uOttawa

L'Université canadienne
Canada's university

FACULTÉ DES ÉTUDES SUPÉRIEURES ET POSTOCTORALES

uOttawa

L'Université canadienne
Canada's university

FACULTY OF GRADUATE AND POSDOCTORAL STUDIES

Fathi Mohamed

AUTEUR DE LA THÈSE / AUTHOR OF THESIS

M.A.Sc. (Civil Engineering)

GRADE / DEGREE

Department of Civil Engineering

FACULTÉ, ÉCOLE, DÉPARTEMENT / FACULTY, SCHOOL, DEPARTMENT

A Semi-Empirical Approach for Interpretation of the Bearing Capacity of Unsaturated Soils

TITRE DE LA THÈSE / TITLE OF THESIS

Dr. Sai Vanapalli

DIRECTEUR (DIRECTRICE) DE LA THÈSE / THESIS SUPERVISOR

CO-DIRECTEUR (CO-DIRECTRICE) DE LA THÈSE / THESIS CO-SUPERVISOR

EXAMINATEURS (EXAMINATRICES) DE LA THÈSE / THESIS EXAMINERS

Dr. S. Sivathayalan

Dr. M. Fall

Gary W. Slater

Le Doyen de la Faculté des études supérieures et postdoctorales / Dean of the Faculty of Graduate and Postdoctoral Studies

A SEMI-EMPIRICAL APPROACH FOR THE INTERPRETATION OF THE BEARING CAPACITY OF UNSATURATED SOILS

Fathi MOHAMED OMAR MOHAMED, B.Sc.,

A Thesis Submitted to the Faculty of Graduate and Postgraduate Studies

under the Supervision of

Dr. Sai K. VANAPALLI, P.Eng.

In partial fulfillment of the requirements for the degree of Master of Applied Science in Civil Engineering

Department of Civil Engineering

University of Ottawa
Ottawa, Ontario
Canada K1N 6N5

August 2006

The Master of Applied Science in Civil Engineering is a joint program between Carleton University and University of Ottawa, which is administrated by the Ottawa-Carleton Institute for Civil Engineering

Your file Votre référence
ISBN: 978-0-494-25809-5
Our file Notre référence
ISBN: 978-0-494-25809-5

ABSTRACT

In many parts of the world, and particularly in semi-arid and arid regions, shallow foundations are located above the ground water table where the soil is typically in a state of unsaturated conditions. The capillary stresses (i.e., matric suction or negative pore water pressure) above the ground water table contribute significantly towards the bearing capacity of unsaturated soils. However, the bearing capacity contribution due to the capillary stresses is routinely ignored in the conventional engineering design of shallow foundations. The bearing capacity of unsaturated soil is also estimated using conventional approaches that are used for saturated soils ignoring the influence of matric suction. This is a conservative approach used in the design of shallow foundations for unsaturated soils. For this reason, it is important to understand the influence of matric suction on the bearing capacity of unsaturated soils. In addition, a framework to interpret the bearing capacity of unsaturated soils that takes account of the influence of matric suction will be useful.

In the present research program, an extensive experimental study is undertaken to investigate the bearing capacity of a compacted coarse-grained soil under both saturated and unsaturated conditions. Two different series of tests were carried out in a bearing capacity tank using two different sizes of square shaped model footings (i.e. 100 mm × 100 mm and 150 mm × 150 mm). The bearing capacity tank of the size 900 mm × 900 mm and 750 mm height was specially designed and fabricated in the University of Ottawa, Faculty of Engineering Student Workshop to undertake the present research program. The first series of tests were conducted to determine the bearing capacity of the compacted sand in the tank under fully saturated conditions. The second series of tests were conducted to study the influence of matric suction on the bearing capacity of compacted sand under different unsaturated conditions.

The ground water table level in the bearing capacity tank was varied using the drainage valves to achieve different values of matric suction below the model footings. The variation of the matric suction with respect to depth below the model footing was measured using four commercial Tensiometers. These Tensiometers were located at different depths in the proximity of the expected stress bulb (i.e., 1.5 times the width of the footing) below the loaded footing. In addition, piezometers were also placed in the tank to monitor the ground water table levels. The model footings were loaded using a loading frame until failure to measure the bearing capacity. The sand used in the bearing capacity tank was compacted to achieve a density index value of approximately equal to 65% such that the failure of the footing will be in accordance with the general shear failure criteria. The experimental results of the test program demonstrated that the bearing capacity of unsaturated soils is approximately 5 to 7 times higher than the bearing capacity of the same soil under saturated conditions. As expected, there was a non-linear increase in the bearing capacity of the compacted sand with an increase in the matric suction under the model footings. In addition, it was also observed that there is a relationship between the soil-water retention curve (SWRC) and the bearing capacity of unsaturated soils.

Terzaghi's bearing capacity theory for saturated soils was extended to interpret the bearing capacity of unsaturated soils using the saturated shear strength parameters and taking account of the influence of the matric suction. In addition, a simple technique was proposed for predicting the variation of the bearing the capacity of unsaturated soils using the soil-water retention curve (SWRC) and the saturated shear strength parameters. The proposed prediction procedure was similar to the procedure used for predicting the shear strength of unsaturated soils by Vanapalli et al. (1996).

The procedure proposed for predicting the bearing capacity of unsaturated soils was extended for another four soils, of which two of them were coarse-grained and the other two were fine-grained in nature. The data for the four soils was obtained from the literature. The results of the study suggest that the variation of the bearing capacity of unsaturated soils can be reasonably well predicted using the SWRC and the saturated shear strength parameters. The framework of the research program developed based on the studies undertaken on model footings shows considerable promise for extending it to field studies.

ACKNOWLEDGMENENTS

I am very grateful to my supervisor Dr. Sai K. Vanapalli for his solid guidance, encouragement, valuable advice and suggestions throughout this research.

I would like to thank Drs. S. Vanapalli, M. Saatcioglu, V. Garga, M. Pérez, S. Sivathayalan and A. Abd El-Halim for sharing their expertise through the graduate courses that I have taken at both universities. Thanks also to all the technical staff in the department of Civil and Mechanical Engineering especially R. Moore and J. Perrins.

Thanks go to my colleagues Dr. Won Taek Oh and Julio Ángel Infante Sedano and Cevat Catana, graduates students at the University of Ottawa for their assistance.

I also express my sincere gratitude to my parents, brothers and sisters whom I dedicate this thesis. I highly appreciate their concrete support and patience.

Finally, I thank the General Public Committee of Higher Education in Libya (G.P.C.H.E) for providing the financial support during this research. I also want to acknowledge the funding received from National Sciences and Engineering Research of Canada (NSERC) for this project.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 Statement of the Problem

Shallow foundations are usually located above the ground water table where the soil is typically in a state of unsaturated condition. About 33% of the earth's surface is considered either arid or semi-arid and the soils in this region are typically in a state of unsaturated condition (Fredlund and Rahardjo 1993). In such regions, the negative pore water pressure due to capillary phenomena plays an important role in the mechanical behaviour of unsaturated soils.

Considerable research has been undertaken during the last 50 years to interpret the engineering behavior of unsaturated soils using two independent stress state variables, namely matric suction and net normal stress (Bishop and Blight 1963, Fredlund and Morgenstern 1977, Alonso et al. 1990). The matric suction is defined as the difference between the pore air pressure, u_a and pore water pressure, u_w. Several studies have shown that the engineering behavior of unsaturated soils including the bearing capacity is significantly influenced by matric suction (Broms 1964, Steensen-Bach et al. 1987, Fredlund and Rahardjo 1993, Schnaid et al. 1995, Miller and Muraleetharan 1998, Oloo et al. 1997, Costa et al. 2003 and Yongfu 2004). However, there are limited studies reported in the literature with respect to the bearing capacity of unsaturated soils.

In this research program, the bearing capacity of saturated and unsaturated compacted coarse-grained soil was measured using the University of Ottawa Bearing Capacity Equipment

(UOBCE). This equipment was specially designed and built for this research program at the University of Ottawa student workshop. The bearing capacity was measured in a controlled laboratory environment using square shaped model footings of two different sizes (i.e., 100 mm × 100 mm and 150 mm × 150 mm). Different series of tests were performed to determine the bearing capacity under both saturated and unsaturated conditions. In the first series tests, the water table was kept at the surface of the soil in the test tank to simulate fully saturated condition. The second series were performed under several different unsaturated conditions.

A framework for interpreting the bearing capacity of unsaturated soils is provided in this research work based on the experimental results undertaken on a compacted sand and experimental data from the literature (i.e., four soils). The experimental results show that there is a relationship between the soil-water retention curve (SWRC) and the bearing capacity similar to the relationship between the SWRC and the shear strength behaviour of unsaturated soils. For this reason, a semi-empirical equation is proposed to predict the bearing capacity of unsaturated soils using the saturated shear strength parameters and the SWRC. Using the proposed semi-empirical equation, comparisons are provided between the measured and predicted values of the bearing capacity of unsaturated soils.

1.2 Objectives of the Thesis

The purpose of this research program is to study the influence of matric suction on the bearing capacity of compacted coarse-grained soils. The main objectives of this research are summarized as follows:

(i) To design a special bearing capacity equipment for the determination of the bearing capacity using model footings in the laboratory.

(ii) To evaluate the bearing capacity of unsaturated compacted coarse-grained soils by undertaking a comprehensive experimental investigation.

(iii) To investigate the relationship between the SWRC and the bearing capacity of unsaturated soils.

(iv) To propose a framework for the interpretation of the bearing capacity of unsaturated compacted coarse-grained soils based on the undertaken experimental program.

(v) To propose a semi-empirical equation for predicting the bearing capacity of unsaturated soils

(vi) To provide comparisons between the measured and predicted bearing capacity of four different soils from the literature in addition to the tested soil in this research using the proposed semi-empirical equation.

1.3 Scope of the Thesis

A comprehensive experimental program was undertaken using a sandy soil to determine the bearing capacity of compacted sand both in saturated and unsaturated conditions. The shear strength parameters, namely; the effective cohesion, c' and the angle of internal friction ϕ' of the tested soil were determined using the direct shear test apparatus. Other soil properties such as the compaction curve, density index were also determined in the laboratory. The shear strength parameters, other soil properties information and footing dimensions were used for computing the bearing of saturated soils using conventional bearing capacity theory. The bearing capacity of the compacted sand in the UOBCE was determined under both saturated and unsaturated conditions. While determining the bearing capacity of the compacted sand under unsaturated conditions, the suction values under the footing were measured using commercial Tensiometers located at different depths in the test tank. In addition, the SWRC for the tested soil was measured in the laboratory using Tempe cell apparatus and also measured directly from the test tank. Based on the experimental results of the tested soil and data from different soils in the literature, a semi-empirical equation was proposed for the prediction of the bearing capacity of unsaturated soils.

1.4 Outline of the Thesis

The research undertaken through this research project is summarized in this thesis in six main chapters. These chapters are organized as follows:

The second chapter, "Literature Review", provides a brief review on topics related to bearing capacity of saturated and unsaturated soils. The limitation of the presently used bearing capacity equations in interpreting the bearing capacity of soils is detailed.

The third chapter, "Equipment Details and Methodology", presents the design details of the University of Ottawa Bearing Capacity Equipment (UOBCE). The methodology followed in collecting all the necessary information related to this research study is summarized. This chapter is summarized by extending a paper published in the 59th Canadian Geotechnical Conference in Vancouver 2006. A copy of this paper is enclosed in Appendix B.

The fourth chapter, "Testing Program and Results", provides details about the testing program undertaken for the present research program.

The fifth chapter, "Interpretation of the Bearing Capacity of Unsaturated Soils", provides details with respect to the proposed semi-empirical equation that can be used in the prediction of the bearing capacity of unsaturated soils. This equation is used for interpreting the bearing capacity of the tested compacted sand and four other soils data from the literature. Based on this study, a relationship is proposed between a bearing capacity fitting parameter, ψ and plasticity index, I_P. Using this relationship and the shear strength parameters and the soil-water retention curve (SWRC), the variation of bearing capacity with respect to suction can be predicted. In this chapter, some of details of the review of literature chapter are reproduced and reported such that all the details are summarized independent of this chapter. A brief version of this chapter is published in a previewed paper in the 2nd International Conference, Mechanics of Unsaturated Soils in Germany, Weimar, 2007. A copy of this paper is enclosed in Appendix B.

The sixth chapter presents the summary and conclusions of this research program and also provides some recommendations for future work and development.

Appendix A consists of several pictures taken during the assembling of the University of Ottawa Bearing Capacity Equipment, (UOBCE). In addition, the conventional bearing capacity derivation from fundamentals using the limit equilibrium method is also summarized.

Appendix B consists of two published papers based on this research program.

CHAPTER 2

REVIEW OF LITERATURE

2.1 General

The research work that was undertaken with respect to the bearing capacity of soils both in saturated and unsaturated conditions is reviewed and synthesized in this chapter. The conventional frameworks provided by various investigators such as Terzaghi 1943, Meyerhof 1951 and Vesic 1973 for the interpretation of the bearing capacity of soils are summarized. In addition, the state-of-the-art information with respect to the bearing capacity of unsaturated soils is presented. Finally, the need for development of a framework for interpreting the bearing of unsaturated soils is highlighted.

2.2 Bearing Capacity of Soils – A Brief Background

The bearing capacity theory has been a subject of interest for several researchers during the last century. Prandtl (1921) was one of the pioneering investigators who studied the bearing capacity of soils. Several techniques and empirical procedures followed Prandtl's study to provide a comprehensive understanding of the bearing capacity of shallow foundations.

Many theoretical and experimental studies were undertaken to study the bearing capacity of soils (Terzaghi 1943, Meyerhoff 1951, Vesić 1963 and 1973, Hansen 1969, Chen 1975; Bolton and Lau 1993, Soubra 1999, Silvestri 2003 and Cerato 2005). However, the influence of suction on the bearing capacity was not considered in the above studies. The focus of all these studies was to provide a framework for interpreting or computing the bearing capacity of saturated soils. Several studies have shown that conventional procedures for determination the bearing capacity of footings in shallow foundations usually provide conservative estimations (De Beer & Ladanyi 1961; Vesić 1973 and Silvestri 2003). This is particularly true for soils that are typically in a state of unsaturated condition.

Summary of some major contributions related to the bearing capacity of soils from the literature is presented in the Table 2.1.

Table 2. 1 Major contributions to the bearing capacity of soils including theoretical solutions, laboratory investigations and field studies

Author	Topic of the Research
Prandtl (1921)	Failure mechanism of strip footing
Terzaghi (1943)	Bearing capacity of shallow foundations
Meyerhof (1951)	The ultimate bearing capacity of foundations
Balla (1962)	Bearing capacity of foundations
Meyerhof (1965)	Shallow foundations
DeBeer (1965)	Bearing capacity and settlement of shallow foundations on sand
Hansen (1969)	Discussion of theoretical bearing capacity of very shallow foundations
Graham and Stuart (1971)	Scale and boundary effects in foundation analysis
Vesié (1973)	Analysis of ultimate loads of shallow foundations
Ingra and Baecher (1983)	Uncertainty in bearing capacity of sands
Bolton and Lau (1989)	Scale effects on the bearing capacity of granular soils
Ismael N. F.(1996)	Loading tests on circular and ring plates in very dense cemented sands
Bolton and Lau (1993)	Vertical bearing capacity factors for circular and

	strip footings on Mohr-Coulomb soil
Kumbhojkar (1993)	Numerical evaluation of Terzaghi's N_γ
Bowles (1996)	Foundation analysis and design
Frydman and Burd (1997)	Numerical studies of bearing capacity factor N_γ
Zhu et al. (2001)	A numerical study of bearing capacity factor N_γ
Silvistri (2003)	A limit equilibrium solution for bearing capacity of strip foundations on sand
Cerato (2005)	Scale effects of shallow foundation bearing capacity on granular material

There are also some studies related to the bearing capacity of unsaturated soils. This information is summarized in later sections of this chapter.

2.3 Key Studies Related to the Bearing Capacity of Soils

In this section, an attempt is made to summarize the key studies undertaken by various investigators towards understanding the bearing capacity of soils.

2.3.1 Prandtl (1921)

Prandtl (1921) presented experimental study results undertaken on a strip footing with a smooth base located at ground surface. A concentric load was applied and increased gradually until the strip footing failed. The unit pressure at failure conditions was defined as the ultimate bearing capacity of the soil. From this study, it was shown that an active pressure zone will be developed immediately below the footing and a passive zone generates where the soil pushes laterally and upwards. The analysis presented in Prandtl's research was based on the assumption that the soil is homogenous and weightless. This study laid a foundation for our present understanding and interpretation of the bearing capacity of soils.

2.3.2 Terzaghi (1943)

Terzaghi (1943) proposed an approach for interpreting the bearing capacity of shallow foundations extending the framework originally proposed by Prandtl (1921).

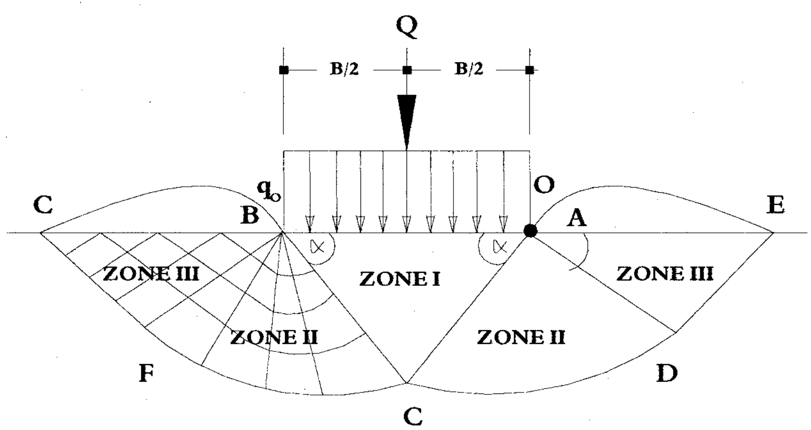


Figure 2.1 Bearing capacity failure in soil under a rough rigid continuous foundation (modified after Terzaghi 1943)

Terzaghi (1943) used two components of cohesion to explain the bearing capacity of a soil. The two components are the true cohesion and the apparent cohesion. The true cohesion in a soil exists from the natural attraction of the soil particles due to molecular forces. The apparent cohesion is due to the action of moisture films in between the soil particles. In the case of sandy type of soil, the apparent cohesion may be attributed to the suction forces arising between the soil particles.

Terzaghi (1943) suggested the failure surface in soil at ultimate load may be assumed to be as shown in Figure 2.1 for a continuous or strip foundation (i.e., one whose width-to-length ratio approaches zero). The soil above the footing was considered as an equivalent surcharge. The failure zone in this figure can be separated into five zones, one marked I and two pairs of zones marked as II and III as follows:

i. the triangular zone (I) under the footing base is considered as a part of the footing and penetrates the soil like a wedge because of friction and adhesion between the footing base and the soil.

ii. zones (II) are located between zone I and zone III and they are known as the radial shear zones because the lines that constitute one set in the shear pattern in these zones radiate from the outer edge of the base of the footing.

iii. zones III are identical with that for Rankine passive state and shear pattern develop in these zones.

Using limit equilibrium method, Terzaghi expressed the ultimate bearing capacity for strip footing using equation [2.1]. The bearing capacity of the soil is dependent on the dimensions of the footing, unit weight and saturated shear strength parameters, c' and ϕ'.

$$q_u = c' N_c + qN_q + 0.5B\gamma N_\gamma \qquad [2.\ 1]$$

where:

q_u = ultimate bearing capacity, *kPa*

c' = effective cohesion, *kPa*

ϕ' = internal friction angle for saturated conditions, *(deg.)*

B = width of the footing, *m*

γ = unit weight of the soil, kN/m^3

The parameters N_c, N_q and N_γ are the bearing capacity factors which are dependent on the angle of internal friction, ϕ' and s_c and s_γ are the shape factors

$$q_u = c' N_c s_c + qN_q + 0.5B\gamma N_\gamma s_\gamma \qquad [2.2]$$

where:

N_c = bearing capacity factor due to cohesion

N_q = bearing capacity factor due surcharge

N_γ = bearing capacity due to unit weight

s_c = shape factor due to cohesion

s_γ = shape factor due to unit weight

The bearing capacity factors proposed by Terzaghi can be determined using equations [2.3] through [2.5].

$$N_q = \frac{a}{\cos^2(45 + \phi'/2)} \qquad [2.3]$$

$$N_c = (N_q - 1)\cot\phi' \qquad [2.4]$$

$$N_\gamma = \frac{\tan\phi'}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'} - 1\right) \qquad [2.5]$$

where:

a = value depends on the angle of internal friction

$K_p = \tan^2(45 + \phi/2)$

Table 2.2 below summarizes the shape factors as defined by Terzaghi (1943).

Table 2. 2 Terzaghi (1943) shape factors for various shallow foundations

Shape factor	Strip footing	Round footing	Square footing
s_c	1.0	1.3	1.3
s_γ	1.0	0.6	0.8

The bearing capacity factors, N_c , N_q and N_γ are plotted as a function of the angle of internal friction angle, ϕ' in Figure 2.2. Terzaghi's equation was found to be applicable for computing the bearing capacity of shallow foundations where the depth, D of footing is less than or equals to the width, B. Terzaghi did not implicitly consider the presence of matric suction; however, the influence of water table level on the bearing capacity was taken into account by reducing the value of the unit weight of the soil.

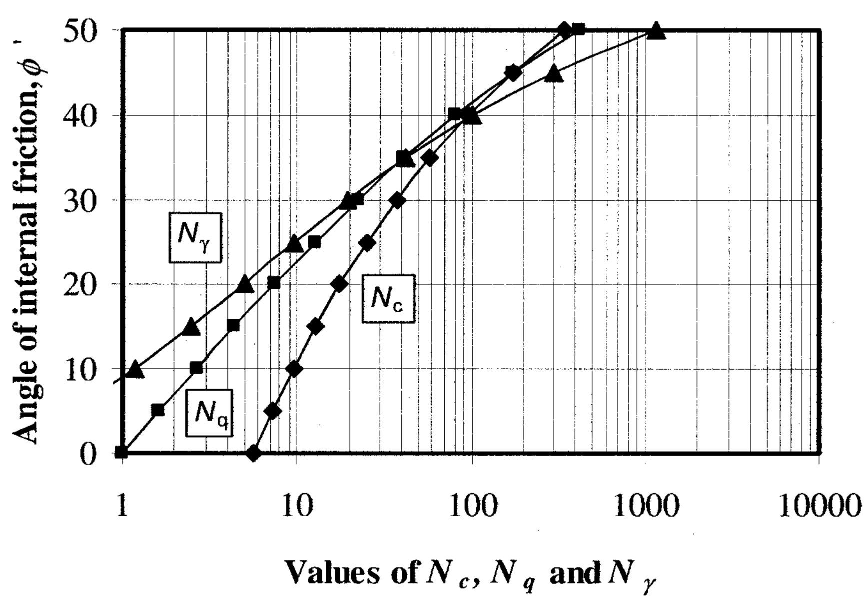


Figure 2. 2 Bearing capacity factors (modified after Terzaghi and Peck 1948)

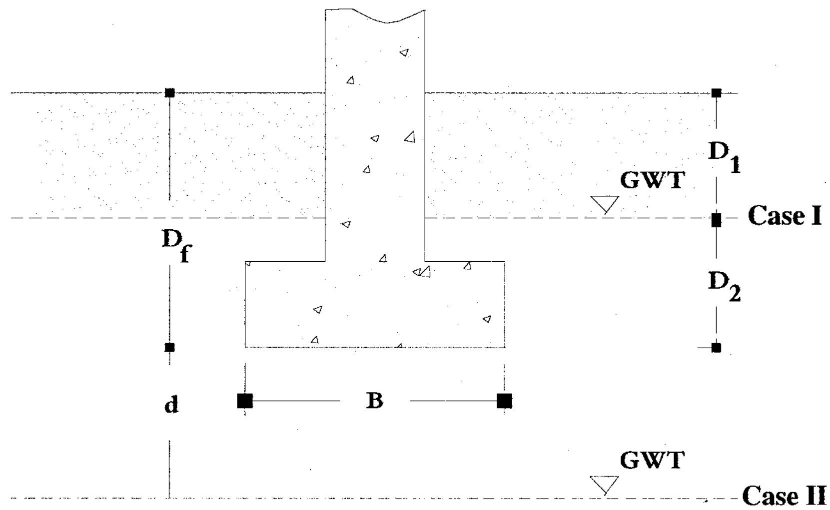


Figure 2. 3 Modification of bearing capacity equations for water table (Modified after Das 2004)

Figure 2.3 shows three different scenarios with respect to the groundwater table level conditions. The bearing capacity of the soil will be significantly influenced due to different groundwater table conditions. The following equations show how the unit weight of the soil should be altered and used in the bearing capacity equation [2.1] to account for the influence of the groundwater table.

In case I, the foundation can be assumed to be under submerged conditions. The surcharge, q term in equation [2.1] should be replaced using equation [2.6]

Case I: $$If \quad 0 \le D_1 \le D_f \longrightarrow \gamma' = \gamma_{sat} - \gamma_w \longrightarrow q = D_1\gamma + D_2\gamma' \qquad [2.6]$$

The submergence condition will cause loss of the apparent cohesion in soil due to the loss of capillary stresses or weak cementation bonds. In such a case, the effective unit weight of the same soil will be reduced to about one-half the unit weight of the soils above water (Bowles 1996 and Ausilio and Conte 2005).

For case II, equation [2.7] should be used in equation [2.1]

Case II: $$If \ \ 0 \leq d \leq B \longrightarrow \bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma') \longrightarrow q = \gamma D_f \qquad [2.7]$$

For case III, equation [2.1] should be used, as there will no effect on the bearing capacity of the soil due to the influence of the ground water table.

Case III : $$If \ \ d \geq B \longrightarrow no\,water\,effect\ on\,the\ bearing\ capacity$$

where:

D_1 = depth from the soil surface to water table level above the footing, *m*

D_2 = depth from the water table level to the bottom of the footing, *m*

D_f = depth from the soil surface to the bottom of the footing, *m*

d = depth of water table below the footing surface, *m*

B = footing width, *m*

Georgiadis et al. (2003) found that bearing capacity of soils upon wetting causes collapse in some soils and contributes to building up positive pore water pressures. Due to this reason, the bearing capacity significantly reduces. The influence of groundwater table conditions on the bearing capacity of soils was also studied by Ausilio and Conte (2005). The results obtained from this study indicate that submergence condition of the soil below the footing may significantly reduce the bearing capacity.

In the case II and III, the soil surrounding the footing can be in a state of unsaturated condition. Nevertheless, the influence of negative pressure on the bearing capacity of soils was not considered in the original derivations of Terzaghi's equation. Due to this reason, using Terzaghi equation for computing the bearing capacity of soils in a state of unsaturated condition may lead to conservative designs.

2.3.3 Meyerhof (1951)

Meyerhof proposed a general bearing capacity equation [2.8], which is similar in form to the Terzaghi's equation.

$$q_u = c' N_c' s_c' d_c' + q N_q' s_q' d_q' + 0.5 B \gamma N_\gamma' s_\gamma' d_\gamma' \qquad [2.\ 8]$$

where:

N_c', N_q', N_γ' = bearing capacity factors due to cohesion, surcharge and unit weight respectively

s_c', s_q', s_γ' = shape factors due to cohesion, surcharge and unit weight respectively

d_c', d_q', d_γ' = depth factors due to cohesion, surcharge and unit weight respectively

The bearing capacity factors and shape factors can be determined using the equations below:

$$N_q' = e^{\pi \tan \phi} \tan^2(45 + \phi'/2) \qquad [2.\ 9]$$

$$N_c' = (N_q - 1)\cot \phi' \qquad [2.\ 10]$$

$$N_\gamma' = (N_q - 1)\tan(1.4\phi') \qquad [2.\ 11]$$

$$s_c' = 1.0 + 0.2 K_p (\frac{B}{L}) \ any \ \phi' \qquad [2.\ 12]$$

$$s_q' = s_\gamma' = 1.0 + 0.1 K_p (\frac{B}{L}) \quad \phi' > 10^o \qquad [2.\ 13]$$

$$s_q' = s_\gamma' = 1.0 \quad \phi' = 10^o \qquad [2.\ 14]$$

where:

K_p = passive pressure coefficient defined as $\tan^2(45 + \phi'/2)$

Meyerhof (1951) studied the effect of the footing base (i.e., smooth and rough) on the bearing capacity concluding that the roughness has little effect in the case of vertical loading condition but increases with roughness in the case of inclined loading condition. The difference between the computation of the bearing capacity values using Terzaghi's and Meyerhof's equations is quite small for smaller *D/B* ratios. Generally, Terzaghi's equation provides relatively low bearing capacity values in comparison to Meyerhof's equation for larger *D/B* ratios.

Similar to Terzaghi, Meyerhof (1951) did not consider the influence of negative pore water pressures on the bearing capacity of soils. For this reason, the bearing capacity of unsaturated soils, if computed using Meyerhof's equation will be conservative.

2.3.4 Vesić (1973)

Vesic (1973) has undertaken studies using footings loaded in homogeneous soil with a vertical load and suggested the use of the original Terzaghi's equation for computing the bearing capacity of soils. However, new bearing capacity factors and shape factors were recommended for reliable computation of the bearing capacity. The bearing capacity equation with the new factors as suggested by Vesić (1973) is given below:

$$q_u = c' N_c \xi_c + q N_{qv} \xi_q + 0.5 B \gamma N_{\gamma v} \, \xi_\gamma \qquad [2.\ 15]$$

The bearing capacity factors, N_{cv} and N_{qv} are not very different from Terzaghi's bearing capacity factors for N_c and N_q. However, the values of $N_{\gamma v}$ are different due to differences in the assumption with respect to the failure angle α (as shown in Figure 2.1). Both Terzaghi and Meyerhof bearing capacity factor due to unit weight, N_γ values are lower than the values proposed by Vesić (1973). The question of which set of N_γ values are more appropriate has remained unsettled, because of difficulties in selecting a representative value of angle of shearing resistance, ϕ' for the bearing capacity factor N_γ (Vesić 1973).

The bearing capacity factors and shape factors proposed by Vesić (1973) are summarized below:

$$N_{qv} = e^{\pi \tan\phi} \tan^2(45 + \phi'/2) \quad [2.16]$$

$$N_{cv} = (N_q - 1)\cot\phi' \quad [2.17]$$

$$N_{\gamma v} = 2(N_q + 1)\tan\phi' \quad [2.18]$$

$$\xi_c = 1.0 + \frac{N_q}{N_c}\left(\frac{B}{L}\right) \quad [2.19]$$

$$\xi_q = 1.0 + \frac{B}{L}\tan\phi' \quad \geq 0.6 \quad [2.20]$$

$$\xi_\gamma = 1.0 - 0.4\frac{B}{L} \quad \geq 0.6 \quad [2.21]$$

Vesić (1973) studies also show the position of the ground water table can influence the bearing capacity of a footing significantly. It was recommended to base the bearing capacity analysis assuming the highest possible ground water table level during the lifetime of the foundation. If the highest ground water table level is within the depth, $z_w \leq B$ below the foundation base, then the effective unit weight, γ of the soil below the foundation base was suggested to be calculated using the equation below:

$$\gamma = \gamma' + \left[\frac{z_w}{B}\right](\gamma_m - \gamma') \quad [2.22]$$

If $z_w = B$, the effective unit weight should be taken as γ_m (moist unit weight) and for a water table at or above the foundation level, the submerged unit weight should be used in the equation [2.15].

2.4 Failure Mechanisms in Shallow Foundations

The ultimate bearing capacity of a shallow foundation can be defined as the peak point of the load-settlement curve determined from load tests. The three principle modes of shear failure under foundations have been described as general shear failure, local shear failure and punching shear failure, (Terzaghi 1943, De Beer and Ladanyi 1961 and Vesic 1963). These three mechanisms of failure are discussed below.

2.4.1 General Shear Failure

If the footing rests on a very dense soil, the failure will most likely be a general shear failure. When the load per unit area equals the ultimate bearing capacity a sudden failure surface will be observed in the soil which was supporting the footing. To develop a well-defined general shear failure, there must be a significant tilting of the footing and surface upheaval and cracks in the soil surface (Oloo 1994). The general shear failure is known by the well-defined failure pattern. This failure consists of a continuous slip surfaces starting from one edge of the footing to up to the ground surfaces as shown in Figure 2.4. Vesić (1973) stated, "*It can be said generally that the failure mode depends on the relative compressibility of the soil in the particular geometrical and loading conditions. If the soil is particularly incompressible and has a finite shearing strength, it will fail in general shear*".

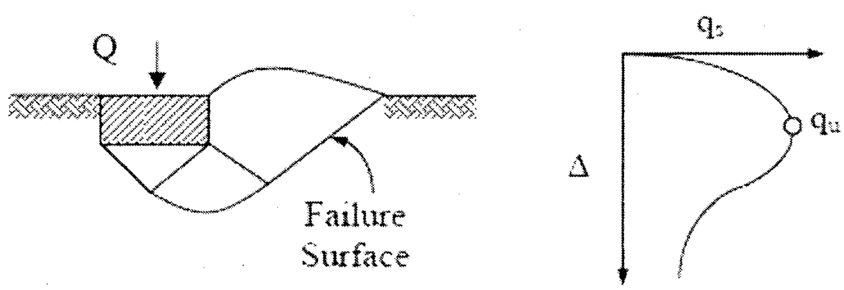


Figure 2. 4 Modes of bearing capacity failure: general shear failure (Vesić 1973 and Das 2004)

2.4.2 Punching Shear Failure

The footing may show a punching shear failure in some cases. For example, if the footing was placed in dense sand at greater depth, the failure mode can be punching shear failure as shown in Figure 2.5.

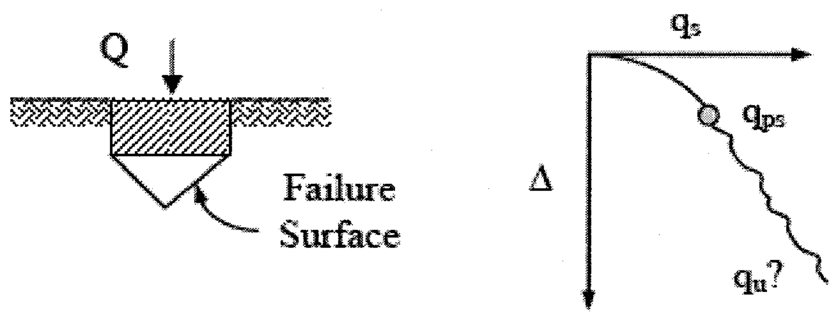


Figure 2. 5 Modes of bearing capacity failure-punching shear failure (Vesic 1973 and Das 2004)

2.4.3 Local Shear Failure

The third type of failure is local shear failure, which can be expected if the footing was on medium compacted sand. The local shear failure is also characterized by a well-defined failure pattern beneath the foundation. As shown in Figure 2.6, it has a wedge and slip surfaces, which start from the edges of the footing. The vertical compression under the footing is important as the slip surfaces disappear in the soil before reaching the ground surface. After the footing settles enough the slip surfaces may appear without a catastrophic failure (i.e., failure surface does not extend to ground surface) or tilt of the footing transmitting the load to a stronger layer. These characteristics make the local shear failure represent both general and punching shear failure. If the footing was constructed on a fairly loose soil, the failure surface will not extend to the ground surface representing a punching shear failure.

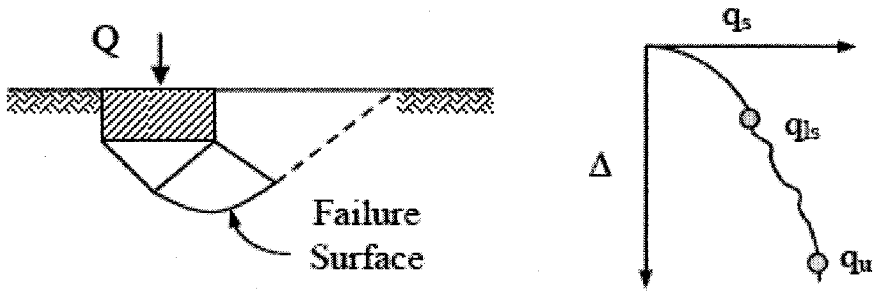


Figure 2. 6 Modes of bearing capacity failure-local shear failure (Vesic 1973 and Das 2004)

Vesić (1973) used several rectangular and circular plates supported by sand at different relative densities of compaction. Based on this study a relationship between the footing depth and the relative density idex, D_r was proposed to define different modes of failure.

If the relative density is more than 70%, the surface shear failure will be general shear failure. The ultimate load may occur at a foundation settlement of 4 to 10% of width, B for shallow foundations at shallow depth; however, in the case of local or punching shear failure, the ultimate load may occur at settlements of 15 to 25% of width, B.

2.4.4 Methods for Ultimate Load Definition

The failure mechanism may be governed by general, local or punching shear failure depending on the type of the soil and the loading condition. The ultimate load could be defined following one of the criteria listed below:

- the peak load value in case of general shear failure
- the maximum constant load with increasing settlement
- the load when the inclination of the load-displacement curve reaches a constant value.
- the stress for which the strain is twice the strain at a 10% smaller stress (Hansan 1961 and Hansan and Christiensen 1969).
- the load where the load-settlement curve breaks on a double log scale (De Beer 1965).

2.5 Pressure Distribution in Shallow Foundations

The stresses from the superstructure are transmitted to the soil below through the sub-structure (i.e. foundation) in the form of a pressure bulb. The depth of the pressure bulb is approximately 2 times the width of the foundation (Poulos and Davis 1974 and Chen 1999). The effective depth is typically assumed to be 1.5 times the footing width, *B* since the effective pressure is only about 20 % of the actual pressure at this depth.

2.5.1 Loaded Depth

Figure 2.7, shows the distribution of vertical stress under a square shaped footing. Based on Boussinesq's equation the pressure bulb will be extended up to a depth 1.5 times the footing width. The pressure transmitted at a depth of 1.5 *B* is only 20% of the applied stress. Table 2.3 presents the pressure distribution within the stress bulb for a square footing.

