March 2010 • Volume 136, Number 3
ISSN 1090-0241 • CODEN: JGGEFK

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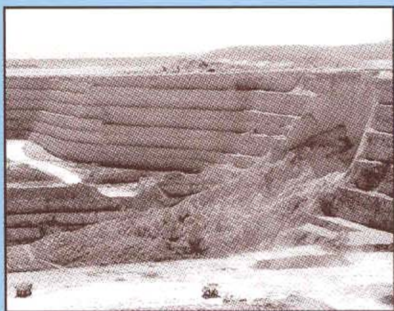

Journal of Geotechnical and Geoenvironmental Engineering

Technical Papers

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Journal of Geotechnical and Geoenvironmental Engineering

VOLUME 136 / NUMBER 3 **MARCH 2010**

Technical Papers

Technical Notes

Journal of
Geotechnical and
Geoenvironmental
Engineering

VOLUME 136 / NUMBER 3
MARCH 2010

Discussions and Closures

Suction-Monitored Direct Shear Testing of Residual Soils from Landslide-Prone Areas

Apiniti Jotisankasa[1] and Warakorn Mairaing[2]

Abstract: The apparent cohesion due to soil suction plays an important role in maintaining the stability of steep unsaturated soil slopes with deep ground water table. In this paper, a modified direct shear box is used to determine the relationships between the value of this additional cohesion and the associated soil suction. The apparatus incorporates a miniature tensiometer which allows for the simple and direct measurement of suction during shearing. The soil-water characteristic curves and shearing behavior of intact residual soils, being low-to-medium plasticity silts, as well as silty sand, taken from four landslide-prone areas in Thailand, have been investigated. The relatively low air-entry suctions (0–7 kPa) and bimodality of the soil-water characteristic curves gives an indication of the structured pore size distribution of the materials tested. Samples with higher suction tend to display stronger bonding at particle contacts and thus are more brittle. The shear strength is found to increase nonlinearly with suction, though linearization can be reasonably assumed for suction below around 30 kPa. Prediction of shear strength based on soil-water characteristic curves agrees better with ultimate than peak values. A simple equation is proposed for the minimum ultimate strength that can be expected in an unsaturated residual soil with a suction lower than about 30 kPa.

DOI: 10.1061/(ASCE)GT.1943-5606.0000225

CE Database subject headings: Unsaturated soils; Soil suction; Soil tests; Residual soils; Landslides.

Author keywords: Unsaturated soils; Soil suction; Soil tests; Residual soils.

Introduction

The shear strength of unsaturated soils has long been the subject of research (e.g., Bishop et al. 1960; Fredlund and Rahardjo 1993; Lu and Likos 2006). One of the simplest relationships for shear strength in unsaturated soils is the following:

$$\tau = c' + (\sigma - u_a) \times \tan \phi' + c^s \tag{1}$$

where c'=effective cohesion intercept; σ=normal total stress; u_a=pore air pressure; and ϕ'=effective angle of friction. c^s is the apparent cohesion in unsaturated soil, which is controlled by interparticle bonding due to capillarity and physiochemical forces as well as distribution of water in soil pore. Lu and Likos (2006) introduced the term "suction stress," σ^s, to lump all factors into a characteristic curve as follows:

$$\sigma^s = \frac{c^s}{\tan \phi'} = \frac{\tau - c' - (\sigma - u_a)\tan \phi'}{\tan \phi'} \tag{2a}$$

$$\sigma^s = -(u_a - u_w) \quad \text{if } (u_a - u_w) \leqslant 0 \tag{2b}$$

$$\sigma^s = f(u_a - u_w) \quad \text{if } (u_a - u_w) > 0 \tag{2c}$$

where u_w=pore-water pressure. Lu and Likos (2006) also suggested an estimation of the suction stress based on the soil-water characteristic curve (SWCC) as follows:

$$\sigma^s = -\frac{\theta - \theta_r}{\theta_s - \theta_r} \times (u_a - u_w) \tag{3}$$

This suction stress, σ^s, is important for the stability analysis of steep unsaturated soil slopes with deep ground water table. Prolonged rainfall and subsequent infiltration can diminish such suction stress to nearly zero at some critical depths along the slope and often become a triggering mechanism of shallow slope failure (e.g., Collins and Znidarcic 2004; Lu and Godt 2008). This paper introduces a simple testing technique for determining the relationship between apparent cohesion and suction for unsaturated soils, as well as presents results on the shearing response of some residual soils obtained from four landslide-prone areas in Thailand.

