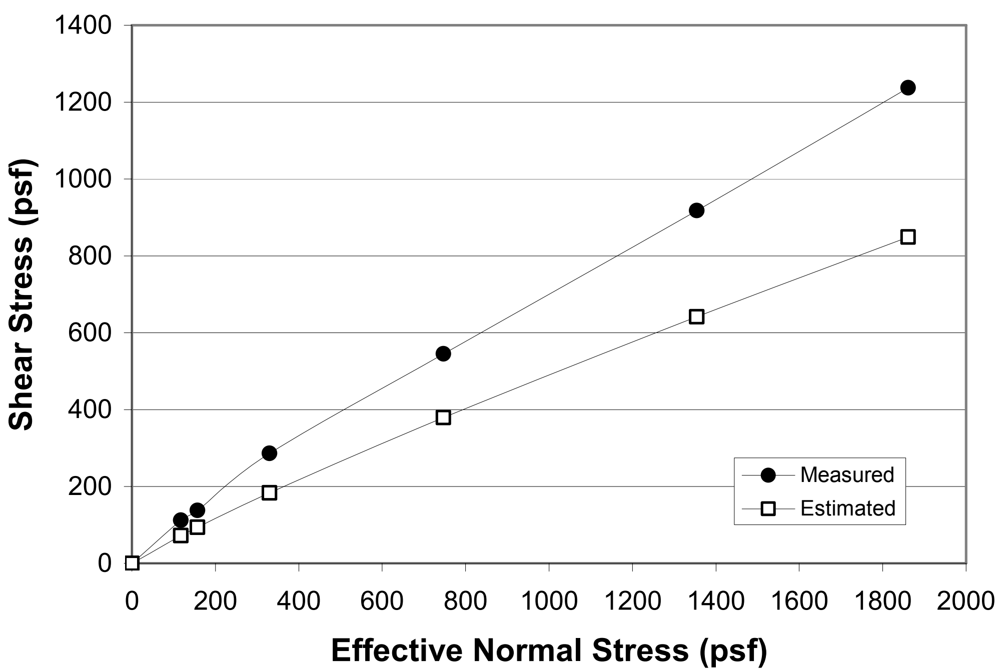


Figure 5.7 Measured and estimated fully-softened shear strength envelopes for Beaumont clay.

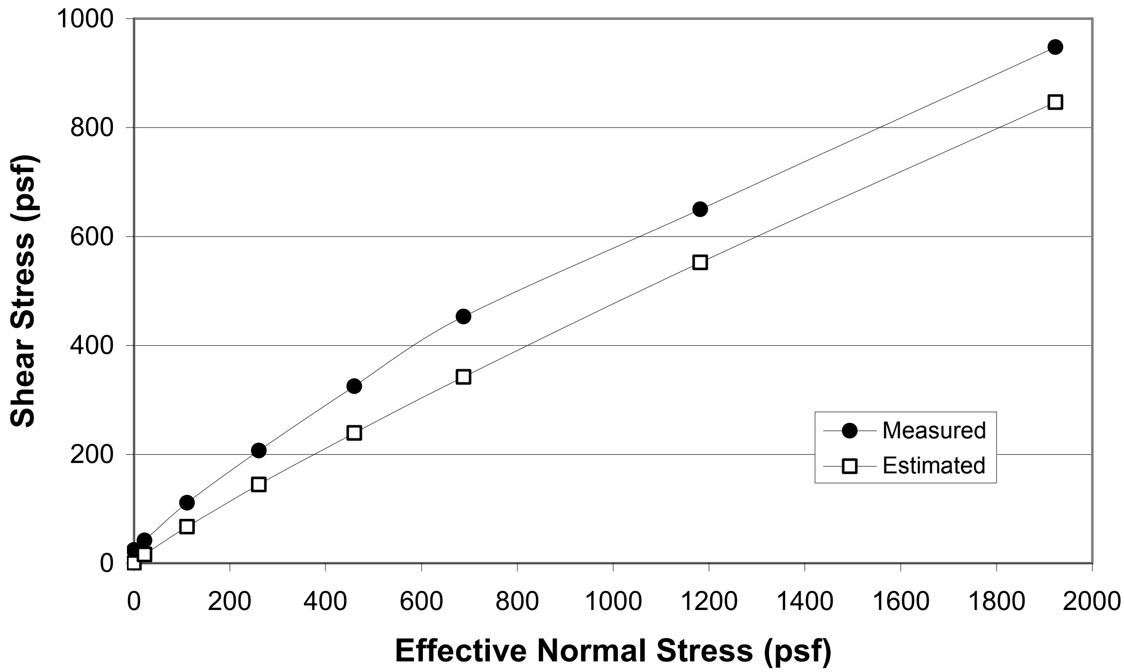


Figure 5.8 Measured and estimated fully-softened shear strength envelopes for Paris clay.