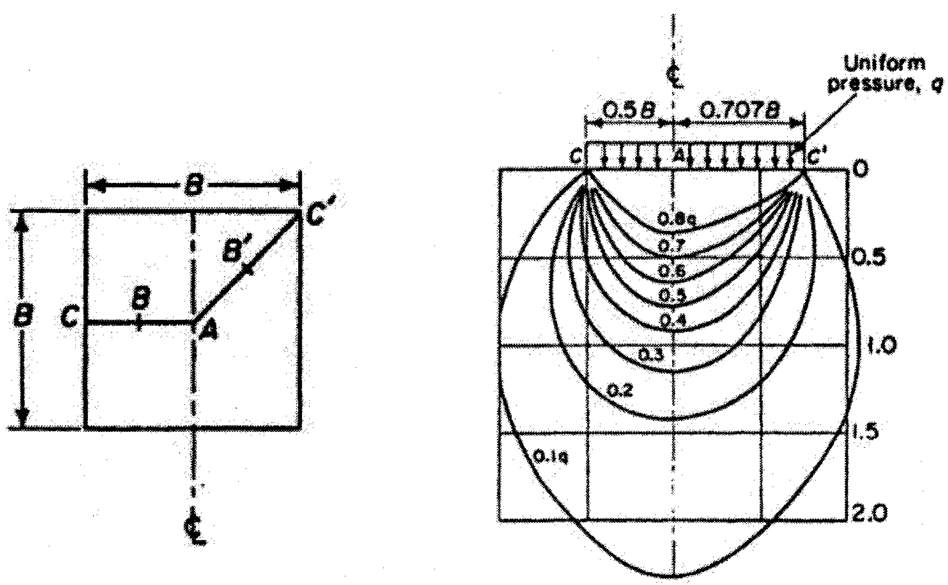


Figure 2. 7 Vertical stresses under a square footing (Chen 1999)

Table 2. 3 Stress distributions within the pressure bulb (Chen 1999)

Depth, (*D*) below footing with (*B*)	Percentage of uniform pressure for square footing
0.5*B*	70%
1.0*B*	35%
1.5*B*	18%
2.0*B*	12%

2.6 Analytical and Empirical Methods for the Estimation of the Bearing Capacity

Several analytical and empirical methods can be used for determining the bearing capacity of soils. Figure 2.9 summarizes some of the key analytical and empirical methods that are commonly used. The bearing capacity of the soils can be determined using analytical techniques (typically mathematical formulations) using some simple properties of the soil. The empirical methods (i.e., SPT and CPT) on the other hand, attempt to relate a form of measurement, usually an in-situ test to the ultimate bearing capacity of soils.

2.6.1 Analytical Methods

Deschenes (1978) stated that the mathematical investigations concerning the state of equilibrium beneath continuous footings under loading are not fully understood. There is general bearing capacity theory that rigorously satisfies the Mohr-Coulomb equation taking account the weight of soil, the influence of the depth of surcharge, *D* and the real distribution of vertical and horizontal forces on the base of the footing. There are three different types of analyses that are commonly used are the limit equilibrium, plastic equilibrium and limit analysis.

Terzaghi (1943) assumed that soil to be weightless in order to determine the mathematical formulation of N_c and N_q. All N parameters (i.e., bearing capacity factors) are derived as functions of the angle of internal friction, ϕ'. However, it is well known that the soil behaviour in the plastic region is nonlinear and thus the concept of superposition is not valid in the bearing capacity analyses. The reason for using the simplified (superposition) method is largely to avoid the difficulties associated with proposing simplified mathematical equations using conventional equilibrium methods.

Several investigators proposed mathematical formulations (using limit equilibrium, limit analysis or numerical methods) that provided approximately same values of bearing capacity based on the angle of internal friction, ϕ' particularly for N_c and N_q values. (Terzaghi 1943, Meyerhof 1951, Vesic 1973, Kumbhojkar 1993, Bolton and Lau 1993, Frydman and Burd 1997, Ausilio and Conte 2005, Cerato 2005). A number of bearing capacity studies have suggested different expressions for N_γ. Researchers agree that a valid theoretical solution for N_γ in case of shallow foundation problems is not available (Bowles 1996). Several equations available in the literature for estimating N_γ are summarized in Table 2.4.

Table 2. 4 Equations for N_γ calculations

Terazaghi (1943)	$N_\gamma = \frac{\tan\phi'}{2}\left(\frac{K_{p\gamma}}{\cos^2\phi'} - 1\right)$
Meyerhof (1951)	$N_\gamma = (N_q - 1)\tan 1.4\phi'$
Hansen (1961)	$N_\gamma = 1.8\,(N_q - 1)\tan\phi'$
Vèsic (1973)	$N_\gamma = 2\,(N_q - 1)\tan\phi'$
Kumbhojkar (1993)	$N_\gamma = \frac{P_{\gamma\min}}{\gamma B^2} - \frac{\tan\phi'}{2}$

where:

$K_{p\gamma} = \tan^2(45 + \phi/2)$

B = footing width, m

γ = unit weight of the soil, kN/m^3

$P_{\gamma \min}$ = minimum passive pressure, kPa

The bearing capacity factor, N_γ is not a unique value and depends on the unit weight, γ, and the friction angle, ϕ' (Cerato 2005). Figure 2.8 below presents a compilation of model footing tests show the relationship between N_γ and ϕ'. The equation was suggested by Cerato (2005) as given below:

$$q_{ult} = 0.5 \gamma B N_\gamma s_\gamma \qquad [2.23]$$

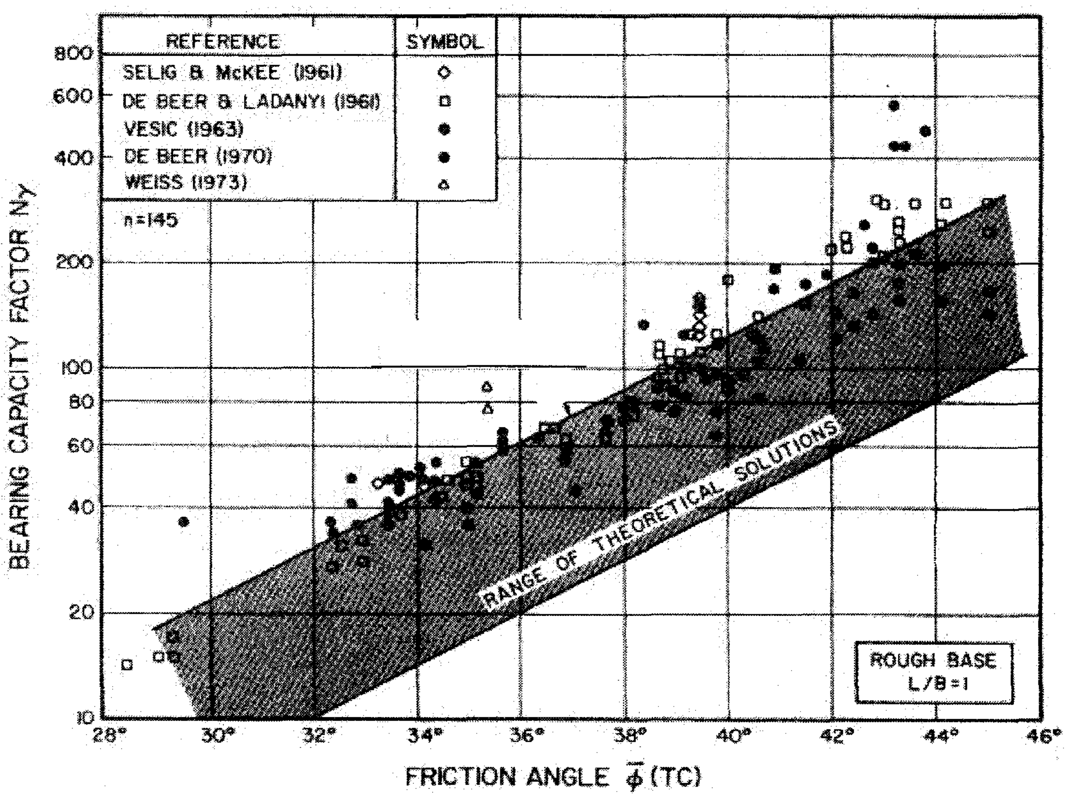


Figure 2. 8 The relationship between the bearing capacity factor, N_γ and internal friction angle, ϕ' from model footing tests (Ingra and Baecher 1983)

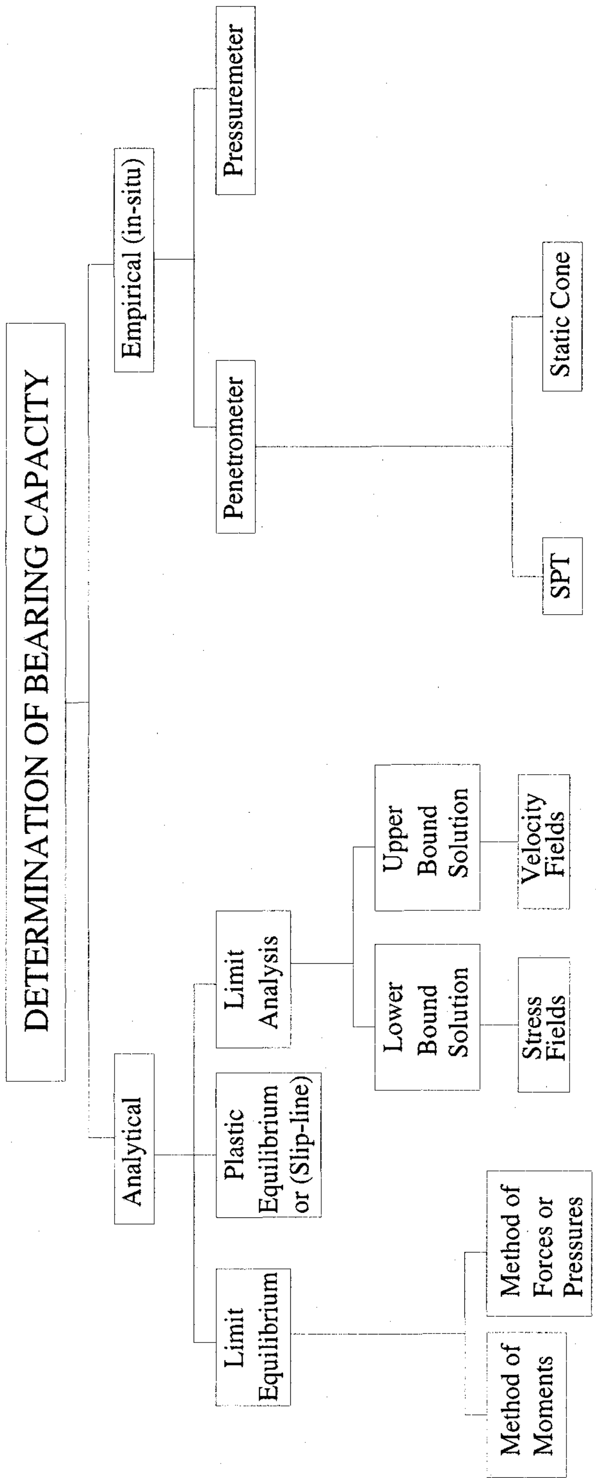


Figure 2. 9 Principle methods for determination of bearing capacity (modified after Deschenes 1978)

The relationship between the bearing capacity factor, N_γ and internal friction angle, ϕ' shown in Figure 2.8 suggests that the N_γ values of the model footings are higher than the range of theoretical solutions. This study clearly demonstrates that scale effect has to be taken into consideration. Cerato (2005) stated, "*For model or prototype scale loading tests, the only unknown in equation* [2.23] *is the bearing capacity factor, N_γ which is only dependent on the friction angle ϕ' of the soil according to all current textbooks. The question that arises is which ϕ' should be used? Most values of friction angle for granular soils are measured on small diameter samples; however, this may introduce an inherent scale effect involved, heretofore undetected*". In other words, it is important to determine an appropriate value of ϕ'such that bearing capacity values can be estimated more reliably.

2.6.1.1 Limit Equilibrium

The limit equilibrium method is an approximation method in which the failure surface is typically assumed to be circular or log spiral. Terzaghi (1943) and Meyerhof (1951) used the limit equilibrium method assuming different shapes for failure surfaces. The shear resistance along the failure surface can be neglected since Terzaghi's equation was intended for shallow foundations where $D \leq B$ extending limit equilibrium method. Meyerhof (1951) considered the failure zone up to the surface with an arc to include the shear along line in his limit equilibrium analysis to derive expressions for the bearing capacity factors (Bowles 1996).

Chen (1975) stated, "*Limit equilibrium methods utilize the basic philosophy of the upper-bound rule, a failure surface is assumed and the least answer is sought. However, it gives no consideration to soil kinematics and the equilibrium conditions are satisfied only in a limited sense. Therefore, limit equilibrium solutions are not necessarily an upper bound or a lower bound. However, any upper-bound solution from limit analysis will obviously be a limit equilibrium solution. Nevertheless, the method has been the most widely used owing to its simplicity and reasonably good accuracy*".

2.6.1.2 Plastic Equilibrium Method (Slip Line Method)

The soil is assumed to be rigid-plastic in this equilibrium method of analysis. A group of slip-line fields are constructed in the method. The plastic equilibrium methods are conventionally used for geotechnical engineering applications and are best described as approximate approaches (Chen 1975). It is possible to locate the most critical surface from which the bearing capacity can be calculated by trail and error.

2.6.1.3 Limit Analysis Method

The limit analysis methods were originally developed for solid mechanics problems. However, they were successfully applied for geotechnical engineering problems. The limit analysis method is simple compared to the limit equilibrium method (Chen 1975, Bolton and Lau 1993 and Soubra 1999). The limit analysis method is efficient and can be extended to solve more difficult problems for which other methods were not successful (Deschenes 1978). The limit analysis methods allow bounds to be placed on the load at which general plastic collapse occurs, even when the problem is quite complex and the determination of the true collapse load is virtually impossible. In addition, it was frequently found that the bounds are close enough to define the true collapse load with sufficient accuracy for engineering purposes. Ausilo and Conte (2005) carried out a theoretical study to analyze the influence of the ground water on the bearing capacity of shallow foundations using the limit analysis.

2.7 Model Footing Tests

Several studies were undertaken in the literature to understand the bearing capacity of soils using model footings (Vesic 1963, Steensen-Bach et al. 1987, Bowles 1996, Oloo et al. 1999 and Cerato 2005). Generally, the bearing capacity of soils can be determined in a laboratory environment under controlled conditions using footing sizes in the range of B = 25 mm to 75 mm × L = 25 mm × 200 mm. There are some advantages and disadvantages associated with using model footings (Bowles 1996).

2.7.1 Advantages of Model Footings

The bearing capacity of soils can be determined relatively easily using model footings in comparison to full-scale tests. These tests are inexpensive and not time consuming. The bearing capacity of different soils under different conditions can be determined in the laboratory under controlled conditions using model footings.

2.7.2 Disadvantages of Model Footings

There are some disadvantages associated with using model footings in the determination of the bearing capacity of soils. The scale effect has an impact on the results as model footings. Typically model footings are under much lower confinement in comparison to full-scale footings tests (Graham and Stuart 1971 and Cerato 2005).

Cerato (2005) studies on granular soils show that the bearing capacity factor, N_γ was dependent on the width of the footing, B. As the footing width increases the bearing capacity factor, N_γ decreases. Cerato 2005 results are consistent with the results obtained by Graham and Stuart (1971) shown Figure 2.10.

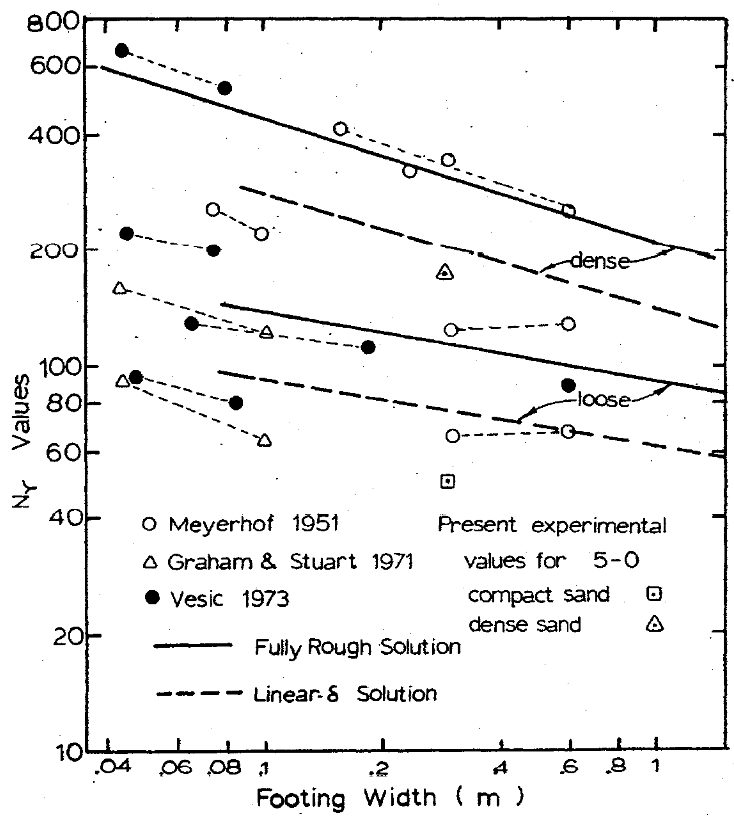


Figure 2. 10 Influence of footing width on the bearing capacity factor N_γ (Graham and Stuart 1971)

2.8 Studies on Bearing Capacity of Unsaturated Soils

The measured bearing capacity from field tests is typically higher than the estimated or computed bearing capacity values using analytical and empirical methods (Terzaghi 1943, Meyerhof 1951 and 1956, Vesic 1973). This behavior may be attributed to neglecting the contribution suction towards the bearing capacity of soils (Steensen-Bach et al. 1987, Fredlund and Rahardjo 1993). There is considerable interest during the last twenty years towards understanding the engineering behavior of unsaturated soils. However, there is limited number of studies undertaken to study the bearing capacity of unsaturated soils. Table 2.5 summarizes some key studies reported in the literature on this topic.

Table 2. 5 Experimental and field studies related to the bearing capacity of unsaturated soils

Author	Topic of the Research
Broms (1964)	The effect of degree of saturation on the bearing capacity of flexible pavements
Siva Reddy and Mogaliah (1970)	Bearing capacity of partly saturated soils
Ismael (1985)	Allowable pressure from loading tests on Kuwaiti soils
Steensen-Bach et al. (1987)	Capillary induced stresses-Fact or fiction?
Fredlund and Rahardjo (1993)	Bearing capacity of unsaturated soils
Oloo (1994)	A bearing capacity approach to the design of low-volume traffic roads
Schnaid et al. (1995)	Load-settlement of shallow foundations in structured unsaturated soils
Oloo et al. (1997)	Bearing capacity of unpaved roads
Miller and Muraleetharan (1998)	In situ testing in unsaturated soils
Costa et al. (2003)	Influence of matric suction on the results of Plate Load tests performed on a Lateritic soil deposit
Youngfu Xu (2004)	Bearing capacity of unsaturated expansive soils

2.8.1 Broms (1964)

Broms (1964) developed a method for the evaluation of the bearing capacity of flexible pavement in unsaturated soils. This method is based on the assumption that the failure takes place in fully saturated subgrade soil. The bearing capacity was found to be a function of the degree of saturation, the apparent cohesion, c and internal friction angle, ϕ. A decrease in the degree of saturation from 100 to 90% increased the bearing capacity of two fine-grained soils (i.e., silt and clay) by approximately 2 and 8% respectively. Broms (1964) studies suggest that the bearing capacity of the subgrade is reduced due to an increase in the degree of saturation. Figure 2.11 shows the relationship between the tire pressure and the ultimate wheel load with respect to degree of saturation. The degree of saturation was found to have a significant effect on the bearing capacity of the pavement.

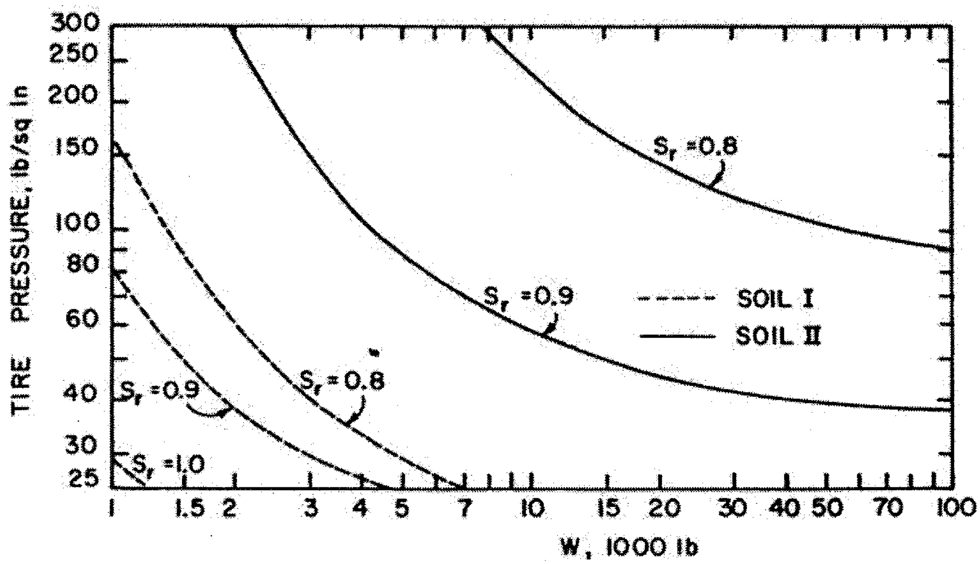


Figure 2. 11 The tire pressure versus wheel load of flexible pavement (Broms 1964)

2.8.2 Steensen-Bach et al. (1987)

The bearing capacity of two different sands of a surface footing under saturated and unsaturated conditions was studied. The capillary pressure below the footings was determined by in-situ measurements using Tensiometers and by laboratory determination of the capillary curves. It was found that the measured bearing capacity of unsaturated sand is typically 4 to 6 times higher than the saturated bearing capacity of the same soils. Steensen-Bach (1987) concluded based on experimental studies on model tests on two sands that the presence of capillary pressures is certainly fact, not fiction, and its effect should be carefully considered.

2.8.3 Fredlund and Rahardjo (1993)

Fredlund and Rahardjo (1993) proposed an extension to conventional bearing capacity equations to account for the increase in bearing capacity due to matric suction. The contribution of matric suction is considered as an additional cohesion component extending total stress approach. A theoretical study was undertaken to evaluate the effect of soil matric suction on the ultimate bearing capacity using a square footing embedded in clay type of soil. Based on this study, it was reported an increase in the bearing capacity by 27% when the suction increased by an amount equal to the undrained shears strength of the fine-grained soil. Figure 2.12 shows the relationship between the bearing capacity of a strip footing for various matric suction values, obtained by Fredlund and Rahardjo (1993).

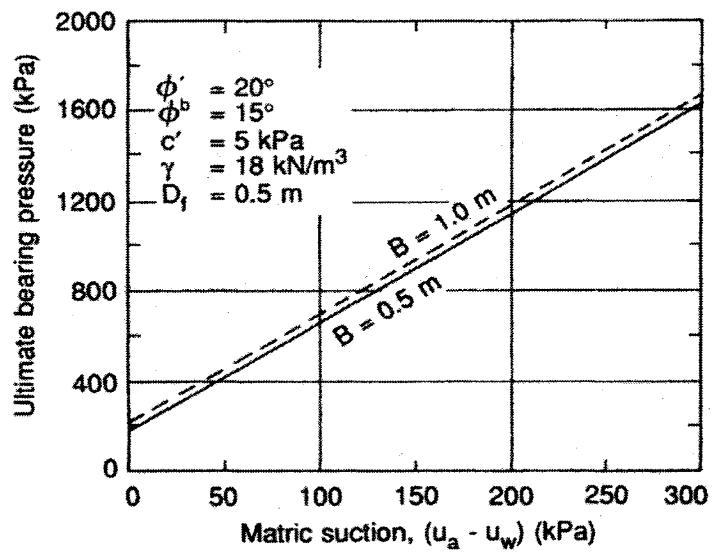


Figure 2. 12 Bearing capacity versus matric suction for a strip footing (Fredlund and Rahardjo 1993)

2.8.3.1 Effects of Matric Suction on Bearing Capacity

The unsaturated soil can be visualized as having a cohesion consisting of two components. One component is the effective cohesion and the other component is due to matric suction contribution. The higher the matric suction in unsaturated soil, the higher will be the bearing capacity due to the fact that the relationship between the matric suction and the bearing capacity is proportional, (Fredlund and Rahardjo 1993). With this concept in mind, the conventional bearing capacity theory can be extended to unsaturated soils by adding the component of suction to effective cohesion. The increase of the cohesion is due to matric suction is illustrated in Figure 2.13 for various ϕ^b values.

$$c_{unsat} = c' + (u_a - u_w)\tan\phi^b \qquad [2.24]$$

where:

c = total cohesion, *kPa*

c' = effective cohesion, *kPa*

$(u_a - u_w)$ = matric suction, *kPa*

ϕ^b = internal friction angle with respect to suction, *(deg.)*

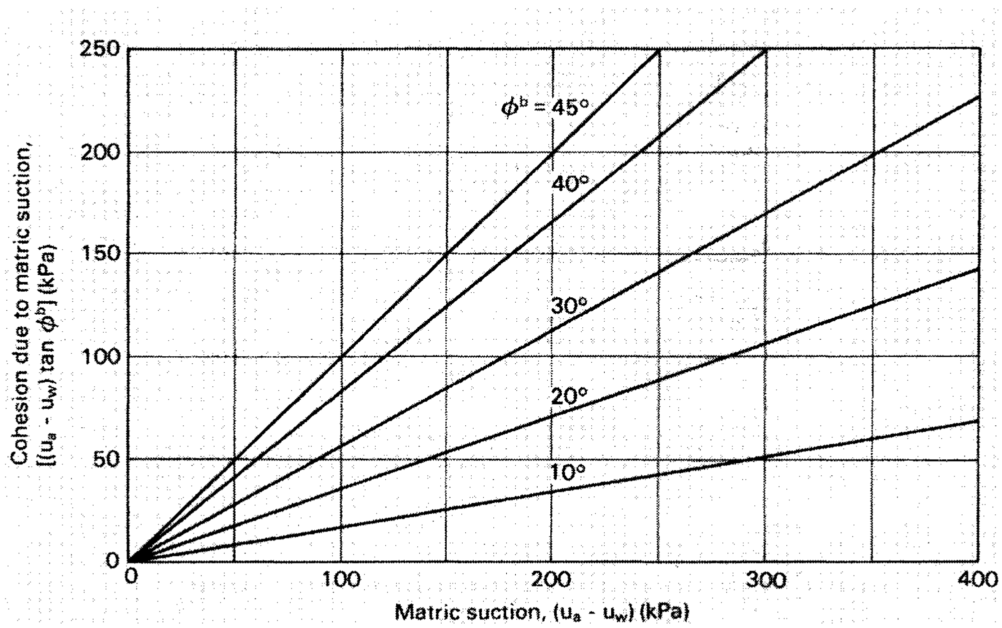


Figure 2. 13 The component of cohesion due to matric suction for angles (Fredlund and Rahardjo 1993)

In Figure 2.14 the matric suction values in this case are obtainable as a percentage of the hydrostatic negative pore-air pressures above the water table level for a ϕ^b. Conventional fully saturated analyses accounting for suctions by using pore pressure Figure 2.14 - profile 1 give higher ultimate loads (low bearing capacity) than do unsaturated analysis.

2.8.4 Oloo (1994)

Oloo (1994) proposed a bearing capacity based method for the design of unpaved roads consisting of a base layer overlying a subgrade. The method of design highlights the significant role-played by matric suction on the bearing capacity of pavement structures. In addition, experimental studies of model footings on silt and till specimens show that the bearing capacity increased non-linearly with an increase in the matric suction. A relationship was proposed to predict the bearing capacity assuming a constant value of ϕ^b (internal friction angle with respect to matric suction). More details about this relationship are summarized in chapter 5. Comparisons between the measured and predicted bearing capacity values using Oloo (1994) equation and a new proposed equation are also provided in this thesis.

2.8.5 Costa et al. (2003)

Costa et al. (2003) carried out a study to investigate the effects of matric suction on the results of plate load tests performed on Lateritic soil deposit, which was in a state of unsaturated condition. The soil suctions were measured by Tensiometers located at different depths as shown in Figure 2.15.

In-situ results suggest that the contribution of matric suction was substantial and the increase of matric suction lead to a nonlinear decrease of settlement.

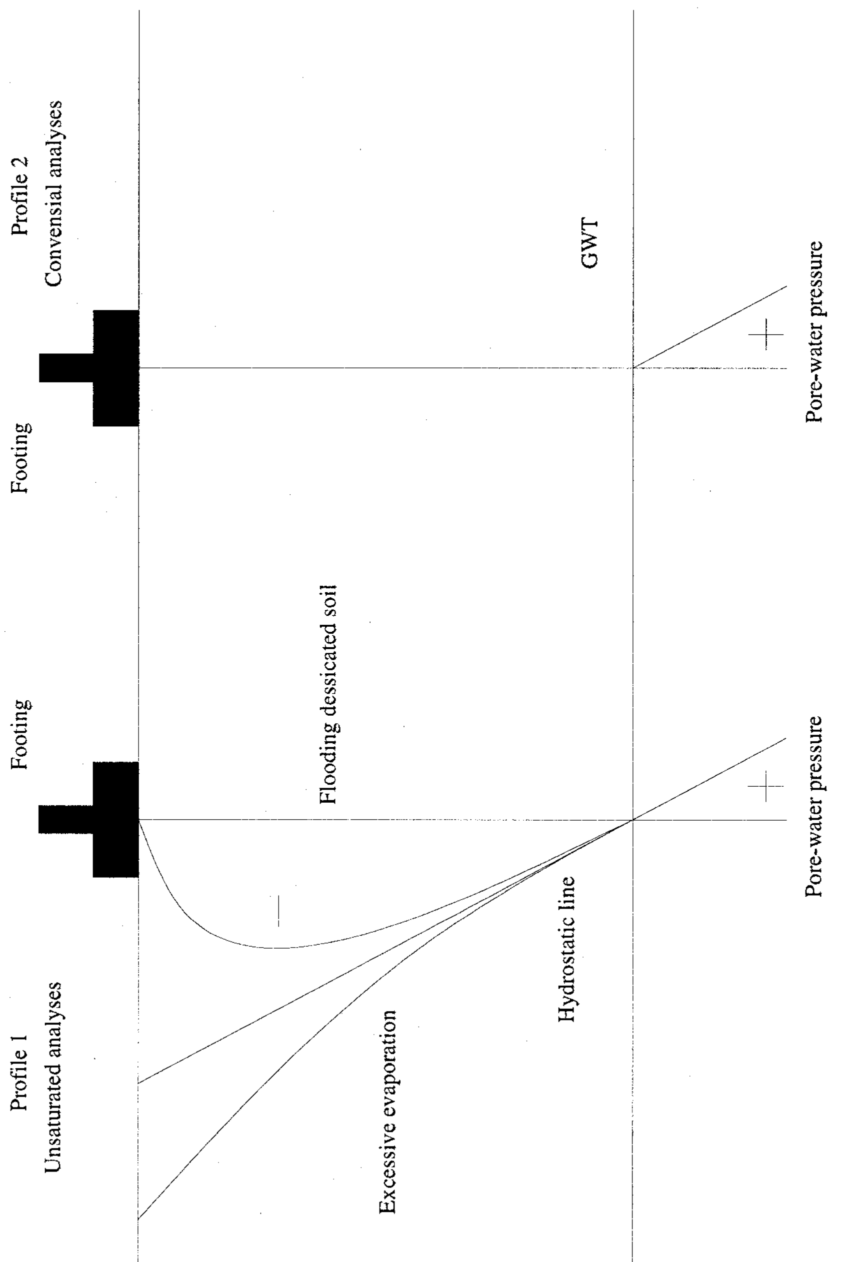


Figure 2. 14 Typical schematic pore-water profiles below a spread footing (Modified after Fredlund and Rahardjo, 1993)

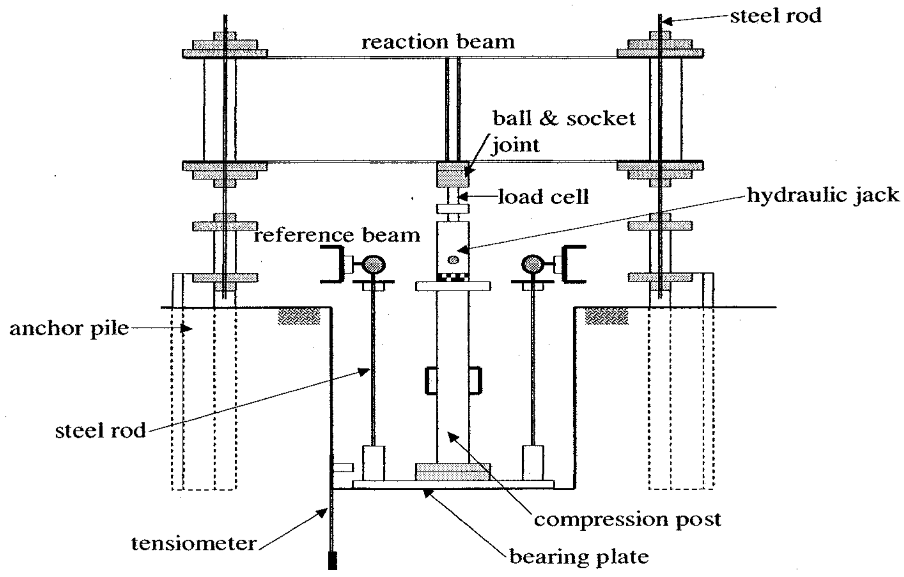


Figure 2. 15 General schematic of the testing assembly and used apparatus (Costa et al. 2003)

2.9 Summary

The influence of matric suction is not typically considered while computing the bearing capacity of soils. Some recent studies have shown that the bearing capacity of unsaturated soils is typically 5 to 7 times higher than the bearing capacity of the same soil under saturated conditions. However, there are limited investigations that are reported in the literature which attempt to provide a theoretical framework for interpreting the bearing capacity of unsaturated soils. Geotechnical engineers are hesitant to take into account of the influence of suction as simple prediction techniques are not available. The literature reviewed in this chapter clearly demonstrates that there is a need for developing a simple framework for interpreting the bearing capacity of unsaturated soils. Such studies will be useful in the design of geotechnical structures such as foundations and pavement structures.

CHAPTER 3

EQUIPMENT DETAILS AND METHODOLOGY

3.1 Introduction

Equipment required for measuring the bearing capacity of both saturated and unsaturated soils using surface model footings (with dimensions of 100 mm × 100 mm and 150 mm × 150 mm) was specially designed and built at the University of Ottawa student workshop. More fabrication and design details of this equipment are available in Appendix A. This chapter primarily describes the key features of the bearing capacity equipment including the methodology of using this equipment. In addition, other equipment used in the testing program, which include the Tempe cell, Tensiometer and data acquisition equipment are also briefly described. This chapter is summarized in a paper published in the 59th Canadian Geotechnical Conference in Vancouver 2006. A copy of this paper is enclosed in Appendix B.

3.2 Bearing Capacity Equipment

Figure 3.1 shows the details of the University of Ottawa Bearing Capacity Equipment (UOBCE). All the key features of this equipment are summarized in this section:

i. The loading frame for this equipment was constructed using an aluminum C-channel (150 mm web × 50 mm flange with 8 mm in thickness). The frame is 2450 mm in height and 1450 mm wide. Four C-channels (with the same section as the loading frame) were used to support the test tank. Two more channels were fastened on top of the frame of the loading machine such that they can offer resistance to the reaction loads. One additional C-channel was placed underneath the loading machine to facilitate the placement of the loading arm and the load cell.

ii. An electrically operated and mechanically controlled loading system was used to load the model footings. A maximum load of 14 kN can be applied using this system.

