A simple method to estimate the bearing capacity of unsaturated fine-grained soils

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ABSTRACT
The bearing capacity of unsaturated fine-grained soils is conservatively estimated extending Terzaghi (1943) bearing capacity theory (i.e. effective stress approach) ignoring the influence of suction. The bearing capacity theory proposed by Skempton (1948) (i.e. total stress approach) is not used for unsaturated fine-grained soils due to the uncertainties associated with the drainage conditions of pore-air and pore-water pressure. However, recent studies by Vanapalli et al. (2007) show that the bearing capacity of unsaturated fine-grained soils can be interpreted reasonably well extending Skempton (1948) equation (i.e. $\phi_u = 0$ approach) using unconfined compression tests results. This means the variation of the bearing capacity with respect to suction for unsaturated fine-grained soils can be estimated if the variation of unconfined compressive strength for unsaturated soils (i.e. $q_{u(unsat)}$) with respect to suction can be predicted. In the present study, a simple model is proposed to predict the variation of shear strength of unsaturated fine-grained soils (i.e. $c_{u(unsat)}$ (= $q_{u(unsat)}/2$)) using the shear strength from the unconfined compression test results of saturated soil specimens (i.e. $c_{u(sat)}$ (= $q_{u(sat)/2}$)) and the soil-water characteristic curve. The research studies presented in this paper show that the variation of unconfined compressive strength with respect to suction can be reasonably well predicted for a variety of fine-grained soils (i.e. $8 \le I_p \le 60$).

RÉSUMÉ
La capacité portante d'un sol à grains fins est estimée de façon conservatrice en prolongeant la théorie de la capacité portante de Terzaghi (1943) (c'est-à-dire l'approche de la contrainte effective). La théorie de la capacité portante proposée par Skempton (1948) (c'est-à-dire l'approche de la contrainte totale) n'est pas utilisée due aux incertitudes associées aux conditions de drainage de la pression de l'air et de l'eau interstitiels. Cependant, de récentes études par Vanapalli et al. (2007) démontrent que la capacité portante de sols non-saturés à grains fins peut être interprétée raisonnablement bien en utilisant l'équation de Skempton (1948) (c'est-à-dire $\phi_u = 0$) en utilisant des résultats d'essais en compression non-confinée. Ceci implique que la variation de la capacité portante en fonction de la succion pour des sols à grains fins non-saturés peut être estimée si la variation de de la résistance en compression non-confinée (c'est-à-dire qu(non-sat)) en fonction de la succion peut être prédite. Dans la présente étude, un modèle simple est proposé pour prédire la variation de la résistance au cisaillement pour des sols non-saturés à grains fins (c'est-à-dire $c_{u(non\text{-}sat)}$ (= $q_{u(non\text{-}sat)}/2$)) en utilisant la résistance au cisaillement des résultats d'essais en compression non-confinée de spécimens de sols saturés (c'est-à-dire $c_{u(non\text{-}sat)}$(= $q_{u(sat)}/2$)) et la courbe de rétention d'eau. L'étude présentée dans cet article démontre que la variation de résultats d'essais en compression en fonction de la succion non-confinée peut être raisonnablement prédite pour une variété de sols à grains fins (c'est-à-dire $8 \le I_P \le 60$).

1 INTRODUCTION

Terzaghi (1943) bearing capacity theory (i.e. effective stress approach) is used for interpreting model footing or in-situ plate load tests results in unsaturated fine-grained soils ignoring the influence of suction (Oloo et al., 1994; Costa et al., 2003). The use of this approach cannot be fully justified due to the following two reasons. Firstly, there is a certain degree of uncertainty with respect to interpreting the bearing capacity of unsaturated fine-grained soils since the drainage conditions of pore-air and pore-water during loading and shearing stages cannot be clearly defined. Secondly, the bearing capacity equation originally proposed by Terzaghi (1943) is based on the general shear failure criteria assuming drained loading conditions. For most cases in unsaturated fine-grained soils well-defined general shear failure is not observed both for model footing or in-situ plate load tests from the stress versus settlement relationships (Oloo, 1994; Schnaid et al., 1995; Costa et al., 2003; Rojas et al., 2007; Vanapalli et al. 2007).

