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Simple Approaches for The Design of Shallow and Deep Foundations for Unsaturated Soils I: Theoretical and Experimental Studies

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Abstract. Bearing capacity is one of the key properties required in the design of shallow and deep foundations. In conventional engineering practice, simple approaches are widely used for determining the bearing carrying capacity of foundations based on the shear strength parameters of saturated soils. However, foundations are typically placed in part or fully in the soil zone above the natural ground water table, which is in a state of unsaturated condition. The shear strength of unsaturated soils are significantly influenced by the matric suction. The bearing capacity of foundations cannot be reliably determined by extending conventional soil mechanics principles for soils that are in a state of unsaturated condition. This Companion Paper I, introduces how the shear strength can be used as a tool in the interpretation and prediction of the bearing capacity of foundations in unsaturated soils. In addition, both theoretical and experimental studies related to the bearing capacity of unsaturated soils are succinctly summarized. Numerical techniques that can be used for predicting the stress versus settlement behavior used for the design of shallow and deep foundations are summarized in Companion Paper II. The succinctly summarized information in the companion papers are valuable for geotechnical engineers for understanding and implementing the mechanics of unsaturated soils in the design of shallow and deep foundations.

Keywords: matric suction, unsaturated shear strength, bearing capacity, shallow foundations, pile foundations

1 Introduction

The behavior of geotechnical infrastructures such as foundations, pavements, retaining structures and slopesare significantly influenced by the shear strength of soils. Due to this reason, in conventional engineering practice, simple approaches are widely used for the design of shallow and deep foundations based on the saturated shear strength parameters. In many scenarios, foundations are placed either in part or fully in the vadose zone, which is above the natural groundwater table, where the soil is typically in a state of unsaturated condition. However, the design of foundations is based on conventional saturated soil mechanics principles ignoring the contribution of matric suction of soil in unsaturated conditions. Such an approach contributes to unrealistic estimation of the bearing capacity and the settlement behavior, which are key parameters required in the design of foundations (i.e., both shallow foundations and pile foundations).

Fredlund and Morgenstern [1] proposed a rational framework for interpreting the mechanical behavior of unsaturated soils in terms of two independent stress state variables; namely, net normal stress, (σ_n-u_a) and matric suction, (u_a-u_w). In 1978, Fredlund et al. [2] proposed a shear strength relationship extending the Mohr-Coulomb failure criterion for unsaturated soils in terms of two stress state variables. Many experimental studies have been conducted over the last few decades to determine and interpret the shear strength of unsaturated soils *(SSUS)* following this framework [3, 4, 5, 6]. Equation proposed by Fredlund et al. [2] was found to be a valuable tool for explaining the shear strength changes from a saturated condition to unsaturated condition and vice versa. However, experimental studies for determining the *SSUS* need elaborate testing equipment that can be performed only with the assistance of trained professionals. In addition, these tests are time-consuming. Due to this reason, several researchers developed empirical and semi-empirical methods for predicting the *SSUS* using the saturated shear strength parameters and the soil-water characteristic curve (*SWCC*) as a tool [for example, 5, 7, 8]. The *SWCC* is defined as a relationship between the soil water content (volumetric or gravimetric) or degree of saturation and soil suction.

This paper provides a succinct theoretical background information of how *SSUS* information can be used in predicting and interpreting the bearing capacity of shallow and deep foundations. In addition, this framework is supported using model and prototype tests performed in the laboratory and field, respectively. The developed theoretical approaches are consistent with the conventional geotechnical engineering practice applications that are based on saturated soil mechanics. The studies summarized in this paper are encouraging for the geotechnical engineers for implementing the mechanics of unsaturated soils in the design of shallow and deep foundations.

2 Background

Bishop [9] extended the Terzaghi [10] shear strength equation to describe the effective shear strength of unsaturated soil, which is given below.

$$\tau = c' + \left[(\sigma_n - u_a) + \chi (u_a - u_w) \right] \tan \phi' \quad (1)$$

where τ is shear strength of unsaturated soil, c' and ϕ' are the effective shear strength parameters, χ is parameter related to the degree of saturation; terms (σ_n- u_a) and (u_a- u_w) are the stress state variables: net normal stress and matric suction.

Fredlund et al. [2] proposed the shear strength equation for unsaturated soil in terms of two independent stress 61 state variables, which isshown as Eq. 2

$$\tau = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w) \tan \phi^b \quad (2)$$

where ϕ^b is the angle of shearing resistance relative to an increase in matric suction. This equation is well established in the literature and provides a rational interpretation of the shear strength behavior of unsaturated soils.

Experimental determination of the *SSUS* however requires expensive equipment, need trained professionals to operate them and are time consuming to perform. For this reason, several researchers [for example, 5, 11-14] proposed simple approaches for predicting or estimating the *SSUS*, which typically have form of Eq. 1 or Eq. 2. More comprehensive discussions on the *SSUS* are available in Vanapalli and Fredlund [15] and Vanapalli [16].

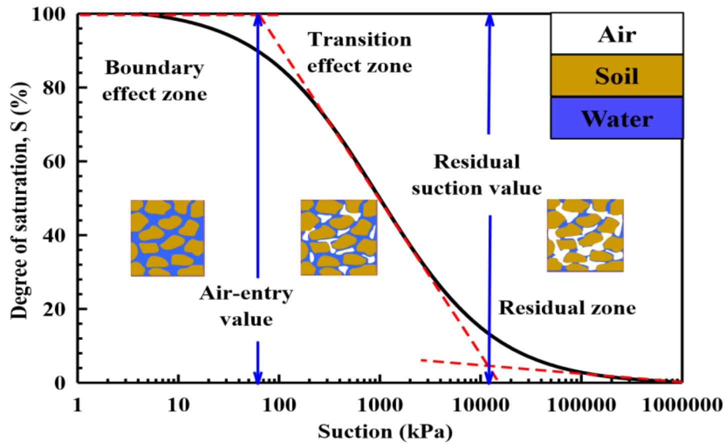
The simple approaches proposed by Vanapalli et al. [5] and Fredlund et al. [14] for predicting the *SSUS* are widely used in the literature. In these approaches, shear strength is predicted using the effective shear strengthparameters (i.e., c' and ϕ') and the *SWCC* (Eq. 3)

$$\begin{aligned} &\tau = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w)(S^\kappa) \tan \phi' \\ &\tan \phi^b = S^\kappa \tan \phi' \end{aligned} \quad (3a)$$

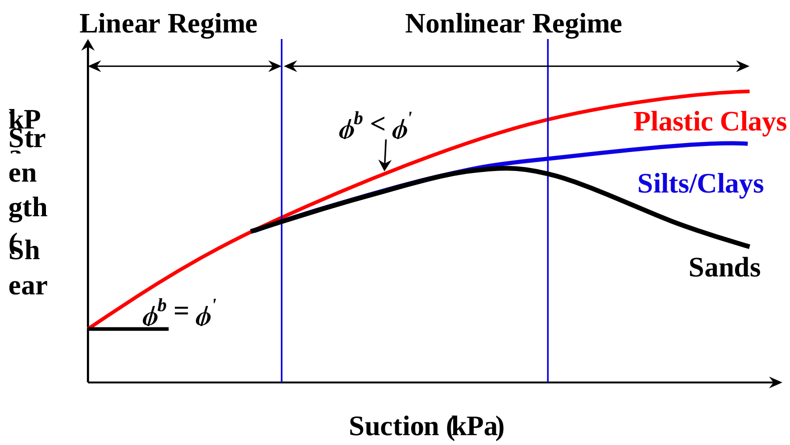
$$\begin{aligned} &\tau = c' + (\sigma_n - u_a) \tan \phi' + (u_a - u_w)\left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right) \tan \phi' \\ &\tan \phi^b = \left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right) \tan \phi' \end{aligned} \quad (3b)$$

where θ is the volumetric water content of the soil, θ_s and θ_r are respectively the saturated and residual volumetric water contents, S is degree of saturation. The fitting parameter κ varies for different soils and is strongly related to the plasticity index, I_p [15, 17, 18]. InEq. 3a and 3b, [c '+ (σ_n- u_a) tan ϕ']) represents the saturated shear strength. The terms,

$[(u_a - u_w)\, S^k \tan \phi']$ and $[(u_a - u_w)\, ((\theta - \theta_s) / (\theta_s - \theta_r) \tan \phi')]$ respectively in Eq. 3a and Eq. 3b, represent the contribution of the matric suction towards the shear strength.



(a) Typical soil water characteristic curve



(b) Shear strength of unsaturated soils in various zones

Fig. 1. Relationship between the *SWCC* and shear strength of unsaturated soils (Modified after Vanapalli [16])

Figure 1a shows drying *SWCC* for a typical fine-grained soil. Three key stages, namely boundary effect zone (BEZ), transition effect zone (TEZ) and residual zone of saturation (RZS) can be identified in this *SWCC*. All the soil pores are filled with water in the BEZ and the soil is in a state of saturated condition, in spite of soil suction. The value of matric suction at which air enters into the largest pores of soil is referred to as the air-entry value. The soil water content starts to reduce rapidly (i.e., desaturates) with a further increase in the matric suction in the TEZ. The water menisci area which is continuous within the soil particles or aggregates in the BEZ becomes discontinuous during the soil desaturation process in the TEZ as shown in Figure 1a. The desaturation phase in the TEZ is typically associated with the movement of water in the liquid phase.In the RZS, significantly large suction changes are necessary to achieve even small changes in the water content. The water content reduction in this zone is mostly in the vapor phase as there are no continuous paths for the water to flow in the liquid phase. The water menisci in the RZS is typically small. As a stress state variable, suction contributes to shear strength along the wetted area of contact of soil particles or aggregates. Due to this reason, the shear strength increases linearly in the BEZ (i.e., $\tan\phi^b = \tan \phi'$) when the matric suction is lower than the air entry value shown in Figure 1b. This is because matric suction is effectively transmitted along the wetted area of contact, which is 100%. However, there will be a non-linear increase in shear strength in the TEZ as the suction increases. The matric suction acts along the discontinuous wetted area of contact of soil particles within the TEZ. Therefore, the angle of shearing resistance with respect to suction ϕ^b is less than the angle of shearing resistance ϕ'. In the RZS, for example, sand desaturates at a relatively fast rate and has a low water content (i.e., low degree of saturation). In other words, there is a limited wetted area of contact. Due to this reason, suction in spite of being a higher value compared to BEZ and TEZ, may not be transmitted effectively to all the soil particles at their contact points [5]. Therefore, shear strength in coarse-grained soils typically decreases. However, fine-grained soils such as clays may not have a well-defined residual state. Considerable water (in the form of adsorbed water) in clay at residual stage may still be available to transmit suction effectively. Due to this reason, the shear strength of clays typically increases in the RZS. The nonlinear shear strength behavior of unsaturated soils should be considered in the rational design of geotechnical infrastructure. The sections that follow summarize how to determine the bearing capacity of both shallow and deep foundations of unsaturated soils using the *SUSS* as a tool.

