An Experimental Investigation of the Bearing Capacity of Unsaturated Sand Using Cone Penetration Tests

Fathi M. O. Mohamed and Sai K. Vanapalli
Department of Civil Engineering – University of Ottawa, Ottawa, Ontario, Canada

ABSTRACT
Vanapalli and Mohamed (2007) have conducted several model plate load tests on sands and proposed a semi-empirical model for predicting the variation of bearing capacity with respect to suction. The saturated shear strength parameters and the soil-water characteristic curve (SWCC) are required for using the proposed model. Several studies have shown that the insitu bearing capacity of sandy soils can be reliably determined from the cone penetration test (CPT) results. The CPTs are simple and can reduce the costs associated with large scale field bearing capacity tests such as the plate load tests. In this paper, CPTs were conducted in a specially designed tank at the University of Ottawa to experimentally investigate the variation of bearing capacity of sand with respect to matric suction. The test results of the CPTs are consistent with the measured bearing capacity values from the model plate load tests. The tests results presented in this paper are encouraging to use the CPTs in estimating the insitu unsaturated bearing capacity of sands.

RÉSUMÉ
Vanapalli et Mohamed (2007) on mené à bien plusieurs essais de chargement sur plaque sur du sable et ont proposé un modèle semi-empirique afin de prédire la variation de la capacité portante en fonction de la succion. Les paramètres de résistance au cisaillement du sol saturé et la courbe de rétention d'eau (CRE) son requis afin d'utiliser le modèle proposé. Plusieurs études ont démontré que la capacité portante in-situ de sols sablonneux peut être déterminée de façon fiable à partir des résultats d'essais de pénétration au cône (CPT). Les CPT son simples et peuvent réduire les coûts associés au essais à grande échelle de capacité portante sur le terrain, tels que les essais de chargement sur plaque. Dans cet article, les CPT ont été effectués dans une cuve spécialement conçue à l'Université d'Ottawa, afin d'étudier expérimentalement la variation de la capacité portante du sable en fonction de la succion matricielle. Les résultats des essais du CPT sont consistants avec les valeurs de la capacité portante obtenues à partir d'essais de chargement sur plaque à échelle réduite. Les résultats présentés dans cet article encouragent l'utilisation du CPT dans l'estimation de la capacité portante du sable sur le terrain.

1 INTRODUCTION

One of the key parameters required in the design of foundations is the bearing capacity of soils. Terzaghi (1943) proposed an equation for estimating the bearing capacity of soils in terms of saturated shear strength parameters (c' and ϕ'), bearing capacity factors and footing dimensions. Several other bearing capacity equations are also available in the literature along similar lines including shape, depth and inclination factors that are used in conventional engineering practice (Terzaghi and Peck 1948, Skempton 1951, Meyerhof 1956, Vesic 1963 and Broms 1964). All these equations are developed assuming saturated conditions for the soil. However, these equations are also used for estimating the bearing capacity of unsaturated soils ignoring the contribution of matric suction. Several studies have shown that the bearing capacity of soils estimated using the conventional approaches leads to conservative estimates (Oloo et al. 1997, Miller and Muraleetharan 1998, Costa et al. 2003, Mohamed and Vanapalli 2006 and Vanapalli and Oh 2007).

Foundations are typically placed above the ground water table where the soil is in a state of unsaturated condition. In semi-arid and arid regions, foundations are more likely to be embedded in soils that are in a state of unsaturated condition for their entire design life. Therefore, determination of the bearing capacity of unsaturated soils is of practical interest to geotechnical engineers who deal with soils that are more commonly in a state of unsaturated condition. Also, the performance of a foundation can be more realistically estimated if the variation of bearing capacity with respect to matric suction is known.

Vanapalli and Mohamed (2007) have proposed a semi-empirical model for predicting the variation of bearing capacity with respect to matric suction under drained loading conditions. This model uses the saturated shear strength parameters (c' and ϕ') and the soil-water characteristic curve (SWCC). The model was proposed based on experimental studies undertaken on sandy soils using surface plate load tests (i.e., model footings). There is a smooth transition between the proposed model valid for soils that are in a state of unsaturated condition to soils that are in a state of saturated condition. The form of the equation of the proposed model by Vanapalli and Mohamed (2007) will be same as Terzaghi's equation when the matric suction is equal to zero (i.e. saturated condition).

