Slope Stability Analyses in Stiff Fissured Clays

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Abstract: Results of torsional ring shear, direct shear, and triaxial compression tests on cohesive soils reveal that the fully softened shear strength is stress-dependent and related to the type of clay mineral and quantity of clay-size particles. An empirical relationship for the fully softened friction angle is presented that is a function of liquid limit, clay-size fraction, and effective normal stress. Studies of first-time slides, i.e., slopes that have not undergone previous sliding, in stiff fissured clay with a liquid limit between 41 and 130% suggest that the mobilized shear strength along the failure surface can be lower than the fully softened shear strength. Recommendations are presented for estimating the mobilized shear strength in first-time slides based on soil plasticity. Soils with a liquid limit greater than 30% exhibit a large difference between the fully softened and residual friction angles. In these soils, the presence or absence of a pre-existing shear surface should be clarified.

INTRODUCTION

Investigations by Skempton et al. (1969) and Skempton (1977) indicate that the development of a fissure can result in softening of surrounding overconsolidated clay. The softening corresponds to an increase in water content caused by soil dilation under the imposed shear stress. Softening reduces the effective stress cohesion component of the Mohr-Coulomb shear strength parameters but does not cause orientation of clay particles or reduction in friction angle (Skempton 1970). Consequently, Skempton (1977) suggests that the shear strength available in a fissure corresponds to the fully softened condition. Skempton (1970) concluded that the fully softened shear strength is numerically equal to the drained peak strength of a normally consolidated specimen.

After studying case histories involving slopes in brown London clay, Skempton (1977) proposed that slopes that have not undergone previous sliding (first-time slides) could be designed using a fully softened shear strength. Based on a number of case histories, Skempton (1964, 1977) also suggested that slopes that have undergone 1–2 m of shear displacement should be designed using the drained residual shear strength. Therefore, the drained residual shear strength pertains to slopes that contain a pre-existing shear surface, such as old landslides or soliflucted slopes, bedding shears in folded strata, sheared joints, or faults. The objectives of the present study are to investigate the effect of clay mineralogy, clay-size fraction, and effective normal stress on the fully softened shear strength of cohesive soils, and to propose a design procedure for stiff fissured clay slopes that have not undergone previous sliding.

SOIL DESCRIPTION AND TEST PROCEDURE

Most heavily overconsolidated clays, mudstones, or shales possess diagenetic bonding that results in aggregates of individual clay particles. The degree of mudstone or shale aggregation that survives a particular sample preparation procedure has an important influence on the measured index properties, such as liquid limit and clay-size fraction (La Gatta 1970; Townsend and Banks 1974). Since the liquid limit and clay-size fraction are used to infer clay mineralogy and quantity of particles smaller than 0.002 mm, respectively, the mudstone or shale particles were disaggregated by ball-milling a representative air-dried sample until all particles passed U.S. Standard Sieve No. 200 (Mesri and Cepeda-Diaz 1986). Remolded silt and clay specimens (soil numbers: 1, 2, 4, and 11 in Table 1) were obtained by air-drying a representative sample, crushing it with a mortar and pestle, and processing it through U.S. Standard Sieve No. 40. Ball-milling was not used for these specimens because they naturally have disaggregated particles, and it would have changed the gradation of the soil. In both cases, distilled water was added to the processed soil until a liquidity index of about 1.5 was obtained. The sample was then allowed to rehydrate for at least one week in a moist room.

A modified Bromhead ring shear apparatus (Stark and Eid 1993) was used for testing the 24 soils listed in Table 1. The original ring shear apparatus is described by Bromhead (1979). The rehydrated remolded soil paste was placed into the annular specimen container using a spatula. The top of the specimen was planed flush with the top of the specimen container using a surgical razor blade. It should be noted that the liquid limit, plastic limit, and clay-size fraction of the specimens were measured using the ball-milled mudstone and shale, or the sieved silt and clay samples.

For each soil, three remolded normally consolidated specimens were sheared at effective normal stresses of 50, 100, and 400 kPa in the modified ring shear apparatus. This range of normal stress was chosen to represent the normal stresses that are typically encountered in first-time failures of slopes and embankments. A shear displacement rate of 0.018 mm/min was used to provide a drained condition during shear. This displacement rate is based on values of the coefficient of consolidation evaluated from oedometer tests. The procedure described by Gibson and Henkel (1954), and a degree of consolidation of 99.5% was used to estimate the drained displacement rate for the high-plasticity shales. Faster displacement rates could have been used for some of the clays and silts. However, to avoid possible rate effects, a displacement rate of 0.018 mm/min was used for all 24 soils.

The fully softened shear strength was measured after the remolded specimen was consolidated to the desired normal stress. Consolidation of specimens to a normal stress of 400 kPa was performed in two stages. The specimen was first consolidated to a normal stress of 200 kPa. Settlement of the loading platen, which causes wall friction during shear, was eliminated by lowering the outer ring and inner core of the specimen container that surround the annular specimen (Stark and Eid 1993). The specimen was then consolidated to a normal stress of 400 kPa. This two-stage loading to 400 kPa minimizes the potential for overconsolidating the specimen during the unloading process required to lower the outer ring and inner core. For normal stresses of 50 and 100 kPa, settlement

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TABLE 1. Clay and Shale Samples Used in Ring Shear Tests

