Bearing capacity of low plastic unsaturated soils using effective and total stress approaches

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ABSTRACT
Effective stress approach (ESA) originally proposed by Terzaghi (1943) for estimating the bearing capacity of saturated soils was modified by Vanapalli and Mohamed (2007) to estimate the variation of bearing capacity of unsaturated coarse-grained soils (I_p = 0%) with respect to matric suction. Total stress approach (TSA) proposed by Skempton (1948) was modified by Vanapalli et al. (2007) to estimate the variation of bearing capacity of unsaturated fine-grained soils (8% ≤ I_p ≤ 60%) with respect to matric suction. The proposed approaches for estimating the bearing capacity of unsaturated soils use the saturated shear strength properties and the Soil-Water Characteristic Curve along with fitting parameters. However, no parametric studies have been undertaken for the soils that have plasticity index values less than 8% (i.e. 0% < I_p < 8%). In the present study, the model footing test results obtained for a silty soil (Botkin silt, I_p = 6%) are analyzed using both the ESA and the TSA to check the validity of the proposed methods for unsaturated soils.

RÉSUMÉ
L'approche de la contrainte effective (ACE) d'abord proposée par Terzaghi (1943) pour estimer la capacité portante des sols a été élargie par Vanapalli et Mohamed (2007) pour estimer la variation de la capacité de sols grossiers non-saturés (I_p = 0%) en fonction de la succion matricielle. L'approche de la contrainte totale (ACT) proposée par Skempton (1948) a été élargie par Vanapalli et al. (2007) pour estimer la variation de la capacité portante de sols fins (8% ≤ I_p ≤ 60%) en fonction de la succion matricielle. Les approches proposées pour l'estimation de la capacité portante de sols non-saturés utilisent les propriétés de résistance au cisaillement du sol sous des conditions saturées et la courbe de rétention d'eau avec des paramètres d'ajustement. Cependant, aucune étude paramétrique n'a été entreprise pour les sols ayant un indice de plasticité de moins de 8% (c.-à-d. 0% < I_p < 8%). Dans la présente étude, les résultats d'essais sur des semelles à échelle réduite obtenus pour un sol limoneux (Botkin silt, I_p = 6%) sont analysés en utilisant aussi bien la méthode ACE que ACT afin de vérifier la validité des méthodes proposées pour des sol non-saturés.

1 INTRODUCTION

Terzaghi (1943) proposed bearing capacity theory for saturated soils extending the effective stress approach (hereafter referred to as ESA). Skempton (1948) suggested that the bearing capacity of saturated fine-grained soils can be more reliably interpreted extending the total stress approach (hereafter referred to as TSA) (i.e. ϕ_u = 0 analysis) based on field observations. These studies imply that the bearing capacity of saturated soils should be estimated taking account of the type of soil (i.e. coarse- or fine-grained) and drainage (i.e. drained or undrained) conditions. However, the bearing capacity of unsaturated soils in engineering practice are typically estimated extending the ESA (Terzaghi, 1943) regardless of the soil type and the drainage condition ignoring the influence of matric suction.

The bearing capacity of unsaturated coarse-grained soils can be reasonably estimated modifying the Terzaghi (1943) approach taking account of influence of matric suction since the drainage condition for both the pore-air and the pore-water are typically under drained loading condition. Vanapalli and Mohamed (2007) proposed such a model and analyzed model footing test results in coarse-grained unsaturated soils (I_p = 0%) (Mohamed and Vanapalli 2006). There was a reasonably good agreement between the measured and the estimated bearing capacity values. The ESA, however, may not be reliable in interpreting the bearing capacity of fine-grained unsaturated soils due to several uncertainties associated with drainage conditions of the pore-air and the pore-water that are dependent on the rate of loading. Vanapalli et al. (2007) conducted model footing tests in an unsaturated fine-grained soil (I_p = 15.5%) and suggested that the bearing capacity of unsaturated fine-grained soils can be more reliably estimated extending the TSA rather than the ESA. In other words, similar to fine-grained saturated soils, the bearing capacity of fine-grained unsaturated soils is also a function of undrained shear strength. Extending this concept, Oh and Vanapalli (2009) proposed a model to estimate the variation of undrained shear strength of fine-grained unsaturated soils (8% ≤ I_p ≤ 60%) with respect to matric suction using two fitting parameters.