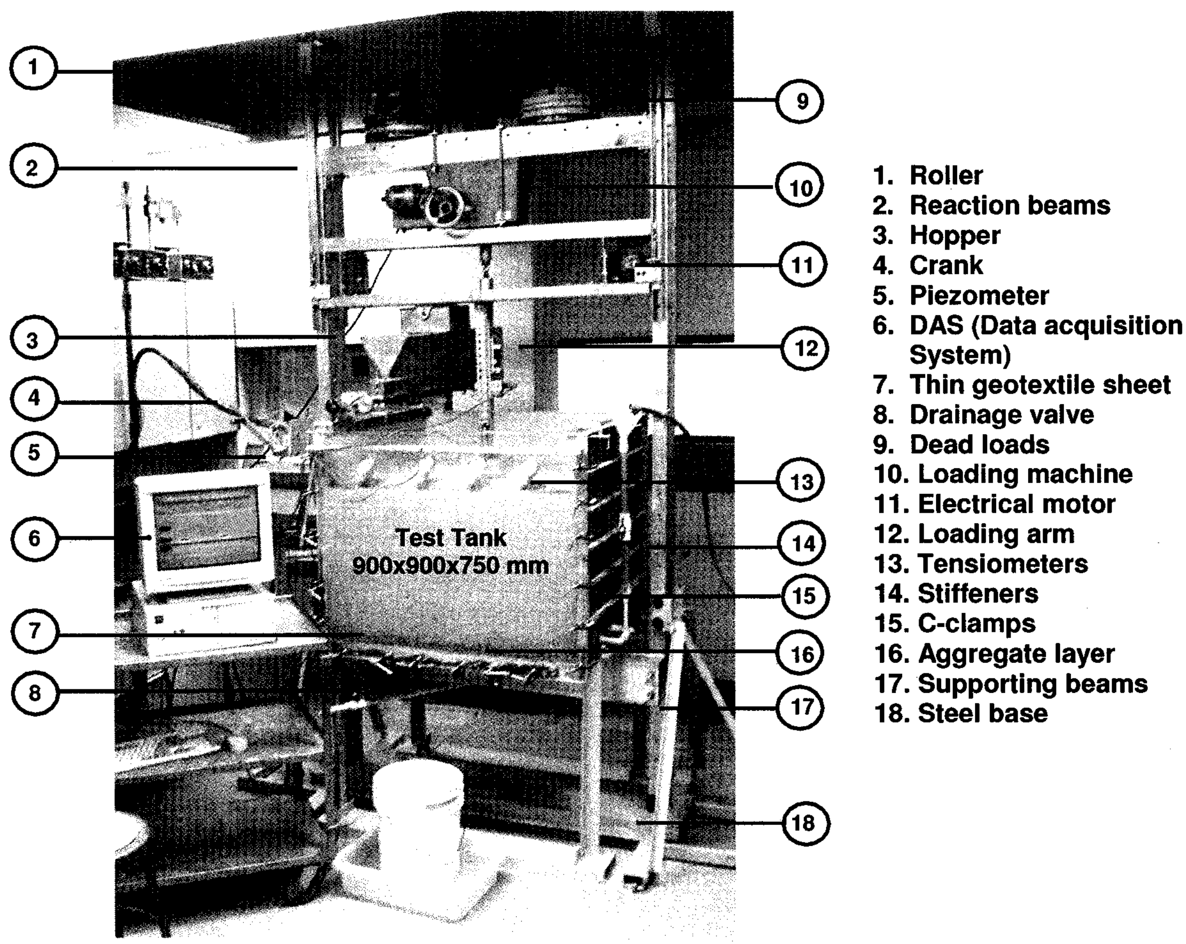


Figure 3. 1 The University of Ottawa Bearing Capacity Equipment (UOBCE)

iii. The test tank dimensions are 900 mm × 900 mm (in plan) and 750 mm (in depth) with provisions for collecting the required data, which include load, deformation, water level in the tank, and matric suction below the surface footing.

iv. The test tank was constructed using 6 mm thick aluminum sheets. Several stiffeners were added along the horizontal direction to prevent any lateral bending or bulging that may occur during the loading of the footing. The clear distance between the model footing edge and the sides of the tank was equal to four times the width of the footings used in the study to avoid the influence of boundary effects in the stress bulb zone, (Poulos and Davis 1974 and Chen 1999). The depth of the tank was deeper than the expected depth of the stress bulb (i.e., 1.5 B to 2 B) below the model footing. Considerably larger depth was used such that the water table can be raised or lowered to vary the matric suction values using drainage valves without any interference from the instrumentation used in the study.

v. Aluminum metal model square footings with the dimensions of 100 mm × 100 mm or 150 mm × 150 mm dimensions were used in this research program (Figure 3.3). The thickness of the footing was equal to 50 mm. The bottom surface of the model footing was corrugated to introduce roughness in the footing. The footings were placed on the surface of the soil in the test tank and subjected to vertical static loading using an adjustable-loading machine.

vi. The soil was placed in the tank through the use of a V- shaped hopper. The hopper movement can be controlled with the aid of a rotating drum (Figure 3.1), which was operated using an electrical motor. The movement of the hopper was monitored in the vertical direction using a side crank and cables on four rollers at the top of the frame.

vii. The maximum density index, D_r value that can be achieved in the tank with respect to the height of free fall of the soil was determined using the V-hopper. A free height of fall of 1 m was found to provide maximum density index to the sand studied in this research program in the test tank. For this reason, the height of free fall of soil was fixed at 1 m from the V-hopper. The soil was placed in the tank in 50 mm lifts. This hopper can also be moved horizontally from left to right and in reverse directions using a side motor with the aid of a horizontal chain.

viii. The front face of the test tank constituted of a transparent acrylic plate. The acrylic plate acted as a window to allow observation of the water table level and marking the thickness of the soil layers during the installation and to empty the soil quickly from the tank after the completion of each test. The acrylic window, which is 25 mm thick, is fastened with C-clamps to the tank as shown in Figure 3.2 item 15.

ix. Linearly Variable Displacement Transducer (LVDT) was attached to the vertical loading arm and the tip of the LVDT was placed directly on the surface of the model footing. A load cell capable of measuring 14 kN was mounted on the loading arm. Both the LVDT and the load cell were connected to a data acquisition system (DAS) as shown in Figure 3.1.

x. A piezometer was used to monitor the level of the water table in the tank, which is on left side of the test tank (item 5 in Figure 3.1). Commercial Tensiometers were installed after saturating the ceramic tips and located at different depths as shown in Figure 3.2.

xi. The base of the equipment was connected to the loading frame using steel angles to ensure the stability during the loading of the footing. No movement or sway of the loading frame was observed during the loading process. A 50 mm thick layer of clean aggregate was laid on the base area of the test tank and a thin geotextile sheet was placed on top of the aggregate to function as a porous barrier between soil and the aggregate. The objective of this layer is to facilitate free and gradual movement of water in the test tank in order to achieve uniform saturation or desaturation conditions as desired by the testing requirements. Drainage pipes with valves connected to the bottom of the test tank were used in monitoring the water level. A water supply pipe of 20 mm in diameter was used to control the amount of water supplied to the tank. This main water supply pipe branches in to four smaller pipes of 12.5 mm, which facilitate to saturate the soil gradually and uniformly from the base of the tank to the surface (i.e., the saturation was progressed from the bottom to the top of the soil surface). Both saturation and desaturation conditions were achieved successfully in the tank using this system.

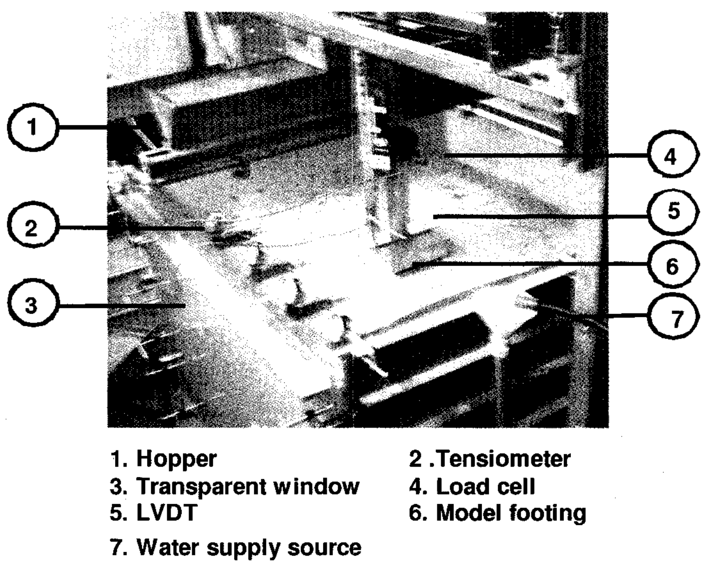


Figure 3. 2 Top view of the UOBCE

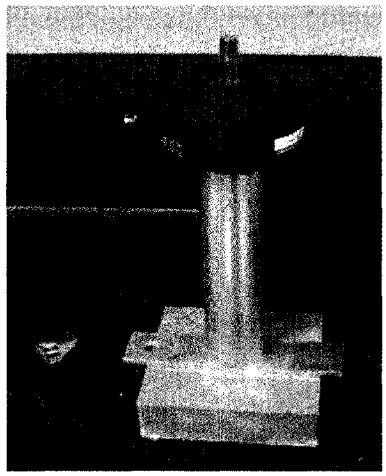


Figure 3. 3 Model square footing and load cell

3.2.1 Soil Delivery System - Calibrations

The soil delivery system as detailed in section 3.1.1 was used to add the soil into the test tank after adjusting the drum speed to 60-rpm. The hopper height was fixed at a height of 1 m as there was no increase in the density index achieved by further increasing the height. The maximum density index value achieved using this technique was 55%. It is likely that the general shear failure conditions may not result when the surface footing is loaded in to the soil in the bearing capacity tank when the density index value is lower than 65%. For this reason, water content of 15% (which was the optimum moisture content value of the sand) was achieved in the top 300 mm of the bearing capacity soil tank by raising the water level from the bottom of the tank using drainage valves. The wet soil with approximately a water content of 12 to 15% was then compacted using a hand compactor of 5 kg. This procedure was useful to increase the density index to a value equal to 64 %. More details of the soil properties and the variation of the density index with depth of the soil in the tank are discussed in Chapter 4.

3.2.2 Calibrations of the Load Cell and the LVDT

A load cell with maximum capacity of 14 kN was used to measure the load that was applied on the model footing in the OUBCE. Figure 3.3 shows the load cell calibration curve using dead loads. The calibration curve for the load cell used in this research program is shown in Figure 3.5. Figure 3.6 shows the calibration of Linearly Variable Displacement Transducer (LVDT). Both the load cell and the LVDT were connected to a data acquisition system for collecting the data.

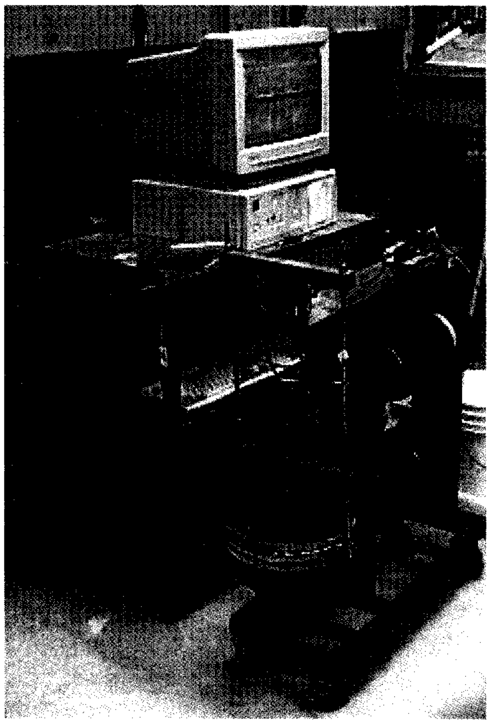


Figure 3. 4 Loading the load cell for calibration

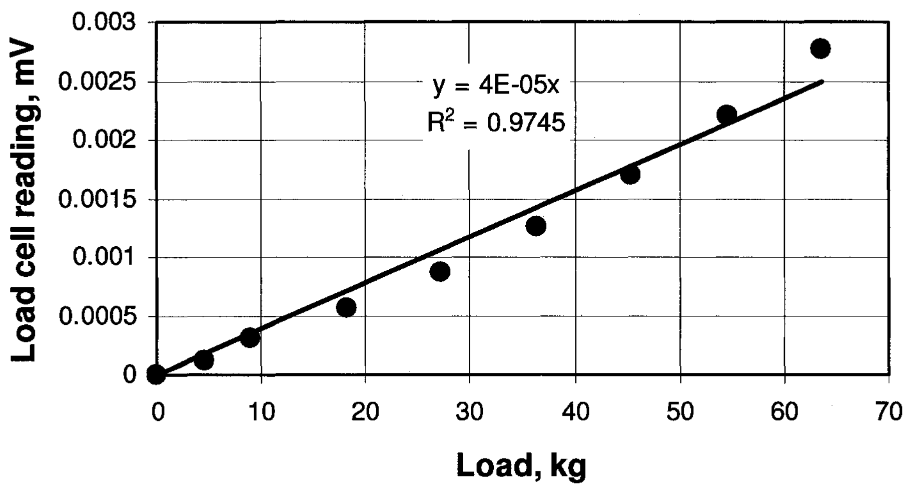


Figure 3. 5 Calibration curve for the load cell

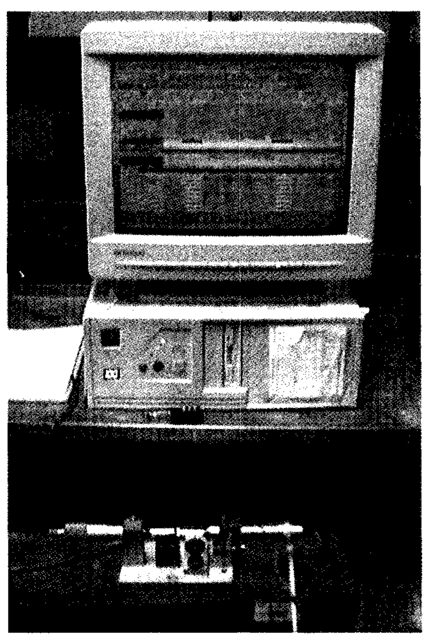


Figure 3. 6 Calibration technique for the LVDT using micrometer

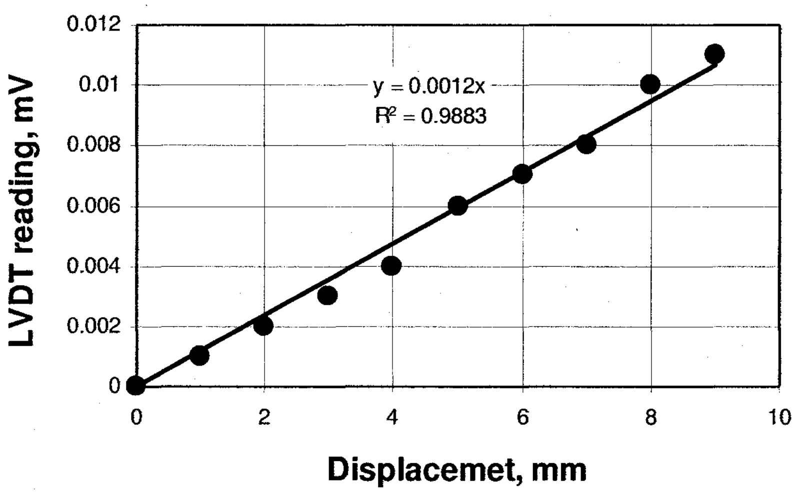


Figure 3. 7 Calibration curve for the LVDT

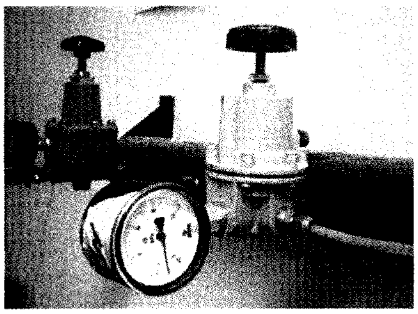
3.3 Tempe Cell Apparatus for Measuring the SWRC

The soil-water retention curve (SWRC) is defined as the relationship between the gravimetric water content, w or the volumetric water content, θ or the degree or saturation, S and the matric suction, $(u_a - u_w)$. The Tempe cell apparatus is commonly used for measuring this relationship using the axis-translation technique (Figure 3.8b). The axis-translation technique facilitates in elevating the air pressure in the Tempe cell and avoids the problems of cavitation when the SWRC relationship has to be measured for suction values greater than 100 kPa. Extending the axis-translation technique, direct measurement of the SWRC can be obtained using the Tempe cell for the matric suction range from 0 to 500 kPa. Tempe cell apparatus is similar to the pressure plate apparatus, which is commonly used for measuring the SWRC for a larger suction range (0 to 1500 kPa). Several specimens SWRC can be measured simultaneously using the pressure plate apparatus. The Tempe cell is smaller in comparison to the pressure plate apparatus and facilitates the measurement of SWRC of individual specimens. The Tempe cell consists of a saturated high air entry disk (HAED) which facilitates in separating air and water phases in a closed vessel. The difference between the applied air pressure, u_a and the pore water pressure, u_w at equilibrium conditions is the matric suction, $(u_a - u_w)$. The pore water pressure connection is typically open to atmosphere; hence the applied air pressure is the matric suction in the soil specimen.

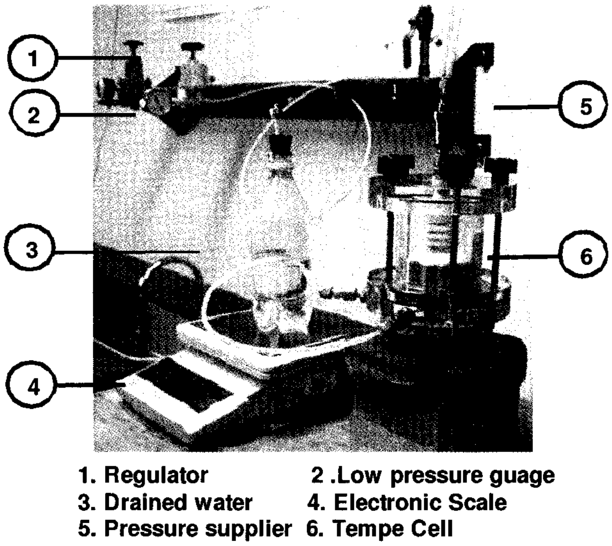
In this research program, the Tempe cell apparatus was used for the measurement of the matric suction in the range of 0 to 20 kPa. Typically, sandy soils such as the one used in the research program attains residual conditions (i.e., low degrees of saturation) by applying a matric suction of 15 to 20 kPa. In other words, the sand fully desaturates and attains low degrees of saturation (typically less than 5%) when the applied suction values greater than 15 kPa.

The setup details for measuring the SWRC in the laboratory using Tempe cell apparatus is shown in Figure 3.8. A pressure gauge with a sensitivity of measuring values of 0.2 kPa is connected to a pressure regulator. The soil sample with the same density as the soil in the bearing capacity tank is placed in the Tempe cell on the saturated ceramic disk. The soil sample is initially saturated by allowing access of water to the soil sample using a bottle of water placed at a level higher than the sample. The suction values are induced by increasing the applied air pressure. After a period of 24 hours, equilibrium water content is determined for each applied pressure increment by determining the mass of the Tempe cell. The mass of water lost due to pore water drainage at each increment is measured. The SWRC relationship is obtained from the water content versus matric suction relationship.

Some amount of air is typically collected below the ceramic disk of the Tempe cell. By using a small flushing system the air bubbles collected below the HAE disk were removed. Figure 3.9 shows the equilibrium conditions of a typical sample in a Tempe cell apparatus.



(a)



(b)

Figure 3. 8 Measuring the SWRC: (a) Sensitive pressure gauge and (b) Tempe cell apparatus

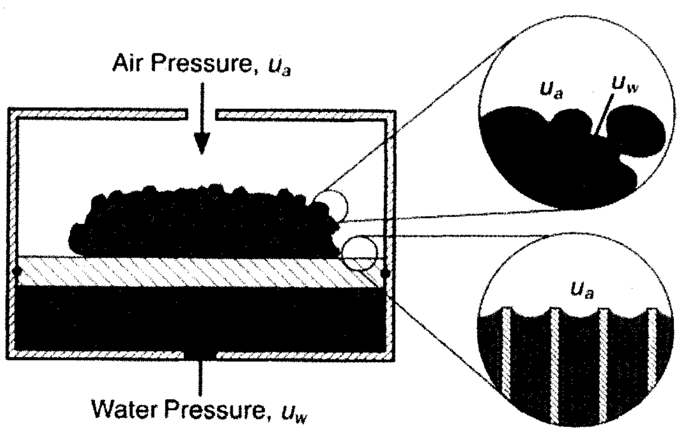


Figure 3. 9 Equilibrium positions for air-water interface in air-water-HAE-soil system (Lu and Likos 2004)

3.4 Tensiometers

Tensiometers are conventionally used for the direct measurement of suction in the range of 0 to 90 kPa. The ceramic tip is typically an inverted cup or small probe that can be filled with water. This tip is used to create a saturated hydraulic connection between the unsaturated soil and the water in the Tensiometer body through the use of a pressure sensor. A setup of typical laboratory commercial Tensiometer is shown in Figure 3.10.

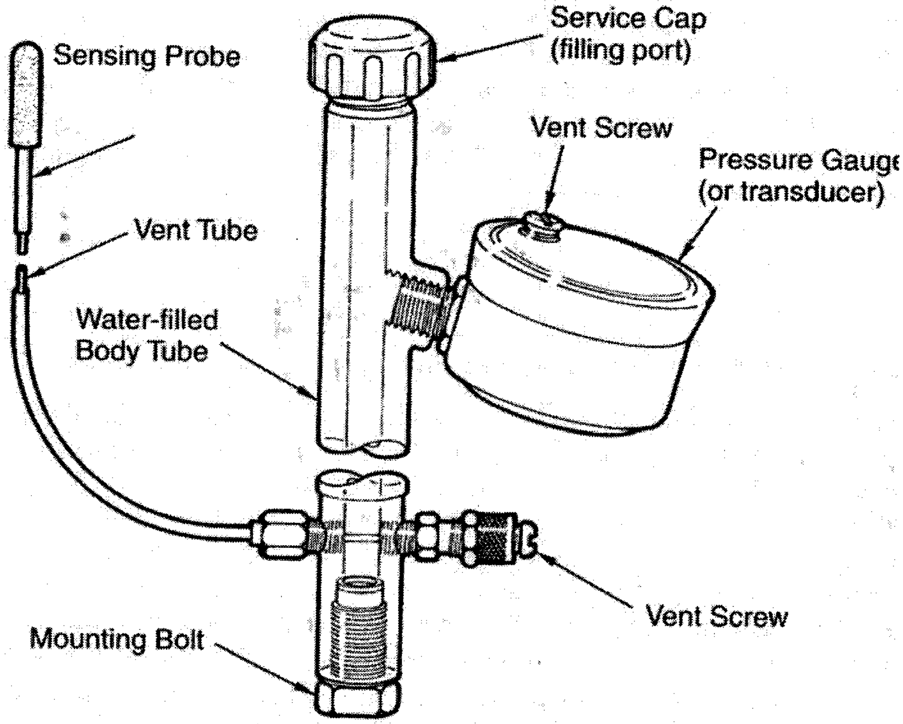


Figure 3. 10 Schematic drawing of small-tip laboratory Tensiometer (Fredlund and Rahardjo 1993)

The technique for measuring the soil suction using the Tensiometer relies on the properties of the high air entry (HAE) materials typically prepared using ceramic materials such as kaolinite. The HAE materials are characterized by microscopic pores of relatively uniform size and size distribution. The surface tension forces maintain the gas-liquid interfaces formed in the material's pores of the saturated HAE material (ceramic disk) used. Physically, surface tension acts as a membrane for separating the two air and water phases.

Figure 3.11 shows a schematic cross-section of a typical saturated ceramic disk.

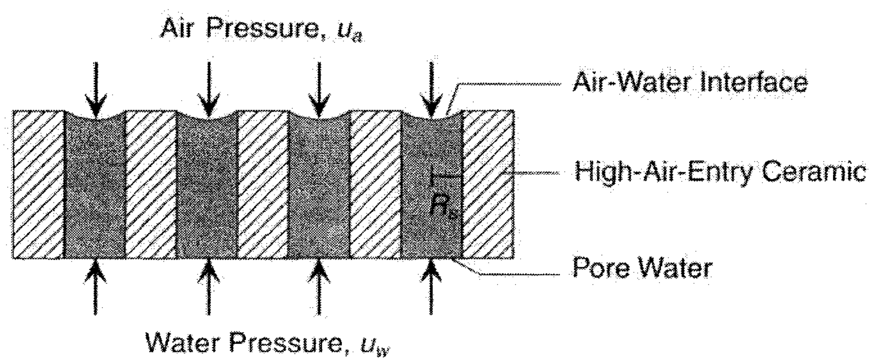


Figure 3. 11 Operating principle of HAE ceramic cup (Lu and Likos 2004)

The maximum sustainable difference between the air pressure above the disk and the water pressure within and below the disk is inversely proportional to the maximum pore size of the material and mathematically expressed using Young-Laplace equation [3.1].

$$(u_a - u_w)_b = \frac{2T_s}{R_S} \qquad [3.\ 1]$$

where; $(u_a - u_w)_b$ = the air entry value, *kPa*

T_S = the surface tension of the air water interface, *kPa*

R_S = the effective radius of the maximum pore size of the HAE material

3.4.1 Measurement of Suction using Tensiometers

Figure 3.12 shows the details with respect the measurement of the suction The negative pore water pressure (i.e., matric suction) is transmitted to through the saturated ceramic disk and facilitates the flow of water (i.e., drying process) to the soil from the Tensiometer and the sensor will read the matric suction at the equilibrium condition when water no longer moves out. If the soil is being saturated (i.e., wetting process) water will flow in the ceramic disk from the soil. It should be noted that there is no effect of osmotic suction on the pressure measurements since the ceramic tip is permeable to dissolved solutes. The matric suction measurements should be corrected for the difference in elevation between the sensor and the pressure gauge. In practice, reliable Tensiometer measurements using standard testing equipment are limited to 70 to 80 kPa approximately. Impurities, dissolved gasses and air bubbles may concentrate in tiny crevices on the walls of the sensor body and that may cause some cavitations to occur and as a result reduce suction values.

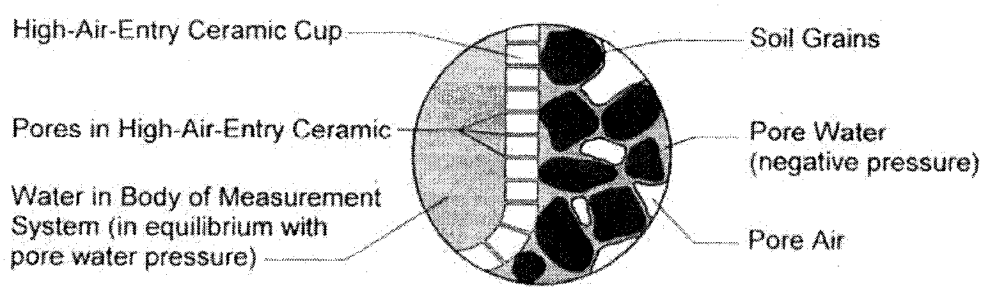


Figure 3. 12 Enlarged schematic showing porous ceramic tip in contact with unsaturated soil grains (Fredlund and Rahardjo 1993)

3.4.2 Calibration of Tensiometer

The following steps should be followed for proper calibration and setting up the Tensiometers:

i. The first step is to immerse the ceramic cup in water for one hour or more to fill the pores with water.
ii. Prior to testing, the Tensiometer system is saturated with de-aired water and temporary vacuum may be applied to remove air bubbles from the system.

iii. After filling the system, a small suction value has to be applied at the filler end to remove air from the bourdon tube in the vacuum dial gauge.
iv. The plastic tube has to be air-free and the tube can be tapped to release the air bubbles then open the service cap.
v. After removing the air the air-free water can be flushed again through the system.
vi. Replace and tighten the service cap and the water vent screw and by then the Tensiometer is ready for laboratory or field installation.
vii. Long-term evaporation from the ceramic cup should be avoided, since it results in evaporation deportation deposits on the surface of the cup, which reduces the sensitivity.

The above details are summarized using the information from the Soilmoisture corporation manual.

3.4.3 Tensiometer Installation

In order to install the Tensiometer in the soil, a good contact is required between the ceramic cup and the soil to maintain continues link between the pore water and the measurement system. A thin wall 6 mm tube can be used to core a hole in the soil to accept the ceramic cup. The plastic tube should be handled carefully and not to be bent. The ceramic cup must be mounted in the region where soil suction values are required (Fredlund and Rahardjo 1993).

After installation is completed, the unit will come to equilibrium with the soil and the soil suction will be read directly on the dial gauge in kPa as shown in Figure 3.13.

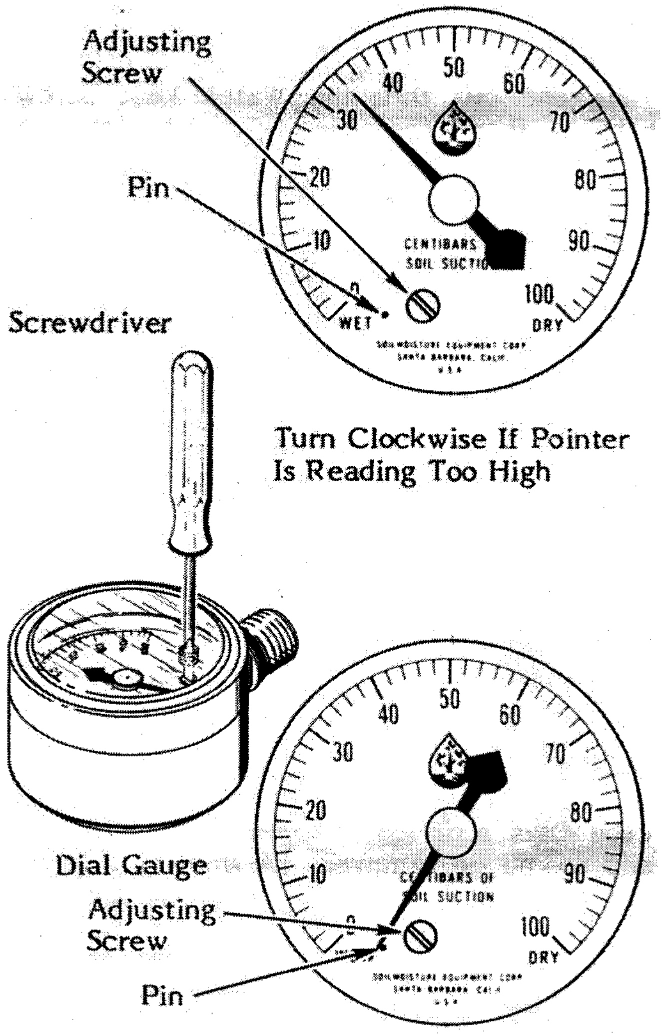


Figure 3. 13 Schematic shows the adjustment process of the gauge (from Soil-Moisture-Manual - 2006)

CHAPTER 4

TESTING PROGRAM AND RESULTS

4.1 General

The testing program undertaken through this research program is presented in three sections in this chapter. In the first section, Experimental Program-I, all the conventional soil property tests results are presented and discussed. In the second section, Experimental Program-II, test results carried out to determine the bearing capacity of the soil under saturated and unsaturated conditions using surface model footings in the bearing capacity equipment are presented. In the third section, Experimental Program- III, test results related to the soil-water retention curves are presented. In addition, all the bearing capacity tests results are presented and the relationship between the soil-water characteristic curve (SWRC) and the variation of the bearing capacity of unsaturated soils are shown providing brief discussions.

4.2 Experimental Program - I

The soil sample used for testing in this research program was commercial sand obtained from Merkley Supply Ltd. in Ottawa. All the soil required for the study was obtained in a single batch. Several conventional tests are undertaken in the laboratory to determine the soil properties. Figure 4.1 shows the various tests conducted in the Experimental Program-I in the form of flowchart.

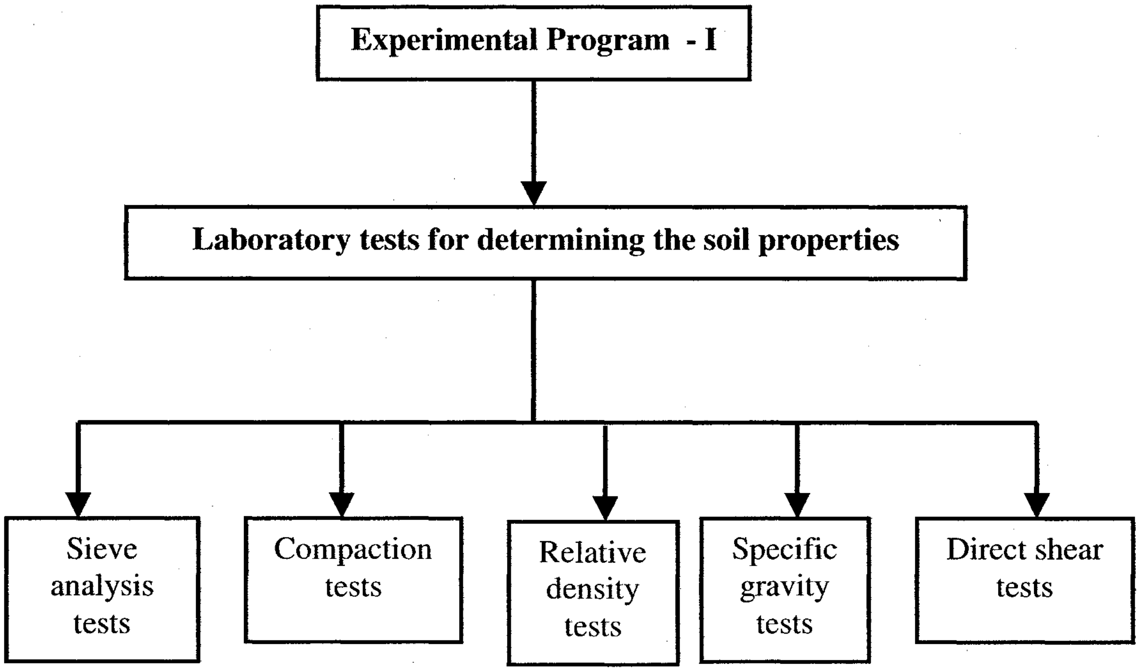


Figure 4. 1 Experimental Program-I

4.2.1 Dry Sieve Analysis Test

A representative sample equal to 0.750 kg was collected from the whole batch of sand proposed for use in this research program to determine the grain size distribution. This soil sample was air-dried for a period of one day. The air-dried soil sample was divided into three equal parts and sieve analysis tests were performed on each of the three soil samples using ASTM, Standards D422 (1994b).

Figure 4.2 shows the average grain size distribution of the soil of the three samples. The soil was classified as poorly graded sand (SP) as per USCS. The fines percentage in the soil was 5 % of silt. Table 4.1 summarizes the some of properties of the tested soil.

Table 4. 1 Properties of the tested soil

Property	Description or Value
Specific gravity, G_s	2.65
D_{60} *(mm)*	0.22
D_{30} *(mm)*	0.18
D_{10} *(mm)*	0.12
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, kN/m^3	16.05
Void ratio, *e* (after compaction)	0.62 – 0.64
Unified soil classification system *(USCS)*	SP

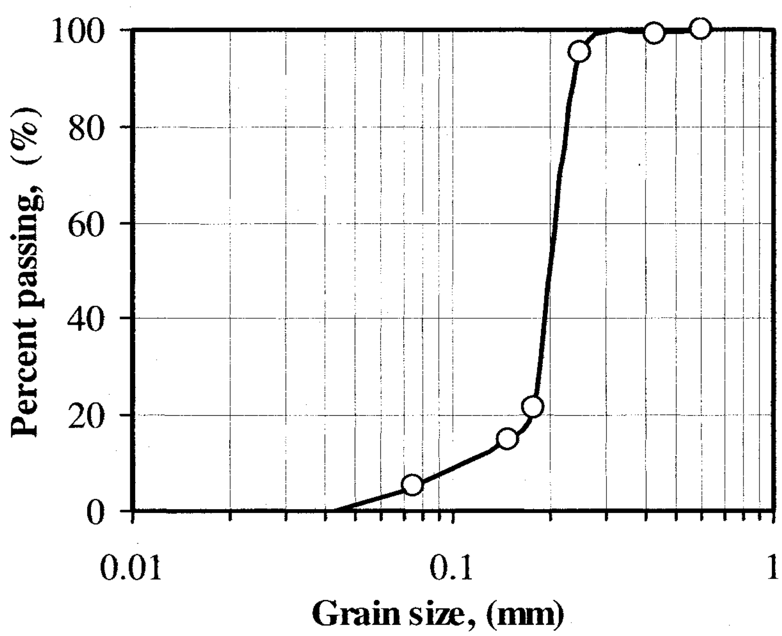


Figure 4. 2 Grain size distribution of the sand used in the study

4.2.2 Compaction Test (Proctor Test)

The compaction test was conducted following the ASTM, Standards D698 (1994c) on the sandy soil used in program. Figure 4.3 shows the relationship between the dry unit density and water content. The optimum moisture content and the maximum dry unit weight from the compaction curve are 14.6 % and 16.8 kN/m^3 respectively.

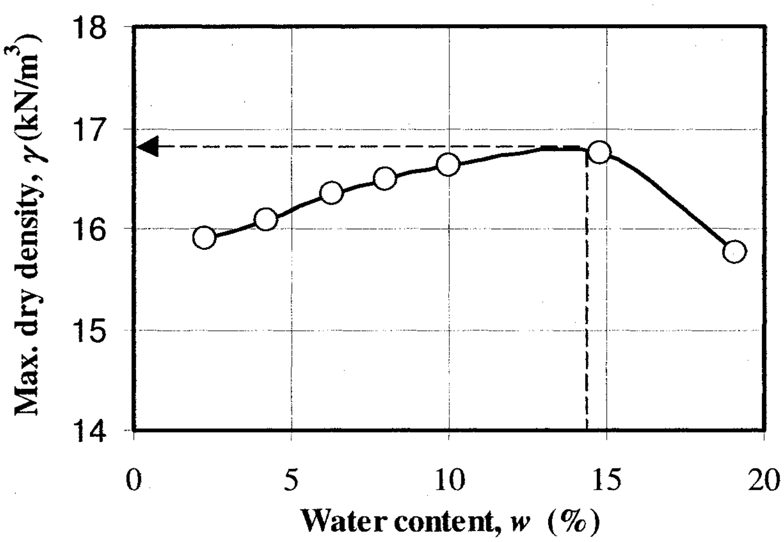


Figure 4. 3 Compaction test results

The soil in the bearing capacity tank was planned to be compacted at optimum moisture content using a 5 kg hammer to achieve highest value of density index. More details with respect to the compaction procedure used in the tank are detailed in later sections.

4.2.3 Relative Density and Specific Gravity Tests

The relative density tests were conducted to determine the maximum and minimum densities for the sand used in this research according to the ASTM Standard, D4235 (1994). The sand was filled in a standard compaction mold in several layers. The mold with the soil was then vibrated by tapping it sharply on the sides with a rubber mallet. The maximum density of the sand was calculated from mass-volume relationships. This value was obtained from an average of three trails. The minimum density of the soil was obtained by filling the mold with the soil without any vibrations.