Soil number (1)	Clay and shale samples (2)	Clay and shale locations (3)	Initial water content (%) (4)	Specific unit weight (kN/m³) (5)	Liquid limit (%) (6)	Plastic limit (%) (7)	Clay-size fraction (%) (8)	Activity (plasticity index/ clay-size fraction) (9)
1	Glacial till	Urbana, Illinois	8.4	16.1	24	16	18	0.44
2	Loess	Vicksburg, Mississippi	14.0	16.5	28	18	10	1.00
3	Duck Creek shale*	Fulton, Illinois	5.3	24.0	37	25	19	0.63
4	Slide debris	San Francisco, California	18.0	19.7	37	26	28	0.39
5	Slope-wash material	San Luis Dam, California	25.0	19.6	42	24	34	0.53
6	Colorado shale*	Montana, Montana	5.6	21.2	46	25	73	0.29
7	Panoche mudstone	San Francisco, California	14.2	19.6	47	27	41	0.49
8	Panoche shale	San Francisco, California	12.0	20.2	53	29	50	0.48
9	Comanche shale*	Proctor Dam, Texas	11.5	23.1	62	32	68	0.44
10	Breccia material	Manta, Ecuador	23.0	19.1	64	41	25	0.92
11	Bay mud	San Francisco, California	73.0	15.0	76	41	16	2.19
12	Patapsco shale*	Washington, D.C.	21.6	20.7	77	25	59	0.88
13	Pierre shale*	Limon, Colorado	24.3	20.1	82	30	42	1.24
14	Lower Pepper shale	Waco Dam, Texas	21.0	20.3	94	26	77	0.88
15	Brown London clay	Bradwell, England	33.0	18.9	101	35	66	1.00
16	Cucaracha shale*	Panama Canal	18.4	20.7	111	42	63	1.10
17	Denver shale*	Denver, Colorado	30.5	18.7	121	37	67	1.25
18	Bearpaw shale*	Saskatchewan, Canada	27.3	19.0	128	27	43	2.35
19	Oahe Firm shale	Oahe Dam, South Dakota	27.6	20.1	138	41	78	1.24
20	Taylor shale*	San Antonio, Texas	35.2	18.0	170	39	72	1.82
21	Pierre shale*	Reliance, South Dakota	42.8	17.7	184	55	84	1.54
22	Oahe bentonitic shale	Oahe Dam, South Dakota	35.4	18.9	192	47	65	2.23
23	Lea Park Bentonitic shale	Saskatchewan, Canada	36.0	17.3	253	48	65	3.15
24	Bearpaw shale*	Ft. Peck Dam, Montana	15.8	21.8	288	44	88	2.77

*Index properties from Mesri and Cepeda-Diaz (1986).

of the loading platen caused by consolidation was small. Therefore, the specimen did not require unloading and lowering of the outer ring and inner core. Thus, these specimens are also normally consolidated prior to shearing.

After consolidation to the desired normal stress, each specimen was sheared until the peak (fully softened) shear strength was mobilized. The ring shear test was usually stopped after a maximum shear displacement of 10 mm. Only one fully softened shear strength was measured for each specimen.

EFFECT OF MODE OF SHEAR ON FULLY SOFTENED SHEAR STRENGTH

Four drained direct shear and 10 isotropically consolidated-drained triaxial compression tests were performed to compare the fully softened shear strengths measured using these apparatuses with the ring shear test results. The Gibson and Henkel (1954) procedure and values of coefficient of consolidation evaluated from oedometer tests and during the last stage of consolidation in the triaxial compression tests were used to determine drained shear displacement rates. Direct shear tests were carried out on remolded specimens of Panoche and Lower Pepper shales (Table 1). Each shale was sheared at effective normal stresses of 100 and 400 kPa using a shear displacement rate of 0.001 mm/min. Remolded specimens were obtained using the ring shear specimen preparation procedure previously described. A spatula was used to place the remolded soil paste into the 60 by 60 mm shear box until a specimen thickness of 30 mm was obtained. Prior to shear, the specimen was consolidated to the desired effective normal stress resulting in a normally consolidated specimen. Then, the specimen was sheared until the peak (fully softened) shear strength was mobilized. The direct shear tests were stopped after completion of one travel of the shear box or a shear displacement of approximately 6 mm.

Fig. 1 shows the stress ratio-displacement relationships for Panoche shale from both ring shear and direct shear apparatuses. The stress ratio is defined as the shear stress, τ, divided by the effective normal stress, σ'_n. It can be seen that the difference in peak or the fully softened stress ratio measured by ring shear and direct shear apparatuses is small. In fact, the stress ratio difference corresponds to a difference in fully softened friction angle of less than 1°. A similar conclusion was reached using Lower Pepper shale (Eid 1996). This is in agreement with the analysis and test results presented by Hvorslev (1939). Hvorslev found that for ratios of the inner to outer ring radii greater than 0.5, the ring shear peak shear stress is approximately equal to direct shear values. The ring shear apparatus used during this study possesses a radius ratio of 0.7. It should be noted that the data in Fig. 1 also show that the fully softened strength is stress-dependent.

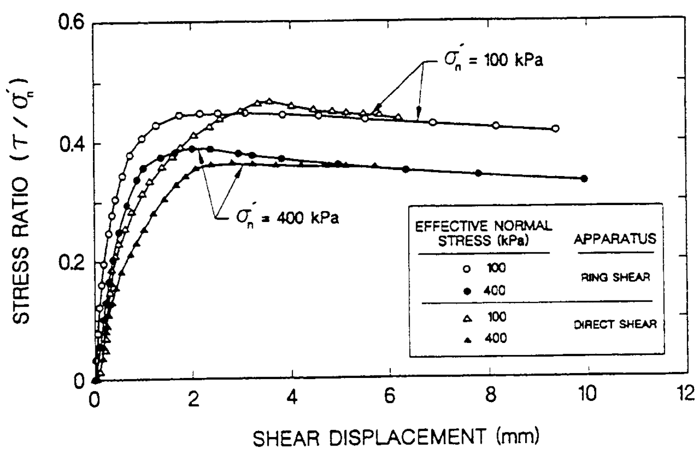

FIG. 1. Stress Ratio-Displacement Relationships for Panoche Shale

Isotropically consolidated-drained triaxial compression tests were performed on remolded specimens of glacial till, Panoche shale, Lower Pepper shale, Oahe Firm shale, and Oahe Bentonitic shale (Table 1) using axial displacement rates of 0.01, 0.001, 0.001, 0.0005, and 0.0002 mm/min, respectively. Remolded samples were obtained using the ring shear specimen preparation except a rehydration water content corresponding to liquidity indices of 1.0, 0.9, 0.87, 0.71, and 0.62 were used for the glacial till, Panoche shale, Lower Pepper shale, Oahe Firm shale, and Oahe Bentonitic shale, respectively. For each soil two triaxial tests were conducted at consolidation stresses (σ_3') of 70 and 275 kPa. Fig. 2 presents the stress-strain relationships for eight of these tests where σ_1 and σ_3 are the major and minor principal stresses. For specimens that exhibited a bulging failure, the shear stress corresponding to 20% axial strain was taken as the fully softened value. For specimens that exhibited a well defined failure plane, the failure criterion of maximum stress difference was used to estimate the fully softened shear strength. Fig. 2 also shows that the fully softened friction angle is stress-dependent.

