FIRST DRAFT

Slope Stability Analyses: Development and Use of Fully Softened Shear Strength

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A Monograph submitted for review and possible acceptance by Springer Briefs

February 12, 2016

Table of Contents

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1. Introduction

Stability analyses for cut, embankment, dam, and levee slopes comprised of fine grained soils have traditionally been conducted using peak strengths or some percentage of peak strength determined from standard laboratory shear strength tests on undisturbed or freshly compacted samples. Using these peak strengths and slope ratios in the range of 3H:1V (3 horizontal to 1 vertical) to 4H:1V with vertical heights of 15 to 25 feet (4.6 to 7.6 m) typically results in calculated factors of safety that are above the regulatory required value of 1.5 and in many cases above 2.0, even with the assumption of a piezometric level or pore-water pressure ratio in the slope. However, many of these slopes have subsequently failed, which implies a factor of safety of approximately unity (one). This disparity indicates the peak strength from standard laboratory shear strength tests are not representative of the long-term soil strength in cut or embankment slopes.

It has long been recognized that stiff fissured clays may become "fully softened" and undergo significant strength loss over time (Skempton, 1964 and 1970). However, the use of the fully softened strength (FSS) in fine grained slopes has only come into use for compacted soil slopes in recent years (Stark and Duncan, 1991; Stark and Eid, 1997; Duncan et al., 2011). In addition, there are situations where there is no recent history of slope movement where a FSS would be expected to be mobilized but instead a residual strength was mobilized (Stark and Eid, 1997; Mesri and Shahein, 2003). This can be problematic because the FSS is significantly greater than the residual for high plasticity materials, i.e., liquid limit greater than 50. As a result, this White Paper explains how to: identify FSS susceptible slopes, measure FSS, and use FSS in stability analyses.

2. Purpose and Use

The purpose of this white paper is to explain:

- fully softened strength (FSS),
- why geotechnical engineers should understand FSS,
- how FSS develops with time,
- slopes susceptible to developing a FSS condition,
- how to measure FSS, use of FSS in stability analyses, and
- measures that can be implemented to limit FSS development.

The importance of the FSS is being recognized by academia and practice and is being adopted by state and federal agencies. For example, the U.S. Army Corps of Engineers Engineering Manual EM 1110-2-1902 SLOPE STABILITY (2003) recommends the FSS be the design shear strength for high plasticity, high clay-size fraction clays, and clay-rich soft rocks (shales and clay shales) in all long-term stability analyses.

It is envisioned that this White Paper will be used by practitioners and academicians to understand and utilize the FSS in their activities. The White Paper also provides references for future study of the various topics covered herein. In particular, this White Paper provides a summary of and references for the various shear strengths that may be applicable to a slope stability analysis, consequences of selecting an inappropriate shear strength for slope design, slope conditions where a fully softened strength (FSS) is likely to develop, techniques for measuring and incorporating the FSS in stability analyses, applicable factors of safety, and techniques for limiting development of a FSS condition for an embankment slope.

3. FSS Definition (What is it?)

This section discusses the various strengths that may be mobilized in soil deposits and when they may be applicable to slope stability analyses. This section starts with determining whether a soil will be drained or undrained and then discusses drained peak, fully softened, and residual strengths and their applicability to drained slope stability analyses.

(a) Drained v. Undrained Strengths

In drained and partially drained loading excess pore-water pressures can dissipate while the soil expands or contracts. During undrained loading pore-water pressures change, which results in a variation of effective stresses, while the total volume remains constant. Generally the following drainage conditions are considered in geotechnical engineering:

<u>Drained Strength:</u> Drained loading implies that loads are applied at a sufficiently slow rate so that no significant pore-water pressures are generated in the soil during shear and the associated volume change. A drained condition usually develops if the Time Factor (T) is greater than 3 (Wright and Duncan, 2005) where T is defined as:

$$T = \frac{C_v * t}{(H_{dr})^2} \quad (1)$$

where C_v is the coefficient of consolidation, t is time, and H_{dr} is the length of drainage path to dissipate shear-induced pore-water pressures.

<u>Undrained Strength:</u> Undrained loading refers to a loading condition where pore-water pressures are generated in saturated and contractive soils while pore-water pressures decrease in saturated, dilatant soils. Undrained loading is a shear condition in which no volume change occurs. An undrained condition usually develops if T is less than 0.01 (Wright and Duncan, 2005).

<u>Partially Drained Strength:</u> Partially drained implies a loading situation somewhere in between the drained and undrained loadings described above. It involves a condition in which some of the pore-water pressures generated during loading have

dissipated but are still significant. Additionally, some related volumetric change has occurred in the form of contraction or dilation.

The strength of soil sheared under drained conditions is usually illustrated with effective stress strength parameters, using the linear Mohr-Coulomb failure criterion as shown below:

$$\tau_f = c' + \sigma'_{ff} * \tan\phi' \qquad (2)$$

where τ_f is the shear stress at failure, i.e., shear strength, c' and ϕ' are the effective stress cohesion intercept and friction angle, respectively, of a linear strength envelope, and σ'_{ff} is the effective normal stress on the failure plane at failure. However, it is now accepted the drained FSS and residual strengths are effective normal stress dependent (Stark and Eid, 1997) and should not be modeled using a linear strength envelope in stability analyses, which is discussed further below.

The focus of this White Paper is drained shear strength for stability analyses so undrained strength parameters and the possible effect of softening on undrained strengths will be addressed in a subsequent White Paper.

(b) Drained Strengths: Peak, FSS, and Residual Strengths

This section discusses the different drained shear strengths that may be applicable to slope stability analyses of cut slopes and embankments containing fine grained soils. In particular, this section discusses the difference between the drained peak, fully softened, and residual strengths and their applicability to various field conditions and slope stability analyses.

Skempton (1985) defines various strengths along a degrading shear stress or shear stress ratio (τ/effective normal stress, (σ'_n)) versus shear displacement relationship (see **Figure 1**) using the following terminology:

- The peak or maximum shear strength achieved for each specimen after the initial nearly elastic behavior is referred to as the "peak strength." Unfortunately there is a range of peak shear strength depending on the level of weathering and/or fissuring the deposit has undergone, which has resulted in some confusion. If little or no weathering has occurred or the sample was obtained from a depth that precludes significant weathering, the resulting strength is referred to as Peak Intact Strength. If significant weathering and/or stress relief has occurred, the soil is usually weathered, fissured, and/or jointed, and the resulting strength is referred to as Peak Weathered- or Fissured-Strength. The Peak Intact Strength Envelope is always higher than the Peak Weathered Strength Envelope, so the level of weathering should be carefully considered before assigning a Peak Intact Strength Envelope in a stability analysis.

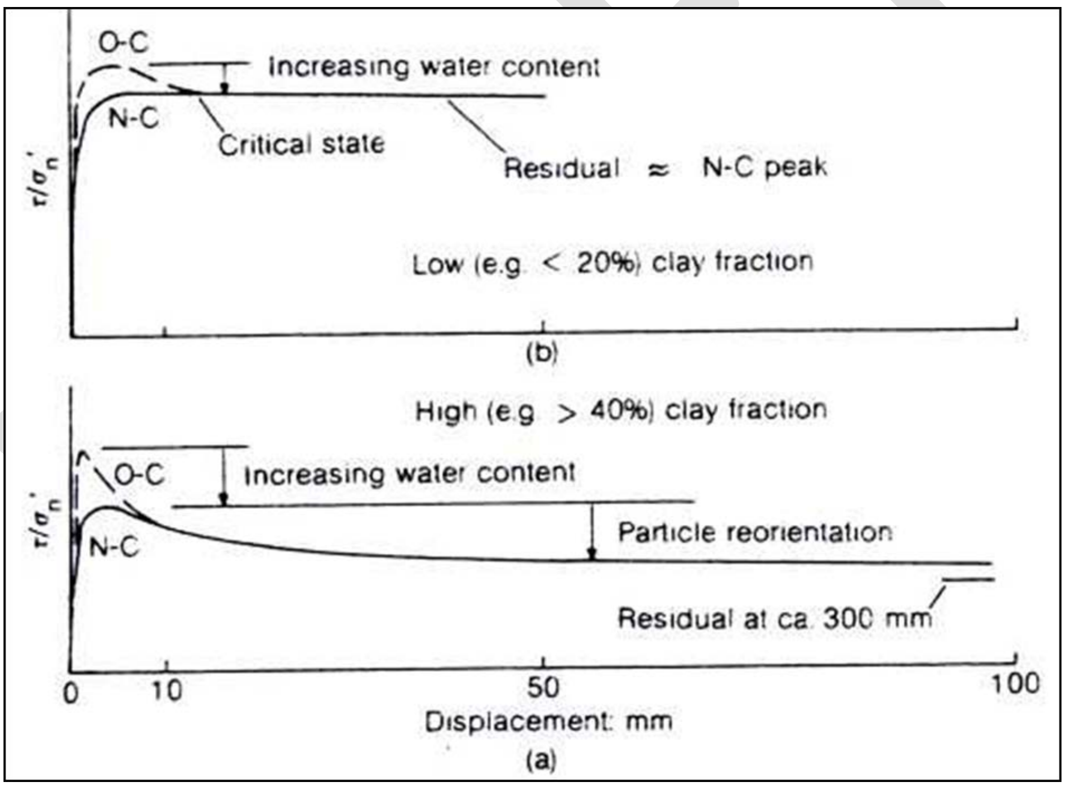


Figure 1. Drained shear behavior for: (a) low and (b) high clay fraction (%< 2 μm) soils (after Skempton, 1985).

- **Figure 1** also shows the shear strength or shear stress ratio decreases after the peak strength is mobilized with additional shear deformation. This post-peak strength loss is marked by an inflection in the shear stress-displacement relationship that eventually reaches the "critical state" (Skempton 1985) as shown in **Figure 1**. The critical state is achieved by an increase in moisture content, i.e., dilation, and to a lesser extent by particle re-orientation in clayey soil, i.e., high clay-size fraction (%< 2μm) soil. This critical state is the FSS as proposed by Skempton (1970 and 1985). The critical strength was developed to explain the strength mobilized in slopes that had not experienced prior sliding but undergone years of softening and other factors. In other words, the FSS corresponds to the average strength mobilized along the observed failure surface in case histories that Skempton (1970 and 1985) investigated while developing recommendations for shear strengths that should be used in stability analyses of slope that had not undergone prior landsliding. These slopes are referred to as first-time slides by Skempton (1970 and 1985). Skempton (1970) then developed a procedure for estimating the FSS for design of first time slide slopes by equating the average strength mobilized along the observed failure surface in a case history to the critical state strength, which could be estimated by measuring the peak strength of a normally consolidated specimen as discussed below.

  Skempton (1970) equated the critical state to the shear strength mobilized along a failure surface after the effects of overconsolidation, particle bonding, and/or interlocking have been removed or lost but the particles are still primarily in an edge-to-face arrangement instead of being primarily oriented parallel to the direction of shear as in a residual strength condition. Therefore, the critical state corresponds to the peak strength of a normally consolidated specimen, which does not exhibit a significant decrease in strength or stress ratio with increasing displacement (see **Figure 1**(a)). Similarly, **Figure 1**(b) shows the critical strength corresponds to the peak strength of a normally consolidated specimen, which <u>does</u> exhibit a significant decrease in strength with increasing displacement because the clay-size fraction (CF) is greater than 40%. This provides some initial insight to the importance of clay-size fraction and plasticity on the magnitude of the FSS discussed below.

- After a large amount of shear displacement, the shear strength reduces to a nearly constant value that is called the "residual strength." The "residual strength" is reached through re-orientation of some to most of the particles parallel to the direction of shear. The level of reorientation depends on the CF and plasticity of the soil (Stark and Eid, 1994). For CFs less than 20% (see **Figure 1**(a)), the particles are primarily granular, so rolling shear behavior occurs and there is limited reorientation of the clay particles parallel to the direction of shear. Conversely for CFs greater than 40% (**Figure 1**(b)), the particles are mainly platy and significant reorientation parallel to the direction of shear can occur causing a significant decrease in strength from the critical state, i.e., FSS, to the residual strength. As shown below in empirical correlations, higher CF soils exhibit lower FSS and residual strengths therefore correlations should include CF and thus shear behavior, e.g., rolling, transitional, and sliding, in their strength estimates. For example, Lupini et al. (1981) present the following three shear behavior groupings: less than or equal to 25%, between 25 and 50%, and greater than or equal to 50%, for rolling, transitional, and sliding shear behavior, respectively.

An example of this shear behavior and the resulting strength envelopes is shown in **Figure 2**, which shows the range of strengths for Upper Cretaceous Pierre Shale from near Rapid City, South Dakota (Contreras, 2003). Consolidated-drained triaxial compression tests were performed to measure the peak strength of intact and weathered Pierre Shale. Torsional ring shear tests were used to measure the FSS and residual strength of reconstituted material from the weathered Pierre Shale. **Figure 2** shows the range of strength envelopes and all of the strength envelopes are effective stress dependent. For illustration purposes, the four effective normal stress dependent strength envelopes are summarized below:

- Peak Intact Strength Envelope: secant friction angle of 45 degrees at low effective normal stress (σ'_n) of 28 kPa and about 29 degrees at high σ'_n of 287 kPa,
- Peak Weathered Strength Envelope: secant friction angle of only about 24 degrees,
- FSS: secant friction angle of 22 degrees at σ'_n of 50 kPa to about 16 degrees at σ'_n of 400 kPa, and
- Residual: secant friction angle of 11 degrees at σ'_n of 50 kPa and about 7.5 degrees at σ'_n of 700 kPa.

This large range of shear strength will result in a large range in calculated factor of safety so selecting the strength envelope that models field conditions is vital to correctly predicting slope behavior and stability.

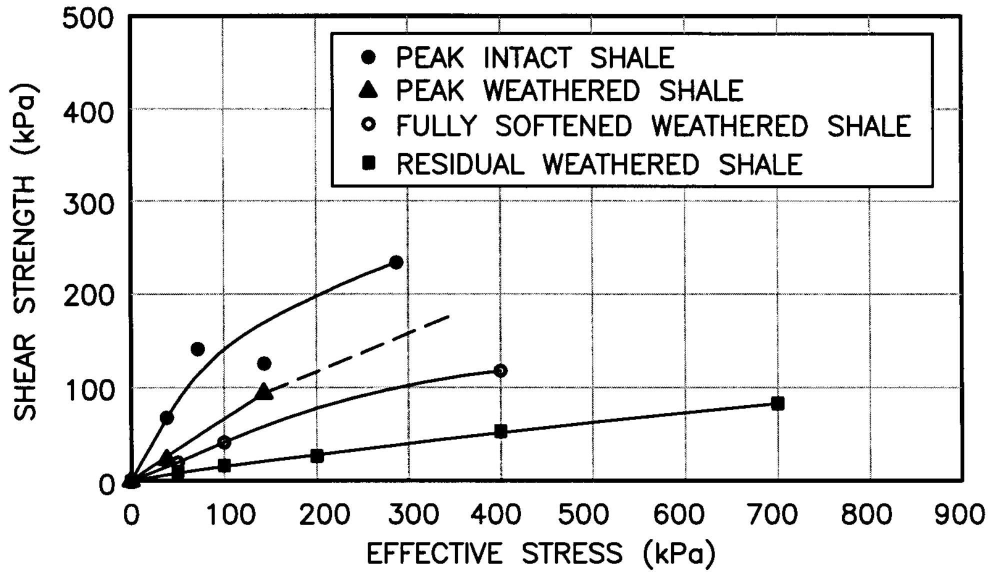


Figure 2. Drained strength envelopes for Pierre Shale from Contreras (2003).

4. Shear Strength Selection Consequences (Why do I care?)

This section uses three case histories to illustrate the adverse consequences that strength loss from the peak strength to the FSS can have on slope stability and why understanding this strength loss is so important for slope stability analyses. Duncan (2013) states the three most important factor in slope stability are: shear strength, shear strength, an shear strength.

The first case involves a railway excavation in Brown London Clay in which failure occurred 46 years after the excavation and illustrates the large strength loss that can occur even in heavily overconsolidated clays due to weathering, softening, and progressive failure processes discussed below. The second involves the decrease in strength that can occur in semi-arid climates due to wet-dry cycles that reduce the peak strength of a compacted soil fill to the FSS. The third case involves the large and rapid decrease in strength from the peak strength to the FSS that can occur upon sustained wetting or soaking of unsaturated soil with high suction pressures. All three cases resulted in slope failure even though using the peak strength resulted in calculated factors of safety significantly greater than unity.

(a) Undisturbed Soil Peak strength v. FSS Case History

The first case history involves a classic natural cut slope in Brown London Clay and is the first site investigated by Skempton (1970) in terms of effective stresses. The railway excavation at Sudbury Hill occurred in about 1903 and the slope failure occurred in 1949 (see **Figure 3**). Effective stress stability analyses by Skempton (1977) of this first-time slide showed the value of effective stress cohesion intercept (c') decreased to a small value during the intervening 46 years. This contributed to Skempton (1970) concluding the softening process results in a decrease in c' with time and it approaches zero (0) after several decades (Skempton, 1970). This decrease in c' with time was attributed to softening caused by many factors including stress relief, applied shear stresses, weathering, and swelling.

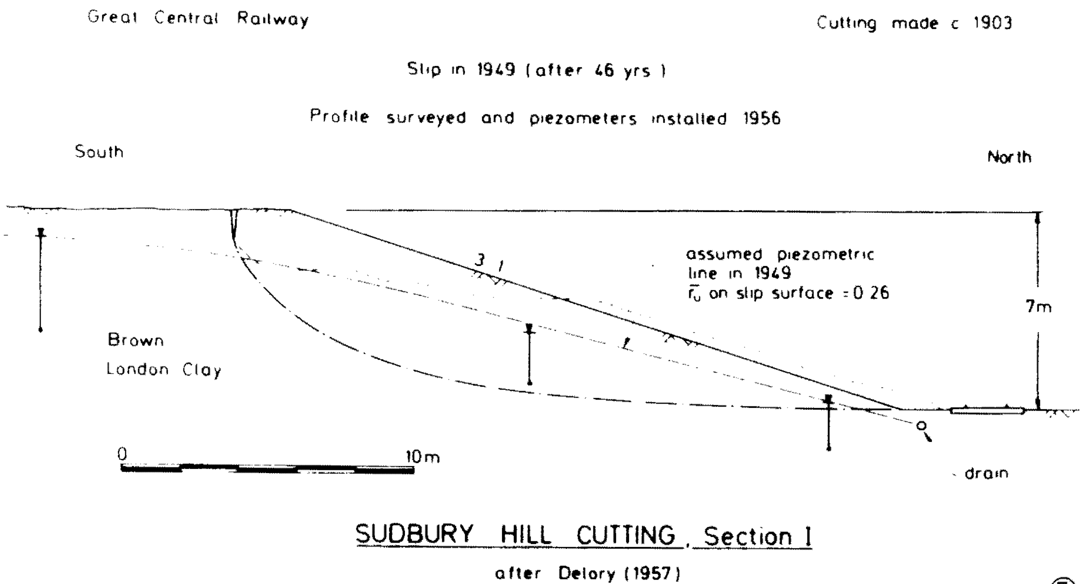


Figure 3. Slope failure in excavated overconsolidated clay at Sudbury Hill from Skempton (1977).

(b) Compacted Peak strength v. FSS Case History

The second case history involves a compacted fill slope that remained stable for about six years after compaction at a Modified Proctor Relative Compaction greater than 90% and a moisture content 1 to 3% above optimum. During this time, the compacted fill slope was wetted by limited precipitation in this semi-arid climate but continuous irrigation by an automatic sprinkler system. **Figure 4** shows the failed compacted fill slope with a shallow failure surface due to a limited depth of softening. Inverse stability analyses yielded a strength that is in agreement with the FSS. This softening from the compacted peak strength to the FSS is primarily due to dissipation of suction pressures, cycles of wetting and drying under the applied shear stresses, and development of a piezometric surface in the compacted fill slope after being initially unsaturated.

Figure 4. Slide in compacted fill slope after six years.

(c) Desiccated Peak strength v. FSS Case History

The third case history involves the 1981 upstream slide in San Luis Dam (now known as B.F. Sisk Dam) in California, which is described in VonThun (1985) and Stark and Duncan (1991). Construction of the 305 ft (93 m) high dam was completed in 1967. It is a major off-stream, pumped-storage facility in the California aqueduct system so the dam is frequently subjected to significant fluctuations in reservoir water level (RWL). The slope failure shown in **Figure 5** occurred during an unprecedented drawdown of the RWL by about 180 ft (55 m) in 120 days. The slide was about 1,800 ft (550 m) long along the centerline of the dam crest (Station 120 to 138). Following the slide, field investigations, laboratory testing, and stability analyses were performed and concluded the clayey slopewash material in the foundation below the embankment was responsible for the slide (Stark and Duncan, 1991). The slopewash is a medium to high plasticity clay (liquid limit (LL) = 37 to 45 and plasticity index (PI) = 18 to 21) derived from the weathering and erosion of the sedimentary rocks of the hill incorporated into the dam at this location where the dam has a height of about 200 ft (60 m) (see **Figure 5**).

Figure 5. Slide in upstream slope of San Luis Dam in September, 1981 (photo from California Department of Water Resources).

Laboratory direct shear tests were performed and the resulting drained peak, fully softened, and residual shear strengths of the slopewash (Stark and Duncan, 1991) are shown in **Table 1**. The insitu strength of the highly desiccated slopewash prior to reservoir filling was high because of the large suction pressures created by the semi-arid environment in the Central Valley of California. However, when the slopewash was wetted in a laboratory direct shear device, its strength was reduced immediately to the fully softened value, i.e., same strength as a normally consolidated reconstituted specimen. This behavior is different from that of mechanically overconsolidated clays in which the effects of overconsolidation are not removed upon wetting and thus require years to develop a FSS condition (Skempton, 1977).

In contrast the slopewash, which is overconsolidated by desiccation, quickly reverted to a FSS condition upon soaking by San Luis Reservoir. This strength loss can be repeated in other situations where desiccated soil is wetted by precipitation or an irrigation system causing a significant strength loss and possibly slope instability. For example, at San Luis Dam, the factor of safety of the upstream slope decreased from about 4.0 to about 2.0 with the decrease in strength from the peak to FSS value. Therefore, consequences of wetting in semi-arid climates on shear strength and slope stability can be significant and immediate. The slide in the upstream slope of San Luis Dam was caused by additional strength loss from the FSS caused by shear stresses induced by the various reservoir cycles (Stark and Duncan, 1991) and possibly the colluvial nature of the slopewash.

Table 1. Summary of Measured Slopewash Shear Strengths (from Stark and Duncan, 1991)

Shear Strength Measured	Effective Stress Cohesion (kPa/psf)	Average Effective Stress Friction Angle (deg.)
Peak	278/5,800	39
FSS	0	25
Residual	0	15

(d) FSS v. Residual Strength Case History

Figure 2 shows the large range of shear strength that results in overconsolidated or compacted materials, which will result in a large range in calculated factor of safety so selecting the strength envelope that models field conditions is vital to correctly predicting slope behavior and stability. This section uses a case history to illustrate the consequences of selecting a FSS when prior shear displacement has occurred and reduced the strength to or near the residual value even though there is no recent evidence of slope movement.

