Technical Note

# Fully Softened Shear Strength at Low Stresses for Levee and Embankment Design

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**Abstract:** Shallow slides in levee and other embankment slopes are usually controlled by effective normal stresses less than 12 kPa (250 psf). For first-time slides in fine-grained soils, the fully softened shear strength is frequently used to model the strength of embankment soils because it represents the shear strength remaining after the effects of overconsolidation, compaction, desiccation, or other strengthening processes have been removed because of wetting, infiltration, stress relief, swelling, and weathering. However, there is limited fully softened strength data at effective normal stresses less than 50 kPa (1,000 psf), so existing correlations in this stress range must be extrapolated. This paper presents new fully softened shear strength data at an effective normal stress of 12 kPa (250 psf) and recommendations for estimating and modeling the stress-dependent fully softened shear strength envelope directly or as a power function in stability analyses. **DOI: 10.1061/(ASCE)GT.1943-5606.0001151.**


**Author keywords:** Fully softened shear strength; Low effective normal stress; Power function; Levees; Highway embankments; Landslides; Slope stability.



## Introduction

In 2007, the Dallas Floodway levee system was assigned an overall rating of unacceptable during a periodic inspection [U.S. Army COE (USACE) 2007]. The overall rating was the cumulative result of significant deficiencies found during the inspection, including slope instability (Fig. 1), underseepage, and desiccation cracking in the levee. Because of these and other deficiencies, USACE determined that the Floodway did not meet current USACE design criteria regarding relevant factors of safety for embankment stability and seepage gradients (USACE 2007). In October 2010, a revision to the Floodway's 100-Year Levee Remediation Plan required site-specific testing to determine the fully softened shear strength for levee design because of uncertainty in previously used values that were estimated using an empirical correlation (Trinity River Corridor Protection Committee 2010). Compounding the issue is the maximum curvature or stress dependence of the fully softened strength envelope that occurs at low effective normal stresses. This lack of data in the area of largest curvature requires engineers to extrapolate a stress-dependent strength envelope, which increases uncertainty and error when using correlations in levee design that do not have data at low effective normal stresses. This paper fills the gap in the fully softened strength correlation by Stark et al. (2005) and updated by Stark and Hussain (2013) using data obtained from modified Bromhead ring shear tests (ASTM 2008c) on 36 soils to measure the fully softened shear strengths at an effective normal stress of 12 kPa. The updated correlation does not directly present the effects of the amount of coarse-grained soil or the amount of organics on the fully softened shear strength behavior for modeling and stability analyses, because the data are presented in terms of clay-size fraction and liquid limit. However, the measured shear strength parameters do reflect the soils tested, which can contain coarse and organic particles. This paper accounts for the amount of clay, clay mineralogy, and expansion via the clay size fraction and liquid limit, respectively, to present the measured fully softened strengths.

### *Slope Failures in Levees*

Levees are subjected to many cycles of wetting and drying. This extended wetting, desiccation, and weathering of levee soil can lead to formation of fissures that allow water to infiltrate deeper into the soil than surficial wetting. Upon seeping into the soil and cracks, the water can cause the soil to swell. This can lead to the outer portions of the levee becoming saturated, trapping air in the inner portions (Terzaghi et al. 1996). The pressure caused by the trapped air can create tension in the soil, which can lead to additional water being absorbed, reduced effective stress, and slope instability (Terzaghi et al. 1996). Failures caused by rapid wetting of desiccated soils are referred to as slaking and usually involve shallow slides (Fig. 1).

These slides are generally shallow, occurring at depths of less than 2.5 m (Fleming et al. 1992). Additionally, the slope failures typically occur between the crown and midslope (Fig. 1). For example, the slides in Fig. 1 have a depth of 1–2 m. The slides range in width from approximately 27.5 to 36.5 m (see Figs. 1 and 2). Because the shear strength of soil is stress dependent, data are needed at low effective normal stresses (e.g., depths of 1–2 m) to accurately represent the fully softened shear strength along the length of these shallow failure surfaces. Because of the shallow slide depths and the large curvature or stress dependence at low normal stresses, fully softened strength at an effective normal stress of 12 kPa was sought. An effective normal stress of 12 kPa corresponds to a slide depth of about 1.5 m, which models the average depth along the failure surface in these shallow slides.