Further Simplifications

To further simplify the estimate of strengths for design, secant friction angles, ϕ'_{secant}, were calculated from Eq 5.3 for stresses corresponding to various depths of overburden. Values were calculated for soils with liquid limits of 50, 75 and 100 percent. Effective overburden stresses were calculated assuming a total unit weight of soil of 125 pcf and zero pore water pressures. These values (γ = 125 pcf and zero pore water pressure) should represent an approximate upper bound for the effective overburden stress, and thus a lower bound in terms of the secant friction angle—friction angles decrease as effective stress increases. The friction angles calculated in this manner are summarized in Table 5.4.

Table 5.4 Friction angles computed for various overburden depths (stresses) and liquid limits.

Depth (feet)	Liquid Limit, w_{LL} = 50	Liquid Limit, w_{LL} = 75	Liquid Limit, w_{LL} = 100
5	30 degrees	27 degrees	25 degrees
10	28 "	25 "	23 "
20	26 "	24 "	21 "
30	25 "	22 "	20 "
40	25 "	22 "	20 "
50	24 "	21 "	19 "

Residual Shear Strength

"Residual" shear strength is a term applied to the shear strength developed at large strains, i.e., the ultimate or critical state strength condition. Residual shear strengths are applicable to slides where there is a preexisting shear plane, resulting either from a previous slope failure or geologic processes. Although residual shear strengths are probably not applicable to first-time slides in embankments, they are applicable to embankments once a slide has occurred.

Correlations by Stark and Co-Workers

Stark et al (2005) developed correlations between residual shear strengths and index properties similar to the ones shown previously for fully-softened shear strengths in Figure 5.3. Their correlations for residual shear strengths are shown in Figure 5.9.

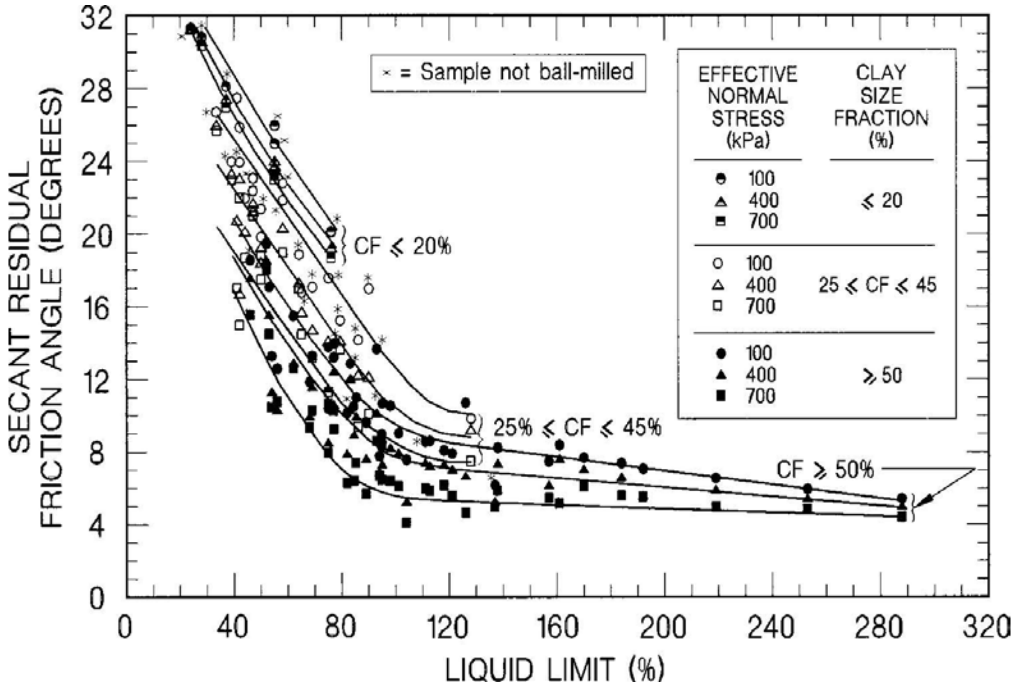


Figure 5.9 Secant residual friction angle relationships with liquid limit, clay-size fraction, and effective normal stress (from Stark et al. 2005).