No attempts were made by earlier investigators to determine the bearing capacity of unsaturated soils using the Cone Penetration Tests (CPTs). Some CPTs were performed in unsaturated soils; however, little is known about the influence of the matric suction on the resistance of the cone penetration in the soil (Russell and Khalili 2006 and Shaqour 2007).

In this paper, a simple cone was specially manufactured and used in a laboratory environment to investigate the influence of matric suction on the bearing capacity of a sandy soil. There is a good comparison between the measured results of the CPTs and the plate load tests (PLTs) using average values of the CPTs along a depth of influence zone which is equal to the depth of the stress bulb used for the PLTs conducted on the same soil (Mohamed and Vanapalli, 2006). The results of the study present in this paper are encouraging towards extending the mechanics of unsaturated soils into engineering practice using the CPT results.

2 BACKGROUND

Mohamed and Vanapalli (2006) have undertaken an extensive experimental investigation program to measure the bearing capacity of a sandy type of soil under saturated and unsaturated conditions using model footing tests (i.e. plate load tests). These tests were conducted in specially designed equipment which is referred as the University of Ottawa Bearing Capacity Equipment (UOBCE). The PLTs are used for reliable determination of the bearing capacity of soils; however, these tests are elaborate, need extensive equipment and hence are expensive.

The CPTs are used in routine geotechnical investigations including determination of the bearing capacity of soils (Campanella et al. 1983, Hryciw and Dowding 1987, Puppala et al. 1993, Salgado et al. 1997, Huang et al. 1999 and Eslami 2006). The soil resistance is estimated from the CPTs relationships to calculate the bearing capacity of both coarse and fine-grained soils (Yu and Mitchell 1998, Lee and Salgado 2005 and Russell and Khalili 2006).

A series of CPTs were carried out recently by Lehane et al. (2004) in the field where the soil is quartz sand with fines less than 5% in two different periods (i.e. wet season and dry season). The results of their study suggest that the matric suction has a significant influence on the cone resistance in unsaturated soils. Some CPT studies were carried out on two different sands under unsaturated conditions by Russell and Khalili (2006). They concluded that the matric suction approximately doubles the cone penetration resistance. Shaqour (2007) reported Dutch-CPT results in calcareous sand under saturated, wet (unsaturated conditions) and dry conditions and found that the penetration resistance is relatively high in wet conditions (i.e. unsaturated conditions). More recently, Muszynski (2008) reported greater cone penetration resistance from tests carried out in a sandy soil with capillary tension (i.e. matric suction) in comparison to saturated soils.

In the present research study, a lab-manufactured cone which was attached to a sleeve was used to determine the variation of the cone resistance with respect to matric suction. Experimental results along with discussions and comparisons between the resistance of the CPTs and bearing capacity determined from the PLTs are provided. The ease of determining the bearing capacity of unsaturated soils from the CPTs is highlighted.

3 EQUIPMENT AND METHODOLOGY

3.1 Test Setup

Mohamed and Vanapalli (2006) designed a test set up to determine the variation of bearing capacity of sands with respect to matric suction using model plate load tests in the University of Ottawa Bearing Capacity Equipment (UOBCE). Several modifications were introduced to the UOBCE for determining the bearing capacity using the CPTs for the present study (Figure 1). The setup consists of a rigid-steel tank of 900 mm-length × 900 mm-width × 750 mm-depth. The test tank can hold 1000 kg of soil and the capacity of the loading machine is 15 kN. Different loading rates can be applied by gears manipulation such that the cone connected to the loading rod can be advanced at a constant rate of strain into the compacted soil into the UOBCE to determine the cone resistance.