3 Theoretical Studies

3.1 Shallow foundations

Effective stress approach (ESA) and total stress approach (TSA) are widely used in conventional geotechnical engineering practice for interpreting the bearing capacity of drained and undrained loading conditions, respectively for saturated soils. Similar approaches can also be used for unsaturated soils and are referred to modified effective stress approach (MESA) and total stress approach (MTSA).

Modified effective stress approach (MESA) . Terzaghi [10] proposed Eq. 4 for determining the bearing capacity of strip foundations assuming general shear failure condition. Vanapalli and Mohamed [19] extended the Terzaghi [10] equation (Eq. 4) and proposed a bearing capacity equation (Eq. 5) for unsaturated soils.Eq. 5a was originally proposed for surface square footing. This equation can be modified as Eq. 5b taking account of the influence of overburden stress associated with embedded foundation. There is a smooth transition between Eq. 5 and Eq. 4 used for unsaturated and saturated soils. In other words, when the matric suction is equal to zero, Eq. 5 takes the form as the Terzaghi's equation (i.e., Eq. 4). Eq. 5 can also be used as a tool to predict the nonlinear variation of the bearing capacity with respect to matric suction using the effective shear strength parameters of saturated soil (i.e., c' and ϕ') and the *SWCC*. The term $S^{\Psi}\tan\phi'$ describes the 119 shear strength contribution with respect to matric suction towards the bearing capacity.

$$q_{\mathrm{ult(sat)}} = c'N_{\mathrm{c}} + \gamma D_{\mathrm{f}} N_{\mathrm{q}} + 0.5B\gamma N_{\gamma} \quad (4)$$

$$q_{\mathrm{ult(unsat)}} = \begin{bmatrix} c' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{b}} \left(1 - S^{\psi}\right)\tan\phi' \\ + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{AVR}} S^{\psi} \tan\phi' \end{bmatrix} N_{\mathrm{c}}\xi_{\mathrm{c}} + 0.5B\gamma N_{\gamma}\xi_{\gamma} \quad (5a)$$

$$q_{\mathrm{ult(unsat)}} = \begin{bmatrix} c' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{b}} \left(1 - S^{\psi}\right)\tan\phi' \\ + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{AVR}} S^{\psi} \tan\phi' \end{bmatrix} N_{\mathrm{c}}\xi_{\mathrm{c}} + \gamma D_{\mathrm{f}} N_{\mathrm{q}}\xi_{\mathrm{q}} + 0.5B\gamma N_{\gamma}\xi_{\gamma} \quad (5b)$$

where $q_{\mathrm{ult(sat)}}$, $q_{\mathrm{ult(unsat)}}$ are the ultimate bearing capacity for saturated soil and unsaturated soils, B is the width of footing, γ is soil unit weight, N_c, N_q, N_γ are the bearing capacity factors [10, 20], $(u_a - u_w)_b$ is the air-entry value, ξ_c, ξ_q,ξ_γ are shape factors [21], D_f is the footing embedment depth. $(u_a - u_w)_{AVR}$ is the average matric suction value.

The average matric suction value is defined as the value at the centroid of the suction distribution profile within depth of 1.5B or 2B (i.e., the predominant stress bulb zone beneath the foundation). Ψ is the fitting parameter that is strongly related to the I_p. Vanapalli and Mohamed [19] proposed an equation (Eq. 6) of Ψ with experiments results of sands, Botkin Pit Silt and glacial till shown in Figure 2.

$$\psi = -0.0031(I_{\mathrm{p}})^2 + 0.34(I_{\mathrm{p}}) + 1 \quad (6)$$

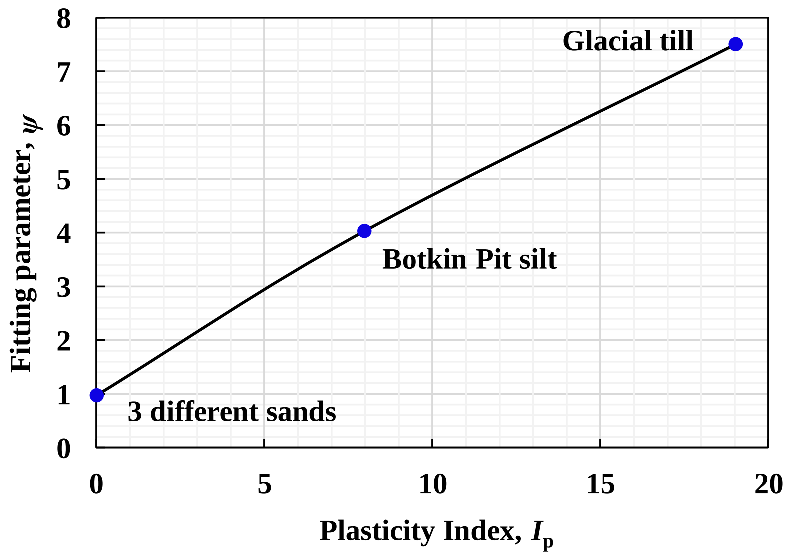


Fig. 2. Relationship between the soil plasticity index and the fitting parameter Ψ

Vanapalli and Oh [22, 23] suggested that Ψ can be typically assumed to be 3.5 for fine grained soils with plasticity index, I_p values greater than 8% based on the results of in-situ plate load tests. A value of Ψ equal to 1 is suggested for coarse grained soils with I_p = 0. They also summarized that different rates of loading can influence values of Ψ.

Oh and Vanapalli [24] also proposed a methodology using Eq. 5 to evaluate the stress versus settlement relationship. This method was developed by assuming the stress and settlement relationship in the elastic and plastic zones as two straight lines (i.e., linear). The slope of the elastic line is equal to the modulus of elasticity of the soil. Oh et al. [25] proposed Eq. 7 for predicting the variation of modulus of elasticity with respect to matric suction.

$$E_{\text{unsat}} = E_{\text{sat}}\left[1+\alpha\frac{(u_a-u_w)}{(P_a/101.3)}\left(S^{\beta}\right)\right] \tag{7}$$

where E_{unsat} and $E_{sat}$are the elastic modulus of unsaturated soil and saturated soil respectively. P_a is the atmosphere pressure (101.3kPa). α and β are fitting parameters, β = 1 for coarse grained soil [25] and β = 2 [26] for fine grained soil. α value is related to the ratio of footing size to soil particle size [25]. The proposed MESA approach has been validated with experimental studies for both coarse- and fine-grained soil with matric suction values lower than the residual suction. More details about the experimental studies and the comparisons between the variation of bearing capacity with matric suction from experimental results and predicted values are discussed in later sections.

Modified total stress approach (MTSA) . Studies by Oh and Vanapalli [27] suggest thatMESA may not provide reliable results for large suction values in unsaturated fine-grained (UFG) soils. This may be associated with bearing capacity converging close to residual suction value. For such a scenario, drainage condition of UFG soil may not be well-defined. Investigations suggest the MTSA providesa reasonable bearing capacity for UFG soils at large suction values. Vanapalli et al. [28]proposed the MTSA to interpret the bearing capacity of foundation in UFG soils under undrained loading condition. The method was based on the equation proposed by Skempton [29]shown as Eq. 8.

$$q_{\text{ult}} = c_{\text{u(sat)}} \times \xi_{\text{c}} \times N_{\text{c}} \tag{8}$$

where q_{ult} is ultimate bearing capacity of the saturated soil; $c_{\text{u(sat)}}$ is the undrained shear strength for saturated soils; $N_{\text{c}}$is bearing capacity factor related to cohesion under undrained loading condition.

Vanapalli et al. [28]replaced the undrained shear strength in Eq. 8 with that of unsaturated soil $c_{\text{u(unsat)}}$. Similar to saturated soil, it is assumed that $c_{\text{u(unsat)}}$ is equal to $q_{\text{u(unsat)}}/2$ from the unconfined compression (UC) test. Therefore, theultimate bearing capacity of UFG soil will take the form as shown in Eq. 9

$$q_{\text{ult(unsat)}} = \left[\frac{q_{\text{u(unsat)}}}{2}\right]\left[1 + 0.2\left(\frac{B}{L}\right)\right] N_{\text{cunsast}} \tag{9}$$

where N_{cunsat} is the bearing capacity factor under unsaturated condition, it is same with that in saturated condition inthis equation. L is the foundation length.

The shear strength of UFG soil is derived from UC test results because of two reasons. The first reason is related to the failure mechanism of the foundations associated with UFG soil under undrained loading condition. Several research studies [27, 28-31]suggest that the failure mechanism of a shallow foundation in UFG soil is closely related to the punching shear failure mode shown as Figure 3. The bearing capacity of the foundation therefore can be considered as a function of the soil compressive strength below the foundation. The soil condition (pore air drained, pore water undrained) can be realized in UC tests. The second reason is UC test is a quick and conventional test 168 compared to other tests such as the constant water content (CW) tests) [27].