Schnaid et al. (1995) performed in-situ plate load tests in unsaturated fine-grained soils to investigate the variation of bearing capacity using different footing sizes (i.e. 0.3, 0.45, 0.6, 0.7, and 1 m). The well-defined general shear failure conditions were not observed from the stress versus settlement relationships. In addition, a well-defined shear zone at the side of the footing was not observed and no heave occurred on the ground surface. These facts indicate that the failure mechanism below the plate was governed more by a punching shear failure. The bearing capacity values estimated by extending the conventional effective stress approach proposed by Terzaghi (1943) were 4 to 6 times higher than the measured values. Schnaid et al. (1995) re-estimated the bearing capacity values by reducing the effective shear strength parameters by two-thirds of the initial values, which is the conventional approach for interpreting local shear failure conditions (Terzaghi, 1943). This approach was extended as there is no other approach available for interpreting the punching shear failure conditions. There was good agreement between the measured and the estimated bearing capacity values using reduction factors approach proposed by Terzaghi (1943). Schnaid et al. (1995) discussing their results stated that the good agreement between the measured bearing capacity values and those estimated using reduction factors approach was a 'surprising' case. Therefore, this approach cannot be generalized for all unsaturated fine-grained soils. In other words, more investigations are necessary to verify whether the reduction factor approach (i.e. assuming local shear failure conditions) can be applied to all types of fine-grained soils and suction values.

Recently, Vanapalli et al. (2007) proposed a simple method to predict the variation of the bearing capacity of unsaturated fine-grained soils using unconfined compression tests results of unsaturated soil specimens. The proposed equation takes the same form as Skempton (1948) equation for interpreting the bearing capacity of saturated soils under undrained loading condition. This approach was tested on a series of model footing tests results conducted on statically compacted fine-grained soil (i.e. Indian Head till) and there was good agreement between the measured and the predicted bearing capacity values.

The study by Vanapalli et al. (2007) suggests that the bearing capacity of unsaturated fine-grained soil can be estimated using only the shear strength from unconfined compression tests results for unsaturated soils (i.e. $c_{u(unsat)}$). In other words, the variation of bearing capacity values for unsaturated fine-grained soils with respect to suction can be predicted by estimating the variation of shear strength, $c_{u(unsat)}$ (= $q_{u(unsat)}/2$) with respect to suction. This concept was extended in the present study, and an equation is proposed to predict the variation of shear strength with respect to suction using the shear strength from the unconfined compression test results under saturated condition (i.e. $c_{u(sat)}$) and the soil-water characteristic curve (SWCC).

2 BACKGROUND

2.1 Behavior of unsaturated fine-grained soils below footings

The unsaturated fine-grained (hereafter referred as UFG) soils below footings can be interpreted using punching shear failure mechanism (Schnaid et al., 1995) as per the discussions presented in the 'Introduction' section. The slip surfaces below footings are typically not extended to the ground surface but instead restrict to vertical planes as shown in Fig. 1. This characteristic behavior indicates that the bearing capacity of the UFG soils is governed by the compressibility of the soil below a footing (i.e. soil A-A'-B-B' in Fig. 1; hereafter referred as soil block). When the soil block, A-A'-B-B' is compressed due to the stress applied by a footing the soil around the soil block acts as confining pressure. In other words, the bearing capacity of the UFG soils can be represented as a function of a compressive strength of the soil block.

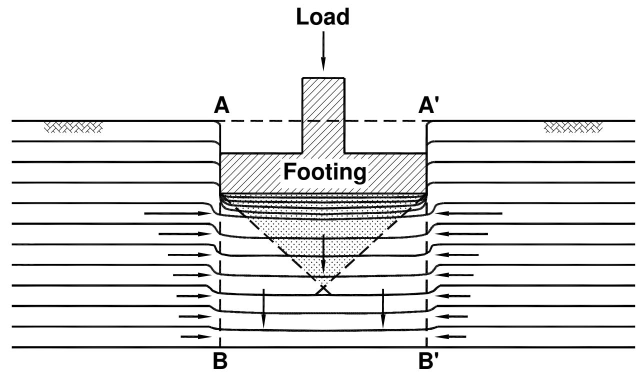

Figure 1. Failure mechanism in unsaturated fine-grained soils below a footing.

An assumption can be made that the pore-air is under drained condition while the pore-water is under undrained condition during the loading of model footings or plate load tests in the UFG soils. This means the pore-air is equal to atmospheric pressure and the water content in the soil is constant throughout the loading stage (Rahardjo et al., 2004).

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, the CW test is time-consuming and needs elaborate testing equipments. Hence, alternatively the shear strength from unconfined compression tests for the UFG soils can be used instead of the conventional CW tests results. This approach can be justified based on the following facts and reasonable assumptions summarized below:

(i) The drainage condition for unconfined compression (UC) test for the UFG soils is the same as the CW test (i.e. pore-air pressure is atmospheric and the water content is constant throughout the test).
(ii) The shear strength increases with an increasing confining pressure for the same matric suction values for the CW test (Rahardjo et al., 2004). Therefore, the shear strength obtained from the unconfined compression test typically provides conservative estimates.