Materials and Site Characteristics

The four sites selected for the study are Site C (Chantaburi), Site N (Nakornnayok), Site O (Omkoi), and Site T (Tak). All sites are situated in hilly areas of Thailand, and their average slope gradient ranges between 25° and 50°. The climate in Sites O and T can be characterized as humid subtropical; with an average annual rainfall of around 1,200 mm. Sites C and N receive a greater annual rainfall of approximately 2,500 mm, due to the effect of tropical monsoon. Numerous shallow failures (depths of less than 1–2 m) were reported on Site C during July of 1999 and 2001; and in May 2003 similar events took place on Sites O and T. In all cases, the failures were triggered by rainfall in excess of about

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Note. This manuscript was submitted on August 19, 2008; approved on August 13, 2009; published online on August 26, 2009. Discussion period open until August 1, 2010; separate discussions must be submitted for individual papers. This technical note is part of the ***Journal of Geotechnical and Geoenvironmental Engineering***, Vol. 136, No. 3, March 1, 2010. ©ASCE, ISSN 1090-0241/2010/3-533–537/$25.00.

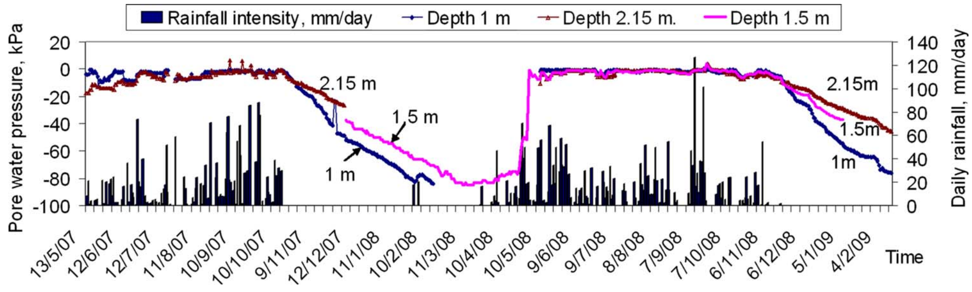

Fig. 1. Variation of rainfall and corresponding pore-water pressure at Site N

300 mm over a period of 4 days, with the fourth day rain exceeding approximately 150 mm. Deforestation appeared to have also contributed to landslide in Sites O and T. Site C, however, was covered with thick rainforest vegetation. As for Site N, a bare cut slope with gradient of about 45° failed in August 2004 (with a depth of failure ~2 m) due to heavy rainfall amounting to 344 mm over 4 days. After its failure, Slope N was regraded to about 26° and densely grassed. Fig. 1 shows the daily rainfall and pore- water pressure variation recorded for 2 years in the middle of soil slope at Site N. The slope remained unsaturated with pore-water pressures ranging from −1 to −80 kPa during most of the year. Positive pore-water pressures were measured within the slope in a number of discrete time intervals, but only at a depth of 2.15 m. These observations coincided with the occurrence of daily rainfalls of about 100 mm during September 2007/2008. The slope did not fail at that point probably due to its low gradient (~2°). However, a steeper gradient (say greater that 45°) in similar fine-grained slopes might result in failure when the pore-water pressure approaches zero (without necessarily becoming positive) as evidenced by Godt et al. (2009).

The geology of Sites C and O consists mainly of granites, whereas Site T is composed primarily of gneiss. Site N is composed of rhyolite, andesite, tuffs, and agglomerate. The basic properties of the residual soils are summarized in Table 1. "Undisturbed" samples were carefully collected at depths of about 0.5–1 m in all four locations, using a thin-walled tube sampler with a diameter of 63 mm.

Soil-Water Characteristic Curves

Fig. 2 shows the SWCC for all four soils, determined using the KU-tensiometer (Jotisankasa et al. 2007) and relative humidity sensor (for suction >1,000 kPa). The test procedure involved incrementally wetting and drying the sample with its suction, weight, and dimensions being measured at each stage. The values of water content at suction of 0.1 kPa, shown in Fig. 2, were in fact arbitrarily chosen to indicate the water contents of soaked samples, θ_s. The value of θ_s is not necessarily the same as the porosity, n, due to the presence of some occluded air. The curve fitting for these results was carried out using the functions proposed by Gitirana and Fredlund (2004), as shown in Fig. 2. For Samples O, T, and N, SWCC bimodalily is evident, due to the soils' aggregated structure. In addition, the blow-through suctions for the wetting SWCC were slightly lower than those of drying SWCC, which indicated their hysteresis.