The relative density can be expressed in terms of unit weight as given below:

$$D_r = \left(\frac{\gamma_f - \gamma_1}{\gamma_2 - \gamma_1}\right)\left(\frac{\gamma_2}{\gamma_f}\right) \qquad [4.\ 1]$$

where:

γ_1 = unit weight of the soil in the loosest state, 14.23 kN/m^3

γ_2 = unit weight of the soil in the densest state, 17.25 kN/m^3

γ_f = unit weight of the soil in the bearing capacity test tank

Table 4. 2 Results of relative density tests from four trials

Trial #	Dry unit weight, (kN/m^3) from the test tank[1]	Relative density, D_r (%) (Density Index)
1	16.02	64
2	15.95	62
3	16.07	66
4	15.99	63
Average values	16.01	63.75 %

[1] A small cup was placed in the test tank at a depth of 200 to 300 mm from the surface of the tank. The tank was filled up with the soil and compacted using 5 kg hammer as detailed in Section 4.3.1. This cup was then removed and the unit weight of the soil in the cup was calculated from mass-volume relationships. Each trial result is an average of 3 sets of data.

The specific gravity tests were performed in the laboratory according to ASTM D854, (1994). The average value of the specific gravity, G_s of the soil used in the study was equal to 2.65 from three tests.

4.2.4 Direct Shear Test

The saturated shear strength parameters c' and ϕ' of the compacted sand were measured using the direct shear apparatus. These parameters are required for predicting the bearing capacity of the soil both in the saturated and unsaturated conditions

The dry sand sample was tamped in to the direct shear box to a dry density value similar to the soil compacted in the bearing capacity tank. The soil sample was saturated prior to loading and shearing in the direct shear box by allowing access to water slowly from the bottom of the soil sample in the shear box. These samples were then loaded under different normal stresses and sheared at a constant strain rate of 1.2 mm per minute. The strain rate used for loading the model footings in the test tank was same as the strain rate used for shearing the sand sample in the direct shear apparatus.

The relationship between the shear stress and the normal stress for two sets of data was plotted in Figure 4.4. The internal friction angle, ϕ' was found to be 32.60 degrees (tested under normal stresses of 25, 50 and 75 kPa) for the first set of data. The internal friction angle, ϕ' was 35.63 degrees (tested under normal stresses of 50, 100 and 150 kPa) from the second set of data. There was a small value of cohesion, c' in the range of 0 to 2.6 kPa. This small value of cohesion may be attributed to the contribution of 5 % of silt in the soil.

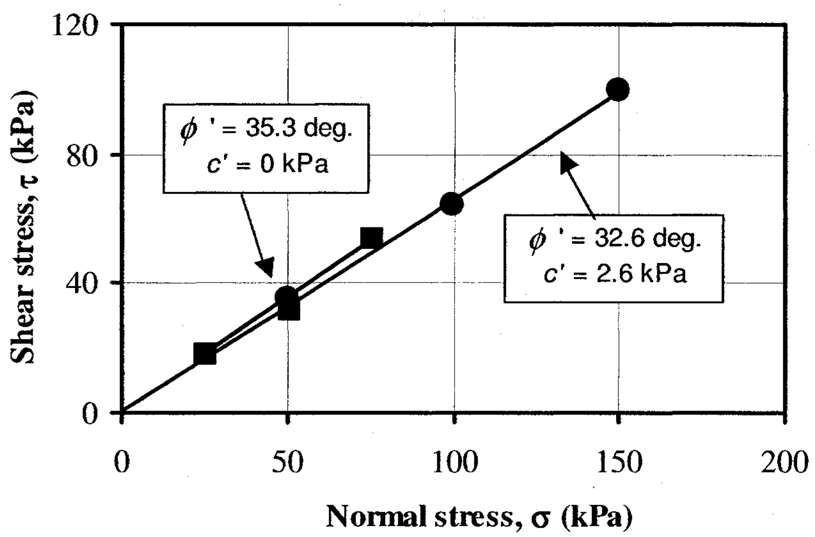


Figure 4. 4 Direct shear test results

4.3 Experimental Program - II

The bearing capacity of the compacted sand in the UOBCE was determined using two square model footings (i.e., 100 mm × 100 mm and 150 mm × 150 mm) under both saturated and unsaturated conditions. The ground water table was raised to the top of the soil using drainage valves in the bearing capacity tank to achieve fully saturated conditions. The water table was adjusted using the drainage valves in the UOBCE to achieve different capillary suction values below the footing. After achieving desired saturated and unsaturated conditions, the model footings were loaded axially to determine the relationship between the load versus settlement and the applied stress versus settlement data. Figure 4.5 shows the details of different tests conducted in Experimental Program-II as a flowchart.

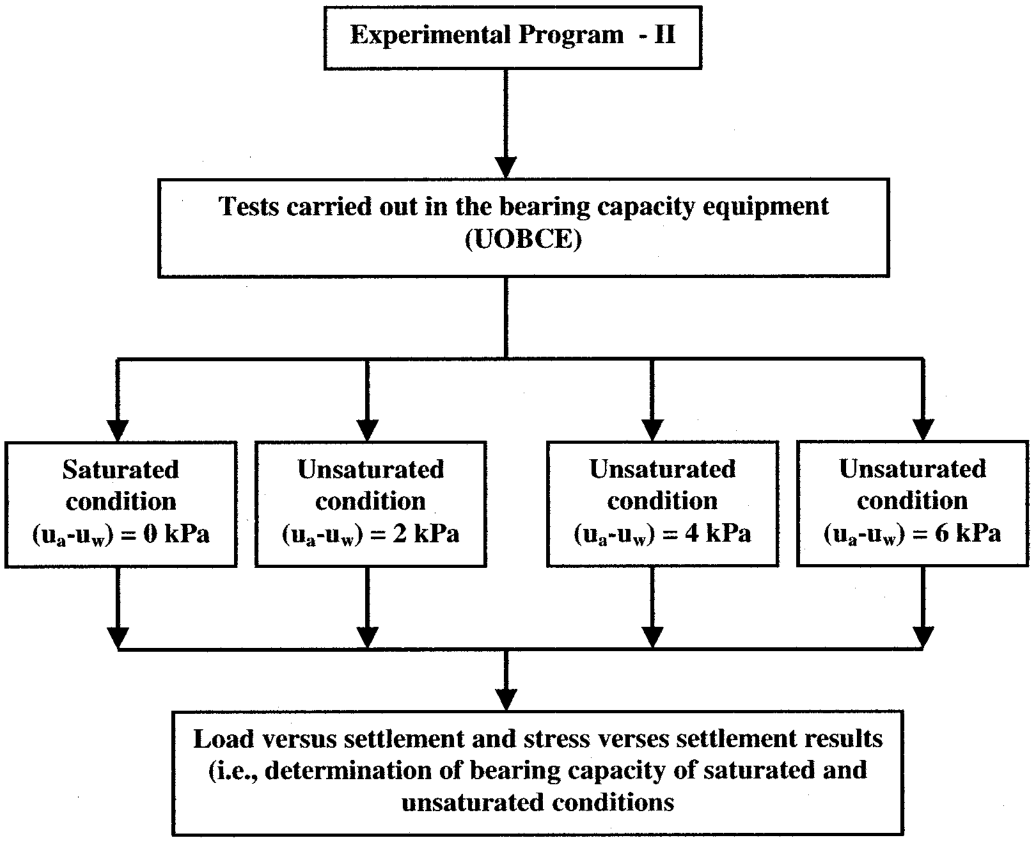


Figure 4. 5 Details of the experimental program undertaken in the OUBCE to determine the bearing capacity of the tested sand under saturated and unsaturated conditions

4.3.1 Preparation of the UOBCE Tank for Testing

The bearing capacity of a sandy soil is dependent on its density index value, D_r value. The relative density index value should be approximately 70% for a surface footing to fail under general shear failure mode (Vesic 1973). The highest density index value that was achieved using the hopper as discussed earlier in Chapter 3 for the sand used in the study was 55%. For this reason, the soil was further compacted using a 5 kg hand compactor after allowing access of water to the sand to achieve a value that was approximately equal to 14 % in the top layers of the soil. This was achieved by raising the ground water table to a value which was approximately 500 mm from the top surface of the soil. The soil in the top layers imbibed water by capillary stresses. Several trials have shown that the water content in the top 300 mm is approximately 14 % (i.e., close to the optimum water content of 14 %). The average density index value that was achieved in the test tank was equal to 64 % (see Table 4.2). The density index was carefully controlled for all the tests to ensure identical conditions. The density of the tested soil was verified by collecting soil samples in aluminum cups for all the tests. The small cups used in the study had perforations as shown in Figure 4.6, which were placed at different levels in the tank to determine the variation of water content with depth.

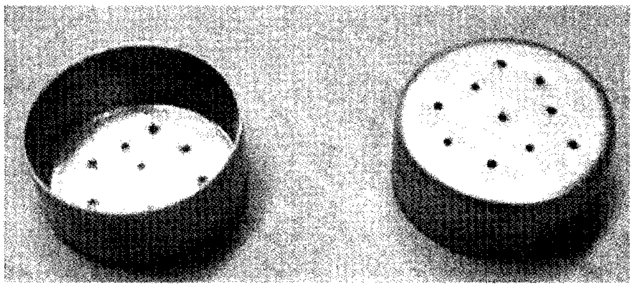


Figure 4. 6 Small aluminum cups

Table 4.2 provides a summary of the data of relative density values along with other information, which will be discussed in later sections of the chapter.

A number of experiments were performed to measure the bearing capacity the compacted coarse-grained soil used in this study under different conditions. The first series of tests performed under fully saturated condition (i.e., zero suction), and the second series of tests were conducted under unsaturated conditions for three different suction values (i.e., 2 kPa, 4 kPa and 6 kPa). A minimum of three bearing capacity tests were conducted and average values were reported. The different tests carried out in this testing program are discussed in the following section.

4.3.2 Bearing Capacity Tests under Saturated Condition

The water table was slowly raised from the base of the tank through the bottom aggregate layer as illustrated in Figure 4.11. This technique facilitated escape of air from bottom to the surface layers of the soil in the tank gradually to ensure a fully saturated condition. The water levels were inspected periodically in the piezometers. The supplier valve was closed once the water level reached the soil surface in the tank. The applied load versus settlement and stresses versus settlement for fully saturated conditions using 100 mm × 100 mm and 150 mm × 150 mm respectively shown in Figure 4.7 through Figure 4.10.

The bearing capacity of the saturated soil was measured by loading the footing until failure. All Tensiometers were indicating zero suction values after saturation and during the testing period.

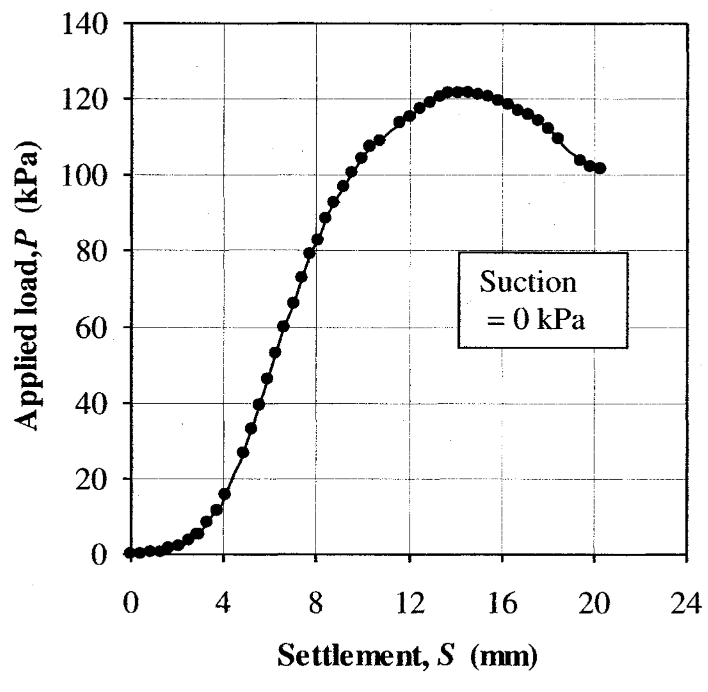


Figure 4. 7 Load versus settlement data from 100 mm × 100 mm footing tested under saturated conditions

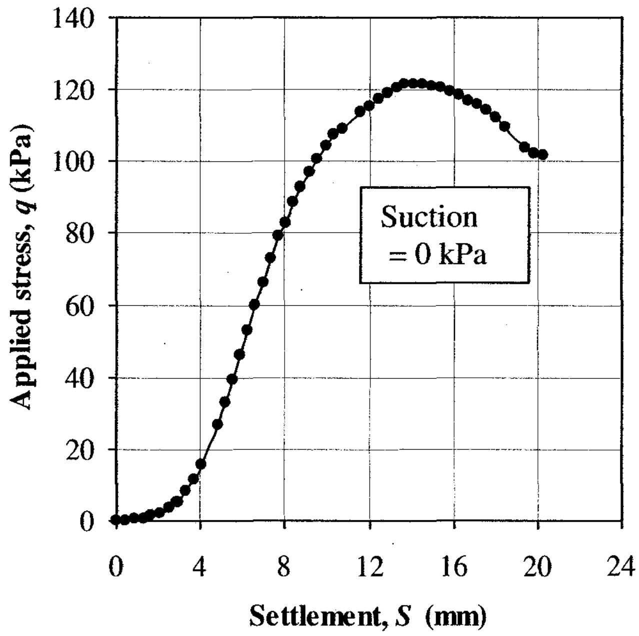


Figure 4. 8 Stress versus settlement data from 100 mm × 100 mm footing tested under saturated conditions

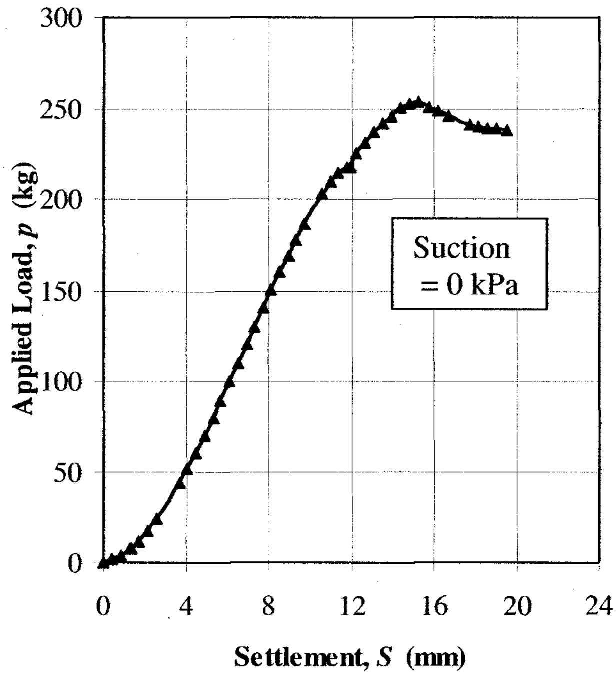


Figure 4. 9 Load versus settlement data from 150 mm × 150 mm footing tested under saturated conditions

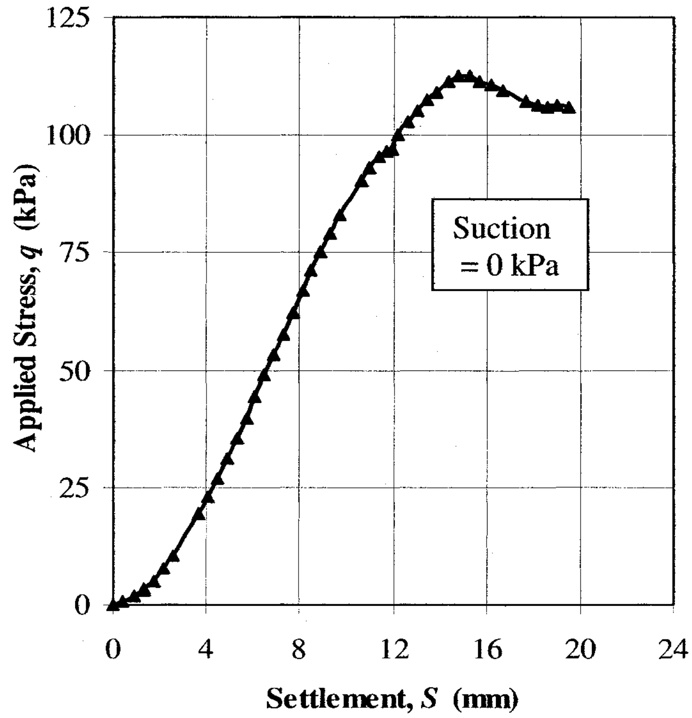


Figure 4. 10 Stress versus settlement data from 150 mm × 150 mm footing tested under saturated conditions

4.3.3 Bearing Capacity Tests under Unsaturated Conditions

The soil in the test tank was first saturated as detailed in section 4.3.2. The water table was then lowered (using drainage valves) to different levels of depth from the soil surface to achieve different capillary suction values. Equilibrium conditions with respect to suction value in the stress bulb zone (i.e., depth of 1.5*B*) were typically achieved in a time period of 24 hours. The bearing capacity of the coarse-grained compacted soil using this technique was measured under different average suction values (i.e., 2, 4 and 6 kPa). While the suction values were measured using the Tensiometers, the gravimetric water contents were determined approximately at the same levels collecting soil specimens in small aluminum cups. These cups with perforations were embedded in the soil below the model footing and close to each ceramic-tip of the Tensiometers. Figure 4.11 shows a typical cross section of the tank in a schematic form and provides the details of the placement of Tensiometers and the aluminum cups (labeled as C_{1-1}, C_{2-1}, C_{3-1} and C_{4-1}) at different elevations.

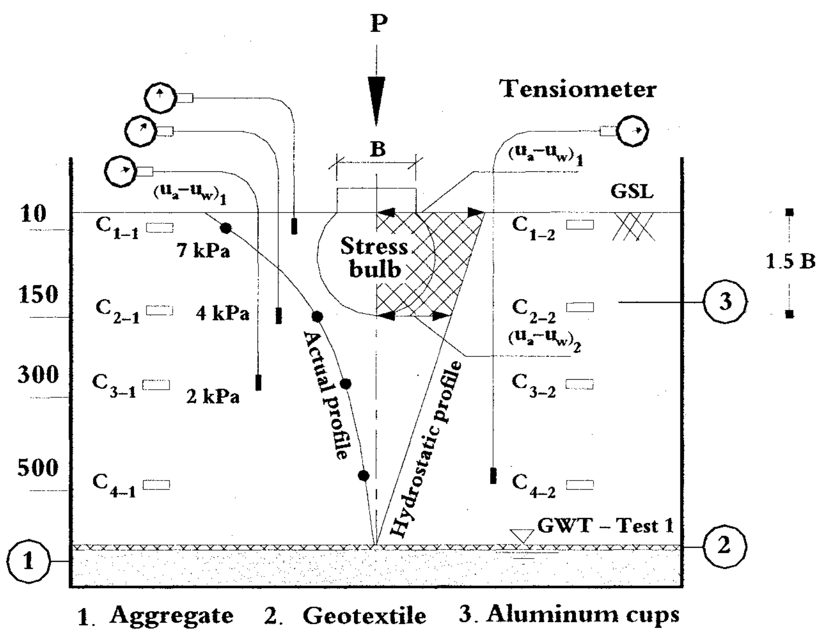


Figure 4. 11 A typical schematic diagram to demonstrate the procedure used for determining the average matric suction below the footing

4.3.3.1 Tests under Average Matric Suction of 2 kPa in the Depth of Stress Bulb

Similar to the procedure undertaken for performing tests of 4 kPa, the tests for 2 kPa were carried out simply by moving the water table up to approximately 150 mm from the soil surface in the test tank (refer to Figure 4.11). The average stress was taken as 2 kPa in the depth of 1.5(*B*), which represents the depth of the stress bulb. The results of the load versus settlement and stress stresses versus settlement for 100 mm × 100 mm and 150 mm × 150 mm are shown through Figures 4.12 to Figure 4.15.

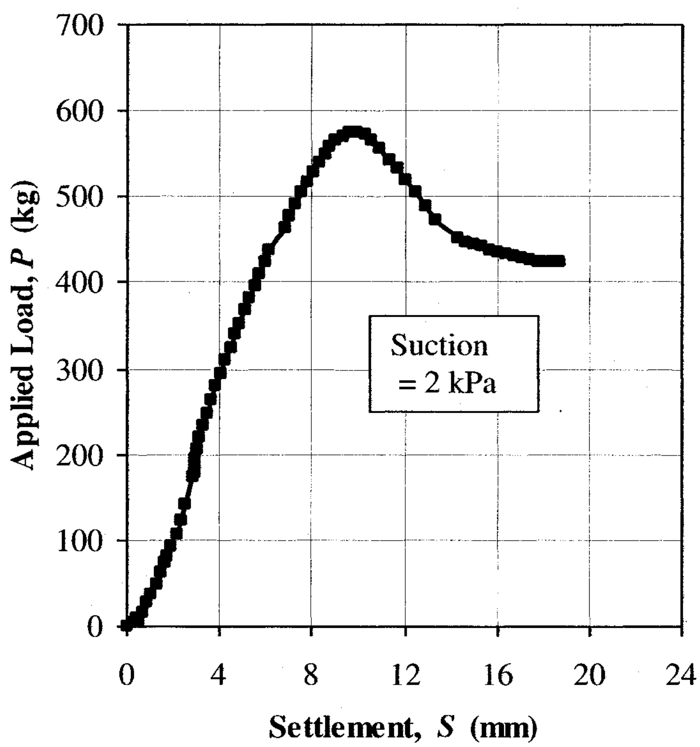


Figure 4. 12 Load versus settlement data from 100 mm × 100 mm footing tested under an average suction value equal to 2 kPa

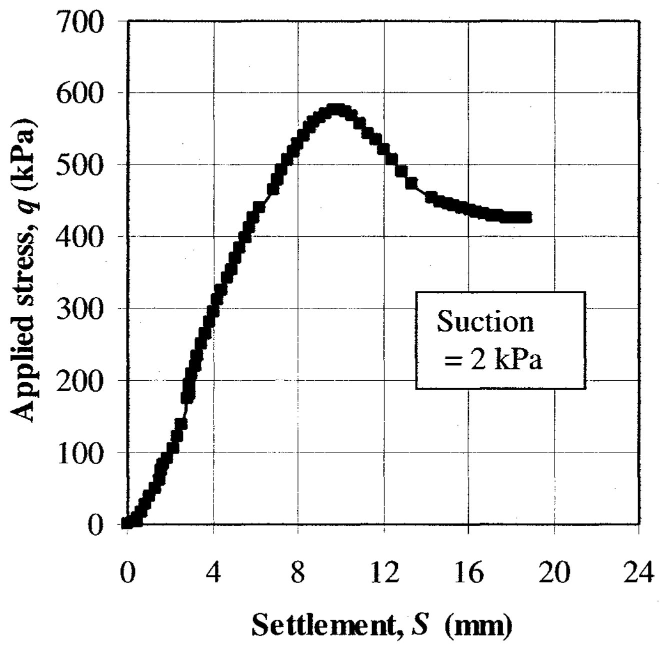


Figure 4. 13 Stress versus settlement data from 100 mm × 100 mm footing tested under an average suction value equal to 2 kPa

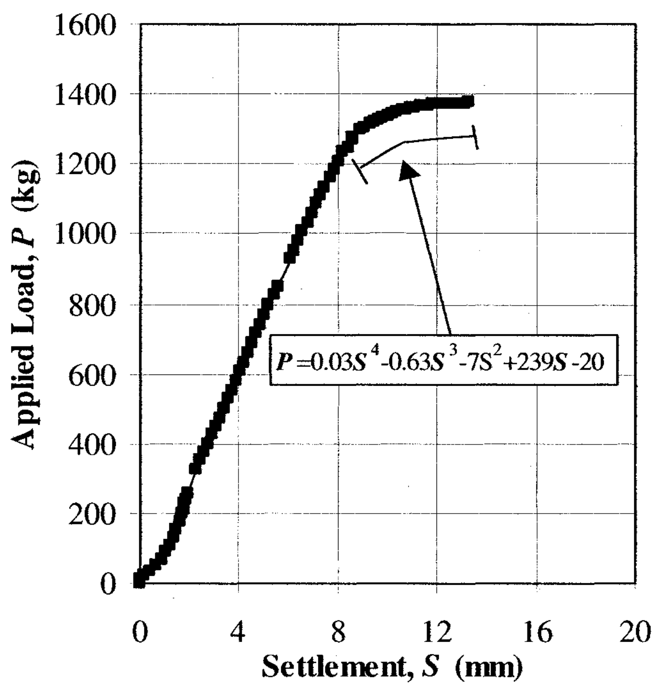


Figure 4. 14 Load versus settlement data from 150 mm × 150 mm footing tested under an average suction value equal to 2 kPa

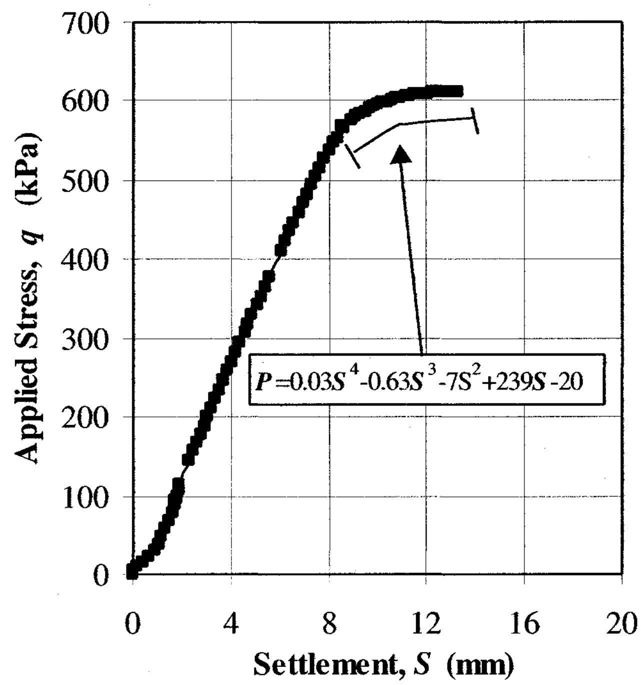


Figure 4. 15 Stress versus settlement data from 150 mm × 150 mm footing tested under an average suction value equal to 2 kPa

4.3.3.2 Bearing Capacity Tests under Average Matric Suction of 4 kPa

The ground water table level was placed at 350 mm from the soil surface in the test tank to achieve an approximate suction value of 4 kPa. The upper Tensiometer was reading 5 kPa and the second Tensiometer from the top was reading 3 kPa. The average suction value was considered as 4 kPa in the stress bulb zone. The footing was only loaded until 14 kN as it was maximum capacity of the loading machine. However, the 150 mm × 150 mm footing did not fail at this load. For this reason, extending the stress versus strain relationship (using hyperbolic relationship) an approximate value of the failure load was estimated.

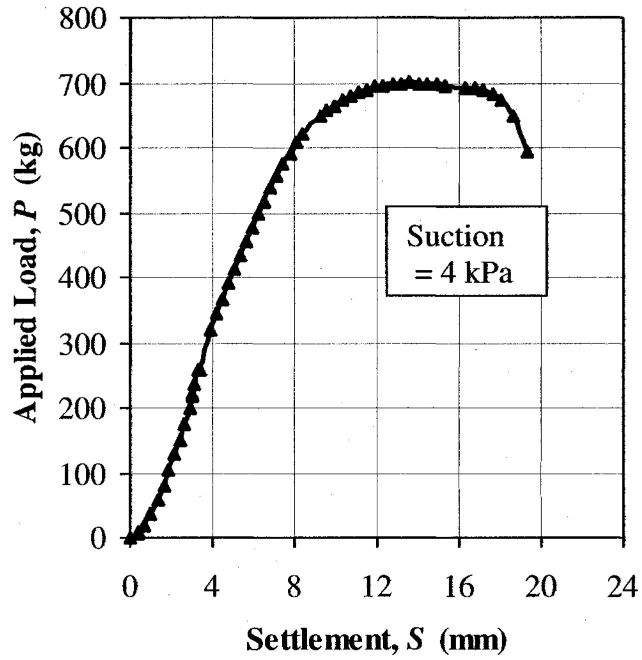


Figure 4. 16 Load versus settlement data for 100 mm × 100 mm footing tested under a suction value of 4 kPa

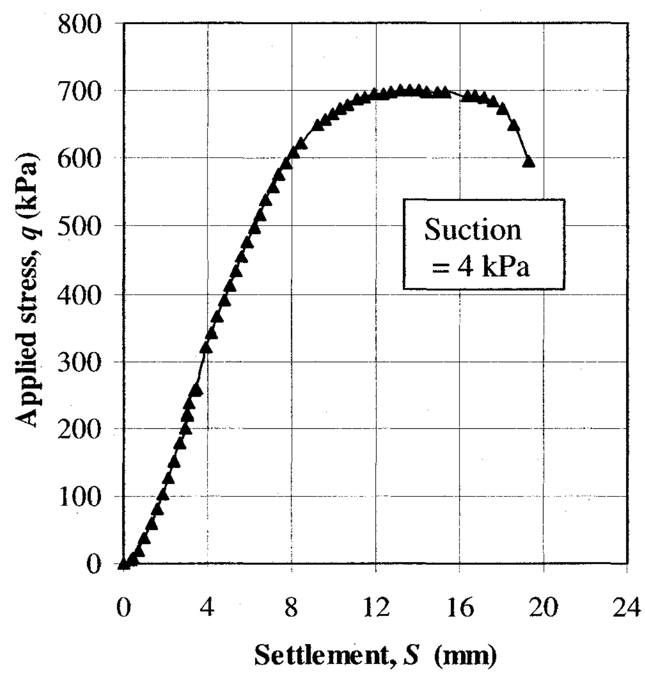


Figure 4. 17 Stress versus settlement data for 100 mm × 100 mm footing tested under a suction value of 4 kPa

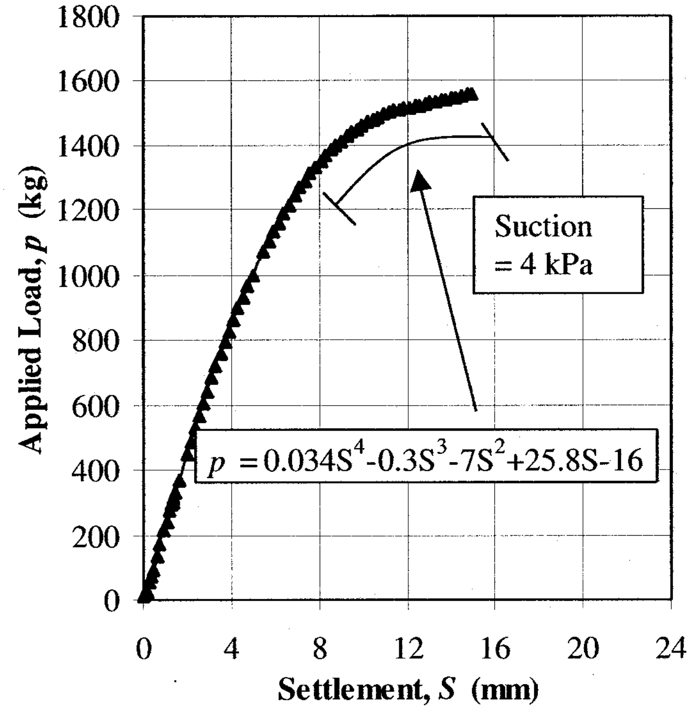


Figure 4. 18 Load versus settlement data for 150 mm × 150 mm footing tested under a suction value of 4 kPa

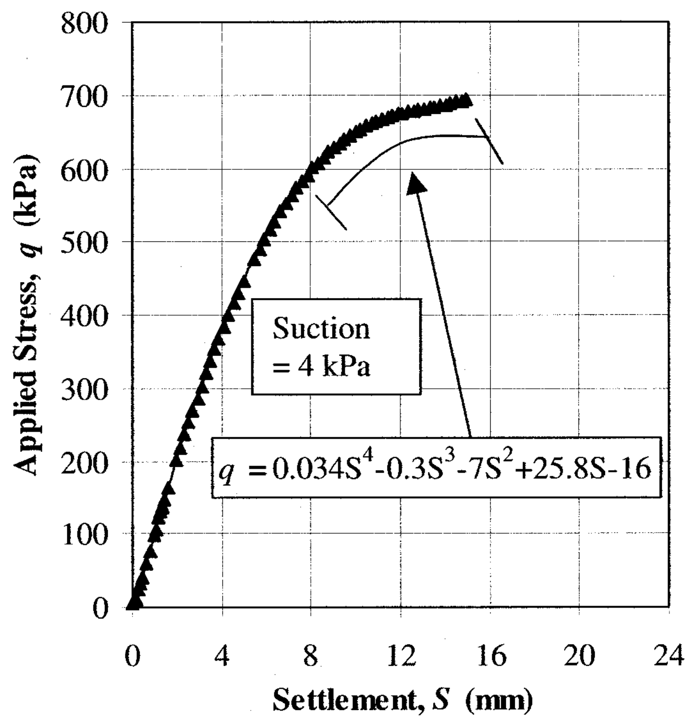


Figure 4.19 Stress versus settlement data for 150 mm × 150 mm footing tested under a suction value of 4 kPa

4.3.3.3 Bearing Capacity Tests under Average Matric Suction of 6 kPa

The model footings were tested following the procedures discussed in earlier sections. The variation of the matric suction with respect to depth underneath the model footing is non-linear as shown in Figure 4.11 (left hand side). However, for simplification purposes the variation of matric suction was assumed to be hydrostatic above the ground water table (GWT) (see right hand side of Figure 4.11). A typical set of data collected is presented in Table 4.3. Load versus settlement and stress versus settlement data are not presented for this suction values.

Table 4. 3 Typical data from the test tank for an average suction value of 6 kPa in the stress bulb zone

D[1] (mm)	γ_t (kN/m³)	γ_d (kN/m³)	e	w (%)	S (%)	AVR[1] $(u_a - u_w)$ (kPa)
10	18.17	15.94	0.63	14.0	58	6
150	18.75	15.85	0.64	18.3	76	4
300	19.27	16.07	0.62	20.0	86	2
500	19.40	15.77	0.64	23	94	1
600	19.75	15.95	0.63	23.8	100	0

[1] AVR: Average value

where:

D = depth from the soil surface of tank, *mm*

γ_t = total unit weight, *kN/m³*

γ_d =dry unit weight, *kN/m)*

e = void ratio

w =water content, *%*

S =degree of saturation, *%*

$(u_a - u_w)$ = matric suction, *kPa*

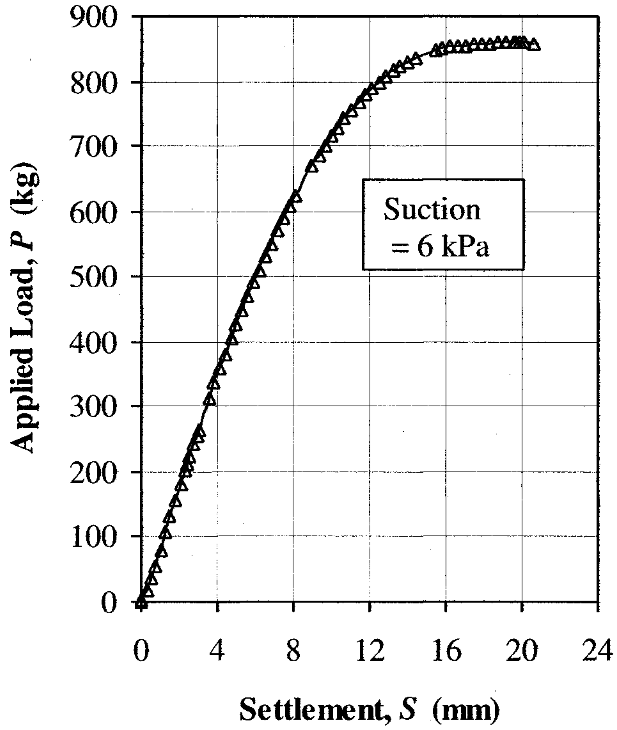


Figure 4. 20 Load versus settlement data for 100 mm × 100 mm footing tested under a suction value of 6 kPa

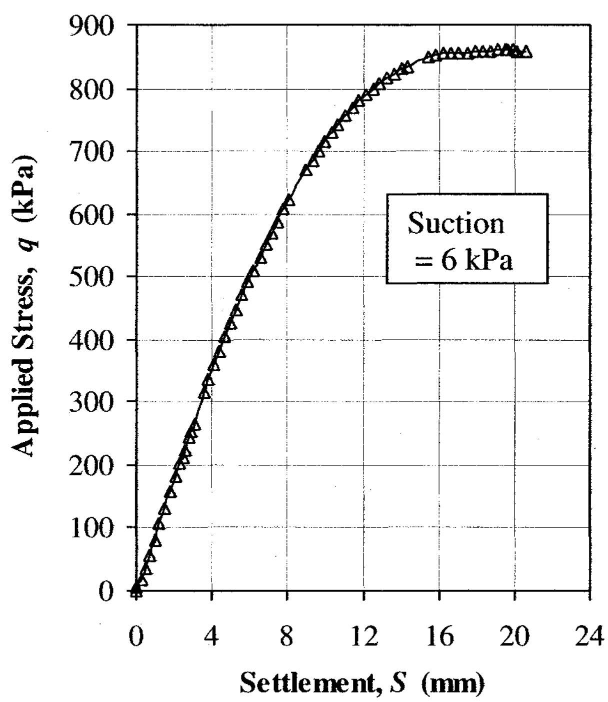


Figure 4.21 Stress versus settlement data for 100 mm × 100 mm footing tested under a suction value of 6 kPa

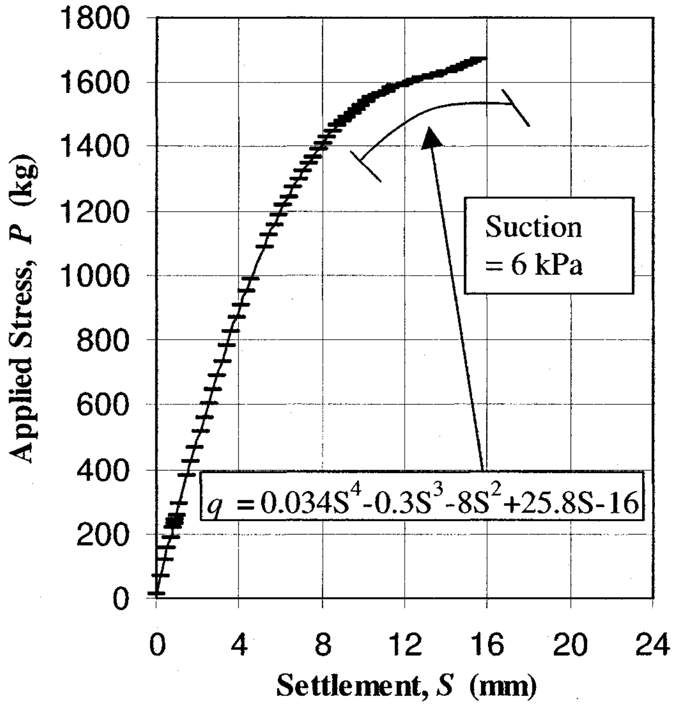


Figure 4. 22 Load versus settlement data for 150 mm × 150 mm footing tested under a suction value of 6 kPa

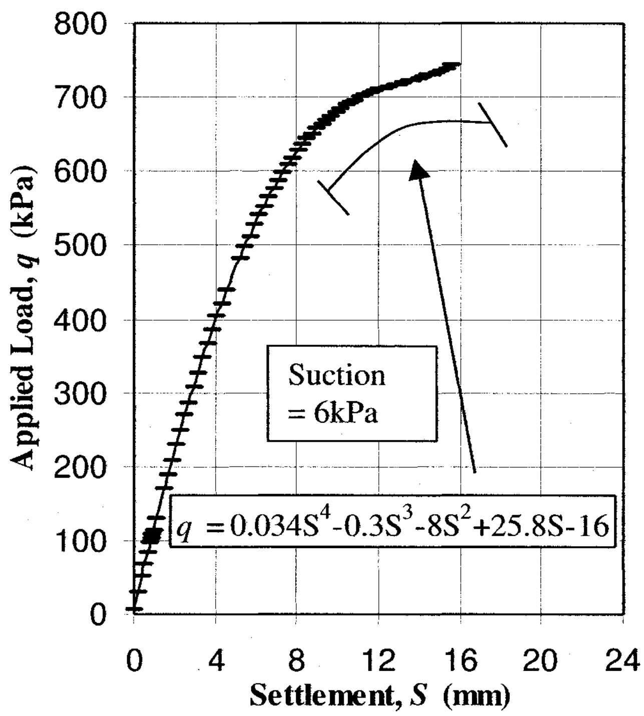


Figure 4.23 Stress versus settlement data for 150 mm × 150 mm footing tested under a suction value of 6 kPa

4.4 Experimental Program - III

The objective of the determination of SWRC was to understand its relationship with the bearing capacity of unsaturated soils. The SWRC is determined using three different methods. Figure 4.24 the details of the three different procedures used in the determination of the SWRC.

