Application of total stress analysis for unsaturated soils in engineering practice

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ABSTRACT

The effective stress approach (ESA) is commonly used in the interpretation of the engineering behavior of unsaturated fine-grained (UFG) soils in comparison to the total stress approach (TSA). This is because of lack of a valid framework for extending the TSA in rationally explaining the UFG soils. In this paper, a frame work is presented for extending the TSA for UFG soils in engineering practice. This approach is successfully used in the interpretation of the UFG soils behavior such as the bearing capacity, the undrained shear strength, the critical height of unsupported vertical trench, the slope stability analysis of compacted embankment and the shaft friction of single piles. There is a smooth transition between the TSA for unsaturated and saturated soils. The TSA can provide more reasonable estimates in comparison to the ESA for several practical geotechnical problems. This approach is also simple and needs only the conventional shear strength tests results and the Soil-Water Characteristic Curve (SWCC).

RÉSUMÉ

L'approche de la contrainte effective (ACE) est communément utilisée pour l'interprétation du comportement de sols fins non-saturés (FNS) en opposition avec l'approche de la contrainte totale (ACT). Ceci est dû à l'absence d'un cadre conceptuel valable pour l'utilisation de l'ACT pour expliquer le comportement mécanique des sols FNS. Dans le présent article, un cadre conceptuel est présenté pour l'application de l'ACT pour les FNS dans le génie appliqué. Cette approche est utilisée avec succès pour l'interprétation, dans des sols FNS, de la capacité portante, la résistance au cisaillement non-drainée, la hauteur critique d'une coupe verticale sans support, la stabilité des pentes de remblais compactés, et la résistance de fût de pieux individuels. Il existe une transition graduelle entre l'ACT pour des sols non-saturés et saturés. L'ACT peut fournir des estimés plus raisonnables que l'ACE dans plusieurs problèmes pratiques de géotechnique. Cette approche est aussi simple et ne nécessite que les paramètres de la résistance au cisaillement obtenus de manière conventionnelle ainsi que la courbe de rétention d'eau (CRE).

1 INTRODUCTION

The engineering behavior of unsaturated soils is commonly interpreted by extending the effective stress approach (i.e. c' - ϕ', and ϕ^b; hereafter referred to as ESA) both for coarse- and fine-grained soils. A well established frame work for extending the ESA is available in the literature (Fredlund and Rahardjo 1993). Several dozen of empirical models or procedures have also been developed in the literature during the last twenty years to predict or estimate the shear strength of unsaturated soils using the effective shear strength parameters (i.e. c' - ϕ') and the Soil-Water Characteristic Curve (SWCC) (Vanapalli, 2009). In other words, the focus of research and practical applications has been directed more towards extending the ESA for unsaturated soils. There are however many geotechnical problems in practice that need to be addressed using the total stress approach (hereafter referred to as TSA) rather than the ESA. This is especially true for unsaturated fine-grained (i.e. $I_p \neq 0$; hereafter referred to as UFG) soils. The pore-air can be either in drained or undrained condition depending on the rate of deformation, while the pore-water mostly remains under undrained condition in the UFG soils including compacted fine-grained soils. In other words, the drainage conditions in the UFG soils during the loading and deformation stages can be either undrained or partially drained. Several investigators have provided theoretical approaches for extending the TSA (Fredlund and Rahardjo 1993, Rahardjo and Fredlund 1997, Vanapalli et al. 1999, Vanapalli and Oh 2010). There is however lack of studies and examples in the literature to establish a valid framework to reliably interpret the engineering behaviour of unsaturated soils using the TSA for routine practical applications. In the present study, the TSA has been extended to the bearing capacity, the undrained shear strength, the critical height of unsupported vertical trench, the slope stability analysis of a compacted embankment and also the shaft friction of the single piles in UFG soils. The focus of this paper is to establish a framework using the TSA for UFG soils, which is similar to the TSA for saturated soils to address several common geotechnical problems in engineering practice.

2 SOIL PROPERTIES

The model footing tests (Vanapalli et al. 2007) to determine the bearing capacity, stability analysis of compacted embankment (Oh and Vanapalli 2010) and model pile tests (Vanapalli and Taylan 2011) were undertaken on compacted Indian Head till (glacial till) and interpreted extending the TSA for the UFG soils.