However, there are no studies reported in the literature to interpret the bearing capacity of low plastic unsaturated soils that have the plasticity index values less than 8% (i.e. 0% < I_p < 8%). In the present study, the model footing test results obtained for a silty soil (Botkin silt, I_p = 6%; Oloo, 1994) were revisited and analyzed using both the ESA (Vanapalli and Mohamed, 2007) and the TSA (Vanapalli et al., 2007). The analyses results showed that the bearing capacity of low plastic unsaturated soils can be interpreted extending either using the ESA or the TSA. Both these approaches provide approximately the same bearing capacity results because the low plastic fine-grained soils exhibit characteristics that are similar in nature to that of coarse-grained soils.

2 BEARING CAPACITY OF UNSATURATED SOILS

2.1 Effective stress approach, ESA

Mohamed and Vanapalli (2006) conducted a series of model footing tests in a sand for four different matric suction values (i.e. 0, 2, 4, 6 kPa) using two different sizes of footings (i.e. B × L = 10 mm × 100 mm and 150 mm × 150 mm). Based on the experimental results and also analyzing other data in the literature, Vanapalli and Mohamed (2007) proposed semi-empirical model to estimate the non-linear variation of bearing capacity of coarse-grained soils with respect to matric suction extending the ESA (Eq. [1]). The model uses the effective shear strength parameters (i.e. c' and ϕ') and the Soil-Water Characteristic Curve (SWCC) along with a fitting parameter, ψ. There was a good comparison between the measured bearing capacity values and those estimated using Eq. [1] (Figure 1).

$$q_{ult(unsat)} = \left[\begin{array}{l} c' + (u_a - u_w)_b (1 - S^{\psi}) \tan\phi' \\ \quad + (u_a - u_w)_{AVR} S^{\psi} \tan\phi' \end{array} \right] N_c \xi_c + 0.5 B \gamma N_{\gamma} \xi_{\gamma} \qquad [1]$$

where $q_{ult(unsat)}$ = ultimate bearing capacity for unsaturated soils, c', ϕ' = effective cohesion and internal friction angle, respectively, $(u_a - u_w)_b$ = air-entry value, $(u_a - u_w)_{AVR}$ = average matric suction value, S = degree of saturation, γ = soil unit weight, ψ = fitting parameter with respect to bearing capacity, B = width of footing, N_c, N_{γ} = bearing capacity factor from Terzaghi(1943) and Kumbhokjar (1993), respectively, and ξ_c, ξ_{γ} = shape factors from Vesić (1973).

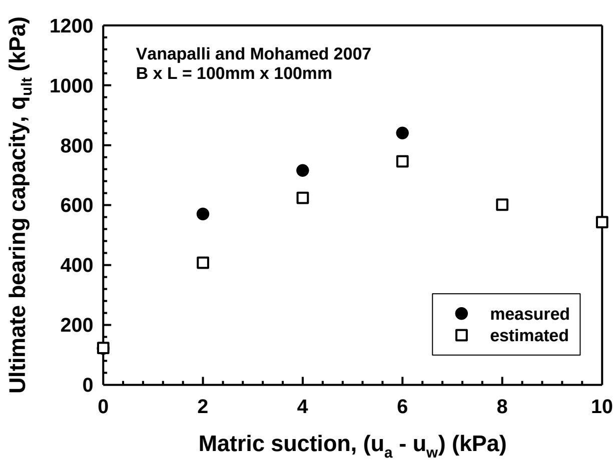

Figure 1. Comparison between the measured bearing capacity values and those estimated extending the ESA (Eq. [1]) (data from Mohamed and Vanapalli 2007).

2.2 Total stress approach, TSA

Vanapalli et al. (2007) carried out model footing (i.e. B × L = 50 mm × 50 mm) tests in an unsaturated fine-grained soil (I_p = 15.5%) for five different matric suction values (i.e. 0, 55, 100, 160, 205 kPa). The failure mode of the model footing tests for the different matric suction values indicated that the slip surfaces below the footing were not extended to the soil surface but instead restricted to vertical planes when a load is applied (i.e. no heave was observed on the soil surface). This characteristic behavior implies that the bearing capacity of the unsaturated fine-grained (hereafter referred to as UFG) soils is governed by the compressibility of the soil block, A-A'-B-B' below a footing as shown in Figure 2. In this case, the soil around the soil block acts as confining pressure and the bearing capacity of an UFG soil can be represented as a function of compressive strength of the soil block.

A methodology was proposed based on this concept to estimate the variation of bearing capacity of UFG soils with respect to matric suction (Eq. [2]).