The site grading operation for this project involves a large 3H:1V slope excavation at an existing facility. Even though the site is part of a landslide complex, the area of the excavation had not experienced slope movement in known history and was designed using the FSS in accordance with Skempton (1977). However, the slope consists of sheared and slickensided shales, claystones, and mudstones and slid shortly before completion of the excavation as shown in **Figure 6**(a). **Figure 6**(b) shows a close-up of the slickensided shale/claystone that was obtained at the toe of the slide mass shown in **Figure 6**(a) where it crossed an access road. The slickensided shale is present along the slide surface. An inverse analysis of the slide indicates the mobilized shear strength in this slickensided shale layer was at or near the residual value at the time of sliding (Stark and Eid, 1997). This is important because the randomly oriented slickensided surfaces probably connected to create a continuous surface with a shear strength at or near the residual value and not the FSS even though this could be termed a "first time slide" according to Skempton (1977).

(a) (b)

Figure 6. Photographs of: (a) deep slide in shale bedrock in area of no known prior landsliding and (b) close-up of slickensided shale along failure surface.

(e) Importance of Difference between FSS and Residual Strengths

Figure 7 presents the numerical difference between the fully softened (ϕ'_{FSS}) and residual (ϕ'_r) friction angles for the 64 soils obtained using torsional ring shear tests from Stark and Hussain (2013). **Figure 7** can be used to evaluate the practical significance of determining whether or not a continuous pre-existing shear surface is present or can develop in a slope. In natural soils with a liquid limit between approximately 80 and 140%, it is important to determine whether the slope has a pre-existing shear surface because of the large difference (up to 16 degrees) between ϕ'_{FSS} and ϕ'_r friction angles. If a pre-existing shear surface is located or significant slickensided material are present, a residual friction angle should be utilized for these portions of the failure surface. A large difference between ϕ'_{FSS} and ϕ'_r also identifies soils that have a greater potential for progressive failure because they exhibit a greater post-peak strength loss. As a result, two stability scenarios should be considered for soils that exhibit a large difference between ϕ'_{FSS} and ϕ'_r, which are are a factor of safety (FS) greater than 1.5 using the FSS and a FS greater than unity with a residual strength as discussed in more detail below and in accordance with Stark et al. (2005).

Figure 7 also shows the difference in ϕ'_{FSS} and ϕ'_r is greater for shallow failure surfaces, i.e., lower effective normal stresses, than for deep failure surfaces. This is important because most the slope failure in clayey embankments are shallow, i.e., less than 10 ft (3.1 m), as discussed below. In soils with a liquid limit less than about 50%, there is a smaller difference (less than 8 and 6 degrees at an effective vertical stress of 1,000 and 8,300 (50 to 400 kPa), respectively) between the fully softened and residual friction angles. However, this difference between the fully softened and residual values may still adversely affect the slope design depending on the magnitude of effective normal stress acting on the failure surface in this material. In slopes where there is not a large difference between ϕ'_{FSS} and ϕ'_r and where some uncertainty exists on whether a pre-existing shear surface is present or not, it may be prudent to verify the slope design by assigning an appropriate value of ϕ'_r to the relevant materials and ensuring the resulting factor of safety is greater than unity Stark et al. (2005).

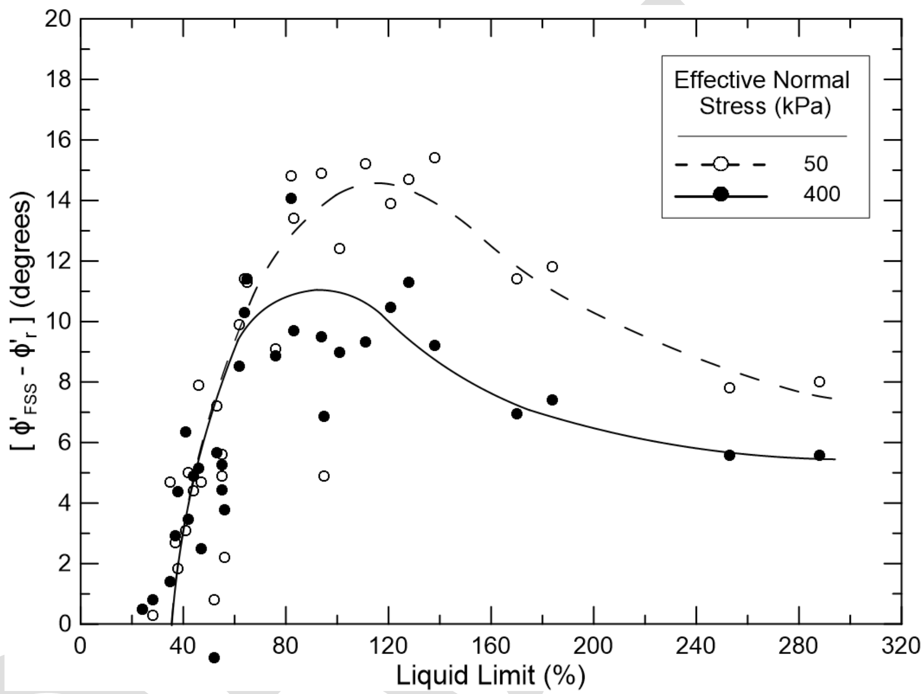


Figure 7. Difference between secant fully softened and residual friction angles as function of liquid limit from Stark et al. (2015).

5. Development of FSS (Where does it develop?)

This section discusses where practitioners should consider use of the FSS because these slope conditions will result in a strength that is close to the peak strength of soil in a normally consolidated state, i.e., FSS condition. The FSS strength is used to reflect the maximum extent of weathering/softening, increase in moisture content/void ratio, stress relief, and/or progressive failure that can occur under these slope conditions. The two main slope conditions that have resulted in development of a FSS condition based on inverse analysis of the slope failure are: (1) cut or excavated slopes in overconsolidated fine grained soils (see **Figure 8**) and (2) compacted fill slopes or embankments (see **Figure 10**) in semi-arid climates, such as highway embankments and levees. The first section below discusses development of a FSS condition in a cut slope while the second major subsection describes development of a FSS condition in a compacted fill slope.

(a) Cut or Excavated Slopes (Stark)

In cut or excavated slopes in heavily overconsolidated fine grained soils a fully softened condition can develop due to stress relief, infiltration of water into the macro and micro-structure of the deposit, and continuing physical and chemical weathering (Terzaghi, 1936). Softening is defined as an increase in water content and/or void ratio due to expansion of soil particles that creates negative pore-water pressures that draw even more water into the soil as well as the joints and fissures, which provides easy access for water to enter and soften the overconsolidated clay.

Figure 8 is a photograph of heavily overconsolidated clay after excavation for a highway widening project. This clay is highly jointed and fissured and it breaks along these discontinuities upon light hand touching as shown in **Figure 8**(a). These discontinuities result in chunks three to five inches (75 to 125 mm) in length being easily removed from the deposit (see **Figure 8**(b)). The presence and opening of these joints and fissures due to a reduction in the applied stresses, e.g., glacial retreat or slope excavation, results in a jointed clay mass that is susceptible to greater infiltration, which is facilitated by the moist climate near Seattle, and softening. In stiff materials joints and fissures can also be facilitated by shear dilation as pointed out by Henkel and Skemption (1954) using the Jackfield Landslide.

(a) (b)

Figure 8. Photographs of highly jointed and fissured overconsolidated clay: (a) in shallow excavation and (b) close-up of joints and fissures and blocky nature of the overconsolidated clay.

Therefore, development of a FSS condition depends on the geologic circumstances present at a site. For example, in heavily overconsolidated clays, such as the excavated clay slope shown in **Figure 8**, a FSS condition can develop via some or all of the steps in the following flowchart (see **Figure 9**), many of which can be occurring at the same time in a slope. For example, infiltration, swelling, weathering will be occurring most, if not, all of the time due to the other factors shown in the flowchart.

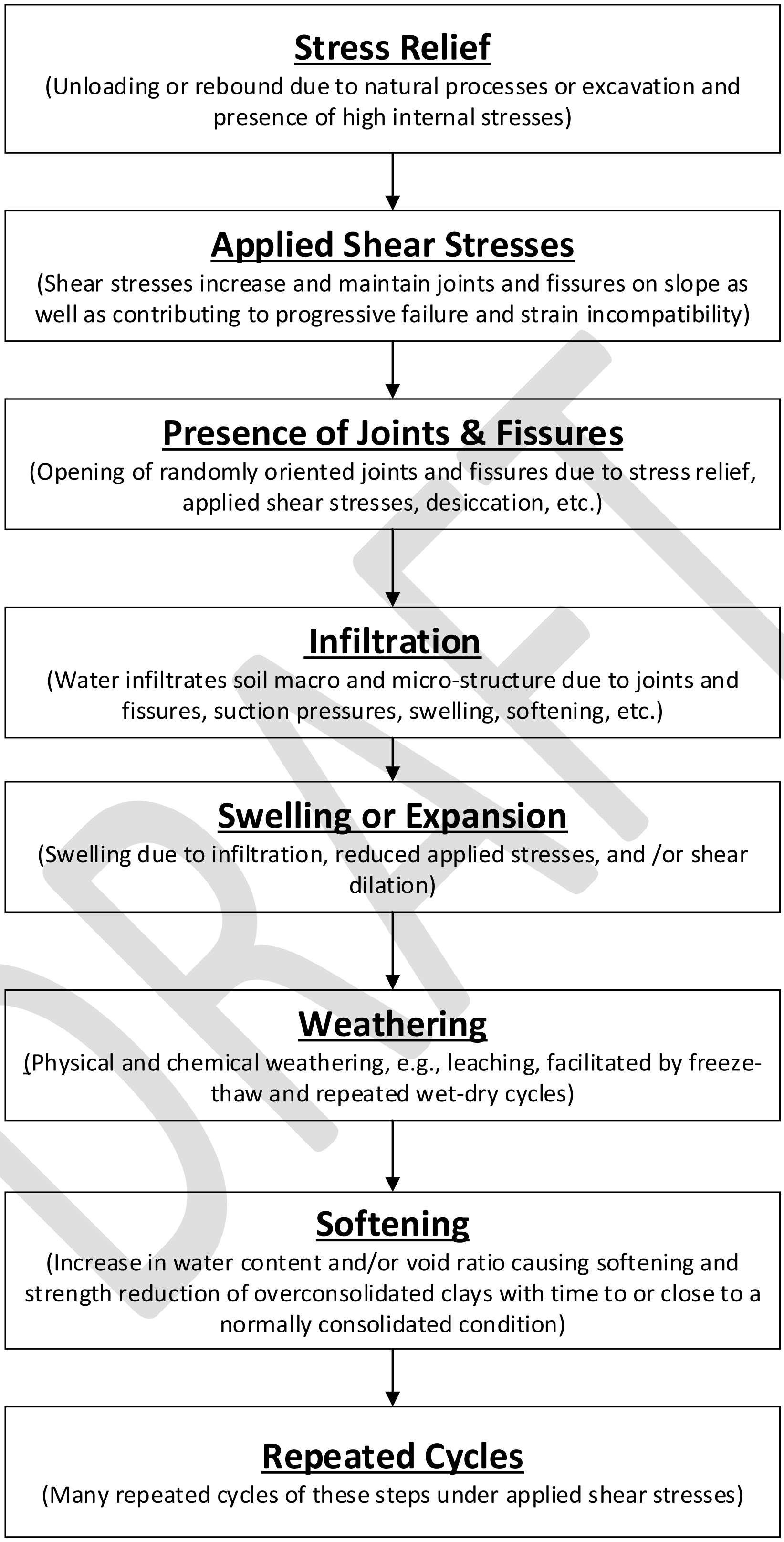


Figure 9. Flowchart depicting development of FSS condition in a cut slope or excavation in heavily overconsolidated clay.

In summary, stress relief and applied shear stress creates, maintains, and increases joints and fissures which allow infiltration into the randomly oriented joints and fissures and subsequent swelling. This can lead to the outer portions of the clay becoming saturated and trapping air in the inner portions (Terzaghi et al. 1996). The pore-water pressures caused by the trapped air can create tension in the soil, which can lead to additional water being absorbed, reduced effective stresses, and softening to a greater depth (Terzaghi et al. 1996). As a result, FSS is more likely to develop quicker and to deeper depths in sloping versus level ground sites. FSS is also more likely to develop in uncovered slopes versus covered slopes because water can infiltrate easier. These generalities are used subsequently to present recommendations for possible mitigation measures for reducing the potential for a FSS condition to develop in Section 8 below.

- Depth of FSS in Cut Slopes

The FSS should be used when analyzing the long-term stability ~~of clay embankments or~~ excavations or cuts in overconsolidated clay because the softening processes discussed above may have sufficient time to occur. If the slope or excavation will be temporary, a strength greater than FSS may be suitable because there is not sufficient time to fully soften the material using the steps in the flowchart above. If the slope or excavation will be permanent or for a long duration, the depth to which full softening can occur becomes important because it dictates the depth to which the FSS should be applied in stability analyses instead of the peak shear strength. In other words, if the overconsolidated clay is below the depth of softening it can be assigned a shear strength greater than that of the FSS, which facilitates stability and slope design.

In general, the FSS is more likely to develop at shallow depths because cracks and joints can open to shallow depths and allow water to infiltrate and start, continue, and expand the softening process. This depth could be estimated from the depth of the active zone (CAGE, 1999). At deeper depths, the insitu stresses can be high enough to prevent sufficient cracks and joints from opening, which eventually limits water infiltration and softening. Recommendations for searching for the critical failure surface using the applicable peak strength and then assigning the FSS (Mesri and Abdel-Ghaffar, 1993) are discussed below.

Table 2 presents depths to failure surfaces in "first-time slides", i.e., slopes that had not experienced prior landsliding, and should have mobilized a FSS (Skempton, 1977). These depths provide an indication of the depth to which all of the mechanisms, not just those shown in **Figure 9**, can occur and reduce the shear strength of a heavily overconsolidated clay from the peak strength to the FSS. **Table 2** shows the depth to the "first time" slip surface varies from 4 to about 65 feet (1.2 to 19.8 m) which suggests a FSS could develop in heavily overconsolidated clays from the ground surface to a depth of about 65 feet (19.8 m). Based on **Table 2**, a depth to application of a FSS could be as deep as 60 feet but most of the case histories indicate a depth of 20 to 25 feet. Therefore, a minimum depth for applying a FSS should should be at least 20 to 25 feet in heavily overconsolidated clay. However, additional research is needed on the depth to which a FSS should be applied in slope stability analyses.

- Time to FSS for Cut Slopes

The FSS condition corresponds to the maximum water content and degree of jointing and fissuring that the overconsolidated clay will experience for the stress relief, applied shear stress, and environmental conditions present and illustrated in **Figure 9**. These phenomena help explain why slope failures in overconsolidated clays can take over 70 years to occur (see **Table 2**). Substantial time is required even for surficial water infiltration, softening, weathering, and strength loss to occur because of the low hydraulic conductivity of the clay matrix. For design purposes, it is recommended that a FSS be used to the depth to which a FSS will develop for permanent structures. If the project involves a temporary structure or excavation, the time shown in **Table 2** can be used to estimate how long a peak strength can be relied upon in stability analyses. In general, if the temporary structure or excavation will not exceed a duration of three years (shortest duration in Table 2), the insitu peak strength can be used based on the data in **Table 2**. Additional research is also needed on the time required to develop a FSS for slope stability analyses because the data in **Table 2** is limited.

Table 2. Summary of "First-Time Sides" in Overconsolidated Clays

Site Name and Location	Time between excavation and Slide (Years)	Depth of Slip Surface (feet)	Slope Inclination	Reference
Watford By-Pass	7.5	4	1.5H:1V	Skempton (1948)
Kensal Green	28	11	1.5:1	Skempton (1948)
Selset	6	12	2:1	Skempton and Brown (1961)
Walthamstow	47	14	1.5:1	Skempton (1948)

West Acton	50	16	3:1	Skempton (1977)
Crews Hill	47	20	3.3:1	Skempton (1977)
Kingsbury	16	20	2.25:1	Skempton (1977)
St. Helier	22	23	2:1	Skempton (1977)
Sudbury Hill	46	23	3:1	Skempton (1977)
Cuffley	35	24	2.75:1	Skempton (1977)
Mill Lane	40	24	1.5:1	Skempton (1948)
Park Village East	18	26	1.5:1	Skempton (1948)
Hadley Wood	~65	34	3.7:1	Skempton (1977)
Grange Hill	48	40	3.25:1	Skempton (1977)
Wembley Hill	13	45	1.5:1	Skempton (1948)
New Cross	3	56	1.5:1	Skempton (1977)

(b) Compacted Fill Slopes (Stark)

In the 1960s, the Texas Highway Department, now called the Texas Department of Transportation, was experiencing problems with shallow slope failures in highways embankments. A survey of these failures, performed by Abrams and Wright (1972), shows the height of the slopes that failed range from 20 ft to 40 ft, with slope inclinations ranging from 2H:1V to 3H:1V. Abrams and Wright (1972) found the majority of the failures were occurring in compacted embankments of high plasticity clays. The failures tended to be shallow and semicircular, did not involve the entire slope, and usually developed numerous years after construction.

Subsequently, Saleh and Wright (1997) suggest an approximation of the slide depth is 20 percent of the slope height (H) or:

$$\text{Slide Depth} = 0.2 * H \qquad (2)$$

Rogers and Wright (1986), Kayyal and Wright (1991), and Wright et al. (2007) used triaxial compression tests to show that cycles of wetting and drying can decrease the drained peak shear strength of compacted high plasticity clays toward a lower boundary that is about equal to the FSS. Similarly, Kovacevic et al. (2001) show that cycles of wetting and drying can also cause progressive failure to develop in compacted clay embankments, thereby decreasing the mobilized shear strength. Wright et al. (2007) found that although cycles of wetting and drying decrease the shear strength, it is not always decreased to the FSS.

Based on these studies, it appears that weathering, described below, is the main cause of the decrease in shear strength in compacted clay embankments although progressive failure may also play a s role in this process. Therefore, development of a FSS condition depends on the environmental circumstances present at a site. For example, in the compacted levee system shown in **Figure 10** a FSS condition can develop via some or all of the steps in the following flowchart (see **Figure 11**), many of which can be occurring at the same time in a slope. For example, infiltration, swelling, weathering will be occurring most, if not, all of the time due to the other factors.

Figure 10. Photograph of shallow slides in levee system embankment

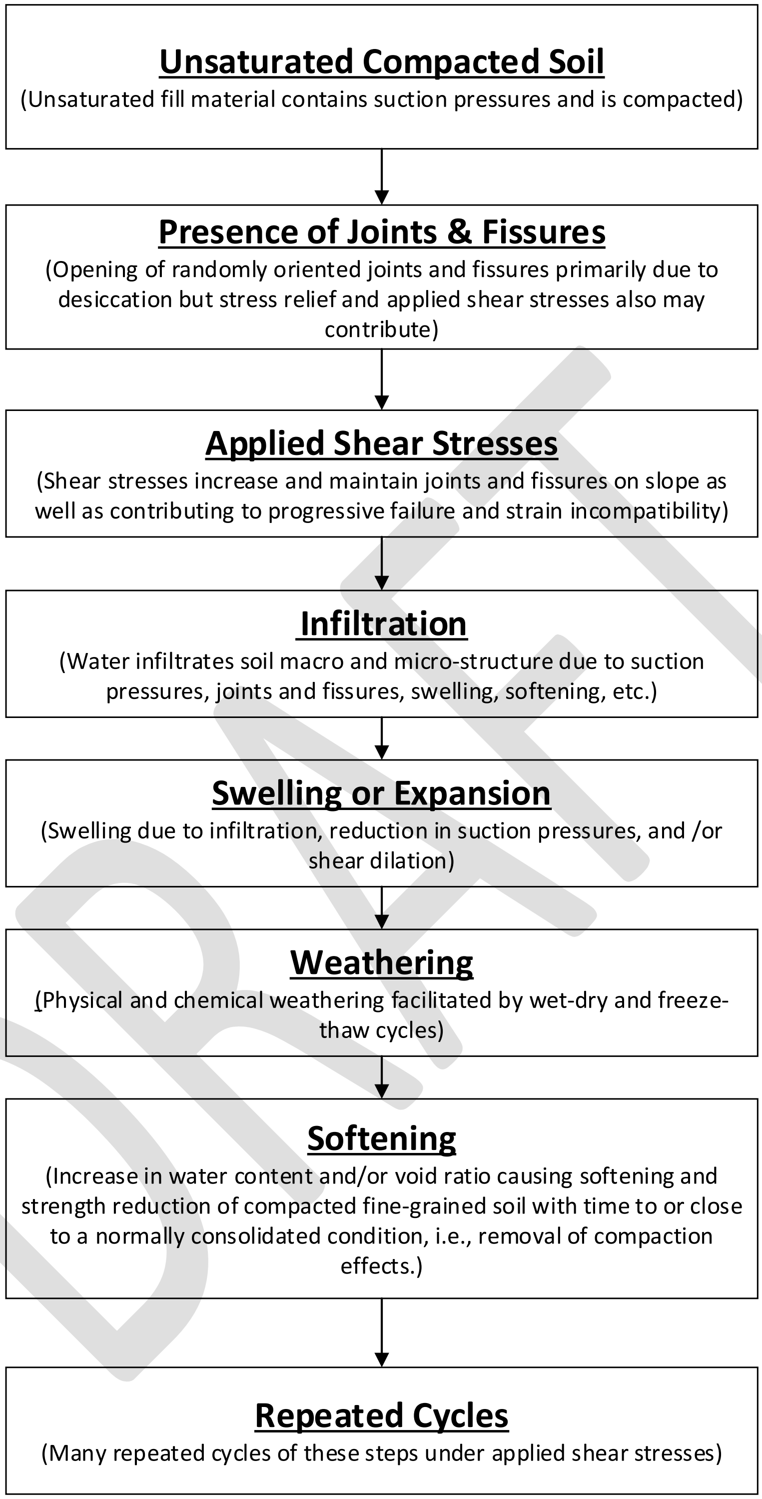


Figure 11. Flowchart depicting development of FSS condition in a compacted embankment.

These steps are in general agreement with Terzaghi (1936) which describes it as water accumulating in fissures and the clay swelling under zero pressure along the walls of the open cracks. This nonuniform swelling weakens the soil fragments and new cracks form causing a progressive softening and loss of strength (Terzaghi, 1936).

The U.S. Army Corps of Engineers (USACE, 1983) and Stauffer and Wright (1984) document 74 case histories of first-time slope failures in compacted clay embankments. **Table 3** presents the 37 first time slope failures from Stauffer and Wright (1984) because data on the time to failure and the depth of the observed slip surface are available. These case histories show that these embankment failures share the following characteristics:

- Soils consist of mostly high plasticity clays with liquid limit and plasticity index greater than 40 and 20, respectively,
- Failures were usually less than 5 feet (1.5 m) deep, and
- Failures mostly occurred less than 20 years after construction.

This suggests that slopes constructed of fine-grained soils with liquid limit and plasticity index less than 40 and 20, respectively, are not susceptible to as large a strength reduction due to the softening mechanisms discussed above.

Table 3. Summary of "First-Time" Sides in Compacted Fill Slopes from Stauffer and Wright (1984).