The stress state at failure for the soil block below a footing (Fig. 1) can be derived from the unconfined compression test represented as a unique Mohr circle (Fig. 2). As discussed previously, the bearing capacity of the UFG soils can be expressed as a function of the shear strength from unconfined compression tests. This approach is similar to Skempton (1948) bearing capacity theory (i.e., ϕ_u = 0 approach) to estimate the bearing capacity of saturated soils under undrained loading condition. Extending this concept, the bearing capacity of the UFG soils can be interpreted using Eq. [1].

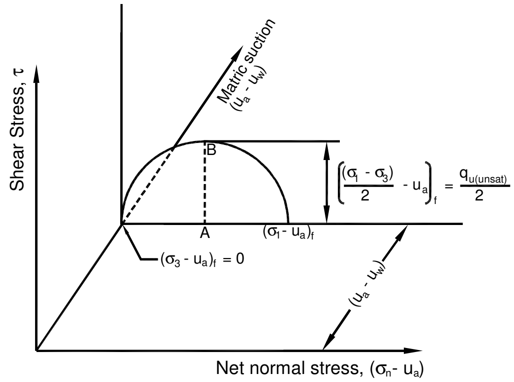

Figure 2. Three-dimensional representation of an unconfined compression test result expressed in terms of stress state variables.

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

where:

$q_{ult(unsat)}$ = ultimate bearing capacity for an unsaturated soil
$q_{u(unsat)}$ = unconfined compressive strength for an unsaturated soil
N_{CW} = bearing capacity factor with respect to constant water content condition
ξ_{CW} = shape factor with respect to constant water content condition.

Justification for estimating the bearing capacity of the UFG soils extending ϕ_u = 0 approach (Skempton, 1948) can be based on the following two reasons:

(ii) The pore-water pressure is under undrained loading condition when a specimen is sheared under constant water content (CW) condition.
(iii) The estimated bearing capacity values extending ϕ_u = 0 approach showed more reasonable results for the data from Costa et al. (2003) in comparison to using the effective stress approach (Oh and Vanapalli, 2009).

Eq. [1] can be rewritten by including the shape factor proposed by Meyerhof (1963) and Vesić (1973) for ϕ_u = 0 condition as below.

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

where:

B,L = width and length of footing, respectively

2.2 Model footing and unconfined compression tests in unsaturated fine-grained soils

Vanapalli et al. (2007) carried out model footing (B x L = 50x50 mm) and unconfined compression tests on statically compacted the UFG soils for five different suction values (i.e. 0, 55, 100, 160, 205 kPa) using specially designed equipments to check the validity of Eq. [2]. The equipments consist of i) high strength plastic tank (HSPT) to compact soil samples and conduct model footing tests and ii) compactor. The experiments were performed by following the procedures shown in Fig. 3. More details on the equipments and testing program are described in Vanapalli et al. (2007).

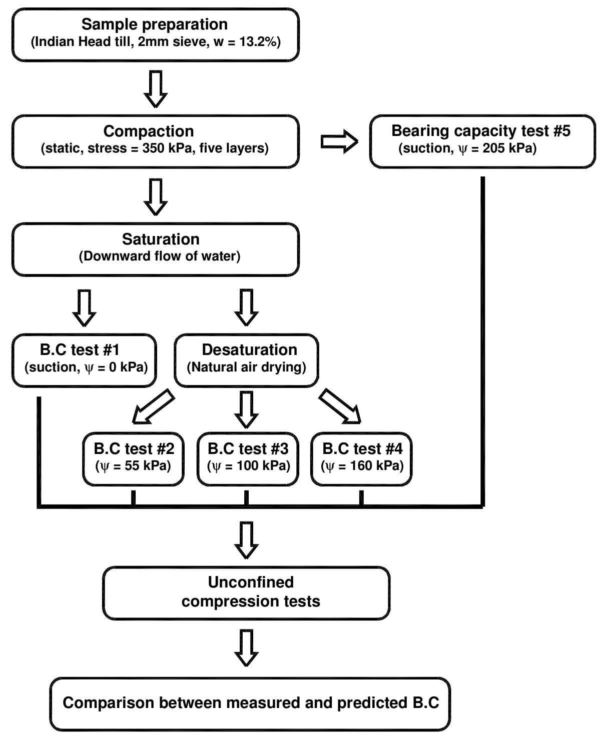

Figure 3. Flow chart of the testing program conducted (Vanapalli et al., 2007).

2.3 Model footing/unconfined compression test results

Fig. 4 shows the model footing test results carried out on both the saturated and the unsaturated soil samples. The results suggest that the stress versus settlement relationships do not reflect well defined general shear failure conditions. The indentation from the model footing tests is also shown in Fig. 4 as an inset, which clearly indicates the typical mode of punching shear failure. This observation is consistent with the discussion in section 2.1 and the in-situ load tests results by Schnaid et al. (1995).