Following procedures like those described earlier for fully-softened shear strengths the actual data points presented in Fig. 5.9 for soils with a clay fraction of 50 percent or greater were digitized. For each soil three values of secant friction angle were determined corresponding to normal stresses of 100, 400 and 700 kPa. Equation 5.2 was then fit to the data for each soil using a least squares approach. Finally, the values of $\phi'_{0,r}$ and $\Delta\phi'_r$ obtained from fitting lines to the data for each soil were plotted versus the liquid limit as shown in Figures 5.10 and 5.11, respectively. Initial examination of the data in Fig. 5.10 suggested that an equation of the form of Eq 5.3 did not fit the data well over the entire range of liquid limits from approximately 50 to nearly 300 percent. Accordingly, an equation was fit to only the data for liquid limits of 150 percent or less, which is the range of greatest interest for Texas soils. The resulting curve fit to the data is shown in Figure 5.12 with the data for liquid limits of 150 percent or less. The equation for the curve shown is,

$$\phi'_{0,r} = 52.5° - 21.3° \log_{10}(w_{LL}) \tag{5.5}$$

Examination of the data for the "slope" parameter, $\Delta\phi'$, in Figure 5.11 suggested that a nominal value of 3 degrees is a reasonable representation of the data for soils with a liquid limit of less than 150. Substituting this value ($\Delta\phi'_r = 3°$) and Eq 5.5 into Eq 5.2 for the secant friction angle then gives,

$$\phi'_{secant,r} = 52.5° - 21.3° \log_{10}(w_{LL}) - 3° \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \tag{5.6}$$

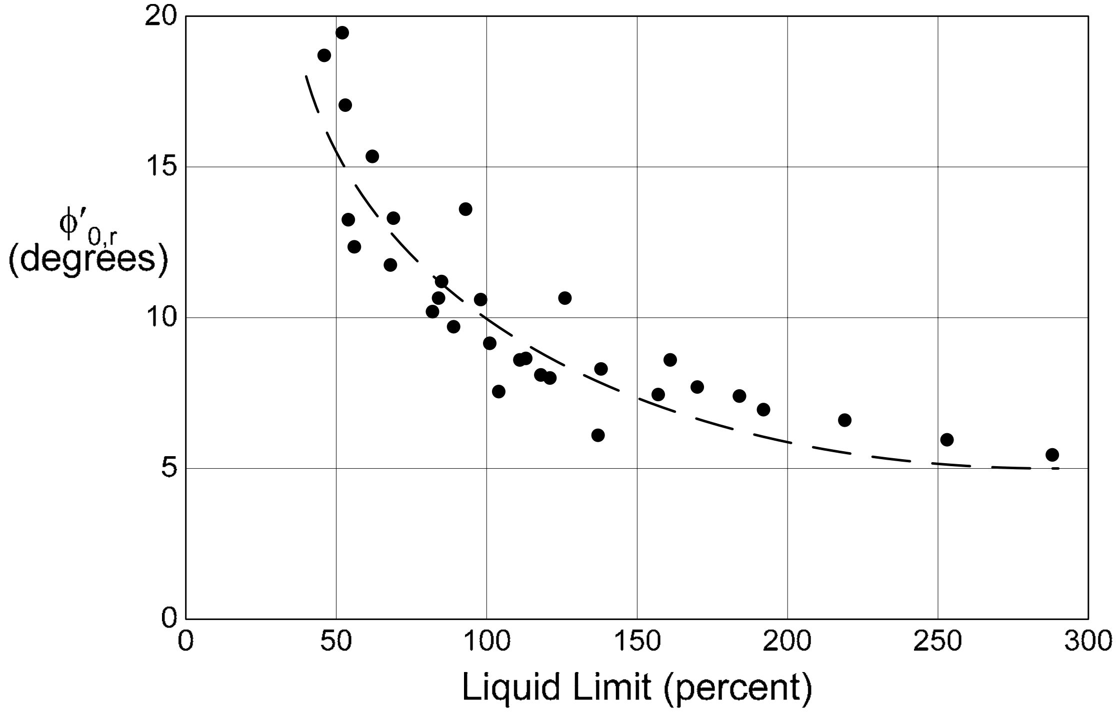


Figure 5.10 Variation in parameter $\phi'_{0,r}$ with liquid limit calculated from Stark et al. (2005) data set for residual shear strengths of soils with at least 50 percent clay fraction.

