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UNSATURATED SHEAR STRENGTH OF EXPANSIVE SOILS FROM NATURAL SLOPES IN QUEENSLAND

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ABSTRACT

Expansive soils exhibit significant changes in the swell and shrink behaviour upon variations of moisture content. Majority of soils in Queensland are expansive; consequently, failures in natural slopes with expansive characteristics are commonly reported under climate-induced soil moisture variations. In order to understand the failure mechanism of such soil slopes, it is vital to analyse slope stability under unsaturated conditions. The current study employed undisturbed soil samples from natural slopes in Queensland, and determined the basic soil properties, including the soil-water characteristic curve (SWCC), and unsaturated shear strength to facilitate the analysis of rainfall-induced slope failure mechanism. A series of conventional direct shear tests were conducted for different moisture contents under various vertical stresses. The test results were analysed to obtain the variation of the apparent cohesion and the effective friction angle of the soil with moisture content and suction. The study presents an empirical equation to predict the cohesive component in the shear strength of unsaturated expansive soils as an exponential function of suction. The formulation originated from multiple linear regression analysis for data sets obtained from shear tests using undisturbed soils with varying moisture contents. The empirical equations can realistically predict the reduction in soil cohesion due to wetting (R^2 = 0.8791). The study provides an alternative to the quantitative estimation of unsaturated shear strength of expansive soils.

Keywords: Unsaturated soil, unsaturated shear strength, Expansive soil, slope stability, Direct shear test

INTRODUCTION

The understanding of soil interface shear strength is vital in designing and analysing the stability of geotechnical structures [1-6]. The cohesive strength of unsaturated soils plays an important role in the stability of both natural and artificial soil slopes, [7, 8] in which the reduction of the soil cohesion due to wetting can cause shear deformation of the slopes at a previously unsaturated shallow layer. Therefore, determination of the critical condition for sliding requires a slope stability analysis, including an effect for the loss of cohesion.

As shear strength tests on unsaturated soils are costly and time-consuming, simple and indirect methods, have been developed to obtain the shear strength of unsaturated soils for engineering purpose. Numerous researchers suggested that the shear strength of unsaturated soils could be predicted using different equations [9]. Based on the soil-water characteristic curve (SWCC), various theoretical and empirical models were proposed for prediction of unsaturated shear strength. However, minimum compelling attempts were made on the analysis of the characteristics of unsaturated shear strength, and the relationship between the shear strength parameters and suction of unsaturated expansive soils.

This paper presents the results of a series of direct shear tests on unsaturated soils from natural slopes in Maleny, Queensland, Australia. Direct shear testing of unsaturated soils is desirable since less time is required to reach the failure of the soil specimen compared to the triaxial test. The time to failure in the direct shear test is significantly reduced because the specimen is relatively thin. However, a lengthy testing period is expected for unsaturated soils due to the low coefficient of permeability of the soil. In this paper, an empirical equation is suggested for the prediction of unsaturated shear strength as a function of suction.

The equation was obtained from a multiple linear regression analysis of the results of direct shear tests using undisturbed expansive soils with varying moisture contents. Even though the equation lacks a theoretical validity from a physical standpoint, but has practical advantages, especially for geotechnical engineering purposes.

STUDY AREA

The study area is located in Lake Baroon catchment, Maleny, Queensland, Australia (26.72 ºS 152.87 ºE). Mapleton - Maleny plateau, which has been documented as a highly susceptible area for rainfall-induced slope failures since the mid - 1950s. (e.g. [10]). Slope failures and mass movements of sediment into the waterways within the Lake Baroon catchment are recognised as a significant risk to water quality and the water storage capacity of Lake Baroon, which is used to supply water to South East Queensland. Approximately 170 mass movement landforms have been identified within the Lake

Baroon catchment, and the study area is one such high-risk slope. This landslide site hosted a voluminous, single-failure rotational landslide in 2008 following heavy rainfall.

TEST MATERIAL

The test series herein employed soils from the Lake Baroon catchment. Undisturbed samples from seven locations of the study area were collected at various depths ranging from the surface down to 4 m. Initially, a trench was created to the desired depth with an excavator and then 500 mm long sampling tubes (diameter 75 mm) were inserted into the soil by connecting a specially modified adaptor to the excavator's arm as in Fig. 1.

Fig. 1 Insertion of soil sampling tube by the modified adaptor to the excavator's arm

Table 1 summarises the results of the laboratory tests conducted to determine the index properties of the soil according to Australian standards.