The water table in the UOBCE can be controlled in the test tank by adding or removing water from the system using water-supply valve and water-drainage valves. Tensiometers were located at different depths above the water table to measure the matric suction. The loading rod passes through a shaft and two horizontal aluminum channel sections to guide the vertical movement and prevent bending or deformation of the rod (see Figure 1).

The strain rate of the cone used for testing in the present research program was equal to 1.2 mm/min which is the same rate used for conducting model PLTs in the UOBCE for determining the bearing capacity of the same soil. More details about the equipment design and setup are available in Mohamed and Vanapalli (2006).

3.2 Design and Manufacturing of the Cone

A simple cone with base diameter, D of 40 mm, cone tip angle, 60^{o} and cone point of radius of 0.1 mm was specially designed for conducting the experimental investigations reported in this paper. The base projected area, A_c of the cone was 1275 mm^2. The cone was securely connected to a steel push rod with a screw connection to form a continuous axis sleeve of 360 mm length and 40 mm diameter. The cone diameter of 40 mm was chosen such that it represents an average value between the minimum and maximum standard cone diameters recommended by the ASTM D 5778 - 07 (i.e.

36 to 44 mm). In addition, the diameter was chosen such that it would be greater than 20 × D_{50} size of the soil to eliminate the scale effect on the results for testing in typical sandy soils (Phillips and Valsangkar 1987 and Bolton et al. 1999 and Muszynski 2008). The cone was manufactured of hardened steel such that it is suitable to resist wear due to abrasion by soil. Figure 1 provides details of the experimental set up used for the present research program.

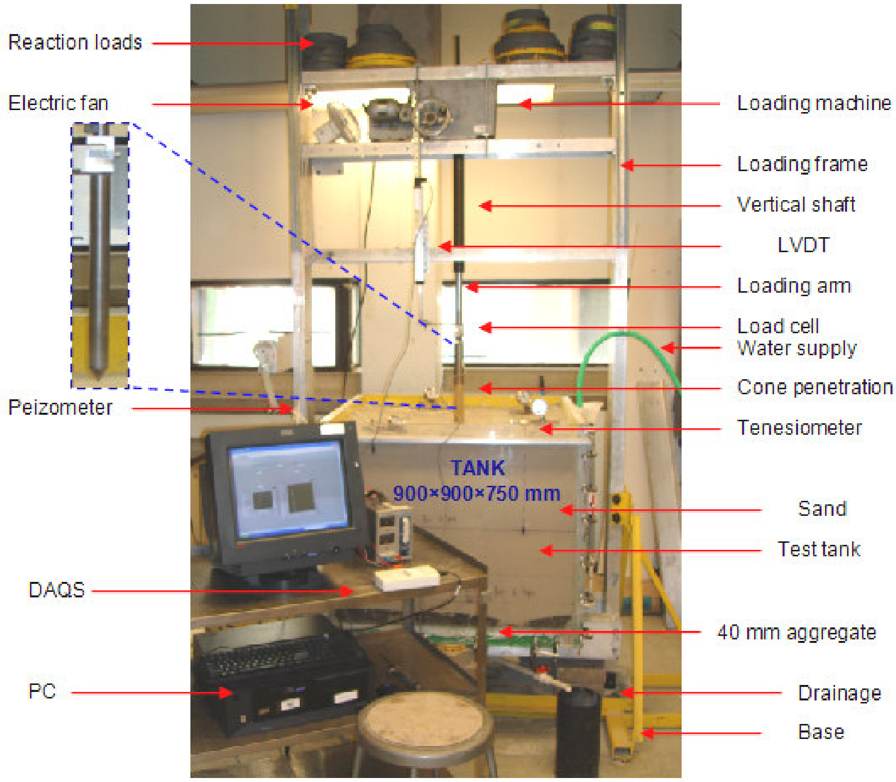

Figure 1. The University of Ottawa Bearing Capacity Equipment (Mohamed and Vanapalli, 2006)

4 MATERIAL DESCRIPTION AND PROPERTIES

Figure 2 provides the grain size distribution of the sand used in the present study. The soil can be classified as poorly graded sand as per the USCS. The properties of the sand are summarized in Table 1. The sand has approximately 5% of silt.