The natural soil was collected at a site in Saskatchewan, Canada. The soil was first air-dried and subjected to gentle pulverization to separate the individual soil particles. The soil sample was then passed through a 2 mm sieve. The grain size distribution curve and soil properties of the Indian Head till are shown in Figure 1. The prepared soil was carefully mixed with predetermined water contents and placed in sealed polyethylene bags and kept in a moisture controlled box to ensure uniform distribution of water throughout the sample. The samples were taken out of the polyethylene bags when they reached equilibrium conditions and statically compacted in a specially designed tank (300 mm in diameter × 300 mm in height for conducting model footing and model pile tests) or a mould (50 mm in diameter × 100 mm in height for conventional and modified triaxial shear tests and unconfined compression tests).

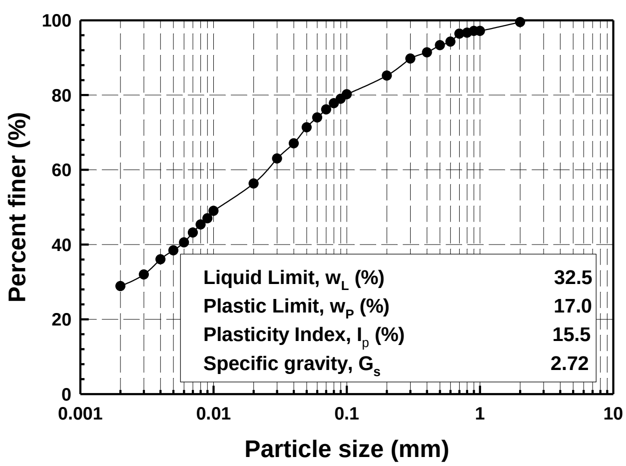

Figure 1. Grain size distribution curve and basic soil properties of Indian Head till.

3 BEARING CAPACITY OF UNSATURATED FINE-GRAINED SOILS

Vanapalli et al. (2007) carried out model footing (B x L = 50 mm x 50 mm) tests in statically compacted UFG soil (i.e. Indian Head till) for five different matric suction values (i.e. 0, 55, 100, 160, 205 kPa) using specially designed equipments. Based on the failure mode (i.e. punching shear failure) observed for different matric suction values, they suggested that the bearing capacity of UFG soils is governed by the compressibility characteristics of the soils below the footing. This implies that the bearing capacity of the model footing in UFG soils can be estimated using the unconfined compression test results as shown in Eq. [1]. The form of Eq. [1] is the same as Skempton (1948) equation used for interpreting the bearing capacity of saturated fine-grained soils under undrained loading conditions (i.e. TSA).

$$q_{ult(unsat)} = \left[\frac{q_{u(unsat)}}{2}\right]\left[1+0.2\left(\frac{B}{L}\right)\right]N_{CW} \qquad [1]$$

where $q_{ult(unsat)}$ = ultimate bearing capacity for unsaturated soil, $q_{u(unsat)}$ = unconfined compressive strength for unsaturated soil, B, L = width and length of footing, respectively, and N_{CW} = bearing capacity factor with respect to constant water content condition.

The back-calculated N_{CW} values using Eq. [1] based on the measured bearing capacity values were between 4.00 and 6.43 and the average value was equal to 5.17. This average value of N_{CW} (i.e. 5.17) is close to the bearing capacity factor proposed by Skempton (1948) for estimating the bearing capacity of saturated fine-grained soils under undrained loading conditions (i.e. 5.14). Figure 2 shows the comparison between the measured bearing capacity values and those estimated using the TSA (i.e. Eq. [1]) along with an inset that shows the indentation from the model footing tests.

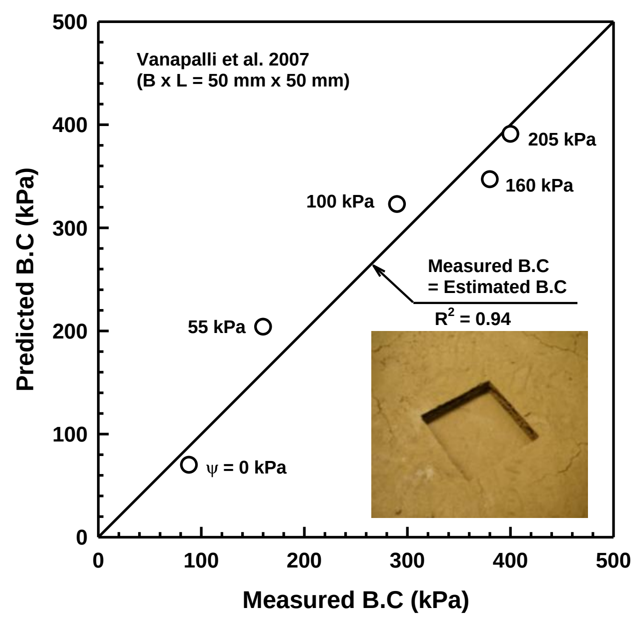

Figure 2. Comparison between the measured bearing capacity values and those estimated using the TSA (date from Vanapalli et al. 2007).