Suction-Monitored Direct Shear Box and Testing Program

Fig. 3 shows the suction-monitored direct shear apparatus. Only minor modifications were made to the top cap, which in the current setup allowed for the tensiometer to be inserted through an orifice and secured in place using a clamping set. To maintain a constant water content during testing, a plastic wrapping and wet clothes were used to cover the shear box. Table 2 gives details of a typical testing program in this study. The first series consisted of slow (consolidated-drained) shearing tests on saturated samples (O-2, O-14) in a standard direct shear device. All shearing tests on unsaturated samples were carried out using the modified apparatus in a constant water content condition with direct suction measurement and a shearing rate of 0.1 mm/min.

Shearing Behavior

Table 3 summarizes the effective shear strength parameters of the four soils from a series of consolidated-drained direct shear tests on saturated samples. The stress-strain relationships for all saturated samples appeared to be strain hardening. Fig. 4 shows the typical shearing behavior of the unsaturated residual soil for Site O. It can be seen that the samples with higher suctions (O-0, O-9) exhibited very clear strain-softening behavior due to breakage of the bonding provided by the water menisci. At the early stages of shearing, both volume and suction of these samples decreased slightly. However, as the samples began to dilate and approached the peak strength, the suction tended to increase slightly. The samples with lower suction (O-1, O-2) appeared to display a contractant response and there was no observed reduction in strength during shearing.

Table 1. Basic Properties of the Studied Materials

Site	LL (%)	PI (%)	% gravel	% sand	% silt	% clay	Soil type
C	41–48	5–12	2	53.2–55.4	22.9–26.5	16.0–22.7	SM
N	46–51	6–18	0.5–5.5	13.7–18.9	38.9–53.9	31.9–36.7	MH/ML
O	41.6	15	2.2	31.4	40.1	26.3	ML
T	48.3	11	0.9	34.2	40.1	24.9	ML

