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# BEARING CAPACITY AND SETTLEMENT OF FOOTINGS IN UNSATURATED SANDS

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**ABSTRACT:** The focus of the study in this paper is directed towards understanding the influence of three parameters; namely, (i) matric suction, (ii) overburden stress, and (iii) dilation, on the bearing capacity and settlement behavior of surface and embedded model footings in unsaturated sands. The results show that the bearing capacity and settlement behavior of unsaturated sands are significantly influenced by all the three parameters. In addition, comparisons are provided between the predicted and measured bearing capacity and estimated and measured settlement values using the proposed modified Terzaghi's equation and modified Schmertmann's CPT-based method, respectively. There is a good comparison between the predicted/estimated and measured bearing capacity and settlement values for the laboratory and field tests using the proposed modified methods.

*Keywords: Bearing capacity, Settlement, Matric suction, PLT, CPT, Sand*

## 1. INTRODUCTION

The two key properties required in the design of shallow foundations are the bearing capacity (i.e., $q_u$) and settlement (i.e., $\delta$) behavior of soils. Structures such as silos, antenna towers, bridges, power plants, retaining walls and house subdivisions can be constructed on shallow foundations (e.g., spread footings near the ground surface) in sandy soils. The shallow footings are typically designed to transfer the loads safely from the superstructure to the supporting soil such that the settlements are in acceptable limits as per the design and construction codes. The bearing capacity of shallow foundations is conventionally estimated using the approaches originally presented by [1] and [2] assuming the soil is in a state of saturated condition. Typically, shallow foundations are placed above the ground water table and the variation of stress with respect to depth associated with the loads from the superstructure are distributed through the substructure (i.e., shallow foundations) predominantly in the vadose zone (i.e., the zone in which soil is in a state of unsaturated condition). This is true for semi-arid and arid regions and also valid in several situations for many other regions of the world. A framework for interpreting the bearing capacity and settlement behavior from experimental and modeling studies for unsaturated sands is recently evolving ([3], [4], [5], [6], and [7]).

Comprehensive data for interpreting the bearing capacity and settlement behavior of footings in unsaturated sands taking account influence of the matric suction, overburden stress and dilation is however are not available in the literature. Due to these reasons, an extensive experimental program is undertaken to study the bearing capacity and settlement behavior of sandy soil from studies on model footings using specially designed equipment. In addition, comparisons are provided between the predicted and measured values of the bearing capacity and estimated and measured settlement behavior of model footings modifying the original contributions of Terzaghi's equation [1] and Schmertmann's equation [8], respectively. The study shows that there is a good comparison between the predicted/estimated and measured values of the bearing capacity and settlement behavior, respectively using the proposed modified equations.

## 2. BACKGROUND

### 2.1 Bearing Capacity of Soils

Terzaghi [1] and others [2], [9] studies were directed towards understanding the bearing capacity of shallow foundations in saturated or dry conditions using conventional soil mechanics. However, soils are typically found in a state of unsaturated condition in semi-arid and arid regions. Therefore, estimation or determination of the bearing capacity of shallow foundations using conventional soil mechanics for these regions may underestimate the bearing capacity and lead to conservative and costly foundation designs.

Several researchers carried out investigations to study the bearing capacity of unsaturated soils ([10], [11], [12], [13], [14], and [15]). Investigators [16] designed special equipment and conducted studies to understand the bearing capacity of surface model footings in a sandy soil. These studies have shown that the unsaturated soils with matric suction values in the range of 2 to 6 kPa contribute to an increase in 5 to 7 times higher bearing capacity values in comparison to saturated condition. A framework was provided by [3] to predict the variation of bearing capacity of a soil with respect to matric suction using the saturated shear strength parameters ($c'$ and $\phi'$) and the Soil-Water Characteristic Curve (SWCC).

### 2.2 Settlement of Shallow Footings

The shallow footings are typically designed in sandy soils such that the allowable settlement is less than 25 mm. In addition, applied loads from the superstructure need to be safely carried to the soil below the footing with a factor of safety recommended by the design and construction codes. Elastic or immediate settlements in sandy soils are assumed to occur instantaneously when static loads are applied.

Several empirical equations are proposed in the literature that can be used in the estimation of the settlement of footings in sands based on the cone penetration tests (CPT) results ([8], [9], and [17]). The presently available methods in the literature overestimate the settlements leading to an overly conservative footing design ([7], and [18]). This can be attributed to ignoring the influence of matric suction below the foundations while determining the settlement of foundations in sands.

Simple relationships are proposed by researchers [7] modifying the Schmertmann's method [8] that is conventionally used in practice for settlement estimations from the CPT results. The modified method was successfully used in the estimation of the settlement behavior of model footing tests and full-scale footings tested in-situ under both saturated and unsaturated conditions in sandy soils. The focus of the present study is to understand the influence of the capillary stresses (i.e., matric suction), overburden stress (i.e., confinement), and dilation on the variation of the bearing capacity and settlement behavior of both surface and embedded footings in unsaturated sand. In the present study, the same sand used by investigators [16] is tested.

## 3. THE TESTED SOIL

### 3.1 Soil Properties

The soil used in this study can be classified according to the *USCS* as poorly graded fine sand, *SP*. The effective internal friction angle, $\phi'$ was 35.3°. The average dry unit weight and specific gravity were 16.02 kN/m$^3$ and 2.64 respectively.

