Engineering Geology 204 (2016) 94–107

Contents lists available at ScienceDirect

Engineering Geology

journal homepage: www.elsevier.com/locate/enggeo

Drained residual shear strength at effective normal stresses relevant to soil slope stability analyses

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ARTICLE INFO

Article history:
Received 6 May 2015
Received in revised form 12 October 2015
Accepted 4 February 2016
Available online 6 February 2016

Keywords:
Clays
Empirical correlation
Failure envelope
Landslides
Nonlinearity
Residual strength
Shale
Shear strength
Slope stability
Software

ABSTRACT

An extensive torsional ring shear testing program has been conducted to measure the drained residual shear strength of soils at the wide range of effective normal stress (10 to 700 kPa) usually mobilized in reactivated and first-time landslides. Soils, mudstones and shales of different plasticity and gradation were tested in the program. The effects of the change in nonlinearity of shear strength envelope over the utilized normal stress ranges on slope stability analyses were investigated. Using this data, new empirical residual shear strength correlations were developed as a function of soil index parameters and wide range of effective normal stresses. In essence, the correlations are presented as revised versions of those previously developed for a limited number of normal stresses utilizing the same soil index parameters. Comparisons were made with a considerable amount of back-calculated shear strength data reported in the literature for reactivated landslides as well as results predicted from existing shear strength correlations to verify the increased suitability of the new correlations for use in slope stability analyses. A numerical expression was also introduced to express the residual shear strength correlations for direct incorporation in slope stability software.

1. Introduction

Shear strength (τ) is a key input in any stability analysis of soil slopes. The drained residual shear strength (τ_r) is a crucial parameter in evaluating the stability of slopes that contain a pre-existing shear surface (Skempton, 1964, 1985). It can be also used along with the fully softened shear strength in determining the factor of safety against first-time sliding in stiff plastic clay slopes (James, 1970; Bishop, 1971; Potts et al., 1997; Stark and Eid, 1997; Mesri and Shahien, 2003). Significant efforts have been reported in the literature for assessing the residual shear strength through laboratory testing and back analysis of failed case histories. Several empirical correlations have been also presented to estimate such strength as a function of soil index parameters. Most of these correlations have been summarized in a subsequent section.

It has been long recognized that the shear strength envelopes of plastic soils are nonlinear, especially at a low effective normal stress ($\sigma'_n < 50$ kPa) range (Terzaghi and Peck, 1948; Penman, 1953; Bishop et al., 1965, 1971 and Ponce and Bell, 1971; Charles and Soares, 1984; Atkinson and Farra, 1985; Skempton, 1985; Day and Axten, 1989; Maksimovic, 1989). Such low normal stresses are usually relevant in slope stability analyses at locations where the critical slip surface intersects the face of the slope or passes through shallow depths or zones with high enough pore-water pressures to reduce effective stresses. In spite of this, parameters derived from laboratory shear tests that have been carried out at higher effective normal stresses, at which the curvature of the shear strength envelope significantly decreases, are commonly used to represent all zones in slope stability analyses. Even most of the existing residual shear strength correlations that incorporate the effect of the normal stress level (e.g., Stark and Eid, 1994; Mesri and Shahien, 2003; Stark and Hussain, 2013) have also been developed based on testing at a limited number of relatively high effective normal stresses. For example, the currently available correlations do not efficiently cover certain low and moderate ranges of the average effective normal stress (i.e., $\sigma'_n < 50$ kPa and 100 kPa $< \sigma'_n <$ 400 kPa) that have been mobilized in several reported reactivated and long-term first-time landslides in stiff plastic clays. Fig. 1 illustrates this limitation for well-documented landslides through English clays (namely; Upper Lias clay, London clay, Oxford clay, Kimmeridge clay, Chalky Boulder clay, Gault clay, Atherfield clay, Etruvia marl, Walton's wood and Jackfield carboniferous mudstone, and Edale shale).