Consoli et al. (1998) conducted in-situ plate load tests in an unsaturated residual homogeneous, cohesive soil (I_p = 20%). The bearing capacity estimated using the ESA were overestimated by 1.5 - 2.5 times compared to the measured values. On the other hand, the bearing capacity values calculated using the TSA (i.e. Eq.[1]) along with the average unconfined compressive strength (i.e. 50.2 kPa) was 155 kPa, which is approximately the same as that of 1 m square concrete footing (i.e. 180 kPa) (Figure 3).

The pore-air can be assumed to be under drained condition while the pore-water is under undrained condition when the UFG soils are loaded. Among the various methods available for estimating the shear strength of unsaturated soils, the constant water content (CW) test is regarded as the most reasonable technique for simulating this loading and drainage condition (Rahardjo et al., 2004). The good agreement between the measured and the estimated values in Figure 3 can be attributed to the reason that the drainage condition for the unconfined compression tests for UFG soils is the same as that of the CW test. The bearing capacity values using the shear strength obtained from the unconfined compression tests can also provide conservative estimates compared to those obtained using CW test results (Vanapalli and Oh 2010).

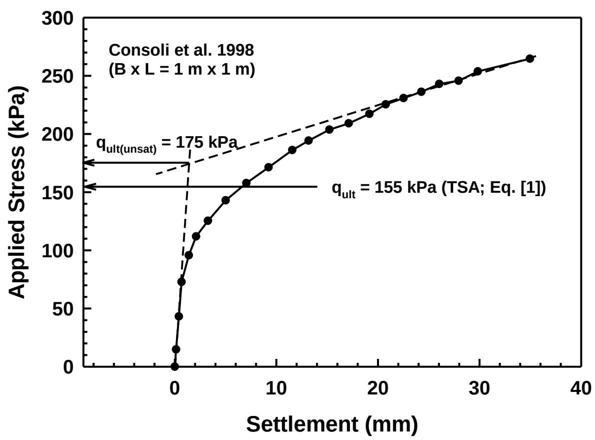

Figure 3. Comparison between the measured bearing capacity values and those estimated using the TSA (data from Consoli et al. 1998).

4 UNDRAINED SHEAR STRENGTH OF UNSATURATED FINE-GRAINED SOILS

The study by Vanapalli et al. (2007) described in Section 3 can be extended to predict the variation of bearing capacity of the UFG soils by estimating the variation of undrained shear strength, $c_{u(unsat)}$ (= $q_{u(unsat)}/2$) with respect to matric suction.

Oh and Vanapalli (2009) extended this concept and proposed a model to predict the variation of shear strength of the UFG soils with respect to matric suction (Eq. [2]). The model uses the shear strength from the unconfined compression tests under saturated condition (i.e. $c_{u(sat)}$) and the SWCC.

$$c_{u(unnsat)} = c_{u(sat)}\left[1+\frac{(u_a - u_w)}{(P_a/101.3)}(S^{\nu})/\mu\right] \qquad [2]$$

where $c_{u(sat)}$, $c_{u(unsat)}$ = shear strength under saturated and unsaturated condition, respectively, P_a = atmospheric pressure (i.e. 101.3 kPa) and ν, μ = fitting parameters.

The fitting parameter, ν = 2 is required for UFG soils. Six sets of unconfined compression tests results for the UFG soils reported in the literature ((1) Chen, 1984; (2) Ridley, 1993; (2) Vanapalli et al. 2000; (3) Babu et al., 2005; (4) Pineda and Colmenares, 2005; (5) Vanapalli et al. 2007) were analyzed and a relationship estimating the fitting parameter, μ was developed. Figure 4 shows the relationship between the fitting parameter, μ and plasticity index, I_p on semi-logarithmic scale for the soils used for the analysis. The fitting parameter, μ was found to be constant with a value of '9' for soils for I_p values in the range of 8% and 15.5%. The value of μ however increases linearly on semi-logarithmic scale with increasing I_p value following the relationship as given in Eq. [3] and Figure 4.