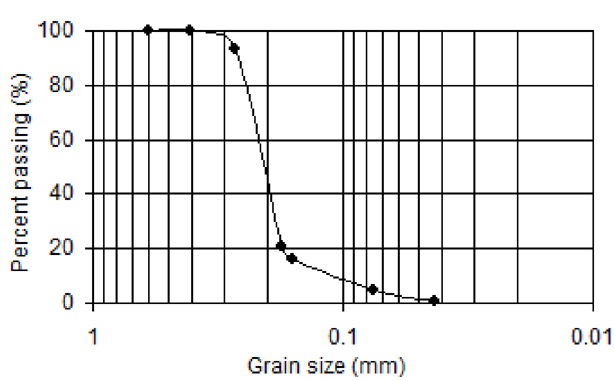

Figure 2. Grain-size distribution for the sand

Table 1. Properties of the tested soil.

Property	Value
Specific gravity, G_s	2.65
Coefficient of uniformity, C_u	1.83
Coefficient of curvature, C_c	1.23
Average dry unit weight of the compacted soil in the tank, γ_d	16.0 kN/m^3
Void ratio, *e* (after compaction)	0.62 – 0.64
Effective shear strength parameters from direct shear tests	$c' =$ 0.6 kPa $\phi' =$ 35.03 ^{o}
Unified soil classification system *(USCS)*	SP

5 TEST PROGRAM

5.1 General

The soil used in the present investigations reported in this paper is the same sand used in an earlier study by Mohamed and Vanapalli (2006). The objective of the study was to determine the bearing capacity of the sand both in saturated and unsaturated conditions using model plate load tests in that study. In the present study, a number of tests were conducted to measure the cone resistance of the compacted sand in the UOBCE under identical test conditions of saturated and unsaturated conditions reported in Mohamed and Vanapalli (2006). The first series of tests were carried out under saturated condition and the later series of tests were conducted under three different unsaturated conditions (i.e. 1 kPa, 2 kPa and 6 kPa).

The ultimate tip resistance typically mobilized at the cone tip by penetrating the cone to a depth of 10 times the cone diameter was measured (Meyerhof 1956, Eslami 2006). Studies by Salgado (2008) have shown that a pile tip mobilizes maximum resistance when the base penetrates into the compacted sand layer by at least 2 × the diameter. In this paper, the cone was penetrated to approximately a depth of 400 mm to satisfy the above criteria.

5.2 CPT Results under Fully Saturated Sand Condition

The compacted sand in the UOBCE tank was saturated by raising the water table by opening the water supply valve gradually. The compacted sand was saturated from the bottom aggregate layer such that water advances in the upwards direction. This technique allowed the air from the compacted sand to be expelled from the top surface of the sand. All the tensiometers in the test tank indicated zero readings when the water level reached the top surface confirming saturated condition (i.e. $(u_a - u_w) = 0$) of the soil. A number of CPTs were conducted after ensuring saturated conditions and average values of the results was used in the results analyses.

5.3 CPT under Unsaturated Sand Conditions

The soil was saturated similar to the procedure followed in the previous section. The water table was then lowered down to different levels of depth (using the drainage valve located at the bottom of the tests tank) to achieve varying capillary stresses (i.e. matric suction) values above the water table. Three different series of CPTs were carried out to achieve three different scenarios of unsaturated conditions. As the objective is to compare the results of the CPTs conducted in this paper with the bearing capacity results of the PLTs carried out in Mohamed and Vanapalli (2006), the same average matric suction value (see equation 1) were achieved in the region of stress bulb (i.e.1.5B). Figure 3 shows a schematic of the procedure used for determining the average matric suction in the stress bulb of the PLTs and Figure 4 shows the procedure for estimating the influence zone for the CPTs.

$$(u_a - u_w)_{AVR} = \left[\frac{(u_a - u_w)_1 + (u_a - u_w)_2}{2}\right] \quad [1]$$

where:

$(u_a - u_w)_{AVR}$: average matric suction in the influence zone as shown in Figure 3 and Figure 4.