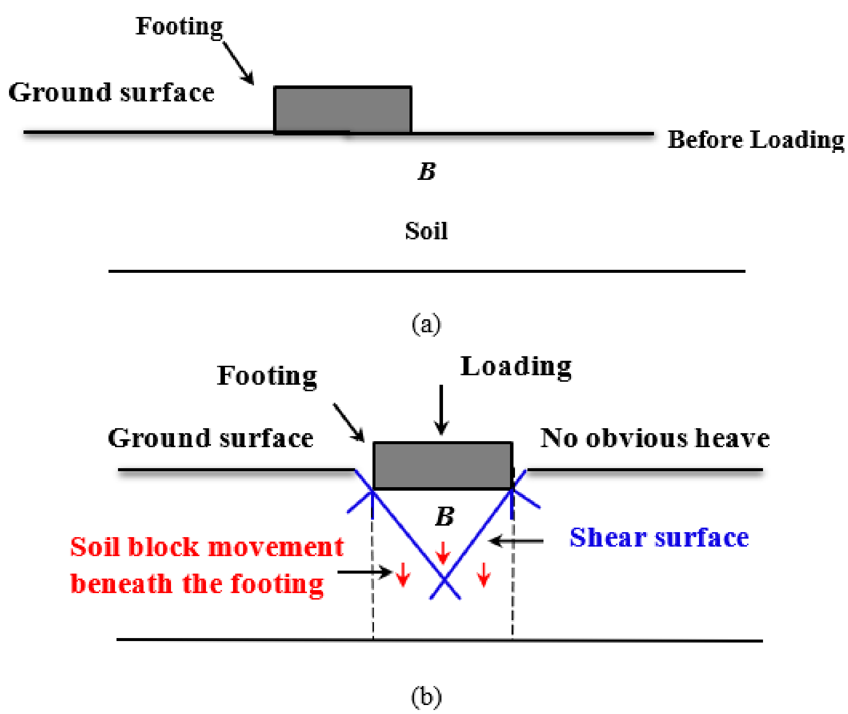


Fig. 3. Schematic of common punching failure mechanism in unsaturated fine-grained (UFG) soils beneath the foundation: (a) Before loading; (b) During loading

Oh and Vanapalli [32] proposed Eq. 10, which facilitates to estimate the variation of the undrained shear strength, $c_{u(unsat)}$ for UFG soils.

$$c_{u(unnsat)} = c_{u(sat)} \left[1 + \frac{(u_a - u_w)}{(P_a / 100)} (S^v) / \mu \right] \tag{10}$$

where v and $\mu$are fitting parameters, v =2 for fine grained soil, μ is related to the plasticity index, I_p , given in Eq.11a and 11b.

$$\mu = 9,\ 8.0 \le I_{\mathrm{p}}(\%) \le 15.5 \quad (11a)$$

$$\mu = 2.1088e^{0.0903(I_{\mathrm{P}})},\ 15.5 < I_{\mathrm{p}}(\%) \le 60.0 \quad (11b)$$

where e is Euler's number. The above relationship was derived from six sets of UC test results [15, 28, 33 - 36]. The results derived by the shear strength equation at suction close to the residual suction value should be carefully interpreted [23]. The proposed approach is valuable; however, more supporting data from field tests can provide more credence for use in geotechnical design practice.

3.2 Pile foundations

End-bearing capacity of piles embedded in unsaturated sands. Theoretical approaches that have been developed for determining the bearing capacity of shallow foundations can be extended with modifications for estimating the end-bearing capacity of pile foundations. For shallow foundations, Vanapalli and Mohamed [19]proposed an approach for predicting the bearing capacity of surface shallow foundation with respect to matric suction, extending the conventional approach used for saturated soils. In this approach, the effective cohesion c' term is replaced with total cohesion c_{a} for determining the variation of bearing capacity of unsaturated soils taking account the influence of matric suction. The same equation (i.e., Eq. 12) can be also used for predicting the variation of total cohesion c_{a} with respect to matric suction. The information of the effective shear strength parameters and the *SWCC* is required for extending this approach.

$$c_{\mathrm{a}} = c' + \left(u_{\mathrm{a}} - u_{\mathrm{w}}\right)_{\mathrm{b}} \left(1 - S^{\Psi_{BC}}\right) \tan\phi' + \left(u_{\mathrm{a}} - u_{\mathrm{w}}\right)_{\mathrm{AVR}} S^{\Psi_{BC}} \tan\phi' \quad (12)$$

where, Ψ_{BC} = fitting parameter with respect to bearing capacity (Ψ_{BC} =1 for nonplastic soil).

Vanapalli et al. [37] extended this approach to predict the end-bearing capacity $Q_{\mathrm{p(us)}}$ of single piles in unsaturated soils. Three conventional methods originally proposed by Terzaghi [10], Hansen [38] and Janbu [39]for estimating the end-bearing capacity of piles in saturated soils were used in extending this approach. The soil above the pile was assumed as an equivalent surcharge, q. Limit equilibrium conditions are used to determine the end-bearing capacity based on failure patterns around the pile tip. The conventional Terzaghi [10]and Hansen [38]/Janbu [39]equations were modified as Eq. 13 and 14, respectively.

$$Q_{\mathrm{p(us)}} = A_{\mathrm{p}}\left(\left[c' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{b}}\left(1 - S^{\psi_{BC}}\right)\tan\phi' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{AVR}} S^{\psi_{BC}} \tan\phi'\right] N_{\mathrm{c}} s_{\mathrm{c}} + q' N_{\mathrm{q}} + \tfrac{1}{2} B\gamma N_{\gamma} s_{\gamma}\right) \quad (13)$$

$$Q_{\mathrm{p(us)}} = A_{\mathrm{p}}\left(\left[c' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{b}}\left(1 - S^{\psi_{BC}}\right)\tan\phi' + (u_{\mathrm{a}} - u_{\mathrm{w}})_{\mathrm{AVR}} S^{\psi_{BC}} \tan\phi'\right] N'_{\gamma} d_{\mathrm{c}} + \eta q' N'_{\mathrm{q}} d_{\mathrm{q}} + \tfrac{1}{2} B\gamma N'_{\gamma}\right) \quad (14)$$

where $A_p$is pile base area, q' is vertical effective stress at the level of pile base, γ is unit weight of soil beneath the pile, B is pile diameter, s_c, s_γ are shape factors with respect to cohesion and unit weight of soil, respectively (s_c=1.3 and s_γ= 0.6 for round pile foundation). N_c, N_q, N_γ, N_c', N_q', N_γ' are bearing capacity factors that are functions of soil friction angle, d_c, d_q are depth factors. More details about the information related to determination of the bearing capacity factors are available in Vanapalli et al. [37].

Pile shaft carrying capacity in UFG soils. Vanapalli and Taylan [40] modified three conventional semi-empirical approaches (i.e., α method proposed by Skempton [41], β method proposed by Burland [42], λ method by Vijayvergiya and Focht [43] for saturated soils) to predict the variation of shaft carrying capacity of pile foundations in UFG soils.

Modified α method . The α method was originally proposed by Skempton [41] to determine the shaft resistance of piles placed in saturated cohesive soils extending the total stress approach (TSA) (i.e., $\phi' = 0$ concept). In this conventional method, the average shaft friction that develops for carrying the load is related to the mean undrained shear strength by an empirical coefficient α which is typically less than unity. Extending the same approach, the shaft carrying capacity of pile Q_f under undrained loading conditions in UFG soils can be estimated using Eq. 15.

$$\begin{aligned} Q_{\mathrm{f}} &= f_{\mathrm{s}} \times A_{\mathrm{s}} = \alpha c_{\mathrm{u}} \pi dL \\ &= \alpha c_{\mathrm{u(sat)}}\left[1 + \frac{(u_{\mathrm{a}} - u_{\mathrm{w}})}{(P_a/101.3)} S^{\upsilon}/\mu\right]\pi dL \end{aligned} \quad (15)$$

where d is the pile diameter and L is the length of pile.

*Modified β method***.** In 1973, Burland [42] proposed β method to calculate shaft carrying capacity by extending effective stress approach (ESA). The total stress approach (TSA) is suitable for clayey soils; however, ESA can be used for all soil types. When the displacement between piles and clay are relatively large, the pile shaft friction is mainly influenced by lateral effective stress. Vanapalli and Taylan [40] proposed a modified β method to determine pile shaft resistance in unsaturated soils. The ultimate shaft capacity of a single pile in unsaturated soils $Q_{f(us)}$ consists of the contribution from matric suction $Q_{(ua-uw)}$ and conventional shaft resistance Q_f under saturated conditions.

$$Q_{\mathrm{f(us)}} = Q_{\mathrm{f}} + Q_{(\mathrm{u_a}\text{-}\mathrm{u_w})} \tag{16}$$

For fine-grained soils, the apparent cohesion c'_a under drained loading condition is due to the contribution of the shaft carrying capacity. Thus, the ultimate shaft capacity of single pile in unsaturated soils $Q_{f(\mathrm{us})}$ can be written as

$$Q_{\mathrm{f(us)}} = \left[c'_{\mathrm{a}} + \beta\left(\sigma'_z\right) + \left(u_{\mathrm{a}} - u_{\mathrm{w}}\right)\left(S^{\kappa}\right)\left(\tan\delta'\right)\right]\pi dL \tag{17}$$

where S is the degree of saturation, K is fitting parameter used for shear strength. The fitting parameter K can be obtained from the relationships proposed by Vanapalli and Fredlund [15]. σ'_z is the horizontal effective stress acting on the pile and δ' is the effective friction angle of the pile-soil interface, β is Burland-Bjerrum coefficient is equal to $K_0\tan\delta'$, where K_0 is earth pressure coefficient.For bored piles, the angle δ' is commonly assumed to be equal to the angle of shearing resistance of the surrounding soil, ϕ', for practical purposes.

Modified λ method . Vijayvergiya and Focht [43]suggested λ method to predict pile shaft carrying capacity, which combines TSA and ESA. This method assumes that the unit skin friction has a relationship with both vertical effective stress and the undrained shear strength. The shaft resistance per unit area $f_{\mathrm{s(avg)}}$can be estimated by the following equation

$$f_{\mathrm{s(avg)}} = \lambda\left(\sigma'_{\mathrm{v(avg)}} + 2c_{\mathrm{u}}\right) \tag{18}$$

where $\sigma'_{\mathrm{v(avg)}}$is the mean effective stress along the pile shaft, λ is frictional capacity coefficient which is a function of entire embedded depth of pile. The λ method was modified by Vanapalli and Taylan [40]to estimate the shaft carrying capacity of single pile $Q_{\mathrm{f(us)}}$ with respect to matric suction.

$$Q_{\mathrm{f(us)}} = \lambda\left[\sigma'_{\mathrm{v(avg)}} + 2c_{\mathrm{u(sat)}}\left(1 + \frac{\left(u_{\mathrm{a}} - u_{\mathrm{w}}\right)}{P_a/101.3}S^{\upsilon}/\mu\right)\right]\pi dL \tag{19}$$

Also, when matric suction (u_a- u_w) equals to zero, Eq. 19 is the same as Eq. 18 for saturated soil conditions. All the modified approaches (i.e., α method, β method, and λ method) provide a smooth transition of pile shaft carrying capacity from unsaturated to saturated soil conditions. In other words, these equations (i.e., Eq. 15, 17, 19) take the form of conventional equations used for saturated soils, when matric suction equals a value of zero.