Site Name and Location	Time between excavation and Slide (years)	De of Slip (fe
IH 610 at Scott St., NE quadrant, Harris Co.	17	1
SH 225 at SH 146, SW quadrant, Harris Co.	31	1
SH 225 at SH 146, NW quadrant, Harris Co.	31	0
SH 225 at SH 146, NE quadrant, Harris Co.	31	1
SH 225 at Southern Pacific RR Overpass, SE quadrant, Harris Co.	20	1
SH 225 at Southern Pacific RR Overpass, SE quadrant, Harris Co.	20	1
SH 225 at Southern Pacific RR Overpass, SW quadrant, Harris Co.	20	1
SH 225 at Southern Pacific RR Overpass, NW quadrant, Harris Co.	20	0
IH 225 at Scarborough, SE quadrant, Harris Co.	17	1
IH 610 at SH 225, SE quadrant, Harris Co.	19	0
IH 610 at Richmond St, SW quadrant, Harris Co.	18	1
IH 10 at Crosby-Lynchburg, NW quadrant, Harris Co.	25	1
IH 45 at SH 146, SE quadrant, Harris Co.	14	1
IH 45 at SH 146, south side, Harris Co.	14	1
IH 45 at FM 2351, NE quadrant, Harris Co.	12	0
IH 45 at College St., NE quadrant, Harris Co.	18	0
U.S. 59 at FM 525, NE quadrant, Harris Co.	24	1
U.S. 59 at Shepard St., SE quadrant, Harris Co.	22	1
U.S. 79 at U.S. 95, SE quadrant, Williamson Co.	11	2
U.S. 77 at SH 21, SW quadrant, Lee Co.	19	1
U.S. 77 at SH 21, NW quadrant, Lee Co.	19	1
U.S. 290–5 miles east of IH 35, NW quadrant, Travis Co.	–	
U.S. 77 at SH 21, NW quadrant, Lee Co.	16	2
U.S. 87 at Loop 175, NW quadrant, Victoria Co.	19	1
Loop 286 at SH 271 Interchange, NW quadrant, Lamar Co. Loop 286 at Missouri Pacific RR Overpass, SW quadrant, Lamar Co.	14	1
Loop 286 at SH 271 Interchange, NW quadrant, Lamar Co.	18	2
Loop 286 at Missouri Pacific RR Overpass, SW quadrant, Lamar Co. Loop 286 at Missouri Pacific RR Overpass, SW quadrant, Lamar Co.	18	1
Loop 286 at Missouri Pacific RR Overpass, NW quadrant, Lamar Co.	–	
Loop 286 at Missouri Pacific RR Overpass, SW quadrant, Lamar Co.	18	3
Loop 286 at FM 79, SW quadrant. Lamar Co.	19	1
Loop 286 at FM 79, SW quadrant. Lamar Co.		

(b) Effect of Clay Mineralogy on FSS

Clay mineralogy has been known to have a significant effect on behavior of soils (Terzaghi et al., 1996). Despite the substantial influence of clay mineralogy on the shear strength of soils, little data is available on its influence on the FSS. Tiwari and Ajmera (2011) prepared thirty-six laboratory samples from sodium montmorillonite, kaolinite, and quartz. These samples were used to measure the FSS at four different normal stresses ranging from 1,000 to 4,200 psf (50 to 200 kPa). **Figure 12** shows the relationship between clay content (% passing No. 200 sieve) and the average fully softened friction angle, which separates the soils in terms of mineral composition. This data shows the type and amount of clay mineral substantially affects the FSS. In particular, the data indicates that mixtures of montmorillonite with quartz exhibit a fully softened friction angle that follows two distinct trends: (1) soils with less than 25% clay mineral content and (2) soils with greater than 25% clay mineral content.

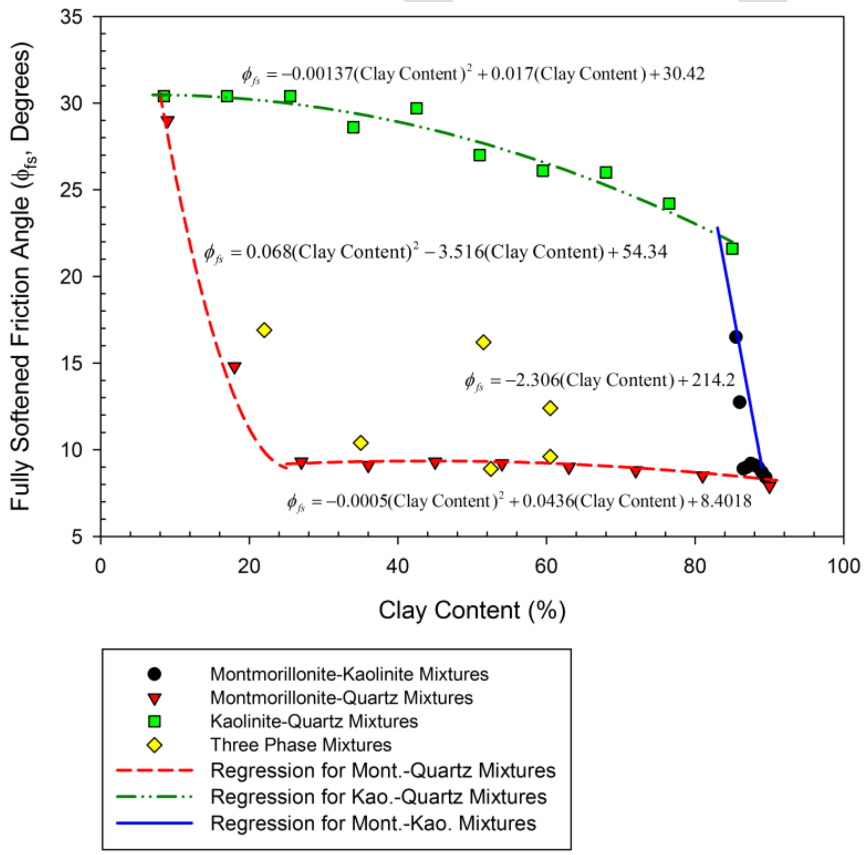


Figure 12. Relationship between fully softened friction angle and clay content from Tiwari and Ajmera (2011).

This data was used to develop a relationship between fully softened friction angle and liquid limit, which is shown in **Figure 13.** In these two phase mixtures, the fully softened friction angle can be estimated from the relationship presented in **Figure 13.**

A similar relationship between the plasticity index and the fully softened friction angle is presented in **Figure 14**. For this case, although it was possible to generalize the relationship in terms of mineralogical composition, different correlations were noted for soils with plasticity indices less than 80 and for soils with plasticity indices greater than 80.

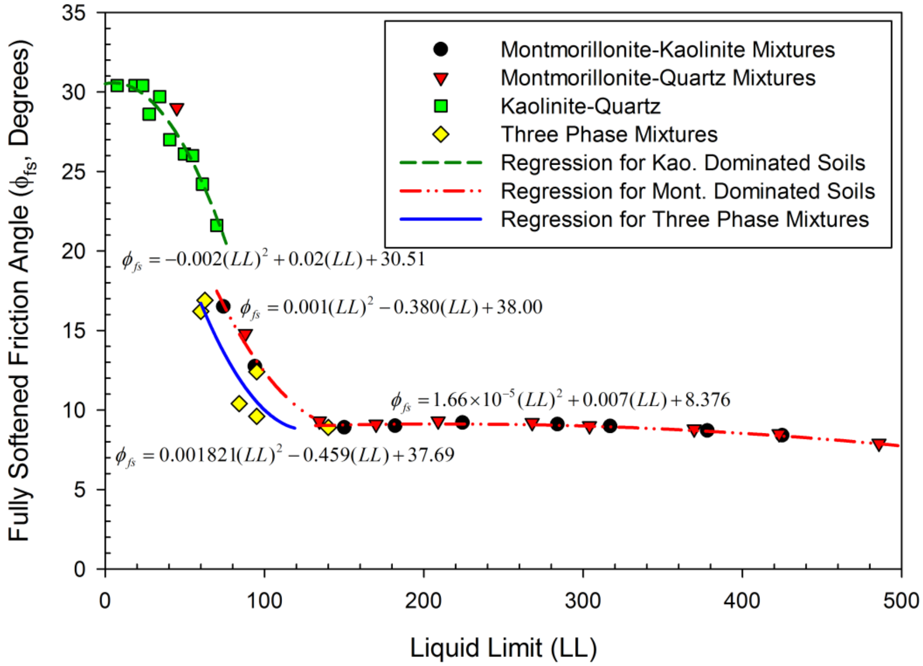


Figure 13. Relationship between liquid limit and fully softened friction angle from Tiwari and Ajmera (2011).

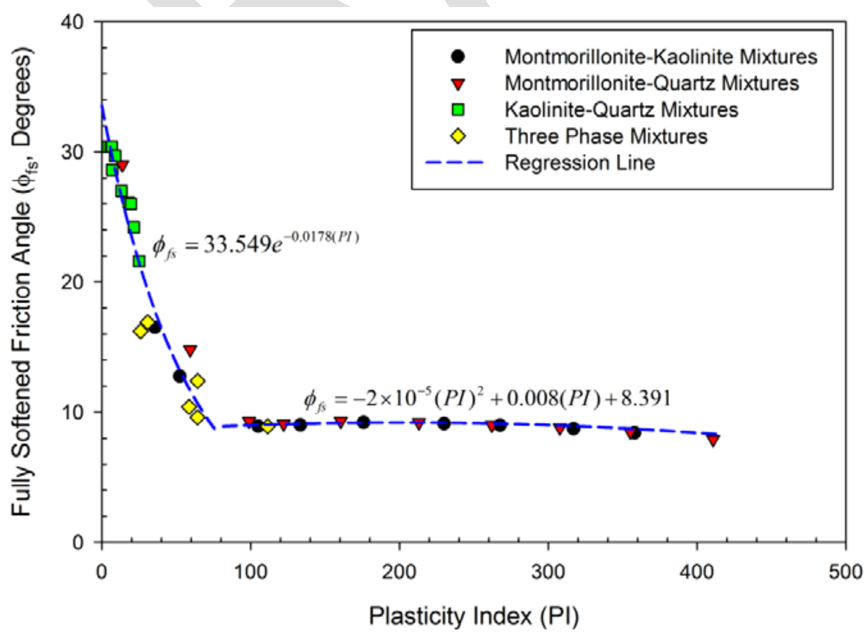


Figure 14. Relationship between fully softened friction angle and plasticity index from Tiwari and Ajmera (2011).

(c) Effect of Pore-Water Chemistry on FSS

It is well known that the shear strength of clays is controlled in part by adsorbed cations (Mitchell 1993). Recent research indicates that clay mineralogy and system chemistry may have a more significant influence on the strength behavior than once thought. The weathering process involves cation exchange of adsorbed ions on clays, particularly smectites. However, little research has been conducted on the effects of system chemistry (pore fluid) on the FSS. Tawari and Ajmera (2014) present data on pore-water chemistry effects on FSS and determined the FSS for mixtures of montmorillonite, kaolinite, quartz, powdered and reconstituted intact rock, and slide surface materials using distilled and saline (NaCl) waters. The test results show little effect on FSS on the mineral mixture samples. However, large effects on FSS were found with the intact rock and sliding surface materials. For these tests, the samples with saline water had significantly higher FSS than those tested with distilled water (see **Figure 15**). Tiwari and Ajmera (2015) use these results to conclude:

- FSS is reduced with reductions in NaCl pore fluid concentration, especially for soils with more than 9,5% montmorillonite content. The reduction in fully softened friction angle with NaCl leaching increases with the initial concentration of NaCl in the pore fluid. As a result, this behavior should be considered in coastal areas or marine deposits.
- The effect of NaCl concentration in the pore water on the FSS was negligible or even opposite for the soils containing 9.5% or less montmorillonite content or kaolinite as the dominant clay mineral.
- There is a good relationship between the ratios of liquid limit or plasticity index and NaCl concentration and FSS.

These findings indicate the chemical makeup of the water used in preparing reconstituted samples can have an effect on the measured FSS and should be a consideration when developing a FSS testing plan and when selecting the FSS for use in stability analyses.

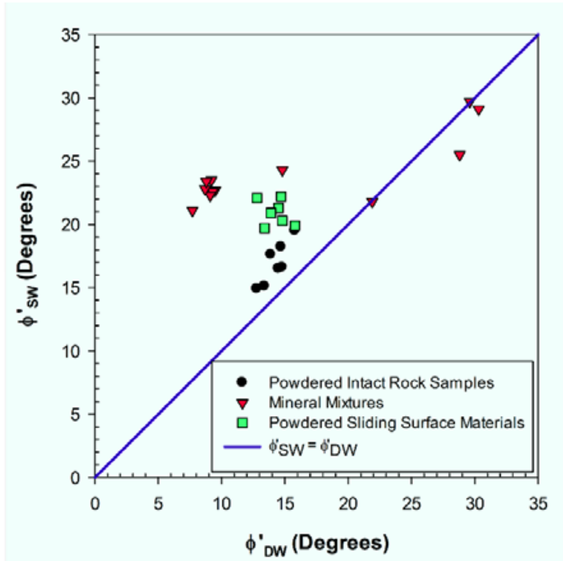


Figure 15. Comparison of fully softened friction angle of mineral mixtures prepared with distilled water (DW) and saline water (SW) for powdered and reconstituted intact rock specimens before and after NaCl leaching and powdered and reconstituted sliding surface materials (from Tiwari and Ajmera, 2014).

6. FSS Measurement and Representation (How do I measure it?) (Stark)

This section discusses techniques for measuring and representing the FSS for use in slope stability analyses. This includes applicable shear device or apparatus, specimen preparation, shear displacement rate, hydration, and data interpretation. The objective of the specimen preparation and shear testing is to measure the peak strength of normally consolidated randomly oriented particles in accordance with Skempton (1970). Techniques for measuring the FSS is a topic of ongoing research so this section presents the current state-of-the-art and state-of-the-practice and references that can be pursued for future study.

(a) Shear Devices

Three laboratory shear devices, i.e., triaxial compression, direct shear, and torsional ring shear, have been used to measure the FSS and each are reviewed in this section. The first test discussed in triaxial compression.

The fully softened shear strength has been measured using the triaxial compression test since Skempton (1970) introduced the FSS concept to slope stability analyses (Gibson 1953; Bishop et al. 1965; Skempton 1977). Values of FSS measured using the triaxial compression test are in agreement with the average mobilized strength along the observed slip surface from inverse analyses of first-time slides (Skempton, 1977). However, there are some challenges with performing consolidated-drained (CD) triaxial compression test (ASTM D7181) on a normally consolidated fine-grained specimen especially at low confining pressures to simulate shallow slip surfaces, such as:

- having to support the specimen with a mold during preparation because of the high moisture content of the reconstituted material,
- difficulty in testing at low confining stresses due to end and piston friction and supporting the specimen, which are important because the FSS strength envelope is effective normal stress dependent at low confining stresses, and
- length of time to perform an isotropically CD triaxial compression test because of the low hydraulic conductivity of fine grained soils and length of drainage path.

The fully softened shear strength should not be measured using isotropically consolidated-undrained (CU) triaxial compression tests (ASTM D4767) with pore-water pressure measurements and using stress-path tangency to define the drained friction angle. The drained friction angles obtained from such CU triaxial compression tests are not in agreement with the values obtained from direct shear (Duncan et al., 2015) and CD triaxial compression tests (Skempton, 1970). For example, **Figure 16** shows drained friction angles for normally consolidated clays from southeast Louisiana and the CU triaxial compression values are significantly greater than the drained direct shear values. The CU triaxial compression values are significantly greater than drained direct shear probably due to: difficulties in achieving full saturation of the specimen prior to shear, i.e., achieving a pore Skempton (1954) pressure coefficient, B, of unity and all of the shear-induced pore pressures not migrating quickly from the mid-height and middle of the specimen to the top and bottom platens to be measured and reflected in Skempton's (1954) pore pressure coefficient A. In particular, if the B value is less than unity the specimen contains suction pressures which contribute to the shear resistance. If all of the shear-induced pore pressures are not measured, the resulting effective stress is over-estimated which reduces the effective stress friction angle or strength envelope by plotting the measured deviator stress at a higher value. In addition, back-pressure saturation of a fine grained specimen to perform a CU triaxial compression test with pore-water pressure measurements is difficult and time-consuming to achieve full saturation in a commercial laboratory.

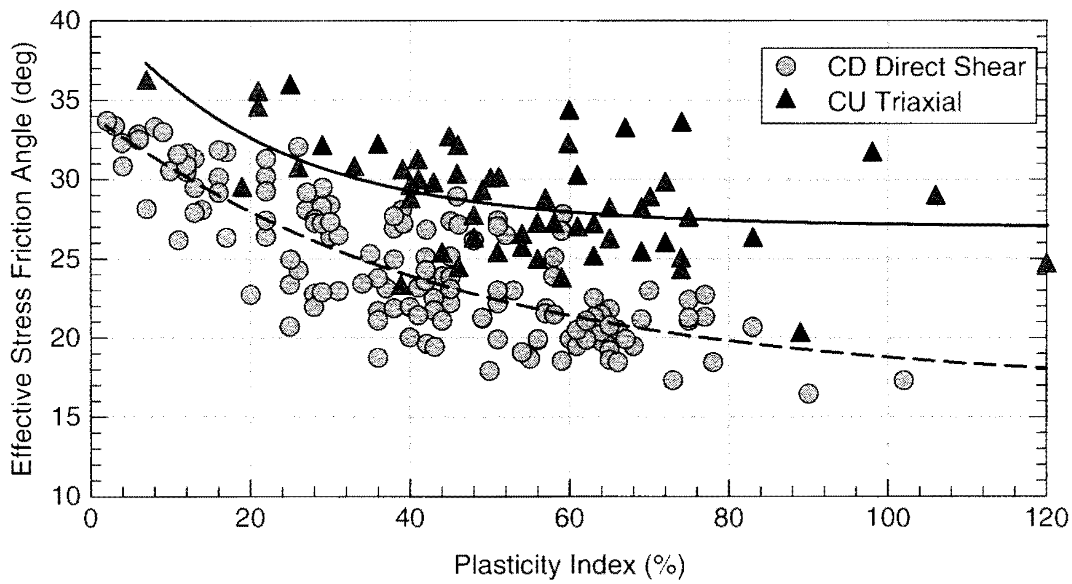


Figure 16. Drained frication angles determined from C-U triaxial compression and C-D direct shear tests on southeast Louisiana alluvial clays (from Duncan et al., 2015).

Because of challenges performing CD triaxial compression tests, the inapplicability of CU triaxial compression tests, and the availability of direct shear devices (ASTM D3080), the FSS also has been measured using the direct shear apparatus. In addition, a direct shear specimen is initially about one-inch thick prior to consolidation so it is possible to locate the shear surface within the specimen if care is taken during consolidation as discussed below. However, the direct shear device also has limitations for measuring the FSS, such as:

- consolidating a high moisture content material and having sufficient specimen above the shear surface after consolidation for proper shearing is difficult because the specimen is near the liquid limit and the top half of the shear box is usually only about one-half inch thick (see **Figure 17**),
- progressive failure occurs in the normally consolidated specimen,
- applying enough shear displacement to form a continuous shear surface across the specimen and mobilize the peak strength of the normally consolidated specimen within one travel, i.e., about 0.2 to 0.3 inches, of the shear box,
- because of the normally consolidated condition of the specimen, a gap can develop at the leading edge of the specimen during shear (see **Figure 17**),
- setting and maintaining the gap between the upper and lower portions of the shear box,
- soil extrusion from this gap during consolidation and shear,
- friction generated if the upper and lower portions of the shear box are not separated sufficiently, and
- variable cross-sectional area during shear so the results need to be corrected.

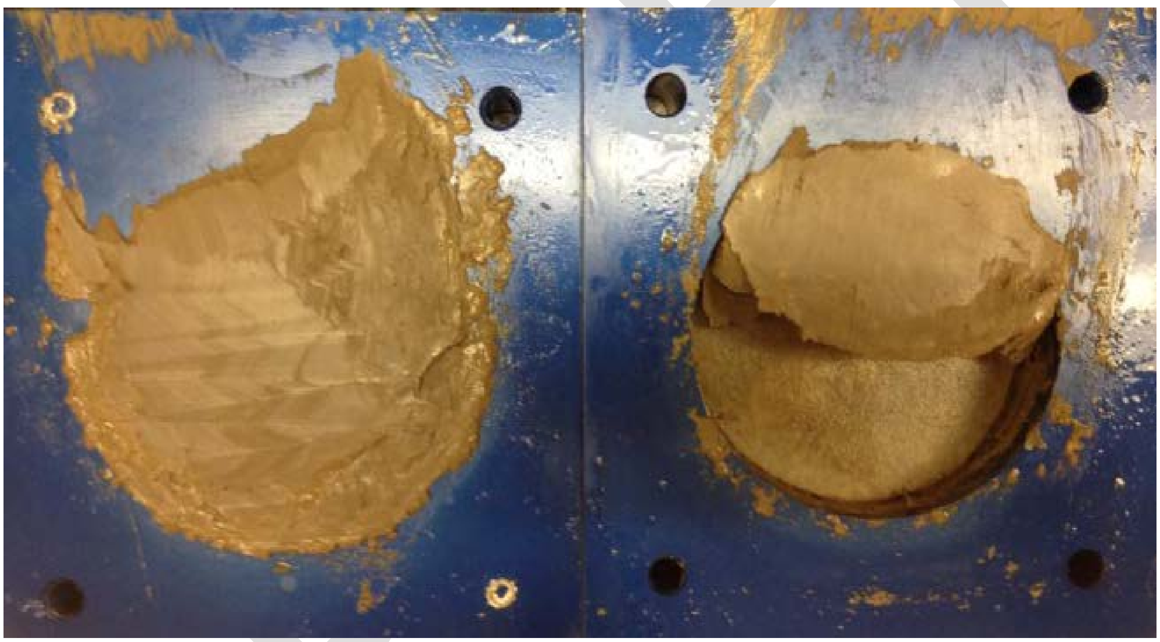


Figure 17. Photograph of direct shear specimen after shearing at an effective normal stress of only 3,016 psf showing little soil in the top shear box and a gap between the soil and the shear box wall in the top box (right photograph) indicating progressive failure and change in specimen area.

Some of the limitations can be overcome by modifying the test protocol in ASTM D3080 to allow repacking the upper shear box with reconstituted material during the consolidation phase to ensure there is sufficient specimen above the shear surface for proper shearing. If a compacted specimen is being prepared, the two halves of the direct shear specimen can be compacted and consolidated in a separate device and the assembled in the shear box so most of the consolidation settlement is removed before placement in the shear box. This ensures sufficient soil is above the shear surface before shearing. Regardless of the procedure, the vertical displacement should be carefully measured during the consolidation and shear phases to determine how much soil is above the gap between the upper and lower portions of the shear box and this information should be included in the test report.

In spite of these limitations, some researchers have shown some agreement between the FSS measured using CD triaxial compression and drained direct shear even though there is a difference in the mode of shear and test limitations some of which are described above, e.g., De Mello (1946), Skempton (1964), Moon (1984), Thomson and Kjartanson (1985), and Maccarini (1993). Review of these studies indicate most of the reported agreement is due to comparison of linear strength envelopes instead of effective normal stress dependent strength envelopes as highlighted by Stark and Edi (1997). It is now accepted that the FSS is effective normal stress-dependent and comparisons should be made between stress-dependent envelopes, which is a focus of ongoing research.

Because of the limitations of CD direct shear devices and challenges in performing CD triaxial compression tests, the torsional ring shear device also has been used to measure both FSS and residual strength (Hvorslev, 1960, 1969; Stark and Eid 1997; Stark et al. 2005, Stark et al., 2015). The ring shear device was designed by Hvorslev (1936) to investigate the shear stress-displacement behavior of soils because Terzaghi was interested in shear resistance after failure. The ring shear device allows large shear displacements to be applied in one direction to a shear surface, which is sufficient to measure both the FSS and residual strengths using the same or different specimens Hvorslev (1936, 1937, and 1939). Therefore, only one device is necessary to measure both fully softened and residual strengths. Some of the other advantages include: constant normal stress, engaging or shearing the entire specimen at one time, an ASTM Test Method (ASTM D7608) is available, and little supervision is required during testing.