### 3.2 Dilation in Sandy Soils

The effective cohesion, $c'$ and the angle of internal friction, $\phi'$ were determined from the direct shear test (i.e., *CD* test) results. Several studies suggest the effective overburden stress (i.e., confinement) and soil density influence the dilatancy behavior of sands ([9], [19], and [20]). The dilatancy angle, $\Psi$ is always less than the effective friction angle, $\phi'$ based on the studies by [21]. The dilation behavior of sand can be attributed to the soil particles rolling on top of each other without crushing during the shearing stage. Experiments (using steel shots that do not break down during shearing) conducted by [22] have shown that an increase in confinement leads to a decrease in the dilatancy angle, $\Psi$. More recent studies show that the dilation of sand decreases with an increase in the effective overburden stress [23]. Therefore, in the analysis of surface model footing results of the present study, the bearing capacity and settlement behavior of shallow footings were interpreted taking into account the influence of dilation on the effective friction angle, $\phi'$ of the tested sand.

## 4. EQUIPMENT AND METHODOLOGY

Fig. 1 shows the details of the modified University of Ottawa Bearing Capacity Equipment (modified UOBCE). Schematic of a sectional view of the equipment is shown in Fig. 2.

The modified UOBCE is specially designed to determine the variation of bearing capacity and settlement of sands with respect to matric suction using model footings which are interpreted similar to plate load tests (i.e., PLTs). In the remainder of the paper, model footing tests are referred to as model PLTs for brevity. The equipment setup consists of a rigid steel frame made of rectangular section pipes with thickness of 6 mm and a steel box of 1500 mm (length) × 1200 mm (width) × 1060 mm (depth). The test box can hold up to 3 tons of soil and the capacity of the loading machine (i.e., Model 244 Hydraulic Actuator) with stroke of 250 mm) is 28.5 kN. The model PLTs were performed using different strain rates of 1.2 mm/min and 2.5 mm/min. The results suggest that the load carrying capacity of the sand is not influenced by the two different strain rates used in the present study.

The equipment used in the present study (see Fig. 1) in terms of test box size and its loading capacity is twice in comparison to the UOBCE designed and used by [16]. The equipment in the present study has special provisions to achieve different degrees of saturation conditions below the model footings similar to the original UOBCE. The variations of matric suctions with respect to depth in the unsaturated zone of the test box were measured using commercial Tensiometers.

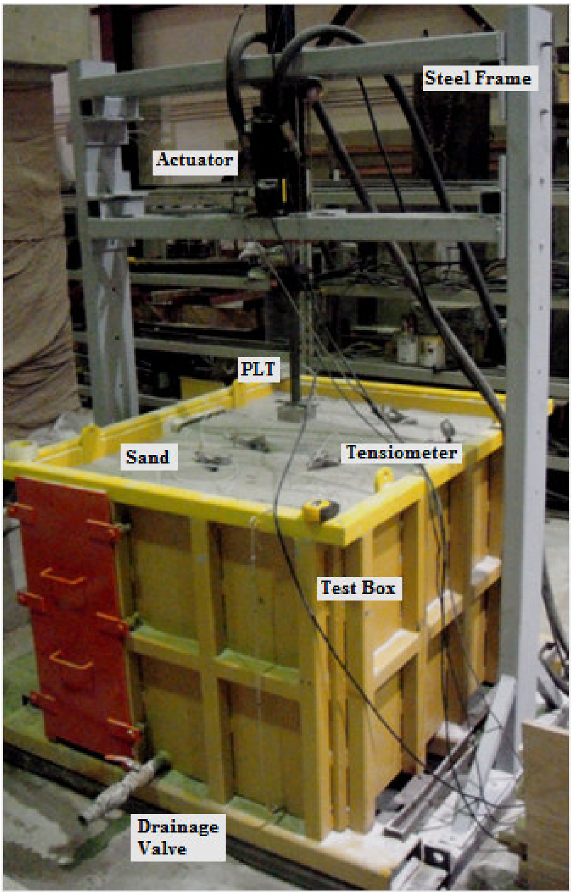


Fig. 1 Modified University of Ottawa bearing capacity equipment (modified UOBCE).

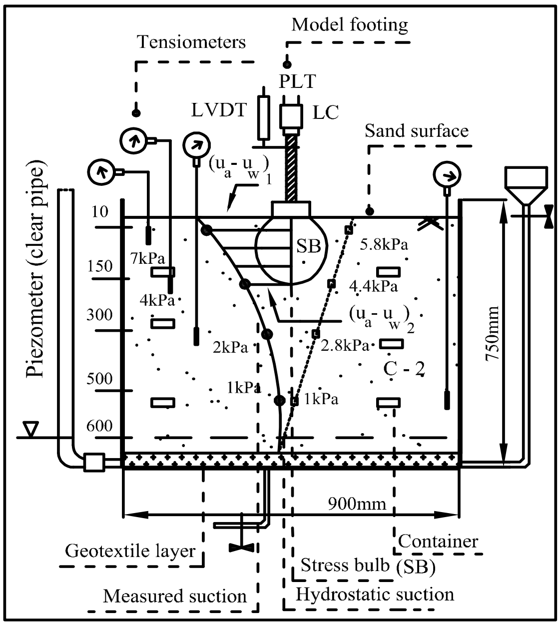


Fig. 2 Schematic to illustrate the procedure used for estimating the average matric suction of 6 kPa within the stress bulb zone of surface footing.