2.4 Estimation of the bearing capacity factor, N_{CW}

Table 1 summarizes the model footing/unconfined compression tests results and the estimated N_{CW} values for each suction value. The back-calculated N_{CW} values using Eq. [2] were between 4.23 and 6.11 and the average value was equal to 5.23. The parameter N_{CW} value is close to the bearing capacity factor of 5.14 proposed by Skempton (1948) for estimating the bearing capacity of saturated fine-grained soils under undrained loading conditions.

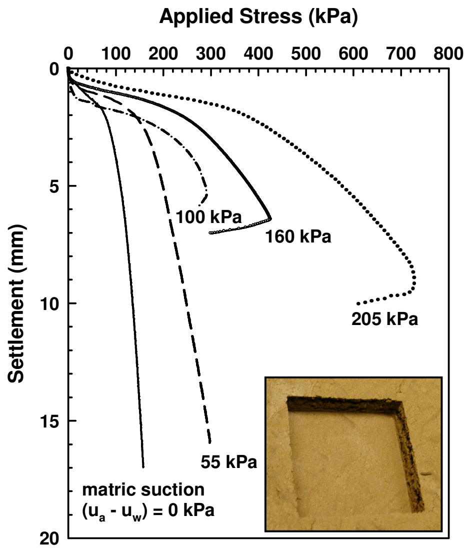

Figure 4. Bearing capacity tests results (modified after Vanapalli et al., 2007).

Table 1. Comparison between the measured and predicted bearing capacity values

ψ[1] (kPa)	Measured B.C[2] (kPa)	$q_{u(unsat)}/2$[3] (kPa)	N_{CW}[4]	Predicted B.C[5] (kPa)
0	88	11.4	6.11	70
55	168	33.3	4.23	204
100	290	52.7	4.61	323
160	380	56.5	5.63	347
205	425	63.7	5.59	391

[1]Initial matric suction
[2]Measured bearing capacity from model footing test
[3]Shear strength obtained from unconfined compression tests for unsaturated soils
[4]Back-calculated N_{CW} using the proposed equation (i.e. Eq. [2])
[5]Predicted bearing capacity using the proposed equation (i.e. Eq. [2]) with N_{CW} = 5.14

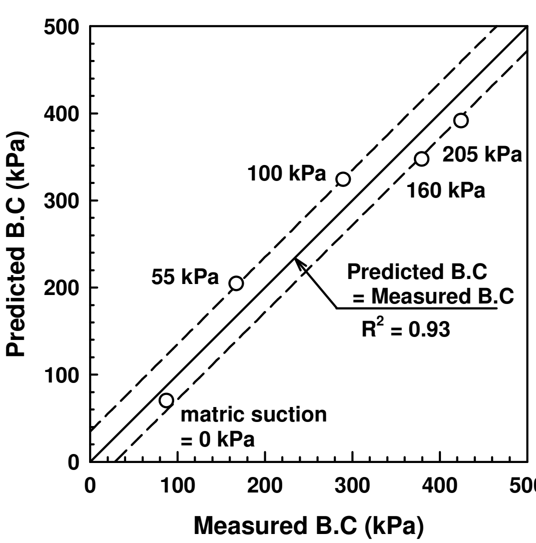

Figure 5. Comparison between the measured and the predicted bearing capacity values (Vanapalli et al., 2007).

Fig. 5 shows the comparison between the measured bearing capacities values from the model the footing tests and predicted values obtained using Eq. [2] with N_{CW} value equal to 5.14. The results show reasonably good agreement. This result indicates that the bearing capacity of the UFG soils can be predicted using unconfined compression test results for unsaturated soils.

3 PREDICTION OF SHEAR STRENGTH WITH RESPECT TO SUCTION FOR UNSATURED FINE-GRAINED SOILS

3.1 Relationship between the SWCC and the modulus of elasticity

Oh and Vanapalli (2008) proposed a model to estimate the modulus of elasticity of unsaturated soils using the SWCC and the modulus of elasticity under saturated condition as given below.

$$E_{unsat} = E_{sat}\left[1+\alpha\frac{(u_a-u_w)}{(P_a/100)}(S^{\beta})\right] \quad [3]$$

where:
E_{sat}, E_{unsat} = modulus of elasticity under saturated and unsaturated condition, respectively
S = degree of saturation
α, β = fitting parameters
P_a = atmospheric pressure (≈ 100 kPa)