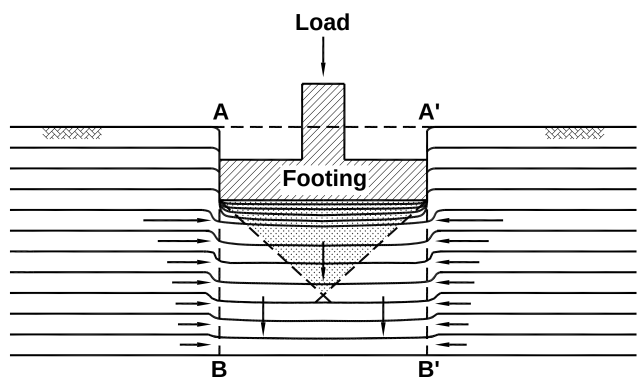

Figure 2. Failure mechanism in unsaturated fine-grained soils below a footing (from Oh and Vanapalli 2010a).

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

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

After comparing the measured and the estimated bearing capacity values, they suggested that the bearing capacity factor, N_c proposed by Skempton (1948) and the shape factor, ξ_c [= 1+0.2(B/L)] proposed by Meyerhof (1963) and Vesić (1973) for saturated fine-grained soils under ϕ_u = 0 condition can also be used for unsaturated fine-grained soils instead of N_{CW} and ξ_{CW}, respectively. Therefore, Eq. [2] can be rewritten as Eq. [3].

$$q_{ult(unsat)} = 5.14 \left(\frac{q_{u(unsat)}}{2} \right) \left(1 + 0.2 \frac{B}{L} \right) \qquad [3]$$

The form of Eq. [3] is the same as Skempton (1948) equation used for interpreting the bearing capacity of saturated fine-grained soils under undrained loading conditions. In other words, the bearing capacity of UFG soils can be estimated extending the TSA. Good agreement was observed between the measured bearing capacity values and those estimated extending the TSA (i.e. Eq. [3]) (Figure 3)

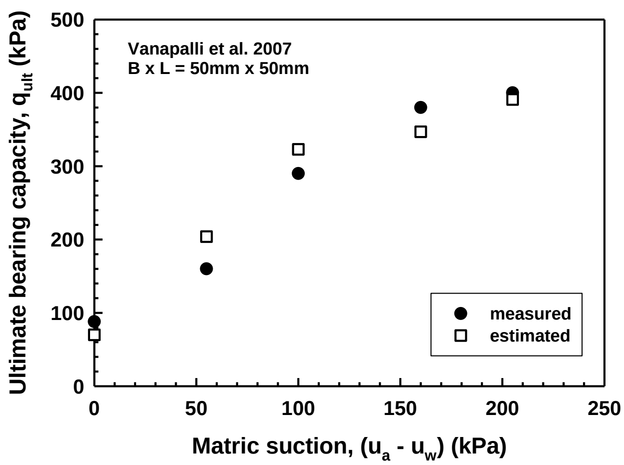

Figure 3. Comparison between the measured bearing capacity values and those estimated extending the TSA (Eq. [3]) (data from Vanapalli et al. 2007).

Eq. [3] also indicates that the bearing capacity of UFG soils can be estimated if the variation of undrained shear strength of unsaturated soils (i.e. $c_{u(unsat)} = q_{u(unsat)}/2$) can be estimated with respect to matric suction. Extending this concept, Oh and Vanapalli (2009) proposed a model to estimate the variation of undrained shear strength of UFG soils with respect to matric suction after analyzing six sets of unconfined compression test results for various UFG soils available in the literature (Eq. [4]). The model uses the unconfined compressive strength under saturated condition and the Soil-Water Characteristic Curve (SWCC) along with two fitting parameters, ν and μ.

$$c_{u(unsat)} = c_{u(sat)}\left[1 + \frac{(u_a - u_w)}{(P_a/100)}(S^{\nu})/\mu\right] \quad [4]$$

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

The fitting parameter, μ was found to be constant value of '9' for soils with 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. [5] and Figure 4.The fitting parameter, ν = 2 is required for UFG soils

$$\begin{aligned} \mu &= 9 && \text{for } 8.0 \le I_p(\%) \le 15.5 \\ \mu &= 2.1088 \cdot e^{0.0903(I_p)} && \text{for } 15.5 < I_p(\%) \le 60.0 \end{aligned} \quad [5]$$

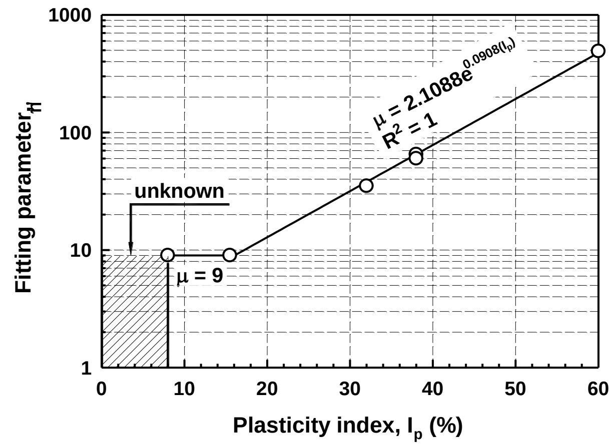

Figure 4. Relationship between plasticity index, I_p and the fitting parameter, μ.