The time required to consolidate the specimen to the desired effective stress as well as the shearing time required to reach the maximum shear stress differed significantly for these apparatuses. For example, a normally consolidated remolded specimen of Lower Pepper shale required a shearing time of approximately 1.5 h, 3 d, and 7 d, using the ring shear, direct shear, and triaxial shear apparatuses. The relatively short shearing time required with the ring shear apparatus is attributed to a thinner specimen, which allows a faster dissipation of excess pore-water pressure. This reduces the consolidation time and allows a faster drained shear displacement rate. The difference in the time required to measure the fully softened shear strength increased with increasing plasticity of the tested soil.

Fig. 3 presents a comparison of the fully softened friction angle (ϕ') obtained from drained ring shear and drained triaxial compression tests. A nonlinear failure envelope was obtained from the triaxial compression test results for each soil. From this nonlinear envelope, a secant fully softened friction angle $(\phi')_{tri}$ was estimated at effective normal stresses of 100 and 400 kPa as shown in Fig. 3. The ring shear apparatus yields a fully softened friction angle $(\phi')_{ring}$ that is approximately 2.5° less than that obtained from a drained triaxial compression test.

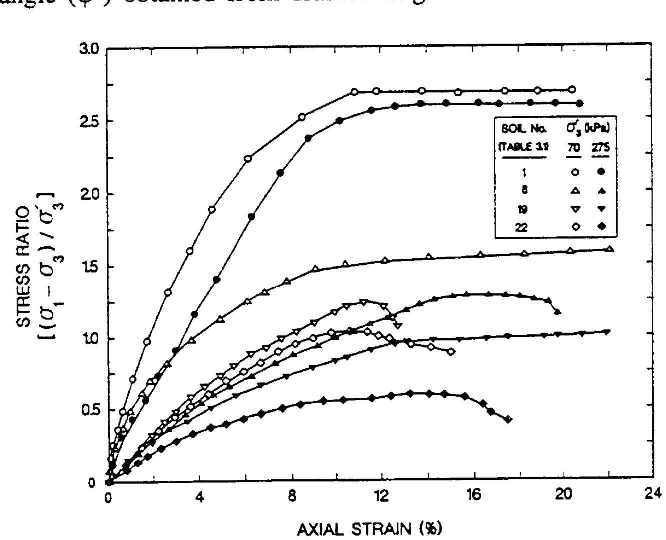

FIG. 2. Stress-Strain Relationships from Drained Triaxial Compression Tests

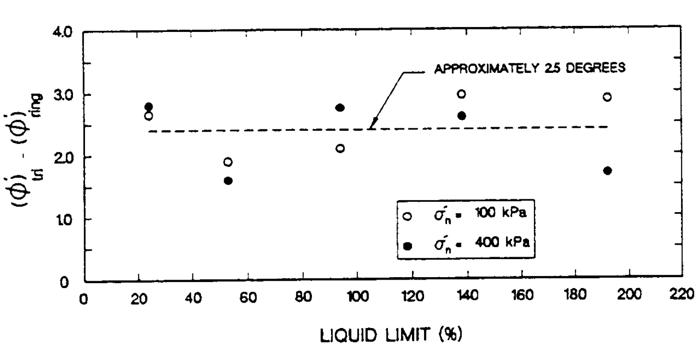

FIG. 3. Comparison of Fully Softened Friction Angles Obtained from Drained Ring Shear and Triaxial Compression Tests

A literature survey reveals few comparisons between the fully softened shear strength measured using different laboratory apparatuses or modes of shear. Taylor (1939), Nash (1953), and Rowe (1969) present results obtained using direct shear and triaxial compression apparatuses for sands with different relative densities. For sands in a loose or contractive state, the direct shear test yields effective stress friction angles that are 1°–4° less than those measured in the triaxial compression apparatus. In a dense or dilative state, the direct shear apparatus yields friction angles that are 2°–5° greater than those measured in the triaxial device. At medium densities, the results from these two apparatuses are similar. Tests by Schultze and Horn (1965) on remolded silt show that the direct shear test yields a friction angle 3°–4° lower than that obtained from a triaxial apparatus.

For clays the literature offers little information on the comparison of the fully softened shear strength measured using direct shear and triaxial apparatuses. This may be attributed to the small range of normal stresses used in comparisons (Casagrande and Hirschfeld 1960; Casagrande and Poulos 1964), expressing the measured shear strength in terms of effective stress cohesion and friction angle without specifying the effective normal stress, problems associated with controlling the molding water content and unit weight (Peterson et al. 1960; Pells et al. 1973), and using consolidated-undrained triaxial compression tests with pore-water pressure measurements and the maximum stress difference failure criterion, which yields a shear resistance that is 8% less than that obtained from drained triaxial compression tests on undisturbed clay specimens (Bjerrum and Simons 1960; Simons 1963).

In summary, the available literature and Fig. 3 indicate that for soil specimens in a contractive state, such as loose sand and normally consolidated clay and silt, the drained triaxial compression test yields a fully softened friction angle that is approximately 2.5° greater than that obtained using a ring shear or direct shear apparatus. This is attributed to the difference in mode of shear and stress state. Since the stress state in a triaxial compression test is closer to the field mode of shear in the development of first time slides, the ring shear fully softened friction angles are increased by 2.5° in subsequent sections of this paper.