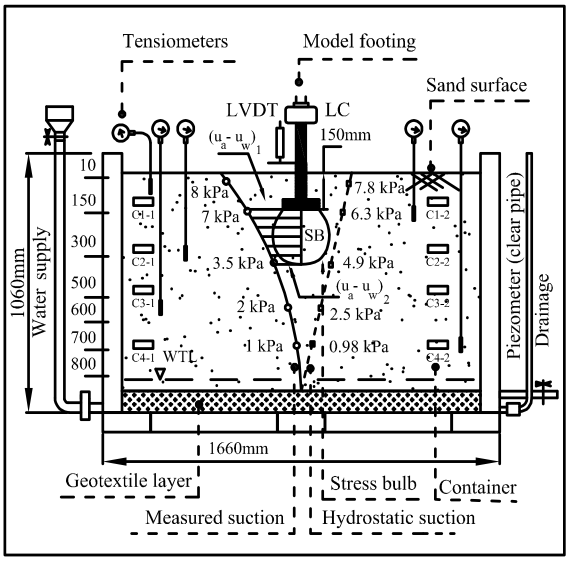


Fig. 3 Sectional view of the modified UOBCE test box with the average matric suction of 6 kPa within the stress bulb zone of embedded footing.

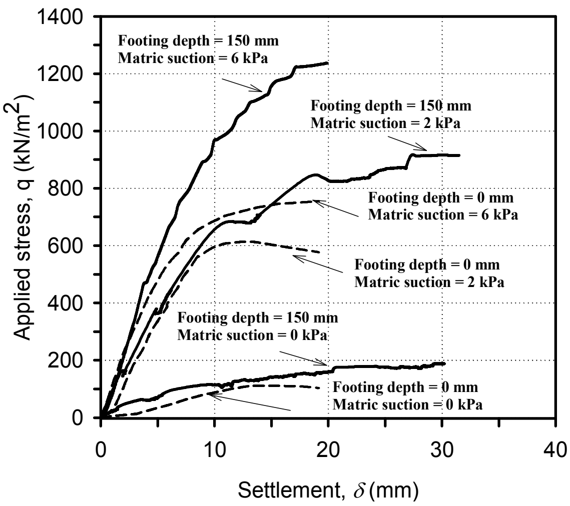


Fig. 4 Relationship between the applied stress versus settlement behavior of surface and embedded model footing tests (PLTs) of 150 mm × 150 mm using the UOBCE and the modified UOBCE.

## 5. LABORATORY PLT AND CPT TESTS

Several tests were conducted to determine the bearing capacity of the tested sand with different values of matric suction using surface PLTs (i.e., model footing depth, $D_f$ = 0 mm), embedded PLTs (i.e., $D_f$ = 150 mm) and CPTs (i.e., cone penetration tests). A minimum of three tests were conducted and average values are reported in this paper.

### 5.1 Surface Plate Load Tests (PLTs)

Model PLTs of 150 mm × 150 mm (i.e., surface footings) were conducted by researchers [16] using the UOBCE (see Fig. 2) which consists of a rigid-steel box of 900 mm (width) × 900 mm (width) × 750 mm (depth). Applied stress versus settlement relationships for surface model footing of 150 mm × 150 mm from that study are summarized in Fig. 4.

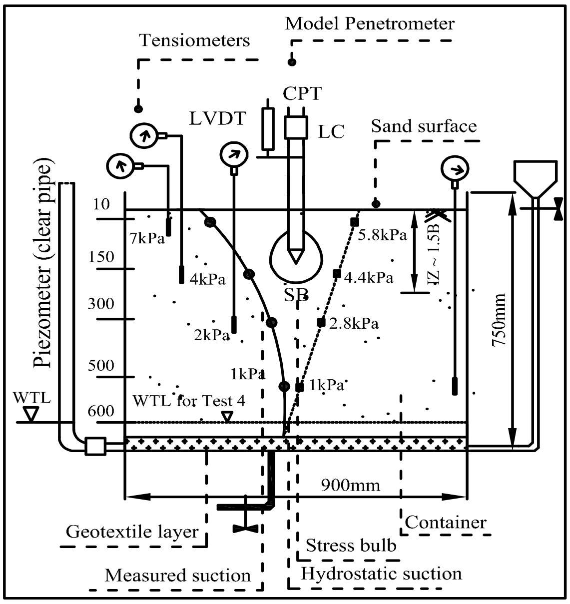


Fig. 5 Schematic to illustrate the procedure used for estimating the average matric suction of 6 kPa within the influence zone (IZ) in the UOBCE.

### 5.2 Embedded Plate Load Tests (PLTs)

In this series of tests, the model footing of 150 mm × 150 mm size is placed at a depth of 150 mm below the soil surface to investigate the effect of the overburden stress. The tests were conducted at two different average matric suction values below the footing (i.e., 2 kPa and 6 kPa) and saturated condition (i.e., matric suction is 0 kPa).

Equilibrium conditions with respect to matric suction were typically achieved in a period of 48 hrs in the test box shown in Fig. 3. Table 1 summarizes a typical set of results in which the average matric suction in the vicinity of the footing base is 6 kPa (see Fig. 3).