In Eq. [3], the terms, S^{β} and α control the nonlinear variation of the modulus of elasticity. The term, $(P_a/100)$ is used for maintaining consistency with respect to dimensions and units on both sides of the equation. The fitting parameter, β is dependent on the soil type (i.e. coarse or fine-grained) and the fitting parameter, α is a function of plasticity index, I_P. The fitting parameter, β equal to 1 and 2 can be used for coarse and fine-grained soils, respectively. Oh and Vanapalli (2008) provided a relationship between the fitting parameter, $(1/\alpha)$ and plasticity index, I_P ($0 \le I_P \le 15.5$).

$$(1/\alpha) = 0.0266(I_P)^2 + 0.1836(I_P) + 0.667 \quad [4]$$

3.2 The relationship between the SWCC and the shear modulus

Oh and Vanapalli (2009) proposed a simple method to predict the variation of shear modulus with respect to matric suction for sandy soils ($0 \le I_P \le 4.9$) using the shear modulus under saturated condition and the SWCC as given below.

$$G_{max(unsat)} = G_{max(sat)}\left[1+\zeta\left(\frac{u_a-u_w}{P_a}\right)(S^{\xi})\right] \quad [5]$$

where:
$G_{max(unsat)}$, $G_{max(sat)}$
= shear modulus under saturated and unsaturated conditions, respectively
ζ, ξ = fitting parameters

For the data analyzed in the study (Picornell and Nazarian, 1998; Kim et al., 2003; Lee et al., 2007), Oh and Vanapalli (2009) suggested that the fitting parameter, ξ value increases from 0.5 to 2.0 as i) uniformity coefficient increases for non-plastic soils and ii) plasticity index increases. This result implies that the fitting parameter, ξ equal to '2' is required to predict the shear modulus of the UFG soils.

3.3 Proposed method for predicting shear strength, $c_{u(unsatu)}$ (= $q_{u(unsat)}/2$)

In the present study, a model is presented to predict the variation of shear strength of the UFG soils with respect to suction using the shear strength derived from unconfined compression test results for saturated condition (i.e. $c_{u(sat)}$) and the SWCC as given below.

$$c_{u(unnsat)} = c_{u(sat)}\left[1+\frac{(u_a - u_w)}{(P_a/100)}(S^{\nu})/\mu\right] \quad [6]$$

where:

$c_{u(unsat)}$, $c_{u(sat)}$ = shear strength under saturated and unsaturated conditions, respectively

ν, μ = fitting parameters

Eq. [6] is similar in form as that of Eqs. [3] and [5] for predicting the variation of modulus of elasticity and shear modulus with respect to suction, respectively. The fitting parameter, ν equal to '2' is used in Eq. [6] by extending the concept discussed in sections 3.1 and 3.2. To obtain the fitting parameter, μ, 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. The basic soil properties (specific gravity, G_s; plasticity index, I_P; optimum moisture content, OMC; maximum dry unit weight, $\gamma_{d(max)}$) for the soils used in this study are summarized in Table 2.

Table 2. Basic physical properties of the soils used for the study

	(1)	(2)	(3)	(4)	(5)	(6)
G_s	2.88	2.61	2.68	2.7	2.61	2.72
I_P	38	32	8	60	38	15.5
OMC (%)	-	-	-	32.5	35	18.3
$\gamma_{d(max)}$ (kN/m^3)	-	-	-	15.35	12.16	17.3

4 ANALYSIS OF THE RESULTS

4.1 Comparison between the measured and the predicted shear strength

Fig. 6 shows the SWCCs for the soils used in the present study. For the data by Chen (1984), the SWCC could not be provided since the experiments were conducted for the specimens compacted at different compaction water contents with the same dry density. Nonetheless, the unconfined compressive strength under saturated condition for each specimen was the same, which implies the effect of the soil structure on the shear strength can be negligible.

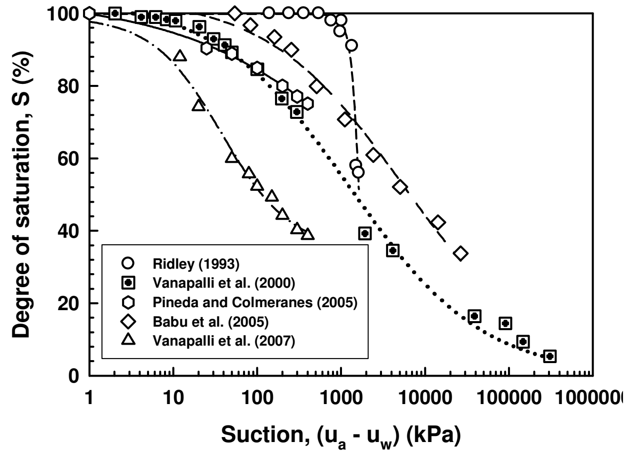

Figure 6. SWCCs for the soils used in the study

Figs. 7 to 11 show the comparison between the measured shear strength from the unconfined compression tests results and the predicted values using the method proposed in the present study (i.e $c_{u(unsat)}$). There is good agreement between the measured and the predicted shear strength values. Explanation with respect to the disagreement for the two points 'a' and 'b' in Fig. 8 is provided in a later section (i.e. section 5).