4 Experimental Studies

4.1 Laboratory tests on shallow foundation

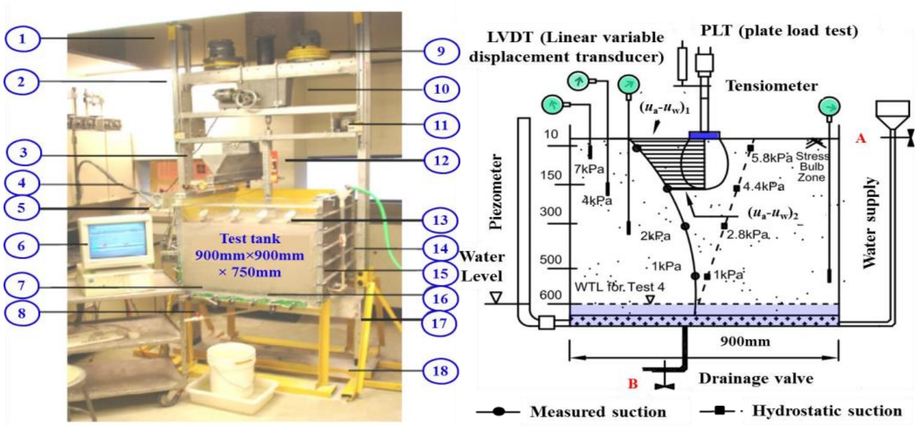
Table 1 provides a summary of the experimental studies information along with the proposed equations for interpreting the bearing capacity and settlement of shallow foundations. Laboratory tests on model shallow foundations using the MESA and MTSA methods are introduced in this section. In addition, in-situ tests such as the standard penetration test (SPT) and plate load test (PLT) are also summarized with the site details and their results. Finally, comparisons between the predicted results from the proposed approach and the measured data from experiments are summarized.

Table 1. Summary of the experimental studies related to the bearing capacity of shallow foundations in unsaturated soils.

	Experiments and Equation Validation			
Bearing Capacity And settlement	Modified total stress approach (MTSA)	Indian Head Till	Unconfined Compression (UC) test, model footing test	Vanapalli et al. [28]
	Suitable for fine grained soil	Lateritic soil deposit	Validation: In-situ Plate Load test (PLT) Costa et al. [44]	Vanapalli and Oh [27]
		Residual cohesive soil	Validation: PLT Consoli et al. [45]	Oh and Vanapalli [27]
	Modified effective stress approach (MESA)	Unimin (7030) sand	Model footing test	Mohamed and Vanapalli [46,47]
	Suitable for coarse and fine grained soil	Huston sand	Model footing test for plain strain condition	Vanapalli and Mohamed [19,48] Lins et al. [49]
		Indian Head Till	Model footing test	Oh and Vanapalli [27]
	In-situ tests	Dark-grey silty sand underneath the	PLT Standard Penetration Test (SPT), CPT, In-	Mohamed and Vanapalli [51]

	septic sand	situ foot loading test validation [50]	
	Poorly graded fine sand (according to USCS)	PLT (modified UOBCE), Cone Penetration test (CPT)	Vanapalli and Mohamed [48]

Laboratory tests on coarse-grained soil . Mohamed and Vanapalli [46], Vanapalli and Mohamed [48] conducted model footing tests in coarse grained sand with the Ottawa Bearing Capacity Equipment (UOBCE, shown in Figure 4) and the modified UOBCE. The features of the modified UOBCE is similar to UOBCE; however, its test box size and its loading capacity is twice to that of UOBCE. The UOBCE is designed to perform the bearing capacity of both model footing tests in both saturated and unsaturated soils. The UOBCE constitutes of an aluminium tank with dimensions of 900mm×900mm×750mm. The front face of the tank is constructed with a transparent acrylic plate. This transparent plate is helpful for the examination of the thickness of the soil layer during installation and for the observation of water table changes. Stiffeners (Figure 4, Item 14) were set along the tank side to prevent lateral bending or bulging. First, 50mm layer of clean aggregate (Figure 4, Item 16) was placed at the bottom of the test tank with a thin geotextile sheet (Figure 4, Item7) on the top. The geotextile sheet is a porous barrier between the soil and the aggregate. Gradual free movement of water is assured through the bottom aggregate and geotextile sheet layer. The sand was then allowed to fall freely from a 1m height with a V-shaped hopper (Figure 4, Item 3) to achieve the maximum relative density. The hopper which is able to hold 25kg soil can be monitored to move horizontally and vertically using a side motor with horizontal chain and the side crank with cables on four rollers on the top of the frame (Figure 4, Item 1). The soil was further compacted using a 5kg compactor after achieving a uniform relative density of 55% by allowing the soil spread with the V-shaped hopper. An average relative density value of 64% was achieved after compaction of soil; which was verified by collecting soil samples at various depths in the tank with an aluminium cups. The sand was then saturated by raising the water level from the bottom of the tank using the water supply valve in Figure 4 (Valve A). The water supply pipe has a diameter of 20mm which branches into 4 small pipes with a diameter of 12.5mm to assure the saturation process is gradual and uniform from the base to the surface of the soil. The water level in the tank can be adjusted by inspecting the piezometers (Figure 4, Item 5). The water supply valve was closed when the water level reached the soil surface. After saturation, the water could be controlled with the drainage valve in Figure 4 (Valve B) to reach the target water level and the corresponding target matric suction. The target matric suction was the average matric suction in the stress bulb beneath the foundation (i.e., 2kPa, 4kPa and 6kPa). The matric suction was measured using four tensiometers placed at different depth levels in the tank.



1.	Roller	7.	Thin geotextile sheet	14.	Stiffeners
2.	Reaction beams	8.	Drainage valve	15.	C-clamps
3.	Hopper	9.	Dead loads	16.	Aggregate layer
4.	Crank	10.	Loading machine	17.	Supporting beams
5.	Piezometer	11.	Electrical motor	18.	Steel base
6.	DAS (Data acquisition system)	12.	Loading arm		
		13.	Tensiometers		

Fig. 4. University of Ottawa Bearing Capacity Equipment (UOBCE) (modified after Vanapalli and Mohamed [48])

The model footing test was conducted after achieving equilibriumconditions with respect to target matric suction value. Mohamed and Vanapalli [46]conducted model footing test with two square model footings with sizes of 100mm×100mm and 150mm×150mm. The distance between the model footing edge and the tank sides was four times of the footing width to alleviate boundary effects. Linearly Variable Displacement Transducer (LVDT) was used to measure the displacement of the footing and was connected to the data acquisition system (DAS, Figure 4, Item 6). The LVDT tip was placed directly on the surface of the model footing. A load cell which was also connected to the DAS was mounted on the loading arm. The bearing capacity of the footing was measured by loading the footing with a rate of 1.2mm/min.

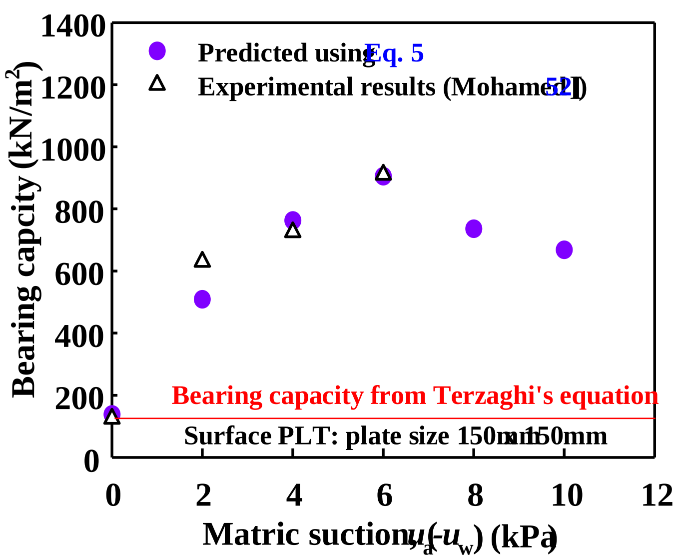


Fig. 5. Comparison between the measured bearing capacity and predicted bearing capacity of sand using Eq. 5

Figure 5 provides comparisons between the measured bearing capacity and the predicted results using Eq. 5. The effective shear strength parameters required for bearing capacity estimation using Eq. 5 are derived from direct shear tests. The average matric suction method has been used in estimation of the bearing capacity. The value of average matric suction is the centroid of the matric suction distribution profile in the stress bulb zone (shown in Figure 4), which was discussed earlier. The degree of saturation corresponding to the average matric suction used for Eq. 5 was derived from the *SWCC* of the soil. Good agreement has been found in Figure 5 between the experimental results and the predicted values. The bearing capacity of footing was found increase until reaching the average matric suction value of 6kPa, which is approximately, the residual suction value. It is important to note that the measured data was restricted to matric suction range 0 to 6kPa because of the limitations with respect to the depth of test box. The reductions in predicted bearing capacity values for matric suction values greater than 6kPa can be explained with the discontinuous water phase shown in Figure 1. For such a scenario, sand desaturates and the discontinuous water phase in soil leads to changes in both stress state and soil-air-water particle contact area, which is typically limited. As a result, suction may not be transmitted effectively to all the soil particles which leads to reduction both in the shear strength and bearing capacity, extending the arguments discussed earlier with Figure 1b.