$$\begin{aligned} \mu &= 9 && \text{for } 8.0 \le I_p(\%) \le 15.5 \\ \mu &= 2.1088 \cdot e^{0.0903(I_p)} && \text{for } 15.5 < I_p(\%) \le 60.0 \end{aligned} \qquad [3]$$

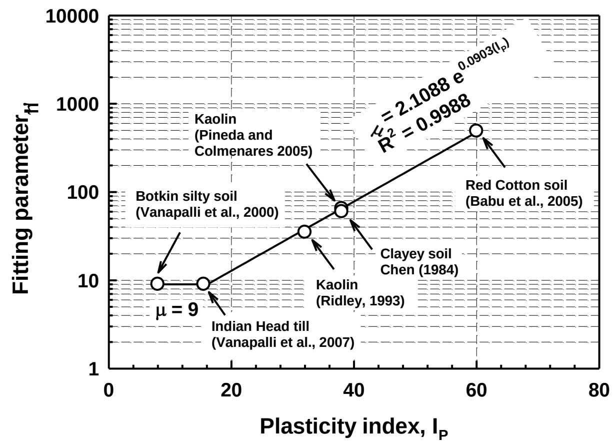

Figure 4. Relationship between plasticity index, I_p and the fitting parameter, μ.

Figure 5 summarizes the comparison between the measured and the predicted undrained strength values using the data presented by Ridley (1993). There is discrepancy between the measured and the estimated shear strength values after a certain suction value (i.e. > 1500 kPa). This behavior can be explained using the differential form of Eq. [2] as shown below.

$$\frac{dc_{u(unnsat)}}{d(u_a - u_w)} = \frac{c_{u(sat)}}{\mu}\left[(S^{\nu}) + (u_a - u_w)\frac{d(S^{\nu})}{d(u_a - u_w)}\right] \qquad [4]$$

The net contribution of matric suction towards shear strength decreases at suction values close to the residual state conditions since the degree of saturation, S is small and the value of $[d(S^{\nu})]/[d(u_a - u_w)]$ in Eq. [4]) is negative (Vanapalli et al. 1996). In other words, the estimated shear strength obtained using the model (i.e. Eq. [2]) starts decreasing at matric suction values close to residual suction value although the measured shear strength continues to increase.

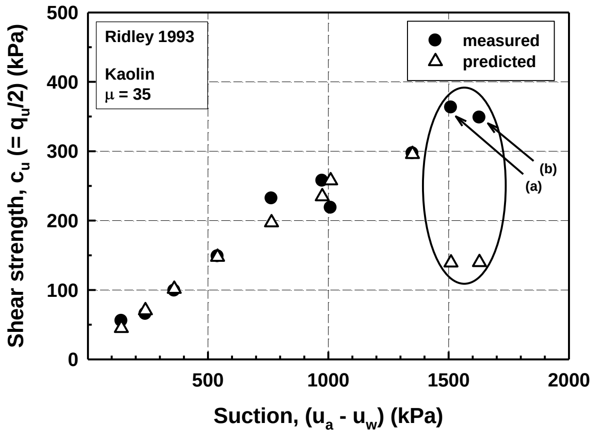

Figure 5. Comparison between the measured and the predicted undrained shear strength (data form Ridley 1993).

5 STABILITY ANALYSIS OF UNSUPPORTED VERTICAL TRECH IN UNSATURATED FINE-GRIANED SOILS

Whenham et al. (2007) studied the stability behaviour of temporary trenches in the natural UFG soils. The trench was excavated in June 2004. The initial local and general failures were observed in January 2005 and February 2005, respectively due to the decrease in matric suction values associated with the precipitation activity (Figure 6). The variation of matric suction values with depth for different cases (i.e. initial, first local failures, general failures) is shown in Figure 7.

The active earth pressure at a certain depth of unsupported unsaturated vertical trench can be calculated using Eq. [5].

$$\sigma_a = \gamma z K_a - 2\sqrt{K_a}\left[c' + u_a - u_w \ \tan\phi^b\right] \qquad [5]$$

where σ_a = active earth pressure and K_a = coefficient of active earth pressure.

Vanapalli et al. (2009) analyzed the stability of the vertical trench cut aforementioned using two different methods by modifying Eq. [5].