Skempton (1970) concludes that fully softened strength is approximately equal to the drained peak strength of a reconstituted

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**Fig. 1.** Typical shallow slides in Dallas Floodway levee system with depths less than 2.5 m and widths from 27.5 to 36.5 m (photograph used with permission from USACE)

**Fig. 2.** Close-up of typical shallow slide in Dallas levees (photograph used with permission from USACE)

normally consolidated specimen. The fully softened strength may be used to model soils in a number of conditions, such as levees and embankments subjected to repeated cycles of weathering (Bhattarai et al. 2006; Duncan et al. 2011; Wright et al. 2007), soils undergoing first-time sliding (Skempton 1970, 1977), and fissured, overconsolidated soils (Skempton 1977). Therefore, the fully softened strength is applicable and used to reflect the shear strength of a normally consolidated fine-grained soil. For levee or embankment design purposes, the soil is assumed to be normally consolidated; that is, the effects of desiccation, compaction, overconsolidation, and so on have been removed due to many cycles of wetting, drying, and weathering (Wright et al. 2007).

## Test Procedure

Prior to ring shear testing, the Atterberg limits and clay size fraction (CF) of each soil were measured in accordance with ASTM D4318 and D422 (ASTM 2008a, b), respectively. Table 1 summarizes the 36 natural soils tested herein to quantify the fully softened strength at low effective stresses. The soil specimens were obtained by air drying a representative sample of each soil. After air drying, the soils were crushed with a mortar and pestle and processed through a U.S. standard No. 40 sieve. Ball-milling was not used for fully softened shear strength testing of the soils in Table 1, because doing so could change the texture and gradation of the soil and this level of disaggregation is not present from surficial weathering. Ball-milling is used for highly indurated shales and claystones to disaggregate clay particles to facilitate development of a residual strength condition (Stark and Hussain 2013). The processed specimens were then mixed to a water content near the liquid limit (LL) and placed in a climate-controlled room at 12.8°C with a 95% relative humidity to hydrate for 5–7 days.

The fully softened shear strength of the soils listed in Table 1 were measured using the modified Bromhead ring shear apparatus described in Stark and Eid (1993) and tested in accordance with ASTM (2008c). The modified Bromhead ring shear apparatus uses an annular specimen with an inside diameter of 70 mm and an outside diameter of 100 mm. A shear displacement rate of 0.018 mm/min was used and is the slowest rate available to ensure drained conditions. Using specimens from the same sample from the step described in the previous paragraph, specimens were placed in the modified ring shear apparatus, normally consolidated to an effective normal stress of 12 kPa, and then sheared under drained conditions. The fully softened shear strength is typically mobilized at a shear displacement of approximately 1.5 mm, and the test was stopped shortly thereafter because residual strength was not needed.

## Augmented Empirical Correlation

Stark and Eid (1997) concluded that a triaxial compression mode of shear and stress state better reflects shallow first-time slides in levees and other embankments than the horizontal mode of shear in the ring shear device, which occurs along a different stress path to failure than would occur in a shallow slide. The ring shear device creates a horizontal shear surface that simulates a weak bedding plane or preexisting shear surface in a natural slope or below a dam or levee. Because the shear testing was performed with a ring shear device, the fully softened friction angles measured were adjusted to reflect a triaxial compression mode of shear (Stark and Eid 1997). To accomplish this adjustment, the ring shear fully softened friction angles measured herein using normally consolidated specimens were increased by 2.5° as suggested by Stark and Eid (1997). Stark et al. (2005) state that increasing the fully softened friction angle by 2.5° should be verified with site-specific testing and existing correlations, such as Bjerrum and Simons (1960), Terzhagi et al. (1996), Naval Facilities Engineering Command (NAVFAC) (1971), and Mesri and Abdel-Ghaffar (1993).

Figs. 3 and 4 present the fully softened strength correlation by Stark and Hussain (2013) augmented by the data obtained at an effective normal stress of 12 kPa. A trend line for an effective normal stress of 12 kPa is added to each CF group. The four trend lines in each CF group can be used to create a fully softened strength envelope for use in stability analyses. The fully softened strength envelope is constructed using the estimated fully softened friction angle, the effective normal stress, and calculating the corresponding shear stress. The resulting strength envelope passing through effective normal stresses of 12, 50, 100, and 400 kPa is drawn through the origin because uncemented, normally consolidated fine-grained soil does not exhibit a cohesion intercept (Stark et al. 2005).