FULLY SOFTENED SHEAR STRENGTH RELATIONSHIP

Fig. 4 presents a relationship between the secant fully softened friction angle and soil index properties based on ring shear tests conducted at effective normal stresses of 50, 100, and 400 kPa. The values of ϕ' in Fig. 4 have been increased by 2.5° to represent the triaxial compression mode of shear. It can be seen that the relationship is divided into three clay-size fraction groups and is a function of the liquid limit. Fig. 4 shows that the secant fully softened friction angle decreases with an increasing liquid limit and increasing clay-size fraction. The secant fully softened friction angle corresponds to a linear failure envelope passing through the origin and the fully softened shear strength at a particular effective normal stress. The liquid limit and clay-size fraction provide an indication of particle shape and particle size, respectively. In general, the plasticity increases as the platyness of the clay particles increases. Increasing the platyness of the clay particles results

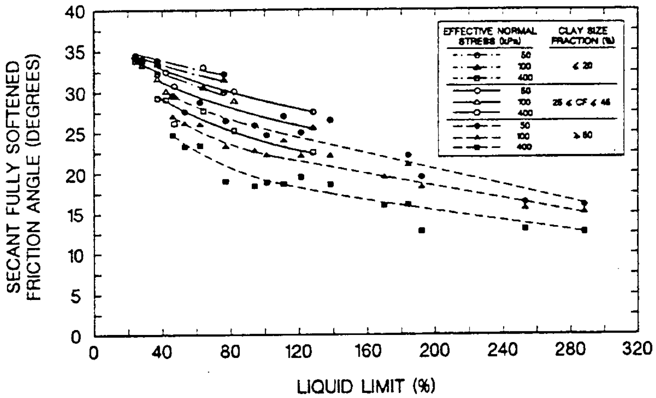

FIG. 4. Relationship between Drained Fully Softened Friction Angles and Liquid Limit for Triaxial Compression Mode of Shear

in a greater tendency for face-to-face interaction, and thus a lower fully softened shear strength.

The proposed fully softened relationship differs from existing correlations of ϕ', e.g., Bjerrum and Simons (1960), Terzaghi and Peck (1967), NAVFAC (1971), and Mesri and Abdel-Ghaffar (1993), because in the proposed relationship the drained fully softened friction angle is a function of the liquid limit, clay-size fraction, and effective normal stress. In correlations presented by Bjerrum and Simons (1960), NAVFAC (1971), and Mesri and Abdel-Ghaffar (1993), there is considerable scatter in ϕ' for the range of plasticity index of 10–100%. Ladd et al. (1977) referred to this scatter by stating that "the scatter in ϕ' 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." It is anticipated that this scatter is caused by omitting the effect of the clay-size fraction and effective normal stress on the fully softened friction angle. For example, at a liquid limit of 120%, the fully softened friction angle ranges from 18°–28° (Fig. 4) for a clay-size fraction greater than or equal to 25%. This variation is explained when the data are separated by the clay-size fraction and effective normal stress as shown in Fig. 4.

Fig. 4 also illustrates the nonlinearity of the drained fully softened failure envelope in terms of the decrease in secant fully softened friction angle with increasing effective normal stress. The nonlinearity is significant for cohesive soils with a clay-size fraction greater than 25%. For example, at a liquid limit of 100% and a clay-size fraction greater than 50%, the secant fully softened friction angle decreases from approximately 25° at an effective normal stress of 50 kPa to about 19° at an effective normal stress of 400 kPa. For clay-size fractions less than 20% and liquid limits less than 80%, the reduction in the secant fully softened friction angle from a normal stress of 50–400 kPa is less than 2°–3°. However, for all clay-size fraction groups, Fig. 4 shows that the rate of reduction in secant fully softened friction angle decreases with increasing the effective normal stress. Therefore, the nonlinearity of the drained fully softened failure envelope is most noticeable at effective normal stress less than approximately 200 kPa.

It is anticipated that the reduction in the fully softened friction angle with increasing effective normal stress is caused by an increase in face-to-face interaction among the plate-shaped particles. However, for high-plasticity soils, the nonlinearity decreases slightly with increasing liquid limit because the flexible platy particles of these soils establish considerable face-to-face interaction by orientation and bending under low, as well as high, effective normal stresses.

The secant fully softened friction angle for a cohesive soil can be estimated for a particular effective normal stress using the liquid limit, clay-size fraction, and interpolation between the curves presented in Fig. 4. Fig. 4 can also be used to estimate the nonlinear fully softened failure envelope by plotting the shear stress corresponding to the drained fully softened friction angle at effective normal stresses of 50, 100, and 400 kPa. A smooth curve can be drawn through the three points and the origin to estimate the failure envelope.

COMPARISON OF FULLY SOFTENED SHEAR STRENGTH RELATIONSHIPS

Fig. 5 shows fully softened friction angles assembled from existing literature for a wide range of soft and stiff clay compositions superimposed on the empirical relationship presented in Fig. 4. Most of these data are based on triaxial compression tests conducted using remolded or undisturbed normally consolidated specimens. Unfortunately, the clay-size fraction was not reported in some references, which limited the number of data points to those shown in Fig. 5. Also, one average fully softened friction angle (corresponding to a linear failure envelope) was usually reported for each soil. It can be seen from Fig. 5 that these external data support the proposed grouping of fully softened friction angle according to liquid limit and clay-size fraction. The scatter of the published data shown in Fig. 5 can be attributed to differences in effective normal stress range, number of test points used to define the failure envelope, test apparatus and procedure, and sample preparation, which especially affects the measured index properties of shales and mudstones (La Gatta 1970; Townsend and Banks 1974).

To illustrate the importance of using the liquid limit, clay-size fraction, and effective normal stress in estimating the fully softened friction angle existing correlations (Bjerrum and Simons 1960; Terzaghi and Peck 1967; NAVFAC 1971; Mesri and Abdel-Ghaffar 1993), the proposed relationship and the fully softened friction angles measured for the 24 soils tested during the present study corrected to the triaxial compression mode of shear are presented in Fig. 6. In Fig. 6, the plasticity index is used to plot the ring shear test data because existing correlations use the plasticity index, instead of the liquid limit, to estimate the fully softened friction angle.