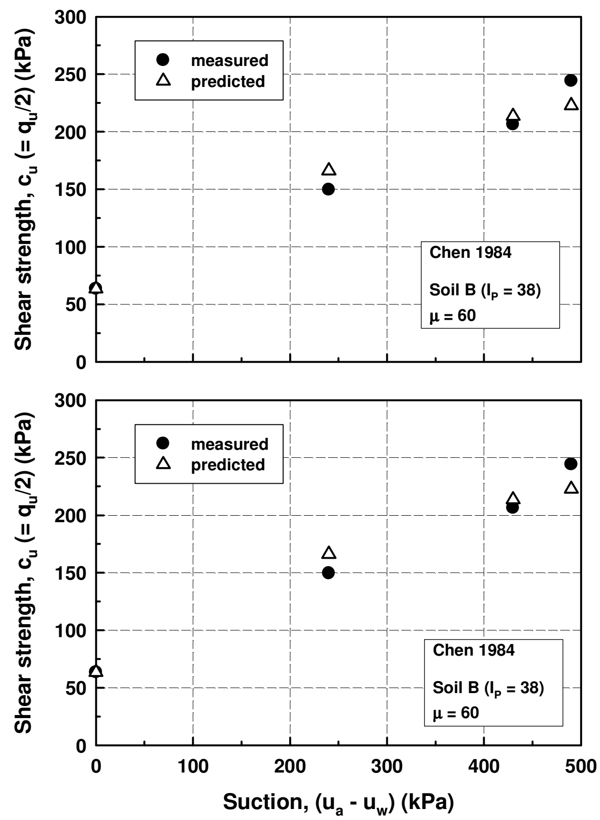

Figure 7. Comparison between the measured and the predicted shear strength (Chen, 1984).

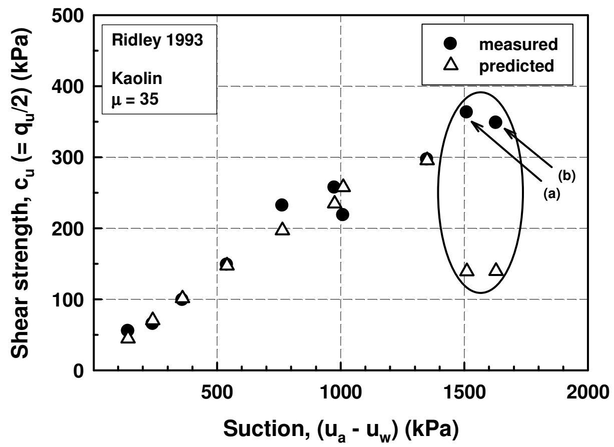

Figure 8. Comparison between the measured and the predicted shear strength (Ridley, 1993).

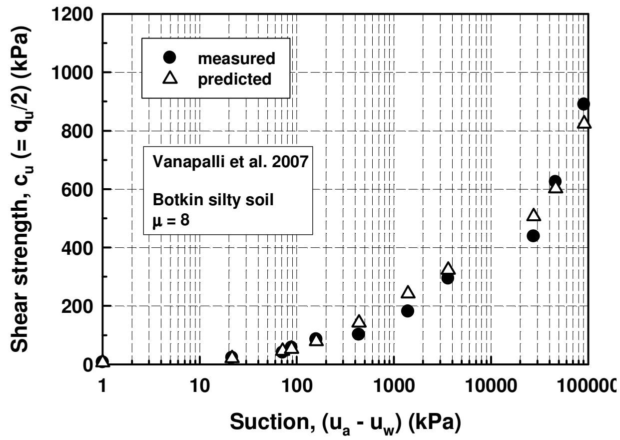

Figure 9. Comparison between the measured and the predicted shear strength (Vanapalli et al., 2000).

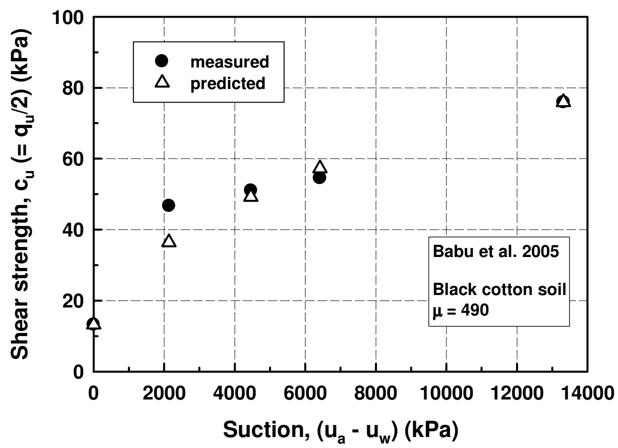

Figure 10. Comparison between the measured and the predicted shear strength (Babu et al., 2005).