Table 1 Index soil properties

Classification Test	Results
Grain size Distribution	% finer than 75 μm > 79% Clay % = 41.0 %
Atterberg Limits	LL = 67.2 % PI = 28.2 %
Linear Shrinkage	LS = 13.4 %
Specific Gravity	G_s = 2.67
X-ray diffraction (XRD)	Presence of Smectite minerals (> 30 %)

Note: LL = Liquid limit; PI = Plastic index; LS = Linear shrinkage

TEST PROCEDURE

Undisturbed soil samples were collected from Lake Baroon catchment. Soil sampling tubes were employed in collecting undisturbed samples from seven locations at various depths. In-situ density and the moisture content of soil were determined for each sampling location. The soil samples were used to determine SWCC, swelling characteristics and unsaturated shear strength.

A dewpoint potentiometer (Fig. 2) was employed in determining SWCC, which is the graphical relationship between soil suction and water content (gravimetric or volumetric) or degree of saturation. The dew point potentiometer measures water potential from 0 to -300 MPa with an accuracy of ±0.1 MPa from 0 to -10 MPa and ±1 % from -10 to -300 MPa. The high total suctions measured from WP4C Dewpoint potentiometer were assumed to consist of negligible osmotic suction effect and hence, used as matric suctions. These samples were initially prepared for known water contents and placed in enclosed containers to prevent any moisture loss prior to testing. Further, to obtain the SWCC, a best fitting relationship was established using the logarithmic model proposed by Fredlund and Xing [11] as presented Eq. (1) and (2).

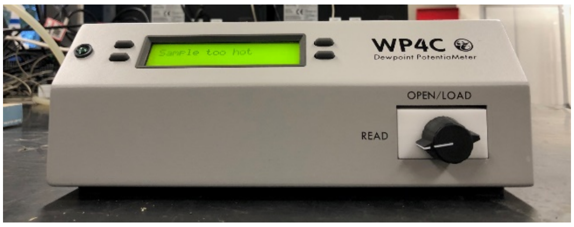

Fig. 2 WP4C Dewpoint potentiometer for suction measurement

$$\theta\ (\psi, a, n, m) = \frac{C\ (\psi)\ \theta_s}{\{\ln\left[\ 2.718\ +\ \left(\frac{\psi}{a}\right)^n\ \right]\}^m} \quad (1)$$

$$C\ (\psi) = \frac{-\ln\ (1 + \frac{\psi}{\psi_r})}{\ln\ (\ 1 + 10^6/\psi_r)} + 1 \quad (2)$$

where a is equal to the air-entry suction; n is the tangent to the curve at the inflexion point (in the transition zone); m is related to the residual water content; $C(\psi)$ is a correction function; ψ_i is the suction corresponding to a given water content, and θ_s is the saturated volumetric water content.

A series of oedometer based one-dimensional consolidation tests (Fig. 3) were conducted as prescribed in AS 1289.6.6.1-1998 for soil samples for various initial water contents and dry densities under different surcharges to determine the rate of shearing for each direct shear test [12]. At the completion of the primary consolidation, the time to reach 50% consolidation (t_{50}) and following empirical equations were used to determine the shearing rate (Eq. (3) and (4)). Taylor's square root of time fitting method was used to determine t_{50} during the study. The selected surcharge values for this study were 50, 100, 150, 200 and 250 kPa. Samples were tested for different initial volumetric water contents from 25% to 43% to replicate in-situ water contents. The samples were cured for four days prior to the oedometer tests. The soil samples prepared for known initial gravimetric moisture contents were then statically compacted to achieve the target density to replicate in-situ density. Subsequently, a representative sample was cut into a consolidation ring, followed by the placement of filter papers and porous disks, at the top and the bottom of the sample. The samples were then subjected to the aforementioned surcharges.

Fig. 3 Conventional oedometer apparatus used for the shear rate determination

$$t_f = 50t_{50} \quad (3)$$
$$R = d_P/t_f \quad (4)$$

where t_f is the time to failure in minutes, d_p the shear displacement at which peak strength is likely to be reached, in millimetres and R is the required shearing rate.

To obtain cohesion (c) and friction angle (φ′) for the same in-situ water content of soil as in oedometer tests, five direct shear tests on five identical soil samples (the same density and water content) were conducted with variation in the normal stresses 50, 100, 150, 200 and 250 kPa as prescribed in AS 1289.6.2.2 - 1998. The maximum failure shear stresses were then plotted with the corresponding normal stress to obtain c and φ′ for the given water content. The shearing rate determined from the oedometer tests were adopted for the direct shear tests. The same procedure was repeated for five different water contents to replicate in-situ water contents.