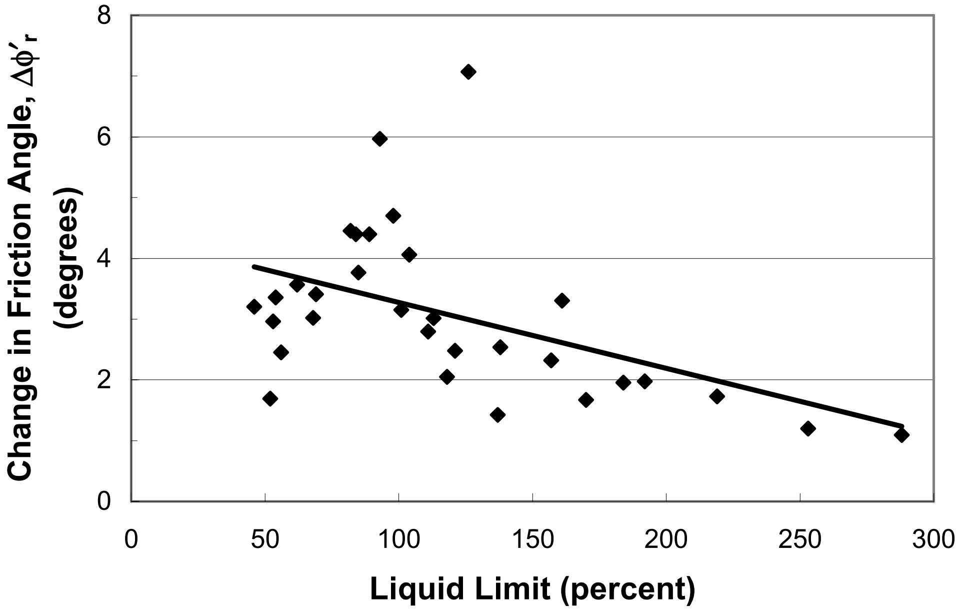


Figure 5.11 Variation in parameter $\Delta\phi'_r$ with liquid limit calculated from Stark et al. (2005) data set for residual shear strengths of soils with at least 50 percent clay fraction.

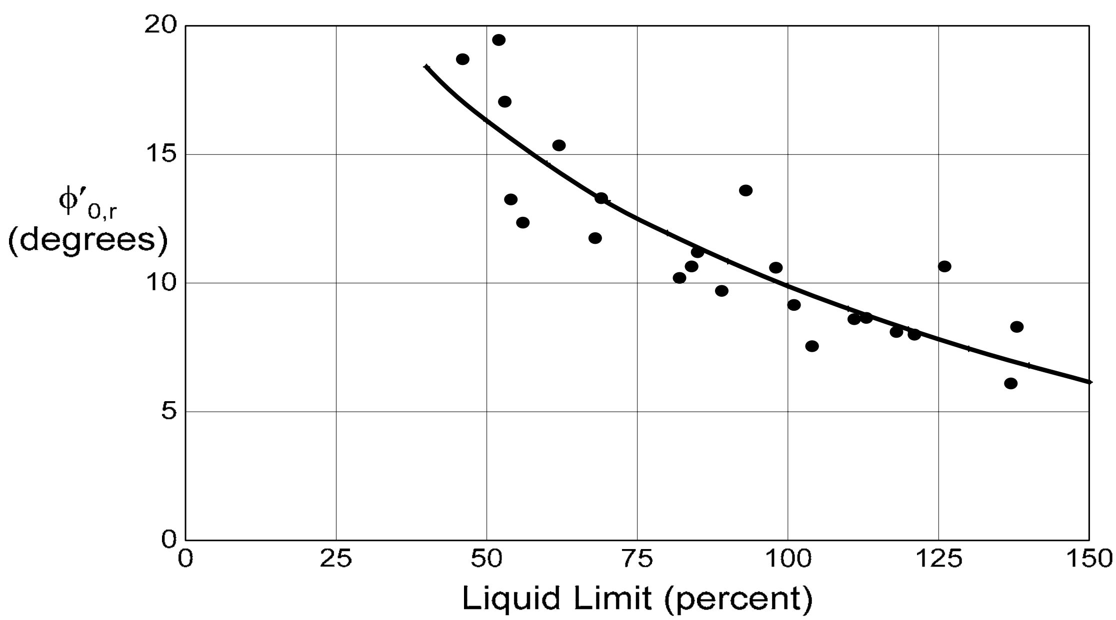


Figure 5.12 Variation in parameter $\phi'_{0,r}$ with liquid limit calculated from Stark et al. (2005) data set for residual shear strengths of soils with at least 50 percent clay fraction and liquid limit less than 150. Curve based on Equation 5.5.

Evaluation of Empirical Equation

To evaluate the applicability of Eq 5.6 for the residual shear strength, data for the residual shear strength of several Texas soils were examined. The first set of data were reported by Fox (1979); the second set of data are from Green and Wright (1986). Fox presents data for two highly plastic clays, known locally as the "Taylor" and "Del Rio" clays. Both soils have liquid limits ranging from approximately 55 to 70 percent. To calculate values from Eq 5.6 a nominal average value of 63 was assumed for the liquid limit of both soils. Values of the secant friction angle were then calculated from Eq 5.6 for a range in normal stresses corresponding to the range in stresses—350 to 3500 psf—used by Fox in his tests. The tangents of the secant friction angles were then multiplied by the corresponding normal stresses to compute a shear stress. Finally, the stresses were plotted on the Mohr diagram shown in Figure 5.13. The measured data for the Taylor and Del Rio clays are also plotted on this same diagram. Because both soils had essentially the same liquid limits (range for both soils: 55 to 70 percent), the data for both soils are plotted on a single Mohr diagram. The failure envelope computed using Eq 5.6 and shown on the Mohr diagram provides a good representation of the data with a tendency to favor slightly the lower of the measured strength values.