There are number different types of ring shear devices available (Sembenelli and Ramirez, 1969; LaGatta, 1970; Bishop et al., 1971; Bromhead, 1979; Sassa, 1992; Gibo et al.; 2002) so understanding the equipment and test procedure is important to obtaining meaningful data (Anderson and Hammoud, 1988) as with direct shear devices. As with the CD triaxial compression and CD direct shear tests, the torsional ring shear device also has limitations for measuring the FSS, such as:

- non-uniform shear displacements across the specimen with the inner portion of the specimen undergoing more displacement than that the outer portion at small shear displacements. This is important in over-consolidated or brittle materials or ring shear devices that have a ratio of the inner radius to the outer radius of the annular specimen less than 0.5 (Hvorslev, 1936). ASTM D7608 requires a ratio greater than 0.6 and a normally consolidated specimen which does not show a large post-peak strength loss.
- limited availability even though the device is available in commercial laboratories and some state and federal laboratories,
- small specimen size depending on the device used,

- soil extrusion if the specimen is shear at mid-height,
- side wall friction if the shear surface is not near the top of the specimen container or the device shears the specimen at mid-height, and
- in some devices the shear surface occurs near the top porous disc so the roughness of the top porous disc should be sufficient to ensure shearing in the soil not along the soil/disc interface (see ASTM D7608).

(b) Shear Displacement Rate/Test Procedure

In CD triaxial compression, direct shear, and ring tests, a shear displacement rate that corresponds to drained conditions must be used to measure the drained FSS. In other words, the shear displacement rate should be slow enough to allow the dissipation of pore-water pressures generated during shear. Guidelines for selecting a drained displacement rate are presented in the ASTM test methods for the triaxial compression device (ASTM D7181), direct shear device (ASTM D 3080), and ring shear device (ASTM D7608). Using higher shear displacement rates can/will result in partial dissipation of the shear-induced pore-water pressures, soil extrusion in the direct shear and ring shear devices, and can decrease the measured FSS depending on the level of soil extrusion. The shearing should continue to a shear displacement that is sufficient to confirm the maximum shear resistance, i.e., FSS, has been exceeded and the specimen shear resistance is decreasing.

(c) Specimen Disaggregation and Preparation

Different sample preparation techniques have been used to measure the FSS. The main difference in these techniques is the level of disaggregation that is achieved during the specimen preparation process, which impacts the measured FSS and associated index properties. In general, a greater degree of disaggregation results in a lower FSS and higher index properties (LaGatta 1970; Townsend and Banks 1974; Stark et al., 2005). The levels of disaggregation commonly used from least to greatest, i.e., highest to lowest FSS, are: soaking (ASTM D4318), mortar and pestle (ASTM D 4318), malt or milk shake mixer, kitchen blender or blenderizing, and ball-milling. The impact of disaggregation on the measured liquid limit (LL) and FSS is illustrated below:

$$LL_{\text{Soaking}} < LL_{\text{Mortar}} < LL_{\text{Mixer}} < LL_{\text{Blender}} < LL_{\text{Ball Milling}} \quad (3)$$

$$FSS_{\text{Soaking}} < FSS_{\text{Mortar}} < FSS_{\text{Mixer}} < FSS_{\text{Blender}} < FSS_{\text{Ball Milling}} \quad (4)$$

The difficulty for practicing engineers is determining the level of disaggregation that will occur during the slope or project service life. The level of disaggregation desired to simulate anticipated field conditions should be determined before starting testing so the specimen preparation procedure can achieve this level of disaggregation. For example, overconsolidated clays, mudstones claystones, and shales display high degrees of induration (Mesri and Cepeda-Diaz 1986). This induration involves diagenetic bonding between clay mineral particles by carbonates, silica, alumina, iron oxides, and other ionic complexes. The degree of induration (aggregation) that survives the field softening process must be selected and simulated in the laboratory sample preparation procedure so the measured values of FSS reflect this remaining level of induration. For example, if it is believed the FSS corresponds to a completely disaggregated material, the overconsolidated clay, claystone, mudstone, shale, etc. should be processed so the specimen material passes the No. 200 sieve to remove most of the bonding(Stark et al., 2005). As a result, Stark and Eid (1994) started ball milling heavily overconsolidated clays, mudstones. claystones, and shales because they possess substantial diagenetic bonding that is usually removed in the field, especially at residual strength situations. This level of disaggregation cannot be achieved using a typical mortar and pestle in the laboratory and passing the material through the No. 40 sieve as required by ASTM D4318 to measure Atterberg Limits.

Conversely, soil compacted into a levee or highway embankment is usually sufficiently disaggregated that it can be processed with a lesser degree of disaggregation than ball-milling, such as, a mortar and pestle, malt or milk shake mixer, or kitchen blender. A judgment decision needs to made by the practitioner on the level of disaggregation that will occur during the field softening process. At present, the industry trend is using a malt mixer or kitchen blender for levee slope design. Regardless, the practicing engineer and testing laboratory should clearly describe the method and level of disaggregation used so designers and regulatory personnel understand the magnitude of FSS being used for slope design.

Another important consideration in the selection of the level of disaggregation is the processing should not breakdown or reduce the quantity of silt and coarse grained particles, i.e., change the gradation, in the representative sample. For example, processing San Francisco Bay mud, which classifies as a silty clay, should involve soaking, mortar and pestle, or at most a milk shake mixer so the silt size particles are not reduced or broken down to a smaller size which could reduce the FSS.

After selecting the level of disaggregation, the specimen preparation process usually involves the following steps to obtain a reconstituted sample that can be used to measure Atterberg Limits (ASTM D4318), clay-size fraction (ASTM D422), and FSS under a normally consolidated condition using soaking or mortar and pestle to disaggregate the material:

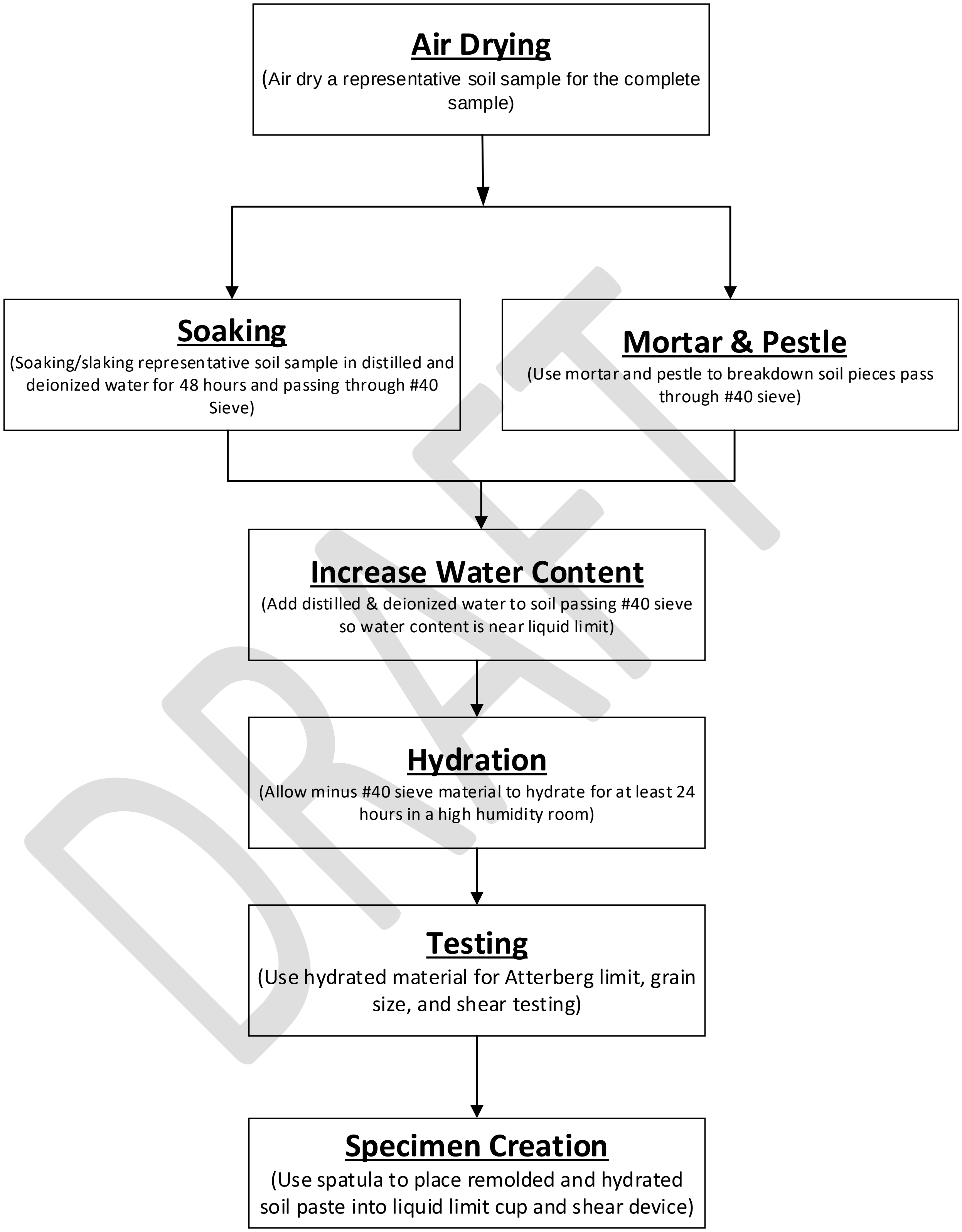


Figure 18. Flowchart depicting reconstituted specimen preparation process by soaking or mortar and pestle for FSS testing.

If a greater level of disaggregation is desired a malt mixer or blender is used. This sample preparation process usually involves the following steps to obtain a reconstituted sample that can be used to measured Atterberg Limits (ASTM D4318), clay-size fraction (ASTM D422), and sheared under a normally consolidated condition:

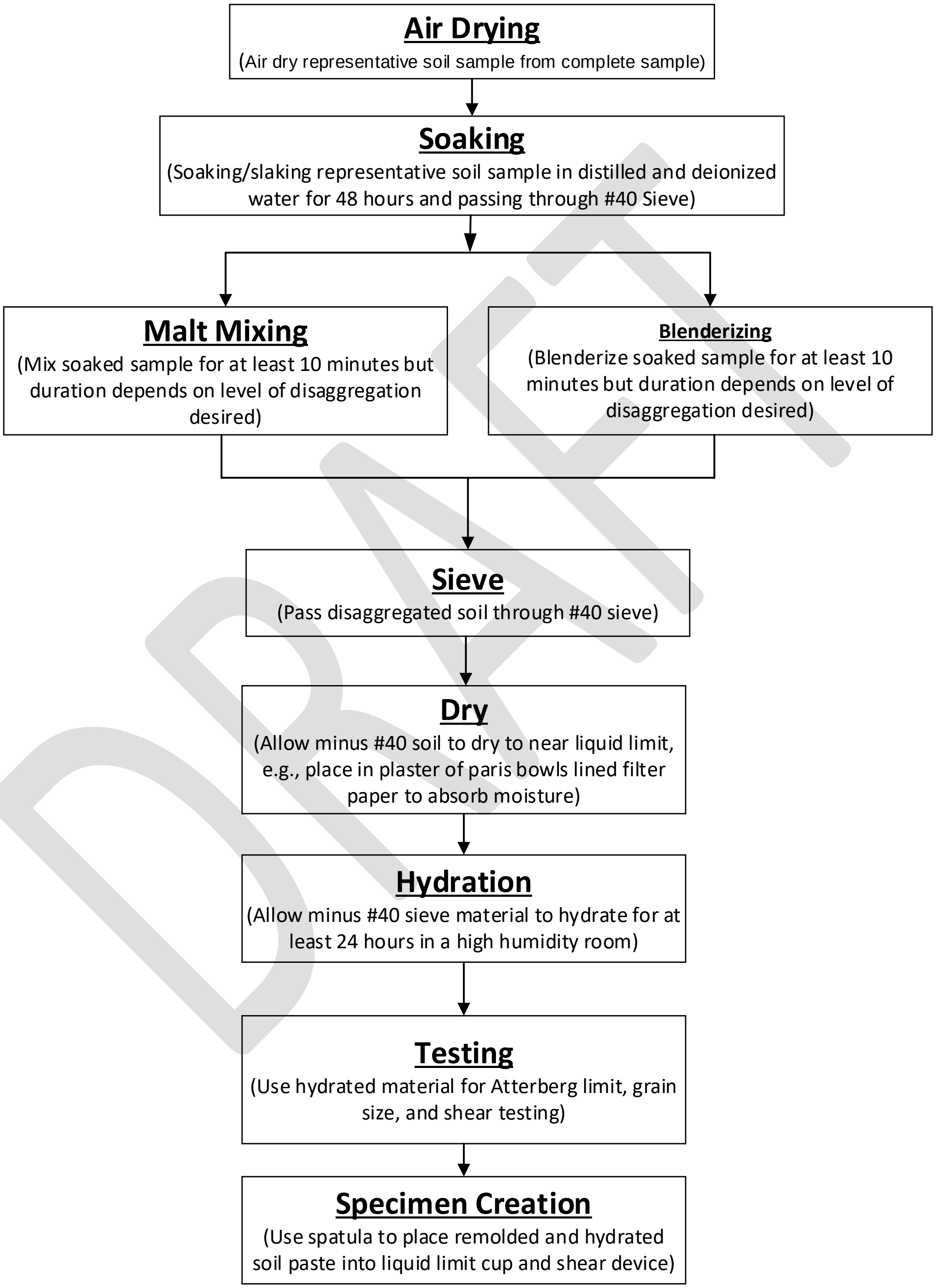


Figure 19. Flowchart depicting reconstituted specimen preparation process by malt mixing or blenderizing for FSS testing.

The practicing engineer and testing laboratory should clearly describe the sample preparation process used so designers and regulatory personnel understand the magnitude of FSS being used in the subsequent stability analyses. Finally regardless of the shear device, level of disaggregation, shear displacement rate, and specimen preparation technique used, the resulting strength envelope should be compared with at least one empirical correlation, e.g., Gamez and Stark (2014), Mesri and Shahien (2003), Wright (2005), to ensure the measured strengths are reasonable as discussed subsequently. For example, if the measured values of FSS envelope plots near a residual strength correlation, e.g., Stark and Hussain (2013), or significantly above an FSS correlation (Gamez and Stark, 2014), the test procedure should be carefully reviewed to understand the cause of the discrepancy. In addition, the results of limited tests should not be used to conclude one correlation or shear device is better than another because of differences and challenges in test procedures and interpretation.

(d) Specimen Water Content and Hydration

In all of the above disaggregation techniques, the soils are reconstituted using molding water contents ranging from below the liquid limit (Stark and Eid 1997; Stark et al. 2005) to a liquidity index of 1.5 (Gibson 1953; Bishop et al. 1965; Cancelli 1981; Bhattarai et al. 2006; Wright et al. 2007). The molding water content does not significantly impact the measured FSS because the specimen is normally consolidated prior to shear. However, the molding water content does impact the magnitude of consolidation settlement during the consolidation phase and thus whether or not there will be sufficient soil in the upper shear box in the direct shear device (see **Figure 17**). To facilitate the consolidation process, a molding water content below the liquid limit is recommended in ASTM D7608.

After reconstituting the specimen at the desired molding water content, it is recommended that it be allowed to hydrate in a high humidity room to ensure the fine-grained particles have sufficient time to absorb as much water as desired. ASTM D7608 requires a hydration time of at least 24 hours. Deionized and distilled water should be used for the hydration process unless the effect(s) of a different hydration fluid on the FSS is being studied such as a site specific fluid or a fluid with specific chemistry as described by Tiwari and Ajmera (2014).

(e) FSS at low effective stresses

First-time slope failures (Skempton, 1970) in compacted embankments (**Figure 10**) and excavated slopes in overconsolidated clay (**Figure 8**) tend to be shallow so the effective normal stress acting on the slip surface is low, i.e., less than 2,000 psf. In addition, Stark and Eid (1997) show the fully softened failure envelope is effective normal stress dependent or nonlinear, which has important application in slope stability analyses. For example, slides in levees and other embankment slopes (**Figure 10**) usually exhibit shallow slope surfaces with depths of less than 10 ft (3.1 m), which corresponds to an effective normal stress of less than 500 psf. Therefore, the shear strength at low effective normal stresses is needed for slope stability analyses because large differences in factor of safety can be calculated using an effective normal stress-dependent or nonlinear strength envelope versus a friction angle that corresponds to the average effective stress along the failure surface. For example, **Figure 20**, shows the FSS strength envelope and a friction angle for an average effective stress of 1,000 psf ($\phi'_{1,000\ psf}$). The $\phi'_{1,000\ psf}$) is lower than the nonlinear envelope at effective stresses less than 1,000 psf and greater at effective stresses greater than 1,000 psf, which can result in different factors of safety depending on the slip surface geometry.

These differences in calculated factor of safety are caused by not accurately modeling the significant stress-dependency of the strength envelope at low effective stresses and the location of the critical failure surface being influenced by the effective stresses acting on the failure surface. In general, the use of a linear strength envelope, instead of a stress-dependent strength envelope, overestimates the depth of the critical failure surface and factor of safety obtained in slope stability analysis (McCook, 1999; 2007; 2012, and Duncan et al., 2011). This is important because the curvature or nonlinearity is not only changing rapidly but is also greatest at effective stresses less than 2,000 psf as shown in **Figure 20**. This also means the fully softened secant friction angle decreases with increasing effective normal stress as shown by Stark and Eid (1997). This decrease in secant friction angle has been attributed to the tendency of the clay particles to a more face-to-face orientation with increasing effective normal confining pressure (Mesri and Cepeda-Diaz 1986; Kayyal and Wright 1991).

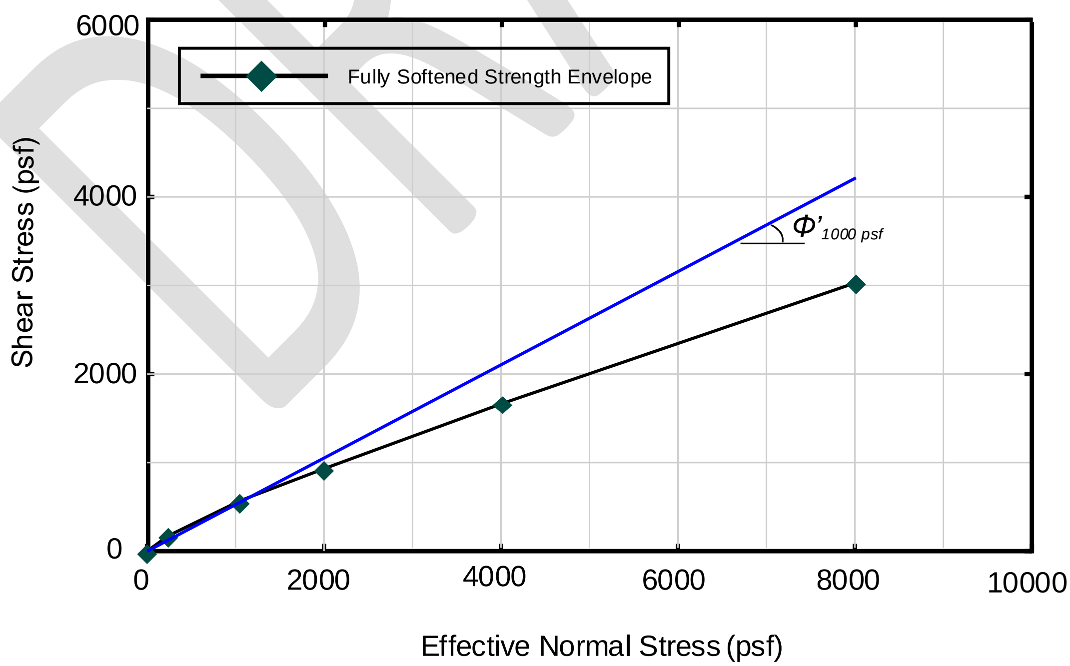


Figure 20. Diagram to illustrate comparison of nonlinear FSS strength envelope and an average friction angle.

Regardless of how the effective normal stress-dependent FSS envelope is modeled, a cohesion intercept should not be included in stability analyses because the drained peak strength of a normally consolidated specimen should pass through the origin and not exhibit a cohesion. This is important because the calculated factor of safety is can be greatly influenced by a cohesion parameter especially if a combination of effective stress cohesion (c') and friction angle (ϕ') are used as detailed by Stark et al. (2005).

(f) Representation of effective normal stress-dependent FSS envelope

The effective normal stress-dependent nature of the FSS envelope is now accepted so techniques for modeling the nonlinear or stress-dependent strength envelope in limit equilibrium and continuum analyses are being developed. This comports with other nonlinear drained strength envelopes, e.g., peak (De Mello, 1977; Charles and Watts, 1980; Mesri and Abdel-Ghaffar, 1993), and residual (Stark and Eid, 1994), being modeled using stress-depdendent strength envelopes instead of linear strength envelopes.

For example, Mesri and Abdel-Ghaffar (1993) present the following expression for the peak strength of overconsolidated clays and shales, which is modified below for the FSS:

$$\tau_p = \sigma_n^{'} \tan[\phi^{'}]\left[\frac{\sigma_p^{'}}{\sigma_n^{'}}\right]^{1-m} \tag{5}$$

where:

τ_p = peak shear strength on a plane with effective normal stress at failure $\sigma_n^{'}$

$\sigma_n^{'}$ = effective normal stress

$\sigma_p^{'}$ = preconsoidation pressure

$\phi^{'}$ = secant friction angle at failure $\sigma_n^{'}$

m = curvature of strength envelope.

Mesri and Shahien (2003) and Lade (2010) rewrote the expression in Equation (5), which is a power function, to model the shear strength in surficial slope failures. With a power function, the slope of the strength envelope is controlled by the parameter "*a*" while the stress-dependency of the strength envelope is controlled by the parameter "*b*" as suggested in Mesri and Shahien (2003). This power function is frequently cited for modeling the stress dependent FSS envelope because large differences in factor of safety can result from using a linear strength envelope (see **Figure 20**). Other functions, e.g., a hyperbolic function (Duncan and Chang, 1970), a secant friction angle in low plasticity soil where the curvature is not as pronounced (Stark and Hussain 2013), or a piece-wise linear envelope based on measured strengths or correlations, also could be used to model the FSS for stability analyses.

Rewriting the expression in Equation (5), the stress-dependent FSS strength envelope can be modeled using the following form as suggested by Mesri and Shahein (2003) and Lade (2010):

$$\tau_{FSS} = a \times P_a \times \left(\frac{\sigma'_n}{P_a}\right)^b \tag{6}$$

where a and b are dimensionless coefficients that control the slope and curvature of the strength envelope; σ'_n = effective normal stress; τ_{FSS} = fully softened shear strength; and P_a = atmospheric pressure in the same units as τ_{FSS} and σ'_n (Lade 2010). With a power function, the overall slope of the strength envelope is controlled by the parameter "*a*" while the stress-dependency of the strength envelope is controlled by the parameter "*b*" as suggested in Mesri and Shahien (2003).