Cone penetration test (CPT) results are also widely used in conventional engineering practice for estimating bearing capacity of soils. Besides model footing tests, Mohamed et al. [53]conducted CPT within the UOBCE discussed earlier to investigate the bearing capacity of sand taking account of the influence of matric suction. The test equipment for the CPT is shown in Figure 6. The test cone was fabricated with harden steel with a tip angle of 60˚. A diameter of 40mm was chosen following the recommendations by ASTM D 5778 [54]. Two horizontal aluminium channel sections and a shaft (Figure 6, Item 6) were set to prevent deformation of the loading rod to achieve vertical penetration of the cone. The water table in the tank was controlled with the water supply valve and drainage valve (Figure 4, Valve A, Valve B). Tensiometers (Figure 6, Item 2) are placed at different depths above the water table for the matric suction measurement. Similar to the model footing test, the cone resistance q_c was determined using a strain rate of 1.2mm/min in four different suction profiles (i.e., average matric suction: 0, 1, 2, 6 kPa).

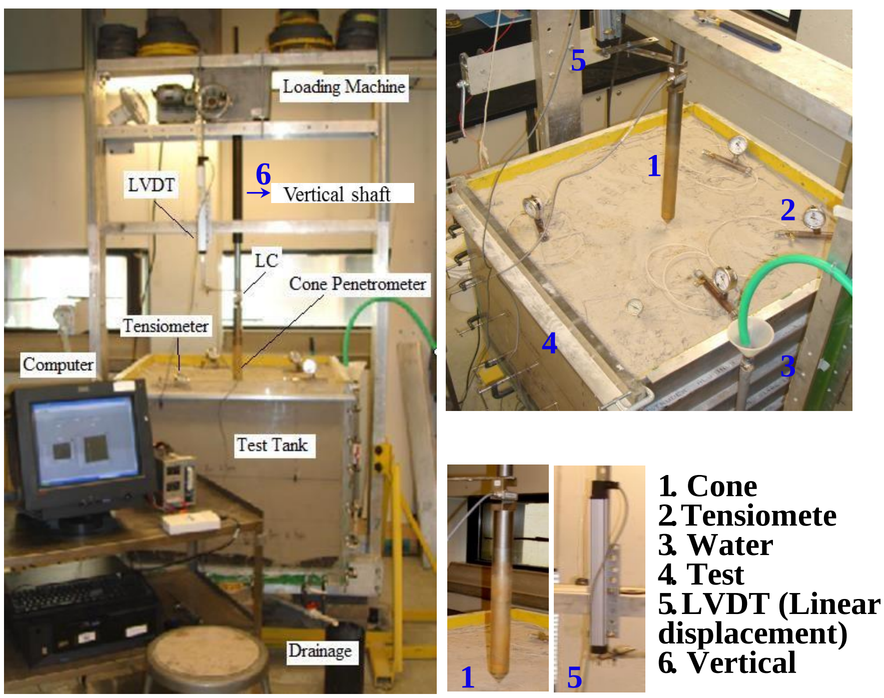


Fig. 6. CPT equipment for determining the bearing capacity of unsaturated soils (modified after Mohamed [52])

Mohamed et al. [53] proposed two equations (Eq. 20a and Eq. 20b) for estimating the bearing capacity of foundations using CPT results by correlating them with the model footing test results

$$q_{\mathrm{usat}} = \Theta\ (q_{\mathrm{csat}}),\ \Theta = 0.15 / B^{0.63} \tag{20a}$$

$$q_{\mathrm{u(unsat)}} = \Omega\ (q_{\mathrm{c(unsat)}}),\ \Omega = 0.19 / B^{0.68} \tag{20b}$$

where q_{usat} and $q_{\mathrm{u(unsat)}}$ are bearing capacity for saturated sand and unsaturated sand, Θ and Ω are correlation factors related to the footing width B, q_{csat} and $q_{\mathrm{c(unsat)}}$ are the average cone resistance in saturated and unsaturated soil. The average cone resistance is similar to the average matric suction method discussed earlier in the paper; the influence zone for the average calculation is set as 1.5*B* from the footing base level.

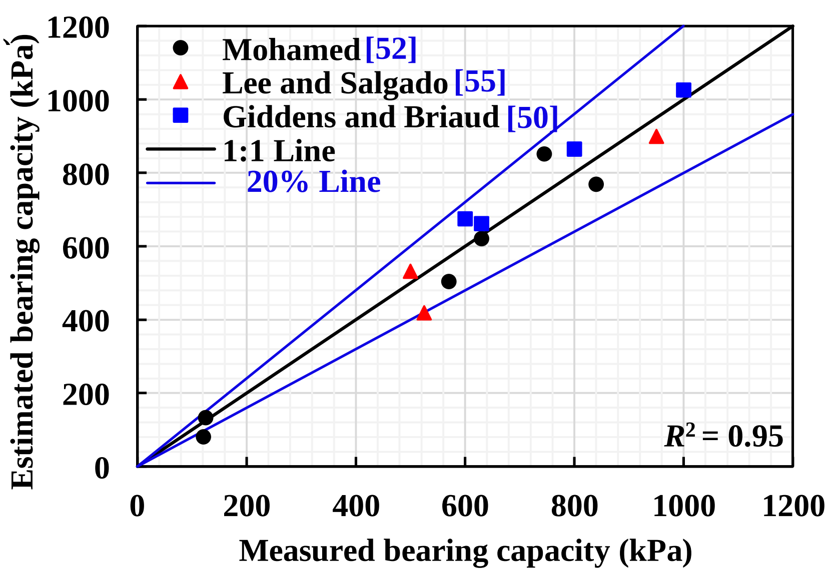
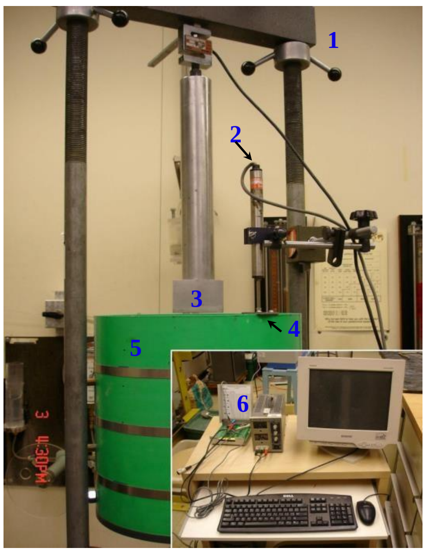


Fig. 7. Comparison between the measured data from laboratory surface PLTs, in-situ footing load test (FLTs) and estimated bearing capacity using Eq. 20

Results summarized in Figure 7 suggest a good agreement between the measured bearing capacity from the model PLTs (i.e., model footing), in-situ footing load tests (FLTs) and that estimated with Eq. 20. The coefficient of determination R^2 is equal to 0.95. Eq. 20 provides a reasonable estimation of the bearing capacity from the CPT results. However, more studies are required with various sizes of footings and soils in the field such that they can be used by geotechnical engineers for practice applications with greater degree of confidence.

Laboratory tests on fine-grained soil . Vanapalli et al. [28] performed a series of model footing tests on a glacial till from Indian Head under undrained loading conditions. Figure 8 provides details of the experiment setup.



1. Load cell
2. LVDT (Linear variable displacement transducer)
3. Model footing
4. Metal plate to place LVDT
5. HSPT (High strength plastic tank)
6.Data acquisition system
7. Holes for conventional tensiometers
8. Compactor with holes for drainage

Fig. 8. Equipment for used for determining the bearing capacity of model footing in unsaturated fine-grained (UFG) soil (modified after Oh and Vanapalli [27])

The glacial till was compacted in a high strength plastic tank (HSPT) (Figure 8, Item 5) with dimensions of 300 mm × 300 mm × 12.7 mm (Diameter × Height × Thickness) to perform model footing tests with 50 mm × 50 mm footing dimensions. The ratio of the diameter of the HSPT to the footing width was set as six based on the published studies to alleviate the influence of boundary conditions [56-59]. Three clamps were placed around the HSPT to alleviate strain during the test. A metal plate (Figure 8, Item 4) was positioned at the top of the tank to place LVDT to measure the displacement of the footing. The soil was prepared with an initial water content of 13.2% and was compacted to a dry density of 14.4 kN/m^3 using static compaction stress of 350 kPa. The soil was statically compacted in five layers with the specially designed compactor (Figure 8, Item 8). Four holes were drilled on the circular compactor for drainage. The compaction of soil layers stopped until no further displacement was observed. Prior to placing a new layer of soil, the surface of previous layer was scarified. After completing the compaction of all five layers, the soil was saturated by allowing water entering the soil through the drainage holes in the compactor (Figure 8, Item 8). At the same time, the special compactor was fixed on the surface of the soil to prevent soil swelling. The HSPT was then submerged into the water for 3 days to ensure the soil was fully saturated. After removing the HSPT from water, four tensiometers were installed at different depth levels in the tank (10, 40, 80, 120mm) (Figure 8, Item 7). The soil was then subjected to air drying for several days to achieve the target matric suction profile which were monitored using the tensiometers. Due to the air drying, the top soil layer would have a rather high matric suction. The HSPT was wrapped tightly and put in a humidity controlled box for more than 14 days to achieve equilibrium conditions with respect to matric suction profile. The model footing test was conducted after taking the HSPT out from the box. Five suction profiles had been considered in the experiments (with average suctions in stress bulb of 0, 55, 100, 160, 205kPa). The bearing capacity of model footing under each suction profile was determined by applying a rate of loading equal to 1.14mm/min.