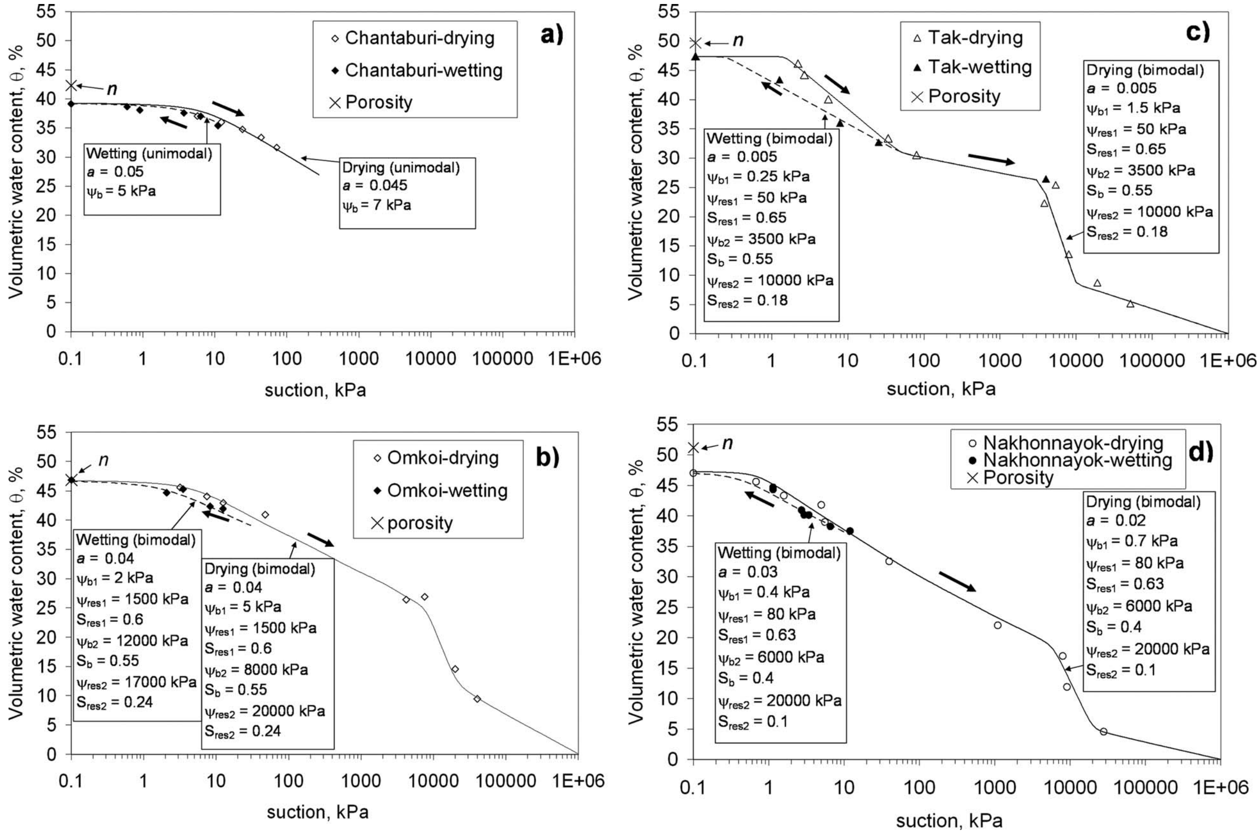

Fig. 2. SWCCs for samples from (a) Chantaburi; (b) Omkoi; (c) Nakornnayok; and (d) Tak

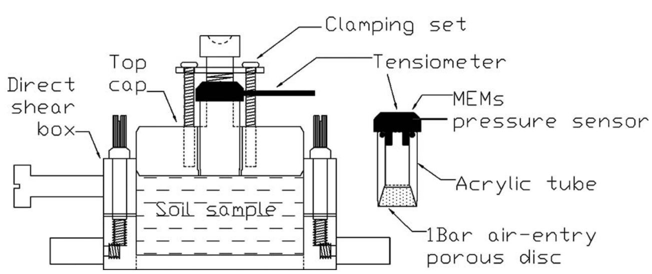

Fig. 3. Experimental setup of suction-monitored direct shear tests and details of KU-tensiometer

Table 2. Testing Program for Site O

Test number	S_r (%)	w (%)	e	Normal stress (kPa)
O-2	~100	—	0.700	31
O-14	~100	—	0.801	15.5/31/62 (multistage)
O-1	89.1	26.9	0.802	31
O-7	90.0	28.4	0.837	31
O-11	83.4	25.2	0.801	31
O-9	81.7	26.2	0.843	31
O-0	81.7	22.9	0.742	31
O-13	66.8	21.7	0.874	15.5/31/62 (multistage)
O-16	82.8	23.6	0.754	15.5/31/62 (multistage)

Note: S_r=initial degree of saturation; w=gravimetric water content during shearing; and e=initial void ratio.

Suction Stress and Unsaturated Shear Strength

To evaluate the contribution of suction to the shear strength, the suction stresses, σ^s, of all samples were determined according to Eq. (2*a*) both for shear strength at peak and ultimate states, as shown in Fig. 5, and were compared with the prediction based on Eq. (3) using both drying and wetting SWCCs. In the single stage tests of soils from Sites C and O [Figs. 5(a and b)], the differences between the peak and ultimate suction stress appear to be greater at higher suction probably due to the presence of stronger bonds between particles and to the greater sample dilatancy. The suction stresses from the multistage tests fall within the boundaries of peak and ultimate states. It must be noted that a further cause for bonding in residual soils can be the depositions of carbonate, hydroxides, organic matter, etc. (Vaughan 1988). There is a possibility of removing such bonding as samples are sheared to a larger strain. It is therefore proposed that the lower bound of the ultimate suction stress observed in the test results corresponds to the destructured state of the material.

The stress-strain response during shearing of samples from Sites T and N, shown in Figs. 5(c and d), is that of a strain-hardening material of low dilatancy. In both cases one would expect for the cementation bonds to be removed prior to shearing. The samples from Site T were sheared at a normal stress of 64 kPa, which would be expected to have destroyed much of the

Table 3. Summary of Effective Shear Strength Parameters

Site	Chantaburi	Nakornnayok	Omkoi	Tak
c' (kPa)	8.7	12.8	17.6	6.5
ϕ' (deg)	38.6	33.1	28.7	37.0