The direct shear apparatus (Fig. 4) accommodated samples with a diameter of 63.5 mm and a height of 25 mm. The soil samples prepared for the predetermined initial moisture contents were statically compacted to achieve the target density. Subsequently, a representative sample was cut into a shear ring, as shown in Fig. 4, followed by the placement of filter papers and porous disks, at the top and the bottom of the sample.

After determining c and φ′ for different water contents, using the SWCC, which was measured for the same density, the suction corresponding to the volumetric water content was obtained and ultimately the variation of c with the suction was adopted to determine ϕ^b.

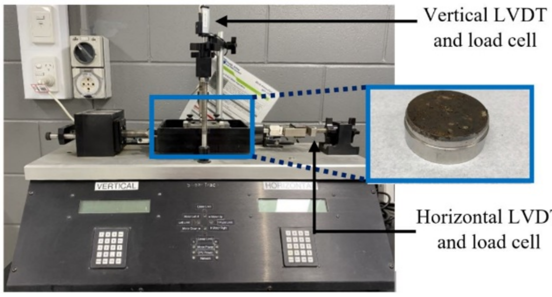

Fig. 4 Direct shear apparatus and the soil sample compacted into the cutting ring for direct shear test

RESULTS AND DISCUSSION

The SWCC

The total suction (ψ) was measured based on the relative humidity concept, and the SWCC equation proposed by Fredlund and Xing [11] was used in this study. However, due to the negligible osmotic suction, the total suctions were assumed as the matric suctions. Figure 5 depicts the SWCC for the test material.

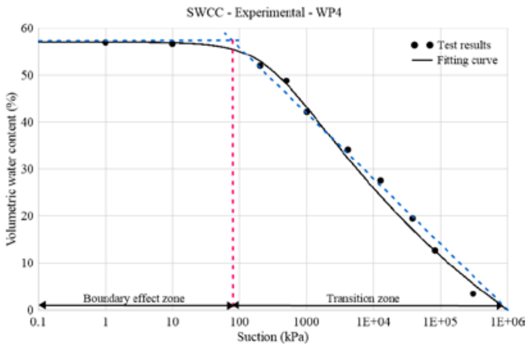

Fig. 5 The SWCC for the test material

The air-entry suction is about 85 kPa, and the residual degree of saturation 8.2%. The SWCC is clearly step-phased with the boundary effect zone and the transition zone well defined while the residual zone is not well defined. In the boundary effect zone, the soil is almost saturated, and the water content is nearly independent of suction, while the shear strength develops linearly with suction. In the transition zone, the water content decreases drastically as the suction increases, and the shear strength varies nonlinearly with suction. In the residual zone, the water content has a residual value. For sandy and silty soils, the shear strength is independent of suction; by contrast for clayey soils, it increases slightly with the suction increase [9].

Shearing rate

The t_{50} determined using Taylor's square root of time fitting method was adopted in calculating the shearing rate. Initially, compression vs square-root-of-time plot was produced as depicted in Fig. 6. Figure 6 presents the compression vs square-root-of-time plot for the volumetric water content of 34.6% under normal stresses 50, 100, 150, 200 and 250 kPa. The construction as prescribed in AS 1289.6.6.1-1998 was then employed in determining shearing rates. Table 2 summarises the shearing rates determined for the volumetric water content of 34.6%, and a similar procedure was followed in determining shearing rates for soil specimens with other moisture contents.

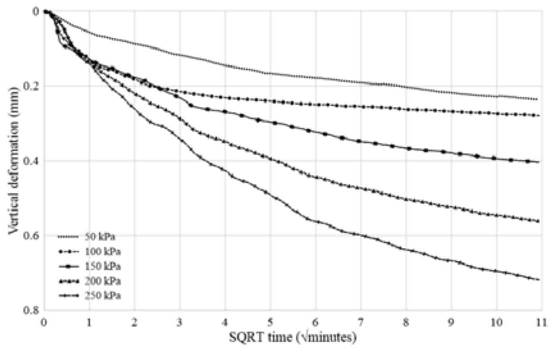

Fig. 6 The compression versus square-root-of-time plot for the volumetric water content of 34.6%