3 TESTING PROGRAM AND SOIL PROPERTIES

Oloo (1994) performed model footing (i.e. B × L = 30 mm × 30 mm and 30 mm in diameter) tests in an UFG soil (Botkin silt, I_p = 6%) for five different matric suction values (i.e. 0, 25, 50, 75, 100 kPa). In the present study, the model footing test results for the square footing were analyzed using both the ESA and the TSA to check the validity of the proposed methods (i.e. Eq. [1] and Eq. [3]) for unsaturated soils.

The experiments were carried out using a specially designed mould equipped with thermal conductivity sensors and ceramic disc (air-entry value = 500 kPa) installed on the cylinder and on the bottom plate, respectively (Figure 5). The thermal conductivity sensors were used to measure matric suction values for the specimens that were statically compacted at different water contents. The specimens compacted at different water contents cannot be regarded as "identical" specimens since each specimen has different density and soil structure. Therefore, the identical specimens at different matric suction values were obtained by using the following procedures.

The specimens compacted at the same water content (i.e. 18%) and density conditions were saturated by allowing water to flow under a small head of water in a temperature and humidity controlled room. The predetermined matric suction values were then achieved by applying air-pressure from the top of the compacted specimens extending the axis-translation technique (Hilf 1956). The model footings were loaded at a rate of 0.18 mm/min after the matric suction values in the compacted specimens reached equilibrium conditions.

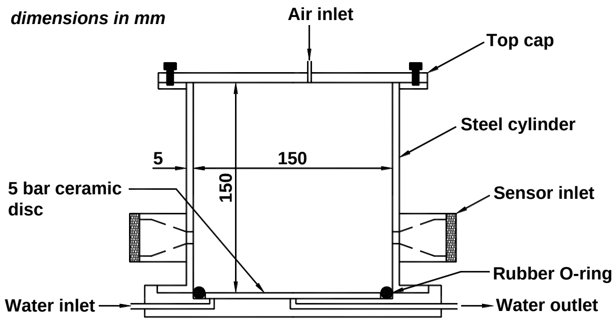

Figure 5. Mould used for compaction and model footing tests (modified after Oloo, 1994).

Figure 6 and Figure 7 and show the grain size distribution curve and the SWCC for the Botkin silt used by Oloo (1994). The reasons associated with showing the grain size distribution curve and the SWCC for the Botkin silt used by Vanapalli et al. (2000) will be discussed later in the paper. The effective shear strength parameters, c' and ϕ' were estimated as 2.5 kPa and 28.1° from consolidated drained direct shear tests (Figure 8).

4 COMPARISON BETWEEN MEASURED AND ESTIMATED BEARING CAPACITY VALUES

4.1 Analysis extending effective stress approach, ESA

4.1.1 General shear failure mode

Figure 9 shows the comparison between the measured bearing capacity values and those estimated using Eq. [1] assuming general shear failure mode. The details are summarized in Table 1.

Vanapalli and Mohamed (2007) suggested that the fitting parameter, ψ in Eq. [1] is a function of I_p and $\psi = 1$ is required for coarse-grained soils after analyzing model footing tests results in three different sands and two fine-grained soils. Extending this concept, Vanapalli and Oh (2010b) analyzed two sets of additional in-situ plate load test results (Costa et al. 2003 (I_p = 8%) and Rojas et al. 2007 (I_p = 12%)) in UFG soils and showed that ψ is constant with a values of 3.5 (Figure 10). Therefore, in the present study, $\psi = 3$ (for I_p = 6%) was used to estimate the variation of bearing capacity with respect to matric suction. As can be seen in Figure 9, the bearing capacity values were significantly overestimated regardless of ψ values (1.5 to 3 times higher for $\psi = 3$) when general shear failure mode is assumed. The discrepancy between the measured and the estimated bearing capacity values increase with decreasing ψ value.