The model footings embedded in the modified UOBCE test box are analyzed considering the influence of average matric suction value in the proximity of the stress bulb zone which is equal to depth 1.5*B*. Fig. 3 provides details of the procedure used in the estimation the average matric suction value for the embedded model footing of 150 mm × 150 mm. The depth 1.5*B* considered is the zone in which stresses are predominant due to the loading of shallow square footings with $D_f /B \leq 1.0$ ([3], [5], and [24]).

Table 1. Typical data from the test box for AVR matric suction of 6 kPa in the stress bulb zone (i.e., 1.5*B*)

| $D^*$ (mm) | $\gamma_t$ (kN/m$^3$) | $w$ (%) | $S$ (%) | $(u_a - u_w)$ AVR (kPa) |
|---|---|---|---|---|
| 12 | 18.16 | 12.10 | 53 | 8.0 |
| 150 | 19.00 | 17.00 | 75 | 7.0 |
| 355 | 19.20 | 19.00 | 82 | 5.0 |
| 500 | 19.50 | 21.00 | 91 | 2.0 |
| 700 | 19.74 | 23.11 | 98 | 1.0 |
| 800 | 19.75 | 23.81 | 100 | 0.0 |

*Depth of Tensiometer from the soil surface

The measured water content and matric suction values from the test box of the modified UOBCE are similar to the corresponding water content and matric suction values in the measured SWCC of the tested sand. More information related to the SWCC is available in a later section.

The measured bearing capacity of the compacted unsaturated sand for both surface and embedded footings were in the range of 5 to 7 times higher than the saturated bearing capacity values.

### 5.3 Cone Penetration Tests (CPTs)

Researchers [25] conducted several CPTs in a compacted sand ($D_r$ = 65 %) in the UOBCE under both saturated and unsaturated conditions (i.e., matric suction values of 1 kPa, 2 kPa and 6 kPa). The test setup, experimental results and analyses of the variation of cone resistance, $q_c$ with penetration depth were presented in [25]. Fig. 5 shows details of the sectional view of the test box used to carry out the CPTs respectively. The measured settlement results of the studies are used to check the validity of the proposed modified Schmertmann's relationships based on the CPTs results in a later section to estimate the settlement of shallow footings in sand under both saturated and unsaturated conditions.

### 5.4 Soil Water Characteristic Curve (SWCC)

Fig. 6 shows the SWCC (drying curve) for the tested sand plotted as a relationship between the degrees of saturation, $S$ and the matric suction, $(u_a - u_w)$ using two different methods. The air-entry value for the sand was found to be between 2.5 kPa and 3 kPa.

In the first method, the SWCC is directly measured from the test box. In the second method, a Tempe Cell apparatus was used in the laboratory for measuring the SWCC. The procedures used in the determination of the SWCC are available in [26].

Fig. 6 shows that there is a good agreement between the SWCC's using both the methods. The objective of the determination of the SWCC was to understand its relationship with the bearing capacity of unsaturated soils and propose a simple method for its prediction.

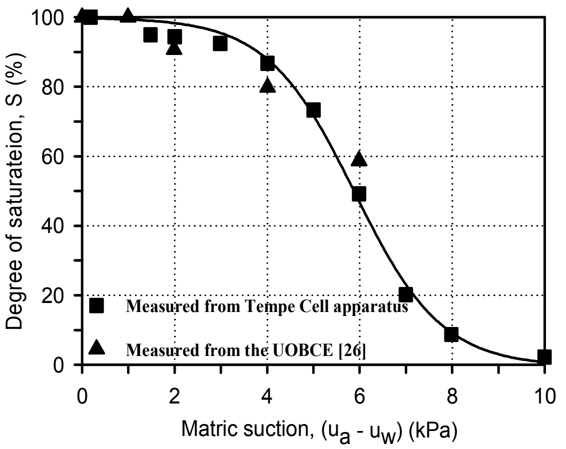


Fig. 6 Measured SWCC from the Tempe Cell apparatus and the test box of the UOBCE.

## 6. BEARING CAPACITY OF UNSATURATED SOILS

Terzaghi [1] suggested Eq. (1) to estimate the ultimate bearing capacity, $q_u$ for strip footings (i.e., plain strain condition) in saturated soils assuming general shear failure conditions:

$$q_u = c' N_c + \gamma D_f N_q + 0.5\gamma B N_\gamma \tag{1}$$

where: $q_u$ = ultimate bearing capacity; kN/m$^2$; $c'$ = effective cohesion, kPa; $\gamma$ = unit weight, kN/m$^3$; $D_f$ = footing base level, m; $B$ = footing width, m; $N_c$ , $N_q$ , $N_\gamma$ = bearing capacity factors which are function of effective friction angle, $\phi'$.

Extending Eq. [1], a semi-empirical equation (i.e., Eq. (2)) was suggested by [3] to predict the variation of bearing capacity with respect to matric suction for surface square footings (using shape factors suggested by [27]) for unsaturated soils using the effective shear strength parameters (i.e., $c'$ and $\phi'$ ) and the SWCC as below:

$$\begin{aligned} q_u &= [c' + (u_a - u_w)_b (\tan\phi' - S^{\psi_{BC}} \tan\phi') \\ &+ (u_a - u_w)_{AVR} S^{\psi} \tan\phi'] N_c \zeta_c + 0.5\gamma B N_\gamma \zeta_\gamma \end{aligned} \tag{2}$$

where: $(u_a - u_w)_b$ = air entry value from SWCC, kPa; $(u_a - u_w)_{AVR}$ = average matric suction, kPa (Fig. 2; Fig. 3); $\phi'$ = effective friction angle,$^o$ ; $S$ = degree of saturation, %; $\psi_{BC}$ = bearing capacity fitting parameter; $\zeta_c$ , $\zeta_\gamma$ = shape factors (from [27]).