Method I:
Fredlund et al. (1996) and Vanapalli et al. (1996) proposed semi-empirical non-linear function to predict $\tan\phi^b$ using the SWCC and fitting parameter, κ (Eq. [6]). The fitting parameter, κ is a function of plasticity index, I_p. (Garven and Vanapalli 2006).

Figure 6. Field tests on the stability of trench in an unsaturated soil deposit (Whenham et al. 2007).

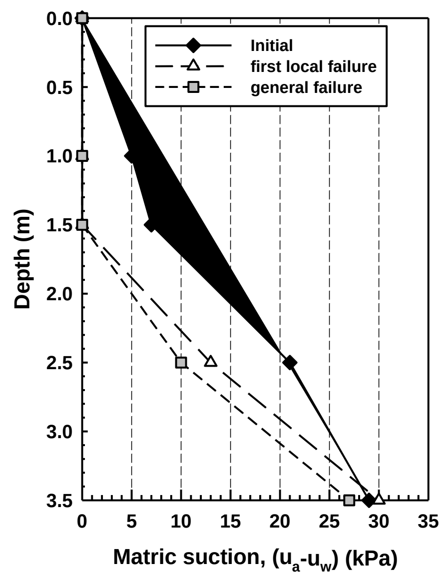

Figure 7. Variation of matric suction with respect to depth for different cases

$$\tan\phi^b = S^\kappa \tan\phi' \qquad [6]$$

where S = degree of saturation.

Substituting Eq. [6] into Eq. [5] yields

$$\sigma_a = \gamma z K_a - 2\sqrt{K_a}\left[c' + u_a - u_w \ \ S^\kappa \ \tan\phi'\right] \qquad [7]$$

Method II:
As addressed in Section 3, the constant water content (CW) test is the most reasonable technique for interpreting the strength behaviour of UFG soils. For CW

tests that are carried out on unsaturated soil specimens, it is commonly postulated that the contribution of matric suction towards shear strength is included in the cohesion term (i.e. c_{CW}). Using this concept, Eq. [5] can be rewritten as Eq. [8]. The shear strength parameters from the CU and CW tests for the soil samples collected at a depth of 1.5m are summarized in Table 1.

$$\begin{aligned}\sigma_a &= \gamma z K_a - 2\sqrt{K_a}\left[c' + \ u_a - u_w \ \ \tan\phi^b\right] \\ &= \gamma z K_a - 2c_{CW}\sqrt{K_a}\end{aligned} \qquad [8]$$

Table 1. Shear strength parameters from CU and CW tests (sampling depth 1.5 m).

Test	Matric suction (kPa)	c' or c_{CW} (kPa)	ϕ' or ϕ (˚)
CU	0	0	32.2
CW	20	19.2	35.7
CW	50	18.1	35.8

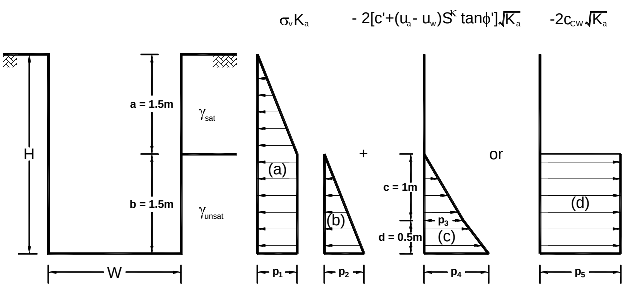

Figure 8. Active pressure distribution diagram (First local and general failure).

Figure 8 shows the distribution of active earth pressures for the case when the first local failures occurred (February, 2005) due to the decrease of matric suction associated with infiltration. The upper 1.5m of the test trench was saturated, but the matric suction at the depth of 3.0m remained unchanged. The equations and values for the pressures, p1, p2, p3, p4, and p5 are summarized in Table 2.

Based on the calculation shown in Table 2, the FS for the cases (a) + (b) + (c) and (a) + (b) + (d) were estimated as 0.31 and 1.14, respectively. The FS obtained using c_{CW} (i.e. 1.14) shows that the failure of the test trench is impending, which ascertains the local failures from the field test. This indicates that the stability analysis using the CW test results provides more realistic estimates when compared with the field observations.