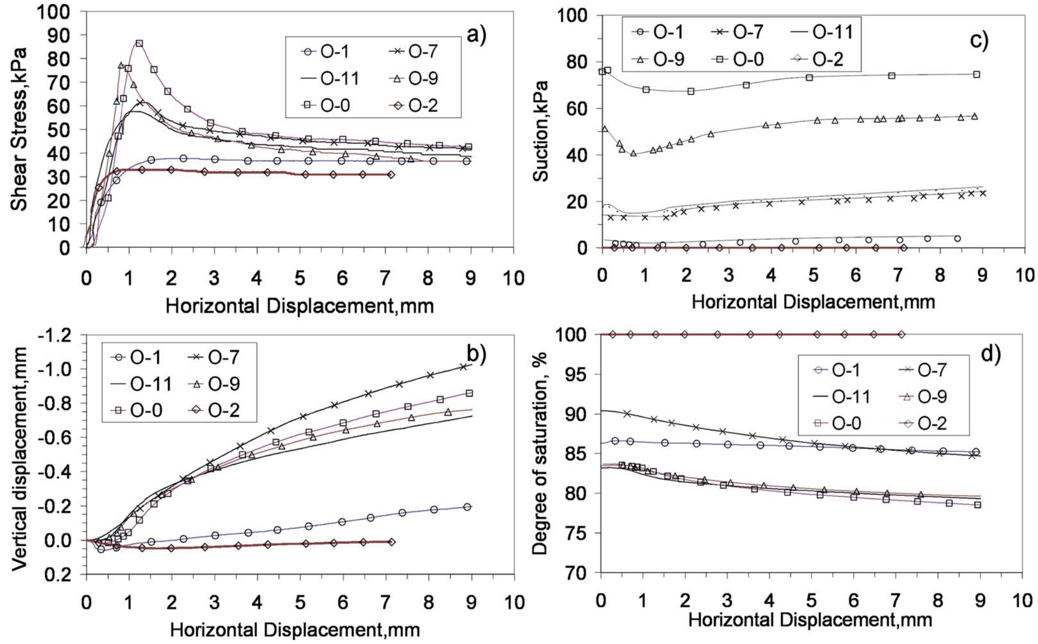

Fig. 4. Typical results of single stage shearing tests (samples from Site O tested at a vertical stress of 31 kPa)

existing cementation. The samples from Site N were taken from a failed slope where degree of inherent cementation was expected to be minimal. The upper bound in Fig. 5(d) envelopes some scatters in suction stress, believed to be due to the heterogeneous cementation characteristics of the material. In most cases, the prediction made with Eq. (3) appears to be in good agreement with the lower bound of the suction stress, especially for suctions lower than about 30 kPa. Predictions based on wetting path are nearly identical to those based on drying path. Several tests at suctions higher than 50 kPa however show the measured values of ultimate suction stress to be significantly lower than the predicted values as shown in Figs. 5(a and b). This is believed to be due to the greater tendency of the sample to dilate, as well as due to the reduced wetted areas around soil particles, when sheared at a higher suction (>50 kPa) and a lower normal stress (<35 kPa). Fig. 5 also shows a further prediction, corresponding to a simplified version of the suggested approach and calculated using Eq. (4) as follows:

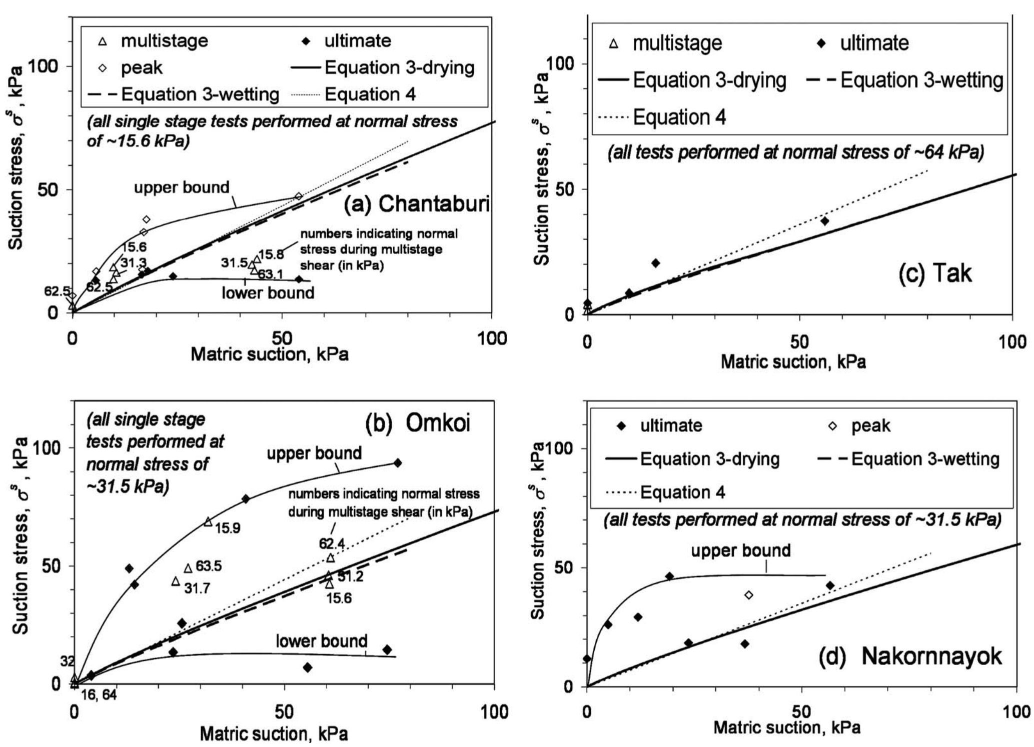

Fig. 5. Variation of suction stress with suction

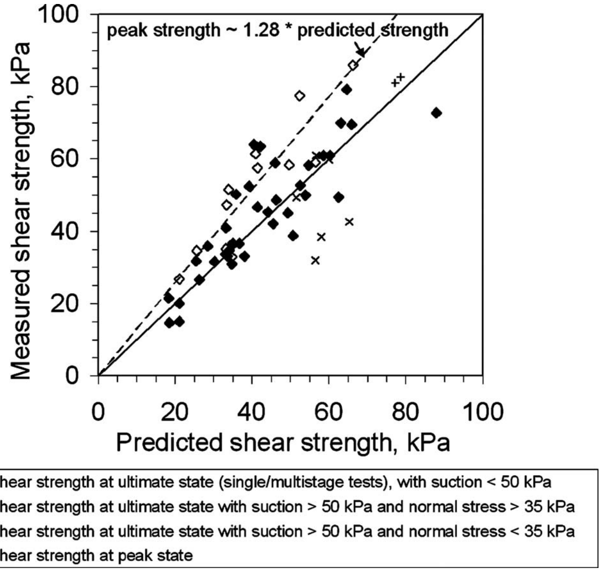

Fig. 6. Comparison between predicted shear strength using Eqs. (1), (2*a*)–(2*c*), and (3) and the experimental values

$$\sigma^s = \left(\frac{\theta_{33}}{\theta_s}\right)(u_a - u_w) \tag{4}$$

where θ_{33}=volumetric water content at suction of 33 kPa (the nominal field capacity). As clearly shown in Fig. 5, the simplified linear approach [Eq. (4)] is practically identical to Eq. (3) for suctions less than about 33 kPa. The validity of this approach is further demonstrated in Fig. 6, where predicted and measured values of shear strength have been plotted together. The measured peak shear strengths appear to be greater than the predicted values by about 0–28%. Regarding stability analysis, many researchers (e.g., Leroueil 2001; Mesri and Shahien 2003) have demonstrated that the shear strength at "fully softened" or "critical state"—which can be assumed to be close to the ultimate state—is shown to be the lower bound for mobilized shear strength of many first-time slope failures. Eq. (3) or Eq. (4) can thus be considered suitable for estimating the ultimate unsaturated strengths for slope stability analysis.

Conclusions

A suction-monitored direct shear box equipped with the miniature tensiometer is proposed as an alternative testing technique for characterizing the shear strength of unsaturated soils for slope stability analysis. When shearing takes place at relatively low normal stress (<35 kPa), residual soils with higher suction tend to display a strong degree of bonding around soil particle contacts and this translates in significant differences between the peak and ultimate shear strength (or suction stress). It appears that this difference increases with suction. When shearing takes place at a normal stress higher that 35 kPa in samples with a low degree of cementation, strain-hardening behavior was observed, with no significant drop in strength after attaining peak strength. This is expected since higher normal stress would suppress dilatancy and could result in the destruction of bonding.

Both predictions of suction stress based on the SWCC, and that based on the field capacity (volumetric water content at 33-kPa suction), agree equally well with the lower bound of ultimate suction stress for suction below about 30 kPa.

Acknowledgments

The writers gratefully acknowledge the research Grant No. MRG4980086 of the Thailand Research Fund and Commissions of Higher Education, Thailand, and thank Mr. Narin Hansachainun, Mr. Nattapong Paiboonsombat, and Mr. Lerdchao Kenganantanon for their assistance in performing some direct shear tests in this research project. The writers also thank Dr. Auckpath Sawangsuriya, Dr. Rafael Monroy, and other anonymous reviewers for their invaluable comments.

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1090-0241(201003)136:3;1-F