There is a smooth transition between the bearing capacity equation proposed by [3] for unsaturated soils and the conventional Terzaghi's bearing capacity equation for saturated soils. In other words, the equation (i.e., Eq. (2) proposed by [3]) will be the same as Terzaghi's bearing capacity equation when the matric suction value is set equal to zero.

The general form of Eq. (2) to estimate the bearing capacity of square footings in unsaturated soils is shown in Eq. (3). This equation takes into account of the influence of overburden stress and the shape factors as follows:

$$\begin{aligned} q_u &= [c' + (u_a - u_w)_b (\tan\phi' - S^{\psi_{BC}} \tan\phi') \\ &+ (u_a - u_w)_{AVR} S^{\psi_{BC}} \tan\phi'] N_c \zeta_c F_c \\ &+ \gamma D_f N_q \zeta_q F_q + 0.5\gamma B N_\gamma \zeta_\gamma F_\gamma \end{aligned} \tag{3}$$

where: $\zeta_q$ = shape factor (from [27]); $F_c$ , $F_q$ , $F_\gamma$ = depth factors

The bearing capacity fitting parameter, $\psi_{BC}$ along with the effective shear strength parameters ($c'$ and $\phi'$ ) and the SWCC are required for predicting the variation of bearing capacity with respect to matric suction assuming drained loading conditions. The bearing capacity fitting parameter, $\psi_{BC}$ can be estimated from relationship provided by researchers [3] in Eq. (4) given below:

$$\psi_{BC} = 1.0 + 0.34(I_p) - 0.0031\,(I_p^{\,2}) \tag{4}$$

Several investigators provided bearing capacity factors for cohesion, $N_c$; surcharge, $N_q$ and unit weight, $N_\gamma$ ([1], [2], and [28]). The values for bearing capacity factors of $N_c$ and $N_q$ provided by various investigators are approximately the same. For this reason, the bearing capacity factors, $N_c$ and $N_q$ originally proposed by Terzaghi [1] were used in the analysis. The values $N_\gamma$ suggested by [28] have been widely used in recent years. For this reason, these values are used in this study.

## 7. COMPARISON BETWEEN THE PREDICTED AND MEASURED BEARING CAPACITY

### Measured and Predicted Bearing Capacity for Surface Plate Load Tests

The bearing capacity values of surface model footings of 150 mm × 150 mm were measured using the UOBCE [16] under both saturated and unsaturated conditions. The test results were interpreted taking account of influence on the dilatancy angle, $\Psi$ for sand.

The dilatancy angle, $\Psi$ was not measured in this study but was approximated for typical sand based on the information reported in the literature. The dilatancy angle, $\Psi$ value assumed to be equal to 10% of effective friction angle, $\phi'$ of 35.3 is equal to 3.53º following [30] (i.e., DS 415. 1984).

Table 2. Bearing capacity (BC), shape and depth factors used in the analysis for the surface PLT

| Effective friction angle, $\phi'$ = 35.3º | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Estimated dilatancy angle, $\Psi$ = 3.53º<br>Modified friction angle, $\phi'_m$ = $(\phi' + \Psi) \approx 39º$ | | | | | | | | |
| B.C. Factors[1] | | | Shape Factors[2] | | | Depth Factors[3] | | |
| $N_c$ | $N_q$ | $N_\gamma$ | $\zeta_c$ | $\zeta_q$ | $\zeta_\gamma$ | $F_c$ | $F_q$ | $F_\gamma$ |
| 86 | 70 | 95 | 1.8 | 1.8 | 0.6 | 1 | 1 | 1 |

[1] from [1] and [28]; [2] from [27]; [3] from [29]

In other words, the modified friction angle, $\phi'_m$ is 38.53º or approximately 39º. Summary of the values of the bearing capacity, shape and depth factors for the surface model footings with a modified friction angle of $\phi'_m$ = 39° are presented in Table 2.

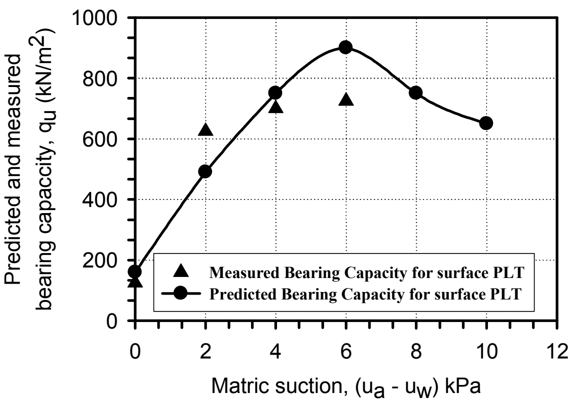


Fig. 7 Comparison between the predicted and measured bearing of 150 mm × 150 mm surface model footing (PLT) in the UOBCE.