The general failure occurred due to the further decrease in matric suction with depth (i.e. 2.5 m: 13 → 10 kPa; 3 m: 30 kPa → 27 kPa) without the change in the depth of saturation (i.e. 1.5 m) after the local failure was triggered. The active earth pressure distribution diagram for the general failure is the same as the first localized failure. The c_{CW} values both for local and general failure can be assumed to be the same since the suction distribution with depth is approximately the same. In other words, the FS value for the general failure is the same as local failure. Hence, the general failure can be attributed to an increase in the unit weight of the soil associated with an increase in the degree of saturation, which resulted in an increase in the active pressure.

Table 2. Active earth pressure values shown in Figure 8.

Pressure (kpa)	Equation Value (first localized failure/general failure)
P_1	$\gamma_{sat} a K_a$ 9.2/9.2
P_2	$\gamma_{unsat} b K_a$ 8.6/8.6
P_3	$2(u_a - u_w)_{2.5} S^{\kappa}_{2.5} \tan\phi'\sqrt{K_a}$ 7.5/5.7
P_4	$2(u_a - u_w)_{3} S^{\kappa}_{3} \tan\phi'\sqrt{K_a}$ 11.3/9.9
P_5	$2c_{CW}\sqrt{K_a}$ 20.6/20.6

6 STABILITY ANALYSIS OF COMPACTED EMBANKMENT

Oh and Vanapalli (2010) conducted embankment stability analysis using finite element software (SEEP/W and SLOPE/W; GEO-SLOPE International Ltd) for a homogeneous compacted embankment (i.e., height of 3 m and slope of 45°) (Figure 9).

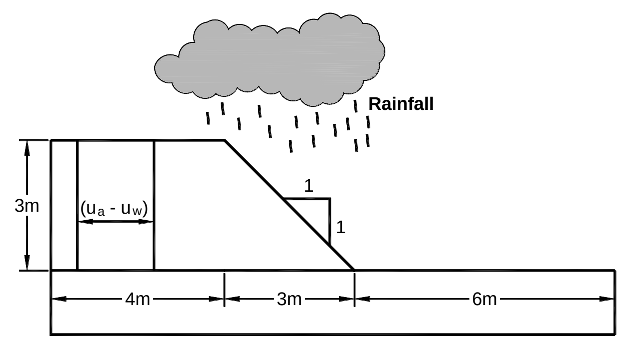

Figure 9. Cross-section of the embankment used in the present study

The analyses were carried out for four different scenarios such as i) conventional stability analysis, ii) ponding condition, iii) stability analysis for unsaturated condition, and iv) short term analysis for unsaturated condition. Table 3 summarizes the details of each scenario (including shear strength parameters used) along with the factor of safety values.

The effective cohesion component of compacted soil is likely to decrease with time since the contribution of matric suction towards effective cohesion decreases and the measured effective cohesion value is dependent on the shearing rate used (Stafford and Tanner 1983). Therefore, the analysis for the Scenario A was performed with two different c' values; i) c' = 36.8 kPa (CU triaxial test) and c' = 5 kPa estimated by Vanapalli et al. (1997) using CD test. The shear strength contribution due to matric suction, ϕ^b was estimated as 12.3°, which corresponds to the initial matric suction value of the compacted embankment at w = 13.2% (Figure 10). The variation of coefficient of permeability with respect to suction (i.e., unsaturated coefficient of permeability function; Figure 11) was predicted using Fredlund and Xing (1994) method using the wetting SWCC (Figure 12) and the saturated coefficient of permeability (i.e. $k_s = 1 \times 10^{-7}$ m/sec). The wetting SWCC was determined using specially designed Tempe cell equipment developed at the University of Ottawa based on axis translation technique (Power and Vanapalli 2010).

Table 3. Summary of the slope stability analyses

Scenario (FS)	Details of analysis
A	• Conventional stability analysis of embankment assuming saturated conditions • Assuming no infiltration • Shear strength parameters (test type)
(5.35)	Case 1: c' (= 36.8 kPa), ϕ' (= 23.1°) ($\overline{CU}$)
(1.35)	Case 2: c' (= 5 kPa), ϕ' (= 23.1°) (CD)
B	• Stability analysis of saturated embankment • Assuming infiltration equal to k_s (= 10^{-7} m/sec)
(1.32)	• c' (= 5 kPa), ϕ' (= 23.1°) (CD)
C	Stage 1 • Stability analysis of unsaturated embankment
(10.26)	• c' (= 5 kPa), ϕ' (= 23.1°) (CD) and ϕ^b (= 12.3°) (MTT) • No infiltration • Long term stability (uniform suction of 200 kPa)
	Stage 2 • Stability analysis of unsaturated embankment
(1.27)	• c' (= 5 kPa), ϕ' (= 23.1°) (CD) and ϕ^b (= 12.3°) (MTT) • Infiltration estimated using permeability function • Long term stability (uniform suction of 200 kPa)
D	• Stability analysis of unsaturated embankment
(10.35)	• c (= 86.9 kPa), ϕ (= 11.2°) (IU) • No infiltration • Short term stability (uniform suction of 200 kPa)