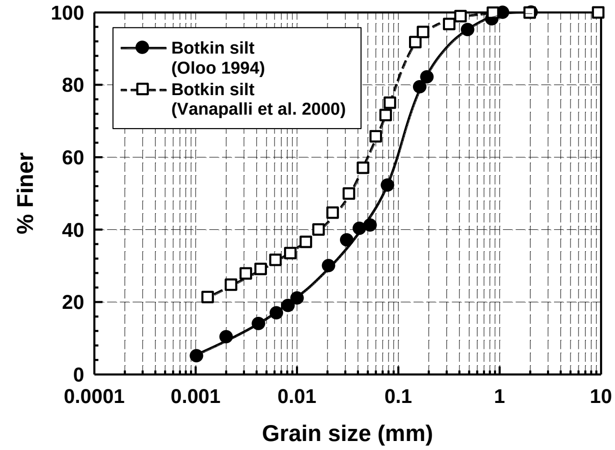

Figure 6. Grain size distribution curves for the Botkin silts used by Oloo (1994, present study) and Vanapalli et al. (2000).

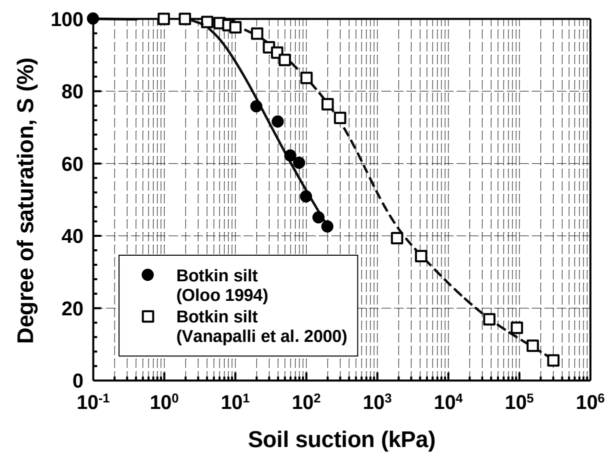

Figure 7. Soil-Water Characteristic Curves for the Botkin silts used by Oloo (1994, present study) and Vanapalli et al. (2000).

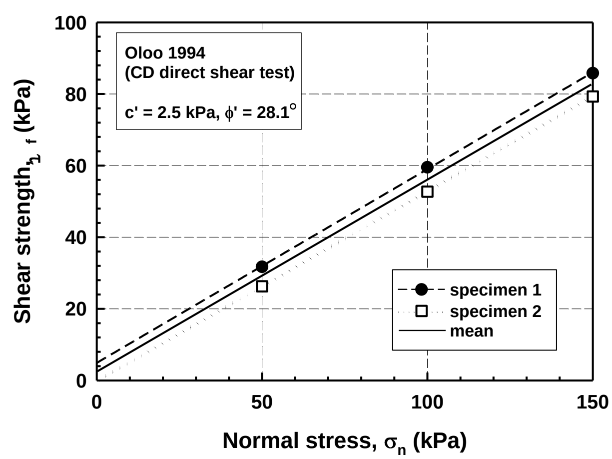

Figure 8. Effective shear strength envelopes for Botkin silt used in the present study.

Table 1. Measured and estimated bearing capacity values extending the ESA assuming general shear failure mode.

$(u_a - u_w)$ (kPa)	Measured (kPa)	Estimated (kPa)		
		$\psi = 1$	$\psi = 2$	$\psi = 3$
0	45	126	126	126
25	250	769	670	595
50	360	1145	874	699
75	450	1444	994	733
100	490	1646	1020	701

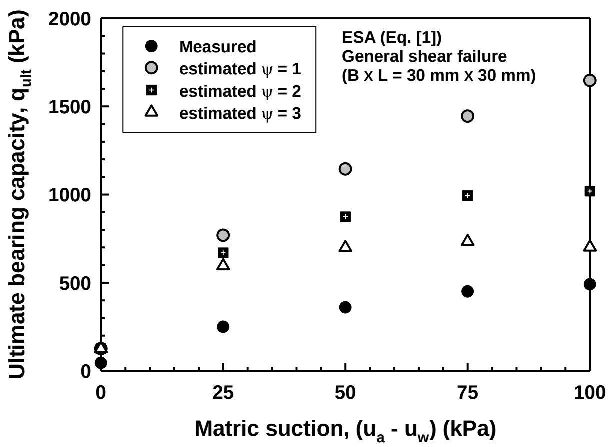

Figure 9. Comparison between the measured and the estimated bearing capacity values extending the ESA assuming general shear failure mode.