Figure 2 presents values of a and b used to predict the FSS envelope for the three clay-size fraction (CF) groups (CF ≤ 20%, 25 ≤ CF ≤ 45%, and CF ≥ 50%) in the FSS correlation by Gamez and Stark (2014). The coefficients a and b can be used with Equation (6) to plot the stress-dependent FSS envelope using more than the four effective normal stresses used in the Gamez and Stark (2014) correlation, i.e., 12, 50, 100, and 400 kPa. While the value of a has a large range for the different CF groups, values of b have little influence on the power function because b ranges from only 0.839 to 0.969 for all of the CF groups. Thus the following average values of the b coefficient can be adopted for each of the CF groups:

- CF≤20%, b=0.96;
- 20%≤CF≤45%, b=0.905; and
- CF≥50%, b=0.852.

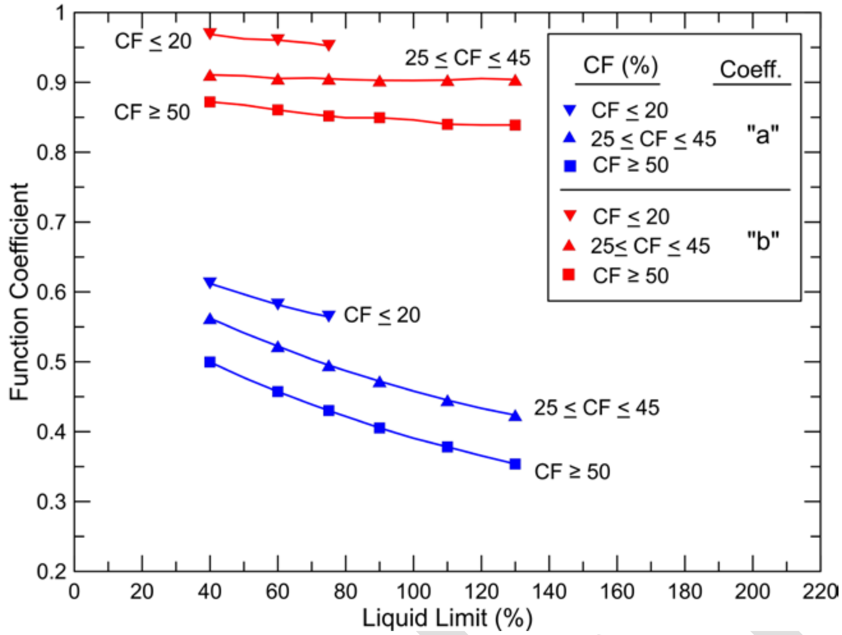


Figure 21. Recommended power function coefficients "a" and "b" to estimate FSS envelope for the three clay-size fraction (CF) groups in the FSS correlation by Gamez and Stark (2014).

(g) Summary

This section discusses techniques for measuring and representing the FSS in slope stability analyses. The objective of the shear testing and specimen preparation is to measure the peak strength of normally consolidated randomly oriented particles to represent the combined effects of weathering, stress relief, applied sehar stresses, infiltration, swelling, softening, and progressive failure (see **Figure 9**) in accordance with Skempton (1970 and 1977). Techniques for measuring the FSS is a topic of ongoing research so this section presents the current state-of-the-art and state-of-the-practice and references that can be pursued for future study.

In general, the following three laboratory shear devices are being used to measure the FSS: triaxial compression, direct shear, and torsional ring shear, and the advantages and limitations of each device are described in this section. In all of these shear devices, a shear displacement rate that corresponds to drained conditions must be used to measure the drained FSS. In addition, the importance of sample disaggregation, e.g., soaking, mortar and pestle, malt mixing, blenderizing, and ball milling, is discussed and should be selected to reflect the level of disaggregation expected in the field or desired in the slope design. Regardless of the shear device and level of disaggregation, the resulting FSS envelope should be compared with at least one empirical correlation, e.g., Gamez and Stark (2014), to ensure the measured strengths are reasonable.

The state-of-the-practice now recognizes the FSS envelope is effective normal stress dependent or nonlinear and should should be modeled in slope stability analyses instead of using a linear strength envelope. In either case, a cohesion intercept should not be included because the drained peak strength of a normally consolidated specimen should pass through the origin and does not exhibit a cohesion. As a result, different expressions are being developed to model the effective normal stress-dependent FSS envelope, such as a power function as recommended by Mesri and Shahien (2003) and Lade (2010) and shown in Equation (6).

6. FSS Empirical Correlations (Is there an easier way?)

This section discusses the use of FSS empirical correlations in cut slope and embankment design and slope stability analyses. The main uses of FSS empirical correlations, particularly the one developed by Stark et al. (2005), are verifying laboratory shear test results, planning level or conceptual stage analyses, and initial borrow site selection. Empirical correlations should not be used for final design unless site specific shear testing, discussed below, confirms the correlation is applicable to the soils present at the project site because no correlation has tested the complete range of soil types. As a result, the topics discussed in this section are: relevant empirical correlation parameters, estimating the parameters for use in the correlations, verification of laboratory shear test results, and calibrating or anchoring the correlation for final design with site specific testing.

(a) Relevant correlation parameters

Three main correlations have been published to estimate the FSS envelope based primarily on the data from Stark and Eid (1997). The first FSS correlation is described by Stark and Eid (1997), which has been augmented by Stark et al. (2005), Stark and Hussain (2013), and Gamez and Stark (2014). The other two correlations are presented by Mesri and Shahien (2003) and Wright (2005) and they use the data developed by Stark and Eid (1997). All three of these correlations were designed to estimate the effective normal stress-dependent FSS envelope identified by Stark and Eid (1997) using the liquid limit (LL), clay-size fraction (CF), and/or Plasticity Index (PI).

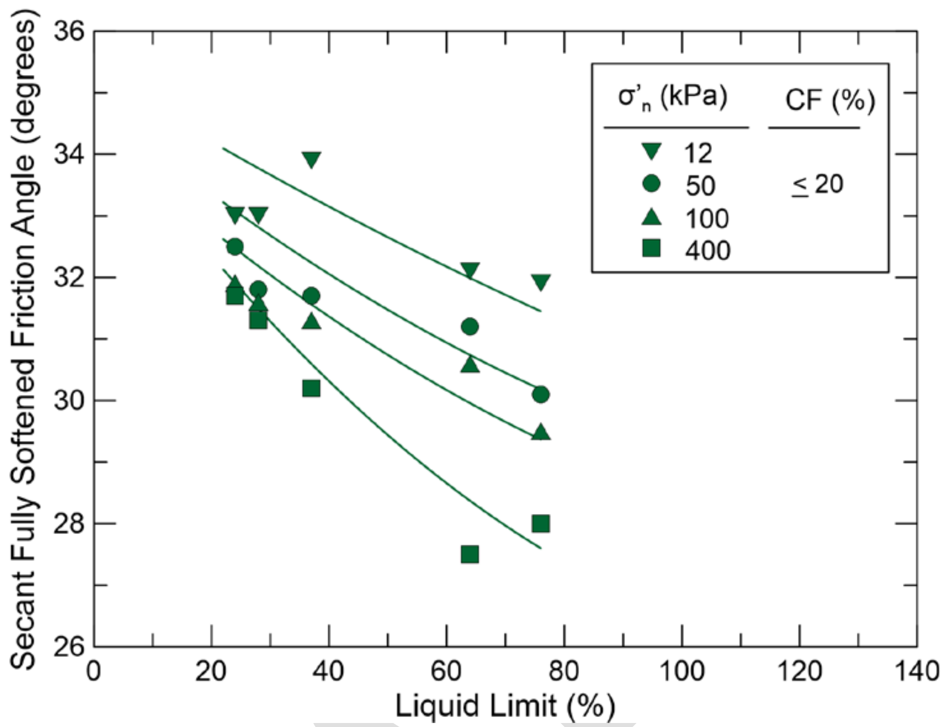
(b) Existing correlations

The liquid limit is used in the FSS correlations as an indicator of clay mineralogy and thus particle size. As the particle size decreases, the particle surface area increases, the LL increases, and the drained FSS decreases. However, CF remains an important predictive parameter of FSS because it indicates the quantity of clay mineralogy, i.e., soil particles smaller than 0.002 mm. **Figure 22** shows the resulting correlation from Stark and Eid (1997) augmented by Gamez and Stark (2014). Gamez and Stark (2014) augment the FSS correlations in Stark and Hussain (2013) to include an effective normal stress of 12 kPa (250 psf) to approximate the average effective normal stress along typical shallow semi-circular to planar slip surfaces in various embankments. The resulting strength envelope passing through effective normal stresses of 12, 50, 100, and 400 kPa is drawn passing through effective normal stresses of 0, 250, 1,044, 2,088, 8,354 psf and the origin because uncemented, normally consolidated fine-grained soil does not exhibit a cohesion intercept (Stark et al. 2005). The resulting FSS envelope can be used directly in a stability analysis or a power function can be used to increase the number of points used to describe the FSS envelope. Equations for the trend lines in **Figure 22** have also been developed (see Gamez and Stark, 2014) and have been incorporated in an EXCEL spreadsheet that is available at www.tstark.net to facilitate estimating an effective normal stress-dependent FSS strength envelope for use in stability analyses.

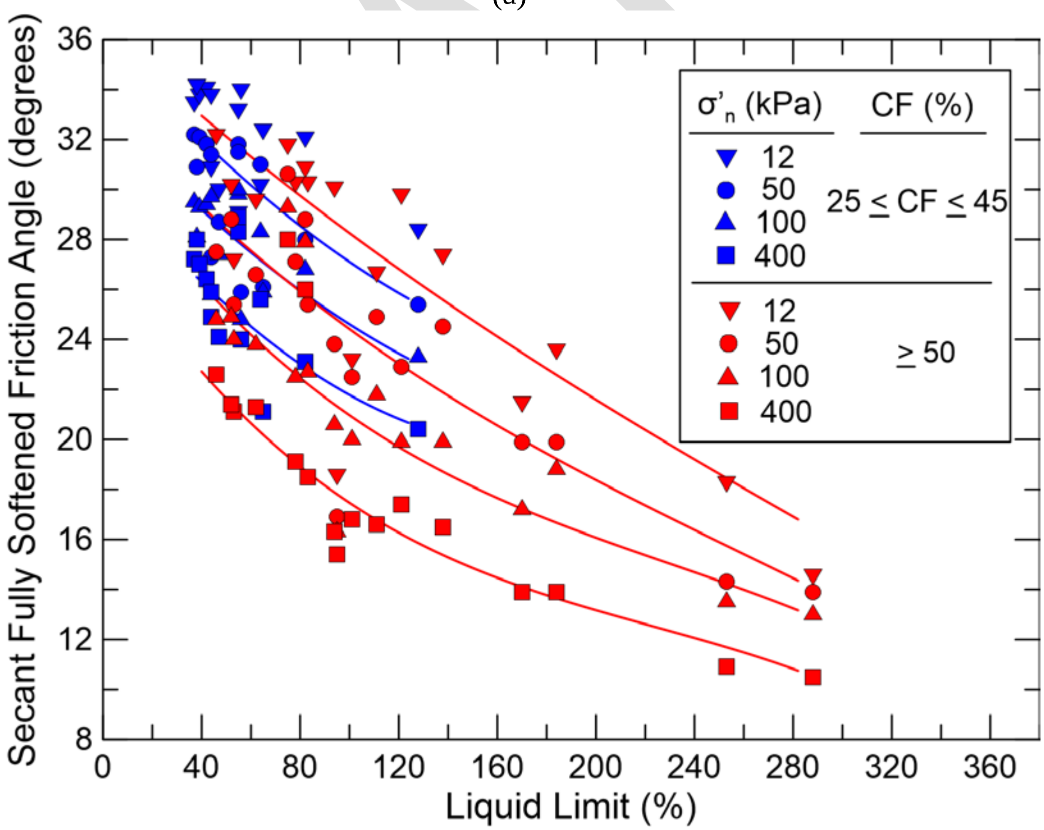
The correlation separates CF into three groups: less than or equal to 20%, greater than or equal to 25% and less than or equal to 45%, and greater than or equal to 50%. These three groupings are similar to those presented by Lupini et al. (1981) and Skempton (1985), which are less than or equal to 25%. between 25 and 50%, and greater than or equal to 50%. The three CF groupings were used by Lupini et al. (1981) to distinguish the boundaries between rolling shear. transitional shear, and sliding shear behaviors, respectively. The data in the correlation does not demonstrate a distinct change from rolling shear to transitional shear so there is a gap in the CF groupings between less than or equal to 20% and greater than or equal to 25% (see **Figure 22**). A distinct transition from transitional to sliding shear behavior also was not observed so there is a gap in the CF groupings between greater than or equal to 45% and greater than 50%. Interpolation can be used to estimate the secant fully softened friction angle between the three CF groups in **Figure 22** for a particular effective normal stress.

Because soils are anisotropic and the field stresses in FSS cases are anisotropic, the laboratory shear conditions should be understood and simulate field shear conditions. The four trend lines in each CF group in **Figure 22** represent the triaxial compression mode of shear not the torsional ring shear device mode of shear. In the beginning Stark and Eid (1997) decided the relevant mode of shear for first slides in cut slopes and embankments is triaxial compression. This is different than residual strength situations where the ring shear mode is relevant because the failure surfaces are usually planar (Stark and Eid, 1994).

Using the results of CD triaxial compression tests, not CU triaxial compression with pore-water pressure measurements, Stark and Eid (1997) introduced an adjustment factor of 2.5 degrees to convert the ring shear mode of shear to the CD triaxial compression mode of shear so the resulting FSS values better correspond to "first time slides" in cut slopes and compacted embankments. In particular, Stark and Eid (1997) used CD D triaxial compression tests at effective confining pressures of 2,088 and 4,178 of to create an average adjustment factor of 2.5 degrees to convert the ring shear mode of shear to the CD triaxial compression mode of shear and the peak strength of normally consolidated randomly oriented particles in accordance with Skempton (1970). Therefore, the fully softened friction angles presented in **Figure 22** have been increased by 2.5 degrees to represent the peak strength of normally consolidated randomly oriented particles in "first time slides" in cut slopes and embankments as described by Stark and Eid (1997) and subsequent papers.



(a)



(b)

Figure 22. Drained fully softened friction angle correlation for: (a) CF ≤ 20% and (b) CF > 20% from Gamez and Stark (2014).

(c) Anchoring correlations

This section discusses the “anchoring” or calibration of a FSS empirical correlation with site specific shear testing to ensure the trend lines used in the correlation are applicable to the soils present at the project site. Soils are highly variable and can even vary significant across a particular site such as a long (12 miles) levee system as shown in **Figure 10**. As a result, the likelihood that existing FSS empirical correlations are representative of all of the soils at a particular site is low because the database of soils used in a correlation is limited. Therefore, FSS empirical correlations should not be used for final design of cut slopes or embankments unless the correlation is anchored or calibrated to determine if it predicts values of FSS that are in agreement with site specific values. This means the test results plot within the scatter of the data used to create the trends lines in **Figure 22**.

If the project involves a large or long area, e.g., 12 miles of levee system as shown in **Figure 10**, that crosses different geologic units, the FSS empirical correlation should be anchored for each geologic unit, e.g., claystone or shale. Afterwards, the anchored or calibrated FSS correlation can be useful because sensitivity studies can be performed using a range of index parameters to investigate slope geometry and inclination, various borrow soures, and the impact of remedial measures.

Gamez and Stark (2014) recommend a minimum of one soil from each CF group be shear tested for the full range of normal stresses to ensure the trend lines produce reasonable effective normal stress-dependent FSS envelopes for other soils across the site. Of course, the number of soils tested in each CF should increase with increasing consequences of slope failure. For example, Stark et al. (2015) illustrate anchoring of the Stark et al. (2005) for the Dallas Floodway project, which serves as a good example for practitioners.

7. FSS in Stability Analyses (How do I apply it?)

This section discusses how the FSS is applied in static slope stability analyses and appropriate values of factors of safety. This includes location of the critical failure surface, appropriate values of factors of safety, applicable limit equilibrium methods, pore-water pressure modeling, and rapid drawdown scenarios. Use of the FSS in dynamic stability analyses is the topic of a future White Paper.

(a) Critical Slip Surface and Applicable Factors of Safety

For clayey embankments, e.g., levees and highway embankments, the critical semi-circular to planar failure surface should be located using the drained peak strength because this is the location of the most detrimental shear stresses in the slope. This is important because the cut slope/excavation or embankment starts with a peak strength that is reduced to a fully softened condition which may not have occurred over the entire length of the slip surface at the time of failure. As mentioned above, applied shear stresses facilitate the softening process and mobilization of a FSS condition. For design purposes, the critical failure surface should also be located using a stress-dependent FSS envelope and the lower factor of safety for these two shear surfaces should be used for design purposes. After locating these static critical failure surfaces, various shear strengths should be considered with corresponding factors of safety as discussed below.

Figure 7 shows a large difference can exist between drained fully softened and residual strengths with different soil plasticity. Where the difference between FSS and residual strength is significant, i.e., liquid limit greater than 50, the potential for progressive failure increases. **Figure 7** also shows shows the difference between FSS and residual strength is greater for shallow failure surfaces, i.e., lower effective normal stresses, than for deep failure surfaces. As a result, the following two factor of safety scenarios are presented for shallow failure surface in clay embankments:

- Use the FSS that represents the level of field disaggregation and full softening because this will likely occur over the service life of the structure, and meet or exceed a two-dimensional factor of safety of 1.5 or 1.4 for levees under U.S. Army Corps of Engineering Manual EM 1110-2-1902 – Slope Stability (2003). If the FSS is being used and the level of softening will be less than a normally consolidated specimen and the consequences of failure are small, lower values of factor of safety may be applicable. These factors of safety are based on a 2D analysis and should be modified if a three-dimensional (3D) stability analysis is used (Akhtar and Stark, 2014).

- Use the residual strength for materials that will undergo softening and possibly shear displacement due to applied shear stresses and meet or exceed a 2D factor of safety above unity (1.0) if a true residual strength is measured using a torsional ring shear device. If a reversal direct shear large displacement strength is used, the factor of safety should meet or exceed a 2D factor of safety of 1.1. If a three-dimensional stability analysis is used, higher factors of safety should be satisfied as outlined in Akhtar and Stark (2014).

(b) Representative pore-water pressures

Pore-water pressures are always an important input category for effective stress slope stability analyses but especially for a FSS analysis because the relevant failure surfaces are usually shallow, i.e., at low total stresses. This is important because at low total stresses, small increases in pore-water pressure can cause significant decreases in effective stresses and thus FSS. The following are some recommendations for pore-water pressures for use in planning level FSS stability analyses in stiff-fissured and compacted soils:

- For excavations and slopes in stiff-fissured clays, a pore pressure ratio, i.e., r_u = pore-water pressure (u) divided by total stress (unit weight*depth), of 0.25 to 0.35 is in agreement with inverse analyses of first-time slides in stiff-fissured clays (James, 1970). James (1970) recommends the lower bound (0.25) for steep slopes and the upper bound (0.35) for flatter slopes with an average value being 0.30. Vaughan and Walbancke (1973), Chandler (1972 and 1974), and Skempton (1977) also use first-time slide case histories to show r_u = varies from 0.2 to 0.30 after 30 years in London Clay slopes. For long-term conditions, pore water pressures corresponding to steady state seepage should be used.

- For compacted fine-grained soil embankments, r_u of 0.4 to 0.6 is in agreement with inverse analyses of first-time slides in compacted embankments by Day and Axten (1989), Lade (2010), and Kayyal and Wright (1991). These values are significantly greater than the values above for stiff-fissured clays and reflect the importance of rainfall on shallow failure surface. These values of r_u correspond to steady state seepage parallel to the slope face for typical embankment inclinations, i.e., 2H:1V to 5H:1V.

Values of r_u can be input directly into most slope stability software or can be used to estimate a phreatic surface that can be input in stability software to perform FSS stability analyses involving in stiff-fissured and compacted soils.

(c) Limit Equilibrium Stability Methods

This section briefly discusses limit equilibrium stability methods that can be used to estimate the factor of safety (FS) that corresponds to the shear strength slope geometry, failure surface, and pore-water pressure condition developed for a FSS stability method using the recommendations above. Duncan et al. (2015) present a much more detailed treatment of limit equilibrium stability methods that are used to estimate the factor of safety, which is defined as the shear strength divided by the shear stress required for equilibrium, i.e., required to close the force polygon.

Each stability method contains a number of assumptions required to make the analysis statically determinate. The following assumptions are used in most, if not all, stability methods:

- FS is constant along the failure surface,
- Strength is fully mobilized along the failure surface assigned that strength, and
- Normal force is located at the base and center of each vertical slice.

In general, it is preferred that the selected stability method satisfy all conditions of static equilibrium because they are better able to accommodate complex slope and failure surface geometries, pseudo-static forces, and pore-water conditions. Spencer's (1967) method is a simplification of the Morgenstern and Price (1965) method, which has made it popular because of its ease and ability to be programmed. However, most slope stability software now accommodates the Morgenstern and Price (1965) method so this is the preferred stability method for the reasons below. This method assumes the shear forces between adjacent vertical slices is related to the normal force at the bottom of the slice, which is beneficial. Available software packages allow the user to selected different functions to represent the shear forces along the failure surface so the sensitivity of the FS to interslice force inclination can be assessed whereas Spencer's (1967) method assumes the resultant interslice forces are parallel, i.e., inclined at a constant angle, along the entire failure surface. This simplifying assumption is not correct but yields reasonable values of FS (Duncan et al., 2015). However, given most slope stability software includes the Morgenstern and Price (1965) method, this method is more flexible, and is useful in cases where interslice forces might have a significant effect on the FS (Duncan et al., 2015), it is recommended for FSS stability analyses.

(d) Time rate of softening in reliability slope stability analyses

Following construction, but prior to reaching a fully-softened condition, cut and fill slopes comprised of fine-grained soils prone to softening are expected to have drained strengths that fall somewhere between the peak strength and the normally-consolidated strength. The use of partially-softened strengths in stability analyses may be warranted when it is desired to estimate the probability of failure over time. The time rate of softening is influenced by many factors that are difficult to model including: climate, soil-pore water interaction, erosion, formation and deepening of cracks from desiccation and freezing, and the availability of surface water and groundwater. Furthermore, softening is not likely to occur at the same rate everywhere within the slope. To overcome the difficulty in modeling the spatial distribution of softening, a practical approach is to estimate the overall impact of softening on strengths along the entire length of a potential sliding surface. The softening model proposed by McGuire and Sleep (2015), shown in Equation (7), assumes that strength along a potential sliding surface decays exponentially. The parameter, p_{fs}, equals the overall degree of softening that has occurred along the entire length of a potential slip surface and k is a rate coefficient. A slip surface with a p_{fs} equal to zero has experienced no softening and has strengths along its length defined by stress-dependent peak strengths. A slip surface with a p_{fs} equal to unity has experienced full softening and has strengths along its length defined by stress-dependent, fully-softened strengths.