Table 2 Shearing rates determined for the volumetric water content of 34.6%,

Normal stress (kPa)	t_{50} (min)	Shear rate (mm/min)
50	7.247	0.055
100	6.317	0.063
150	5.898	0.069
200	4.628	0.086
250	6.450	0.062

Direct shear tests

Direct shear tests were conducted for all the five moisture contents under the same normal stresses 50, 100, 150, 200 and 250 kPa. Figure 7 presents the variation of shear stress with horizontal displacement for the soil specimens with the volumetric water content of 34.6%. Figure 7 depicts that the peak shear stresses were increased with the normal stress and curves are clearly step-phased. In the beginning, the shear stress increases very rapidly with the increase of shear displacement. Then, after a certain stress value, the slope of the curve declines. It also suggests that when the normal stresses increase from 50 to 250 kPa, the stress-strain relationship of the specimen changes from strain-softening to strain-hardening.

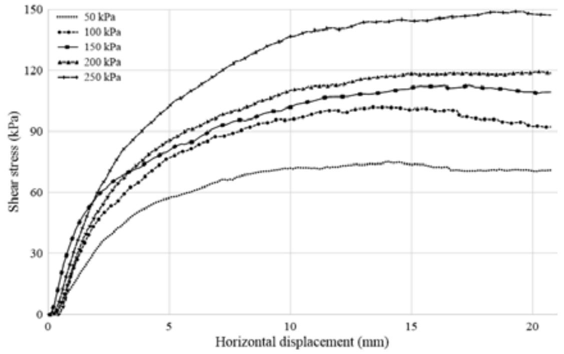

Fig. 7 Variation of shear stress with horizontal displacement under different normal stresses for the volumetric water content of 34.6%

Shear strength

The shear strength of the soils decreased with the increase in moisture content. Average volumetric water contents ranged from 0.25 to 0.43 for the test material (44.0 - 75.4% saturation). Figure 8 shows the results with a simple linear regression for each of the volumetric water content. Table 3 lists the values of the y-intercept and the inclination of the regression lines (i.e. cohesive strength and angle of shearing resistance in terms of simple linear regression), along with the corresponding suction values as per the Fredlund and Xing [11] fitting curve presented in Fig. 5.

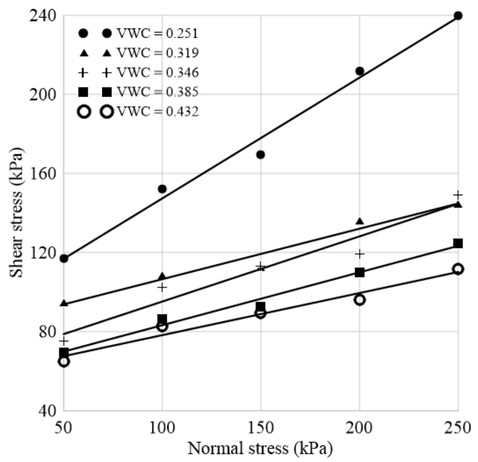

Fig. 8 Results of the direct shear tests (The solid lines indicate the simple linear-regression lines for each specimen group)

Table 3 Shear strength parameters obtained by simple linear regression with suction

Volumetric water content (m^3/m^3)	Cohesion (kPa)	Friction angle (Degree)	Suction (kPa)
0.251	86.28	31.42	11258.7
0.319	81.01	14.32	4364.8
0.346	65.25	18.24	3043.5
0.385	56.52	14.97	1827.7
0.432	56.93	12.02	991.8

The inclinations of the regression lines are largest in the driest conditions and drastically decreases for the wetter samples, converging at 12–18° (Fig. 8; Table 3). In other words, the angle of shearing resistance of the moist soils seems to be constant, independent of volumetric water content, except in the driest condition.

The y-intercepts of the regression lines decreased with increasing moisture content and approached a minimum value at the more saturated condition (Table 3). Figure 9 shows the relationships between the corresponding suction values for the average volumetric water content in each group and the y-intercept of the regression lines. The values of the y-intercept tend to increase linearly on the semi-log scale (i.e. the cohesive strength of the soils exponentially increase with an increase in suction).