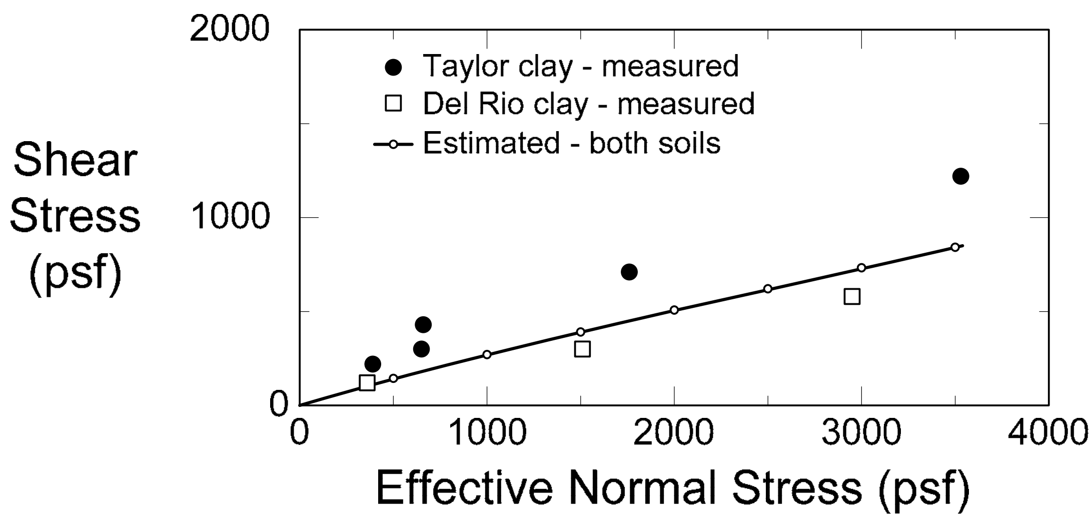


Figure 5.13 Estimated and measured residual shear strengths for Taylor and Del Rio clays (Data from Fox, 1979).

Green and Wright (1986) presented data for residual shear strengths of compacted specimens of the Beaumont clay, which has a nominal liquid limit of 70. Using a liquid limit of 70, shear strengths were calculated from Eq 5.6 following the procedures described above for the Taylor and Del Rio clays. The resulting strengths are plotted on the Mohr diagram in Figure 5.14. Very good agreement can be seen between the measured and calculated values, with the calculated values tending toward a lower bound of the measured values.

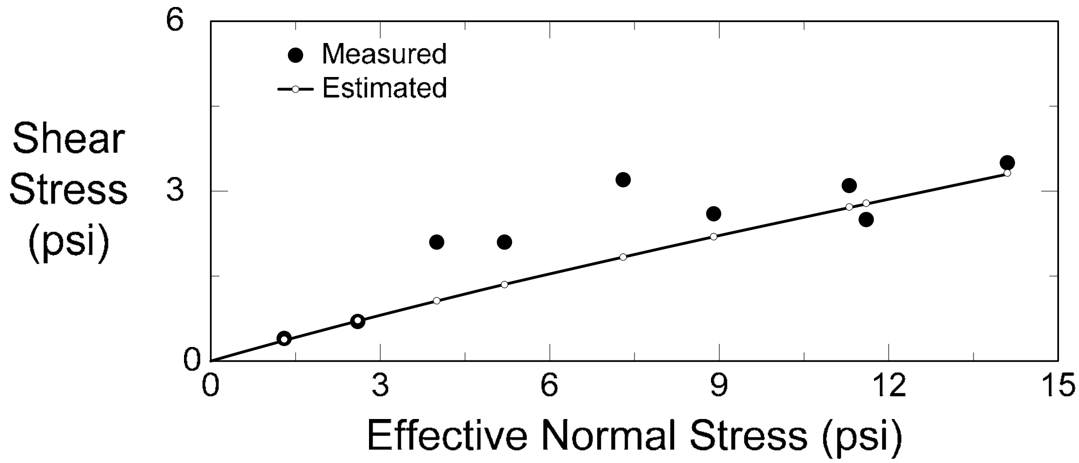


Figure 5.14 Estimated and measured residual shear strengths for Beaumont clay (data from Green and Wright, 1986).

Overall the agreement between the values of residual shear strength for the highly plastic Taylor, Del Rio and Beaumont clays and those calculated from Eq 5.6 is excellent, considering the empirical nature of the equation. Although the empirical equation was not developed using data for the Texas soils, the equation seems to be applicable to these soils.