The analysis results suggest that the conventional stability analysis assuming saturated conditions for compacted embankments may not necessarily be a conservative design approach. The main conclusions obtained from the study were as follows:

i) The CU test results can provide a relatively high effective cohesion, c', which results in a higher FS. High cohesion value from the tests may be attributed to the reason that the compacted soils behavior is similar to that of an overconsolidated soil. It is important to use relatively slower shear strain rates for CU tests or CD tests to reliably determine the effective cohesion, c'.

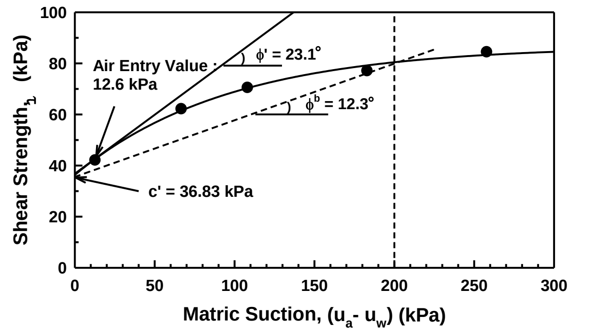

Figure 10. Variation of shear strength with respect to matric suction.

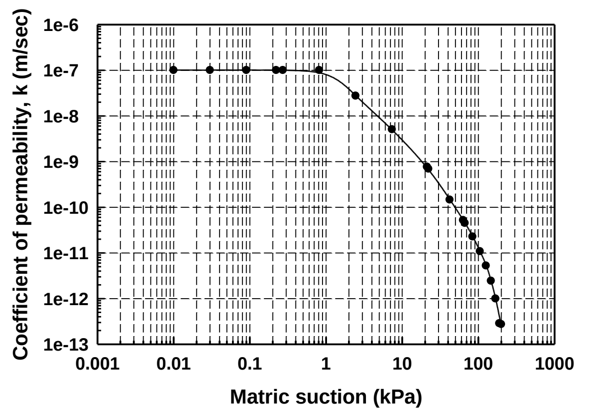

Figure 11. The variation of coefficient of permeability with respect to matric suction of Indian Head till.

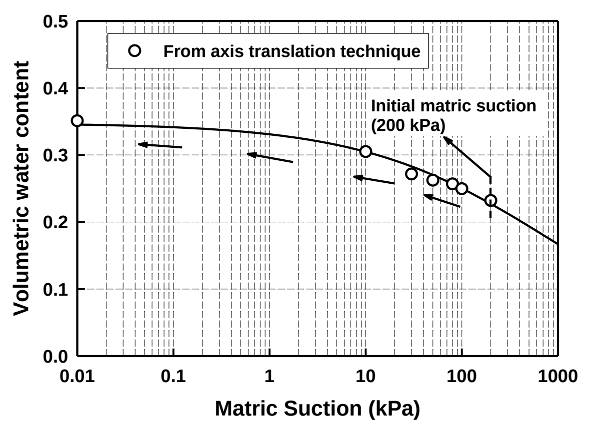

Figure 12. Wetting soil-water characteristic curve of the Indian Head till.

ii) The stability of compacted embankment is dependent on the ponding depth (Scenario B: Ponding condition). The analysis for this scenario should be undertaken using the environmental data information of maximum rainfall for the region where the embankment is constructed.

iii) The results of stability analyses performed using modified triaxial test (MTT) (Scenario C – Stage 1) and IU test (Isotropic confinement undrained test for unsaturated soil) (Scenario D) results without considering infiltration are approximately the same. This result implies that the stability analysis for homogeneous unsaturated embankment may also be carried out using the isotropic confinement undrained tests (i.e. IU) (Oh et al. 2008). In other words, the MTT results which are time consuming are not necessary in analyzing the short term stability using the TSA. The FS values for both these methods are approximately the same. Such behaviour may be attributed to the effect of matric suction being included in the IU results. Studies on other homogeneous compacted embankments using different fine-grained soils along this direction would provide more credence to this conclusion.