Fig. 6 shows that existing correlations can be represented by one trend lineup to a plasticity index of 100%. This trend line is in agreement with the median of the ring shear data, and thus the proposed relationship considering the different clay-size fractions and effective normal stresses. For example, at a plasticity index of 60%, the estimated fully softened friction angle using the proposed relationship ranges from 20° to 29°. The trend line from existing correlations yields an estimate of ϕ' equal to 25°. However, the trend line overestimates the fully softened friction angles at effective normal stresses greater than 100 kPa for soils with a clay-size fraction greater

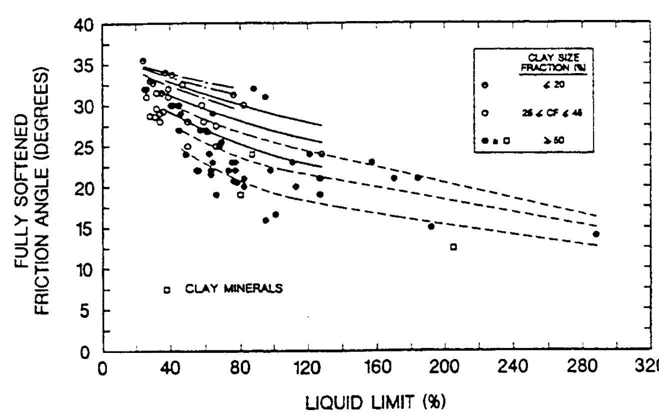

FIG. 5. Comparison of Published Fully Softened Friction Angles and Proposed Relationship

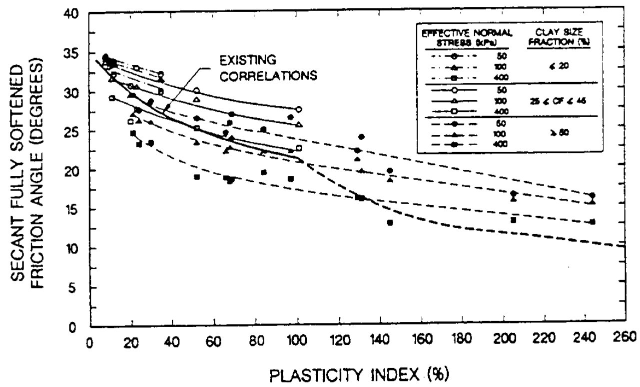

FIG. 6. Relationship between Fully Softened Friction Angle and Plasticity Index

than 50%. This is caused by existing correlations ignoring the importance of the clay-size fraction.

The fully softened friction angle correlation presented by Mesri and Abdel-Ghaffar (1993) extends the fully softened friction angle trend line for soils that exhibit a plasticity index greater than 100%. However, a smooth line cannot be drawn between the correlation for low plasticity index soils (<100%) and this extension. This is attributed to two reasons: (1) soils with a plasticity index greater than 100% usually exhibit a clay-size fraction greater than 50%; and (2) the extension represents fully softened friction angles measured using triaxial compression tests with high confining pressures (up to 550 kPa), and consequently high effective normal stresses (Mesri 1969).

LONG-TERM STABILITY OF STIFF FISSURED CLAYS

Fourteen field case histories involving first-time slides (Table 2) were located to study the mobilized shear strength of stiff fissured clay. These case histories were selected because no previous shear displacement or landsliding were reported for the slope, and the shear surface and piezometric, or ground-water level were carefully determined. Except for case histories numbers 3, 4, 7, 9, and 12, the mobilized cohesion (c_{mob}) and friction angle (ϕ'_{mob}) during failure were backcalculated by Abdel-Ghaffar (1990) using Spencer's (1967) stability method as coded in UTEXAS2 (Wright 1986). For case history number 3, a reduction in shear strength was applied to allow for the shear resistance developed along the sides of the actual three-dimensional slide mass using a length-to-depth ratio of 5.0 (Baligh and Azzouz 1975). No reduction in shear strength is taken for the rest of the cases because either the sliding mass length-to-depth ratio is greater than 10 and consequently the reduction is negligible, or the failure occurred along the whole length of the slope such as in case history number 11. The average effective normal stress on the slip surface of each case history was calculated using reported values of unit weight and the reported location of the ground-water surface.

For case histories in which a nonlinear fully softened or residual shear strength failure envelope were reported, a secant friction angle that corresponds to the average effective normal stress acting on the slip surface was used to estimate the fully softened and residual friction angles. For case histories in which the fully softened or residual strength was not reported, the liquid limit, clay-size fraction, and average effective normal stress acting on the slip surface were used to estimate ϕ'_r (Stark and Eid 1994) and ϕ' (Fig. 4). In case histories numbers 5 and 8, the clay-size fraction was not reported. Stiff fissured clays of these two cases exhibit relatively high plasticity (liquid limit = 25–84% with an average of 58%, and liquid limit = 47–91% with an average of 72% for cases 5 and 8). This range of plasticity is usually associated with a clay-size fraction higher than 25%. Accordingly, the fully soft-