**Table 1.** List of Soils Used for Fully Softened Ring Shear Testing

| Number | Soil name | Location | LL | PL | CF | Activity (PI/CF) |
|---|---|---|---|---|---|---|
| 1 | Glacial till | Urbana, Illinois | 24 | 16 | 18 | 0.44 |
| 2 | Loess | Vicksburg, Mississippi | 28 | 18 | 10 | 1.00 |
| 3 | Duck Creek Shale | Fulton, Illinois | 37 | 25 | 19 | 0.63 |
| 4 | Slide debris | San Francisco, California | 37 | 26 | 28 | 0.39 |
| 5 | Sky Valley | Vallejo, California | 39 | 22 | 36 | 0.47 |
| 6 | Slope-wash material | San Luis, California | 42 | 24 | 34 | 0.53 |
| 7 | Crab Orchard shale | Peoria, Illinois | 44 | 24 | 32 | 0.63 |
| 8 | Cardinal Fly Ash Dam | Brilliant, Ohio | 44 | 19 | 39 | 0.64 |
| 9 | Colorado shale | Sunlight, Montana | 46 | 25 | 73 | 0.29 |
| 10 | Panoche mudstone | San Francisco, California | 47 | 27 | 41 | 0.49 |
| 11 | Panoche shale | San Francisco, California | 53 | 29 | 50 | 0.48 |
| 12 | Colluvium | Marietta, Ohio | 54 | 25 | 48 | 0.60 |
| 13 | Slide plane material | Los Angeles, California | 55 | 24 | 27 | 1.15 |
| 14 | Illinois Valley shale | Peru, Illinois | 56 | 24 | 45 | 0.71 |
| 15 | Comanche shale | Proctor Dam, Texas | 62 | 32 | 68 | 0.44 |
| 16 | Breccia material | Manta, Ecuador | 64 | 41 | 25 | 0.92 |
| 17 | Silty clay | Esperanza Dam, Ecuador | 64 | 41 | 21 | 1.10 |
| 18 | Claystone | Big Bear, California | 75 | 22 | 54 | 0.98 |
| 19 | Siltstone/claystone | Orange County, California | 75 | 37 | 48 | 0.79 |
| 20 | Bay mud | San Francisco, California | 76 | 41 | 16 | 2.19 |
| 21 | Patapsco shale | Washington, DC | 77 | 25 | 59 | 0.88 |
| 22 | Pierre shale | Limon, Colorado | 82 | 30 | 42 | 1.24 |
| 23 | Hollywood Landslide | Los Angeles, California | 82 | 31 | 50 | 1.02 |
| 24 | Lower Pepper shale | Waco Dam, Texas | 94 | 26 | 77 | 0.88 |
| 25 | Serpentinite clay | Marion County, California | 95 | 27 | 54 | 1.26 |
| 26 | Brown London clay | Bradwell, England | 101 | 35 | 66 | 1.02 |
| 27 | Cucaracha shale | Panama Canal, Panama | 111 | 42 | 63 | 1.10 |
| 28 | Denver shale | Denver, Colorado | 121 | 37 | 67 | 1.25 |
| 29 | Bearpaw shale | Saskatchewan, Canada | 128 | 27 | 43 | 2.35 |
| 30 | Pierre shale | Newcastle, Wyoming | 137 | 30 | 54 | 1.98 |
| 31 | Oahe Firm shale | Oahe Dam, South Dakota | 138 | 41 | 78 | 1.24 |
| 32 | Taylor shale | San Antonio, Texas | 170 | 39 | 72 | 1.82 |
| 33 | Pierre shale | Reliance, South Dakota | 184 | 55 | 84 | 1.54 |
| 34 | Bentonitic shale | Oahe Dam, South Dakota | 192 | 47 | 65 | 1.96 |
| 35 | Lea Park Bentonitic shale | Saskatchewan, Canada | 253 | 48 | 65 | 3.15 |
| 36 | Bearpaw shale | Ft. Peck Dam, Montana | 288 | 44 | 88 | 2.77 |

Note: PI = plasticity index; PL = plastic limit.