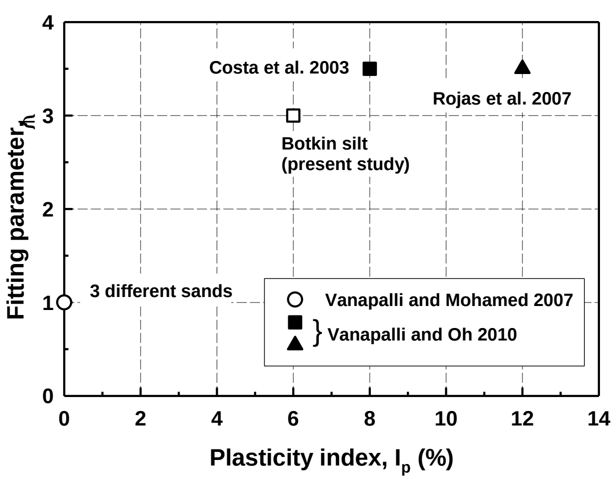

Figure 10. Relationship between fitting parameter, ψ and plasticity index, I_p.

4.1.2 Local shear failure mode

Figure 11 shows the comparison between the measured bearing capacity values and those estimated using Eq. [1] for different ψ values assuming local shear failure mode (i.e. Eqs. [6] and [7]). The details are summarized in Table 2.

$$c^* = 0.67c' \qquad [6]$$

$$\tan\phi^* = 0.67\tan\phi' \qquad [7]$$

where c^*, ϕ^* = modified effective cohesion and internal friction angle for local shear failure mode, respectively

Table 2. Measured and estimated bearing capacity values extending the ESA assuming local shear failure mode.

$(u_a - u_w)$ (kPa)	Measured (kPa)	Estimated (kPa)		
		$\psi = 1$	$\psi = 2$	$\psi = 3$
0	45	41	41	41
25	250	205	174	151
50	360	322	238	183
75	450	415	275	194
100	490	478	283	184

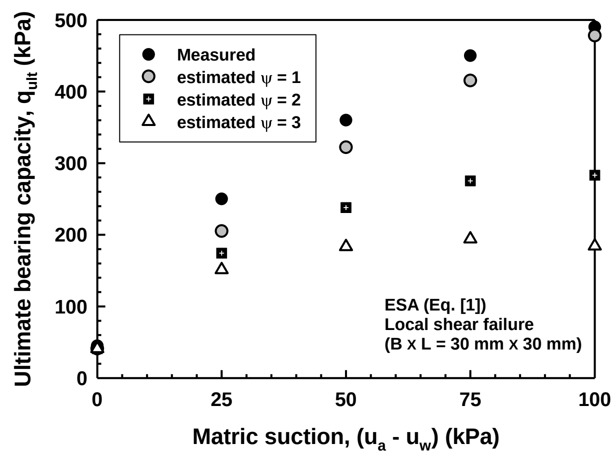

Figure 11. Comparison between the measured and the estimated bearing capacity values extending the ESA assuming local shear failure mode.

Good comparison was obtained when the bearing capacity values were estimated using ψ = 1; on the other hand, the bearing capacity values were significantly under estimated when calculated using ψ = 3.

The results in Figure 9 and Figure 11 show that the best comparison between the measured and the estimated bearing capacity values can be obtained when the bearing capacity values are estimated using ψ = 1 assuming local shear failure mode. The reason that the fitting parameter, ψ = 1 proposed for unsaturated coarse-grained soils (Vanapalli and Mohamed 2007) can also be used for an UFG soil may be attributed to the fact that because the mechanical properties of the low plasticity soils are close to that of coarse-grained soils.

As explained in section 2.2 with Figure 2, the bearing capacity of the UFG soils is governed by the compressibility of the soil block below a footing. This behavior justifies the use of local shear failure mode in interpreting the bearing capacity of UFG soils (i.e. $I_p \neq 0$).

Schnaid et al. (1995) carried out in-situ plate (0.3, 0.45, 0.6, 0.7 and 1 m) load tests in UFG soils. The bearing capacity values interpreted extending the ESA was 4 to 6 times greater than the measured values. The same trend was also observed for the in-situ plate (dia. = 0.8 m) load tests results by Costa et al. (2003). Schnaid et al. (1995) and Costa et al. (2003) also estimated the bearing capacity values with reduced effective shear strength parameters (i.e. local shear failure mode using Eq. [6] and Eq. [7]). Reasonably good agreement between the measured and the estimated bearing capacity values was observed for the results presented by Schnaid et al. (1995); however, the estimated bearing capacity values were still higher than the measured values by 3 to 5 times for the results by Costa et el. (2003). The studies by Schnaid et al. (1995) and Costa et al. (2003) indirectly suggest that using the reduced shear strength parameters may not be applicable for all types of UFG soils and matric suction values.