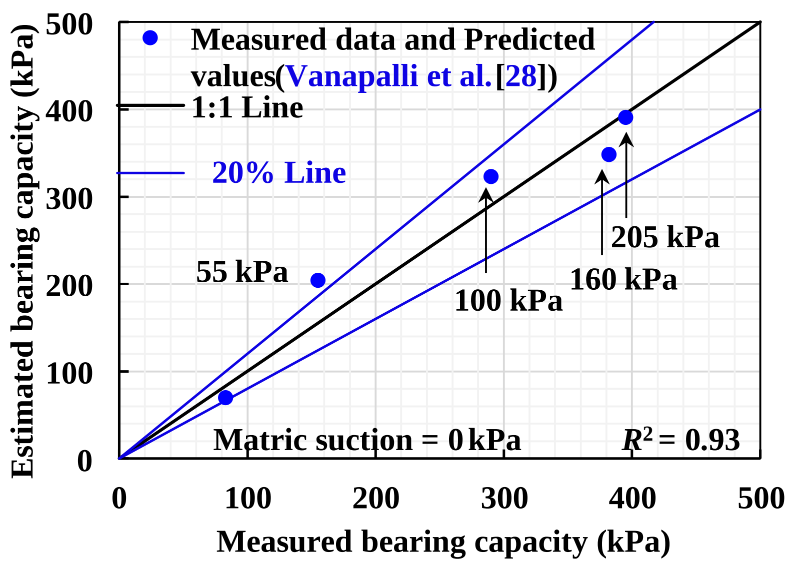
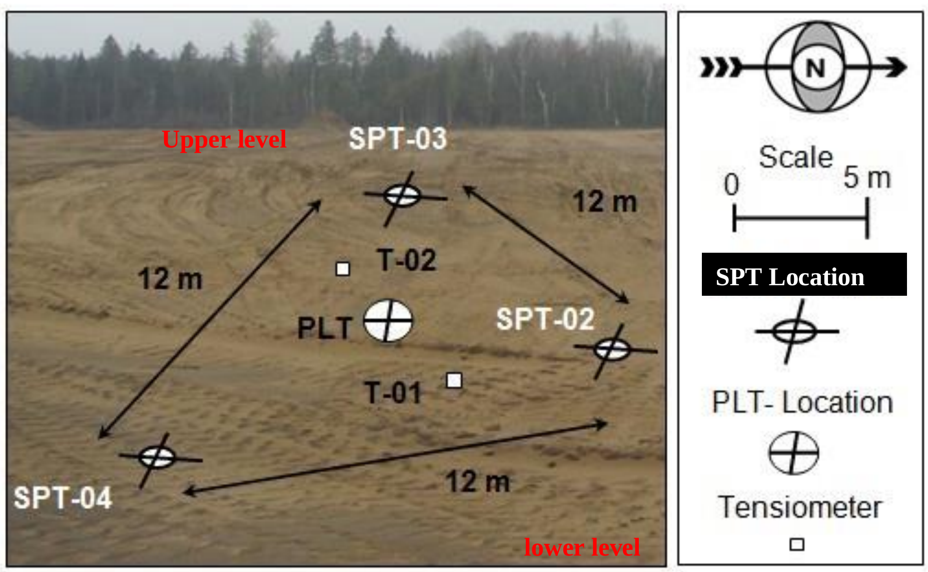


Fig. 9 Comparison between the measured and predicted bearing capacity of square model footing with width of 50mm

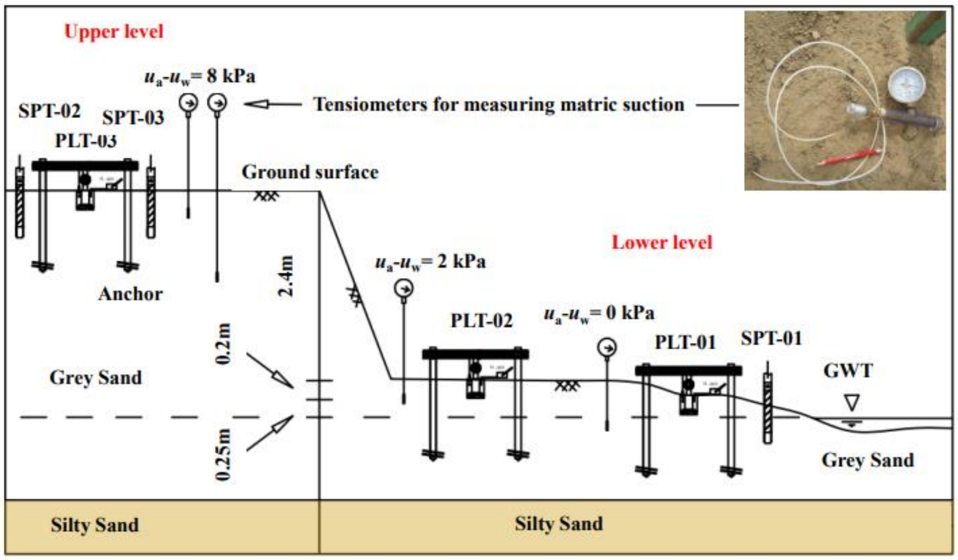
Figure 9 shows the relatively good agreement with $R^2 = 0.93$ between the estimated bearing capacity from Eq. 9 and measured bearing capacity from the model footing test. The shear strength used in Eq. 9 is derived from UC test. The soil specimens prepared for UC test was extracted from the compacted soil outside of the stress bulb below the foundation in the tank. The soil sample may therefore have a similar suction distribution with that in model footing test. Unlike coarse grained soil, the bearing capacity of footing on fine grained soil increases with an increase in matric suction for the current study range. This can be explained with Figure 1b which highlights the shear strength or the bearing capacity of fine grained soil will increase after the air entry value and keep increasing or stay constant beyond the residual suction value.

4.2 In-situ tests on shallow foundations

In conventional engineering practice, the in situ bearing capacity of soils is typically determined from PLT, CPT or SPTs [51]. Mohamed and Vanapalli [51]conducted in-situ tests to determine the bearing capacity of sand taking account of the influence of matric suction. The tests were conducted at Carp region in Ottawa, Ontario, Canada. The test site and SPTs, PLTs and tensiometers location are shown in Figure 10. The site has a sloping terrain with a difference of 2.4m between the upper and lower levels. Dark-grey silty sand was found underneath the 4.7m depth of grey sandy soil (known as septic sand) from the upper level surface.



a) Test terrain and SPT, PLT and tensiomenter locations



(b) Side view of the test site and location

Fig. 10. Test site details (Modified after Mohamed and Vanapalli [51])

SPT-01 in Figure 10b was used to simulate saturated condition test since it was close to natural ground water table (GWT). The other three SPT tests were conducted under unsaturated conditions in the upper level region. Each of the three tests had a distance about 12m between them as shown in Figure 10a. The SPTs were conducted with a truck mounted equipment following the ASTM D1586 [60]up to depth of 3.5m from the natural soil surface. The SPT energy efficiency was equal to 60%. More details about the SPT and blow counts are discussed in Mohamed and Vanapalli [51]. A steel plate of 0.2m×0.2m had been used in the PLTs. Two PLTs were conducted at lower level in different zones of varying matric suction values. PLT-01 were conducted in a saturated condition with zero matric suction recorded on tensiometer. PLT-02 were carried out on a soil suction of 2kPa. PLT-03 was conducted at upper level with a uniform suction of 8 kPa. All of the three tests were conducted at a depth of 0.15m. The tensiometers measured the matric suction at the mid-height depth of the stress bulb zone.

Results of the in-situ SPTs showed that the blow count *N* under unsaturated soil condition was much higher than that of saturated soil. The in-situ PLTs also showed that the bearing capacity of the plate on soil with a suction value of 8kPa was about three times higher than that on saturated soil [51]. Correlations of the SPTs results and PLTs results have been proposed by Mohamed and Vanapalli [51]as Eq. 21a and Eq. 21b. The equations are based on the CPT and PLT correlation equations proposed by Mohamed et al. [53] that were discussed earlier.

$$q_{\text{all(sat)}} = \frac{0.15}{B^{0.63}}\left[0.37\,(N_{\text{(sat)}})^{0.73}\right] \times 1000 \tag{21a}$$

$$q_{\text{all(unsat)}} = \frac{0.19}{B^{0.68}}\left[0.45\,(N_{\text{(unsat)}})^{0.83}\right] \times 1000 \tag{21b}$$

where $q_{\text{all (sat)}}$ and $q_{\text{all(unsat)}}$ are allowable bearing capacity for footings on saturated sand and unsaturated sand. The allowable bearing capacity were determined at a settlement of 6mm of PLT. $N_{\text{(sat)}}$ and $N_{\text{(unsat)}}$ are the average corrected SPT blow count value in the influence zone (i.e., stress bulb zone within depth of 1.5B beneath the footing). Comparisons between the bearing capacity predicted with Eq. 21a, Eq. 21b and that from different published in-situ test [50, 61] are shown in Figure 11. Results show a good agreement between the predicted and measured values for various plate sizes. The measured and estimated values of bearing capacity are within the ±20% deviation line. The proposed equation is promising for use in geotechnical engineering practice.

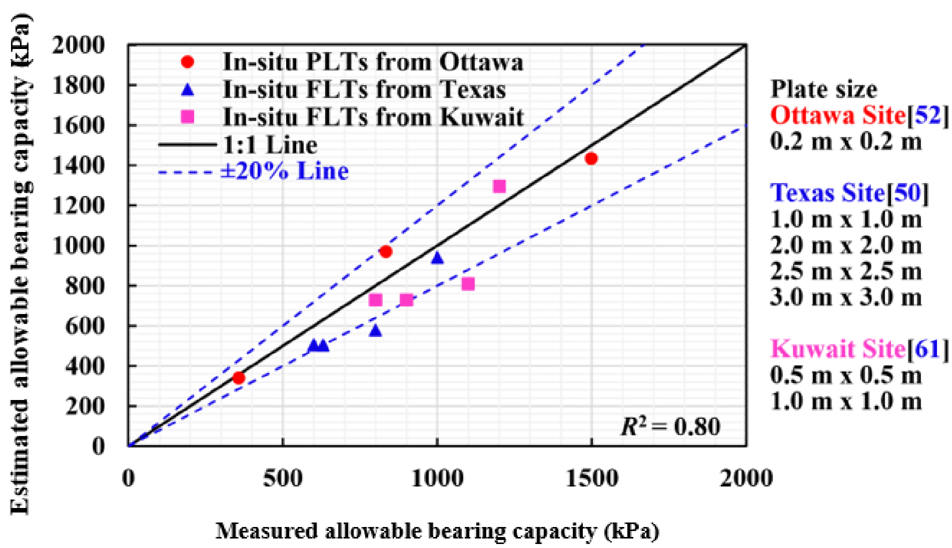
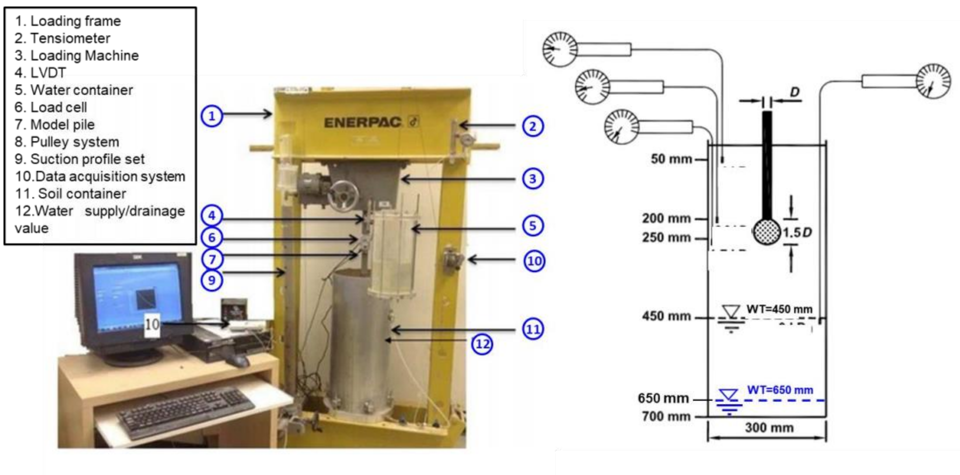