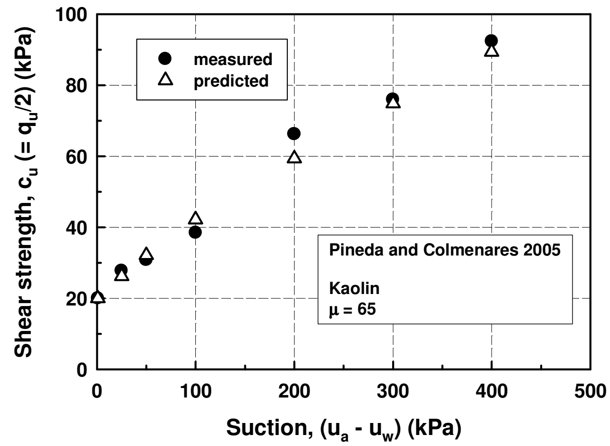

Figure 11. Comparison between the measured and the predicted shear strength (Pineda and Colmenares, 2005).

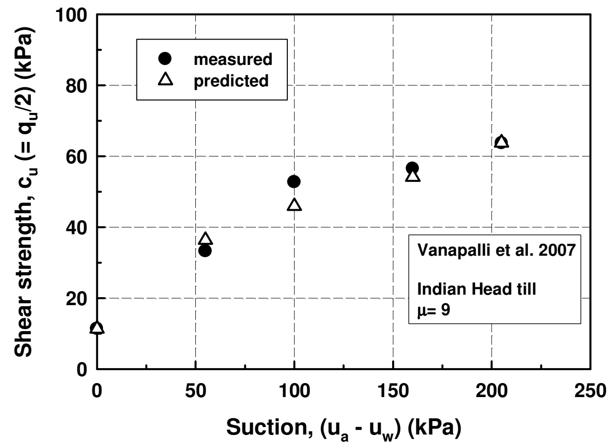

Figure 12. Comparison between the measured and the predicted shear strength (Vanapalli et al., 2007).

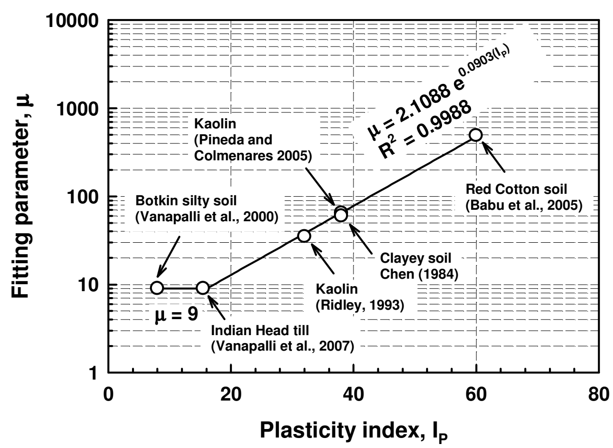

Figure 13. Relationship between plasticity index, I_P and the fitting parameter, μ.

4.2 Relationship between the fitting parameter, μ and plasticity index, I_P

The fitting parameter, μ in Eq. [6] used to estimate the shear strength, $c_{u(unsat)}$ for each data set is summarized in Table 3 and plotted on semi-logarithmic scale in Fig. 13.

Table 3. Fitting parameter, μ for the soils used in this study

	I_P	μ
Vanapalli et al. (2000)	8	9
Vanapalli et al. (2007)	15.5	9
Ridley (1993)	32	35
Chen (1984)	38	60
Pineda and Colmenares (2005)	38	65
Babu et al. (2005)	60	490

The fitting parameter, μ shows constant value of '9' for the plasticity index, I_P between 8 and 15.5 (i.e. low plastic soils). The value of μ then increases linearly on semi-logarithmic scale with increasing plasticity index, I_P following the relationship as given in Eq. [7]. The data from Chen (1984) was not taken into account in developing Eq. [7] due to the reason discussed in section 4.1.

$$\mu = 2.1088 \cdot e^{0.0903(I_P)} \quad (15.5 \le I_P \le 60) \qquad [7]$$

The initial effective stress in the soil specimen for the unconfined compression test can be approximately equal to negative pore-water pressure (i.e. suction value; Eq. [8]) since the pore-air is atmospheric pressure (i.e. $u_a = 0$) (Rahardjo et al., 2004).

$$\sigma' = -(u_a - u_w) \qquad [8]$$

The increment of pore-water pressure during shearing stage for the soils that has more percentage of finer fractions (i.e. higher I_P value) is higher in comparison to the other soil when unconfined compression tests are conducted on two different soils at the same suction value In other words, the effective stress for the soil that has higher plasticity index, I_P value will be less than the other soil. As a result, the higher value of the fitting parameter, μ is required to obtain reasonable estimates since the ratio between the predicted and measured values becomes larger.