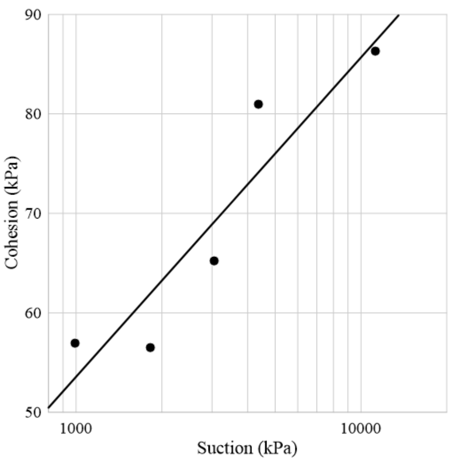

Fig. 9 Relationship between the corresponding suction values for the average volumetric water content and cohesive strength

Formulation of shear strength as a function of suction

On the basis of the shear strength reduction characteristics (Figs. 8 and 9), it was assumed that: (1) the angle of shearing resistance takes a constant value; (2) an exponential function is valid to express the relationship between apparent cohesion and suction.

From this, the regression function can be postulated as follows:

$$\tau = \sigma' \tan \phi' + Ce^{\mu\psi} \quad (5)$$

where τ is shear strength, σ′ is net normal stress, ψ is suction, φ′ is the effective angle of shearing resistance, C is a hypothetical minimum value of cohesion (when ψ = 0), and μ is a coefficient related to the susceptibility of strength increment (μ>0).

To determine the values of unknowns (i.e. φ′, C and μ), Eq. 5 should be linearised as in below Eq. 6:

$$\ln\left[\tau - \sigma' \tan \phi'\right] = \mu\psi + \ln C \quad (6)$$

Using the linearised Eq. 6, multiple regression analyses were conducted by considering arbitrary φ′. For arbitrary φ′ values, simple linear regression analyses were conducted for data sets of $\ln[\tau - \sigma' \tan \phi']$, and ψ to determine the φ′ with the highest coefficient of determination, as shown in Fig. 10.

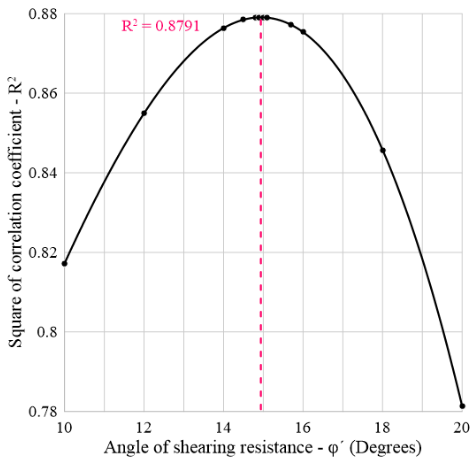

Fig. 10 Variation in square of correlation coefficient of Eq. 6 with respect to the varied value of φ′

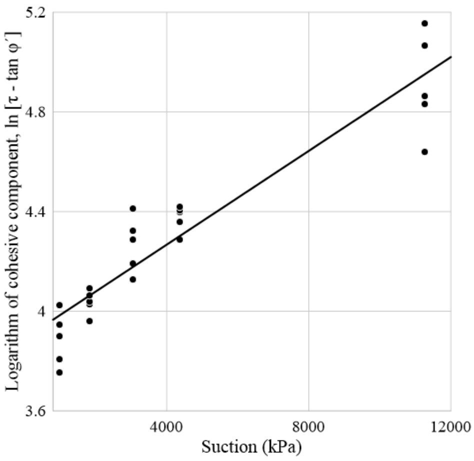

Fig. 11 The relationships between the logarithm of cohesive shear strength and suction

The shear strength of test material was well represented by the below regression function $\tau = \sigma' \tan 14.9^0 + 56.61e^{3.89\psi}$ as depicted in Fig. 11 (R^2 = 0.8791). The regression variables in the equations include the liquefaction characteristics of the soils, effects of lubrication and susceptibility to changes in pore-water pressure during the shear deformation. For this reason, the values should be considered purely as empirical parameters for the soils, each of which has inherent geotechnical behaviours.

CONCLUSIONS

The cohesive strength of an unsaturated expansive soil was formulated as an exponential function of suction. In the formulation, shear strength τ was expressed as $\boldsymbol{\tau = \sigma' \tan \phi' + Ce^{\mu\psi}}$; where σ´ is net normal stress, φ´ is the effective angle of shearing resistance, C is minimum cohesion, μ is a susceptibility coefficient, and ψ is suction of soil. An advantage of this formulation is that all the parameters required are available without any extensive soil testing. The variables can be obtained by a basic shear test and a subsequent regression analysis. It is considered that this empirical method provides a convenient alternative for engineering practice.

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