The same approach has been extended for analyzing surface model footing results of another three sands tested by [11]. Similar to the test results of sand used in the present study, good comparison was observed between the predicted and measured bearing capacity values for the three sands considering a dilatancy angle value equal to 10 % of effective friction angle, $\phi'$. Details of these discussions are available in [3] and [11]. More recently, [5] have undertaken numerical modeling studies to predict the variation of bearing capacity with respect to matric suction. These modeling results show acceptable comparisons were possible between the predicted bearing capacity values using a dilatancy angle, $\Psi$ which is equal to 10 % of effective friction angle, $\phi'$ and the measured bearing capacity values as shown in Fig. 7.

### 7.1 Measured and Predicted Bearing Capacity for Embedded Plate Load Tests

The bearing capacity of 150 mm × 150 mm embedded model footing (sandy soils in both saturated and unsaturated conditions) was measured using the modified UOBCE (see Fig. 1). The model footing of 150 mm × 150 mm is located at a depth, $D_f$ of 150 mm below the soil surface simulating an overburden stress which also acts as a confinement all around the footing.

Equation (3) is used in the interpretation of the bearing capacity results of embedded footings in saturated and unsaturated sandy soils taking account of the influence of the overburden stress and the shape factors. However, the influence of dilatancy angle, $\Psi$ was not considered.

Table 3. Bearing capacity (BC), shape and depth factors used in the analysis for the embedded PLT

| Effective friction angle, $\phi'$ = 35.3º | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Estimated dilatancy angle, $\Psi \approx 0º$ | | | | | | | | |
| B.C. Factors[1] | | | Shape Factors[2] | | | Depth Factors[3] | | |
| $N_c$ | $N_q$ | $N_\gamma$ | $\zeta_c$ | $\zeta_q$ | $\zeta_\gamma$ | $F_c$ | $F_q$ | $F_\gamma$ |
| 58 | 41 | 45 | 1.7 | 1.7 | 0.6 | 1.4 | 1.2 | 1 |

[1] from [1] and [28]; [2] from [27]; [3] from [29]

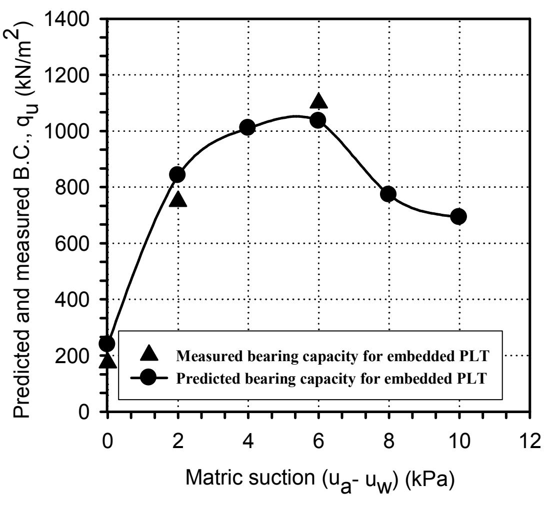


Fig. 8 Comparison between the predicted and measured bearing capacity for embedded model footing (PLT) of 150 mm × 150 mm in the modified UOBCE.

In other words, the bearing capacity factors, shape factors and depth factors were obtained using the effective friction angle, $\phi'$ = 35.3º (see Table 3). There is a good comparison between the predicted and measured bearing capacity values for interpreting the embedded model footing results without taking account influence of the dilatancy angle, $\Psi$ as shown in Fig. 8. Such a behavior can be attributed to the influence of the confinement with a depth which is equal to the width, $B$ of the footing ($D_f$ = 150 mm; as illustrated in Fig. 4). These results are also consistent with the studies of several investigators who have shown that the influence of dilation in the sand decreases with an increase in overburden effective stress or confinement ([19], [23], [31]).

## 8. CORRELATIONS BETWEEN THE CONE RESISTANCE AND THE SETTLEMENT OF FOOTINGS IN UNSATURATED SAND

### 8.1 Proposed Technique

The modulus of elasticity, $E_s$ typically increases with depth in sandy soils and the stresses associated with the applied load decrease with an increase in depth. In other words, settlement will be less in deeper layers in comparison to shallow layers. Schmertmann et al. [8] suggested equation (i.e., Eq. (5)) by extending this philosophy for the estimation of footing settlement in sands using average cone penetration resistance, $q_{ci}$ over a depth of 2$B$ from the bottom of the footing.

$$\delta = C_1\, C_2\, (q_a - \sigma'_{z,d}) \sum_{0}^{2B} \left( \frac{I_{zi}\, \Delta_{zi}}{E_s} \right) \tag{5}$$

where: ($C_1 = 1 - 0.5[\sigma'_{z,d} / (q_a - \sigma'_{z,d})]$) = depth factor; ( $C_2 = 1 - 0.21 \log[t/0.1]$ ) = time factor; $\delta$ = settlement, $q_a$ = footing pressure; $\sigma'_{z,d}$ = vertical effective stress at footing base level; $E_s = f \times q_{ci}$ (i.e., elastic modulus of soil); $I_{zi}$ = influence factor, $B$ = footing width; $q_{ci}$ = resistance of each layer; $f$ = coefficient; $\Delta_{zi}$ = thickness of each layer; and $t$ = time.

This method is widely used in geotechnical engineering practice. One of the key limitations of this method is that it does not take into account the influence of capillary stress or matric suction and is used for sands both in saturated and unsaturated conditions.