$(u_a - u_w)$: measured matric suction by tensiometers, kPa

The CPTs were conducted under different matric suction values after ensuring equilibrium conditions. Two more series of tests performed under average matric suction values of 2 kPa and 6 kPa by placing the water table at a depth of 150 mm and 600 mm respectively (Mohamed and Vanapalli, 2006). The tensiometers were used to measure the matric suctions in the soil above the water table. The gravimetric water contents were also measured by collecting specimens using small containers with perforations. The small containers were embedded in the unsaturated zone close to ceramic tip of the tensiometers. Figure 4 shows the cross-section of the test tank and provides details of the placement and locations of tensiometers and the small containers (see Table 2).

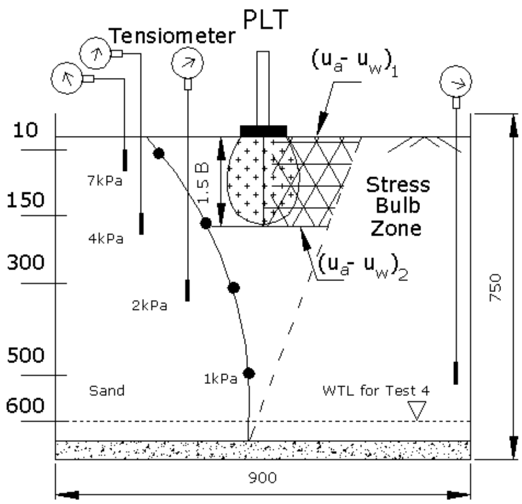

Figure 3. Schematic to illustrate the procedure used for determining AVR matric suction below the PLTs (from Mohamed and Vanapalli, 2006)

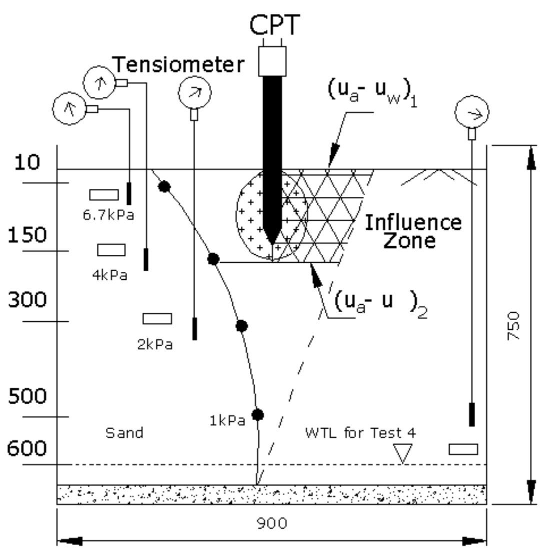

Figure 4. Schematic to illustrates the procedure used for determining AVR matric suction 6 kPa below the CPT

Table 2.Typical data from the test tank for an average matric suction value of 6 kPa in the influence zone.

d (mm)	γ_t (kN/m³)	e	w.c (%)	S (%)	AVR[1] $(u_a - u_w)$ (kPa)
10	18.17	0.63	14.00	58	6.0
150	18.76	0.64	18.33	76	4.0
300	19.20	0.62	19.48	83	2.0
500	19.30	0.64	22.38	93	1.0
600	19.74	0.63	23.76	100	0.0

[1] AVR: average value.

where:

: depth from the soil surface of tank, mm

γ_t : total unit weight, kN/m³

: void ratio

c : gravimetric water content, %

: degree of saturation, %

$(u_a - u_w)$: matric suction, kPa

6 CONE RESISTANCE AND SLEEVE FRICTION FROM THE CPT RESULTS

The cone resistance, q_c can be determined using different methods proposed by researchers (Fleming and Thorburn 1983 and Eslami and Fellenius 1997, ASTM D 5778 - 07). In this paper, since the total applied loads (friction and tip resistance) were evaluated, the cone resistance for the tested soil was determined using the following equation (ASTM D 5778 - 07):