Recommended Strengths

Experience with a number of slides in both natural and compacted fill slopes of highly plastic clays suggests that the long-term strength is less than the peak strength of the soil. This may be due either to softening that occurs as a result of repeated wetting and drying of the soil or as a result of some other mechanism, e.g., partial progressive failure. For slopes which have not experienced large strains due either to previous sliding or geologic processes, the fully-softened strength appears to be applicable and is recommended. Based on the discussion earlier in this chapter, the fully-softened strength of high plasticity soils (liquid limit 50 or greater) can be expressed by a secant friction angle estimated from the following empirical equation:

$$\phi_{secant} = 55.3° - 16.7° \log_{10}(w_{LL}) - 6° \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \tag{5.4}$$

For slopes that have failed or experienced previous large shear deformations due to geologic processes, use of the lower, residual shear strengths instead of fully-softened shear strengths is recommended. Residual strengths for highly plastic soils can be estimated from the following equation:

$$\phi_{secant,r} = 52.5° - 21.3° \log_{10}(w_{LL}) - 3° \log_{10}\left(\frac{\sigma'_f}{p_a}\right) \tag{5.6}$$

Both of the above equations were derived for soils with a clay fraction of at least 50 percent and, for Eq 5.6 the data were further restricted to soils with liquid limits of 150 or less. These conditions, however, encompass the highly plastic soils commonly found in Texas. These highly plastic soils represent the soils of greatest interest and the ones causing the greatest difficulties for slope stability. For lower plasticity soils and/or soils with a lower clay fraction the reader can refer to the additional relationships by Stark et al. (2005) shown in Figures 5.3 and 5.9.

Chapter 6 – Summary and Recommendations

Summary

TxDOT seeks to update its Geotechnical Manual and provide improved guidance on the appropriate soil shear strength properties to be used for stability analyses of slopes and retaining walls. Important details of stability analyses and the selection of shear strength have been presented and discussed in Chapter 2. Many of these details are currently omitted or only briefly addressed in the current Geotechnical Manual. Accordingly, it is anticipated that some of the information presented in Chapter 2 of this report can be incorporated into future versions of the TxDOT Geotechnical Manual (Texas Department of Transportation, 2000).

During the past approximately thirty years a substantial amount of research has been conducted for TxDOT on the stability of slopes and the appropriate shear strengths to be used for design. This research is reviewed and summarized in Chapter 3. The research has shown that the majority of slope problems experienced by TxDOT are *long-term* stability problems, governed by the *drained*, rather than undrained strength of the soil. The research has also led to the conclusion that the *fully-softened* shear strength is the controlling shear strength in most cases, but that the *residual* shear strength may be applicable once a slide has occurred. Most failures of embankments have been restricted to the portion of the compacted fill above the level of the toe of the slope, with relatively few failures involving the natural foundation soils. However, when failures do involve the foundation, the undrained, rather than drained strength controls the stability and must be evaluated.

Appropriate shear strengths for both undrained and drained conditions are presented and discussed in Chapter 4 and 5, respectively. Undrained shear strength values can vary widely and depend on the past stress history at a particular site. Accordingly, undrained shear strengths must be evaluated on a site-specific basis. Correlations that relate the undrained shear strength to the stress history (present and past maximum effective stresses) and to the Standard Penetration Resistance blow count ("N-value") are presented in Chapter 4.

Drained shear strengths are discussed in Chapter 5. Based on previous research conducted for TxDOT as well as correlations between shear strength and soil index properties by Stark and his co-workers, a suitable empirical equation has been developed and is presented for estimating the fully-softened and residual shear strengths of highly plastic (liquid limit of 50 or greater) clays. Such highly plastic clays represent the most problematic soils encountered by TxDOT for slope and retaining wall stability.

This report has focused on the shear strength of fine-grained soils, and highly plastic clays in particular which present the greatest stability problems for TxDOT. No attention has been given to coarse-grained, cohesionless soils because the writer is aware of no instance where the strength of such materials has been an issue in a failure. The primary problem with cohesionless soils has apparently been with settlement, rather than shear strength. Although this does not warrant complete neglect of the shear strength, it is believed that the strength can usually be estimated reasonably well based on current experience and knowledge.

Pore Water Pressures

The stability of most of the embankment slopes constructed of highly plastic clay fill are governed by the long-term, drained shear strength of the soils. These strengths are expressed as a function of the effective stresses in the soil. Application of the strengths in slope stability analyses requires that effective stresses be used in the analyses and that appropriate pore water pressures be determined. The pore water pressures are an important element in the evaluation of slope stability. Although the determination of pore water pressures is independent of the determination of the shear strength parameters and is beyond the scope of this report, careful attention should be paid to the pore water pressures that are used to evaluate stability. The research by Kayyal and Wright (1991) reviewed in Chapter 3 indicates that the pore water pressures may be quite high in embankment slopes with a perched water table nearly coincident with the face of the slope. Such high pore water pressures should be considered when computing the stability of exposed embankment slopes.