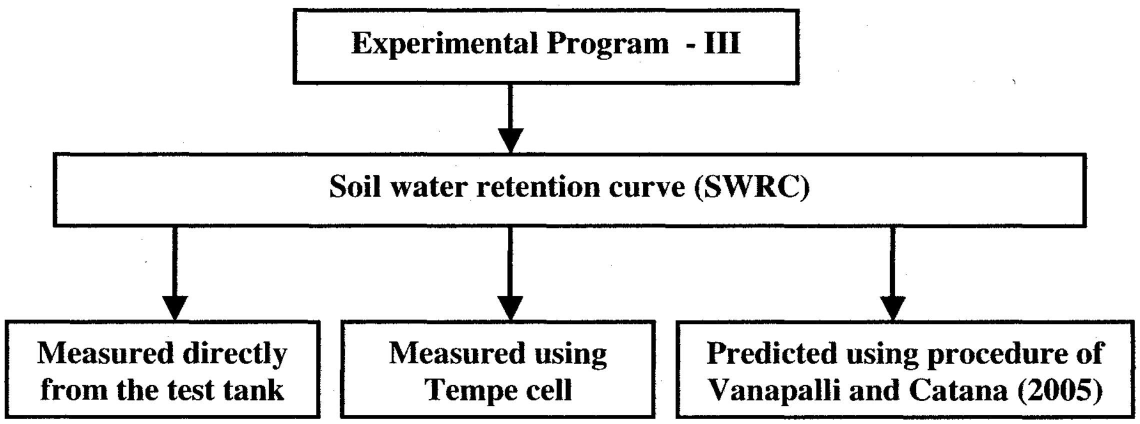
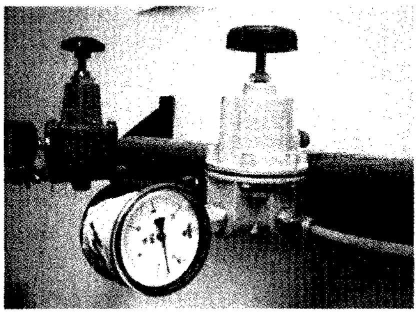


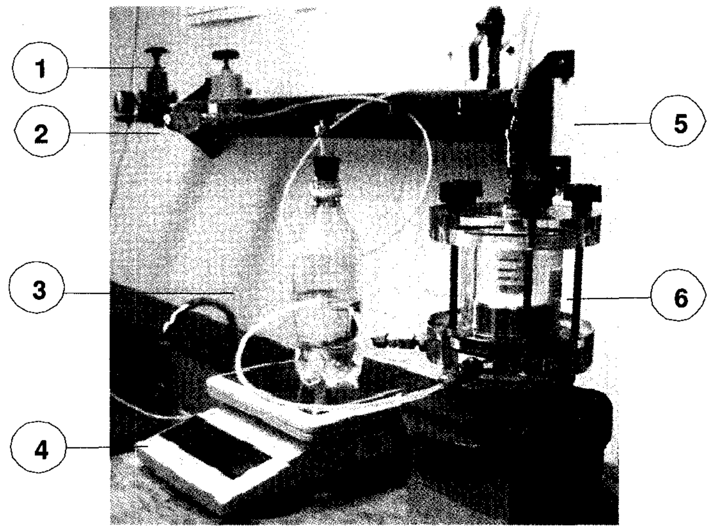
Figure 4. 24 Experimental Program - III

4.4.1 Measuring the SWRC Using the Tempe Cell Apparatus

The setup details for measuring the SWRC in the laboratory using Tempe cell apparatus is shown in Figure 4.25. The procedure for determine the SWRC was discussed in Chapter 3. The objective of this test is to desaturate a saturated specimen by applying suction and determine the water content that is available in the specimen at different values of suction. The electronic scale as shown in Figure 4.25 (a) facilitated in measuring the mass of the specimen at different applied suction values. The measured SWRC using the Tempe cell is plotted in Figure 4.26 below.



(a)



1. Regulator 2 .Low pressure guage
3. Drained water 4. Electronic Scale
5. Pressure supplier 6. Tempe Cell

(b)

Figure 4. 25 Measuring the SWRC; (a) Sensitive pressure gauge and (b) Tempe cell apparatus

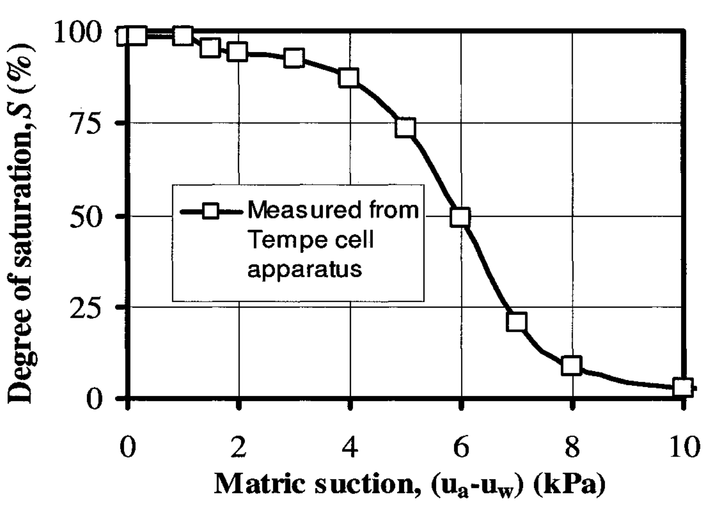


Figure 4. 26 Measured SWRC from the Tempe cell

4.4.2 Measuring the SWRC from the Test Tank

The SWRC was measured directly from the test tank. The water content measurements were determined from the test tank using small aluminum cups (with perforations). The small test cups were embedded in the soil inside the tank at different levels and close to the tip of the Tensiometers located at different depths. Four cups were place in each depth. After adjusting the ground water table at a certain level, different suction readings were recorded from the Tensiometers.

The saturated water content was also measured from the soil specimens in aluminum cups (without perforations) under the ground water table level when the Tensiometer readings were reporting zero suction values (the level of the water is at the surface of the soil in the test tank). Figure 4.27 shows the SWRC measured from the test tank for 5 points.

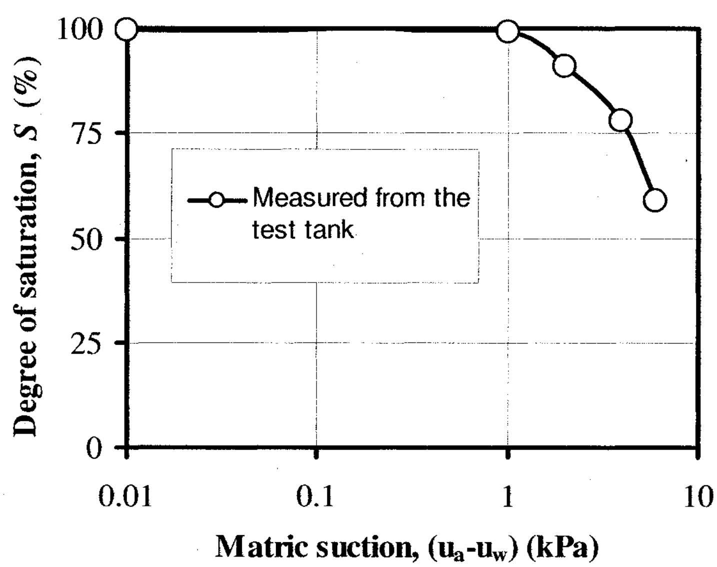


Figure 4. 27 Measured SWRC from the bearing capacity test tank

4.4.3 Predicting the SWRC

The third method that was used to obtain the SWRC was a prediction method. The method can be used to estimate the SWRC of coarse-grained soils using parameters derived from the grain size distribution curve and the volume mass properties. In this method, a correlations were developed between the fitting parameters, *a* and *n* which are functions of the coefficient of uniformity, C_u and the dominant particle size, d_e and the void ratio, *e*. The SWRC can be estimated using the proposed equation by Vanapalli and Catana (2005) and one measured data point, (i.e., matric suction, $(u_a - u_w)$ and water content, *w*). This step-by-step procedure of this method is summarized as follows:

- One measured point of suction versus water content is needed for estimating the SWRC; the data point used had a water content, *w* and matric suction, $(u_a - u_w)$ was 4 kPa and 18.3% respectively.
- The fitting parameters *a* and *n* were estimated using equation 4.2 and 4.3 respectively.

$$a = \frac{1.33}{(de)^{0.86}} \qquad [4.2]$$

$$n = \frac{7.78}{(C_u \times e)^{1.14}} \qquad [4.3]$$

- The one measured point is used for determination of the third fitting parameter, *m*.
- Equation [4.2] and Equation [4.3] along with the information from the grain-size distribution can be substituted in Fredlund and Xing (1994) equation to develop the following equation:

$$f = \left(e + \left(\frac{\psi}{1.33/de^{0.86}} \right)^{7.78/(C_u \times e)^{1.14}} \right)^{mf} \qquad [4.4]$$

where:

a = first fitting parameter

n = second fitting parameter

e = void ratio

d_e = dominant diameter, *mm*

C_u = coefficient of uniformity

ψ = soil suction, *kPa*

Figure 4.28 shows the SWRC predicted using a one-point prediction method following the previous procedures, which proposed by Vanapalli and Catana (2005).

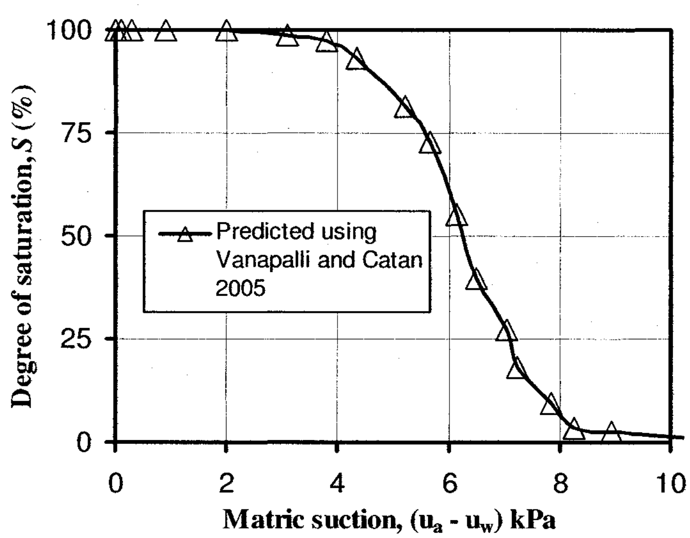


Figure 4. 28 Predicted SWRC

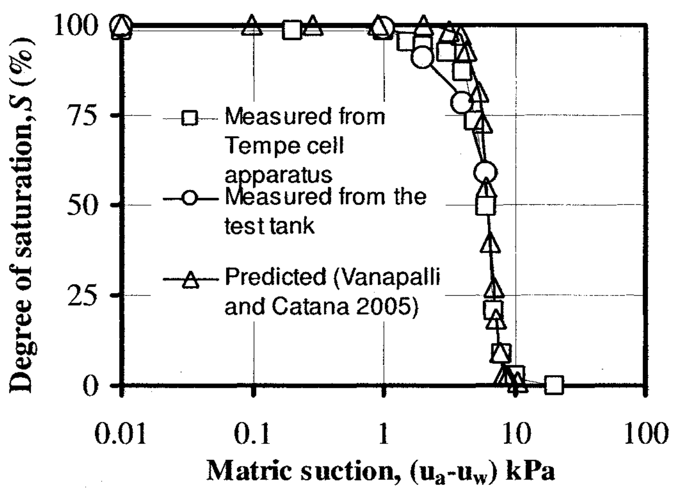


Figure 4. 29 Measured and Predicted SWRC for the tested soil

There was a good comparison between the SWRCs using all the three different methods. The air-entry value is approximately in the range of 2.5 to 3 kPa from all the three methods. There is a steep transition zone in the suction range of 3 to 10 kPa (Figure 4 .29). Such a behavior is consistent with the nature of relatively graded sand used in the research study.

4.5 Experimental Results of Bearing Capacity

The variation of the bearing capacity of surface square model footings of 100 mm × 100 mm and 150 mm × 150 mm on compacted coarse-grained soil under vertical static loads both under fully saturated conditions and unsaturated conditions were measured. The objective of this study was to determine the increase of the bearing capacity of the model footings under unsaturated condition in comparison to the bearing capacity of model footings in saturated condition. To develop such a relationship the failure loads have to be defined.

4.5.1 Determination of the Failure Loads

The stresses versus settlement for the tests using 100 mm × 100 mm and 150 mm × 150 mm model footing were plotted as presented earlier in section 4.3. The failure mode in all test results was similar and following general shear failure (GSF) criteria where the peak of the load is well defined.

4.6 Analysis of Results and Discussion

In this research program experimental results of the ultimate bearing capacity from all model footings in unsaturated conditions were higher than the bearing capacity of the same model footing in fully saturated condition. The SWRC is plotted on semi-log and arithmetic scale in Figure 4.30 and Figure 4.31 respectively. The loads versus settlement relationship for the two model footings were also plotted in Figure 4.29. The results show that the larger the size of footing, the higher its capacity to carry the applied loads and stresses.

The variation of the bearing capacity with respect to matric suction for two model footings of different sizes (i.e., 100 mm × 100 mm and 150 mm × 150 mm) is shown in Figure 4.33. This relationship demonstrates that there is a significant increase in the bearing capacity of the model footing due to the contribution of suction in the range 0 to 6 kPa for the tested coarse-grained soil. The results also suggest the bearing capacity approximately increases linearly with matric suction up to the air-entry value. There is a non-linear increase in the bearing capacity with respect to matric suction beyond the air-entry value. The trends of the results of the bearing capacity of an unsaturated soil are similar to the shear strength behavior of unsaturated soils. For this reason, it will be useful to propose prediction procedures for estimating the bearing capacity of unsaturated soils along similar lines as the shear strength of unsaturated soils using the SWRC and the effective shear strength parameters (i.e., c' and ϕ').

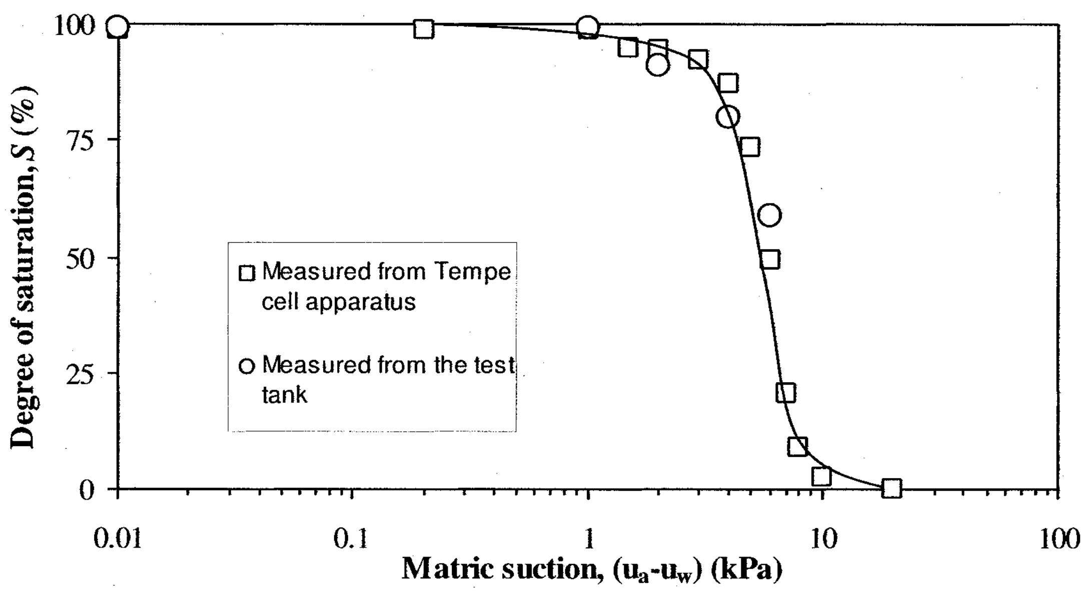


Figure 4. 30 SWRC for the tested soil using two methods

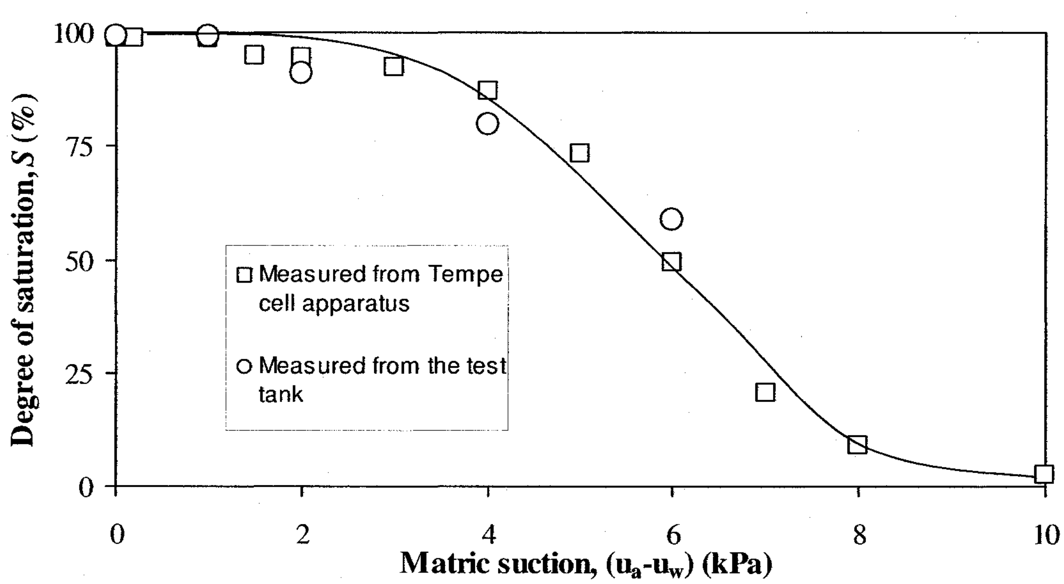


Figure 4. 31 SWRC for the tested soil (arithmetic in scale)

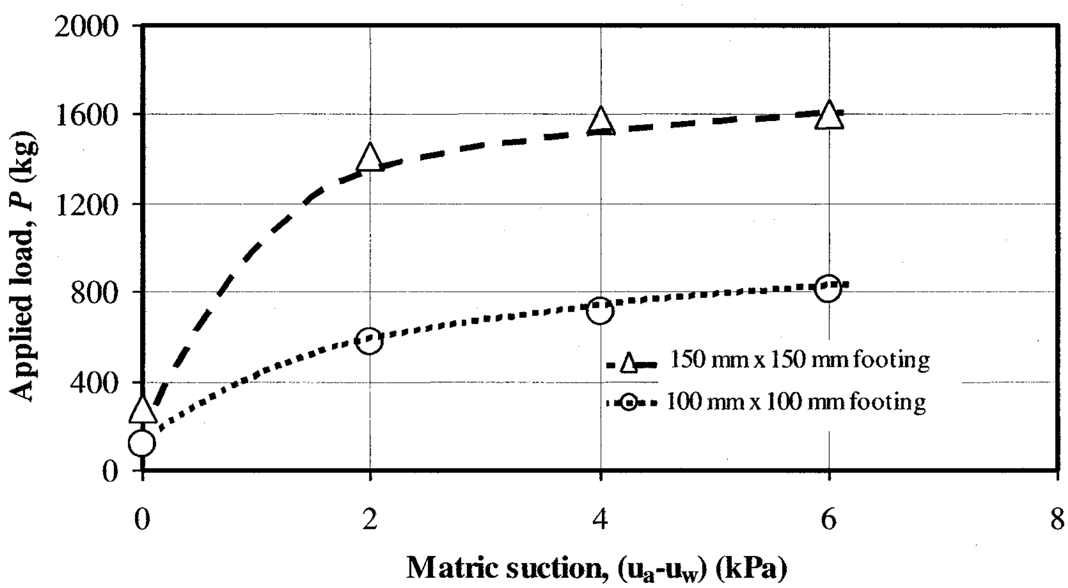


Figure 4. 32 The variation of loading carrying capacity with respect to matric suction for two model footings

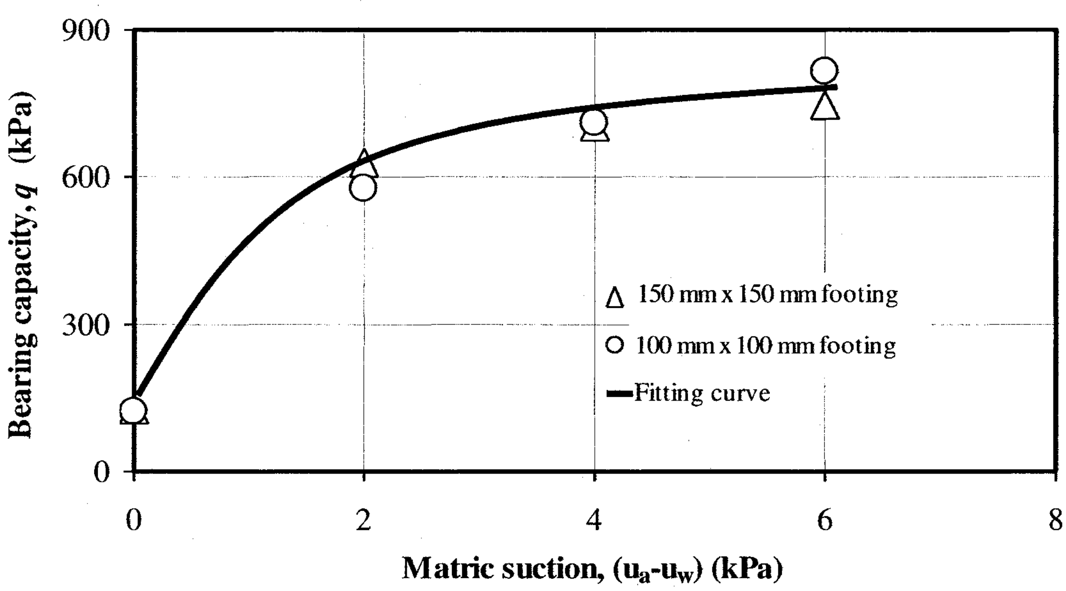


Figure 4. 33 Bearing capacity versus matric suction for two model footings

The bearing capacity for the compacted unsaturated coarse-grained soil measured in this study was observed to be 5 to 7 times higher than the ultimate bearing capacity of the same soil under saturated condition.

The results of this study are consistent with the observations of Steensen-Bach et al. (1987) who reported the bearing capacity of a coarse grained unsaturated soil to be 4 to 6 times higher than the bearing capacity of the same soil under saturated conditions. Ismael (1985) studies also show that the measured bearing capacity of moist coarse grained was 3 to 5 times higher than the bearing capacity determined using Terzaghi's (1943) equation and 2 to 3.5 times higher than Meyerhof's equation (1951 and 1956).

4.7 Summary

In this chapter, the bearing capacity of a surface footing under both saturated and unsaturated conditions were measured for a compacted coarse-grained soil. The bearing capacity was measured in a controlled laboratory environment using two square shaped model footings of different sizes (i.e., 100 mm × 100 mm and 150 mm × 150 mm). The experimental bearing capacity of the tested coarse-grained soil under unsaturated conditions was found to be approximately 5 to 7 times higher than the bearing capacity of the same soil under fully saturated conditions. The results of this experimental program suggest the conventional bearing capacity theory used in the engineering practice is conservative when it is applied for unsaturated soils. The results of the study also suggest that there is a relationship between the SWRC and the bearing capacity of an unsaturated soil is similar to the relationship between the SWRC and the shear strength of an unsaturated soil.

CHAPTER 5

INTERPRETATION OF THE BEARING CAPACITY OF UNSATURATED SOILS

5.1 General

A semi-empirical equation is proposed for interpretation and prediction of the bearing capacity of unsaturated soils. The form of the proposed equation is consistent with the framework originally proposed by Terzaghi (1943) for computing the bearing capacity of soils using the bearing capacity factors based on the angle of internal friction, ϕ'. The proposed equation is presented as a functional relationship such that the variation of the bearing capacity of an unsaturated soil with respect to matric suction can be predicted. When the matric suction value is set to zero, the form of the proposed equation will be similar to the conventional expression used for calculating the bearing capacity of soils that are in a state of saturated condition. This proposed procedure is developed extending the concepts for predicting the shear strength of unsaturated soils proposed by Vanapalli et al. (1996) and Vanapalli and Fredlund (2000). Using the proposed equation, the bearing capacity of an unsaturated soil can be predicted using the saturated shear strength parameters, c' and ϕ' and the soil-water retention curve (SWRC).

The experimental results undertaken on compacted sand using the University of Ottawa Bearing Capacity Equipment (UOBCE) and presented in Chapter 4 and the bearing capacity results of four other different soils published in the literature are interpreted and reanalyzed in this chapter. Of the five soils analyzed, three of them were coarse-grained soils (i.e., sands) while the remainder two soils (i.e., Botkin Pit silt and glacial till) were fine-grained in nature.

Comparisons are provided between the measured bearing capacity and the predicted bearing capacity values using the proposed semi-empirical equation. There is a reasonably a good comparison between the measured and the predicted bearing capacity of the unsaturated soils.

5.2 Relationship between the Shear Strength of Unsaturated Soils and the SWRC – Background

Fredlund et al. (1978) proposed an equation for interpreting the experimental results of the shear strength of unsaturated soil using the equation below:

$$\tau_f = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \qquad [5.1]$$

where:

τ_f = the shear strength of an unsaturated soil, *kPa*

c' = the effective cohesion of a soil, *kPa*

σ_n = the total normal stress, *kPa*

ϕ' = the effective angle of internal friction of a saturated soil

u_a = the pore air pressure, *kPa*

u_w = the pore water pressure, *kPa*

ϕ^b = the angle of internal friction with respect to matric suction, *(deg.)*

$(u_a - u_w)$ = matric suction, *kPa*

$(\sigma_n - u_a)$ = net normal stress, *kPa*

Vanapalli et al. (1996), extended this equation and proposed a semi-empirical function to predict the variation of shear strength with respect to suction using the effective shear strength parameters *c'* and ϕ' along with the SWRC. Fredlund et al. (1996) also proposed a similar function based on the theoretical formulations developed by Vanapalli (1994).

$$\tau = [c' + (\sigma_n - u_a)\tan\phi'] + [(u_a - u_w)S^{\kappa}\tan\phi'] \qquad [5.2]$$

where:

κ = a fitting parameter used for obtaining a best-fit between the measured and predicted shear strength

S = degree of saturation

A relationship between the fitting parameter, κ and plasticity index, I_P was developed by Vanapalli and Fredlund (2000) using the shear strength data of five unsaturated soils.

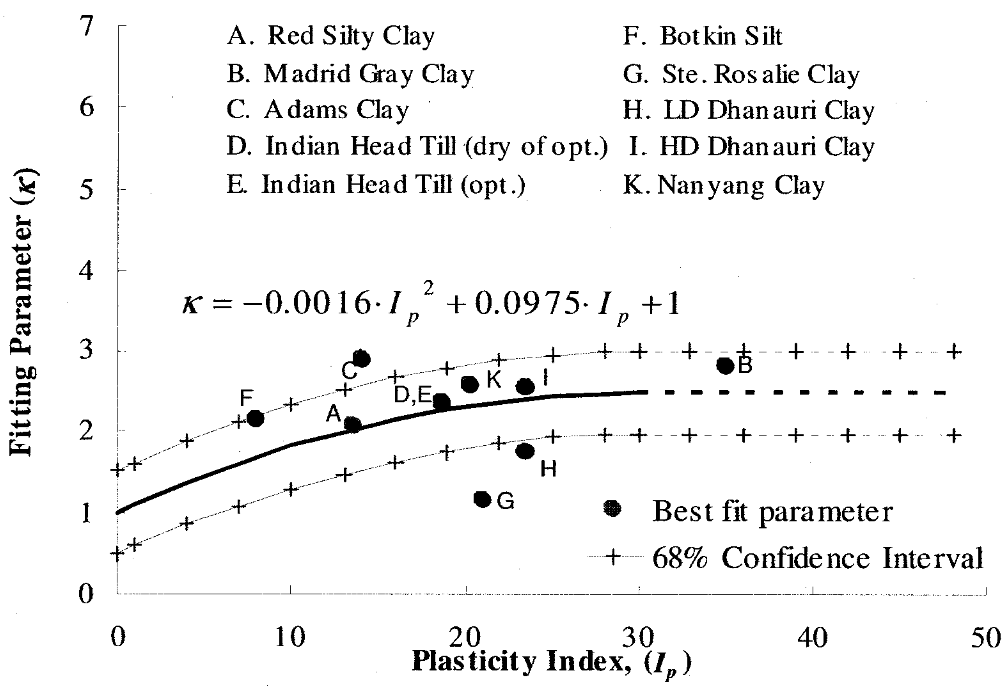


Figure 5. 1 Relationship between κ and I_P for natural, statically compacted soil (Garven and Vanapalli 2006)

The fitting parameter, κ value can be estimated from this relationship based on the plasticity index, I_p value and used in Equation [5.2] to predict the variation of shear strength with respect to matric suction for unsaturated soils. More recently, Garven and Vanapalli (2006) proposed a new relationship for estimating the fitting parameter κ as shown in Figure 5.1 using data of sixteen statically compacted soils.

The bearing capacity of a soil in a state of saturated condition is strongly dependent on the saturated shear strength parameters, c' and ϕ'. Several researchers have developed empirical relationships to compute the bearing capacity of soils using the saturated shear strength parameters, bearing capacity factors, foundation dimensions and shape factors (Terzaghi 1943, Meyerhof 1951, Hansen 1970 and Vesic 1973). Retaining the conventional framework used in the interpretation of the bearing capacity of saturated soils, a semi-empirical relationship is proposed for predicting the variation of bearing capacity of unsaturated soils with respect to matric suction in this thesis using the saturated shear strength parameters, c' and ϕ' and the soil-water retention curve (SWRC).

5.3 Bearing Capacity of Saturated Soils

Terzaghi (1943) proposed an equation for computing the bearing capacity (i.e., Equation [5.3]) for shallow strip footings extending Prandtl (1921) assumptions for the failure mechanisim. This equation is valid for strip footings resting in a homogenous soil and subjected to vertical loading.

$$q_u = c'N_c s_c + qN_q s_q + 0.5B\gamma N_\gamma s_\gamma \qquad [5.3]$$

where:

q_u = ultimate bearing capacity, kPa

q = overburden pressure, kPa

c' = effective cohesion, kPa

N_c, N_q, N_γ = bearing capacity factors due to cohesion, surcharge and unit weight respectively

s_c, s_q, s_γ = shape factors due to cohesion, surcharge and unit weight respectively

γ = soil unit weight, kN/m^3

B = footing width, m

L = footing length, m

Meyerhof (1951) modified this equation by proposing shape and depth factors as shown in Equation [5.4]. This equation is conventionally referred to as the general bearing capacity equation.

$$q_u = c' N_c' s_c' d_c' + q N_q' s_q' d_q' + 0.5 B \gamma N_\gamma' s_\gamma' d_\gamma' \quad [5.4]$$

where:

N_c', N_q', N_γ' = bearing capacity factors

s_c', s_q', s_γ' = shape factors

d_c', d_q', d_γ' = depth factors

Vesic (1973) suggested using Terzaghi (1943) bearing capacity equation with slightly different bearing capacity factors and shape factors. The shape factors proposed by Vesić (1973) are different from Meyerhof (1951) and are shown in equations [5.5], [5.6] and [5.7]. The shape factors can be used for different shapes of footings as function of both the bearing capacity factors, N_c, N_q and N_γ and the dimensions of a footing, B and L.

$$\xi_c = 1.0 + \frac{N_q}{N_c}\left(\frac{B}{L}\right) \quad [5.5]$$

$$\xi_q = 1.0 + \frac{B}{L}\tan\phi' \quad \geq 0.6 \ (\textit{for all values of } \phi') \quad [5.6]$$

$$\xi_\gamma = 1.0 - 0.4\frac{B}{L} \quad \geq 0.6 \quad [5.7]$$

where:

ξ_c = shape factor due to cohesion

ξ_q = shape factor due to surcharge

ξ_γ = shape factor due to unit weight

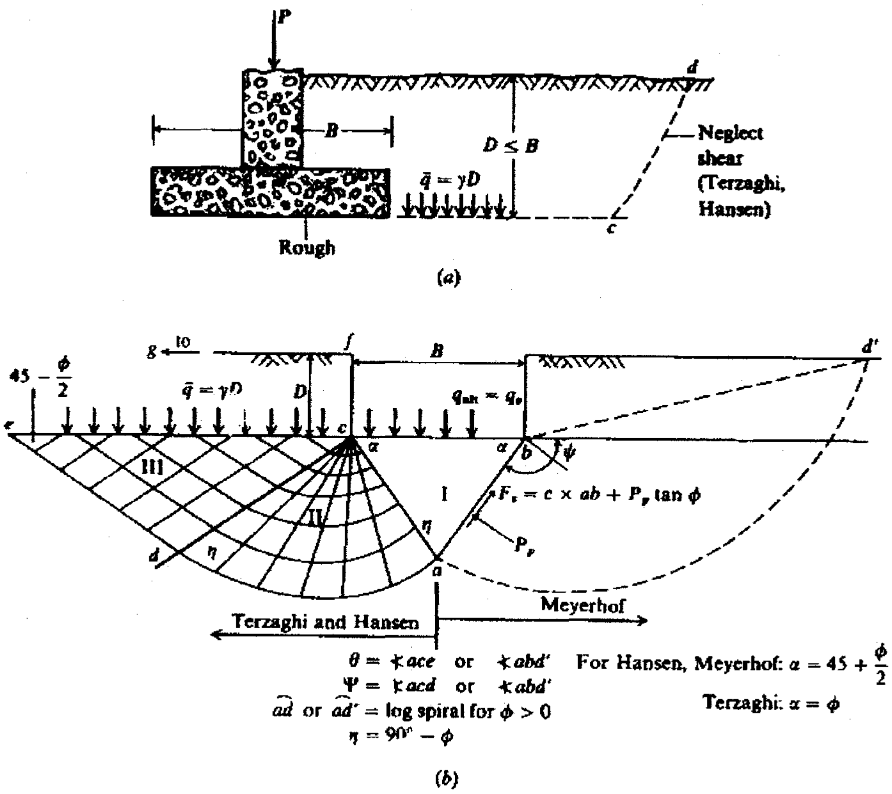


Figure 5. 2 (a) Shallow foundation with rough base defined, (b) general footing-soil interaction for bearing capacity equations (Bowles 1996)

The bearing capacity factors N_c and N_q suggested by Terzaghi (1943), Meyerhof (1951) and Vesic (1973) have small differences and are approximately the same. However, the values of bearing capacity factors N_γ suggested Terzaghi (1943), Meyerhof (1951) and Vesic (1973) and also by various other investigators are significanlty different from one another. These differences may be attributed to the differences in assumptions related to the failure mechanism and the assumptions used in the shape of the failure wedge. For example, see Figure 5.2 for noting the differences in the assumptions used by Terzaghi (1943) and

Meyerhof (1951). The differences in assumption leads to signifcant differences in the values cacluated for the passive pressure, P_p generated below the footing . This in turn translates to differences in the computed N values and praticularly N_γ values (Bowles 1996).