Because the clay is exposed to environmental conditions that cause weathering, and thus softening, from the time it is placed or exposed, it seems reasonable to measure time relative to the end of construction. If $p_{fs,r}$ and t_r equal reference values of softening and time, respectively, the rate coefficient can be estimated using Equation (8). Information compiled from first-time slope failures, e.g., Skempton, 1970, can be used to make reasonable estimates of $p_{fs,r}$ and t_r. For example, Henkel (1957) studied strength loss over time for London Clay in terms of the decrease in effective cohesion, c', from peak strength (c'=250 psf, $p_{fs} = 0$) to what we now refer to as the fully-softened condition (c'=0, $p_{fs} = 1$). The solid black line in **Figure 23** can be closely represented by Equation (7) using a rate coefficient equal to -0.182 years^{-1} because there is a 72 percent loss of cohesion over 70 years (i.e. $P_{fs,r} = 0.72$, $t_r = 70$ years). It is possible to determine reliable rate coefficients for certain regional soils and construction methods by reviewing available data and collecting new data when slope failures occur.

$$p_{fs} = 1 - e^{kt} \tag{7}$$

$$k = \frac{\ln\left(1 - p_{fs,r}\right)}{t_r} \tag{8}$$

The single value of p_{fs} estimated using the proposed softening model for the time of interest is applied in Equation (9) to determine the most likely estimate of partially-softened strength at a particular normal effective stress, $\tau_{f,ps}$ from the most likely estimates of peak strength, $\tau_{f,p}$, and fully-softened strength, $\tau_{f,fs}$.

$$\tau_{f,ps} = \left(p_{fs}\right)\tau_{f,fs} + \left(1 - p_{fs}\right)\tau_{f,p} \tag{9}$$

The coefficient of variation (COV) of the partially-softened strength, $COV_{\tau,ps}$, can be estimated from the COV of the peak strength, $COV_{\tau,p}$, and the COV of the fully-softened strength, $COV_{\tau,fs}$, using Equation (10). For $0 < p_{fs} < 1$, the value of $COV_{\tau,ps}$ is higher than

$COV_{\tau,p}$ and $COV_{\tau,fs}$ due to the difference between $\tau_{f,p}$ and $\tau_{f,fs}$. McGuire and Sleep (2015) describe a method for estimating values of $COV_{\tau,fs}$ and $COV_{\tau,fs}$ using the results from laboratory shear strength tests.

$$\mathrm{COV}_{\tau,ps} = \frac{1}{\tau_{f,ps}}\sqrt{(p_{fs})[A]+(1-p_{fs})[B]}$$
$$A = \left(\mathrm{COV}_{\tau,fs}\cdot\tau_{f,fs}\right)^2 + \left(\tau_{f,fs}-\tau_{f,ps}\right)^2 \tag{10}$$
$$B = \left(\mathrm{COV}_{\tau,p}\cdot\tau_{f,p}\right)^2 + \left(\tau_{f,p}-\tau_{f,ps}\right)^2$$

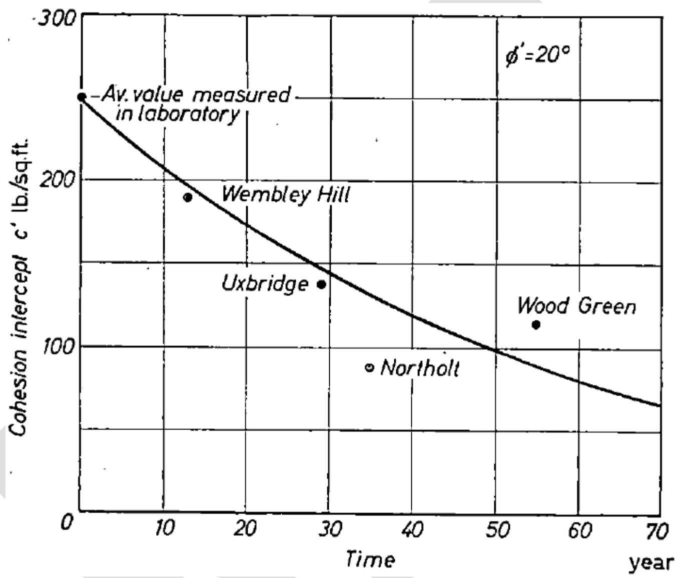


Figure 23. Degradation of soil strength with time for cut slope failures in London Clay from Henkel, (1957).

8. FSS Mitigation Measures (How do I limit FSS development?)

This section briefly presents some of the situations where a FSS condition may not develop and remedial measures the can be implemented to delay or prevent development of a FSS condition and thus increase the stability of clay embankment slopes.

The FSS may not be applicable is some situations, such as the long-term stability excavations or cuts in overconsolidated clay because the softening processes discussed above may not have sufficient time to occur. If the slope or excavation will be temporary, a strength greater than FSS may be suitable because there is not sufficient time to fully soften the material using the steps in the flowchart in **Figure 9**. Another situation where the FSS may not develop is in unsaturated material if it will not have access to water, which could occur if the slope is covered.

Saleh and Wright (1997) nicely tabulate a range possible remedial measures for limiting development of a FSS condition and improving embankment stability. Some of these measures are better suited as a preventive measure than a remedial measure. For brevity, the remedial measures are separated into three broad topic areas: decrease applied shear stress, limit infiltration, and soil stabilization. A few possible remedial measures are presented under each topic area below and interested readers should review Saleh and Wright (1997):

i. Decrease shear stress
   - flatten or reduce slope inclination,
   - install toe buttress,
   - use lightweight fill, and/or
   - reduce stress relief using retaining wall, excavation bracing, anchors, drilled shafts, etc.

ii. Limit infiltration
   - cover slope – vegetation, concrete (drains)
   - decrease mowing because it reduces vegetation and can create depressions that facilitate infiltration,

iii. Soil Stabilization
   - mixing lime into slope material to decrease plasticity
   - adding cement to increase soil shear strength.

9. Summary and Recommendations (DRAFT)

The purpose of this White Paper is to explain: fully softened strength (FSS), why geotechnical engineers should understand FSS, how FSS develops with time, slopes susceptible to developing a FSS condition, how the FSS can be measured, use of the FSS in stability analyses, how the FSS differs from drained peak and residual strengths, and measures that can be implemented to limit FSS development. The following is a summary of these topics and the main recommendations presented herein:

- Cuts or excavations in overconsolidated fine-grained soils exhibit semi-circular to planar slope failures with a depth from 4 to about 65 feet with a minimum depth of 20 to 25 feet (6.1 to 7.6 m) for design purposes.
- Compacted fine-grained soil embankments usually experience shallow semi-circular to planar slope failures with a depth less than about 10 ft (3.1 m) or about 20 percent of the slope height,
- Fully softened strength is effective normal stress-dependent at low effective stresses, i.e., shallow depths, and must be modeled in stability analyses because the factor of safety and location of the critical failure surface is influenced by the applied effective stresses and shear strength.
- A cohesion intercept or strength parameter also should not be used to model the FSS because it can dramatically increase the factor of safety and does not correspond to a normally consolidated specimen.
- Consolidated-drained triaxial compression, direct shear, and torsional ring shear can be used to measure the FSS using normally consolidated specimens. However, the sample preparation and disaggregation, test procedure, and results must be extensively documented to understand the applicability of the measured strength values. Regardless of the shear device, test procedure, and disaggregation used, the resulting strength envelope should be compared with at least one empirical correlation, e.g., Stark and Eid (1997), Mesri and Shahien (2003), or Wright (2005), to ensure the measured FSS envelope is reasonable.
- FSS empirical correlations should be used to verify laboratory shear test data and can be used for planning level design. Empirical correlations should not be used for final design unless the correlation is "anchored" or calibrated with site specific testing as described herein.
- Because of the potential for progressive failure, two stability scenarios should be considered: (1) use the FSS and exceed a two-dimensional factor of safety of for cut slopes and embankments 1.5 or 1.4 for levees and (2) use the residual strength and exceed a two-dimensional factor of safety of unity.

10. Cited References

Abrams, T. G., and Wright, S. G. (1972). *A survey of earth slope failures and remedial measures in Texas*. Center for Transportation Research, University of Texas at Austin, 109.

Akhtar, K. and Stark, T.D., "Importance of Side Resistance in 3D Stability Analysis," *Proceedings of 12th International Conference on Geo-Disaster Reduction-ICGdR*, Fullerton, CA, September, 2014, pp. 110-118.

Anayi, J. T., Boyce. J. R., and Rodgers. C. D. F. 1988, "Comparisons of Alternative Methods of Measuring the Residual Strength of a Clay." *Transportation Research Record* 1192, Transportation Research Board, Washington, DC. pp. 16-26.

Anayi. J. T., Boyce. J. R., and Rodgers. C. D. F., 1989, "Modified Ring Shear Apparatus," ASTM *Geotechnical Testing Journal.* Vol. 12, No.2, pp. 171-173.

Anderson. W. F. and Hammoud. F. 1988, "Effect of Testing Procedure in Ring Shear Tests." ASTM *Geotechnical Testing Journal.* Vol. 11. No.3, pp. 204-207.

Anson, R.W.W. and Hawkins, A.B.. (1998). "The effect of calcium ions in pore water on the residual strength of kaolinite and sodium montmorillonite." *Geotechnique*, v. 48(6):787-800.

American Society for Testing and Materials (ASTM). (2008). "Standard test method for particle-size analysis of soils." (*D 422) 2008 annual book of ASTM standards*, Vol. 04.08, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). (2011). "Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions." (*D3080) 2008 annual book of ASTM standards*, Vol. 04.08, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). (2008). "Standard test method for liquid limit, plastic limit, and plasticity index of soils." *(D 4318) 2008 annual book of ASTM standards*, Vol. 04.08, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). (2008). "Standard test method for torsional ring shear test to determine drained fully softened shear strength and nonlinear strength envelope of cohesive soils (using normally consolidated specimen) for slopes with no pre-esixting shear surfaces." (*D7608) 2008 annual book of ASTM standards*, Vol. 04.08, West Conshohocken, PA.

American Society for Testing and Materials (ASTM). (2011). “Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.” (*D7181) 2008 annual book of ASTM standards,* Vol. 04.08, West Conshohocken, PA.

Atkinson, J. H., and Farrar, D. M. (1985). “Stress path tests to measure soil strength parameters for shallow landslips.” *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, 4, 983–986.

Bhattarai, P., Marui, H., Tiwari, B., Watanabe, N., and Tuladhar, G. R. (2006). “Influence of weathering on physical and mechanical properties of mudstone.” *Proceedings of the International Symposium on Disaster Mitigation of Debris Flows, Slope Failures and Landslides*, 467–479.

Bishop, A. W., Webb, D. L., and Lewin, P. I. (1965). “Undisturbed samples of London Clay from the Ashford Common Shaft: Strength–effective stress relationships.” *Géotechnique*, 15(1), 1–31.

Bishop, A. W., Green. G. E., Garaga, V. K., Andresen, A., and Brown. J. D. (1971). "A New Ring Shear Apparatus and its Application to the Measurement of Residual Strength." *Geotechnique,* Vol. 21. No.4. W·273-328.

Branch, A., Stephens, I., Olsen, R., and Pearson, M. (2011). Lecture slides titled “Caution on Use of Non Site-Specific Fully Softened Shear Strength Correlations,” presented at U.S. Army Corps of Engineers Infrastructure Systems Conference (ISC), Atlanta, GA, 41 pages.

Bromhead, E. N. (1979). “A simple ring shear apparatus.” *Ground Engineering*, 12(5), 40–44.

Bromhead, E. N. and Curtis. R. D. 1983, "A Comparison of Alternative Methods of Measuring the Residual Strength of London Clay," *Ground Engineering.* Vol. 16. pp. 39-41.

CAGE (Colorado Association of Geotechnical Engineers), (1999). “Commentary on Geotechnical Practices: Drilled Pier Design Criteria for Lightly Loaded Structures in the Denver Metropolitan Area,” 30 p.

Cancelli, A. (1981). “Evolution of slopes in over-consolidated clays.” *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, 3, 377–380.

Castellanos, B.A., Brandon, T.L., VandenBerge, D.R. (2015). “Use of fully softened shear strength in slope stability analysis.” *Journal of the International Consortium on Landslides*, Vol. 12, No. 3, pp. 1-13.

Castellanos, B., Brandon, T. L., Stephens, I., and Walshire, L. (2013). “Measurement of fully softened shear strength.” *Proceedings of Geo-Congress 2013: Stability and Performance of Slopes and Embankments III*, 234–244.

Chandler, R. J. (1972). “Lias clay: Weathering process and their effect on shear strength.” *Géotechnique*, 22(4), 403–431.

Chandler, R. J. (1974). “Lias clay: The long-term stability of cutting slopes.” *Géotechnique*, 24(1), 21–38.

Charles, J. A., and Watts, K. S. (1980). “The influence of confining pressure on the shear strength of compacted rockfill.” *Géotechnique*, 30(4), 353–367.

Contreras, I. (2003). “Investigation and Stabilization of a Landslide on Pierre Shale.” *Proc. 51st Annual Geotechnical Enginering Conf., University of Minnesota, Minneapolis, MN*, 1-21.

Crabb, G. I., and Atkinson, J. H. (1991). “Determination of soil strength parameters for the analysis of highway slope failures.” *Proceeding of the International Conference on Slope Stability Engineering*, 13–18.

Day, R. W., and Axten, G. W. (1989). “Surficial stability of compacted clay slopes.” *Journal of Geotechnical Engineering*, ASCE, 115(4), 577–580.

De Mello, V. F. B. (1946). “Laboratory Investigation of Shearing Resistance of Clays.” Thesis presented to the Massachusets Institute of Technology in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 262.

De Mello, V. F. B. (1977). “Reflections on design decisions of practical significance to embankment dams.” *Géotechnique*, 27(3), 281–355.

Di Maio, C. (1996). “Exposure of bentonite to sail solution: Osmotic and mechanical effects.” *Géotechnique*, 46(4), 695-707.

Di Maio, C. and Fenilli, G.B.. (1994). “Residual strength of kaolin and bentonite: the influence of their constituent pore fluid.” *Géotechnique*, 44(4):217-226.

Di Maio, C. and Oronati, R. (2000). “Influence of pore liquid composition on the shear strength of active clay.” In: Bromhead, E., N. Dixon and M.L. Ibsen (eds), *Proceedings of the 8th International Symposium on Landslides*, Cardiff, 1: 463-468.

Duncan, J.M., Wright, S.G., and Brandon, T.L., *Soil Strength and Slope Stability*, 2nd Ed., John Wiley and Sons, 2015, 317 p.

Duncan, J. M., Brandon, T. L., and VandenBerge, D. R. (2011). *Report of the workshop on shear strength for stability of slopes in highly plastic clays*. Center for Geotechnical Practice and Research, Blacksburg, VA, 79 p.

Duncan, J.M. and Wright, S.G. (2005). *Soil Strength and Slope Stability,* 1st Ed., John Wiley and Sons, 2015, 297 p.

Duncan, J.M., Wright, S.G., and Brandon, T.L. (2015). *Soil Strength and Slope Stability,* 2nd Ed., John Wiley and Sons, 2015, 317 p.

Duncan, J.M. and Chang, C.Y. (1970). “Nonlinear Analysis of Stress and Strain in Soils.” *ASCE Journal of the Soil Mechanics and Foundations Division,* September. 1970, 96(SM5), pp. 1629-1653.

Duncan, J. M., Brandon, T. L., and VandenBerge, D. R. (2011). *Report of the workshop on shear strength for stability of slopes in highly plastic clays*. Center for Geotechnical Practice and Research, Blacksburg, 79.

Gamez, J. and Stark, T.D. (2014). “Fully Softened Shear Strength at Low Stresses for Levee and Embankment Design.” *ASCE Journal of Geotechnical and Geoenvironmental Engineering, June.* 2014, pp. 06014010-1-06014010-6.

Gibson, R. E. (1953). “Experimental determination of the true cohesion and true angle of internal friction in clays.” *Proceedings of the 3rd International Conference in Soil Mechanics*, 1, 126–130.

Gibo, S., Egashira, K., Ohtsubo, M., and Nakamura, S. (2002). “Strength recovery from residual state in reactivated landslides.” *Geotechnique*, 52(9), 683–686.

Gourlay, A. W., and Wright, S. G. (1984). *Initial laboratory study of the shear strength properties of compacted, highly plastic clays used for highway embankment construction in the area of Houston, Texas*. Center for Transportation Research, University of Texas at Austin, 224.

Green, R., and Wright, S. G. (1986). *Factors affecting the long term strength of compacted Beaumont Clay*. Center for Transportation Research, University of Texas at Austin, 222.

Henkel, D.J. (1957). “Investigations of two long-term failures in London Clay slopes at Wood Green and Northolt.” Proceedings of the fourth International Conference on Soil Mechanics and Foundation Engineering, London, Vol. 2, pp. 315-320.

Hvorslev, M. J. (1936). “A ring shear apparatus for the determination of the shearing resistance and plastic flow of soils.” *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering*, 2, 125–129.

Hvorslev, M. J. (1939). “Torsion shear tests and their place in the determination of the shearing resistance of soils.” *Proceedings of the American Society Testing Material*, 39, 999–1022.

Hvorslev, M. J. (1960). “Physical components of the shear strength of saturated clays.” *Research Conference on Shear Strength of Cohesive Soils,* Boulder, Colorado, 169–273.

Hvorslev, M. J. (1969). *Physical Properties of Remolded Cohesive Soils*. *Translation 69-5*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, 165.

James, P. M. (1970). “Time Effects and Progressive Failure in Clay Slopes.” Thesis presented to University of London in partial fulfillment of the requirements for the degree of Doctor of Philosophy, 210 pp.

Kayyal, M.K. and Wright, S.G. (1991). “Investigation of Long-Term Strength Properties of Paris and Beaumont Clays in Earth Embankments,” Research Project No. CTR 3-8-89/1-1195-2F, Center for Transportation Research, University of Texas, Austin, TX, Research Report 1195-2F, 123 pp.

Kovacevic, N., Potts, D. M., and Vaughan, P. R. (2001). “Progressive failure in clay embankments due to seasonal climate changes.” *Proc. 15th Int. Conf. Soil Mech. Geotech. Eng.*, 3, 2127–2130.

Lade, P. V. (2010). “The mechanics of surficial failure in soil slopes.” *J. Eng. Geo.*, ASCE, 114(1-2), 57–64.

La Gatta, D. P., 1970, "Residual Strength of Clays and Clay-Shales by Rotation Shear Tests," Ph.D. thesis reprinted as *Harvard Soil Mechanics Series* No. 86, Harvard University. Cambridge. MA, p. 204.

Lupini, J. F., Skinner, A. E.. and Vaughan, P. R. (1981). "The drained residual strength of cohesive soils." *Geotechnique*, 31(2), 181-213.

Maccarini, M. (1993). “A comparison of direct shear box tests with triaxial compression tests for a residual soil.” *Geotechnical and Geological Engineering*, 11(2), 69–80.

Maksimovic, M. (1989). “Nonlinear failure envelope for soils.” *Journal of Geotechnical Engineering*, 115(4), 581–586.

McCook, D. (1999). “Surficial Slides on Highly Plastic Clay Embankments,” ASDSO Annual Meeting, St. Louis, MO. August, 1999. Voted one of best of conference papers.

McCook, D. (2007). Lecture on Slope Stability presented at FEMA Seminar “Pitfalls with Embankments”, Emmitsburg, MD, February, 2007.

McCook, D. (2012). “Modeling for Surficial Slope Failures,” Paper accepted for presentation at 2012 USSD Annual Meeting.

McCook, D. K. (2012). “Discussion of Modeling for Analyses of Fully Softened Levees.” *Innov. Dam and Levee Design and Const. for Sustain. Water Mngmt.*, New Orleans, 483–523.

McGuire, M.P. and Sleep, M.D. (2015). “Stability of embankment slopes with time-dependent strength.” *28th Central Pennsylvania Geotechnical Conference*, Hershey, Pennsylvania.

Mesri, G., and Cepeda-Diaz, F. (1986). “Residual shear strength of clays and shales.” *Géotechnique*, 36(2), 269–274.

Mesri, G., and Abdel-Ghaffar, M. E. M. (1993). “Cohesion intercept in effective stress-stability analysis.” *Journal of Geotechnical Engineering*, ASCE, 119(8), 1229–1249.

Mesri, G., and Shahien, M. (2003). “Residual shear strength mobilized in first-time slope failures.” *J. Geotech. Geoenviron. Eng.*, 129(1), 12–31.

Moon, A. T. (1984). “Effective shear strength parameters for stiff fissured clays.” *4th Australia-New Zealand Conference on Geomechanics*, 107–111.

Moore, R. (1991). “The chemical and mineralogic controls upon the residual strength of pure and natural clays.” *Géotechnique*, 41(1):35-47.

Morgenstern, N.R. and Price, V. E. (1965). “The analysis of the stability of of general slip surfaces.” *Geotechnique*, 15(1), 79-93.

Noor, M. J. M., and Anderson, W. F. (2006). “A comprehensive shear strength model for saturated and unsaturated soils.” *Unsaturated Soils 2006 (GSP 147)*, ASCE, 1993–2003.

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*. Center for Transportation Research, University of Texas at Austin, 146.

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*. Center for Transportation Research, U. of Texas at Austin, 156 p.

Sassa, K. (1992). “Access to the dynamics of landslides during earthquakes by a new cyclic loading ring shear apparatus,” Proceedings of the 6th International Symposium on Landslides, 3, Balkema, 1919-1937, 1992.

Schaefer, V. R. and Lohnes, R. A. 2001. “Landslide failure mechanisms in Pierre shale, South Dakota, U.S.A.” *Proceedings of the International Conference on Landslides: Causes, Impacts and Countermeasures*, Davos, Switzerland, pp. Verlag Glückauf Essen, pp. 87-96.

Sembenelli, P. and Ramirez, AL. “Measurement of residual strength of clay with a rotating shear machine,” *Proceedings of the 7th International Conference on Soil Mechanics*, Mexico, 3, 528-529, 1969.

Skempton, A. (1954). “The pore-pressure coefficients A and B.” *Geotechnique,* 4, 143-147.

Skempton, A.W. (1964). "Long-Term Stability of Clay Slopes.” *Geotechnique*, 14(2), 77-102.

Skempton, A. W. (1970). “First-time slides in over-consolidated clays.” *Geotechnique*, 20(3), 320–324.

Skempton, A. W. (1977). “Slope stability of cuttings in Brown London Clay.” *Proc. 9th Int. Conf. Soil Mech. Found. Eng.*, 3, 261–270.

Skempton, A.W. (1985), "Residual Strength of Clays in Landslides, Folded Strata, and the Laboratory," *Geotechnique*, Vol. 35, No. 1, pp. 3-18.

Skempton, A.W. and Petley, D.J. (1967), "The Strength of Structural Discontinuities in Stiff Clay," *Proceedings of the Geotechnical Conference*, Oslo, Vol. 2, pp. 29-46.

Spencer, E. (1967). “A method of analysis of the stability of embankments assuming parallel interslice forces.” *Geotechnique*, 17(1), 11-26.

Stark, T. D., Choi, H., and McCone, S. (2005). “Drained shear strength parameters for analysis of landslides.” *Journal of Geotechnical and Geoenvironmental Engineering*, 131(5), 575–588.

Stark, T. D. (1987). Mechanism of strength loss in stiff clays. Ph.D. dissertation. Virginia Polytechnic Institute and State University, Blacksburg, VA

Stark, T. D. and Duncan, J. M. (1991). Mechanism of strength loss in stiff clays. *Journal of Geotechnical Engineering*, ASCE, 117(1): 139 – 154.

Stark, T.D. and Eid, H.T. (1997). "Slope Stability Analyses in Stiff Fissured Clays," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 123, No. 4, April, 1997, pp. 335-343.

Stark, T.D., Choi, H., and McCone, S. (2005). "Shear Strengths for Analysis of Landslides," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 131, No. 5, May, 2005, pp. 575-588.