$$q_c = \frac{Q_c}{A_c} \quad [2]$$

where:

q_c : the cone resistance, MPa

Q_c : load carried by the cone, kN

A_c : the cone base area, mm^2

The sleeve friction, f_s (due to sleeve soil interaction) can be directly measured using an electronic cone. However, the sleeve friction in the present testing program was estimated following the ASTM D (5778 - 07) guidelines using the equation below:

$$f_s = \frac{Q_f}{A_s} \quad [3]$$

where:

f_s : the sleeve friction, kPa

Q_f : load carried by the sleeve in saturated condition, kN

A_s : surface area of the sleeve, mm^2

The friction ratio, R_f which is defined as the sleeve friction, f_s divided by the cone resistance, q_c was estimated following the ASTM D 5778 – 07 guidelines:

$$R_f = \frac{f_s}{q_c} \times 100 \quad [4]$$

The sleeve friction is much less than the cone resistance for cohesionless soils such as sands (Nes 2004). Vos (1982) and Bakker (2004) used an electronic cone penetrometer and determined the friction ratio, R_f values and summarized empirical friction ratio values for various soil types (see Table 3).

Table 3. Soil type as function of friction ratio (from Vos 1982 and Bakker 2004).

Soil type	Friction ratio R_f (%) (Vos 1982)	Friction ratio R_f (%) (Bakker 2004)
Coarse sand and gravel	< 0.5	0.2 - 0.6
Fine sand	1.0 - 1.5	0.6 - 1.25
Silt or loam	1.5 - 3.0	1.2 - 4.0
Clay	3.0 - 7.0	3.0 - 5.0
Peat	> 5.0	5.0 - 10.0

Vanapalli and Eigenbrod (2009) extended β – Method in the design of piles using the mechanics of unsaturated soils and proposed equation [5] taking account of matric suction for determining the shaft capacity (sleeve friction) of open-end pile (D = 65 mm) in sandy soil under moist (i.e., unsaturated) conditions.

$$Q_{fus} = Q_f + Q_{(u_a - u_w)} \quad [5]$$

$$Q_{fus} = \beta\left(\frac{L\gamma_d}{2}\right)(A_s) + \left[(u_a - u_w)\left\{\Theta^{\kappa}\left(\tan\phi'\right)\right\}\right](A_s) \quad [6]$$

where:

Q_{fus} : total shaft capacity (sleeve friction), kN

$Q_{(u_a - u_w)}$: shaft capacity (sleeve friction) due to unsaturated condition, kN

β : factor (0.36 – 0.45) as function of tan ϕ and coefficient of earth pressure along the shaft, Ki

γ_d : dry unit weight, kN/m^3

embedded length, m

A_s : surface area of shaft or sleeve, mm^2

$(u_a - u_w)$: matric suction, kPa

Θ : normalized water content (equivalent to degree of saturation)

κ : fitting parameter as function of plasticity index (1 for sandy soils)

ϕ : soil / pile interface friction angle, ^{o}

A sample calculation of the sleeve friction along the influence zone applying equation [5] (for AVR matric suction of 6 kPa along a depth of 150 mm as shown in Figure 4) with κ equal to 1 is summarized below:

$$Q_{fus} = 0.35\left(\frac{16 \times 0.15}{2}\right)(0.01885) + \left[(6.0)\left\{0.58^{1}(\tan 26)\right\}\right](0.01885) = 0.04 \text{ kN}$$

By substituting the value of sleeve friction from Eq. [6] in Eq. [3], the sleeve friction can be determined:

$$f_s = \frac{Q_{fus}}{A_s} = \frac{0.04}{0.01885} = 2.12 \text{ kPa}$$

$$Q_t = Q_c + Q_{fus} \quad [7]$$

where:

Q_t : total applied force (taken by sleeve and cone), kN

For the experimental results conducted in this research, the average measured applied force, Q_t at a depth of 150 mm is 2.093 kN. Using Eq. [7], the applied load on the cone can be determined as 2.035 kPa. By substituting the value of Q_c value back in Eq. [2], the cone resistance, q_c can be estimated.