Recommendations

Based on a review of the TxDOT Geotechnical Manual it is recommended that the coverage of soil shear strength for slope and retaining wall stability analyses be expanded to include material presented in Chapter 2 of this report. Chapter 2 covers important principles related to soil shear strength for stability analyses and provides guidance for selecting appropriate test conditions for measuring the shear strength.

Specific recommendations and suitable empirical equations for estimating both undrained and drained shear strengths are presented in Chapters 4 and 5, respectively. These empirical equations can be used by TxDOT as a baseline for estimating and evaluating shear strengths and at TxDOT's discretion may be incorporated into the Geotechnical Manual as well as provided to designers as guidelines.

Clearly, one of the best ways to determine design shear strengths for clays is by appropriate laboratory tests on representative samples of the soil. Empirical equations such as the ones presented in this report are useful, but it should be recognized that the estimates involve significant approximations and higher factors of safety may be required than when strengths are based on testing the particular soil of interest. The cost for designs employing soil shear strengths based on conservative empirical guidelines and higher factors of safety should always be weighed against the additional costs of laboratory or field testing.

References

Abrams, T. G. and Wright, S. G. (1972). "A Survey of Earth Slope Failures and Remedial Measures in Texas." Research Report 161-1, Center for Highway Research, University of Texas, August 1972.

ASTM. (2000). “Designation D 2166-00, Standard Test Method for Unconfined Compressive Strength of Cohesive Soil.” American Society for Testing and Materials, Philadelphia.

———. (2002). “Designation D 4767-02, Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.” American Society for Testing and Materials, Philadelphia.

———. (2003). “Designation D 3080-03, Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.” American Society for Testing and Materials, Philadelphia.

Bjerrum, L. (1972). "Embankments on Soft Ground." Proceedings of the Specialty Conference on Performance of Earth and Earth-Supported Structures, Lafayette, ASCE, Vol. 2, pp. 1–54.

Bjerrum, L. and Simons, N. E. (1960). "Comparison of shear strength characteristics of normally consolidated clays." *Proceedings of the Research Conference on Shear Strength of Cohesive Soils*, ASCE, Boulder, Colorado, 1960, pp. 711–726.

Cedergren, H. R. (1989). *Seepage, Drainage, and Flow Nets*. Third Edition, John Wiley and Sons, Inc.

Corps of Engineers. (1970). “Laboratory Soils Testing.” *Engineer Manual EM 1110-2-1906*, Department of the Army, U. S. Army Corps of Engineers, Washington, D. C., November 1970 with Change 1 dated May 1980.

——. (1986). “Engineering and Design – Seepage Analysis and Control for Dams.” *Engineer Manual EM 1110-2-1901*, Department of the Army, U. S. Army Corps of Engineers, Washington, D. C., 30 September 1986.

——. (2003). “Engineering and Design – Slope Stability.” *Engineer Manual EM 1110-2-1902*, Department of the Army, U. S. Army Corps of Engineers, Washington, D. C., 31 October 2003.

Djoenaidi, W. J. (1985). "A compendium of soil properties and correlations." M. Eng. Sc. Thesis, University of Sydney, Sydney, 836 pages.

Duncan, J. M., Horz, R. C., and Yang, T. L. (1989). *Shear Strength Correlations for Geotechnical Engineering*. Virginia Polytechnic Institute and State University, Blacksburg, August, 1989, 100 pages.

Duncan, J. M., and Wright, S.G. (2005). *Soil Strength and Slope Stability*. John Wiley and Sons, Inc., New Jersey, 2005.

Focht, J. A., and Sullivan, R. A. (1969). "Two slides in overconsolidated Pleistocene clays." *Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering*, Mexico City, 1969, Vol. 2, pp. 571–576.

Fox, T. J. (1979). "A study of slope stability in stiff-fissured clays in the Austin area, Texas." Thesis presented to the faculty of the graduate school of The University of Texas at Austin in partial fulfillment of the requirements for the degree of Master of Science in Engineering, The University of Texas at Austin, May, 1979.

Fredlund, D. G. and Rahardjo, H. (1993). *Soil Mechanics for Unsaturated Soils*. John Wiley and Sons, New York.

Fredlund, D. G. (2000). “1999 R. M. Hardy Lecture: The implementation of unsaturated soil mechanics into geotechnical engineering.” *Canadian Geotechnical Journal*, Vol. 37, No. 5, pp. 963–986.

Gourlay, A. W., and Wright, S. G. (1984). "Initial laboratory study of the shear strength properties of compacted clays used for highway embankment construction in the area of Houston, Texas." Report on laboratory testing performed under Interagency Contract Nos. (82–83) 2187 and (84–85) 1026, for Texas State Department of Highways and Public Transportation – District 12, by Center for Transportation Research, The University of Texas, September, 1984.

Green, R. and Wright, S. G. (1986). "Factors Affecting the Long Term Strength of Compacted Beaumont Clay." Research Report 436-1, Center for Transportation Research, The University of Texas at Austin, October 1986.