4.2 Analysis extending total stress approach, TSA

The unconfined compression test results for the saturated condition (i.e. $q_{u(sat)}$) was not available in the literature (i.e. Oloo 1994); therefore, it was back-calculated using the measured bearing capacity value for the saturated condition (i.e. 14.6 kPa). The $q_{u(sat)}$ value for the Botkin silt (I_p = 8%) experimentally determined by Vanapalli et al. (2000) was 13 kPa. The variation of undrained shear strength, c_u with respect to matric suction was estimated using Eq. [4]. As explained in section 2.2, Oh and Vanapalli (2009) suggested that ν = 2 is required for fine-grained soils and μ = 9 can be used for the I_p values in the range of 8% to 15.5%. However, the I_p value of the soil (i.e. I_p = 6%) used in the present study does not fall in this range. Therefore, the analyses were carried out for ν = 1 and 2 with different μ values.

Figure 12 (and Table 3) and Figure 13 (and Table 4) provide the comparison between the measured bearing capacity values and those estimated using Eq. [3] (i.e. TSA) for ν = 1 and 2 with different μ values, respectively. The best comparisons were obtained with the combinations of ν = 1; μ = 5 and ν = 2; μ = 3. This indicates that the bearing capacity of low plastic UFG soils can be estimated using either ν = 1 or 2 with low μ values. This may be attributed to the fact that the mechanical properties of low plasticity soils lie between coarse- and fine-grained soils. Similar trends in results were also observed from the ESA results (see section 4.1 for more details).

5 BEARING CAPACITY OF LOW PLASTIC UNSATURATED FINE-GRAINED SOILS

The analyses carried out extending both the ESA and TSA shows that different fitting parameters are required to estimate the bearing capacity of low plastic UFG soils. The fitting parameters proposed for the ESA and TSA and those obtained from the present study are summarized in Table 5.

Table 3. Measured and estimated bearing capacity values extending the TSA for ν = 1.

$(u_a - u_w)$ (kPa)	Measured (kPa)	Estimated (kPa) (ν = 1)		
		μ = 3	μ = 5	μ = 9
0	45	45	45	45
25	250	326	214	139
50	360	533	338	208
75	450	698	437	263
100	490	807	502	299

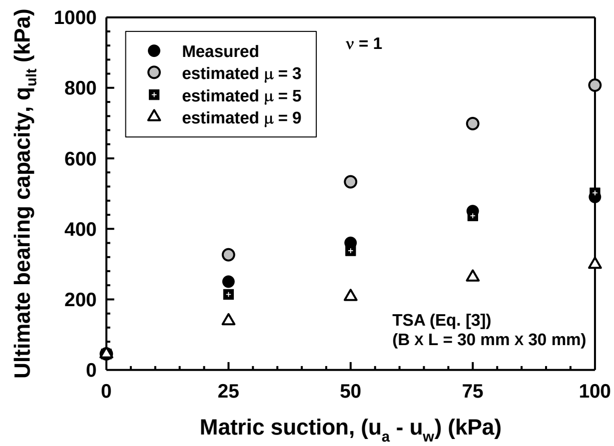

Figure 12. Comparison between the measured and the estimated bearing capacity values extending the TSA (ν = 1 and different μ values).

Table 4. Measured and estimated bearing capacity values extending the TSA for ν = 2.

$(u_a - u_w)$ (kPa)	Measured (kPa)	Estimated (kPa) (ν = 2)		
		μ = 3	μ = 5	μ = 9
0	45	45	45	45
25	250	256	172	115
50	360	362	235	151
75	450	424	272	171
100	490	432	277	174

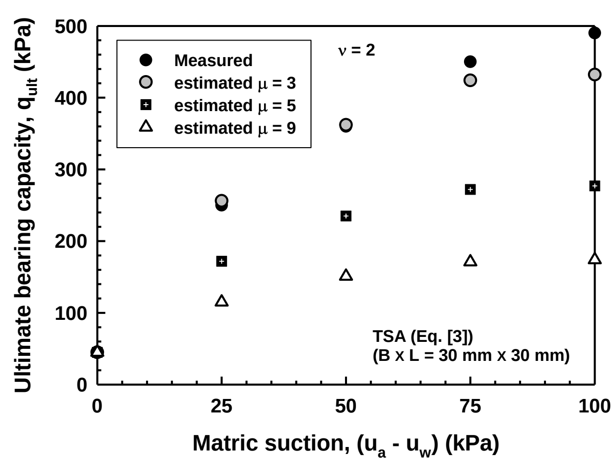

Figure 13. Comparison between measured and estimated bearing capacity values extending the TSA (ν = 2 and different μ values).