TABLE 2. Case Histories Involving First-Time Slides in Stiff Fissured Clay

Number (1)	Case history (2)	Liquid limit (%) (3)	Plastic limit (%) (4)	Clay-size fraction (%) (5)	Average effective normal stress acting on slip surface (kPa) (6)	Mobilized cohesion intercept (kPa) (7)	Mobilized friction angle (8)	Estimated fully softened friction angle (9)	Estimated residual friction angle (10)	Reference (11)
1	Slope failure in China	41	21	41	116	11.81[a]	38.3°[a]	31.0°[b]	24.0°[b]	Li and Zhao (1984)
2a	Slope failure in Sri-Lanka	47	23	15	79	8.14[a]	31.0°[a]	33.5°[b]	25.5°[b]	Balasubramaniam et al. (1977)
2b	Slope failure in Sri-Lanka	47	23	15	25	2.26[a]	31.0°[a]	34.5°[b]	26.0°[b]	Balasubramaniam et al. (1977)
3	River Albedosa slide	52	26	40	99	0.00	21.7°	29.5°	13.0°	Cancelli (1981)
4	Marly clay failure	58	20	96	62	0.00	20.0°	25.0°	16.0°	Sotiropoulos and Cavounidis (1980); Cavounidis and Sotiropoulos (1980)
5	Shellmouth test fill	58	21	>25%[c]	69	0.00[a]	22.0°[a]	(27.5–31.0)°[b]	(16.5–20.5)°[b]	Rivard and Kohuska (1965); Rivard and Lu (1978)
6	Amuay slides	63	23	98	45	0.00[a]	18.5°[a]	27.5°	12.5°	Lambe et al. (1981); Lambe (1985)
7	Lias clay failures	64	28	52	30	1.00	23.0°	31.9°	14.5°	Chandler (1974); Chandler and Skempton (1974); Skempton (1985a)
8	North Ridge Dam	72	21	>25%[c]	64	0.00[a]	22.6°[a]	(26.5–30.0)°[b]	(14.0–18.0)°[b]	Peterson et al. (1957); Rivard and Lu (1978)
9	Carsington Dam	75	32	62	235	0.00	16.0°	20.0°	12.0°	Skempton (1985b); Skempton and Vaughan (1993)
10	Santa Barbara coal mine	80	45	30	577	0.00[a]	17.0°[a]	24.0°[b]	11.0°	Esu et al. (1984)
11	Otford test embankment	85	31	50	100	0.00[a]	18.1°[a]	23.0°[b]	11.5°[b]	Simons (1976)
12	London clay failures	87	35	52	35	1.00	20.0°	25.4°	14.2°	Chandler and Skempton (1974); Skempton (1977); Skempton (1985a)
13	Seven sisters dikes	97	30	75	48	0.95[a]	16.0°[a]	25.0°[b]	10.5°[b]	Peterson et al. (1957); Rivard and Lu (1978)
14	Slope failure at Wettern	130	42	50	76	0.00[a]	13.2°[a]	22.0°[b]	9.0°[b]	Marivoet (1948)

[a]After Abdel-Ghaffar (1990).
[b]Predicted using liquid limit, clay-size fraction, and σ'_n.
[c]Estimated.

ened and residual friction angles were estimated using both the intermediate (25% < clay-size fraction < 45%) and high (clay-size fraction > 50%) clay-size fraction groups.

Table 2 shows that most of the cases exhibit a mobilized cohesion of approximately zero and a mobilized friction angle that is lower than that predicted from the proposed fully softened relationship in Fig. 4 using the reported liquid limit, clay-size fraction, and the average effective normal stress acting on the slip surface. The empirical correlations (Fig. 6) based on the plasticity index also yield friction angles that are larger than the backcalculated values. This suggests that even for the first-time slides, a shear strength less than the fully softened shear strength may be mobilized. The same conclusion was previously reached by Mesri and Abdel-Ghaffar (1993).

Fig. 7 presents a comparison of the fully softened, residual, and mobilized stress ratios for the case histories in Table 2. It can be seen that first-time slides in stiff fissured clays with a liquid limit greater than 50% mobilize a shear strength that is less than the fully softened value. The scatter around the fully softened and residual stress ratio trend lines may be attributed to the difference in clay-size fraction and the average effective normal stress on the failure surface for the case histories.

Skempton (1977) suggested that for a first-time slide, the shear strength of a stiff fissured clay at failure can be approximated by the fully softened strength. A similar conclusion is made by Chandler (1984a) for case histories involving Upper Lias clay and by Cancelli (1981) for a first-time slide through Lugagnano clay along the Albedosa River. However, Chandler (1984a) questioned this close agreement between the field shear strength and laboratory fully softened strength for London and Upper Lias clays by stating that "since both clays are brittle, and hence susceptible to progressive failure, this agreement is surprising and perhaps fortuitous." Chandler (1984b) also comments on this agreement by stating that "the reason for the close agreement of the field strength with the fully softened strength is not fully understood, particularly since the majority of the field evidence seems to support progressive failure rather than softening."

Fig. 8 shows that for the liquid limit range of 50–130%, the mobilized shear strength (τ_{mob}) can be as low as the average between the fully softened strength (τ_s) and residual shear strength (τ_r). This suggests that during global instability the mobilized shear strength for a first-time slide through stiff fissured clay may be as low as the average between the fully softened and residual shear strengths. A possible explanation for this result is that softening, which is a reduction in the available drained strength resulting from an increase in water content under constant effective stress, reduces the shear strength of the mass to the fully softened value. After reaching the fully softened shear strength, progressive failure reduces the average shear strength along the failure surface to a value between the fully softened and residual shear strength values.

Fig. 9 shows that it takes less than 15 mm of displacement to reduce the clay shear strength from the fully softened value to a value that is half-way between the fully softened and residual strengths. The point that represents the average between the fully softened and residual strengths approximately separates the steep and flat portions of the stress ratio-displacement curve (Fig. 9). This may explain the mobilized shear strength being close to the average between the fully softened and residual strength values. Softening and progressive failure are not pronounced for low-plasticity stiff clays (Bjerrum 1967). Therefore the mobilized shear strength can be greater than or equal to the fully softened value (Fig. 7). Mesri and Abdel-Ghaffar (1993) also showed that for stiff clays with plasticity indices less than 20%, the mobilized shear strength can be greater than the fully softened shear strength. In addition, they concluded that the mobilized shear strength in these low-plasticity stiff clays can be equal to the intact peak strength.

It is anticipated that the agreement between the field shear strength and the laboratory fully softened strength may be attributable to ignoring the nonlinearity of the fully softened shear strength envelope and/or the lack of laboratory shear strength data in the range of the effective normal stresses acting on the slip surface of the studied case histories. This was observed for the fully softened failure envelopes obtained from

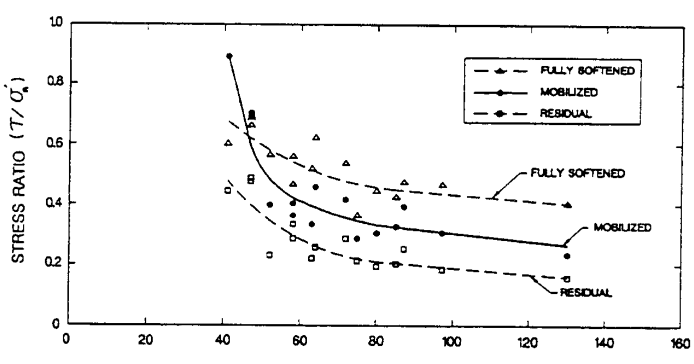

FIG. 7. Mobilized, Fully Softened, and Residual Stress Ratios for Field Case Histories