To fill the knowledge gap described above, this paper mainly presents results from a laboratory research program involving residual shear strength tests conducted at effective normal stresses of 10, 25,

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http://dx.doi.org/10.1016/j.enggeo.2016.02.003
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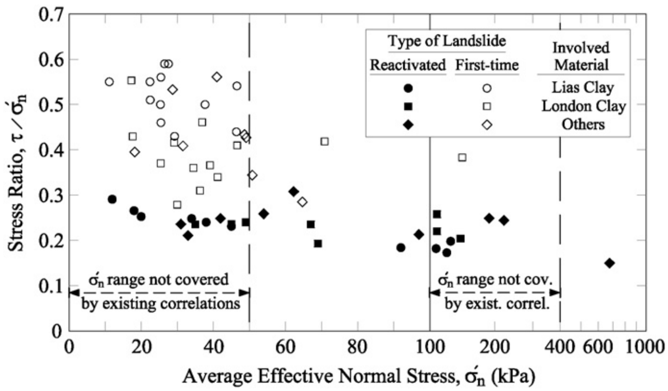

Fig. 1. Average normal stresses reported for landslides in English soil formations and the normal stress ranges not covered by testing utilized to develop the commonly-used residual strength correlations that incorporate the effect of normal stress (data points from Skempton, 1964, 1972, 1977, 1985; James, 1970; Chandler, 1974, 1976, 1977, 1982, 1984; Chandler and Skempton, 1974; Bromhead, 1978).

200, and 300 kPa on clays, silts, mudstones, and shales with plasticity and gradation varying over a wide range as presented in a table in the subsequent section. The residual shear strengths of these materials at effective normal stress of 50, 100, 400, and 700 are also presented. This is intended to: (i) complement the data set developed through conducting a series of torsional ring shear tests by Eid (1996); and (ii) to consequently revise and update the drained shear strength correlations that have been developed entirely (e.g., Stark and Eid, 1994; Mesri and Shahien, 2003) or chiefly (e.g., Stark et al., 2005; Stark and Hussain, 2013) based on such a data set. The testing program is limited to sedimentary fine-grained materials (i.e., materials with mostly plate-like clay particles); as such, the revised correlations presented herein can be used to predict the drained residual friction angles of such materials. The correlations exclude soils such as carbonate soils (White et al., 2012) and soils composed chiefly of allophane or halloysite (i.e., soils with non-platy particles such as volcanic ashes) that are unlikely to have particle rearrangement towards some preferred orientation (Wesley, 1977, 1992, 2003). They also exclude marine soils that contain numerous skeletal remains or foraminifera (Mesri et al., 1975; Najjar et al., 2007). Most of these soils exhibit high friction angle and small or no difference between the drained peak shear strength and the drained residual shear strength regardless of their plasticity (Saldivar and Jardine, 2005).

2. Testing method

A total of 50 clay, silt, mudstone, and shale samples were used in torsional ring shear testing to measure the drained residual shear strength at 8 different effective normal stresses (Table 1). To avoid unnecessary testing repetition, no shear strength tests were conducted on soils which were tested by Stark and Eid (1994) at effective normal stress of 50, 100, 400, and 700 kPa using the same testing procedure followed in the present study. As shown in Table 1, the utilized test samples cover a wide range of liquid limit (*LL*), plastic limit (*PL*), plasticity index (*PI*), and clay-size fraction (*CF*). Except for the heavily overconsolidated clay, mudstone, and shale samples, Atterberg limits and clay-size fractions were determined in accordance with the particle disaggregation standard procedure. Most heavily overconsolidated clays, mudstones, and shales possess diagenetic bonding that results in particle aggregation (induration). This aggregation usually survives the standard sample preparation procedure and consequently influences the measured index parameters and the accuracy of their correlations to the results of shear strength testing in which the aggregated particles would be battered (La Gatta, 1970; Townsend and Banks, 1974; Airo'Farulla and La Rosa, 1977; Eid, 2001, 2006). As a result, these materials were disaggregated by ball-milling of representative air-dried samples until all particles passed the standard sieve No. 200. The hydrated ball-milled materials were used in determining the index parameters as well as the shear strengths. This sample preparation procedure was adopted from that used by Mesri and Cepeda-Diaz (1986) to determine liquid limit and clay-size fraction that better infer the clay mineralogy and gradation of shales.

Remolded specimens for ring shear testing were obtained by adding distilled water to the air-dried and processed samples until a liquidity index of 1.5 was obtained. The sample was then allowed to rehydrate for at least one week in a moist room. The ring shear specimen is annular with inside and outside diameters of 7 cm and 10 cm, respectively. Drainage is provided by two bronze porous stones secured on the loading platen and the bottom of the specimen container. The specimen is confined radially by the specimen container which is 5 mm deep. The

Table 1
Soil, mudstone, and shale samples used in ring shear tests.