Table 5. Proposed and required fitting parameter for the ESA and the TSA.

Reference	Fitting parameter					
	ESA (Eq. [1])		TSA (Eq. [4])			
	ψ_{pro}	ψ_{pre}	ν_{pro}	μ_{pro}	μ_{pre}	μ_{pre}
1)	3	1				
			2	N/A		
2)					1	5
					2	3

1) Vanapalli and Mohamed (2007)
2) Oh and Vanapalli (2009)
pro: proposed, pre: present study

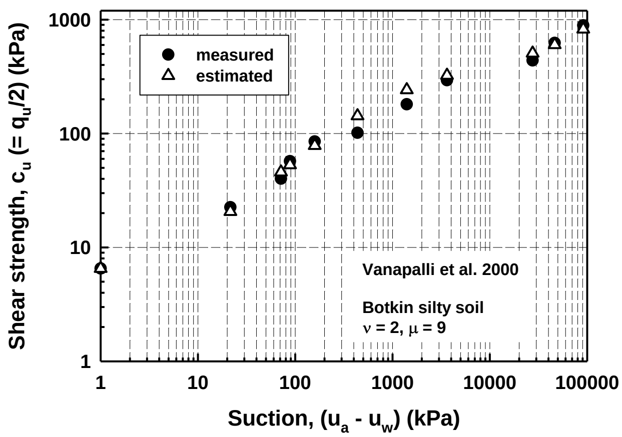

Figure 14. Comparison between measured and estimated unconfined compressive strength of Botkin silt used by Vanapalli et al. (2000) (from Oh and Vanapalli 2009).

In the previous studies, ψ = 3 (ESA) and ν = 2 (TSA) were proposed for the unsaturated fine-grained soils but in the present study ψ = 1 (assuming local shear failure mode) and ν = 1 and 2 (with lower fitting parameter value, μ) provide good comparisons between the measured and the estimated bearing capacity values.

Vanapalli et al. (2000) conducted unconfined compression tests on Botkin silt (hereafter referred to as Botkin_V). The I_p value of the soil was estimated as 8% that is 2% higher compared to the Botkin silt used by Oloo (1994) (hereafter referred to as Botkin_O). This may be attributed to the collection of samples at different periods of time and also due to the different procedures used in the preparation of the natural soil sample collected. As can be seen in Figure 6, the Botkin_V is finer than that of Botkin_O, which leads to higher degree of saturation at the same matric suction values (see Figure 7). Figure 14 shows the comparison between the measured undrained shear strength values and those estimated using Eq. [4] with ν = 2 and μ = 9 for the Botkin_V. There is a good agreement between the measured and the estimated undrained shear strength values. However, as discussed with Figure 13, good comparison between the measured and the estimated bearing capacity values was not observed for the Botkin_O with the same fitting parameter (i.e. ν = 2 and μ = 9). Hence, it can be concluded that the mechanical properties of unsaturated low plasticity soils lie between coarse- and fine-grained soils and the bearing capacity of unsaturated soils of low plasticity can be more reliably estimated using the fitting parameters (i.e. ψ and ν) proposed for coarse-grained soils.

6 SUMMARY AND CONCLUSION

Bearing capacity of unsaturated soils is commonly estimated extending effective stress approach (ESA) regardless of soil types and drainage conditions due to the uncertainties associated with drainage conditions of the pore-water and the pore-air. However, the previous studies showed that the bearing capacity of unsaturated fine-grained (UFG) soils can be more reliably estimated extending the total stress approach (TSA) rather than effective stress approach (Vanapalli et al. 2007).

In the present study (Vanapalli et al. 2007 and Oh and Vanapalli 2009), the model footing test results on unsaturated soil of low plasticity (i.e. I_p = 6%) were analyzed using the methods proposed by Vanapalli and Mohamed (2007) and Vanapalli et al. (2007) (along with the study by Oh and Vanapalli 2009) that are based on the ESA and the TSA, respectively. The analyses results showed that the fitting parameter ψ = 1 is required instead of 3 as per the ESA assuming the local shear failure mode. The bearing capacity of unsaturated soils of low plasticity can also be estimated extending the TSA using the fitting parameters ν = 1 and 2 with μ values less than 9 that was originally proposed by Oh and Vanapalli (2009).

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