The bearing capacity factor, N_γ is significantly different especially for ϕ' values greater than 35°. Vesić (1973) stated "*The question of correct, N_γ values has remained unsettled, because of difficulties in selecting a representative value of angle of shearing resistance, ϕ' for the bearing capacity computations*". Some investigators, for example Terzaghi (1943) have not provided details of how the bearing capacity factor, N_γ was calculated (Bowles 1996). More discussions with respect to choosing an appropriate ϕ' value for bearing capacity computations is discussed in a later section.

Kumbhojkar (1993) provided different bearing capacity factor values for N_γ based on results of a numerical solution and compared with the relationship of N_γ versus ϕ' relationship proposed by Terzaghi (1943). The explicit analytical expressions for calculating, N_γ by Kumbhojkar (1993) show higher values in comparison to studies reported in the literature (Terzaghi, 1943, Meyerhof, 1951 and Vesic, 1973). Kumbhojkar (1993) study also shows that bearing capacity computations provide better comparisons with the measured bearing capacity values using the N_γ values based on his study results.

In the present analysis the bearing capacity factors, N_c and N_q proposed by Terzaghi (1943) and N_γ proposed by Kumbhojkar (1993) are used. These bearing capacity factors are summarized in Table 5.1. In addition, the shape factors proposed by Vesic (1973) (i.e., Equation [5.5], [5.6], and [5.7]) are used in this research program since they take into account the bearing capacity factors and the footing dimensions.

Table 5. 1 Bearing capacity factors used in the research study, N_c and N_q from Terzaghi (1943) and N_γ from Kumbhojkar (1993)

ϕ'	N_c	N_q	N_γ
0	5.70	1.00	0.00
5	7.34	1.64	0.14
10	9.61	2.69	0.56
15	12.86	4.45	1.52
20	17.69	7.44	3.64
25	25.13	12.72	8.34
30	37.16	22.46	19.13
36	63.53	47.16	54.36
39	85.97	70.61	95.06
40	95.66	81.27	115.31
42	119.67	108.75	171.99
44	151.95	147.74	261.60
45	172.28	196.22	363.34

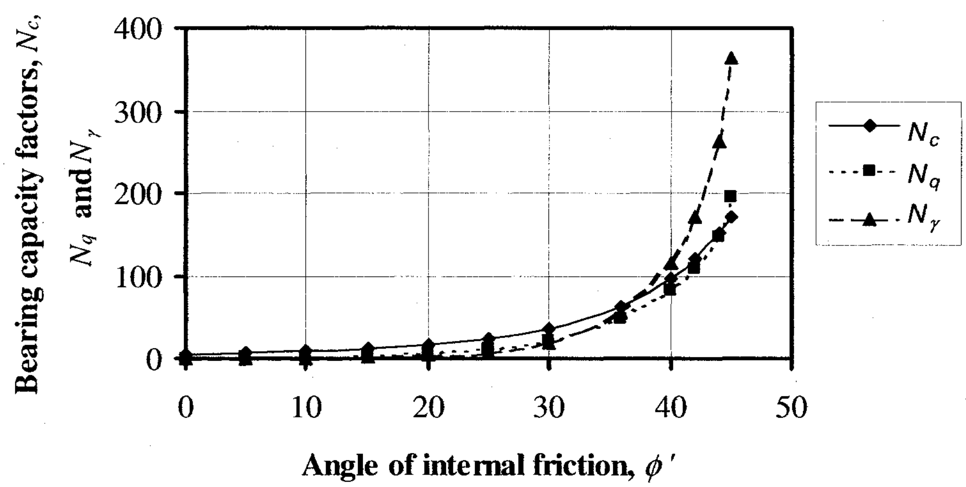


Figure 5. 3 Variation of the bearing capacity factors with respect to angle of internal friction, ϕ' (Note: N_c and N_q values are proposed by Terzaghi (1943) and N_γ values are proposed by Kumbhojkar (1993))

The bearing capacity factors were also plotted in Figure 5.3 to demonstrate how they vary with the angle of internal friction, ϕ'. This figure shows the bearing capacity factors, N_c , N_q

and N_γ rapidly increase for ϕ' values greater than 30°. For this reason, the internal friction angle, ϕ'should be determined carefully for the reliable determination of the bearing capacity of soils. This is particularly important for coarse-grained soils as they have angle of internal friction, ϕ' values typically higher than 30°. In addition, various other parameters such as the dilatancy, applied normal stress also influence the internal friction, ϕ'. More discussions are offered in a later section in this chapter about the various factors that influence the parameter internal friction, ϕ' and its impact on computing the bearing capacity of soils.

5.4 Bearing Capacity of Unsaturated Soils – Early Studies

Several investigators reported that matric suction has a significant influence on the bearing capacity of unsaturated soils (Broms 1964, Steensen-Bach et al. 1987, Fredlund and Rahardjo 1993, Oloo 1994, Schnaid et al. 1995, Oloo et al. 1997 and Costa et al. 2003). Experimental studies undertaken in this research program (see Chapter 4) also show that the bearing capacity of compacted, unsaturated coarse-grained soil increases non-linearly with respect to matric suction. The bearing capacity of the compacted sand tested was found to be 5 to 7 times higher than the bearing capacity of the same soil under saturated condition. These results are consistent with the findings of Steensen-Bach et al. (1987).

Oloo (1994) proposed a theoretical solution that can be used for interpreting the bearing capacity of unsaturated soils and tested it on two fine-grained soils (i.e., Botkin silt and glacial till). Two different scenarios were considered in this equation. In the first scenario, the matric suction is considered to be uniform below the footing. In the second scenario, the equation considers the variation of the matric suction under the footing as hydrostatic above the ground water table.

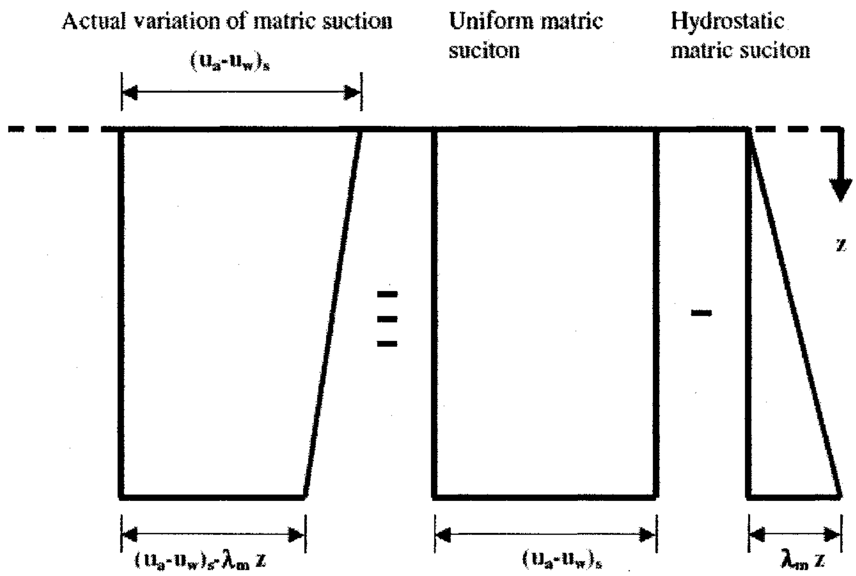


Figure 5. 4 An Idealized variation of matric suction in an unsaturated soil with depth (Oloo 1994)

5.4.1 Uniform Matric Suction Profile

Oloo (1994) derived a general bearing capacity equation that can be used for computing the bearing capacity of unsaturated soils for the two scenarios discussed in the earlier section as given below:

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - \tan\phi^b) + (u_a - u_w)_s \tan\phi^b]N_{c_o} + 0.5(\gamma - \lambda_m \frac{\tan\phi^b}{\tan\phi'})N_{\gamma_o} \qquad [5.8]$$

where:

λ_m = rate of decrease of suction with depth

$N_{c_o}, N_{q_o}, N_{\gamma_o}$ = bearing capacity factors proposed by Oloo (1994)

$(u_a - u_w)_b$ = matric suction at the air entry value, *kPa*

$(u_a - u_w)_s$ = matric suction at the surface, *kPa*

For the case of uniform matric suction throughout the depth of the stress bulb, the rate of decrease of matric suction can be assumed to be zero (Figure 5.4). Equation [5.8] will take the following form for computing the bearing capacity assuming uniform matric suction:

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - \tan\phi^b) + \gamma_w h_w \tan\phi^b] N_{c_o} + 0.5 B\gamma N_{\gamma_o} \quad [5.9]$$

If the matric suction is assumed to be uniform within a layer of an unsaturated soil and the air entry value, $(u_a - u_w)_b$ is close to zero, the bearing capacity Equation [5.10] can be simplified as given below:

$$q_u = [c' + (u_a - u_w)\tan\phi^b] N_{c_o} + 0.5 B\gamma N_{\gamma_o} \quad [5.10]$$

Oloo et al. (1997) used Equation [5.10] interpreting the bearing capacity of soils and pavements.

5.4.2 Hydrostatic Matric Suction Profile

Oloo (1994) also proposed slightly different equation for computing the bearing capacity of unsaturated soils assuming a linear decrease in variation of matric suction with respect to depth as shown in Figure 5.5 below.

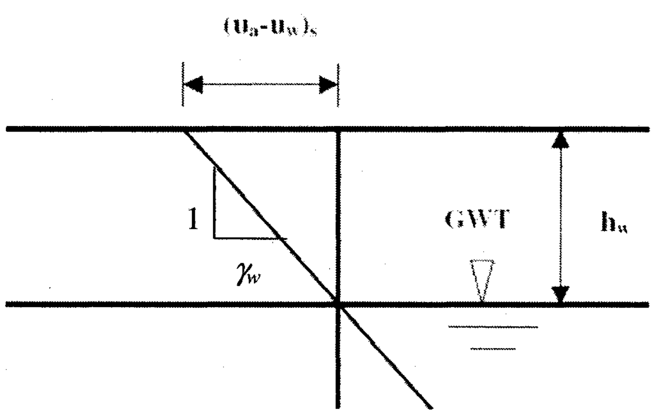


Figure 5. 5 Hydrostatic variation of matric suction with respect to depth above the ground water table

Equation [5.8] takes the form as Equation [5.11] if the negative pore water pressures in the unsaturated soils were assumed to be in equilibrium with the position of the static water table (i.e., hydrostatic matric suction profile as shown in Figure 5.5).

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - \tan\phi^b) + \gamma_w h_w \tan\phi^b]N_{c_o} + 0.5(\gamma - \gamma_w \frac{\tan\phi^b}{\tan\phi'})N_{\gamma_o} \quad [5.11]$$

Detailed derivations for the bearing capacity of unsaturated soils as provided by Oloo (1994) are summarized separately in Appendix.

5.4.3 Oloo (1994) Bearing Capacity Factors

Oloo (1994) proposed new bearing capacity factors and outlined calculations for the bearing capacity factors, N_{c_o} and N_{γ_o} for a 2-layered system. The objective of that study was to check whether there is a difference between the bearing capacity factors of 2-layered soil and one layer soil (i.e., homogenous soil). However, the solution developed for 2-layered soils were found to be applicable for homogenous soils as well.

The bearing capacity factor, N_{c_o} proposed by Oloo (1994) for homogeneous soils is approximately about 20% lower than bearing capacity factors proposed by Terzaghi (1943). Unlike the factor N_{c_o}, the bearing capacity factor due to self weight, $N_{\gamma o}$ has no closed form solution. The proposed solution by Oloo (1994) for the bearing capacity factor, N_{γ_o} was 10 to 40 % higher than the bearing capacity factors proposed by Terzaghi (1943). The bearing capacity factor, N_{q_o} was calculated as $(1+N_{co} \tan\phi')$. The bearing capacity factors proposed by Oloo (1994) are summarized in Table 5.2.

Table 5. 2 Bearing capacity factors proposed by Oloo (1994)

ϕ'	N_{c_o}	N_{q_o}	N_{γ_o}
0	5.15	1	0.0
10	8.34	2.47	1.2
20	14.8	6.4	5.9
30	30.1	18.4	26.7
35	55	40	60.4
40	75.3	64	147
44	100	115	220

Oloo (1994) used Equation [5.10] for interpreting the bearing capacity values using model footing tests on two fine-grained soils (i.e., Botkin Pit silt and glacial till) considering the shape factors proposed by De Beer (1970). The De Beer's shape factors are summarized below:

$$s_c = 1 + \frac{N_{q_o}}{N_{c_o}} \qquad [5.12]$$

$$s_\gamma = 0.6 \qquad [5.13]$$

5.4.4 Limitations of Bearing Capacity Equations Proposed by Oloo (1994)

The shear strength contribution due to matric suction, $\tan \phi^b$ was suggested to be considered as a constant value equal to $0.5 \tan \phi'$ in the interpretion of the bearing capacity of unsaturated soils for simplification purposes (see Equation [5.8]). If this equation is used for interpreting the bearing capacity of unsaturated soils, the variation of bearing capacity with matric suction will be bilinear in nature. The bearing capacity of an unsaturated soil increases almost linearly with matric suction up to air-entry value and beyond the air-entry value the bearing capacity increases linearly with a relatively flatter slope.

Oloo et al. (1997) further simplified Equation [5.8] to take the form as Equation [5.10] assuming matric suction to be uniform within a layer of an unsaturated soil and the air entry value, $(u_a - u_w)_b$ is close to zero. The bearing capacity of an unsaturated soil increases linearly with matric suction if this equation used. Equation [5.10] suggests that the soil desaturates immediately after application of even small values of suction. Such an assumption is valid more for coarse-grained soils in comparison to fine-grained soils. This assumption is simple for practical purposes, however, it is not valid as several studies related to unsaturated soils have shown that the engineering properties vary in a non-linear fashion with the matric suction. (Fredlund et al. 1987, Vanapalli et al. 1996). The experimental studies conducted in this thesis with respect to the bearing capacity of unsaturated soils also support this behavior.

In this research program, comparisons between the predicted bearing capacity of unsaturated soils using Oloo et al. (1997) equation (i.e., Equation [5.10]) and the proposed semi-empirical equation are presented for five soils in later sections of this chapter.

5.5 A Semi-empirical Equation for Predicting the Bearing Capacity of Unsaturated Soils

A semi-empirical equation for predicting the bearing capacity of soils in terms of saturated shear strength parameters was provided by Terzaghi (1943) based on Prandtl (1921) failure mechanism for a strip footing as given below.

$$q_u = c'N_c + \gamma D N_q + 0.5\gamma B N_\gamma \quad [5.14]$$

The contribution of surcharge, N_q can be neglected for estimating the bearing capacity of shallow foundations placed directly on the soil surface. In other words, Equation [5.14] takes the form:

$$q_u = c'N_c + 0.5\gamma B N_\gamma \quad [5.15]$$

Equation [5.15] can be written as given below by introducing Vesic (1973) shape factors:

$$q_u = c' N_c \xi_c + 0.5 \gamma B N_\gamma \xi_\gamma \quad [5.16]$$

The shear strength contribution due to suction can be added to the effective cohesion term to determine the bearing capacity of unsaturated soils extending the philosophy presented by Oloo (1994) (see Equation [5.9]).

$$q_u = [c' + (u_a - u_w)_b (\tan\phi' - \tan\phi^b) + (u_a - u_w)\tan\phi^b] N_c \xi_c + 0.5 \gamma B N\gamma \xi_\gamma \quad [5.17]$$

Equation [5.17] can be used for interpreting the bearing capacity of unsaturated soils if all the data is available. This equation can also be modified for predicting the bearing capacity of unsaturated soils extending the philosophy proposed by Vanapalli et al. (1996) for predicting the shear strength of unsaturated soils.

Vanapalli et al. (1996) proposed a semi-empirical function for estimating the internal friction angle with respect to matric suction, tan ϕ^b using the SWRC and the saturated shear strength parameters as $\tan\phi^b = S^\kappa \tan\phi'$, where S = degree of saturation value from the SWRC. This form of the equation takes account of the non-linear variation of the shear strength of unsaturated soils. Extending these concepts, Equation [5.16] can be modified to predict the bearing capacity of unsaturated soils as given below:

$$q_u = [c' + (u_a - u_w)_b (\tan\phi' - S^\psi \tan\phi') + (u_a - u_w) S^\psi \tan\phi'] N_c \xi_c + 0.5 \gamma B N\gamma \xi_\gamma \quad [5.18]$$

Equation [5.18] can be used for providing comparisons between the measured and predicted bearing capacity values using a fitting parameter, ψ. The parameter, ψ is defined hereafter as a bearing capacity fitting parameter. In this equation, the bearing capacity contribution due to matric suction can be obtained from a part of Equation [5.18], which is equal to $(u_a - u_w) S^\psi \tan\phi'$ beyond the air-entry value, $(u_a - u_w)_b$. Up to the air-entry value, $(u_a - u_w)_b$, the contribution to bearing capacity will be equal to $(u_a - u_w)_b (\tan\phi' - S^\psi \tan\phi')$. This technique is similar to the predicting the shear strength contribution due to matric suction, which is $[(u_a - u_w)S^{\kappa} \tan\phi']$ from Equation [5.2].

The philosophy of using the fitting parameter, ψ in the bearing capacity of unsaturated soils is similar to using the fitting parameter, κ for predicting the shear strength of unsaturated soils. The limitation of using Equation [5.17] for predicting the bearing capacity of unsaturated soils is similar to using Equation [5.2] for determining the shear strength of unsaturated soils. In other words, Equation [5.18] can only be used for predicting the bearing capacity when the experimental results are available. To alleviate this limitation for the shear strength of unsaturated soils, Vanapalli and Fredlund (2000) provided a relationship between κ versus plasticity index, I_p for predicting the shear strength of unsaturated soils. Based on more recent studies Garven and Vanapalli (2006) provided a relationship as given below:

$$\kappa = 1 + 0.0975 I_p - 0.0016 I_p^{\,2} \qquad [5.19]$$

This relationship was shown in Figure 5.1. Using this relationship and Equation [5.2], shear strength of unsaturated soils can be predicted.

In the present study, the same philosophy has been extended to propose a relationship between the bearing capacity fitting parameter, ψ versus plasticity index, I_P such that the Equation [5.18] can be used for predicting the bearing capacity of unsaturated soils using the saturated shear strength parameters, c' and ϕ' and the SWRC. More details of the ψ versus plasticity index, I_P relationship are available in a later section.

Equation [5.18] will take the form as given below for predicting the bearing capacity if a square model footing is used:

$$q_u = 1.3\,[c' + (u_a - u_w)_b(\tan\phi' - S^{\psi}\tan\phi') + (u_a - u_w)_{AVR}\,S^{\psi}\tan\phi']N_c\xi_c + 0.4B\gamma N_\gamma\,\xi_\gamma$$

[5.20]

where:

S	= degree of saturation
ψ	= fitting parameter in bearing capacity equation
$(u_a - u_w)_{AVR}$	= average matric suction in the depth of the stress bulb, *kPa*
N_c	= bearing capacity factor due to cohesion (proposed by Terzaghi 1948)
N_γ	= bearing capacity due to unit weight (proposed by Kombojkar 1993)
ξ_c	= shape factor due to cohesion (proposed by Vesic 1973)
ξ_γ	= shape factor due to unit weight (proposed by Vesic 1973)

The following assumptions are used for the proposed bearing capacity equation:

- The soil is isotropic and homogeneous in nature.
- The ultimate bearing capacity is the maximum load per area at failure for dense soils where a well-defined failure load can be determined from the stress-settlement curve (i.e., GSF). Local shear failure conditions are assumed when it was difficult to define failure load from the stress-settlement curve extending Terzaghi (1943) approach.
- The proposed equation is valid for vertical static loads.
- The non-linear variation of matric suction with respect to depth is assumed to be linear.
- The matric suction value (i.e., subsituted in Equation [5.20]) was taken as the average suction values in the proximity of the stress bulb (more details are provided for calculating the average suction value in a later section).

5.6 Differences in the Proposed Bearing Capacity Approach and Oloo (1994) Equation

Equation [5.18] proposed in this research is similar in form to Equation [5.9] proposed by Oloo et al. (1997). Equation [5.18] can be used for predicting the variation of the bearing capacity using the shear strength parameters, c' and ϕ' and the SWRC. However, Equation [5.9] cannot be used for predicting the variation of the bearing capacity of unsaturated soils with respect to matric suction. The other key differences are also summarized below:

- The shape factors, ξ_c , ξ_q and ξ_γ proposed by Vesic (1973) are used in the proposed equation. Oloo (1994) used the shape factors proposed by De Beer (1970)
- The bearing capacity factors, N_c , N_q were proposed by Terzaghi (1943) and N_γ proposed by Kumbhojkar (1993) were used in the proposed equation. However, Oloo (1994) proposes a new set of bearing capacity factors.
- Oloo (1994) simplifies the form of Equation [5.9] and suggest using Equation [5.10]. The influence of air entry value, $(u_a - u_w)_b$ is neglected in this equation.
- A bearing capacity fitting parameter, ψ is required for providing a best-fit between the measured and estimated values of the bearing capacity of soils using the proposed equation (Equation [5.18]). A simple relationship is later presented in this thesis to estimate the bearing capacity fitting parameter, ψ from plasticity index, I_P value of the soil. These details are provided later after the analysis of the test results.

5.7 Details for Using the Proposed Bearing Capacity Approach

This section provides details about how the matric suction profile below the footing can be taken into account and also show details of how the average matric suction value can be calculated used in the proposed equation for predicting the bearing capacity of unsaturated soils.

5.7.1 Procedure for Calculating Average Matric Suction Value

The variation of matric suction with respect to depth below the footing was non-linear in nature from the experimental results. However, matric suction variation was assumed to be linear with respect to depth in the present analysis. This assumption is detailed with typical experimental results of the present research program (refer to chapter 4 Figure 4.11). The suction was taken as an average value in the depth zone of the stress bulb, which typically extends over the depth of 1.5 times the width of the foundation from the base of the foundation. This assumption is based on Poulos et al. (1974) and Chen (1999) studies, which show the stress bulb extends typically to a depth of 1.5 times the width of the foundation. The average suction value in the proposed semi-empirical Equation [5.20] can be calculated using the relationship below:

$$(u_a - u_w)_{AVR} = \frac{[(u_a - u_w)_1 + (u_a - u_w)_2]}{2} \qquad [5.21]$$

where:

$(u_a - u_w)_{AVR}$ = average matric suction in the depth of the stress bulb, *kPa*

$(u_a - u_w)_1$ = the measured matric suction at the surface of the soil, *kPa*

$(u_a - u_w)_2$ = the measured matric suction at a depth of 1.5(B), *kPa*

Equation [5.20] will take the form as given below for interpreting the experimental results of the square model footing by including Vesić (1973) shape factors:

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - S^{\psi}\tan\phi') + (u_a - u_w)_{AVR}\, S^{\psi}\tan\phi']N_c\left[1.0 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right)\right] + 0.5B\gamma N_\gamma\left[1.0 - 0.4\left(\frac{B}{L}\right)\right] \quad [5.22]$$

Equation [5.22] will take the conventional equation form for calculating the bearing capacity of soils in a state of saturated condition when the matric suction, $(u_a - u_w)$ value is set to zero.

5.7.2 Summary of the Measured Matric Suction Data from the Experimental Program

As detailed in Chapter 4, the ground water table was located at different depths below the footing such that there is a non-linear variation of matric suction with respect to the depth above the ground water table in zone of stress bulb (i.e., $D = 1.5B$). This non-linear variation of matric suction in the depth zone can be attributed to the capillary stresses above the ground water table. Such a condition is a typical scenario in engineering practice where the surface suction values are much higher in comparison to lower depths.

Three different average matric suction values of 2 kPa, 4 kPa and 6 kPa were simulated in the test tank by varying the ground water table below the footing. For achieving an average matric suction value of 2 kPa matric suction, the ground water table was located at a depth of 250 mm from the soil surface in the test tank as shown in Figure 5.7. Similarly, an average matric suction of 4 kPa and 6 kPa were achieved by varying the ground water table location to depths of 400 mm and 600 mm respectively. Figures 5.8 and 5.9 provide the details as a schematic for both these cases.

The hydrostatic variation of matric suction (i.e., $\gamma_w h_w$) values were also calculated and presented below along with the non-linear distribution of the measured matric suction for the tested compacted coarse-grained soil under the three different conditions (i.e., 2 kPa, 4 kPa and 6 kPa). Typically, in sands capillary stresses are linear in nature up to the air-entry value of the soil and later show non-linear nature. They provide conservative estimate of increase of suction above the ground water table for the present study. This information is also useful to indirectly check how the tensiometers used in the study are performing. The results of the study shows that the tensiometers functioned well and measured readings are reliable.

The data of unit weight, degree of saturation and water contents were directly collected from the test tank for the three cases (i.e., for average suction values of 2, 4, and 6 kPa) and summarized in Table 5.3, Table 5.4 and Table 5.5 respectively.

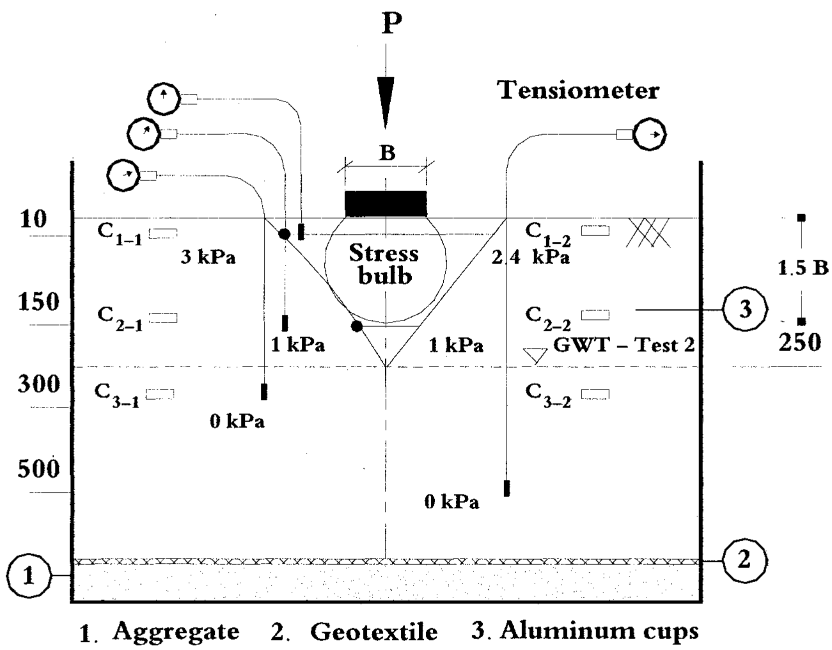


Figure 5. 6 Variation of measured matric suction with respect to depth along with hydrostatic distribution for an average suction of 2 kPa in the stress bulb zone

The suction range considered in the study (i.e., 0 to 6 kPa) represents the degree of saturation from 100 to 58% (see Tables 5.3 thru 5.5).

Table 5. 3 Typical data from the test tank for an average suction value of 2 kPa in the stress bulb zone

D[1] (mm)	γ_t (kN/m³)	γ_d (kN/m³)	e	w (%)	S (%)	AVR[1] $(u_a - u_w)$ (kPa)	Estimated[2] $(u_a - u_w)$
10	19.55	15.93	0.64	22	89	2	2.4
150	19.72	15.99	0.625	23.5	97	1	1.0
300	19.74	15.95	0.63	23.8	100	0	0

[1] AVR: Average value of measured matric suction.

[2] Estimated $(u_a - u_w)$: This value of capillary stress (i.e., matric suction) was estimated assuming hydrostatic variation above ground water table conditions.

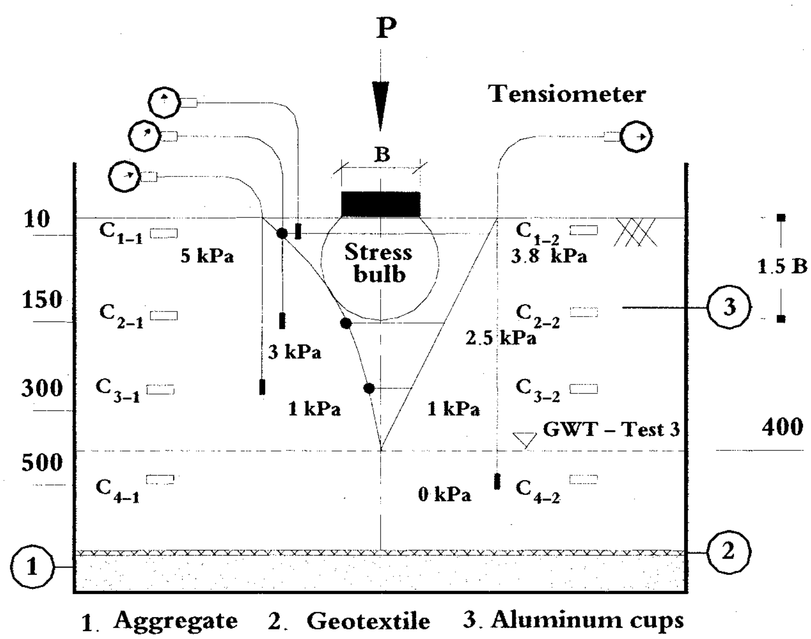


Figure 5. 7 Variation of measured matric suction with respect to depth along with hydrostatic distribution for an average suction of 4 kPa in the stress bulb zone

Table 5. 4 Typical data from the test tank for an average suction value of 4 kPa in the stress bulb zone

D[1] (mm)	γ_t (kN/m³)	γ_d (kN/m³)	e	w (%)	S (%)	AVR[1] $(u_a - u_w)$ (kPa)	Estimated [2] $(u_a - u_w)$
10	19.08	16.01	0.62	19.2	81	4	3.8
150	19.54	15.98	0.623	22.2	86	2	2.5
300	19.72	15.99	0.625	23.3	98.7	1	1.0
500	19.74	15.95	0.63	23.8	100	0	0

[1] AVR: Average value of measured matric suction.

[2] Estimated $(u_a - u_w)$: This value of capillary suction (i.e., matric suction) was estimated assuming hydrostatic variation above ground water table conditions.

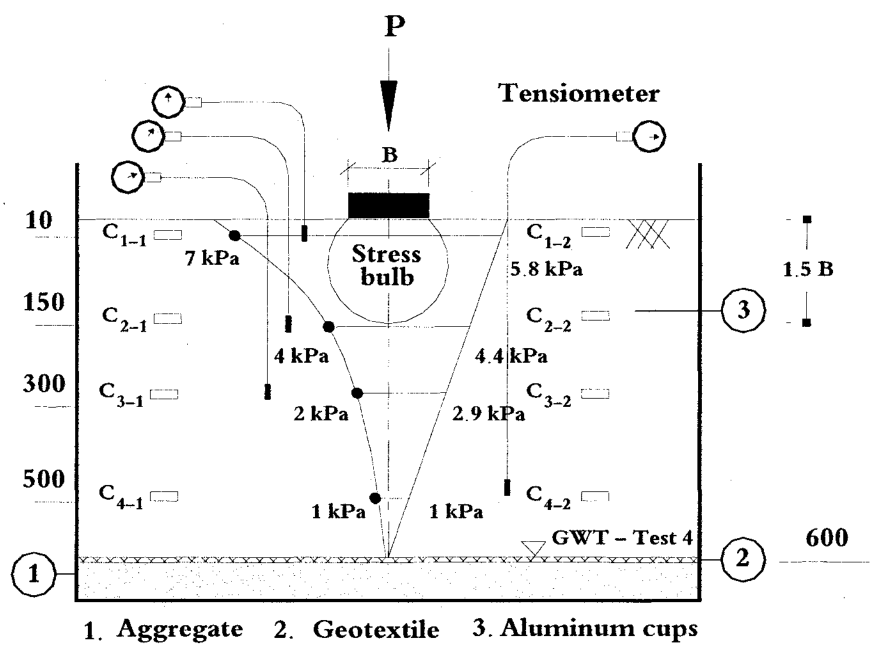


Figure 5. 8 Variation of measured matric suction with respect to depth along with hydrostatic distribution for an average suction of 6 kPa in the stress bulb zone

Table 5. 5 Typical data from the test tank for an average suction value of 6 kPa in the stress bulb zone

D[1] (mm)	γ_t (kN/m³)	γ_d (kN/m³)	e	w (%)	S (%)	AVR[1] $(u_a - u_w)$ (kPa)	Estimated[2] $(u_a - u_w)$
10	18.17	15.94	0.63	14.0	58	6	5.8
150	18.75	15.85	0.64	18.3	76	4	4.4
300	19.27	16.07	0.62	20.0	86	2	2.9
500	19.40	15.77	0.64	23.0	94	1	1.0
600	19.74	15.95	0.63	23.8	100	0	0

[1] AVR: Average value of measured matric suction.

[2] Estimated $(u_a - u_w)$: This value of capillary suction (i.e., matric suction) was estimated assuming hydrostatic variation above ground water table conditions.

5.7.3 Appropriate Saturated Shear Strength Parameters for Use in the Bearing Capacity Analysis

Several studies show that the computed bearing capacity of soils using Terzaghi (1943) and Meyerhof (1953) equation provides lower estimates than the measured values (Steensen-Bach et al. 1987, Kumbhokar 1993). Some investigators have found that better comparisons can be provided between the measured and computed bearing capacity values by using ϕ' values about 10 to 15% higher than the measured values in the laboratory (Steensen-Bach et al. 1987). This may be attributed to several factors that influence the measured angle of internal friction, ϕ value.

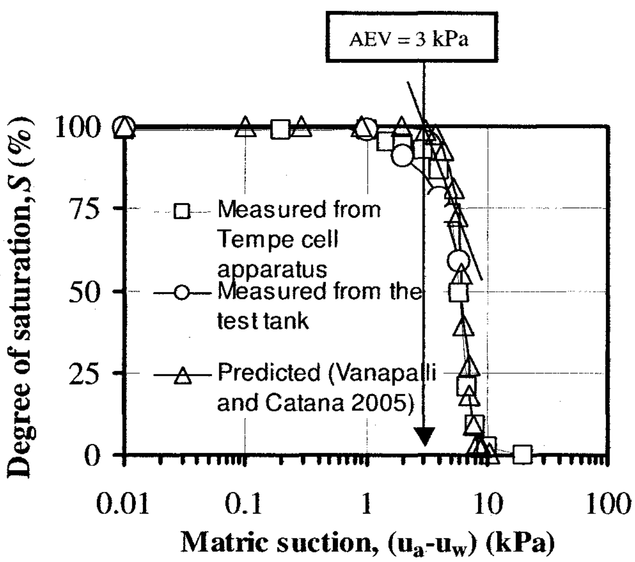
De Beer studies (1965) show that the angle of internal friction, ϕ' for a soil is higher when tested under low normal stresses or confining pressures. However, ϕ' decreases for the same soil when tested with the application of higher normal stresses. Bolton (1986) studies on sands show that the effective stress and soil density affect the rate of dilatancy of soils and thereby influence the internal friction angle, ϕ' value. The dilatancy of the soil has a significant effect on the internal friction angle and the value obtained from triaxial shear tests was suggested to be modified as $\phi' = (\phi' + 0.8$ of dilatancy angle). Studies by (Chen 1999) suggest that this is true for sandy soils with ϕ' values in the range of 35 to 45°.

Steensen-Bach et al. (1987) suggested using higher values of ϕ' values in the computation of the bearing capacity of soil because the bearing capacity theory is based on plane strain calculations coupled with empirical corrections. The plane strain angle of friction value was suggested to be modified to $\phi_{pl} = 1.1\ \phi_{triaxial}$. Steensen-Bach et al. (1987) increased their measured internal friction angle, ϕ' for the Sollerod sand soil studied by 10 % (refer to Table 5.6 and Table 5.7 below). This increase was recommended for soils with typically with friction angle, ϕ' values less than 40°. For the Lund sand used in their study, there was no need to increase ϕ' value by 10 % due to the fact the value of ϕ' was 44° and good agreement was found between the predicted and measured bearing capacity values using the measured ϕ' values from the laboratory. The soils studied by Steensen-Bach et al. (1987) were reanalyzed in this study using the proposed bearing capacity equation and Oloo's (1994) equation.

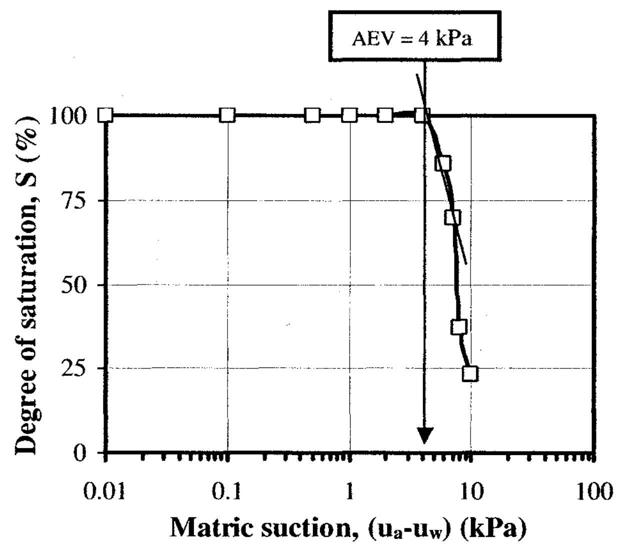
The shear strength parameters, c' and ϕ' were determined under two different sets of normal stresses (both low and high) for the soil tested in this research program (see Chapter 4 for more details) using direct shear apparatus. The measured internal friction angle, $\phi'$was equal to 32.6° (under high normal stresses) and 35.3° (under low normal stresses). The effective cohesion was 2.6 kPa under high normal stresses and equal to zero under the application of low normal stresses. Reasonably good comparisons were obtained between the measured bearing capacity of the soil for saturated conditions from model footing tests using proposed bearing capacity equation and Terzaghi's bearing capacity theory (Equation [5.3]) using shear strength parameters c' and ϕ' values of 0.6 kPa and 39 ° respectively. The internal friction angle, ϕ' value of 39 ° was10 % higher than the measured internal friction angle, ϕ' value 35.3°.