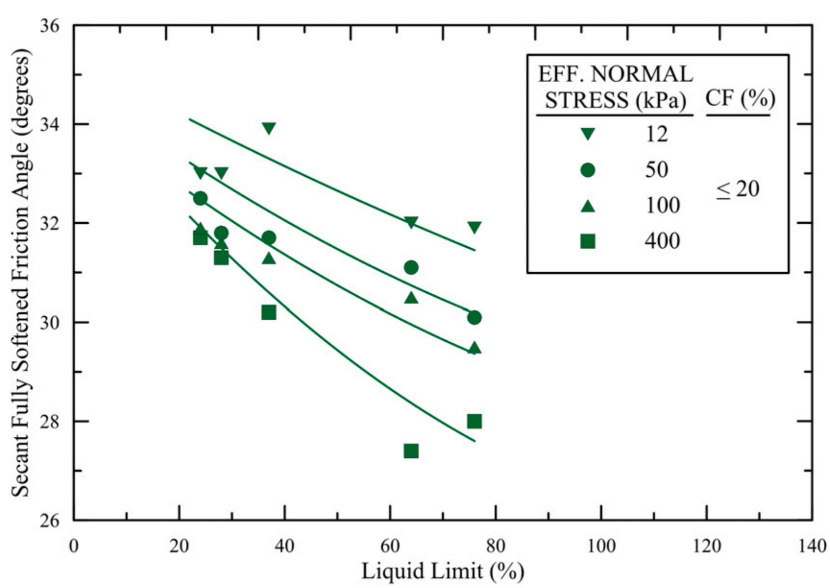


**Fig. 3.** Augmented drained fully softened friction angle correlation for CF ≤ 20%

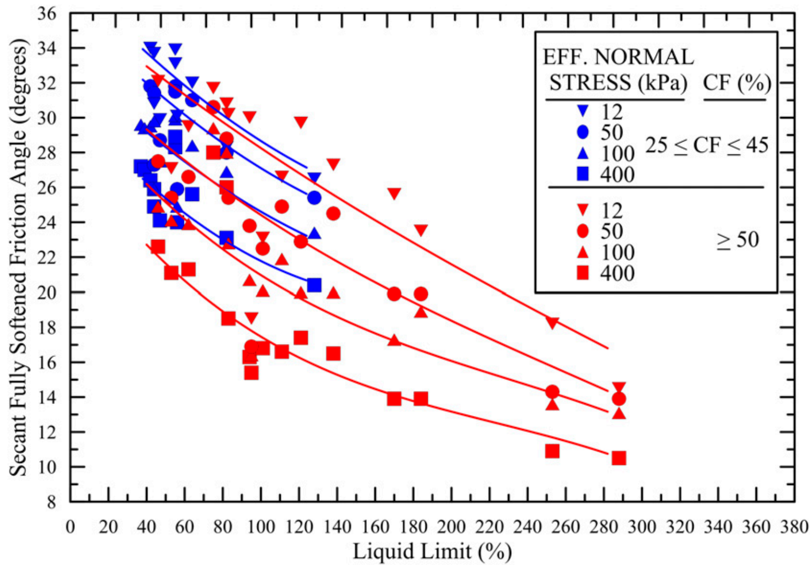


**Fig. 4.** Augmented drained fully softened friction angle correlation for CFs 25–45% and ≥50%

The fully softened friction angle relationship is presented in terms of CF and LL, and follows the same form of the correlations in Stark and Eid (1997), Stark et al. (2005), and Stark and Hussain (2013). In using LL rather than the plasticity index (PI) in the fully softened relationship, Stark and Eid (1997) and Stark et al. (2005) state that, in prior correlations [e.g., Bjerrum and Simons (1960), NAVFAC (1971), and Mesri and Abdel Ghaffar (1993)], there is considerable scatter in the fully softened friction angle for the plasticity index range of 10–100% and that Ladd et al. (1977) referred to this scatter by stating that "the scatter of $\phi'$ for the plasticity index range of 15–40%, where most of the data exist is considerable, which has prompted some to question the worth of such correlations." Stark and Eid (1997) and Stark et al. (2005) anticipated that the scatter is caused by omitting the effect of CF and effective normal stress on the fully softened friction angle. Furthermore, Mesri and Shahien (2003) asserted that the empirical correlations by Stark and Eid (1994, 1997) using LL and CF include less scatter because all samples for both LL and CF and for measurement of friction angles were prepared consistently following the procedure developed by Mesri and Cepeda-Diaz (1986), and because they include the influence of effective normal stress on measured friction angles.