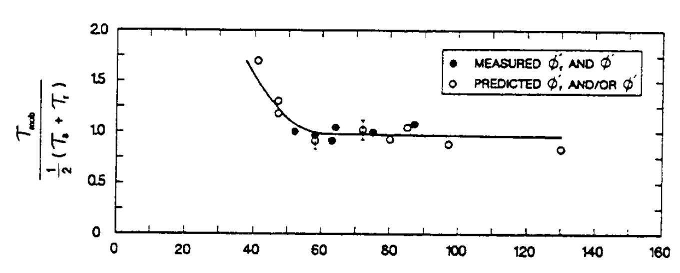

FIG. 8. Comparison of Mobilized, Fully Softened, and Residual Shear Strengths for Stiff Fissured Clay

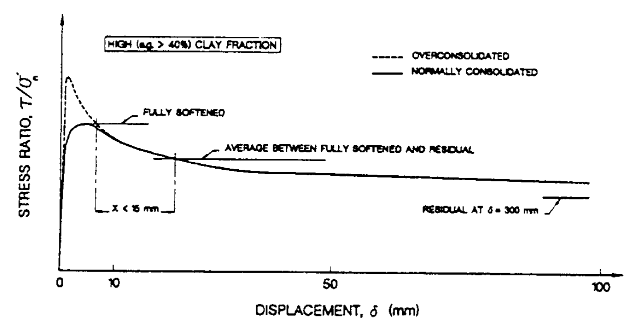

FIG. 9. Diagrammatic Stress-Displacement Curves at Constant σ_n' [after Skempton (1985a)]

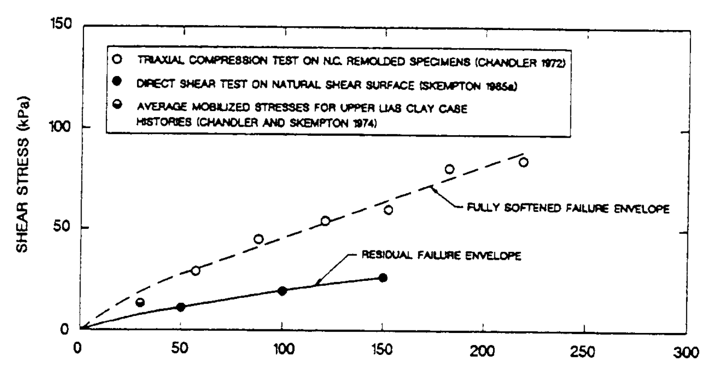

FIG. 10. Drained Shear Strength Failure Envelopes for Upper Lias Clay

laboratory test results on London clay (Skempton 1977), Upper Lias clay (Chandler 1972), and Lugagnano clay (Cancelli 1981). For example, Fig. 10 shows that the lowest effective normal stress used in the laboratory tests to estimate the fully softened strength of Upper Lias clay is approximately 60 kPa, and the average effective normal stress in the Upper Lias clay case histories is about 30 kPa. Fig. 10 also shows that the fully softened and residual failure envelopes are nonlinear, and the average between the fully softened and residual shear strengths yields a reasonable estimate of the mobilized shear strength. Similar figures can be drawn for London clay, River Albedosa, and the Carsington Dam case histories (Eid 1996).

After reviewing studies conducted by Potts et al. (1990, unpublished report; 1994) on the Carsington Dam slide and slides in highway cuttings and embankments, Skempton and Vaughan (1995) stated that "residual strength can indeed play a part in first-time slides in clay fills and cuttings." This statement appears to support the conclusion presented herein that the residual shear strength can influence the stability or behavior of slopes that have not undergone prior sliding.

In summary, the mobilized shear strength in first-time slides may be less than the fully softened strength. A study of 14 field case histories suggests that the mobilized shear strength can be as low as the average between the fully softened and residual shear strengths in first-time landslides through stiff fissured clay with a liquid limit between 50 and 130%. Both the fully softened and residual friction angles should correspond to the average effective normal stress acting on the critical slip surface. It should be noted again that this conclusion is based on only 14 case histories. Additional case histories should be located and the effect of other geological factors, such as fissure spacing and bedding, should be studied to confirm this statement.

Location of Critical Slip Surface in Stiff Fissured Clay Slopes

The critical slip surface through a slope that has not undergone previous sliding should be located to estimate the average effective stress acting on the slip surface and the minimum factor of safety against failure. The concept of progressive failure assumes that at the initiation of failure, the peak shear strength of the soil mass (mass strength) is mobilized in an overstressed zone. Therefore, the path of least resistance, i.e., the potential slip surface, should be located using the peak shear strength of the soil mass. As failure progresses along this surface, it becomes a surface of weakness along which the average available shear strength decreases below the peak shear strength of the soil mass. Therefore, it can be concluded that the peak shear strength of the soil mass should be used to determine the location of the critical slip surface in slopes that have not undergone previous sliding (Abdel-Ghaffar 1990; Mesri and Abdel-Ghaffar 1993). The critical slip surface in slopes that have undergone previous sliding is usually well defined, and thus does not need to be located.

After shearing triaxial specimens with different diameters of Oxford clay (liquid limit = 56%, plasticity index = 34%, and clay-size fraction = 56%), Petley (1984) concluded that in stiff fissured clays, the peak shear strength of the soil mass is controlled by the shear strength along fissures. For clays that have undergone softening, the shear strength available in a fissure corresponds to the fully softened condition. This was concluded based on testing of brown London clay and Oxford clay by Skempton (1977) and Petley (1984), respectively. Therefore, the fully softened strength and shear strength of the soil mass are similar for stiff fissured clays with high plasticity (liquid limit > 50%). This was primarily observed for case histories involving shallow slip surfaces because the stiff fissured clay was able to undergo weathering or softening. As a result, the fully softened shear strength can be used to determine the location of the critical slip surface through high-plasticity stiff fissured clays. The nonlinearity of the fully softened failure envelope should be incorporated in locating the critical slip surface. The fully softened shear strength usually yields a shallower critical slip surface and consequently a lower factor of safety than the intact peak shear strength of soil. This may be suitable for locating the critical slip surface, which is sensitive to site conditions such as piezometric head level. In cases where the softening process is not pronounced, such as first-time slides through low-plasticity stiff clays, the intact peak shear strength may be used to determine the critical slip surface location.