Stark, T.D. and Hussain, M. (2013). "Drained Shear Strength Correlations for Slope Stability Analyses," *J. of Geotechnical Engineering*, ASCE, 139(6), June, 2013, pp. 853-862.

Stark, T.D. and Poeppel, A.R. (1994). "Landfill Liner Interface Strengths from Torsional Ring Shear Tests," *Journal of Geotechnical Engineering*, ASCE, Vol. 120, No. 3, March, pp. 597-615.

Stark, T.D. O'Donnell, D.P., and Lopez, S. (2015). "Use of Fully Softened Strength Correlations for Design." UNDER REVIEW BY *Geotechnique, November.* 2015, 30 p.

Stauffer, P. A., and Wright, S. G. (1984). *An examination of earth slope failures in Texas*. Center for Transportation Research, University of Texas at Austin, Austin, TX, 273.

Steward, H.E. and Cripps, J.C. (1983). "Some engineering implications of chemical weathering of pyretic shale." *Quarterly Jour. Engng. Geology*, v.16:281-289.

Terzaghi, K. (1936). "Stability of slopes of natural clay." *Proceedings of the 1st International Conf. on Soil Mechanics and Foundation Engineering*, 1, 161–165.

Thomson, S., and Kjartanson, B. H. (1985). "Study of delayed failure in a cut slope in stiff clay." *Can. Geotech. J.*, 22(2), 286–297.

Tiedemann, B. (1937). "Uber die Schubfestigkeit bindiger Boden." *Bautechnik*, 15(30 and 33), 400 – 403 and 433 – 435.

Tiwari, B., and Ajmera, B. (2011). "A new correlation relating the shear strength of reconstituted soil to the proportions of clay minerals and plasticity characteristics." *Applied Clay Science*, Elsevier B.V., 53(1), 48-57.

Tiwari, B. and Ajmera, B. (2014). "Reduction in Fully Softened Shear Strength of Natural Clays with NaCl Leaching and Its Effect of Slope Stability." *Journal of Geotechnical and Geoenvironmental Engineering*, 04014086-1– 04014086-10.

Tiwari, B. and Marui, H. (2005). "A new method for the correlation of residual shear strength of soil with mineralogical composition." *Journal of Geotechnical and Geoenvironmental Engineering*, 131 (9), 1139-1150.

Tiwari, B., Tuladhar, G.R., and Marui, H. (2005). "Variation in residual shear strength of the soil with salinity of pore fluid." *J. Geotech. Geoenviron. Eng.*, ASCE, 131:12, 1445-1456.

USACE (1983) Long-term reduction and slough slides in Mississippi river levees, Waterways Experiment Station, Vicksburg, MS.

VandenBerge, D. R., Duncan, J. M., and Brandon, T. L. (2013). “Fully softened strength of natural and compacted clays for slope stability.” *Proceedings of Geo-Congress 2013: Stability and Performance of Slopes and Embankments III*, 221–233.

VandenBerge, D. R., Duncan, J. M., and Brandon, T. L. (2015). “Undrained Strength of Compacted Clay under Principal Stress Reorientation.” *ASCE Journal of Geotechnical and Geoenvironmental Engineering, August.* 2015, pp. 04015035-1-04015035-11.

Vaughan, P. R., and Walbancke, H. J. (1973). “Pore pressure changes and the delayed failure of cutting slopes in overconsolidated clay.” *Géotechnique*, 23(4), 531–539.

VonThun, J. L. (1985). San Luis dam upstream slide. I*n proceedings of the eleventh international conference on soil mechanics and foundation engineering*, San Francisco, 12 – 16 August, pp. 2593 – 2598.

Wright, S. G. (2005). *Evaluation of soil shear strengths for slope and retaining wall stability analyses with emphasis on high plasticity clays*. Center for Transportation Research, University of Texas at Austin, 100.

Wright, S. G., Zornberg, J. G., and Aguettant, J. E. (2007). “The fully softened shear strength of high plasticity clays.” Center for Transportation Research, University of Texas at Austin, 132.

11. Additional/Uncited References

(a) Measurement

- ASTM test methods
- Other test methods
- Journal of Geotechnical Testing articles
- Other Papers/Sources

(b) FSS in overconsolidated clays

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(c) FSS in compacted clays

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(d) Drained strength v. undrained strength

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(e) FSS at low effective normal stresses

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(f) Empirical correlations for slope stability analyses

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(g) Other possible topics

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12. Uncited References

(a) Measurement

American Society for Testing and Materials (ASTM). (2008). "Standard test method for particle-size analysis of soils." (*D 422) 2008 annual book of ASTM standards,* Vol. 04.08, West Conshohocken, Pa.

American Society for Testing and Materials (ASTM). (2011). “Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.” (*D3080) 2008 annual book of ASTM standards,* Vol. 04.08, West Conshohocken, Pa.

American Society for Testing and Materials (ASTM). (2008). “Standard test method for liquid limit, plastic limit, and plasticity index of soils.” *(D 4318) 2008 annual book of ASTM standards,* Vol. 04.08, West Conshohocken, Pa.

American Society for Testing and Materials (ASTM). (2008). “Standard test method for torsional ring shear test to determine drained fully softened shear strength and nonlinear strength envelope of cohesive soils (using normally consolidated specimen) for slopes with no preesixting shear surfaces.” (*D7608) 2008 annual book of ASTM standards,* Vol. 04.08, West Conshohocken, Pa.

American Society for Testing and Materials (ASTM). (2011). “Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.” (*D7181) 2008 annual book of ASTM standards*, Vol. 04.08, West Conshohocken, Pa.

(b) FSS in overconsolidated clays

Skempton, A. W. (1970). “First-time slides in over-consolidated clays.” *Geotechnique*, 20(3), 320–324.

(c) FSS in compacted clays

Green, R., and Wright, S. G. (1986). “Factors affecting the long term strength of compacted Beaumont clay.” Center for Transportation Research, University of Texas at Austin.

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.” Center for Transportation Research, University of Texas at Austin, 146

(d) Drained strength v. undrained strength

Borden, R. H., and Putrich, S. F. (1986). "Drained-strength parameters from direct shear tests for slope stability analyses in overconsolidated fissured residual soils." *Transportation Research Record*, N1089, 102-113.

Stark, T. D. (1995). "Measurement of drained residual strength of overconsolidated clays." *Transportation Research Record*, 1479, 26–34.conso

Stark, T. D., Choi, H., and McCone, S. (2005). "Drained shear strength parameters for analysis of landslides." *Journal of Geotechnical and Geoenvironmental Engineering*, 131(5), 575-588.

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

Dewoolkar, M. M., and Huzjak, R. J. (2005). "Drained residual shear strength of some claystones from Front Range, Colorado." *Journal of Geotechnical and Geoenvironmental Engineering*, 131(12), 1543-1551.

Eid, H. T. (1996). "Drained shear strength of stiff clays for slope stability analyses." PhD Dissertation. University of Illinois.

Kamei, T. (1987). "Some drained tests on normally consolidated cohesive soils." <http://ir.nagaokaut.ac.jp/dspace/bitstream/10649/397/1/K9_9.pdf> (May. 12, 2011).

Kanji, M. A. (1974). "The relationship between drained friction angles and Atterberg limits of natural soils." *Géotechnique*, 24(4), 671–674.

Lupini, J. F., Skinner, A. E., and Vaughan, P. R. (1981). "The drained residual strength of cohesive soils." *Géotechnique*, 31(2), 181-213.

Stark, T. D., Choi, H., and McCone, S. (2005). "Drained shear strength parameters for analysis of landslides." *Journal of Geotechnical and Geoenvironmental Engineering*, 131(5), 575-588.

Stark, T. D., and Eid, H. T. (1994). "Drained residual strength of cohesive soils." *Journal of Geotechnical Engineering*, 120(5), 856–871.

(e) FSS at low effective normal stresses

Pedersen, R. C., Olson, R. E., and Rauch, A. F. (2003). "Shear and interface strength of clay at very low effective stress," *Geotechnical Testing Journal*, March, Vol. 26, No. 1, 8 pp.

(f) Empirical correlations for slope stability analyses

Aubeny, C. P., and Lytton, R. L. (2004). "Shallow slides in compacted high plasticity clay slopes." *Journal of Geotechnical and Geoenvironmental Engineering*, 130(7), 717–727.

Bjerrum, L. (1967). “Progressive failure in slopes of overconsolidated plastic clay and clay shales.” *Journal of Soil Mechanics and Foundations Division*, 93(SM5), 3-49.

Day, R. W. (1992a). “Swell versus saturation for compacted clay.” *Journal of Geotechnical Engineering*, 118(8), 1272–1278.

Day, R. W. (1996a). “Design and repair for surficial slope failures.” *Practice Periodical on Structural Design and Construction*, 1(3), 83–87.

Cancelli, A. (1977). “Residual shear strength and stability analysis of a landslide in fissured overconsolidated clays.” *Bulletin of the International Association of Engineering Geology*, 16, 193-197.

Charles, J. A., and Soares, M. M. (1984). ‘‘Stability of compacted rockfill slopes.’’ *Geotechnique,* 34(1), 61–70.

Day, R. W. (1994b). “Surficial stability of compacted clay: Case study.” *Journal of Geotechnical Engineering*, 120(11), 1980–1990.

Day, R. W., and Axten, G. W. (1989a). “Surficial stability of compacted clay slopes.” *Journal of Geotechnical Engineering*, 115(4), 577.

Chandler, R. J., and Skempton, A. W. (1974). “The design of permanent cutting slopes in stiff fissured clays.” *Géotechnique*, 24(4), 457-466.

Day, R. W., and Axten, G. W. (1990). “Softening of fill slopes due to moisture infiltration.” *Journal of Geotechnical Engineering*, 116(9), 1424–1427.

Henkel, D. J. (1957). “Investigations of two long-term failures in London clay slopes at Wood Green and Northolt.” *Proceedings of the 4th International Conference in Soil Mechanics*, 2, 315–320.

Lade, P. V. (2010). “The mechanics of surficial failure in soil slopes.” *Engineering Geology*, Elsevier, 114(1-2), 57–64.

Putrich, S. F., Borden, R. H., and Lambe, P. C. (1986). “Analysis of slope failure in overconsolidated fissured residual soils: A case study.” *Transportation Research Record*, (1089), 114-123.

Vaughan, P. R., and Walbancke, H. J. (1973). “Pore pressure changes and the delayed failure of cutting slopes in overconsolidated clay.” *Géotechnique*, 23(4), 531–539.

Skempton, A. W. (1964). “Long-term stability of clay slopes.” *Géotechnique*, 14(2), 77-102.

Skempton, A. W. (1977). “Slope stability of cuttings in brown London clay.” *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering*, 3, 261-270.

Stauffer, P. A., and Wright, S. G. (1984). “An examination of earth slope failures in Texas,” Center for Transportation Research, University of Texas at Austin.

Terzaghi, K. (1936). “Stability of slopes of natural clay.” *Proceedings of the 1st International Conference of Soil Mechanics and Foundations*, 161-165.

USACE (2003). “Slope stability, Engineer Manual 1110-2-1906,” U.S. Army Corps of Engineers, Washington, D.C.

Collins, I. F., Gunn, C. I. M., Pender, M. J., and Wang, Y. (1988). ‘‘Slope stability analyses for materials with a nonlinear failure envelope.’’ *International Journal for Numerical and Analytical Methods in Geomechanics.*, 12(5), 533–550.

Crabb, G. I., and Atkinson, J.H. (1988). “Determination of soil strength parameters for the analysis of highway slope failures,” *Slope Stability Engineering – Developments and Applications, Proceedings of the international conference on slope stability, Institution of Civil Engineers*, 1991.

Lade, P. V. (2010). The mechanics of surficial failure in soil slopes,” *Engineering Geology*, 114, 57-64.

Perry, J. (1994). “A technique for defining non-linear shear strength envelopes and their incorporation in a slope stability method of analysis.” *Quarterly Journal of Engineering Geology*, 27(3), 231–241.

Abrams, T. G., and Wright, S. G. (1972). *A survey of earth slope failures and remedial measures in Texas*. Center for Transportation Research, University of Texas at Austin, 109.

Bhattarai, P., Marui, H., Tiwari, B., Watanabe, N., and Tuladhar, G. R. (2006). “Influence of weathering on physical and mechanical properties of mudstone.” *Proceedings of the International Symposium on Disaster Mitigation of Debris Flows, Slope Failures and Landslides*, 467-479.

Cancelli, A. (1981). “Evolution of slopes in over-consolidated clays.” *Proceedings of the 10th International Conference on Soil Mechanics and Foundations Engineering*, 3, 377–380.

Chandler, R. J. (1974). “Lias Clay: The long-term stability of cutting slopes.” *Géotechnique*, 24(1), 21–38.

Chandler, R. J. (1976). “The history and stability of two Lias Clay slopes in the Upper Gwash Valley, Rutland [and discussion].” *Philosophical Transactions of the Royal Society of London. Series A, Mathematical and Physical Sciences*, 283(1315), 463–491

Chandler, R. J., and Skempton, A. W. (1974). “The design of permanent cutting slopes in stiff fissured clays.” *Géotechnique*, 24(4), 457-466.

Charles, J. A., and Bromhead, E. N. (2008). “Contributions to Géotechnique 1948–2008: Slope stability and embankment dams.” *Géotechnique*, 58(5), 385-389.

Charles, J. A., and Soares, M. M. (1984). “The stability of slopes in soils with nonlinear failure envelopes.” *Canadian Geotechnical Journal*, 21(3), 397-406.

Chin, T. Y., and Sew, G. S. (2001). “The determination of shear strength in residual soils for slope stability analysis.” *Seminar Cerun Kebangsaan 2001, Cameron Highlands*.

Ching-Chuan, H., Yih-Jang, J., Lih-Kang, H., and Jin-Long, L. (2009). “Internal soil moisture and piezometric responses to rainfall-induced shallow slope failures.” *Journal of Hydrology*, Elsevier B.V., 370(1-4), 39-51.

Cho, S. E., and Lee, S. R. (2002). "Evaluation of surficial stability for homogeneous slopes considering rainfall characteristics." *Journal of Geotechnical and Geoenvironmental Engineering*, 128(9), 756-763.

Day, R. W. (1993). "Surficial slope failure: A case study." *Journal of performance of constructed facilities*, 7(4), 264–269.

DDay, R. W., and Axten, G. W. (1990). "Softening of fill slopes due to moisture infiltration." *Journal of Geotechnical Engineering*, 116(9), 1424–1427.

Duncan, J. M., and Dunlop, P. (1969). "Slopes in stiff-fissured clays and shales." *Journal of Soil Mechanics and Foundations Division*, 95(SM2), 467-491.

Dunlop, P., and Duncan, J. M. (1970). "Development of failure around excavated slopes." *Journal of the Soil Mechanics and Foundations Division*, 96(2), 471–493.

Early, K. R., and Skempton, A. W. (1972). "Investigations of the landslide at Walton's Wood, Staffordshire." *Quarterly Journal of Engineering Geology*, 5(1-2), 19-41.

Eigenbrod, K. D. (1975). "Analysis of the pore pressure changes following the excavation of a slope." *Canadian Geotechnical Journal*, 12, 429-440.

Esu, F. (1966). "Short-term stability of slopes in unweathered jointed clays." *Géotechnique*, 16(4), 321-328.

Fourie, A. B. (1996). "Predicting rainfall-induced slope instability." *Proceedings of Institution of Civil Engineers, Geotechnical Engineering*, 119, 211-218.

Henkel, D. J. (1957). "Investigations of two long-term failures in London Clay slopes at Wood Green and Northolt." *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering*, 2, 315–320.

James, P. M. (1970). "Time effects and progressive failure in clay slopes." PhD Thesis, University of London.

Kassiff, G., and Alpan, I. (1973). "A slope failure in swelling clay." *Canadian Geotechnical Journal*, 10(3), 531-536.

Kenney, T. C., and Uddin, S. (1974). "Critical period for stability of an excavated slope in clay soil." *Canadian Geotechnical Journal*, 11, 620–623.

Lade, P. V. (2010). "The mechanics of surficial failure in soil slopes." *Engineering Geology*, Elsevier, 114(1-2), 57–64.

Lambe, P. C., Silva, F., Marr, W. A., and Lambe, T. W. (1989). "Instability of natural slopes in Puerto Rico." *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering*, Rio de Janeiro, 13-18.

Ling, H. I., Wu, M. H., Leshchinsky, D., and Leshchinsky, B. (2009). "Centrifuge modeling of slope instability." *Journal of Geotechnical and Geoenvironmental Engineering*, 135(6), 758-767.

Lo, K. Y., and Lee, C. F. (1973). "Stress analysis and slope stability in strain-softening materials." *Géotechnique*, 23(1), 1-11.

McGown, A., Saldivar-Sali, A., and Radwan, A. M. (1974). "Fissure patterns and slope failures in till at Hurlford, Ayrshire." *Quarterly Journal of Engineering Geology*, 7(1), 1-26.

esri, G., and Shahien, M. (2003). "Residual shear strength mobilized in first-time slope failures." *Journal of Geotechnical and Geoenvironmental Engineering*, 129(1), 12–31.

Mitchell, R. J. (1975). "Strength parameters for permanent slopes in Champlain Sea clays." *Canadian Geotechnical Journal*, 12(4), 447–455.

Morgenstern, N. R. (1977). “Slopes and excavations in heavily over-consolidated clays.” *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo*, State of t, 567–581.

Noor, M. J. M., and Hadi, B. A. (2010). “The role of curved-surface envelope Mohr-Coulomb model in governing shallow infiltration induced slope failure.” *Electronic Journal of Geotechnical Engineering*, 15.

Peck, R. B. (1967). “Stability of natural slopes.” *Journal of Soil Mechanics and Foundations Division*, 93(SM4), 403-417.

Picarelli, L., Urciuoli, G., Mandolini, A., and Ramondini, M. (2006). “Softening and instability of natural slopes in highly fissured plastic clay shales.” *Natural Hazards and Earth System Sciences*, 6(1985), 529–539.

otts, D. M., Kovacevic, N., and Vaughan, P. R. (1997). “Delayed collapse of cut slopes in stiff clay.” *Géotechnique*, 47(5), 953-982.

Pradel, D., and Raad, G. (1993a). “Effect of permeability on surficial stability of homogeneous slopes.” *Journal of Geotechnical Engineering*, 119(2), 315-332.

Quigley, R. M., Matich, M., Horvath, R. G., and Hawson, H. H. (1971). “Swelling clay in two slope failures at Toronto, Canada.” *Canadian Geotechnical Journal*, 8(3), 417–424.

Reid, M. E., Nielsen, H. P., and Dreiss, S. J. (1988). “Hydrologic factors triggering a shallow hillslope failure.” *Bulletin of the Association of Engineering Geologists*, 25(3), 349–361.

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*. Center for Transportation Research, The University of Texas at Austin, 154.

Skempton, A. W. (1964). “Long-term stability of clay slopes.” *Géotechnique*, 14(2), 77-102.

Skempton, A. W. (1969). “General report on stability of natural slopes and embankment foundations.” *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, 3, 151-155

Skempton, A. W. (1977). “Slope stability of cuttings in Brown London Clay.” *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering*, 3, 261-270.

Skempton, A. W., and DeLory, F. A. (1957). “Stability of natural slopes in London Clay.” *Proceedings of the 4th International Conference on Soil Mechanics and Foundations Engineering*, 2, 378–381.

Skempton, A. W., and Hutchinson, J. N. (1969a). “Stability of natural slopes and embankment foundations.” *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, 4(State of the Art), 291-340.

Skempton, A. W., and Hutchinson, J. N. (1969b). “Discussion of stability of natural slopes and embankment foundations.” *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, 3, 377-414.

Stark, T. D., and Eid, H. T. (1992). “Comparison of field and laboratory residual strengths.” *Stability and Performance of Slopes and Embankments II Proceedings*, 876-889.

soils.” *Journal of Geotechnical Engineering*, 121(9), 670-673.

Stark, T. D., and Eid, H. T. (1997). “Slope stability analyses in stiff fissured clays.” *Journal of Geotechnical and Geoenvironmental Engineering*, 123(4), 335-343.

Stauffer, P. A., and Wright, S. G. (1984). *An examination of earth slope failures in Texas*. Center for Transportation Research, University of Texas at Austin, 273.

Terzaghi, K. (1936). “Stability of slopes of natural clay.” *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering*, 1, 161-165.

Thomson, S., and Kjartanson, B. H. (1985). “Study of delayed failure in a cut slope in stiff clay.” *Canadian Geotechnical Journal*, 22(2), 286-297.

Tohari, A., Nishigaki, M., and Komatsu, M. (2007). “Laboratory rainfall-induced slope failure with moisture content measurement.” *Journal of Geotechnical and Geoenvironmental Engineering*, 133(5), 575-587.

Turnbull, W. J., and Hvorslev, M. J. (1967). “Special problems in slope stability.” *Journal of Soil Mechanics and Foundations Division*, 93(SM4), 499-528.

Weeks, A. G. (1969). “The stability of natural slopes in south-east England as affected by periglacial activity.” *Quarterly Journal of Engineering Geology*, 2(1), 49-61.

Wright, S. G. (2005). *Evaluation of soil shear strengths for slope and retaining wall stability analyses with emphasis on high plasticity clays*. Center for Transportation Research, University of Texas at Austin, 100.

(g) Other possible topics

Bishop, A. W. (1966). “The strength of soils as engineering materials.” *Geotechnique*, 16(2), 89-130.

Bishop, A. W., Green, G. E., Garga, V. K., Andresen, A., and Brown, J. D. (1971). “A new ring shear apparatus and its application to the measurement of residual strength.” *Géotechnique*, 21(4), 273-328.

Brooker, E. (1967). “Strain energy and behaviour of overconsolidated soils.” *Canadian Geotechnical Journal*, 4(3), 326–333.

Charles, J. A., and Watts, K. S. (1980). ‘‘The influence of confining pressure on the shear strength of compacted rockfill.’’ *Geotechnique*, 30(4), 353–367.

Chandler, R. J. (1984a). “Recent Ecompaureopean experience of landslides in overconsolidated clays and soft rocks.” *Proceedings of the 4th International Symposium on Landslides*, Toronto, 61–81.

Day, R. W. (1992b). “Effective cohesion for compacted clay.” *Journal of Geotechnical Engineering*, 118(4), 611-619.

Day, R. W. (1994a). “Swell-shrink behavior of compacted clay.” *Journal of Geotechnical Engineering*, 120(3), 618–623.

Duan, S., Nakagawa, K., and Sakaida, T. (1991). “A mathematical model to approach the fracture process of overconsolidated clay.” *Engineering Fracture Mechanics*, 38(6), 361-369.

Gourlay, A. W., and Wright, S. G. (1984). *Initial laboratory study of the shear strength properties of compacted, highly plastic clays used for highway embankment* *construction in the area of Houston, Texas*. Center for Transportation Research, University of Texas at Austin, 224.

Green, R., and Wright, S. G. (1986). *Factors affecting the long term strength of compacted Beaumont Clay*. Center for Transportation Research, University of Texas at Austin, 222.

Henkel, D. J., and Skempton, A. W. (1955). "A landslide at Jackfield, Shropshire, in a heavily over-consolidated clay." *Géotechnique*, 5(2), 131–137.

Insley, A. E. (1965). "A study of a large compacted clay embankment-fill failure." *Canadian Geotechnical Journal*, 2(3), 274-286.

Ladd, C. C. (1959). "Mechanism of swelling by compacted clays." *Water Tensions; Swelling Mechanisms; Strength of Compacted Soil. Highway Research Board Bulletin 245*, 10-26.