Investigators [7] suggested two empirical relationships that can be used in the Schmertmann's equation [8] to estimate the settlements in sands. These empirical relations are useful in estimating the modulus of elasticity, $E_s$ of sands in saturated and unsaturated conditions. The relationships are proposed based on the analysis from the PLT and CPT results. Equation (6) is suggested to estimate the modulus of elasticity, $E_s$ for saturated sands (i.e., $(u_a - u_w)$ = 0 kPa) as given below:

$$E_{s(sat)} = f_1 \times q_{c\ sat} \tag{6}$$

where: $E_{s\ (sat)}$ = modulus of elasticity for saturated homogenous sand, $f_1 = 1.5 \times ((D_r/100)^2 + 3)$ (i.e., $f_1$ is a correlation factor and $D_r$ is the relative density in % ), $q_{c\ (sat)}$ = average cone resistance under saturated sands condition (e.g. within an influence zone, IZ equal to 1.5$B$ from the footing base level) and $B$ = footing width.

Equation (7) is suggested to estimate the modulus of elasticity, $E_s$ for unsaturated sands (i.e., $(u_a - u_w) > 0$ kPa):

$$E_{s(unsat)} = f_2 \times q_{c\ unsat} \tag{7}$$

where: $E_{s\ (unsat)}$ = modulus of elasticity for unsaturated homogenous sands, $f_2 = 1.2 \times ((D_r/100)^2 + 3.75)$ for sands with $D_r < 50$ % or $f_2 = 1.7 \times ((D_r/100)^2 + 3.75)$ for sands with $D_r \geq 50$ %, (i.e. $f_2$ is a correlation factor and $D_r$ is the relative density in %), $q_{c(unsat)}$ = average cone resistance under unsaturated sands conditions (e.g., within influence zone, IZ equal to 1.5$B$ from the footing base level) and $B$ = footing width.

The modulus of elasticity, $E_s$ from Eq. (6) or Eq. (7) can be substituted into Schmertmann's equation (i.e., Eq. (5)) [8] to estimate the immediate settlement.

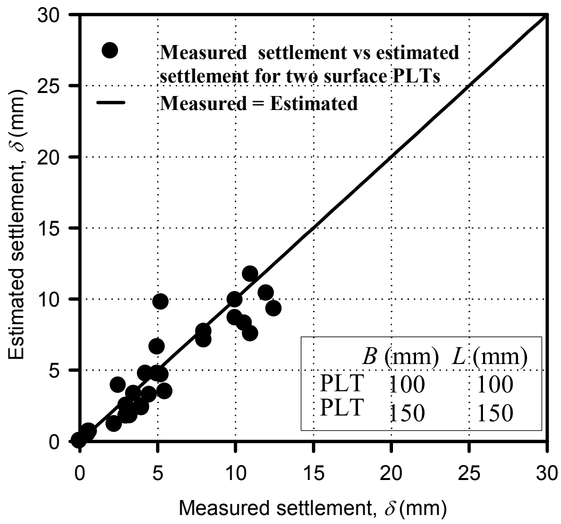


Fig. 9 Comparison between the measured settlements for two surface model footings (PLTs) carried out in the UOBCE and the estimated values using modified Schmertmann's equation.

Fig. 9 provides a typical comparisons between the estimated and measured settlement values for surface model footing of 150 mm × 150 mm in the tested sand with different average matric suction values (i.e., 0 kPa, 2 kPa and 6 kPa) using the proposed relationships into the Schmertmann's equation [8]. The footing settlements decrease with an increase in the matric suction and the overburden stress.

## 8.2 Validation of the Proposed Technique

Comparisons are provided between the estimated settlement (from the proposed technique) and measured settlement (from in-situ large-scale footing load tests (i.e., FLTs) conducted by [32]) values for both saturated and unsaturated sands to validate the proposed technique.

Analysis and comparisons between the estimated and measured settlement values using the proposed relationships in Eq. (5), and measured settlement values for both saturated and unsaturated sands using four in-situ FLTs are provided in this section. Fig. 10 shows the settlement values estimated using the proposed technique for a typical full-scale footing in unsaturated sand from in-situ presented in [32]. The proposed technique provides reasonable settlement estimations in comparison to the measured values.

This can be attributed to the use of different correlations (i.e., $f_1$ and $f_2$) between the cone resistance, $q_c$ and modulus of elasticity, $E_s$ in the proposed relationships which are better in representing the settlement behaviour of sands. Furthermore, the selection of these correlations are not only functions of the CPTs results but also are based on the soil condition (e.g., saturated or unsaturated) and the relative density, $D_r$.

## 9. DISCUSSION OF RESULTS

The correlation factor, $f_1$ value for estimating reliable settlement behaviour of shallow footings in sands under saturated condition is typically in the range of 4.5 to 5.0. On the other hand, the correlation factor, $f_2$ value for estimating the settlement of unsaturated sands falls between 4.5 and 7.5 for the sands evaluated. The need for using such a wide range of $f_2$ values (i.e., 4.5 to 7.5) can be attributed to the influence of matric suction on the cone resistance, $q_c$ values which contributes to a reduction of the settlement, $\delta$ of sands under unsaturated conditions (i.e., $(u_a - u_w) > 0$ kPa).

The correlation factors, $f_1$ and $f_2$ values increase proportionally with an increase in the relative density, $D_r$. These observations are consistent with the conclusions drawn by researchers [33]. Estimated and measured settlement values (using the proposed procedure) of a large-scale footing (FLT) from a reported in-situ case study presented in [32] are compared in this study as shown in Fig. 10. Estimated versus measured settlement values from the proposed procedure and the experimental results respectively for seven footing (four FLTs from [32] and three FLTs from [37]) are presented in Fig. 11.