5 LIMITATIONS OF THE PROPOSED METHOD

There are two main limitations of the proposed method for predicting the variation of the shear strength, $c_{u(unsat)}$ (= $q_{u(unsat)}/2$) with respect to suction as follows:

a) The relationship between the fitting parameter, μ and plasticity index, I_P shown in Fig. 13 and Eq. [7] is developed with limited data (i.e. six data sets) for a certain range of I_P values (i.e. $8 \le I_P \le 60$). The proposed method can be used in geotechnical engineering practice with greater confidence if more supporting data is available.

b) The results by Ridley (1993) in Fig. 8 show that there is a discrepancy between the measured and the predicted shear strength values after a certain suction value (i.e. > 1,500 kPa). This behavior can be explained using the differential form of Eq. [6] as shown in Eq. [9].

$$\frac{dc_{u(unnsat)}}{d(u_a-u_w)} = \frac{c_{u(sat)}}{\mu}\left[(S^v) + (u_a-u_w)\frac{d(S^v)}{d(u_a-u_w)}\right] \qquad [9]$$

Eq. [9] indicates that at suction values close to the residual state conditions, the net contribution of matric suction towards shear strength decreases since the degree of saturation, S is small and the value of $[d(S^v)]/[d(u_a - u_w)]$ is negative (Vanapalli et al. 1996). In other words, the predicted shear strength obtained using the proposed method in the present study starts decreasing at suction values close to residual suction value although the measured shear strength continues to increase. It can be seen that the SWCC for the soil used by Ridley (Fig. 8) desaturates at a rapid rate, which leads to the fact that the suction values for the point 'a' and 'b' are close to the residual suction value.

The residual suction value of the soil used by Ridley (1993) can be estimated as about 1,500 kPa based on the SWCC in Fig. 8. The movement of water at this suction value is governed by vapor movement for several soils (van Genuchten, 1980).

The tests results for the Kaolin (the plasticity index for the material was not available in the literature) by Aitchison (1957) also showed the similar trend as Ridley (1993)'s data (see Fig. 14).

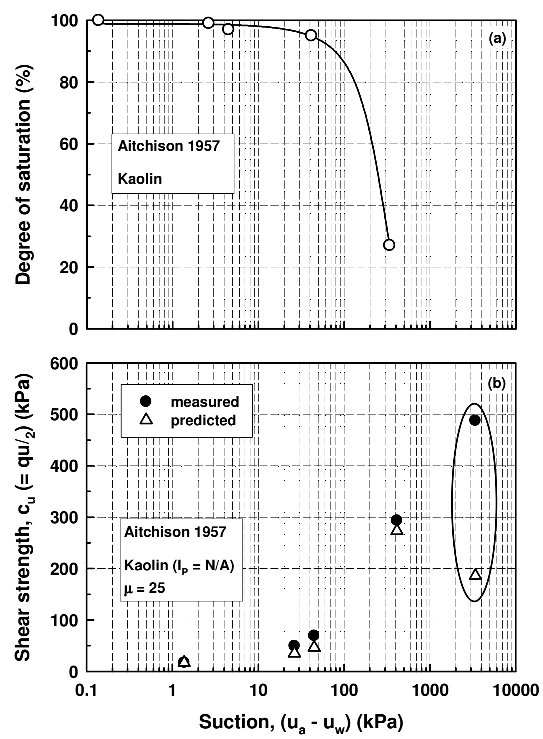

Figure 14. (a) SWCC and (b) comparison between the measured and the predicted shear strength (Aitchison, 1957).

6 SUMMARY AND CONCLUSION

Vanapalli et al. (2007) conducted a series of model footing tests on statically compacted unsaturated fine-grained soils. Based on the tests results, they suggested that the bearing capacity of unsaturated fine-grained soils can be reliably estimated using only unconfined compression tests results extending Skempton (1948) bearing capacity theory.

An equation is proposed in the present study to predict the variation of shear strength (derived from unconfined compression tests) with respect to suction for the unsaturated fine-grained soils ($8 \le I_P \le 60$) using the unconfined compression test results under saturated condition and the Soil-Water Characteristic Curve (SWCC).

There was good agreement between the measured shear strength from the unconfined compression tests for unsaturated fine-grained soils and the predicted values using the method proposed in the present study.

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