$$q_c = \frac{Q_c}{A_c} = \frac{2.053}{0.001257(1000)} = 1.62 \text{ MPa}$$

The friction ratio, R_f can be determined by substituting the values of sleeve friction, f_s and the cone resistance, q_c in Eq. [4]. The friction ratio was equal to 0.13% at midpoint of the assumed influence zone length (150 mm). The results of the friction ratio, R_f are lower than the values summarized in Table 3. These results suggest that the load carrying capacity contribution from the sleeve friction is negligible. These observations are consistent with the behavior of pile foundations in sandy soils in which the load carrying capacity is primarily carried at the tip of the pile. The results of the cone resistance for the tested soil under saturated and unsaturated conditions are plotted with respect to the depth in Figure 6.

7 DISCUSSION OF TESTS RESULTS

In CPTs, the cone forces a failure in the soil in-front of the advancing cone tip and forms a mechanism of rupture which is comparable to single pile failure behaviour. Similar pattern of failure mechanisms based on the bearing capacity theory was proposed by many researchers (Terzaghi 1943, Vesic 1963 and De Beer 1965) to approximately visualize the failure load when conducting the CPTs. Along similar lines, a method was proposed by Salgado (2008) to determine the end bearing capacity of a single pile which takes an average load over an influence zone length equal to the summation of pile diameter, D above the pile base and 1.5 × the pile diameter, D below the pile base. Assuming the sleeve with cone as a single or model pile, similar concept can be extended for the CPTs conducted herein to obtain representative value of cone resistance, q_c for comparison with bearing capacity results of PLTs. Three influence zones were assumed below the CPT illustrated in Figure 5 and labeled as: Case (i) represents an influence zone length of 100 mm, Case (ii) represents an influence zone length of 140 mm and Case (iii) represents an influence zone length of 150 mm. A representative value of cone resistance, q_c of each of the series of the CPT results is plotted with respect to matric suction values in Figure 7.

From the measured CPTs results, the cone resistance, q_c under unsaturated conditions (average matric suction values of 1 kPa, 2 kPa and 6 kPa) found to be two to three times higher than the cone resistance for saturated condition as shown in Figure 7. These results are consistent with the observations of Russell and Khalili (2006). The increase in the CPT values can be attributed to the contribution of the matric suction to the bearing capacity of the tested soil similar to the PLTs results (Mohamed and Vanapalli, 2006). The cone resistance increased as the soil condition changes from saturated (0 kPa) condition to unsaturated (1 kPa, 2 kPa and 6 kPa) conditions in the capillary zone as shown in Figure 6.

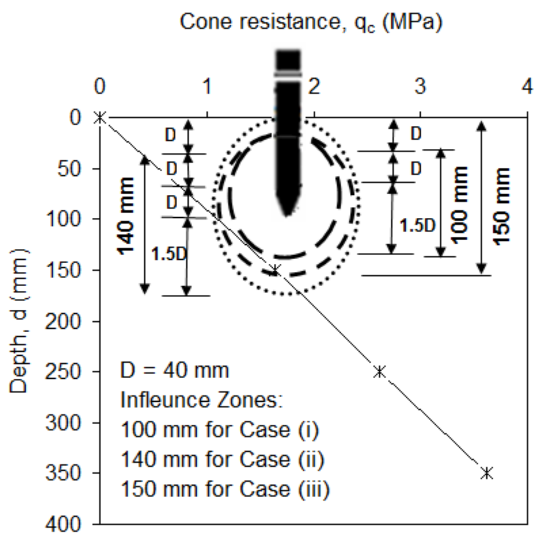

Figure 5. Schematic illustrates the CPT results of three for three cases (i.e. Case (i), Case (ii) and Case (iii))

Figure 7 presents the details of measured bearing capacity of the PLTs (100 mm × 100 mm) from Mohamed and Vanapalli (2006) and the cone resistance obtained from this study. Both model PLTs and the cone resistance from the CPTs show similar contributions of matric suction (i.e., capillary tension) towards the bearing capacity.

