PIERCE CHANDLER

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

Stephen G. Wright

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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.

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

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., $45^\circ + \phi'/2 = 45^\circ + 30^\circ/2 = 60^\circ$. 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.

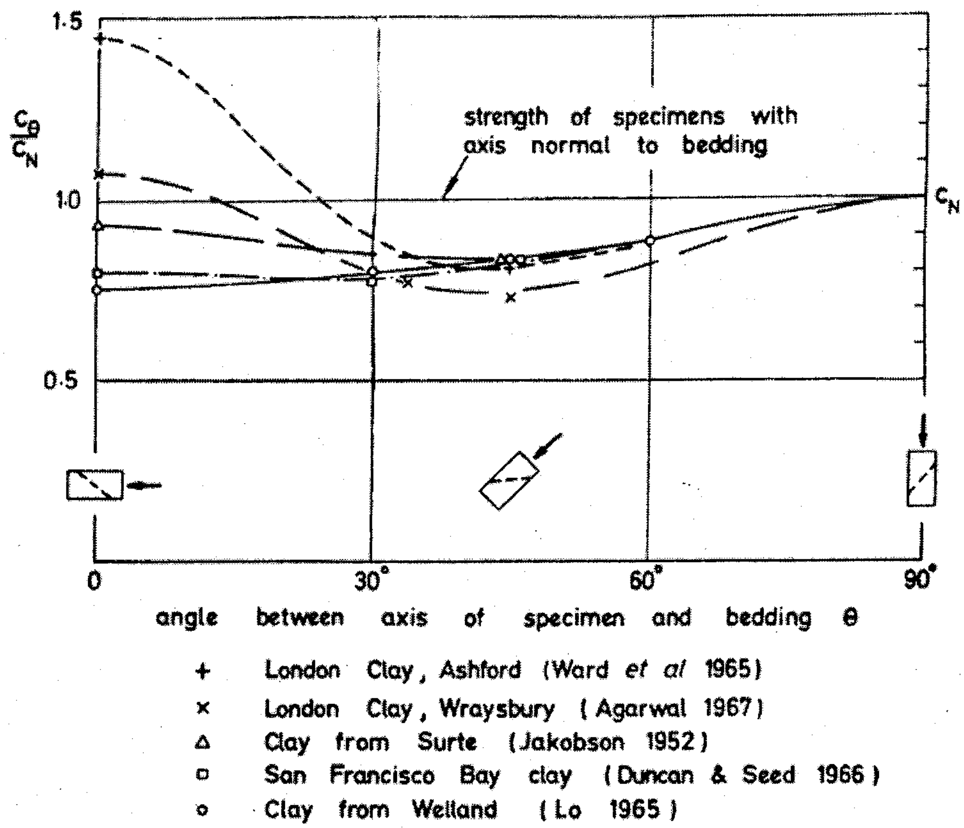

Figure 4.2 Effect of sample orientation on undrained shear strength (from Skempton and Hutchinson, 1969).

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.