No.	Soil name	location	*LL* (%)	*PL* (%)	*CF* [a] (%)	*A*
1	Fraser-river silt[b]	Vancouver, BC, Canada	21	18	9	0.33
2	Glacial till[b,d]	Urbana, IL, USA	24	16	18	0.44
3	Loess[b,d]	Vicksburg, MS, USA	28	18	10	1.00
4	Gray silt[b]	Vancouver, BC, Canada	34	17	26	0.65
5	Bootlegger Cove clay[b,d]	Anchorage, AK, USA	35	18	44	0.39
6	Duck Creek shale[c,d]	Fulton, IL, USA	37	25	19	0.63
7	Slide debris[b]	San Francisco, CA, USA	37	26	28	0.39
8	Chinle (red) shale[c,d]	Holbrook, AZ, USA	39	20	43	0.44
9	Slopewash material[b]	San Luis Dam, CA, USA	42	24	34	0.53
10	Colorado shale[c,d]	Montana, MT, USA	46	25	73	0.29
11	Panoche mudstone[d]	San Francisco, CA, USA	47	27	41	0.40
12	Kaolinite clay[b,e]	Hephzibah, GA, USA	48	26	32	0.69
13	Four Fathom shale[c,d]	Durham, England	50	24	33	0.79
14	Mancos shale[d]	Price, UT, USA	52	20	63	0.51
15	Panoche shale[d]	San Francisco, CA, USA	53	29	50	0.48
16	Gulf-bed deposit[b]	Doha, Qatar	53	34	18	1.06
17	Red Sea white shale	Alsokhna, Egypt	55	22	50	0.66
18	Comanche shale[c,d]	Proctor Dam, TX, USA	62	32	68	0.44
19	Breccia material[b]	Manta, Ecuador	64	41	25	0.92
20	Bearpaw shale[c,d]	Billings, MT, USA	68	24	51	0.86
21	Slide debris[d]	San Francisco, CA, USA	69	22	56	0.84
22	Bay Mud[b,d]	San Francisco, CA, USA	76	41	16	2.19
23	Patapsco shale[c,d]	Washington D.C., USA	77	25	59	0.88
24	Nile deposit[b]	Damanhur, Egypt	82	27	58	0.95
25	Pierre shale[c,d]	Limon, CO. USA	82	30	42	1.24
26	Red Sea gray shale	Alsokhna, Egypt	84	27	44	1.30
27	Upper Pepper shale	Waco, TX, USA	89	29	72	0.83
28	Santiago claystone[d]	San Diego, CA, USA	89	44	57	0.79
29	Toshka shale	Toshka, Egypt	91	30	58	1.05
30	Lower Pepper shale[d]	Waco Dam, TX, USA	94	26	77	0.88
31	Altamira Bentonitic tuff[d]	Portuguese Bend, CA, USA	98	37	68	0.90
32	Brown London clay[d]	Bradwell, England	101	35	66	1.00
33	Mokattam yellow shale	Cairo, Egypt	103	33	43	1.63
34	Cucaracha shale[c,d]	Panama Canal	111	42	63	1.10
35	Otay Bentonitic shale[d]	San Diego, CA, USA	112	53	73	0.81
36	Denver shale[c,d]	Denver, CO, USA	121	37	67	1.25
37	Bearpaw shale[c,d]	Saskatchewan, Canada	128	27	43	2.35
38	Mokattam gray shale	Cairo, Egypt	134	37	79	1.23
39	Pierre shale	New Castle, WY, USA	137	32	67	1.57
40	Oahe firm shale[d]	Oahe Dam, SD, USA	138	41	78	1.24
41	Claggett shale[c,d]	Benton, MT, USA	157	31	71	1.78
42	Bentonitic claystone	Fayoum, Egypt	164	55	79	1.38
43	Taylor shale[c,d]	San Antonio, TX, USA	170	39	72	1.82
44	Pierre shale[c,d]	Reliance, SD, USA	184	55	84	1.54
45	Oahe bentonitic shale[d]	Oahe Dam, SD, USA	192	47	65	2.23
46	Panoche clay gouge[d]	San Francisco, CA, USA	219	56	72	2.26
47	Midra gray shale	Sealine, Qatar	231	74	79	1.99
48	Lea Park bentonitic shale[d]	Saskatchewan, Canada	253	48	65	3.15
49	Bearpaw shale[c,d]	Fort Peck Dam, MT, USA	288	44	88	2.77
50	Bentonitic clay[b,e]	El Hammam, Egypt	293	46	89	2.78

[a] Quantity of particles <0.002 mm.
[b] Samples not ball-milled.
[c] Index properties from Mesri and Cepeda-Diaz (1986).
[d] Samples reprocessed from those utilized in Eid (1996).
[e] Received in the form of powder passed through sieve # 200.