## Fully Softened Strength Envelope

The fully softened shear strength envelope exhibits maximum curvature, or stress dependency, at low effective normal stresses because the strength envelope passes through the origin. The fully softened strength envelope displays curvature because, even for a random arrangement of particles, low effective normal stresses promote edge-to-face interactions, whereas high effective normal stresses promote face-to-face interaction of plate-shaped particles (Terzaghi et al. 1996). Thus as effective stress increases, face-to-face interaction among plate-shaped particles increases and friction angle decreases (i.e., the strength envelope flattens). This effect is more pronounced in soils with a high CF. Consequently, soils with a low CF will not exhibit as much curvature in their strength envelope as those with a high CF. The fully softened friction angles, $\phi'$, for fine-grained soil can be estimated using Figs. 3 and 4 for a particular effective normal stress using the LL and CF and interpolating between the empirical relationships presented in the next section. To be conservative, CF ≥ 50% can be used for design purposes with a representative value of LL.

For stability analyses, it is recommended that the entire strength envelope, or a secant friction angle corresponding to the average effective normal stress acting on the slide surface in that particular material, be used to estimate the applicable shear strengths. A number of slope stability software packages allow the entire strength envelope to be input using combinations of shear and normal stress to model the stress-dependent strength envelope.

## Equations and Spreadsheet for Empirical Correlations

Equations were developed for the 12-kPa effective normal stress trend line in each CF group. These equations augment those in Stark and Hussain (2013). The trend line for CF Group 1 (CF ≤ 20% and 30% ≤ LL < 80%) is presented as Eq. (1), Group 2 (25% ≤ CF ≤ 45% and 30% ≤ LL < 130%) as Eq. (2), and Group 3 (CF ≥ 50% and 30% ≤ LL < 300%) as Eq. (3). The upper and lower bounds of LL are specified because no ring shear data are available outside each LL range. These are

$$\phi'(\mathrm{LL}) = 35.33 - 5.85 \times 10^{-2}(\mathrm{LL}) + 9.71 \times 10^{-5}(\mathrm{LL})^2 \quad (1)$$

$$\phi'(\mathrm{LL}) = 38.10 - 1.19 \times 10^{-1}(\mathrm{LL}) + 2.48 \times 10^{-4}(\mathrm{LL})^2 \quad (2)$$

$$\phi'(\mathrm{LL}) = 36.45 - 9.18 \times 10^{-2}(\mathrm{LL}) - 1.09 \times 10^{-4}(\mathrm{LL})^2 + 1.10 \times 10^{-7}(\mathrm{LL})^3 \quad (3)$$

Stark and Hussain (2013) present a spreadsheet that utilizes only two parameters, CF and LL, as input and generates values of fully softened secant friction angles at effective normal stresses of 0, 50, 100, and 400 kPa and residual secant friction angles at effective normal stresses of 0, 50, 100, 400, and 700 kPa and the resulting strength envelopes. The spreadsheet is revised herein to include the fully softened strength trend lines at an effective normal stress of 12 kPa. Although these data augment the fully softened correlation at an effective normal stress of 12 kPa, both the estimated stress-dependent residual and fully softened shear strength envelopes (with

**Table 2.** Recommended Power Function Coefficients $a$ and $b$ to Estimate Fully Softened Shear Strength Envelopes for Three CF Groups

| | CF ≤ 20% | | | 25% ≤ CF ≤ 45% | | | CF ≥ 50% | | |
|---|---|---|---|---|---|---|---|---|---|
| Power function coefficient | Correlation | +95% | −95% | Correlation | +95% | −95% | Correlation | +95% | −95% |
| $a$ | 0.61 | 0.64 | 0.59 | 0.52 | 0.55 | 0.49 | 0.38 | 0.43 | 0.33 |
| $b$ | 0.97 | 0.97 | 0.96 | 0.92 | 0.92 | 0.92 | 0.88 | 0.88 | 0.87 |

the exception of the residual correlation at 12 kPa) are plotted in a single plot in the spreadsheet, as well as tabulated for use in stability software packages.