Hamoudi, M. N., Coyle, H.M., and Bartoskewicz, R. E. (1974). “Correlation of the Texas Highway Department cone penetrometer with unconsolidated-undrained shear strength of cohesive soils.” *Research Report 10-1*, Texas Transportation Institute, Texas A & M University.

Kanji, M. A. (1974). "The relationship between drained friction angles and Atterberg limits of natural soils." Technical Note, *Geotechnique*, Vol. 24, No. 4, December, 1974.

Kayyal, M. K. and Wright S.G. (1991). "Investigation of Long-Term Strength Properties of Paris and Beaumont Clays in Earth Embankments." Research Report 1195-2F, Center for Transportation Research, The University of Texas at Austin, November, 1991.

Kulhawy, F. H. and Mayne, P. W. (1990). *Manual on Estimating Soil Properties for Foundation Design*. EPRI EL-6800, Research Project 1493-6, Final Report, Cornell University, Ithaca, August, 1990.

Jamiolkowski, M., Ladd, C. C., Germaine, J. T., and Lancellotta, R. (1985). “New developments in field and laboratory testing of soils.” Proceedings, Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, Aug. 1985, Vol. 1, pp. 57–153.

Ladd, C. C. and Foott, R. (1974). "New Design Procedure for Stability of Soft Clay, *Journal of the Geotechnical Engineering Division*.” ASCE, Vol. 100, No. GT7, pp. 763–786.

Ladd, C. C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H. G. (1977). "Stress-deformation and strength characteristics." State-of-the-Art Reports, *Proceedings*, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 2, pp. 421–494.

Mesri, G. (1989). “Reevaluation of S_u(mob) = 0.22 σ′max using laboratory shear tests.” *Canadian Geotechnical Journal*, Vol. 26, No. 1, pp. 162–164. 1989.

O'Malley, E. S.,and Wright, S. G. (1987). "Review of Undrained Shear Strength Testing Methods used by the Texas State Department of Highways and Public Transportation." Research Report 446-1F, Center for Transportation Research, The University of Texas at Austin, November 1987.

Peterson, R., Jaspar, J. L., Rivard, P. J., and Iverson, N. L. (1960). “Limitations of laboratory shear strength in evaluating stability of highly plastic clays.” *Proceedings of the Research Conference on Shear Strength of Cohesive Soils*, ASCE, Boulder, Colorado, 1960, pp. 765–791.

Rogers, L. E. and Wright, S. G. (1986). "The Effects of Wetting and Drying on the Long-Term Shear Strength Parameters for Compacted Beaumont Clay." Research Report 436-2F, Center for Transportation Research, The University of Texas at Austin, November 1986.

Saleh, A. A. and Wright, S. G. (1997). “Shear Strength Correlations and Remedial Measure Guidelines for Long-Term Stability of Slopes Constructed of Highly Plastic Clay Soils.”

Research Report 1435-2F, Center for Transportation Research, The University of Texas at Austin, October, 1997.

Skempton, A. W. (1957). Discussion of "The planning and design of the new Hong Kong airport." *Proceedings of the Institution of Civil Engineers*, London, Vol. 7, pp. 305–307.

—— (1964). "Long-term Stability of Clay Slopes." *Geotechnique*, Institution of Civil Engineers, Great Britain, Vol. 14, No. 2, pp. 75–101.

Skempton, A. W. and Hutchinson, J. (1969). "Stability of natural slopes and embankment foundations." State of the Art Volume – Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico City, pp. 291–340.

Skempton, A. W. (1977). "Slope stability of cuttings in brown London clay." *Proceedings*, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Vol. 3, pp. 261–270.

Stark, T. D., and Eid, H. T. (1997). "Slope stability analysis in stiff-fissured clays." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 123, No. 4, April, pp. 335–343.

Stark, T. D., Choi, H., and McCone, S. (2005). "Drained shear strength parameters for Analysis of landslides." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 131, No. 5, May, 2005, pp. 575–588.

Stauffer, P. A. and Wright, S. G. (1984). "An Examination of Earth Slope Failures in Texas." Research Report 353-3F, Center for Transportation Research, The University of Texas, November 1984.

Terzaghi, K. and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice*. 2nd edition, John Wiley and Sons, New York, 1967.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil Mechanics in Engineering Practice*. 3rd edition, John Wiley and Sons, New York.

Texas Department of Transportation (2005). *Geotechnical Manual*. October, 2000 [available online at http://manuals.dot.state.tx.us/dynaweb/colbridg/geo as of 8 September 2005].

Wright, S. G., and Duncan, J. M. (1972). "Analyses of Waco Dam slide." *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 98, No. SM9, Sept., pp. 869–877.

Wright, S. G. (1999). *UTEXAS4: A Computer Program for Slope Stability Analyses*. Shinoak Software, Austin, Texas.