Lovell, C. W., and Johnson, J. M. (1981). "Shearing behavior of compacted clay after saturation." *Laboratory Shear Strength of Soil. ASTM STP 740*, R. N. Yong and F. C. Townsend, eds., American Society for Testing and Materials, 277-293.

McBrayer, M. C., Mauldon, M., Drumm, E. C., and Wilson, G. V. (1997). "Infiltration tests on fractured compacted clay." *Journal of Geotechnical and Geoenvironmental Engineering*, 123(5), 469-473.

Palmer, A. C., and Rice, J. R. (1973). "The growth of slip surfaces in the progressive failure of over-consolidated clay." *Proceedings of the Royal Society of London. A Mathematical and Physical Sciences*, The Royal Society, 332(1591), 527-548.

Rao, S. M., and Revanasiddappa, K. (2000). "Role of matric suction in collapse of compacted clay soil." *Journal of Geotechnical and Geoenvironmental Engineering*, 126(1), 85-90.

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*. Center for Transportation Research, University of Texas at Austin, 146.

Seed, H. B., Mitchell, J. K., and Chan, C. K. (1960). "The strength of compacted cohesive soils." *Research Conference on Shear Strength of Cohesive Soils*, ASCE, Boulder, Colorado, 877–964.

Singh, R., Henkel, D. J., and Sangrey, D. A. (1977). "Shear and Ko swelling of overconsolidated clays." *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering*, 367-376.

Smith, R. E., Jahangir, M. A., and Rinker, W. C. (2006). "Selection of design strengths for overconsolidated clays and clay shales." *40th Symposium on Engineering Geology and Geotechnical Engineering*, 1-11.

Tinjum, J. M., Benson, C. H., and Blotz, L. R. (1997). "Soil-water characteristic curves for compacted clays." *Journal of Geotechnical and Geoenvironmental Engineering*, 123(11), 1060-1069.

Wang, X., and Benson, C. H. (1995). "Infiltration and saturated hydraulic conductivity of compacted clay." *Journal of Geotechnical Engineering*, 121(10), 713-722.

Yoshida, N., Morgenstern, N. R., and Chan, D. H. (1991b). "Finite-element analysis of softening effects in fissured, overconsolidated clays and mudstones." *Canadian Geotechnical Journal*, 28(1), 51-61

Kayyal, M. K., and Wright, S. G. (1991). *Investigation of long-term properties of Paris and Beaumont Clays in earth embankments*. Center for Transportation Research, University of Texas at Austin, Austin, 134

McCook, D. K. (1997). "Surficial slides in highly plastic clay embankments." *Infrastructure Condition Assessment: Art, Science, and Practice*, ASCE, 227–236.

Skempton, A. W. (1985). "Residual strength of clays in landslides, folded strata and the laboratory." *Géotechnique*, 35(1), 3-18.

Tiwari, B., and Ajmera, B. (2010). *A new correlation for shear strength of reconstituted soil with proportions of clay minerals and plasticity characteristics*. Fullerton, 88.

Tiwari, B., and Ajmera, B.. (2011). "A new correlation relating the shear strength of reconstituted soil to the proportions of clay minerals and plasticity characteristics." *Applied Clay Science*, Elsevier B.V., 53(1), 48-57.

United States Army Corps of Engineers (USACE) (1970). "Laboratory soils testing, Engineer Manual 1110-2-1906," U.S. Army Corps of Engineers, Washington, D.C.

USACE (2000). "Design and construction of levees, Engineer Manual 1110-2-1913," U.S. Army Corps of Engineers, Washington, DC.

Wright, S. G., Zornberg, J. G., and Aguettant, J. E. (2007). "The fully softened shear strength of high plasticity clays." Center for Transportation Research, University of Texas at Austin.

Aguettant, J. E. (2006). "The fully softened shear strength of high plasticity clays," Thesis submitted in partial fulfillment of the requirements for Master of Science in Engineering, The University of Texas at Austin.

Atkinson, J. H. and Farrar, D. M. (1985). "Stress path tests to measure soil strength parameters for shallow landslips", *Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, 1985, Vol. 2, 983-986.

de Mello, V. F. B. (1977). "Reflections on design decisions of practical significance to embankment dams," *Geotechnique*, Vol. 27, No. 3, 281-354.

Kayyal, M. K., and Wright, S. G. (1991). *Investigation of long-term properties of Paris and Beaumont Clays in earth embankments*. Center for Transportation Research, University of Texas at Austin, Austin, 134.

Maksimovic, M. (1989). "Nonlinear failure envelope for soils.'' *Journal of Geotechnical Engineering,* 115(4), 581–586.

Al-Homoud, A. S., Basma, A. A., and Al Bashabsheh, M. A. (1995). "Cyclic swelling behavior of clays." *Journal of Geotechnical Engineering*, 121(7), 562-565.

Albrecht, B. A., and Benson, C. H. (2001). "Effect of desiccation on compacted natural clays." *Journal of Geotechnical and Geoenvironmental Engineering*, 127(1), 67-75.

Allred, B. J. (2000). “Survey of fractured glacial till geotechnical characteristics: Hydraulic conductivity, consolidation, and shear strength.” *Ohio Journal of Science*, 100(3-4), 63-72.

Asmaranto, R., Aryani, R. A., and Anwar, N. (2010). “Changes of soil erodibility due to wetting and drying cycle repetitions on the residual soil.” *International Journal of Academic Research*, 2(5), 149-152.

Baker, R. (2004). “Nonlinear Mohr envelopes based on triaxial data.” *Journal of Geotechnical and Geoenvironmental Engineering*, 130(5), 498-506.

Bell, D. O. N., and Wright, S. G. (1991). *Numerical modeling of the response of cylindrical specimens of clay to drying*. Center for Transportation Research, University of Texas at Austin, 48.

Bishop, A. W., Green, G. E., Garga, V. K., Andresen, A., and Brown, J. D. (1971). “A new ring shear apparatus and its application to the measurement of residual strength.” *Géotechnique*, 21(4), 273-328.

Bishop, A. W., Webb, D. L., and Lewin, P. I. (1965). “Undisturbed samples of London Clay from the Ashford Common Shaft: Strength–effective stress relationships.” *Géotechnique*, 15(1), 1-31.

Bishop, A. W. (1966). “The strength of soils as engineering materials”, *Géotechnique*, 16(2), 89-130.

Botts, M. E. (1998). “Effects of slaking on the strength of clay shales: A critical state approach.” *Proceedings of the 2nd International Symposium on the Geotechnics of Hard Soils/Soft Rocks*, 1(October), 447–458.

Burland, J. B. (1990). “On the compressibility and shear strength of natural clays.” *Géotechnique*, 40(3), 329–378.

Burland, J. B., Longworth, T. I., and Moore, J. F. A. (1977). “A study of ground movement and progressive failure caused by a deep excavation in Oxford Clay.” *Géotechnique*, 27(4), 557-591.

Cascini, L., Cuomo, S., Pastor, M., and Sorbino, G. (2010). “Modeling of rainfall-induced shallow landslides of the flow-type.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(1), 85-98.

Chandler, R. J. (1970). “A shallow slab slide in the Lias Clay near Uppingham, Rutland.” *Géotechnique*, 20(3), 253-260.

Chandler, R. J. (1972). “Lias Clay: Weathering process and their effect on shear strength.” *Géotechnique*, 22(4), 403-431.

.

Chandler, R. J. (1984b). “Delayed failure and observed strength of first- time slides in stiff clays.” *Proceedings of the 4th International Symposium on Landslides*, Toronto, 19-26.

Chandler, R. J., Pachakis, M., Mercer, J., and Wrightman, J. (1973). “Four long-term failures of embankments founded on areas of landslip.” *Quarterly Journal of Engineering Geology*, 6(3-4), 405-422.

Chandler, R. J., and Apted, J. P. (1988). “The effect of weathering on the strength of London Clay.” *Quarterly Journal of Engineering Geology*, 21, 59-68.

Collins, B. D., and Znidarcic, D. (2004). “Stability analyses of rainfall induced landslides.” *Journal of Geotechnical and Geoenvironmental Engineering*, 130(4), 362-372.

Cooling, L. F., and Skempton, A. W. (1942). “A laboratory study of London Clay.” *Journal of the ICE*, 17(3), 251-276.
Crawford, C. B., and Eden, W. J. (1965). “A comparison of laboratory results with in-situ properties of Leda Clay.” *Proceedings of the 6th International Conference on Soil Mechanics and Foundations Engineering*, 1, 31-35.
Eigenbrod, K. D., and Morgenstern, N. R. (1972). “A slide in cretaceous bedrock, Devon, Alberta.” *Geotechnical Practice for Stability in Open Pit Mining*, 223-238.
Ellis, E., and O’Brien, T. (2007). “Effect of height on delayed collapse of cuttings in stiff clay.” *Proceedings of the Institution of Civil Engineers-Geotechnical engineering*, Telford, 160(2), 73–84.
Feda, J., Bohac, J., and Herle, I. (1995). “Shear resistance of fissured Neogene clays.” *Engineering Geology*, 39(3-4), 171-184.
Fox, P. J., and Stark, T. D. (2004). “State-of-the-art report: GCL shear strength and its measurement.” *Geosynthetics International*, 11(3), 141-175.
Graham, J., and Au, V. C. S. (1985). “Effects of freeze–thaw and softening on a natural clay at low stresses.” *Canadian Geotechnical Journal*, 22(1), 69-78.
Graham, J., and Shields, D. H. (1985). “Influence of geology and geological processes on the geotechnical properties of a plastic clay.” *Engineering Geology*, Elsevier, 22(2), 109–126.
Gregory, C. H. (1844a). “On railway cuttings and embankments; with an account of some slips in the London Clay, on the line of the London and Croydon railway.” *Minutes of the Proceedings*, 3, 135–145.
Gregory, C. H. (1844b). “Discussion on railway cuttings and embankments; with an account of some slips in the London Clay, on the line of the London and Croydon railway.” *Minutes of the Proceedings*, 3, 145-173.
Griffiths, D. V., and Li, C. O. (1993). “Analysis of delayed failure in sloping excavations.” *Journal of Geotechnical Engineering*, 119(9), 1360-1378.
Guan, G. S., Rahardjo, H., and Choon, L. E. (2010). “Shear strength equations for unsaturated soil under drying and wetting.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(4), 594-606.
Henkel, D. J. (1956). “The effect of overconsolidation on the behaviour of clays during shear.” *Géotechnique*, 6(4), 139-150.
Henkel, D. J. (1960). “The shear strength of saturated remoulded clays.” *Research Conference on Shear Strength of Cohesive Soils*, ASCE, Boulder, Colorado, 533–554.
Hussain, M. (2010). “Analysis and behavior of preexisting landslides.” PhD Thesis. University of Illinois.
Hvorslev, M. J. (1936). “A ring shear apparatus for the determination of the shearing resistance and plastic flow of soils.” *Proceedings of the 1st International Conference on Soil Mechanics and Foundations*, 2, 125-129.
Hvorslev, M. J. (1939). “Torsion shear tests and their place in the determination of the shearing resistance of soils.” *Proceedings of the American Society Testing Material*, 39, 999–1022.

Hvorslev, M. J. (1969). *Physical properties of remolded cohesive soils*. *Translation 69-5*, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, 165.

Johnson, J. M., and Lovell, C. W. (1979). "The effect of laboratory compaction on the shear behavior of a highly plastic clay after saturation and consolidation : Interim report." *Joint Transportation Research Program. Paper 849*, <http://docs.lib.purdue.edu/jtrp/849/> (Jun. 30, 2011).

Kamai, T. (1998). "Monitoring the process of ground failure in repeated landslides and associated stability assessments." *Engineering Geology*, Elsevier, 50(1-2), 71–84.

Kaya, A. (2009). "Residual and fully softened strength evaluation of soils using Artificial Neural Networks." *Geotechnical and Geological Engineering*, 27(2), 281-288.

Kayyal, M. K., and Wright, S. G. (1991). *Investigation of long-term properties of Paris and Beaumont Clays in earth embankments*. Center for Transportation Research, University of Texas at Austin, Austin, 134.

Kim, C., Snell, C., and Medley, E. (2004). "Shear strength of Franciscan complex melange as calculated from back-analysis of a landslide." *5th International Conference on Case Histories in Geotechnical Engineering*, 8.

Kwan, D. (1971). "Observations of the failure of a vertical cut in clay at Welland, Ontario." *Canadian Geotechnical Journal*, 8(2), 283–298.

Lo, K. Y. (1970). "The operational strength of fissured clays." *Géotechnique*, 20(1), 57–74.

Maksimovic, M. (1989). "Nonlinear failure envelope for soils." *Journal of Geotechnical Engineering*, 115(4), 581-586.

Marcotte, M., and Silvestri, V. (1982). "Shear strength of the weathered Champlain clay measured in a large diameter triaxial test." *Canadian Geotechnical Journal*, 19(5), 413-420.

Marsland, A. (1971). "The shear strength of stiff fissured clays." *In Stress-strain behavior of soils. Roscoe Memorial Symposium*, Foulis, Henley-on-Thames, UK, 59-68.

Marsland, A., and Butler, M. E. (1967). "Strength measurements on stiff fissured Barton Clay from Fawley (Hampshire)." *Proceedings of the Geotechnical Conference Vol. 1*, Building Research Station, Ministry of Public Building and Works, Oslo, Norway, 139-145.

McCook, D. K. (1997). "Surficial slides in highly plastic clay embankments." *Infrastructure Condition Assessment: Art, Science, and Practice*, ASCE, 227–236.

McGown, A., and Radwan, A. M. (1975). "The presence and influence of fissures in the Boulder Clays of west central Scotland." *Canadian Geotechnical Journal*, 12, 84–97.

McManis, K., and Arman, A. (1986). "Sampling and testing in stiff crustal clays." *Geotechnical Aspects of Stiff and Hard Clays (GSP 2)*, ASCE, New York, New York, 1-13.

Mesri, G., and Abdel-Ghaffar, M. E. M. (1993). "Cohesion intercept in effective stress-stability analysis." *Journal of Geotechnical Engineering*, 119(8), 1229–1249.

Mesri, G., and Cepeda-Diaz, F. (1986). “Residual shear strength of clays and shales.” *Géotechnique*, 36(2), 269-274.

MMoon, A. T. (1984). “Effective shear strength parameters for stiff fissured clays.” *4th Australia-New Zealand Conference on Geomechanics*, 107-111.

Morgenstern, N. R., and Eigenbrod, K. D. (1972). “Seepage into an excavation in a medium possessing stress-dependent permeability.” *Proceedings of the Symposium on Percolation through Fissured Rock*, 1–15.

Morgenstern, N. R., and Eigenbrod, K. D. (1974). “Classification of argillaceous soils and rocks.” *Journal of the Geotechnical Engineering Division*, 100(10), 1137–1156.

Mullervonmoos, M., and Loken, T. (1989). “The shearing behaviour of clays.” *Applied Clay Science*, 4(2), 125-141.

Nonveiller, E., and Suklje, L. (1955). “Landslide Zalesina.” *Géotechnique*, 5(2), 143–153.

Noor, M. J. M., and Anderson, W. F. (2006). “A comprehensive shear strength model for saturated and unsaturated soils.” *Unsaturated Soils 2006 (GSP 147)*, ASCE, 1993-2003.

Olson, R. E., and Mitronovas, F. (1960). “Shear strength and consolidation characteristics of calcium and magnesium illite.” *Symposium on the Engineering Aspects of the Physico-Chemical Properties of Clays*, 185-209.

Osipov, V. I., Bik, N. N., and Rumjantseva, N. A. (1987). “Cyclic swelling of clays.” *Applied Clay science*, 2(4), 363–374.

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.” *Research Conference on Shear Strength of Cohesive Soils*, ASCE, 765–791.

Potts, D. M., Dounias, G. T., and Vaughan, P. R. (1987). “Finite element analysis of the direct shear box test.” *Geotechnique*, 37(1), 11-23.

PRivard, P. J., and Lu, Y. (1978). “Shear strength of soft fissured clays.” *Canadian Geotechnical Journal*, 15(3), 382-390.

Rowe, P. W. (1972). “The relevance of soil fabric to site investigation practice.” *Géotechnique*, 22(2), 195–300.

Saada, A. S., Bianchini, G. F., and Liang, L. (1994). “Cracks, bifurcation and shear bands propagation in saturated clays.” *Géotechnique*, 44(1), 35–64.

Sevaldson, R. A. (1956). “The slide in Lodalen, October 6th, 1954.” *Géotechnique*, 6(4), 167-182.

Skempton, A. W. (1945a). “A slip in the west bank of Eau Brink Cut.” *Journal of the ICE*, 24(7), 267–287.

Skempton, A. W. (1945b). “Discussion of a Slip in the west bank of the Eau Brink Cut.” *Journal of the ICE*, 24(8), 535-553.

Skempton, A. W. (1948). “The rate of softening in stiff fissured clays, with special reference to London Clay.” *Proceedings of the 2nd International Conference on Soil Mechanics and Foundations Engineering*, 2, 50-53.

Skempton, A. W. (1949). “A study of the geotechnical properties of some post-glacial clays.” *Géotechnique*, 1(1), 1-16.

Skempton, A. W. (1970). “First-time slides in over-consolidated clays.” *Géotechnique*, 20(3), 320–324.

Skempton, A. W. (1985). “Residual strength of clays in landslides, folded strata and the laboratory.” *Géotechnique*, 35(1), 3-18.

Skempton, A. W., Petley, D. J., and Schuster, R. L. (1969). “Joints and fissures in the London Clay at Wraysbury and Edgware.” *Géotechnique*, 19(2), 205-217.

Skempton, A. W., Vaughan, P. R., Rocke, G., and Penman, A. D. M. (1994). “Discussion on the failure of Carsington Dam.” *Géotechnique*, 44(4), 719-739.

Skempton, A. W., and Brown, J. D. (1961). “A landslide in Boulder Clay at Selset, Yorkshire.” *Géotechnique*, 11(4), 280-293.

Skempton, A. W., and Henkel, D. J. (1957). “Tests on London Clay from deep borings at Paddington, Victoria and the south bank.” *Proceedings of the 4th International Conference on Soil Mechanics and Foundation Engineering*, 1, 100-106.

Skempton, A. W., and Henkel, D. J. (1960). “Field observations on pore pressure in London Clay.” *Conf. on Pore Pressure and Suction in soils*, 81–84.

Skempton, A. W., and LaRochelle, P. (1965). “The Bradwell Slip: A short-term failure in London Clay.” *Géotechnique*, 15(2), 221-242.

Skempton, A. W., and Petley, D. J. (1967). “The strength along structural discontinuities in stiff clays.” *Proceedings of the Geotechnical Conference*, Thomas Telford, Oslo, 29-46.

Skempton, A. W., and Vaughan, P. R. (1993). “The failure of Carsington Dam.” *Géotechnique*, 43(1), 151-173.

Stark, T. D., and Duncan, J. M. (1991). “Mechanisms of strength loss in stiff clays.” *Journal of Geotechnical Engineering*, 117(1), 139-154.

Stark, T. D., and Eid, H. T. (1993). “Modified Bromhead ring shear apparatus.” *Geotechnical Testing Journal*, 16(1), 100-107.

Stark, T. D., and Eid, H. T. (1995). “Discussion of drained residual strength of cohesive Stark, T. D., and Hussain, M. (2010). “Shear strength in preexisting landslides.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(7), 957.

Taha, M. R. (2009). “Geotechnical properties of soil-ball milled soil mixtures.” *Nanotechnology in Construction 3*, Springer, 377–382.

Terzaghi, K., Peck, R. B., and Mesri, G. (1996). *Soil mechanics in engineering practice*. Wiley-Interscience, 547.

Thomson, S., and Morgenstern, N. R. (1979). “Landslides in argillaceous bedrock, Prairie Provinces, Canada.” *Rock Slides and Avalances: Vol.2*, 515-540.

Thomson, S., and Tweedie, R. W. (1978). “The Edgerton Landslide.” *Canadian Geotechnical Journal*, 15(4), 510-521.

Tiwari, B., Brandon, T. L., Marui, H., and Tuladhar, G. R. (2005). “Comparison of residual shear strengths from back analysis and ring shear tests on undisturbed and remolded specimens.” *Journal of Geotechnical and Geoenvironmental Engineering*, 131(9), 1071-1079.

Tiwari, B., Tuladhar, G. R., and Marui, H. (2011). “Effect of shearing speed on residual shear strength of natural soil obtained from mudstone.” *Geo-Frontiers 2011: Advances in Geotechnical Engineering*, 2786-2793.

Tiwari, B., and Ajmera, B. (2010). *A new correlation for shear strength of reconstituted soil with proportions of clay minerals and plasticity characteristics*. Fullerton, 88.

Tiwari, B., and Ajmera, B. (2011). “A new correlation relating the shear strength of reconstituted soil to the proportions of clay minerals and plasticity characteristics.” *Applied Clay Science*, Elsevier B.V., 53(1), 48-57.

Tiwari, B., and Marui, H. (2004). “Objective oriented multistage ring shear test for shear strength of landslide soil.” *Journal of Geotechnical and Geoenvironmental Engineering*, 130(2), 217-222.

Tiwari, B., and Marui, H. (2005). “A new method for the correlation of residual shear strength of the soil with mineralogical composition.” *Journal of Geotechnical and Geoenvironmental Engineering*, 131(9), 1139-1150.

Townsend, F. C., and Gilbert, P. A. (1974). “Preparation effects on clay shale classification indexes.” *Proceedings of National Meeting on Water Resources*, ASCE, Los Angeles, CA, 1-30.

Toyota, H., Nakamura, K., Sugimoto, M., and Sakai, N. (2009). “Ring shear tests to evaluate strength parameters in various remoulded soils.” *Géotechnique*, 59(8), 649–659.

Ward, W. H., Butler, M. E., and Samuels, S. G. (1959). “Further studies of the properties of London Clay.” *Géotechnique*, 9(2), 33-58.

Watson, J. D. (1956a). “Earth movement affecting L.T.E. railway in deep cutting east of Uxbridge.” *Proceedings of Engineering Divisions*, 5(5), 302-316.

Watson, J. D. (1956b). “Discussion of earth movement affecting L.T.E. railway in deep cutting east of Uxbridge.” *Proceedings of Engineering Divisions*, 5(5), 316-331.

Wright, S. G., Zornberg, J. G., and Aguettant, J. E. (2007). “The fully softened shear strength of high plasticity clays.” Center for Transportation Research, University of Texas at Austin, 132.

Wu, T. H., Randolph, B. W., and Huang, C. S. (1993). “Stability of shale embankments.” *Journal of Geotechnical Engineering*, ASCE, 119(1), 127–146.

Yoshida, N. (1991). “Time-dependent instability in fissured, over-consolidated clays and mudstones.” Ph.D. Thesis, University of Alberta, Edmonton.

Yoshida, N., Morgenstern, N. R., and Chan, D. H. (1990). “A failure criterion for stiff soils and rocks exhibiting softening.” *Canadian Geotechnical Journal*, 27(2), 195-202.

Yoshida, N., Morgenstern, N. R., and Chan, D. H. (1991a). “Analysis of softening effects in mudstone and over-consolidated clays.” *Soils and Foundations*, 31(1), 121–130.

Zornberg, J. G., Kuhn, J., and Wright, S. G. (2007). *Determination of field suction values, hydraulic properties, and shear strength in high PI clays*. Center for Transportation Research, University of Texas at Austin, 88.