Fig. 11. Comparison between the measured allowable bearing capacity from various test sites and that from estimation using Eq. 21

4.3 Laboratory tests on deep foundation

Laboratory tests on coarse-grained soil . Vanapalli et al. [37] carried out experimental studies to investigate the load versus displacement behavior of single piles in an unsaturated coarse-grained soil (i.e., Unimin Sand). The details of test program are presented in Figure 12.



(a) (b)

Fig. 12. Details of test program: (a) Details of the test setup for determining the load carrying for model pile (b) Cross-sectional schematic of testing program with the water level at 450 mm and 650 mm from the soil surface (Modified after Vanapalli et al. [37])

The soil container was 300 mm in diameter, 700 mm in height and 8 mm in thickness. The model pile used in the tests were stainless steel piles with three different diameters (i.e., 38.30, 31.75, and 19.25 mm) with a length of 350 mm. Three tensiometers were installed at different depths (i.e., 50, 200, 250 mm) from soil surface to measure and monitor the matric suction values. The water container was connected to water supply/drainage value at the base of soil container to obtain required water levels using a pulley system. The end-bearing capacity and shaft-bearing capacity of model piles were measured separately. The pile was placed through a hollow sleeve in order to eliminate the contribution of pile shaft resistance towards the total bearing capacity for measuring the end-bearing capacity of the model pile. The cylindrical tube was covered with a thin flexible plastic sheet film on the top of soil sample for measuring the shaft-bearing capacity of the model pile. The plastic film facilitates to prevent the connection between the pile base and soil surface in order to reliably measure of pile shaft bearing capacity. Two different matric suction distribution profiles were achieved by setting one water level at 450 mm deep and the other at 650 mm deep from soil surface in the compacted sand (Figure 12). The water level was raised from the bottom of the sand to slightly above soil surface to achieve saturated soil conditions. The elevation of water level was adjusted using a pulley system through a thin plastic tube connected to the water container. After an equilibrium time period of 24 to 48 hours, the fully saturated soil condition was ensured. For the purpose of convenience, the average matric suction values of 2 kPa, 4 kPa were used in this study which were achieved maintaining water levels at 450 mm and 650 mm deep from the soil surface, respectively. After achieving equilibrium conditions with respect to targeted matric suction values, the load was applied to the top of the model pile at a rate of 0.7 mm/min to ensure a drained loading condition.

The experiment results showed that the end-bearing capacity of single piles in unsaturated sands were between 2 to 2.5 times higher than that in saturated sands. These results motivated to propose theoretical approaches [19], which were discussed earlier (i.e., Eq. 13 and 14), for estimating pile carrying capacity taking account of the influence of matric suction in unsaturated soils. Figure 13 shows a good agreement with less than 10% deviation between the measured end-bearing capacity values and those calculated by the modified methods. Relatively high R square values for all cases are also shown in this study. Among three methods, the modified Terzaghi [10]method provides results that are closer to the measured values compared to the other two modified methods.

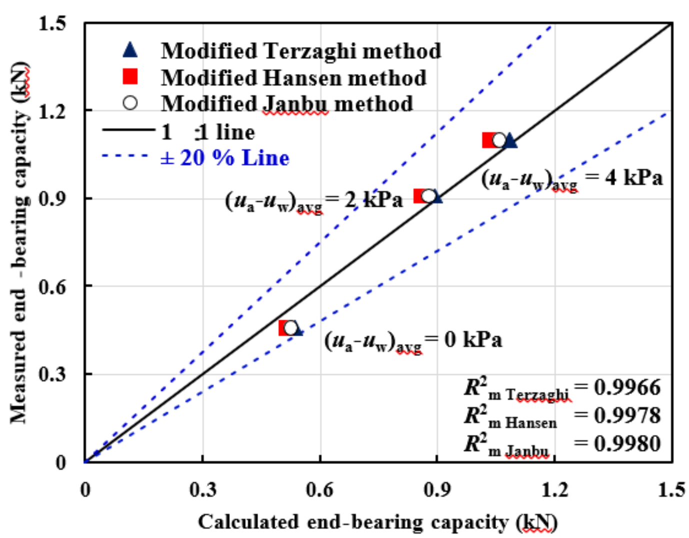


Fig. 13. Comparison between measured end-bearing capacity values and those estimated using the three different modified end-bearing capacity equations (Eq. 13 and 14) (*D*=38.3 mm).

Al Khazaali and Vanapalli [62]conducted experimental studies to investigate the behavior of single model piles and 2 × 2 pile groups in saturated and unsaturated sands in the modified UOBCE equipment, which was succinctly discussed earlier. Model piles of 38.1 mm diameter and 300 mm embedded length with smooth and rough shafts with three different pile center-to-center spacing (i.e., 3*D*, 4*D*, and 5*D*) for the pile group, were used in the tests, to investigate the influence of matric suction, roughness of soil-shaft interface, dilation and group effects. A total of 40 tests were performed by varying the water table levels (i.e., 0, 300, 400, 550, and 850 mm deep from the sand surface) to achieve different matric suction profiles. The test setup and preparation for pile groups performed in modified UOBCE is shown in Figure 14. First, a square pit was dug in the middle of soil container in Figure 14c. Then, the model pile or pile group was placed in this pit. The sand surrounding the pile was manually compacted to optimum moisture content while model piles or pile group was supported by wooden frame in Figure 14b. The water supply/drainage valve connected to the soil container was used to control the saturation and desaturation procedures in the sand. The load of 0.5 mm/s rate was applied at the top of model pile or pile groups to simulate a drained loading condition.

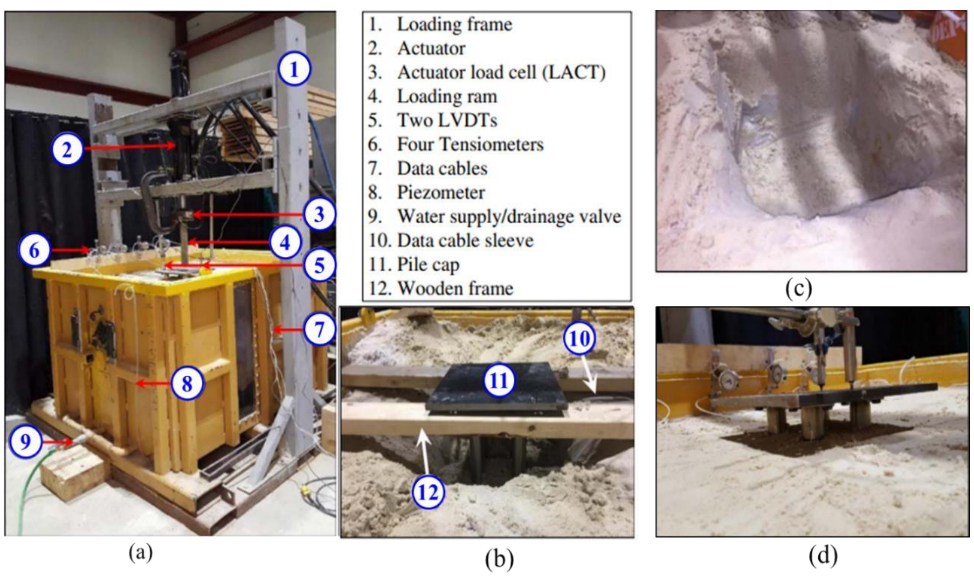


Fig. 14. Test setup for model pile group test (Modified after AI Khazaali and Vanapalli [62])

The results of tests showed that the bearing capacity of both single piles and pile groups were significantly improved due to the contribution of matric suction to shear strength and stiffness of unsaturated soils. The ultimate bearing capacity$P_{ult}$was observed toincrease linearly in the boundary effect zone (BEZ) until suction reached airentry value (i.e., AEV). Then P_{ult} increases nonlinearly in the transition effect zone (TEZ) and reduced in the residual zone of saturation (RZS). Based on the tests on model piles fabricated with smooth and rough surfaces, the results indicated that the shaft roughness has significant effect on pile shaft resistance since threaded shaft have more notable influence on behavior of a single pile in comparison with smooth shafts. The results also showed that the stress bulb zone generated by pile groups is several times deeper than it generated by individual piles. The stress state in the unsaturated bulb zone changes due to group action effects, which contribute to variation of moisture regime. Such changes will influence both shear strength and stiffness of soil and thereby influence the bearing capacity of pile groups in unsaturated soils. More comprehensive information is summarized in Al Khazaali and Vanapalli [62].

Laboratory tests on fine-grained soil . Vanapalli and Taylan [40]have conducted a series of single model pile tests to study the contribution of matric suction on the pile carrying shaft capacity in UFG soils under undrained and drained loading conditions. A glacial till, IHT was used for performing model pile tests. The schematic of the model pile test is shown in Figure 15.