7 ESTIMATION OF SHAFT FRICTION OF MODEL PILES

Vanapalli and Taylan (2011) proposed a modified α-method by combining the conventional α-method for undrained loading conditions with the semi-empirical model to estimate the variation of undrained shear strength with respect to matric suction proposed by Oh and Vanapalli (2009) (i.e. Eq. [2]) as shown in Eq. [9].

$$Q_{f(unsat)} = \alpha c_{u(unnsat)}\ \pi dL$$
$$= \alpha c_{u(sat)}\left[1+\frac{(u_a - u_w)}{(P_a/100)}(S^{v})/\mu\right]\pi dL \qquad [9]$$

where $Q_{f(unsat)}$ = ultimate shaft capacity of unsaturated soils, d and L = diameter and length of a pile and α = coefficient.

To check the validity of the modified α-method (Eq. [9]), Vanapalli and Taylan (2011a) conducted a series of model pile tests in compacted UFG soils (i.e. Indian Head till) using the following procedures.

After the soil samples were compacted under static loading conditions in five layers, a thin wall sampling tube (18.7 mm in outer diameter with 1 mm of wall thickness) was used to create a hole down to a depth of 220 mm. The sampling tube along with the soil column embedded into it was removed out from the compacted soil. The model pile used in the study was made out of stainless solid steel cylindrical rod with 20 mm diameter and 400 mm in length (hereafter referred to as D20 pile). The thickness of the wall was equal to 1 mm. The D20 pile was slightly larger in diameter in comparison to the diameter of sampling tube (i.e. 18.7 mm). After the borehole drilling was completed, model pile was jacked down to a predetermined depth of 200 mm. A void of 20 mm in length was intentionally left below the pile toe. The pile was loaded at a rate of 1 mm/min.

Table 4 shows the comparison between the measured ultimate shaft capacity values and those estimated using the modified α-method for D20 pile (Eq. [9]; TSA). The adhesion factor, α = 0.9 and 0.75 were chosen for saturated and unsaturated conditions, respectively based on undrained strength values as suggested by Sowers & Sowers (1970). Good agreement was observed between the measured ultimate shaft capacity values and those estimated using the modified α-method (i.e. TSA). Only typical results are summarized in this paper. More details are available in another paper presented in Vanapalli and Taylan (2011a and 2011b)

Table 4. Comparison between the measured ultimate shaft capacity values and those calculated using modified α-method for D20 pile.

ψ (kPa)	S (%)	α	c_u (kPa)	Cal. $Q_{f(unsat)}$ (kN)	Meas. $Q_{f(unsat)}$ (kN)
0	43	0.90	11.5	0.13	0.10
203	96	0.75	68.0	0.60	0.68

8 SUMMARY

The effective stress approach (ESA) for unsaturated soils requires the saturated shear strength parameters (i.e. c' - ϕ') from conventional tests and angle of internal friction due to matric suction, ϕ^b from the modified shear strength on unsaturated soil specimens tested under drained condition with respect to both the pore-air and the pore-water. However, the interpretation of the engineering behaviour of the UFG soils using the shear strength parameters under drained condition may not be reasonable since typically the pore-air is under drained condition while the pore-water is under undrained condition when the UFG soils are loaded. Among the various methods available for estimating the shear strength of unsaturated soils, the constant water content (CW) test is regarded as the most reasonable technique for simulating this loading and drainage condition. However, determination of the shear strength parameters using the CW tests is time-consuming and requires elaborate testing equipment. Due to this reason, various attempts have been made by the authors to interpret the engineering behaviours of UFG soils using simple unconfined compression test or Isotropic confinement undrained test (IU) results for UFG soils instead of CW test results. This approach (i.e. total stress approach; TSA) can be justified based on two reasons; i) the drainage condition for the unconfined compression tests for UFG soils is the same as the CW test and ii) the influence of matric suction on the engineering behaviour of UFG soils is included in the unconfined compression and IU test results. In other words, there is no need of controlling or measuring matric suction values during experiments by using these test results. In the present study, the TSA was extended to interpret the engineering behaviour of unsaturated soil, which includes i) bearing capacity, ii) undrained shear strength, iii) critical height of unsupported vertical trench, iv) slope stability analysis of compacted embankment and v) shaft friction of model pile. The analyses showed that there was good comparison between the measured values and those estimated extending the TSA. The TSA is encouraging for practicing engineers since it can provide reasonable analyses for several practical problems.

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