NUMERICAL DIFFERENCE BETWEEN FULLY SOFTENED AND RESIDUAL FRICTION ANGLES

Fig. 11 presents the numerical difference between the residual and fully softened friction angles for 24 soils shown in Table 1. The difference between the fully softened and residual friction angles is maximized for soils of intermediate plasticity. In these soils a large displacement is required to convert initial edge-to-face particle interactions to face-to-face interactions and establish a residual strength condition. This orientation leads to a low residual friction angle, and thus a large difference between ϕ' and ϕ'_r. For example, at a liquid limit of 130%, the difference between ϕ' and ϕ'_r is approximately 16° and 12° for effective normal stresses of 50 and 400 kPa, respectively.

For low-plasticity soils the relatively rotund or bulky particles and/or stiff clay plates result in large values of ϕ' and ϕ'_r. Consequently, the difference between ϕ' and ϕ'_r is small. This is caused by the rotund particles and/or stiff clay plates establishing edge-to-face interaction during shear in the fully softened and residual cases. For high-plasticity soils the flexible and platy particles of these soils can establish face-to-face interaction at low and high normal stresses in a remolded specimen without large shear displacement. This mechanism leads to a smaller difference between ϕ' and ϕ'_r than that of the intermediate plasticity soils.

Fig. 11 also shows that the difference between ϕ' and ϕ'_r depends on the effective normal stress at which the two secant friction angles are estimated. Increasing the effective normal stress decreases the difference between ϕ' and ϕ'_r. This suggests that the fully softened failure envelope exhibits a larger nonlinearity than the drained residual failure envelope. This is caused by the effective normal stress being the only factor that affects initial particle orientation in the fully softened strength condition, whereas both effective normal stress and large shear displacement affect particle orientation in the drained residual strength condition. The difference in ϕ' and ϕ'_r can be esti-

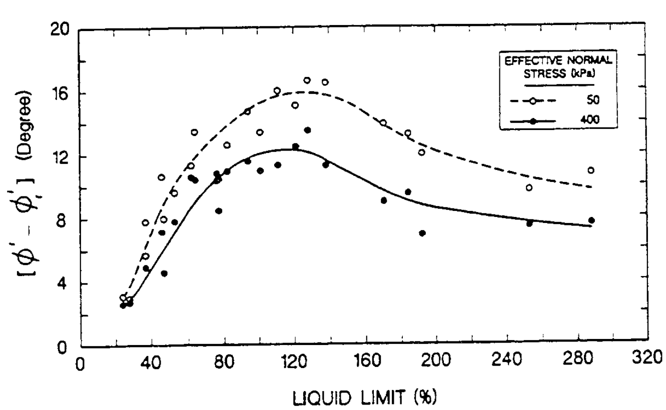

FIG. 11. Difference between Secant Fully Softened and Residual Friction Angles

mated for other effective normal stresses by interpolation between the relationships shown in Fig. 11.

Fig. 11 can be used to evaluate the practical significance of determining whether or not a pre-existing shear surface is present in a slope. Clearly, natural soils with a liquid limit greater than 30% exhibit a large difference (up to 16°) between the fully softened and residual friction angles. In these soils it is important to determine whether the slope has undergone previous sliding, and thus contains a pre-existing shear surface. If a pre-existing shear surface is located, a residual friction angle should be used for slope design, which will significantly impact the design. The difference in ϕ' and ϕ'_r will be greater for shallow failure surfaces, i.e., lower effective normal stresses.

In soils with a liquid limit less the 30%, there is a small difference (less than 3°) between the fully softened and residual friction angles. Therefore, the difference between the fully softened and residual values may not significantly affect the slope design. In slopes where there is no large difference between ϕ' and ϕ'_r and some uncertainty exists on whether a pre-existing shear surface is present, it may be prudent to verify the slope design using an appropriate value of ϕ'_r.

CONCLUSIONS

The following conclusions are based on results of torsional ring shear, direct shear, and triaxial compression tests on clays, mudstones, and shales and studies of first-time slides in stiff fissured clays.

The magnitude of the fully softened shear strength is stress-dependent and controlled by the type of clay mineral and quantity of clay-size particles. A fully softened shear strength relationship is presented that is a function of the liquid limit, clay-size fraction, and effective normal stress. The relationship can be used to estimate the entire nonlinear fully softened failure envelope or a secant fully softened friction angle that corresponds to the average effective normal stress acting on the critical slip surface.

The mobilized shear strength along the failure surface in first-time slides through stiff fissured clay with a liquid limit between 50 and 130% can be lower than the fully softened shear strength. A study of 14 first-time slides through stiff fissured clay suggests that the mobilized shear strength can be as low as the average between the fully softened and residual shear strengths. Additional case histories should be located and the effect of other geological factors, such as fissure spacing and bedding existence, should be studied to verify this conclusion.

The peak shear strength of the soil mass should be used to locate the critical slip surface in slopes that have not undergone previous sliding. In high-plasticity stiff fissured clays (liquid limit > 50%), the fully softened shear strength is approximately equal to the peak shear strength of the soil mass and can be used to locate the critical failure surface. The nonlinearity of the fully softened failure envelope should be incorporated into this analysis. In slopes that have undergone previous sliding, the critical slip surface is usually well defined, and thus does not need to be located.

The numerical difference between the fully softened and the residual friction angles is a function of clay mineralogy and effective normal stress. Natural soils with a liquid limit greater than 30% exhibit the largest difference in these friction angles (up to 16°). In these soils, the presence or absence of a pre-existing shear surface should be clarified during the subsurface investigation.

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