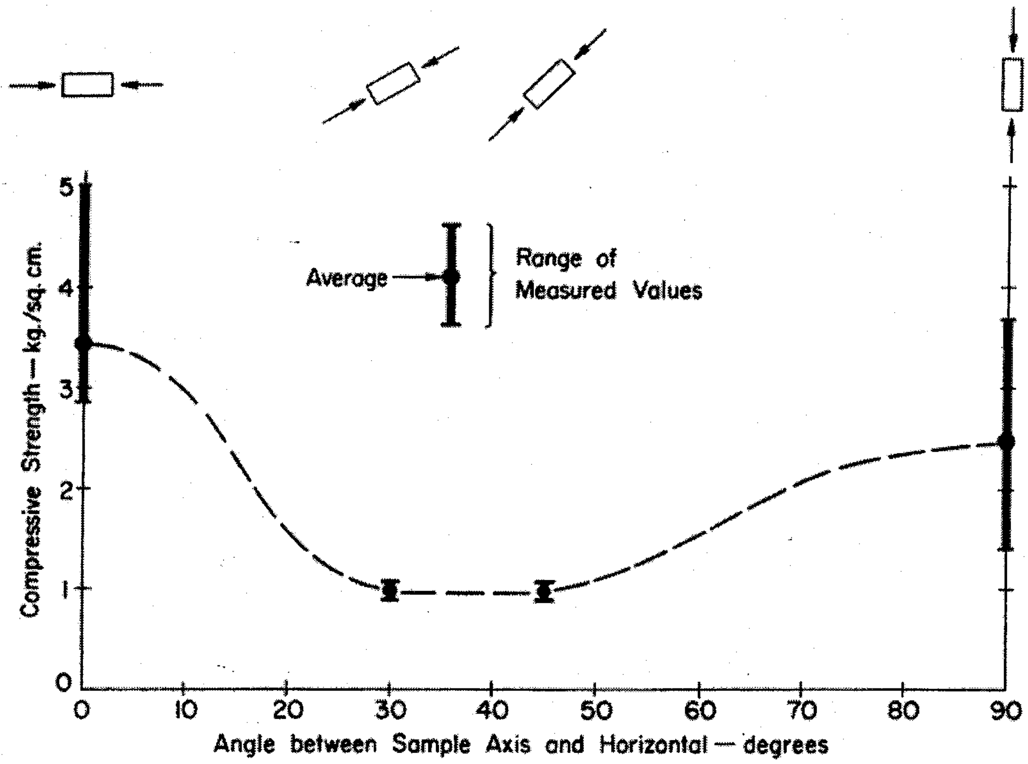

Figure 4.3 Effect of sample orientation on undrained shear strength of Pepper Shale from Waco Dam (from Wright and Duncan, 1972).

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.

accepted correlations for estimating undrained shear strengths are reviewed and discussed in this section.

Relationships to Effective Consolidation Pressure

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.

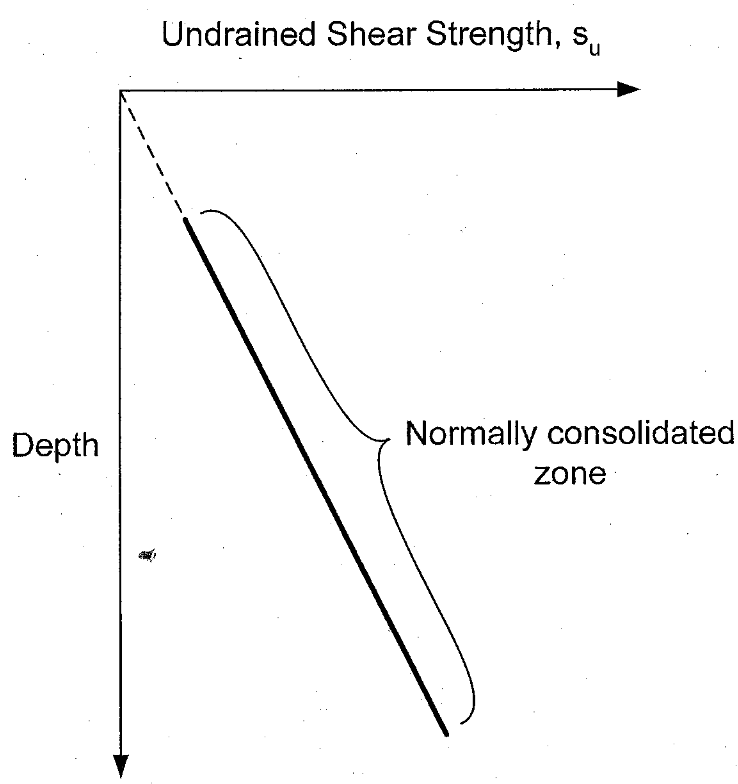

Figure 4.5 Typical variation in undrained shear strength with depth for a normally consolidated clay.

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.