5.7.4 Air Entry Values of the Soils from the SWRC

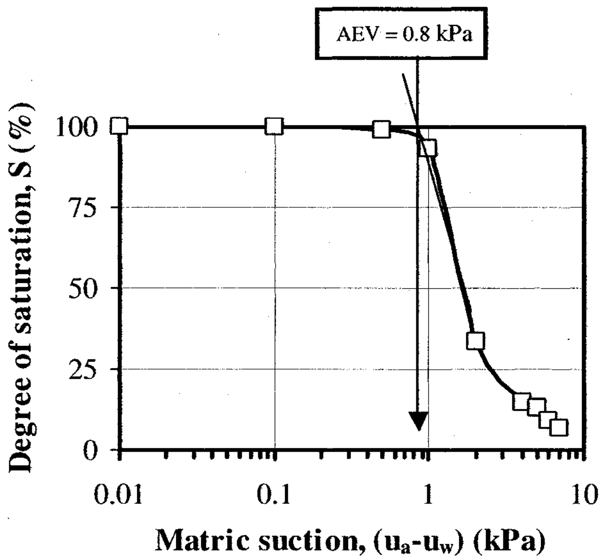
The soil-water retention curves (SWRC) for the five different soils are shown in Figure 5.10. The SWRC was plotted as a relationship between the degree of saturation and the matric suction using a semi-logarithmic plot. The air-entry value (AEV) for each soil is estimated as the intersection between the tangent from the boundary zone and the tangent from the transition zone as detailed in Vanapalli et al. (1999). For the three coarse-grained soils (i.e., Soil A, Soil B and Soil C), the air entry values were in the range from 0.8 kPa to 4 kPa. The air entry values for the two fine-grained soils were 35 kPa and 40 kPa for Botkin Pit silt (i.e., Soil D) and glacial till (i.e., Soil E) respectively. This implies that the finer the soil the higher is the air-entry value which is consistent with the results of other studies. Also, it should be noted that the degree of saturation drops down rapidly to values lower than 10% values for the coarse-grained soils when the matric suction in the specimens reaches around 10 kPa.



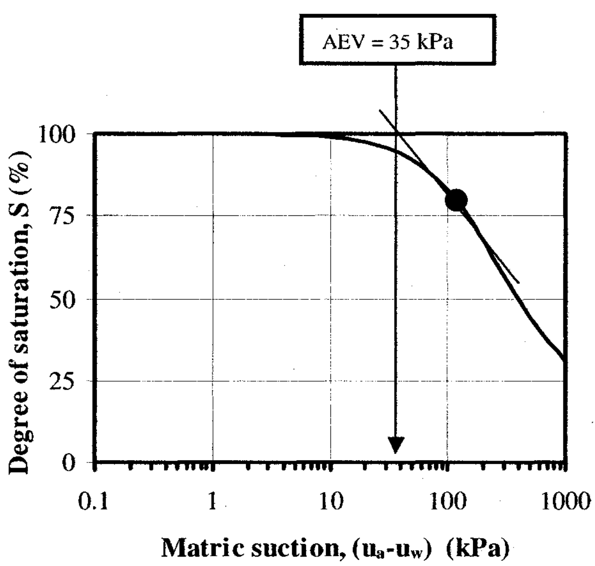
(a) SWRC for the compacted sand (Soil A)



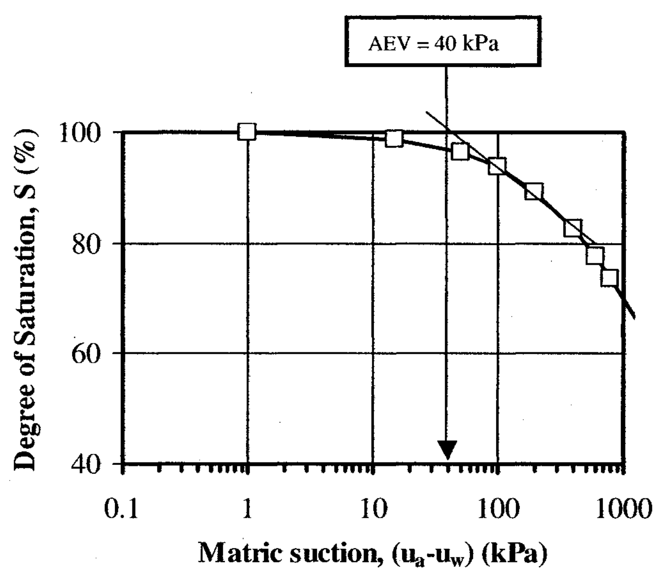
(b) SWRC for the Sollerod sand (Soil B from Steensen-Bach et al. 1987)



(c) SWRC for the Lund sand (Soil B from Steensen-Bach et al. 1987)



(d) Predicted SWRC of the Botkin Pit silt



(e) Measured SWRC for Till (Vanapalli et al. 1999)

Figure 5. 9 Air entry values from SWRCs of the five soils studied

Except of the Botkin Pit silt, the SWRC measured for all the other soils. The SWRC for the Botkin Pit silt was predicted using one point method following the procedures detailed in Catana and Vanapalli (2006). More details about this procedure are detailed in a later section.

5.8 Comparisons between the Measured and Predicted Bearing Capacity of Soils from Model Footing Tests

Comparisons are provided between the measured and predicted bearing capacity values using Equation [5.22] for five different soils including the compacted sand studied in this research program (i.e., Sandy soil, A). The other four soils results are summarized from the literature (Soil B thru E).

The bearing capacity for all the five soils were determined using square model footings tests with sizes in the range 20 mm to 150 mm. In addition to the proposed equation (Equation [5.22]), Oloo et al. (1997) equation (i.e. Equation [5.10]) was also used for interpreting the bearing capacity of all the five soils.

The conventional properties of the soils are summarized in Table 5.6. Summary of all the data required for predicting the variation of the bearing capacity of the soils with respect to matric suction using the proposed equation (i.e., Equation [5.22]) is summarized in Table 5.7. Table 5.8 summarizes the data for computing the bearing capacity using the equation proposed by Oloo (1994) (i.e., Equation [5.10]).

The failure mode for all the analyzed five soils including the tested soil was defined based on the behaviour of the actual stress versus settlement relationships. The general shear failure (GSF) mode of failure was used for dense soils (Soil A, B and C). The local shear failure (LSF) criterion was used for the soils D and E, which were fine-grained soils (i.e., Botkin Pit silt and glacial till). The angle of internal friction angle $\tan \phi^* = 0.67 \tan \phi'$ was used for LSF conditions following the approximation proposed by Terzaghi (1943) and Terzaghi and Peck (1948).

Table 5. 6 Basic properties and description of the different soils

PROPERTY OR PARAMETER	**Soil (A)**	**Soil (B)**	**Soil (C)**	**Soil (D)**	**Soil (E)**
Specific gravity, G_s	2.65	2.65	2.65	2.68	2.73
Sieve analysis results					
D_{10} *(mm)*	0.12	0.08	0.3	0.002	-
D_{30} *(mm)*	0.18	0.11	0.4	0.015	0.002
D_{60} *(mm)*	0.22	0.14	0.55	0.019	0.02
Coefficient of uniformity, C_u	1.83	2.04	1.52	9	80
Coefficient of curvature, C_c	1.23	1.08	0.97	5.92	1.8
Sand, (%)	95	96	100	52.5	28
Silt, (%)	5	4	0	37.5	42
Clay, (%)	0	0	0	10	30
Atterberg limits					
Liquid limit, w_L	-	-	-	22	36
Plastic limit, w_P	-	-	-	14	17
Plasticity index, I_P	-	-	-	8	19
Void ratio, *e*	0.62	0.58	0.587	0.502	0.575
Dry unit weight, (kN/m^3)	16.05	16.46	16.37	17.5	17
Cohesion, *c'* (kPa)	0.6	0.8	0.5	2.5	11
Measured Friction angle, ϕ' (deg.)	35.3	35.78	44	28	22
Unified Soil Classification System, *USCS*	Coarse-grained Sand SP	Sollerod Sand SP	Lund Sand SP	Botkin Pit Silt CL-ML	Glacial Till CL
Reference	Present study (2006)	Steensen-Bach et al. (1987)	Steensen-Bach et al. (1987)	Oloo (1994)	Oloo (1994)

Table 5. 7 Summary of the data for predicting the bearing capacity of the different soils using the proposed Equation [5.22]

Property or parameter	**Soil (A)**	**Soil (B)**	**Soil (C)**	**Soil (D)**	**Soil (E)**
B (m)	0.1 and 0.15	0.022	0.022	0.03	0.03
L (m)	0.1 and 0.15	0.022	0.022	0.03	0.03
D (m)	0.0	0.0	0.0	0.0	0.0
Saturated unit weight, (kN/m^3)	20	20.05	20	20.78	20.58
Average total unit weight, (kN/m^3)	18.0	18.25	18.19	19.14	18.79
Dry unit weight, (kN/m^3)	16.05	17	16.5	17.5	17
Cohesion, c' (kPa)	0.6	0.8	0.5	1.7	11.32
Measured friction angle, ϕ' (deg.)	35.3	35.78	44	28	21.9
Modified friction angle, ϕ' (deg.)	39 [1]	39.4 [1]	-	20 [2]	15 [2]
ψ	1.0	1.0	1.0	3.99	7.5
Air entry value, $(u_a - u_w)_b$ (kPa)	3	0.5	4	35	45
N_c [3]	85.97	90	151.95	17.69	12.86
N_q [3]	70.61	75	147.74	7.44	4.45
N_γ [4]	95.0	104	261.60	3.64	1.52
ξ_c [5]	1.820	1.833	1.972	1.420	1.346
ξ_γ [5]	0.6	0.6	0.6	0.6	0.6

[1] Friction angle was increased by 10 %

[2] Modified as $\tan \phi^* = 0.67 \tan \phi'$ (local shear failure) (Terzaghi 1943)

[3] Bearing capacity factors, N_c and N_q proposed by Terzaghi (1943) (Table 5.1)

[4] Bearing capacity factor, N_γ proposed by Kumbhojkar (1993) (Table 5.1)

[5] Shape factors proposed by Vesic (1973) (Eq. [5.5] & Eq. [5.7])

Table 5. 8 Summary of the data for calculating the bearing capacity of the different soils using the Equation [5.10] proposed by Oloo (1994) and Oloo et al. (1997)

Property or parameter	**Soil (A)**	**Soil (B)**	**Soil (C)**	**Soil (D)**	**Soil (E)**
B (m)	0.1 and 0.15	0.022	0.022	0.03	0.03
L (m)	0.1 and 0.15	0.022	0.022	0.03	0.03
D (m)	0.0	0.0	0.0	0.0	0.0
Saturated unit weight, (kN/m^3)	19.80	20.05	20	20.78	20.58
Average total unit weight, (kN/m^3)	18.0	18.25	18.19	19.14	18.79
Dry unit weight, (kN/m^3)	16.05	17	16.5	17.5	17
Cohesion, c' (kPa)	0.6	0.8	0.5	1.7	11.32
Measured friction angle, ϕ' (deg.)	35.3	35.78	44	28	21.9
Modified friction angle, ϕ' (deg.)	39 [1]	39.4 [1]	-	19.6 [2]	15.1 [2]
ϕ^b (deg.)	19.5 [3]	19.8 [3]	22 [3]	7.74 [4]	13.17 [4]
Air entry value, $(u_a - u_w)_b$ (kPa)	3	0.5	4	35	45
N_{c_o} [5]	60	62	100	14.6	11
N_{q_o} [5]	70	72	115	6.2	4.0
N_{γ_o} [5]	125	135	220	5.2	2.7
s_c [6]	2.167	2.16	2.15	1.425	1.364
s_γ [6]	0.6	0.6	0.6	0.6	0.6

[1] Friction angle was increased by 10 %

[2] Modified as $\tan \phi^* = 0.67 \tan \phi'$ (local shear failure)

[3] ϕ^b value considered was equal as 50 % of ϕ' (assumption proposed by Oloo 1994)

[4] Measured using modified direct shear test apparatus (from Oloo 1994)

[5] Bearing capacity factors, proposed by Oloo (1994) (Table 5.2)

[6] Shape factors proposed by De Beer (1970) (Eq. [5.12] & Eq. [5.13])

5.8.1 Compacted Sand, Soil (A)

The bearing capacity of soil A (i.e., compacted sand) was determined both under saturated and unsaturated conditions using two different sizes of model footings (i.e., 100 mm × 100 mm and 150 mm × 150 mm in size) (as in Chapter 4). A comparison between the predicted and measured bearing capacity for the tested compacted coarse-grained soil using the two model footings are presented in the following sections.

5.8.1.1 Soil (A) Results with Model Footing 100 mm × 100 mm

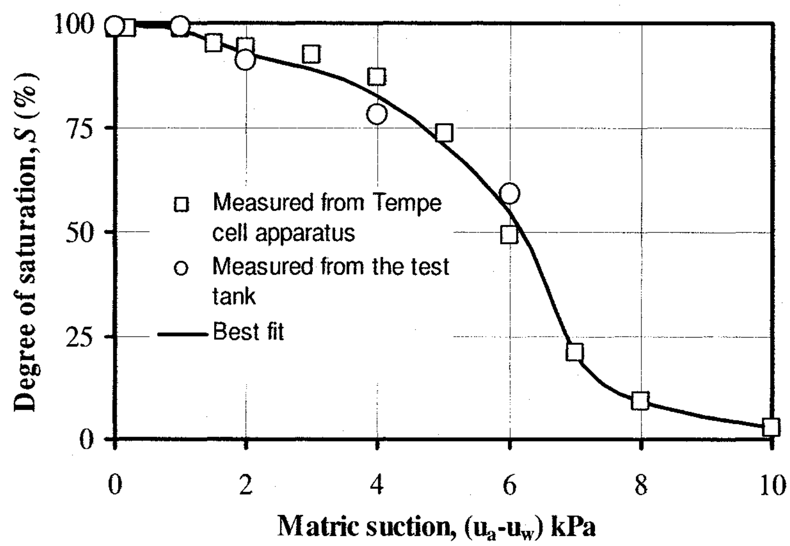
The predicted and measured SWRC for the tested soil (i.e., Soil A) is shown in Figure 5.10 (a). The variation of the predicted bearing capacity with respect to matric suction using Equation [5.22] is shown in Figure 5.10 (b). The computed bearing capacity using Oloo's approach (i.e, Equation [5.10]) for various values of matric suction values is also shown in Figure 5.10 (b). The data used for predicting and computing the bearing capacity using Equations [5.22] and [5.10] are summarized in Table 5.7 and Table 5.8 respectively. More detailed calculations using the two equations are available in Appendix A.

There is a good comparison between the measured bearing capacity and the computed bearing capacity of the soil under saturated conditions (i.e., matric suction value equal to zero) using both the equations (i.e., Equations [5.22] and [5.10]). However, the computed bearing capacity using Equation [5.10] varies linearly with respect to matric suction and is much lower than the measured bearing capacity. This due to the reason of assuming tan ϕ^b as a constant value was. At higher suction values (i.e., greater than 9 kPa) this equation computes higher values of bearing capacity than the Equation [5.22].

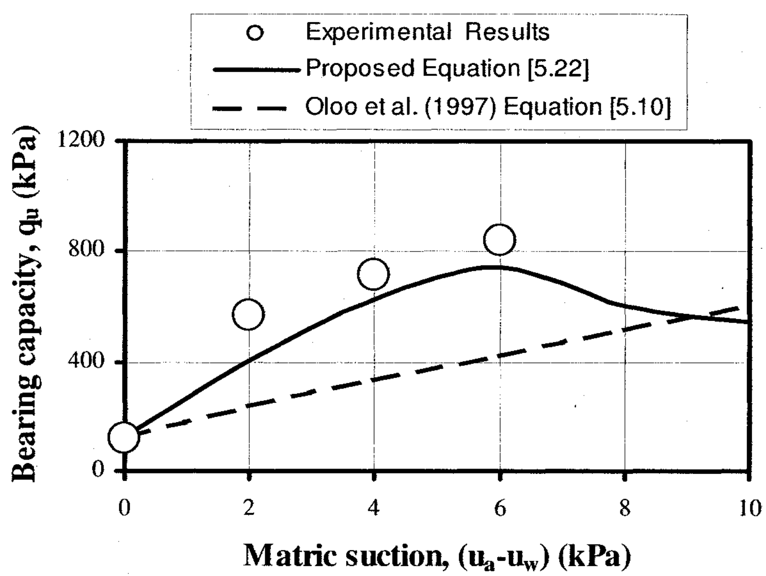
Equation [5.22] provides better estimates and is close to measured bearing capacity values for the tested range of matric suction. The predicted bearing capacity values are slightly lower than the measured bearing capacity values and may be attributed to lower average matric suction values calculated in the stress bulb. Typically, the stress bulb with higher intensities (i.e., greater than 80%) due to the applied load are distributed in the depth zone of 0.5 times the width of the footing (Poulos et al. 1974, Chen 1999). The matric suction close to surface is higher in comparison to the depths below 0.5 times the width of foundation. However, the average matric suction is calculated assuming the stresses to be distributed below a depth of 1.5 times the width of footing from the surface. This is assumption is likely to be conservative and provides slightly lower estimates than the measured bearing capacity values.

It is of interest to note that the measured and the predicted bearing capacity increases linearly up to the air-entry value of 3 kPa (see Figure 5.9 (a) and Table 5.7). The bearing capacity for suction values greater than 3 kPa increases non-linearly with decreasing rate of increase in the matric suction. The predicted bearing capacity starts dropping at higher suction values (i.e., 6 to 10 kPa). This behavior is consistent with the relationship between the SWRC and the shear strength behavior of unsaturated soils as discussed by Vanapalli et al. (1996). Theoretically, the bearing capacity should reduce at higher values of suction as the degree of saturation drops rapidly. As can be seen from Figure 5.10 (a) the degree of saturation is low for suction values greater than 7 kPa and suction cannot be transmitted effectively as the water contact area between the soil particles decreases predominantly in the transition and residual zones. This residual zone of occurs in the suction range of 7 to 10 kPa for the soil tested in this research program. It is likely at slightly higher values of suction (say 12 to 15 kPa), the bearing capacity of the soil may be similar to the bearing capacity of saturated soils as degree of saturation is close to zero (i.e., dry conditions). Terzaghi (1943) commented that the bearing capacity of a saturated soil is close to bearing capacity of a dry soil as the stresses distributed are essentially in two phase media (i.e., soil and water or soil and air). In other words, the effective stress is the same both under saturated and dry conditions. Thus, the proposed equation essentially captures all the salient features expected for the bearing capacity of soils from a fully saturated condition to total dry conditions.

The bearing capacity computations using Oloo's approach (Equation [5.10]) on the other hand increases linearly with an increase in the matric suction. This equation thus cannot capture all the essential features expected from the experimental behavior from a saturated condition to total dry conditions.



(a) SWRC for the tested compacted sand

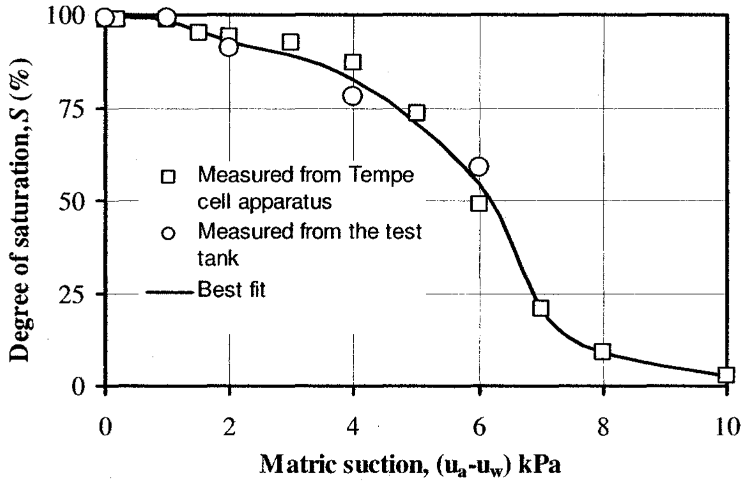


(b) Bearing capacity versus suction

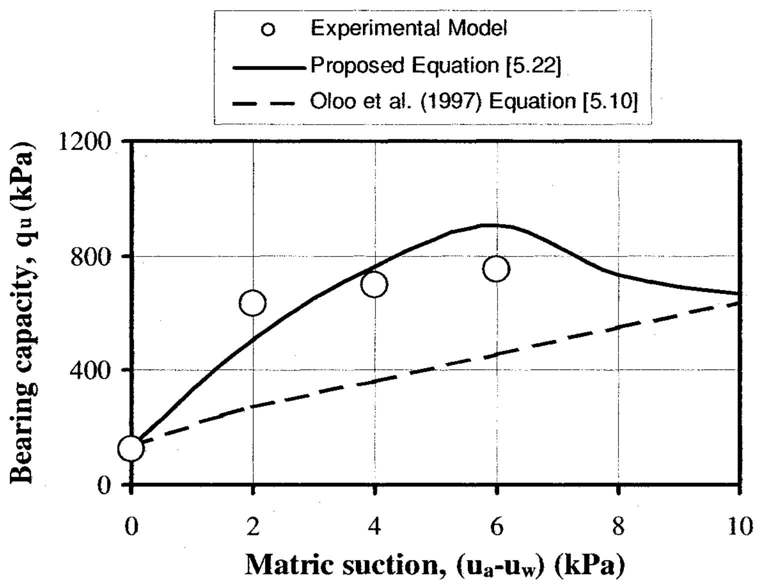
Figure 5. 10 Measured and predicted Bearing capacity for 100 mm × 100 mm footing

5.8.1.2 Soil (A) Results with Model Footing 150 mm × 150 mm

Comparisons between the predicted and measured bearing capacities using both the equations as discussed in the earlier section are provided for the experimental results on 150 mm x 150 mm model footings in Figure 5.12 (a) and Figure 5.12 (b).



(a) SWRC for the tested compacted sand



(b) Bearing capacity versus suction

Figure 5. 11 Measured and predicted Bearing capacity for 150 mm × 150 mm footing

All salient features discussed for the 100 mm × 100 mm model footing in the previous section can also be observed for the 150 mm × 150 mm footing.

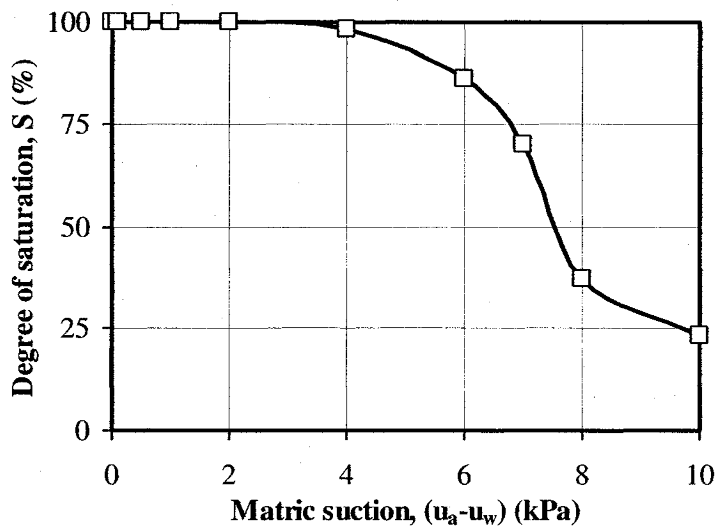
5.8.2 Soil (B) Sollerod Sand

A laboratory program was carried out by Steensen-Bach et al. (1987) using small scale plate load tests on Sollerod sand. The Sollerod sand was collected from field test site and is a glacial sedimentary deposit. The model footings were 22 mm × 22 mm in size with rough surfaces in a circular test pit with a diameter of 200 mm. The ground water table was located at different levels and different tests were carried out.

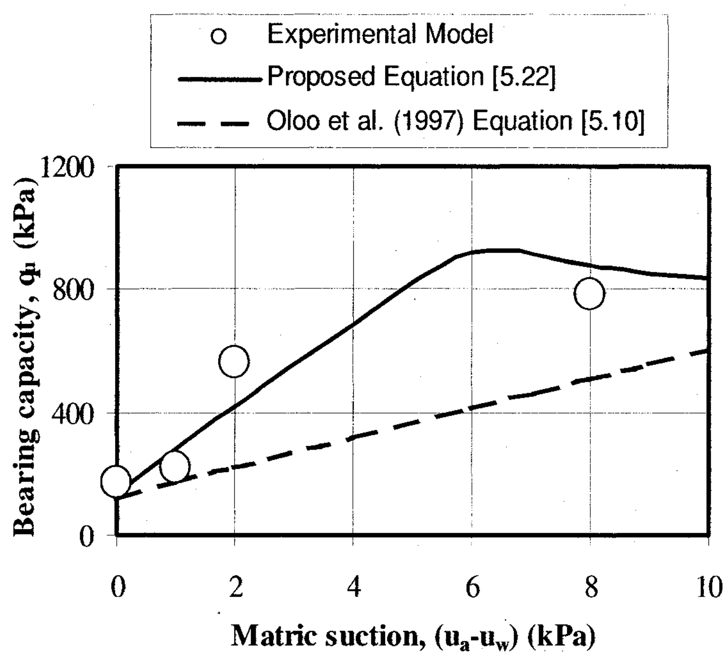
The capillary pressure curves (i.e., SWRC) established experimentally in the laboratory using the capillary rise phenomena are shown in Figure 5.12 (a). The air entry value of the sand was 4 kPa. All the parameters necessary for predicting and computing the bearing capacity of the soil using Equation [5.22] and Equation [5.10] are summarized in Table 5.7 and Table 5.8.

The bearing capacity of the Sollerod sand without suction (i.e., saturated conditions) shows a slightly higher bearing capacity values than computed value as shown in Figure 5.13(b). This may be attributed to the dilation of the Sollerod sand during the test which was caused due to lowering of the water table from the sand surface. The lowering of the water table induced a small suction in the upper sand layer (Steensen-Bach et al. 1987).

The bearing capacity of the Sollerod soil was measured using model footings in the suction range of 0 to 8 kPa. It is of interest to note that the bearing capacity of the tested sand increases linearly up to the air-entry value and starts falling around 6 kPa. The degree of saturation starts reduces at a rapid rate when the matric suction is in the range of 6 to 8 kPa (see Figure 5.9 (b)). In other words, in this suction range the bearing capacity starts decreasing due to the reduction of the water skeleton which leads to less attraction or cementation between the soil particles. Equation [5.22] was able to model the experimental behavior results reasonably well.



(a) SWRC for Sollerod Sand (Steensen-Bach et al. 1987)



(b) Bearing capacity versus suction

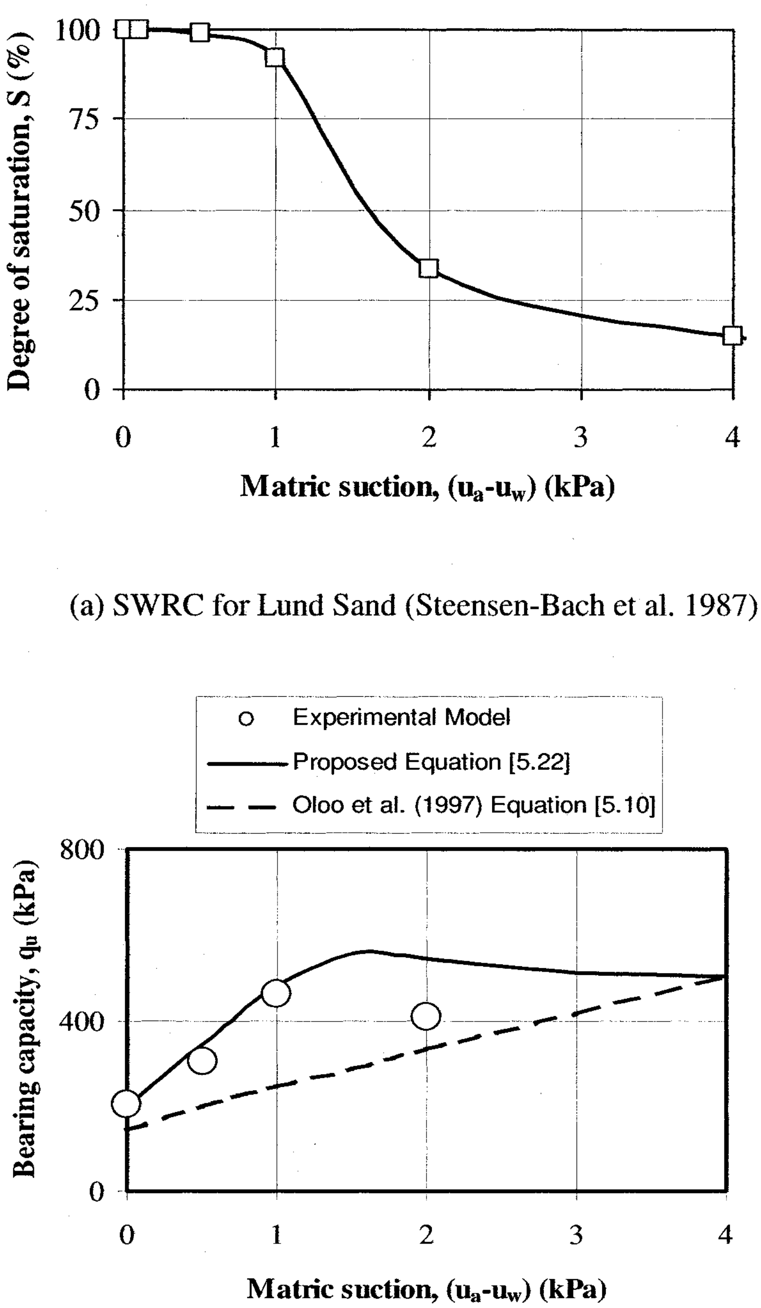
Figure 5. 12 Measured and predicted Bearing capacity for Sollerod Sand

5.8.3 Soil (C) Lund Sand

Another laboratory program was carried out by Steensen-Bach et al. (1987) using a small-scale plate load tests on Lund sand. Similar procedures were followed for measuring the measuring the Lund sand as detailed in the previous section for Sollerod sand. The Lund sand is coarser than the Sollerod sand and has a lower air-entry value of 0.8 kPa.

The properties of the Lund sand are listed in Table 5.6. The shear strength parameters and the bearing capacity factors along with other data are listed in Table 5.7 and Table 5.8 respectively. The soil was tested under matric suction values of 0 to 5 kPa. The findings on the Lund sand are similar to the Sollerod sand (Figure 5.13 (a) and (b)).

For all the three different sands studied (i.e., Soil A, Soil B and Soil C), the bearing capacity fitting parameter ψ value was found to be equal to 1. Along similar lines, a fitting parameter value of κ equal to 1 was for providing a best-fit between the measured and predicted shear strength values for sandy soils (using Equation [5.2]) (Vanapalli et al. 1996, Fredlund et al. 1996, Garven and Vanapalli 2006).



(a) SWRC for Lund Sand (Steensen-Bach et al. 1987)

(b) Bearing capacity versus suction

Figure 5. 13 Measured and predicted Bearing capacity for Lund Sand

5.8.4 Soil (D) Botkin Pit Silt (Wet of optimum)

Oloo (1994) measured the bearing capacity of the Botkin Pit silt that was compacted at several different water contents. The matric suction values of the compacted Botkin Pit specimens were also determined by Oloo (1994) using the axis translation technique. However, the SWRC of the compacted specimens at different water contents were not determined. To use Equation [5.22] for predicting the variation of bearing capacity with respect to matric suction, along with the shear strength parameters, c' and ϕ', the SWRC is also required. In other words, the experimental results of the bearing capacity values of specimens compacted at different water contents are available from Oloo (1994) but the variation of the bearing capacity with respect to matric suction is not available.

Several factors such as the stress history, compaction water content, soil structure, mineralogy influence the SWRC (Vanapalli et al. 1999, Vanapalli et al. 2002, Catana and Vanapalli 2006). The SWRC is a function of compaction water content and each soil specimen at specific compaction water content has a different SWRC. This is due to differences in soil structure of specimens compacted at different compaction water contents for different fine-grained soils.

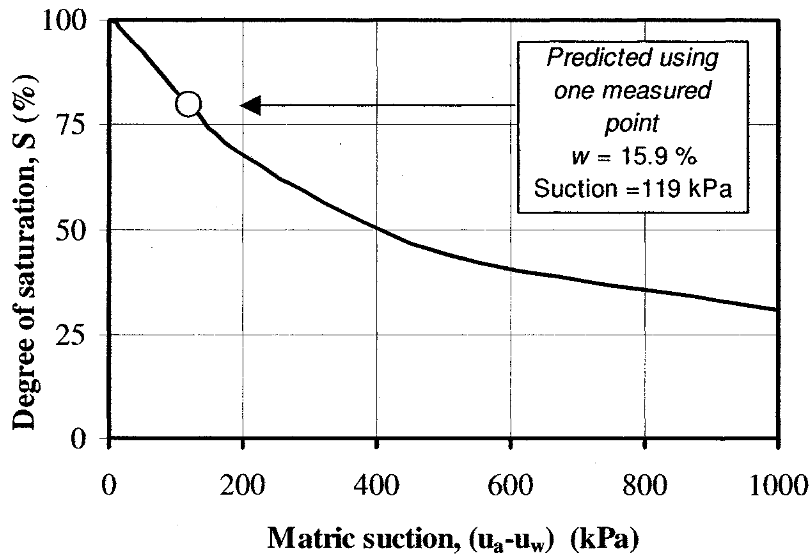
Catana and Vanapalli (2006) recently proposed a simple technique for estimating the SWRC for a compacted soil. This technique can be used for estimating the SWRC with the information of two data points using the Brustaert equation (1966) for fitting the SWRC. In this method it is suggested to use one measured point for the SWRC (i.e., water content and matric suction) in the suction range of 0 to 500 kPa.

The second point required was suggested to be obtained from a relationship that was developed between the suction capacity, C and a parameter, β (defined as liquid limit x clay fraction). This relationship was developed using the data on Atterberg limits coupled with clay fraction and the SWRC of 16 compacted clayey soils. This study shows that the SWRC can be reasonably well estimated for compacted fine-grained soils.

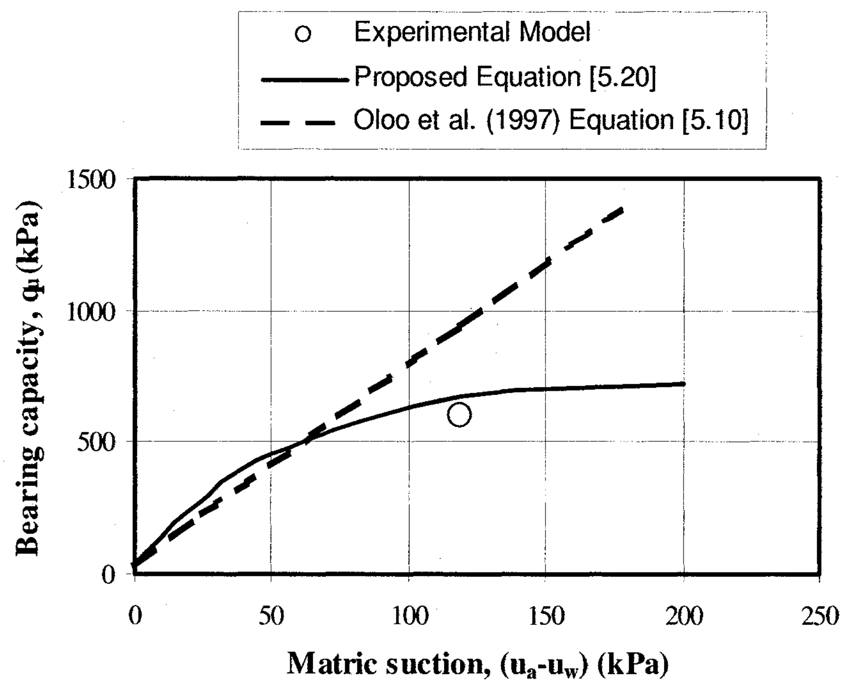
Catana and Vanapalli (2006) technique was used for estimation of the Botkin Pit silt SWRC compacted at wet of optimum conditions. The one measured point of matric suction and water content used in this estimation was 119 kPa and 15.9 % respectively and shown in Figure 5.14 (a). The estimated SWRC using the procedure suggested by Catana and Vanapalli (2006) is plotted as a continuous line in Figure 5.14 (a).

The saturated shear strength parameters, bearing capacity factors and the shape factors used for the prediction of the bearing capacity using the proposed Equation [5.22] are listed in Table 5.7. The measured bearing capacity at a matric suction value of 119 kPa is compared with the predicted variation of bearing capacity values with respect to matric suction. The predicted and measured values of the bearing capacity at the matric suction value of 119 kPa are approximately the same (Figure 5.14 (b)).

As summarized in Table 5.7, local shear failure conditions were assumed for the Botkin Pit Silt as well defined failure was not identified from the stress versus settlement relationship.