### Power Function Strength Envelope

The stress-dependent fully softened strength envelope can be modeled using a power function of the following form as suggested by Lade (2010)

$$\tau_{FS} = a \times P_a \times \left(\frac{\sigma'_n}{P_a}\right)^b \tag{4}$$

where $a$ and $b$ = dimensionless coefficients that control the scale and curvature of the strength envelope; $\sigma'_n$ = effective normal stress; $\tau_{FS}$ = fully softened shear strength; and $P_a$ = atmospheric pressure in the same units as $\tau_{FS}$ and $\sigma'_n$ (Lade 2010).

Table 2 presents values of $a$ and $b$ used to predict the fully softened strength envelope for the three CF groups in the correlation shown in Figs. 3 and 4. The values of $a$ and $b$ that provide a 95% confidence interval for each CF group are also presented in Table 2 and can be used to plot a 95% confidence interval for the fully softened strength envelope obtained from the correlations in Figs. 3 and 4. The 95% confidence intervals were calculated using the standard normal table (Z Table) and assuming that the data are normally distributed. Fig. 5 presents the estimated fully softened strength envelope for a soil with CF ≥ 50% at effective normal stresses of 12, 50, 100, and 400 kPa. Fig. 5 also includes a 95% confidence interval for the strength envelope. The recommended power function coefficients for the three CF groups (≤20%, 25–45%, and ≥50%), and the various shear strength envelopes are presented in Table 2. The coefficients $a$ and $b$ can be used with Eq. (4) to plot the stress-dependent fully softened strength envelope for factor of safety calculations, and the 95% confidence interval can be used to calculate a probability-based factor of safety as recommended by Duncan (2000). The ±95% strength envelopes correspond to the highest and lowest conceivable values as defined by Duncan (2000) and can be used to calculate the coefficient of variation of the factor of safety.

## Summary

The shear strength of *first-time* slides in repeatedly weathered soils can be modeled using fully softened shear strength (Skempton 1970, 1977; Bhattarai et al. 2006). Slope failures in levees and other embankments caused by repeated cycles of wetting, drying, and weathering usually involve shallow slides with depths of less than 2.5 m. As a result, the fully softened strength at effective stresses less than 12 kPa (250 psf) is frequently sought for levee and other embankment slope design. This paper presents fully softened strengths at an effective normal stress of 12 kPa from torsional ring shear tests on 36 soils to augment the correlation by Stark et al. (2005) and Stark and Hussain (2013).

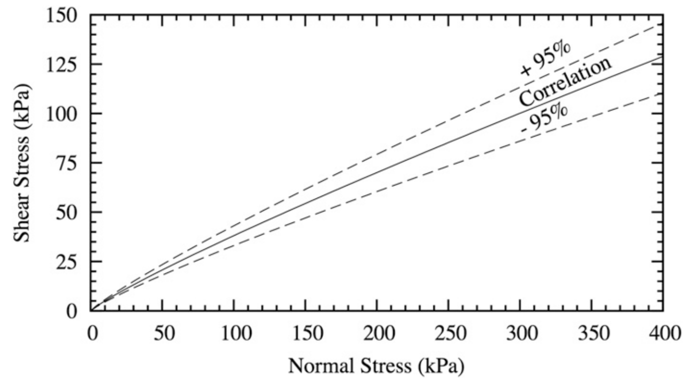


**Fig. 5.** 95% confidence interval on fully softened shear strength envelope for CF ≥ 50% estimated using the correlation in Fig. 4

The maximum curvature or stress dependency of the fully softened shear strength envelope occurs at effective normal stresses less than 100 kPa; thus, using current correlations without data below 50 kPa for the design of levees and other embankments can introduce considerable uncertainty and error in the slope design and evaluation. The empirical correlation for the fully softened secant friction angle using LL, CF, and effective normal stress proposed by Stark and Hussain (2013) is augmented by including trend lines for an effective normal stress of 12 kPa in Figs. 3 and 4.

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