Because of the limited depth of the test box of both the UOBCE and the modified UOBCE in the laboratory, the maximum average matric suction value (i.e., $(u_a - u_w)_{AVR}$) simulated in the box was 6 kPa. The proposed technique was developed based on test results using average matric suction values ≤ 6 kPa; however, it provides good comparisons for footings constructed in sands for higher matric suction values (i.e., 10 kPa) for the FLTs. The average matric suction value $(u_a - u_w)_{AVR}$ for the FLTs in sand using [32] results was determined assuming a constant matric suction distribution as the depth of groundwater table is at 4.9 m and the natural water content over this depth was constant.

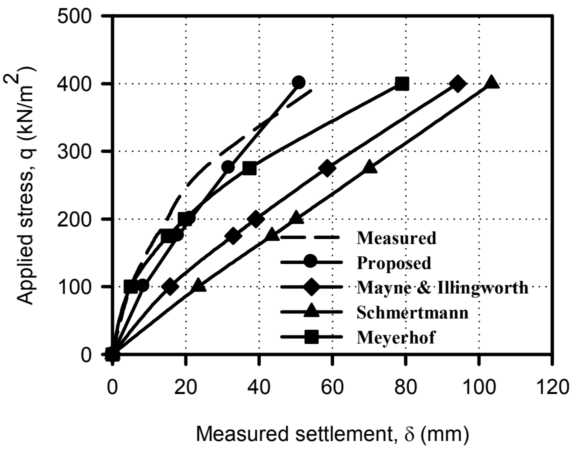


Fig. 10 Comparison between estimated and measured settlement using the modified Schmertmann's equation for a large-scale footing (i.e., FLT of 1500 mm × 1500 mm from [32]).

## 10. SUMMARY AND CONCLUSIONS

The bearing capacity and settlement behaviour of sandy soil under saturated and unsaturated conditions using surface and embedded model footings tests are studied in this research program. The bearing capacity values are underestimated for surface model footings (i.e., the depth of the model footing is equal to zero) when calculations are based on effective friction angle, $\phi' = 35.3°$ for the tested sand both in saturated and unsaturated conditions. Typical value of dilatancy angle for sands is equal to 10 % of effective friction angle, $\phi'$ (see Table 2). Reasonably good comparisons were observed between the predicted bearing capacity values (using Eq. (2) particularly when the influence of the dilatancy angle, $\Psi$ was taken into account) and the measured bearing capacity values of surface footings.

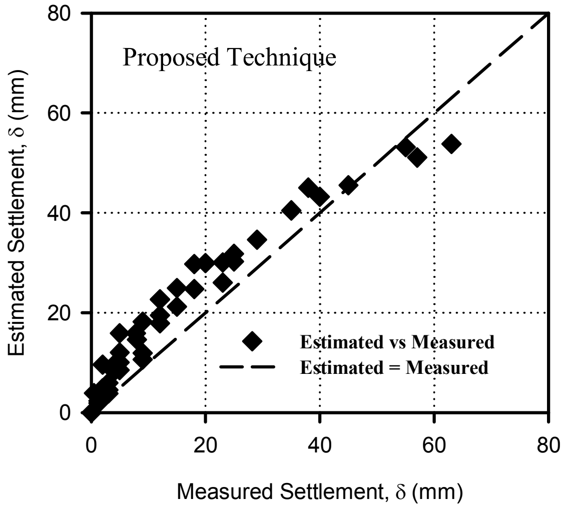


Fig. 11 Comparison between estimated and measured settlements of seven large-scale footing load tests (four FLTs from [32] and three FLTs from [37]) using the modified Schmertmann's equation.

There is no need to increase the effective friction angle, $\phi'$ by 10 % to obtain reasonable comparison between the predicted and measured bearing capacity values (see Table 3) using Eq. (3) for embedded footings. In other words, the dilatancy angle, $\Psi = 0$ for shallow foundations whose $D_f / B \approx 1$. Such a contrasting behavior between surface and embedded footings may be attributed to the contribution of the overburden stress which eliminates the influence of dilation. These observations are consistent with the conclusions drawn in ([19], [23], and [34]) with respect to dilation effects in sandy soils.

The bearing capacity of unsaturated sands increases with matric suction in a linear fashion up to the air-entry value (saturation zone). There is a non-linear increase in the bearing capacity in the transition zone (i.e., from air-entry to the residual suction value). The bearing capacity however decreases in residual zone of unsaturation. Fig. 7 and Fig. 8 show two sets of typical results and behaviour of variation of bearing capacity with respect to matric suction. The behaviour of bearing capacity of unsaturated soils is consistent with the shear strength behaviour of unsaturated sands ([35], and [36]).

Schmertmann's equation [8] (i.e., Eq. (5)) with proposed relationships for modulus of elasticity, $E_s$ for saturated and unsaturated conditions (i.e., Eq. (6) and Eq. (7)) provide good comparisons between estimated and measured settlements for both model PLTs and in-situ FLTs (see Figs. 9, 10, and 11).

The modified Terzaghi and modified Schmertmann's procedures for predicting the bearing capacity and settlement behavior of sandy soils, respectively for both saturated and unsaturated conditions are promising and can be used by practicing engineers.

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