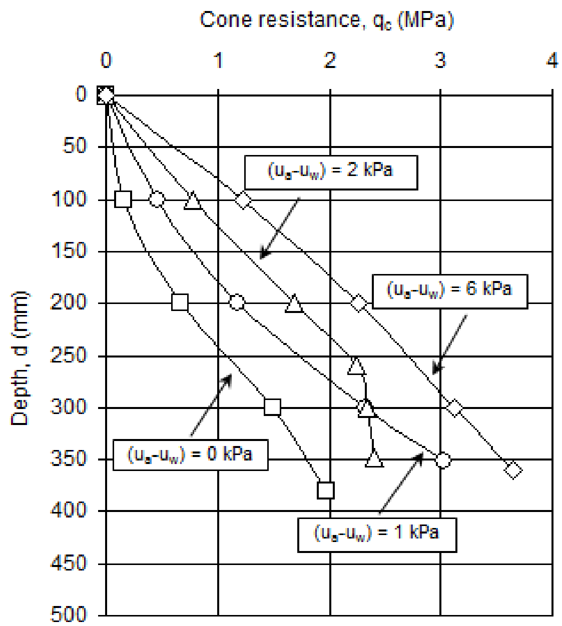

Figure 6. Variation of cone resistance (CPTs) with depth under saturated and unsaturated conditions

8 SUMMARY AND CONCLUSIONS

The results presented in the paper demonstrate significantly low sleeve friction contribution to the load carrying capacity but dramatic increase of the cone resistance when the CPTs were performed under unsaturated conditions. The increase of the cone resistance can be attributed to the contribution of matric suction.

The comparison between the resistances obtained from the CPTs in this study and the bearing capacity results from the PLTs under similar conditions show approximately the same values when the tests are conducted under saturated conditions. In contrast, there was a difference of 20% to 50% between the CPTs results and the PLTs results under unsaturated conditions. The differences may be attributed to the assumed influence zone length presented in Case (i) and Case (ii) respectively. In addition, the cone resistance is also affected by the weight of the soil (i.e. overburden stress) above the cone which causes confinement around the tip and higher resistance than that of the surface PLTs. However, the cone resistance of CPTs and the bearing capacity values of PLTs were approximately the same (for both saturated and unsaturated conditions) using the influence zone length presented in Case (iii) (150 mm). This validates the procedure used for averaging the values of matric suction and cone resistance, q_c over an influence zone length of 150 mm which is equal to the depth of the stress bulb proposed for the PLTs by Vanapalli and Mohamed (2007). The trend of both the resistance of the CPTs and the bearing capacity of the PLTs demonstrate a linear increase up to the air-entry value (AEV = 3 kPa) and then a nonlinear increase beyond that value.

Based on the results reported in this paper, it appears that the influence zone length over which the average cone resistance values should be considered has a significant role on deciding the appropriate and representative cone resistance value (end-bearing capacity) to be used in the design of foundations in unsaturated soils.

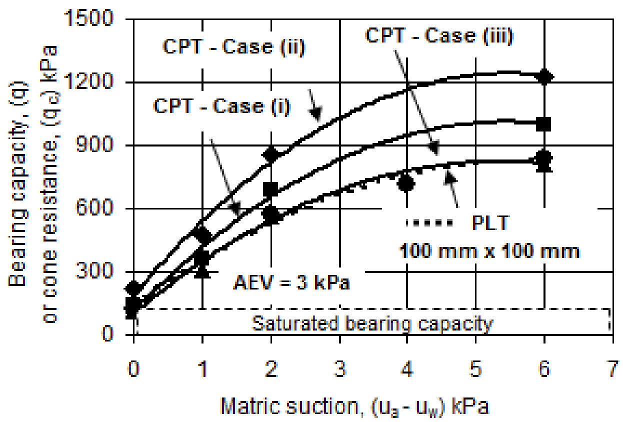

Figure 7. Comparison between the cone resistance from the CPTs and the bearing capacity from the PLTs tests with respect to matric suction

9 ACKNOWLEDGEMENTS

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