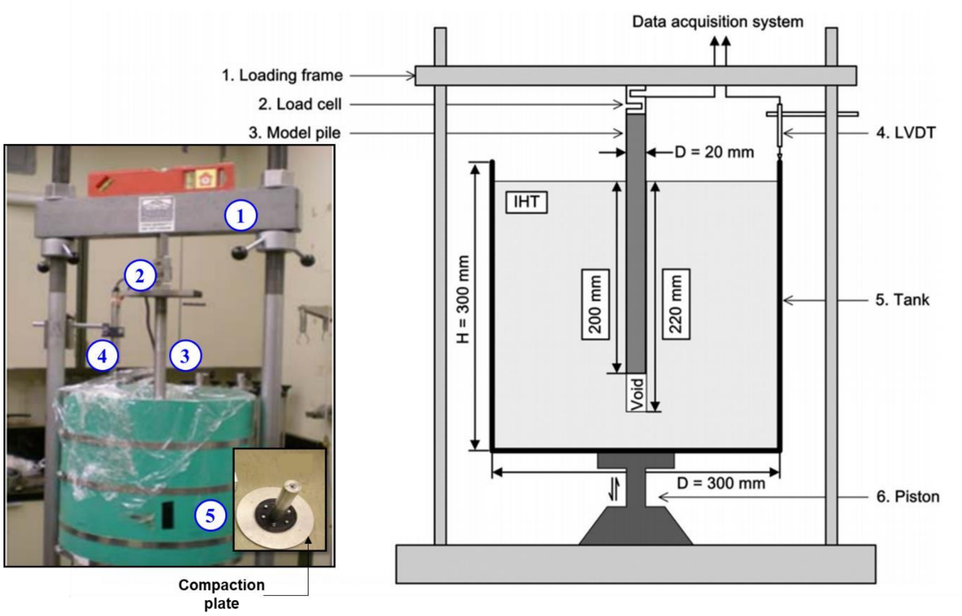


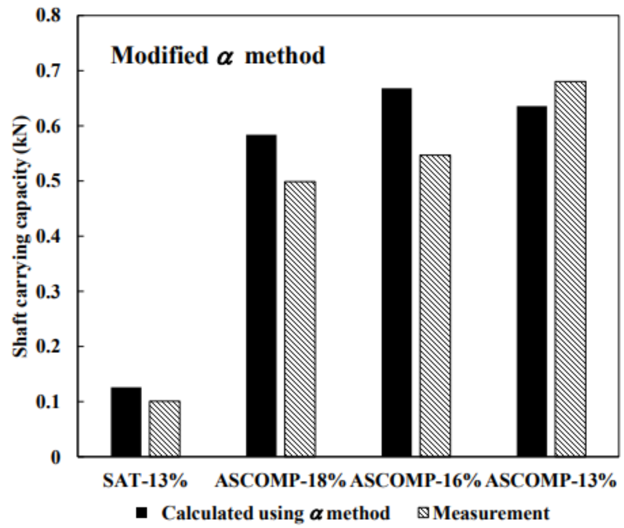
Fig. 15. Test setup for single model pile loading test (Modified after Han and Vanapalli [66])

The soil was placed in a 300 mm depth and 300 mm diameter cylinder tank. It was compacted statically under 350 kPa stress using a specially designed compaction plate. The soil samples were compacted at four different initial water contents: (i) w = 13% (i.e., as-compacted condition, referred as ASCOMP-13%), (ii) w = 16% (ASCOMP-16%), (iii) w = 18% (ASCOMP-18%), and (iv) w = 13% in fully saturated condition (SAT-13%). The SAT-13% condition was achieved by allowing the water to flow downward into the ASCOMP-13% soil sample through the apertures of compaction plate. Axis-translation technique with a modified null pressure plate [63]was used to measure matric suction of soil samples.

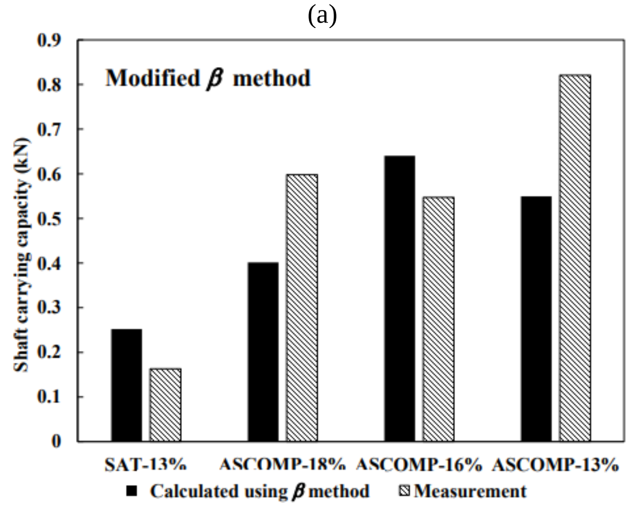
After the soil was compacted, a thin wall sampling tube of 18.7 mm was used to drill a vertical hole down to 220 mm deep from the soil surface. Model piles used in this test were stainless steel piles with 20 mm diameter, which was slightly larger than the diameter of sampling tube in order to obtain a good contact between model piles and surrounding soils. The model pile was loaded using a triaxial test loading frame. LVDT was used for measuring the pile displacement and load cell was for measuring the applied load at the top of pile. Model piles were installed to a depth of 200 mm after the borehole drilling was done. A 20 mm depth gap under the base of pile was set up to eliminate the end bearing resistance while loading the model pile. After the preparation of tests, model piles were subjected to a strain rate of 0.0120 mm/min loading to simulate drained loading conditions. This loading rate is consistent with the tests performed by Gan et al. [3]and Vanapalli et al. [5]on the same soil under drained conditions. A relatively faster loading rate of 1.4 mm/min were controlled to simulate undrained loading conditions.

Sample	Matric suction (kPa)	S (%)	Measured c_u (kPa)	Estimated c_u (kPa)	α from [64]	Back-calculated α
SAT-13%	0	100	11.5	11.5	0.9	0.71
ASCOMP-18%	55	83	58	60	0.82	0.7
ASCOMP-16%	110	65	68	71	0.67	0.57
ASCOMP-13%	205	44	80	62	0.75	0.79

The comparison between the measured from model tests and estimated shaft carrying capacity using the modified α ,β , and λ methods for UFG soil were shown in Figure 16. The measured matric suction was 205 kPa, 110 kPa and 55 kPa for ASCOMP-13%, ASCOMP-16%, ASCOMP-18%, respectively. Table 2 shows the comparison between the measured and estimated undrained shear strength of IHT using Eq.12. The determination of α values obtained from the correlation charts [64] and back-calculated from experimental results were also summarized in Table 2. The coefficient, β = 0.3 was used for both the saturated and unsaturated soils based on the value of soil-pile interface friction angle δ′. Vanapalli and Taylan [65]suggested a relationship between λ and the ratio of pile diameter to pile length *d*/*L*. A value of λ = 0.32 was used in the study. The results show significant increase in shaft carrying capacity due to the influence of matric suction. Also, the results calculated by the modified α, β, and λ methods provide a good agreement with those measured results in model pile tests.



(a)



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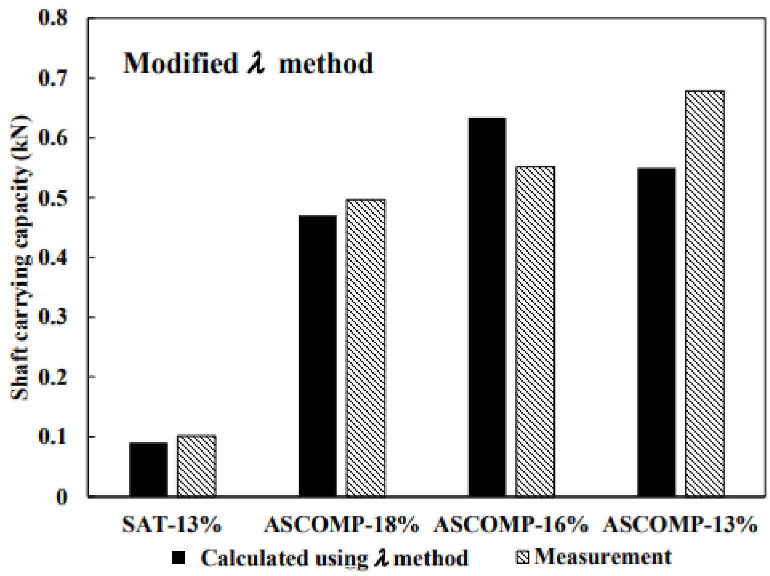

(c)

Fig. 16. Comparison between measured and predicted shaft carrying capacity the modified α, β, and λ methods

5 Summary

The bearing capacity and settlement behavior are two key properties required in the design of shallow and deep foundations. There is a strong relationship between the shear strength and the bearing capacity of soils. Due to this reason, geotechnical engineering pioneers have developed several approaches for determining the bearing capacity of shallow and deep foundations under drained and undrained loading conditions using the shear strength properties of the soils. These approaches are widely used in conventional engineering practice because they are simple and provide valuable information required for the design of foundations. However, the bearing capacity of foundations cannot be reliably determined by extending conventional soil mechanics principles for soils that are in a state of unsaturated condition. Significant research has been undertaken during the past three decades for determining the shear strength of unsaturated soils. This paper summarizes the research that has been undertaken at the University of Ottawa, Canada during the past 15 years for determining, interpreting and predicting the bearing capacity of unsaturated soils in which shear strength of unsaturated soils has been used as a tool. The required information for extending these approaches include the saturated shear strength parameters and the soil-water characteristic curve. The modified approaches for interpreting and predicting the bearing capacity of unsaturated soils are consistent with the approaches used for saturated soils in conventional geotechnical engineering practice. There is a good comparison between the predicted or estimated results from the proposed modified approaches and the experiments undertaken both in the laboratory and field for both shallow and deep foundations. The approaches proposed in this Companion Paper I are promising for implementing our present understanding of the mechanics of unsaturated soils into geotechnical engineering practice for determining the bearing capacity of unsaturated soils. More studies on different unsaturated soils that include large-scale field tests are necessary to better understand the strengths and limitations of the proposed modified approaches for use in the design of foundations in unsaturated soils.

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