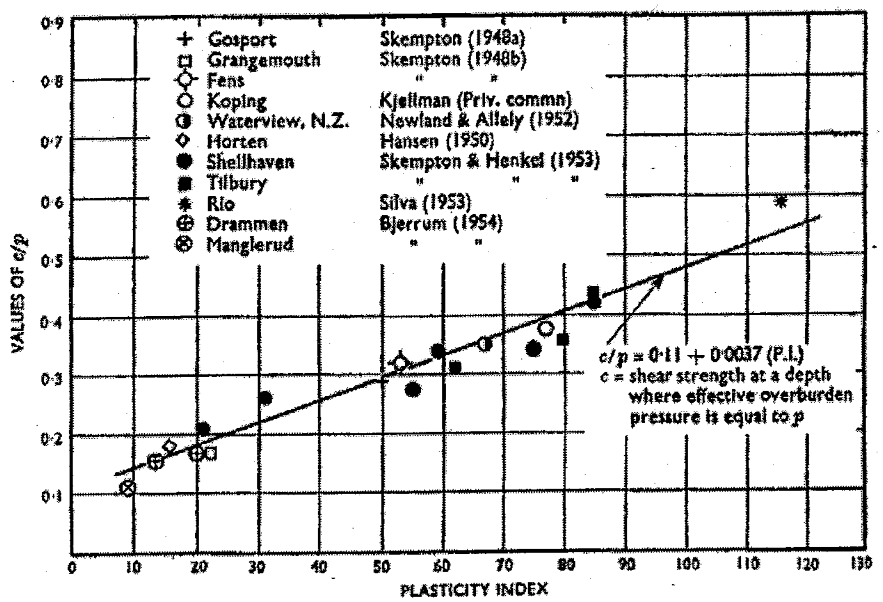

Figure 4.6 Relationship between c/p ratio and plasticity index suggested by Skempton (1948).

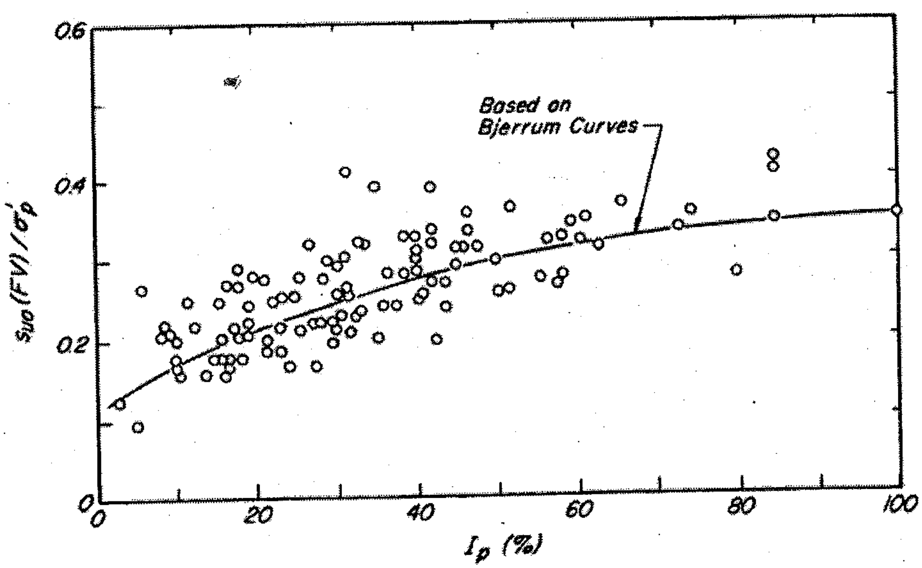

Figure 4.7 Relationship between c/p ratio and plasticity index (from Terzaghi, Peck and Mesri, 1996).

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.

<u>Correlations with Standard Penetration Test Blow Count</u>

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.

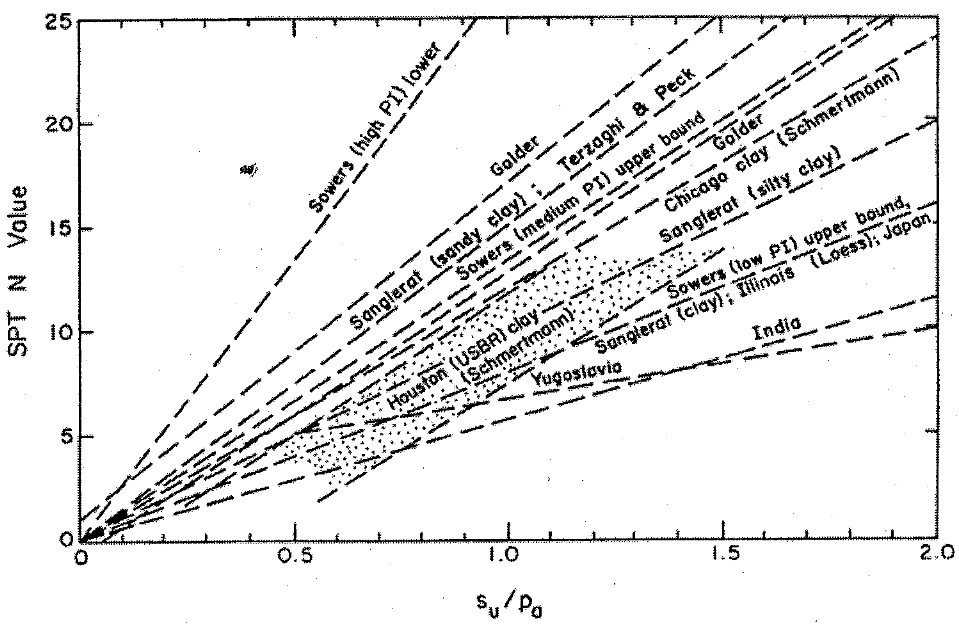

Figure 4.8 Relationships between undrained shear strength and Standard Penetration Resistance N values (from Kulhawy and Mayne, 1990; based on Djoenaidi, 1985).

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.

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.

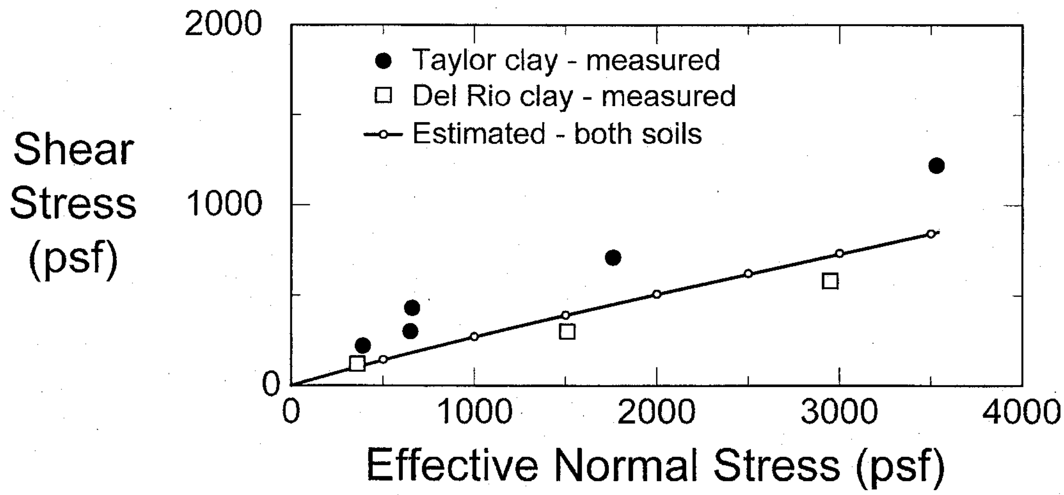

Figure 5.13 Estimated and measured residual shear strengths for Taylor and Del Rio clays (Data from Fox, 1979).

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

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.