(a) Predicted SWRC for Botkin Silt



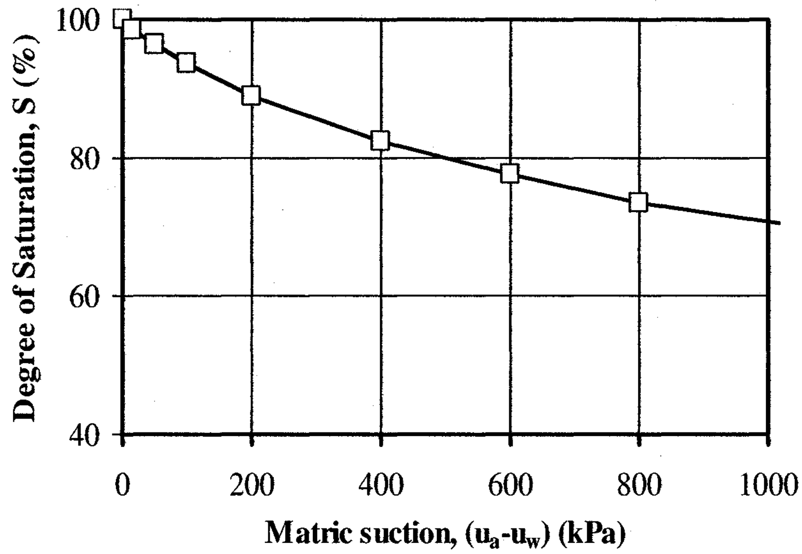
(b) Bearing capacity versus suction

Figure 5. 14 Measured and predicted Bearing capacity for Botkin Silt compacted at a wet of optimum water content

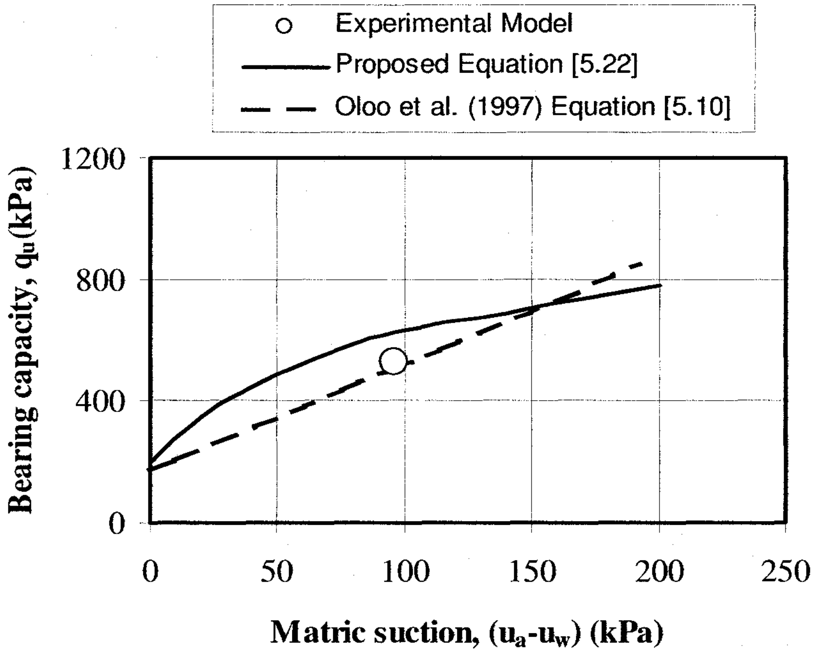
5.8.5 Soil (E) Glacial Till (Wet of optimum)

Oloo (1994) measured the bearing capacity on glacial till specimen compacted at wet of optimum water content conditions using a model footing. Similar to studies under taken on Botkin Pit silt (which were discussed in the earlier section), Oloo (1994) did not measure the variation of the bearing capacity of the glacial till with respect to matric suction and the SWRC. However, in a different study, Vanapalli (1994) measured the matric suction of the compacted specimen and the SWRC of the glacial till specimen compacted under identical conditions (Figure 5.15 (a)). Using the SWRC and the data information summarized in Table 5.7, comparisons are provided between the predicted variation of the bearing capacity with respect to matric suction using the Equation [5.22] and measured bearing capacity value at one value of matric suction in Figure 5.16 (b). Equation [5.10] was also used for providing the comparisons in Figure 5.15 (b) (using the data from Table 5.8). There is acceptable comparison between the measured and predicted bearing capacity values using Equations [5.22]. However, on a comparative basis, the other four soils provided better comparisons (i.e, Soil a, Soil B, Soil C and Soil D) than this soil. This may be attributed to the difficulties in defining the failure load values for local shear failure conditions.

Equation [5.10] provides reasonable comparison between the measured and computed values.



(a) Measured SWRC for Glacial Till (wet of optimum) (Vanapalli 1994)



(b) Bearing capacity versus suction

Figure 5. 15 Measured and predicted Bearing capacity for Glacial Till compacted at a wet of optimum water content

5.9 Relationship between the Bearing Capacity Fitting Parameter, ψ and the Plasticity Index, I_P

The bearing capacity fitting parameter, ψ was used in Equation [5.22] for providing a best-fit between the measured and predicted values of the variation of bearing capacity with respect to matric suction. The ψ parameter values for the five soils studied in this research program are summarized in Table 5.7. A best fit between the measured and predicted bearing capacity values for the three sands analyzed in this research program (i.e., Soil A, Soil B, and Soil C) was achieved using ψ value equal to 1. However, higher values were required for the other two fine-grained soils (i.e., Soil D and Soil E).

The variation of the bearing capacity fitting parameter, ψ is plotted versus the plasticity index, I_P in Figure 5.16 for the five soils.

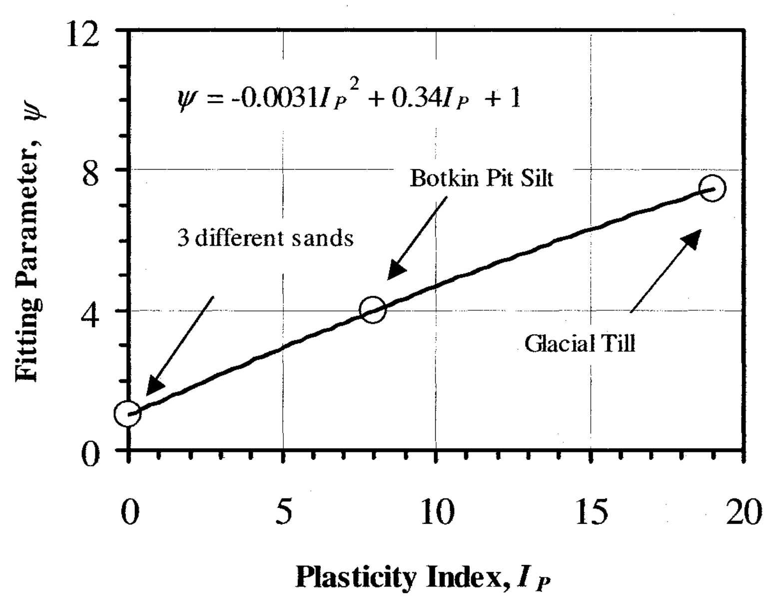


Figure 5.17 Relationship between, ψ and I_P for prediction bearing capacity of unsaturated soils

This relationship can be mathematically represented as given below:

$$\psi = -0.0031(I_P^2) + 0.34(I_P) + 1 \quad [5.23]$$

This relationship can be used for obtaining the y value. In other words, the bearing capacity of an unsaturated soil can be predicted using this relationship (i.e., Equation [5.23]) and the SWRC and the shear strength parameters, $c'$and ϕ'.

This study shows that is also possible to estimate the SWRC of the coarse-grained use one data point information of water content and suction using the method proposed by Vanapalli and Catana (2005) soils. Along similar lines, the SWRC of a fine-grained soil can be estimated using the method proposed by Catana and Vanapalli (2006). In other words, the SWRC required for prediction of the bearing capacity of unsaturated soils can be estimated using one point methods.

It is of interest to note that he form of the Equation [5.23] is similar to Equation [5.19], which the relationship between the fitting parameter $\kappa$and the plasticity index, I_P.

5.10 Relationship between the Shear Strength Fitting Parameter, $\kappa$and the Bearing Capacity Fitting Parameter, ψ

There is an approximate relationship between the bearing capacity fitting parameter, ψ (developed from this study) and the κ versus I_P relationship proposed by Garven and Vanapalli (2006) as given below.

$$\psi = \kappa^{2.5} \quad [5.24]$$

There are two main potential failure modes that are used in the literature to explain the bearing capacity of the footings. The failure of footing is commonly explained using shear resistance along the perimeter of the slip zone or punching of the soil (refer to Figure 5.2). Both these failure modes are developed extending Mohr's equation for shear strength of soils (Bowles 1996). This confirms that there is a strong relationship between the bearing capacity and the shear strength of a soil. Based on the study undertaken in this research program, the bearing capacity for each of the studied unsaturated soils was expressed as function of the shear strength parameters, c' and ϕ' and ϕ^b. The SWRC was used as tool to indirectly estimate the shear strength contribution due to suction. Consequently, a relationship was developed between the bearing capacity fitting parameter, ψ and the shear strength fitting parameter, κ as described in Equation [5.24]. This equation can be used as a tool for predicting the fitting parameter, ψ and used in the bearing capacity of unsaturated soils.

5.11 Summary

A semi-empirical equation was proposed in this research program to predict the bearing capacity of unsaturated soils using the saturated shear strength parameters and the SWRC. The proposed equation was tested on five soils. The data of four soils was obtained from the literature and one soil was studied in this research program. The proposed semi-empirical equation was found suitable to predicting reasonably the variation of bearing capacity with respect to matric suction of all the five soils with a plasticity index, I_P value in the range of 0 to 20.

The bearing capacity of each of the five studied soils increases almost linearly with respect to matric suction up to the air entry value. As the area of water content along which suction is communication reduces with desaturation, the bearing capacity increasing nonlinearly in the transition zone). The bearing capacity starts decreasing in the residual zone. In the residual zone the bearing capacity starts reducing. The test results support the observations of Terzaghi (1943) that the bearing capacity of a dry soil is close to the bearing capacity of a saturated soil.

It will be of interest to extend the proposed model for predicting the in-situ bearing capacity of soils that are in a state of unsaturated condition. This study can be of considerable interest to practicing engineers to estimate the bearing capacity of shallow foundations and foundation structures of pavements.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

A comprehensive experimental program was undertaken to determine the bearing capacity of compacted sand both in saturated and unsaturated conditions in a laboratory environment using model footings. All the required data related to soil properties, shear strength parameters, soil-water retention curves (SWRC), and suction measurements were collected. The bearing capacity theory proposed by Terzaghi's was extended for interpreting the bearing capacity of unsaturated soils taking account of the influence of suction. A semi-empirical equation was proposed for predicting the variation of bearing capacity with respect to suction using the SWRC and the saturated shear strength parameters. Based on the studies undertaken on the five different soils (one soil studied in this research program and the other four soils data collected from the literature) a relationship is developed between the bearing capacity factor, ψ and the plasticity index, I_p. This relationship along with the proposed semi-empirical equation can be extended based on field studies on an unsaturated soil at any value of suction for both coarse-grained soils and fine-grained soils.

6.2 Conclusions

The following conclusions can be drawn from the studies undertaken through this research program:

- The bearing capacity equipment designed and fabricated at the University of Ottawa functioned well for determining the bearing capacity of both saturated and unsaturated soils using the model footings.
- The bearing capacity of the unsaturated compacted sand of 5 to 7 times higher than the saturated bearing capacity.
- Terzaghi's bearing capacity theory was extended for interpreting the bearing capacity of unsaturated soils taking account of the suction.
- The study shows that there is a relationship between the bearing capacity of unsaturated soils and the soil water retention curve (SWRC). Based on this observation, a semi-empirical equation was developed using the experimental results from this research study along with results of other four soils reported in literature. The proposed equation can be used for prediction of the bearing capacity of unsaturated soils using the saturated shear strength parameters and the SWRC. The predicted bearing capacity values have shown good agreement with the measured bearing capacity values for the soil tested in this research program as well as the other four soils data collected from the literature.

6.3 Recommendations

The following recommendations are offered for further research work:

- To study of the effect of different footing sizes and shapes on the bearing capacity of unsaturated soils (i.e., to understand the scale effects)
- To study the effect of depth of the footing on the bearing capacity of unsaturated soils.

- To encourage the practicing engineers to apply the simple technique proposed in this thesis towards estimating the bearing capacity of unsaturated soils.

- To determine bearing capacity of fine-grained unsaturated soils using model footings over a large suction range.

- To determine the in-situ bearing capacity of soils using plate load tests and provide comparisons with the estimated bearing capacity using the framework proposed in this research work.

References

ASTM, D2216. (1994a). Laboratory determination of water (moisture) content of soil and rock by mass. *In annual Book of ASTM Standards, American Society of Testing Material*, Philadelphia, PA, 4.08, 177 – 180.

ASTM, D422. (1994b). Standard test method for particle size analysis of soils. *In annual Book of ASTM Standards, American Society of Testing Material*, Philadelphia, PA, 4.08, 10– 16.

ASTM, D698. (1994c). Test method for laboratory compaction characteristics of soils using standard effort. *In annual Book of ASTM Standards, American Society of Testing Material*, Philadelphia, PA, 4.08, 69 – 76.

ASTM, D854. (1994d). Standard test method for specific gravity of soils. *In annual Book of ASTM Standards, American Society of Testing Material*, Philadelphia, PA, 4.08, 80 – 83.

Alonso, E. E., Gens, A. and Josa, A. (1990). A constitutive model for partially saturated soils. *Geotechinque*, Vol. 40, No. 3: 405 – 430.

Ausilio E. and Conte E. (2005). Influence of groundwater on the bearing capacity of shallow foundations. Canadaian Geotechnical Journal, 42: 663 – 672.

Balla, A. (1962). Bearing capacity of foundations. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 88 (SM5): 13 – 34.

Bishop, A. W. and Blight, G. E. (1963). Some aspects of effecive stress in saturated and unsaturated soils. *Geotechnique*, Vol. 13, No. 3: 177 – 197.

Bolton, M. D. and Lau, C. K. (1989). Scale effects on the bearing capacity of granular soils. Proceedings of the 12^{th} International Conference of Soil Mechanics and Foundation Engineering. Vol. 2 : 895 – 898.

Bolton, M. D. and Lau, C. K. (1993). Vertical bearing capacity factors for circular footings on Mohr-Coulomb soil. *Canadian Geotechnical Journal*, 30: 1024-1033.

Bolton, M. D. (1986). The strength and dilatancy of sands. Geotechnique 36, No. 1: 65 – 78.

Bowles, J. E. (1996). *Foundation Analysis and Design*. 5th edition. McGaw – Hill, NY, USA.

Broms, B. B., (1964). The Effect of Degree of Saturation on the Bearing Capacity of Flexible Pavements. *Highway Research Recoed*, No. 71: 1 – 14.

Catana, M. C. and Vanapalli, S. K. (2006). The water retention charactristics of compacted clays. Proceedings of the Fourth International Conference on Unsaturated Soils, Carefree, Arizona, American Society of Civil Engineers Geotechnical Special Publication No. 147, Vol. 2, pp. 1-12.

Cerato A. (2005). Scale effects of shallow foundation bearing capacity on granular material. PhD. Thesis, Oklahoma University, OK, USA.

Coduto, D. P. (2001). Foundation design: principle and practices. Prentice-Hall, Inc. Upper Saddle River, NJ, USA.

Costa, Y.D, Cintra J. C. And Zornberg J. G. (2003). Influence of matric suction on the results of plate load tests performed on a lateritic soil deposit. Geotechnical Testing Journal, Vol. 26, No. 2: 219 – 226.

Chen, Fu Hua. (1999). Soil engineering, testing, design and remediation. CRC Press LLC, 2000 Corporate Blvd, N.W, Boca Ration, Florida, USA.

Chen, W.F., (1975). Limit analysis and soil plasticity. Elsevier, Amsterdam.

Das, B. M. (2004). *Principles of Foundation Engineering*. 5th ed., Brooks, CA, USA.

De Beer, E. E. and Ladanyi, B. (1961). Etude expérimental de la capacité portante du sable sous des foundations circulares établies en surface. *Proceeding of the 5th International Conference on Soil Mechanics and Foundation Engineering*, Paris, France, Vol. 1: 577– 581.

De Beer, E. E. (1965). Bearing capacity and settlement of shallow foundations on sand. *Proceedings of a Symposium on Bearing Capacity and Settlement of Foundations*, Duck University, Duhram: 15 – 32.

Deschenes, J.H. (1978). Bearing capacity of footings close to slopes cohesionless soil. PhD. thesis, University of Ottawa, Ottawa, ON, Canada.

Fredlund, D. G. and Morgenstern, N. R., (1977). Stress state variables for unsaturated soils. *Journal of the Geotechnical and Engineering Division*, ASCE, 103(GT5):447 – 466.

Fredlund, D. G. and Morgenstern, N. R. And Widger, R. A. (1977). Shear strength of unsaturated soils. *Canadian Geotechnical Journal,* Vol. 15 (3):313 - 321.

Fredlund, D. G. and Rahardjo H. (1993). *Soil Mechanics for Unsaturated Soils*, 1st edition, Wiley, New York.

Frydman, S. and Burd, H. J. (1997). Numerical studies of bearing capacity factor N_γ. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123. No. 1: 20 – 29.

Garven, E. and Vanapalli, S.K. (2006). Evaluation of empirical procedures for predicting the shear strength of unsaturated soils. Proceedings of the Fourth International Conference on Unsaturated Soils, Carefree, Arizona, American Society of Civil Engineers Geotechnical Special Publication No. 147, Vol. 2, pp. 2570-2581.

Georgrids, K., Potts, D. M. and Zdravkovic, L. (2003). The influence of partial soil saturation on pile behaviour. *Géotechnique* 53 No. 1:11 – 25.

Graham, J. and Stuart, J. G. (1971). Scale and boundary effects in foundation analysis. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 97 (SM11): 1533 – 1548.

Hanna, A. M., and Meyerhof, G. G. (1979). Ultimate bearing capacity of foundations on three-layer soil, with special reference to layered sand. *Canadian Geotechnical Journal*, 16: 412 – 414.

Hansen, B (1961). The bearing capacity of sand, tested by loading circular plates. Proceedings of the 5th International Conference of Soil Mechanics and Foundation Engineering. Vol. 1: 659 – 664.

Hansen, B. and Christiensen, N. H. (1969). Discussion of theoretical bearing capacity of very shallow foundations. *Journal of Soil Mechanics and Foundation Engineering Division*, ASCE, 95 (SM6): 1568 – 1572.

Ingra, T. S. and Baecher, G. B. (1983). Uncertainty in bearing capacity of sands. Journal of Geotechnical Engineering, Vol. 109, No. 7: 899 – 913.

Ismael, N. F. (1985). Allowable Pressure from Loading Tests on Kuwaiti Soils. *Canadian Geotechnical Journal*, 22: 151 – 157.

Ismael, N. F. (1996). Loading tests on circular and ring plates in very dense cemented sands. Journal of Geotechnical Engineering, Vol. 122, No. 4: 281 – 287.

Lawrence, A. L. (1968). Theoretical bearing capacity of very shallow footings. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 94, No. (SM6): 1347 – 1357.

Krishnamurthy, S. and Kameswara R. (1975). The effect of submergence on bearing capacity. Soils and Foundations. Vol. 15, No. 3: 61 – 66.

Kumbhokjar, A.S. (1993). Numerical evaluation of Terzaghi's N_γ. Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 1999, No. 3, pp. 598-607.

Lu, N. and Likos, W. (2004). Unsaturated soil mechanics. John Wiley and Sons. New York, NY, USA.

Meyerhof, G. G. (1951). The ultimate bearing capacity of foundations. Geotechnique, Vol. 2: 301 – 332.

Meyerhof, G. G. (1956). Penetration tests and bearing capacity of cohesionless soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 82, No. (SM1): Paper: 866.

Meyerhof, G. G. (1965). Shallow foundations. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 91, No. (SM2): 21 – 31.

Meyerhof, G. G. (1982). The bearing capacity and settlement of foundations. Technical University in Novascotia, Halifax, Canada.

Miller, G. A. and Muraleetharan, K. K. (1998). In situ testing in unsaturated soil. *Proceedings of the 2nd International Conference on Unsaturated Soils*, Beijing, China, Vol. 1: 416 – 421.

Mohamed, F. M. O. and Vanapalli, S. K. (2006). Laboratory investigations for the measurement of the bearing capacity of an unsaturated coarse-grained soil. 59th Canadian Geotechnical Conference, BC, Vancouver. (In print).

Nainan P. K., (2005). Design of foundation systems: Principles and practices, 3th edition. Alpha Science International Ltd., Harrow, UK.

Oloo, S. Y. (1994). A bearing capacity approach to the design of low-volume traffic roads. PhD. Thesis, University of Saskatchewan, Saskatoon, SK, Canada.

of an International Symposium on Advanced Experimental Unsaturated Soil Mechanics: 401 – 410.

Vanapalli, S. K., Fredlund, D. G., Pufahal, D. E. and Clifton, A. W., (1996). Model for the prediction of shear strength with respect to soil suction. Canadian Geotechnical Journal, No. 33: 379 – 392.

Vanapalli, S. K. and Fredlund, D. G. (1999). Comparison of empirical procedures to predict the shear strength for unsaturated soils. XI Asian Regional Conference, International Society for Soil Mechanics and Foundation Engineering, Seoul, August 16-20, 1999, Sung-Wan Hong et al., A. A. Balkema, Vol. 1.

Vanapalli, S. K., Fredlund, D. G. and Pufahl, D. E. (1999). The influence of soil structure and stress history on the soil-water characteristics of compacted till. *Geotechnique* 49, No. 2: 143 – 159.

Vanapalli, S.K. and Fredlund, D.G. (2000). Comparison of empirical procedures to predict the shear strength of unsaturated soils using the soil-water characteristic curve, Geo-Denver 2000. American Society of Civil Engineers, Geotechnical Special Publication. No. 99, pp. 195-209.

Vesić, A. S. (1963). Bearing capacity of deep foundations in sand. *Highway Research Record*, No. 39: 112 – 153.

Vesić, A. S. (1973). Analysis of ultimate loads of shallow foundations. *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 99, No. (SM1): 45 – 73.

Yongfu Xu (2004). Bearing capacity of unsaturated expansive soils. *Geotechnical and Geological Engineering,* No. 22: 611 – 625.

Zhu D. Y., Lee C. F. and Jiang H. D. (2001). A numerical study of bearing capacity factor N_γ. *Canadian Geotechnical Journal*, 38: 1090 – 1096.

APPENDIX A

A 1. BEARING CAPACITY DERIVATIONS (Oloo 1994)

Effect of Matric Suction on the Bearing Capacity of Unsaturated Soils

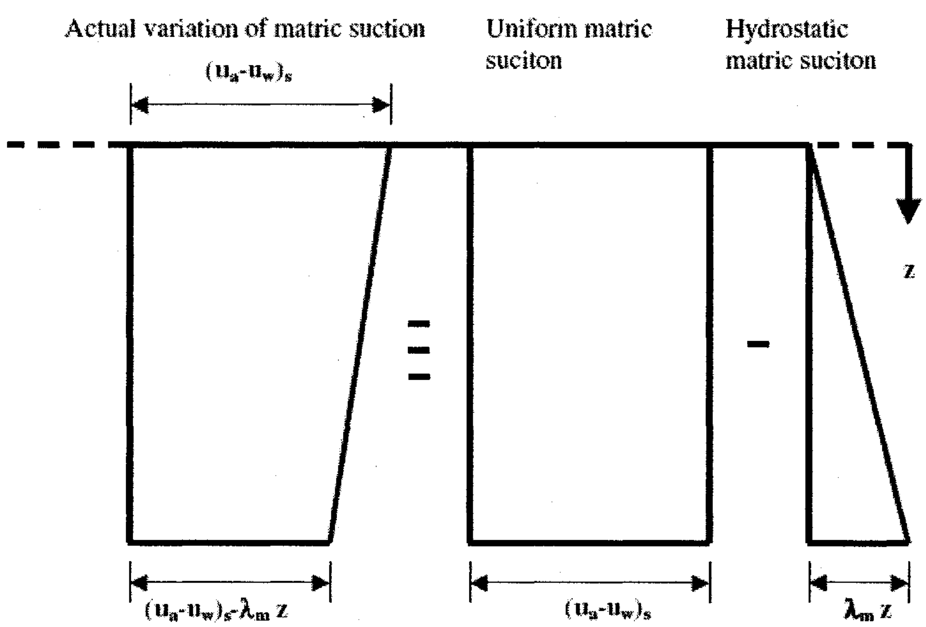


Figure A. 1 Idealized of the variation of matric suction with depth (Oloo 1994)

$$(u_a - u_w)_z = (u_a - u_w) - \lambda_m z \qquad [A. 1]$$

where:

z = depth under consideration, *m*

$(u_a - u_w)_s$ = matric suction at the surface of the soil, *kPa*

λ_m = rate of decrease of suction with depth

$(u_a - u_w)_z$ = matric suction at depth z, kPa

In the derivation of the influence of decreasing cohesion, c_o on the bearing capacity, the shear strength at depth, z is given by:

$$c_z = c_o - \lambda z \qquad [A. 2]$$

$$\lambda = \lambda_m \tan\phi^b \qquad [A. 3]$$

The solution developed for cohesion decreasing with depth can be modified for matric suction by recognizing that:

$$c_o = [(u_a - u_w)_b(\tan\phi' - \tan\phi^b) + (u_a - u_w)_s \tan\phi^b \qquad [A. 4]$$

By substituting in the following equation:

$$q_u = c_o N_{c_o} - \frac{B}{2}\lambda N_{\gamma_o} \qquad [A. 5]$$

$$q_u = [(u_a - u_w)_b(\tan\phi' - \tan\phi^b) + (u_a - u_w)_s \tan\phi^b]N_{c_o} - \frac{B}{2}\lambda_m N_\lambda \tan\phi^b \qquad [B. 6]$$

If the soil has an air entry value approaches zero:

$$q_u = [(u_a - u_w)_s \tan\phi^b]N_c - \frac{B}{2}\lambda_m N_\lambda \tan\phi^b \qquad [A. 7]$$

In the matric suction is below the air entry value the soil is saturated and under negative pore water pressure:

$$\tan\phi^b = \tan\phi' \qquad [A. 8]$$

$$q_u = [(u_a - u_w)_s \tan\phi']N_{c_o} - \frac{B}{2}\lambda_m N_\lambda \tan\phi' \qquad [A. 9]$$

The bearing capacity factor, N_λ was proposed and can be expressed in terms of N_γ:

$$N_\lambda = \frac{N_{\gamma_o}}{\tan\phi'} \quad \text{[A. 10]}$$

Therefore, the equation for the bearing capacity (i.e., general linear variation of matric suction) of a surface footing resting on a soil in which the matric suction decreases at the rate λ_m with depth is given by:

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - \tan\phi^b) + (u_a - u_w)_s \tan\phi^b]N_{c_o} + \frac{B}{2}(\lambda - \lambda_m N_\lambda \frac{\tan\phi^b}{\tan\phi})N_{\gamma_o} \quad \text{[A. 11]}$$

A.1.1 For uniform matric suction profile:

$$q_u = [c' + (u_a - u_w)_b(\tan\phi' - \tan\phi^b) + (u_a - u_w)_s \tan\phi^b]N_{c_o} + \frac{B}{2}\gamma N_{\gamma_o} \quad \text{[A. 12]}$$

A.1.2 For hydrostatic matric suction variation:

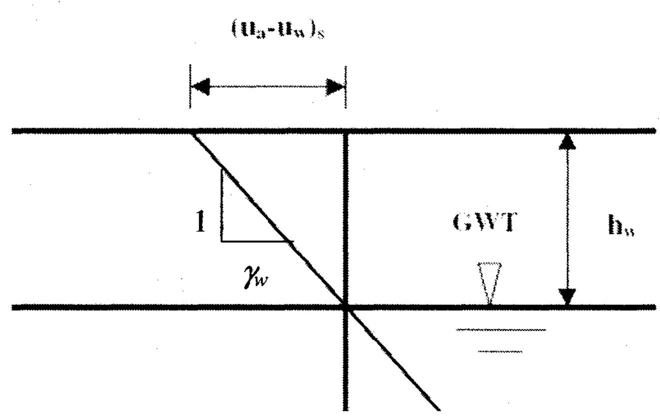


Figure A. 2 Hydrostatic variation of matric suction

$$(u_a - u_w)_s = \gamma_w h_w \quad \text{[A. 13]}$$

$$\lambda_m = \gamma_w \quad \text{[A. 14]}$$

$$q_u = [c' + (u_a - u_w)_b (\tan\phi' - \tan\phi^b) + \gamma_w h_w \tan\phi^b] N_{c_o} + \frac{B}{2}(\gamma - \gamma_w \frac{\tan\phi^b}{\tan\phi}) N_{\gamma_o} \quad \text{[A. 15]}$$

where:

q_u = bearing capacity at failure, *kPa*

c' = effective cohesion, *kPa*

B = footing width, *m*

h_w = depth of ground water table, *m*

λ_m = rate of decrease of suction with depth

N_{c_o}, N_{γ_o} = bearing capacity factors due to cohesion and surcharge respectively

$(u_a - u_w)_b$ = matric suction at the air entry value, *kPa*

ϕ^b = angle of internal friction with respect to matric suction, *deg.*

γ = total unit weight of the soil, *kN/m*^{3}

γ_w = unit weight of water, *kN/m*^{3}

A 2. BEARING CAPACITY CALCULATIONS

Soil (A)

Soil (A) Results with Model footing 100 mm × 100 mm Size

Table A. 1 Comparisons between the predicted bearing capacity for the tested sand using present model (100 mm × 100 mm)

SOIL TYPE	Coarse-grained sand		
B	0.1	m	
L	0.1	m	
Shear Failure	GSF		
c'	0.6	kPa	
γ_{sat}	20.0	kN/m^3	
γ	18	kN/m^3	
$(u_a-u_w)_b$	3.0	(kPa)	Air Entry Value
ψ	1.0	-	
ϕ'	35.3	*degrees*	From Lab. Test
ϕ'	39	*degrees*	Modified
N_c	85.97	From Table 5.1	
N_q	70.61	From Table 5.1	
N_γ	95.03	From Table 5.1	
ξ_c	1.82	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	
Measured and predicted SWRC		**Proposed Eq. [5.22]**	**Experimental**
S, (%)	**(u_a-u_w), (kPa)**	**B.C., (kPa)**	**B.C., (kPa)**
100	**0**	**123**	**121**
93	**2**	**408**	**570**
78	**4**	**624**	**715**
58	**6**	**746**	**840**
12	**8**	**601**	-
2	**10**	**543**	-

Table A. 2 Predicted B.C. for the tested sand using Oloo et al. (1997) model (100 mm × 100 mm)

SOIL TYPE	Coarse-grained sand		
B	0.1	*m*	
L	0.1	*m*	
Shear Failure	**GSF**		
c'	0.6	*kPa*	
γ_{sat}	20.0	kN/m^3	
γ	18	kN/m^3	
ϕ'	35.3	*degrees*	From Lab. Test
ϕ'	39	*degrees*	Modified
ϕ^b	19.5	*degrees*	$\phi^b = 0.5\ \phi'$
N_{c_o}	60	From Table 5.2	
N_{q_o}	70	From Table 5.2	
N_{γ_o}	125	From Table 5.2	
s_c	2.167	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	
Measured matric suction $(u_a$-$u_w)$, (kPa)		**Oloo et al. (1997) Eq.[5.11] B.C., (kPa)**	**Experimental B.C., (kPa)**
0		**116**	**121**
2		**238**	**570**
4		**330**	**715**
6		**422**	**840**
8		**514**	**-**
10		**606**	**-**

Soil (A) Results with Model footing 150 mm × 150 mm Size

Table A. 3 Predicted B.C. for the tested sand using present model (150 mm × 150 mm)

SOIL TYPE	Coarse-grained sand		
B	0.15	m	
L	0.15	m	
Shear Failure	GSF		
c'	0.6	kPa	
γ_{sat}	20.0	kN/m^3	
γ	18	kN/m^3	
$(u_a-u_w)_b$	3.0	(kPa)	Air Entry Value
ψ	1.0	-	
ϕ'	35.3	*degrees*	From Lab. Test
ϕ'	39	*degrees*	Modified
N_c	85.97	From Table 5.1	
N_q	70.61	From Table 5.1	
N_γ	95.06	From Table 5.1	
ξ_c	1.82	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	

Measured and predicted SWRC		Proposed Eq. [5.22]	Experimental
S, (%)	(u_a-u_w), (kPa)	B.C., (kPa)	B.C., (kPa)
100	0	138	125
93	2	509	630
78	4	762	700
58	6	905	745
12	8	736	-
2	10	667	-

Table A. 4 Predicted B.C. for the tested sand using Oloo et al. (1997) model (150 mm × 150 mm)

SOIL TYPE	Coarse-grained sand		
B	0.15	m	
L	0.15	m	
Shear Failure	GSF		
c'	0.6	kPa	
γ_{sat}	20.0	kN/m^3	
γ	18	kN/m^3	
ϕ'	35.3	*degrees*	From Lab. Test
ϕ'	39	*degrees*	Modified
ϕ^b	19.5	*degrees*	$\phi^b = 0.5\ \phi'$
N_{c_o}	60	From Table 5.2	
N_{q_o}	70	From Table 5.2	
N_{γ_o}	125	From Table 5.2	
s_c	2.167	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	

Measured matric suction (u_a-u_w), (kPa)	Oloo et al. (1997) Eq.[5.11] B.C., (kPa)	Experimental B.C., (kPa)
0	**135**	**121**
2	**271**	**570**
4	**363**	**715**
6	**455**	**840**
8	**547**	-
10	**639**	-

Soil (B) Sollerod Sand

Table A. 5 Predicted B.C. for the tested sand using present model for Sollerod sand

SOIL TYPE	Sollerod sand		
B	0.022	m	
L	0.022	m	
Shear Failure		GSF	
c'	0.8	kPa	
γ_{sat}	20.05	kN/m^3	
γ	18.25	kN/m^3	
$(u_a-u_w)_b$	4.0	(kPa)	Air Entry Value
ψ	1.0	-	
ϕ'	35.78	*degrees*	From Lab. Test
ϕ'	39.4	*degrees*	Modified
N_c	90	From Table 5.1	
N_q	75	From Table 5.1	
N_γ	104	From Table 5.1	
ξ_c	1.833	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	
Measured SWRC		Proposed Eq. [5.22]	Experimental
S, (%)	(u_a-u_w), (kPa)	B.C., (kPa)	B.C., (kPa)
100	0	139	170
99	1	284	220
99	2	418	560
99	4	686	-
86	6	917	-
37.2	8	877	780
26	9	850	-
20	10	835	-

Table A. 6 Predicted B.C. for the tested sand using Oloo et al. (1997) model for Sollerod sand

SOIL TYPE	Sollerod sand		
B	0.022	m	
L	0.022	m	
Shear Failure		GSF	
c'	0.8	kPa	
γ_{sat}	20.05	kN/m^3	
γ	18.25	kN/m^3	
ϕ'	35.78	*degrees*	From Lab. Test
ϕ'	39.4	*degrees*	Modified
ϕ^b	19.7	*degrees*	$\phi^b = 0.5\ \phi'$
N_{c_o}	62	From Table 5.2	
N_{q_o}	72	From Table 5.2	
N_{γ_o}	135	From Table 5.2	
s_c	2.16	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	

Measured matric suction $(u_a\text{-}u_w)$, (kPa)	Oloo et al. (1997) Eq.[5.11] B.C., (kPa)	Experimental B.C., (kPa)
0	116	170
1	171	220
2	219	560
4	315	-
6	411	-
8	507	780
9	555	-
10	603	-

Soil (C) Lund Sand

Table A. 7 Predicted B.C. for the tested sand using present model for Lund sand

SOIL TYPE	Lund sand		
B	0.022	m	
L	0.022	m	
Shear Failure	GSF		
c'	0.6	kPa	
γ_{sat}	20.00	kN/m^3	
γ	18.19	kN/m^3	
$(u_a-u_w)_b$	0.8	(kPa)	Air Entry Value
ψ	1.0	-	
ϕ'	44	*degrees*	
N_c	151.95	From Table 5.1	
N_q	147.74	From Table 5.1	
N_γ	261.063	From Table 5.1	
ξ_c	1.972	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	

Measured SWRC		Proposed Eq. [5.22]	Experimental
S, (%)	(u_a-u_w), (kPa)	B.C., (kPa)	B.C., (kPa)
100	0	197	200
98	0.5	342	300
98	0.75	413	-
95	1	482	460
65	1.5	558	-
33.5	2	543	405
13	3	510	-
8	4	501	-

Table A. 8 Predicted B.C. for the tested sand using Oloo et al. (1997) model for Lund sand

SOIL TYPE	Lund sand		
B	0.022	m	
L	0.022	m	
Shear Failure	GSF		
c'	0.6	kPa	
γ_{sat}	20.00	kN/m^3	
γ	18.19	kN/m^3	
ϕ'	44	*degrees*	
ϕ^b	22.0	*degrees*	$\phi^b = 0.5\ \phi'$
N_{c_o}	100	From Table 5.2	
N_{q_o}	115	From Table 5.2	
N_{γ_o}	220	From Table 5.2	
s_c	2.15	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	

Measured matric suction (u_a-u_w), (kPa)	Oloo et al. (1997) Eq.[5.11] B.C., (kPa)	Experimental B.C., (kPa)
0	**144**	**200**
0.5	**199**	**300**
0.75	**221**	**-**
1	**242**	**460**
1.5	**286**	**-**
2	**329**	**400**
3	**416**	-
4	**503**	-

Soil (D) Botkin Silt (Wet of optimum)

Table A. 9 Predicted B.C. for the tested sand using present model for Botkin Silt (Wet of optimum)

SOIL TYPE	Botkin silt		
B	0.03	m	
L	0.03	m	
Shear Failure		LSF	
c'	2.5	kPa	
$0.67c'$	1.70	kPa	
γ_{sat}	20.78	kN/m^3	
γ	19.14	kN/m^3	
$(u_a-u_w)_b$	35.0	(kPa)	Air Entry Value
ψ	3.85	-	
ϕ'	28.00	*degrees*	
ϕ^*	20	*degrees*	$\tan\phi^* = 0.67 \tan\phi'$
N_c	17.69	From Table 5.1	
N_q	7.44	From Table 5.1	
N_γ	3.64	From Table 5.1	
ξ_c	1.42	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	
Predicted SWRC		**Proposed Eq. [5.22]**	**Experimental**
S, (%)	**(u_a-u_w), (kPa)**	**B.C., (kPa)**	**B.C., (kPa)**
100	**0**	**43**	-
97	**20**	**240**	-
92	**50**	**460**	-
80	**119**	**684**	**600**
70	**200**	**741**	-
SWRC predicted using one poit of experimental data along with liquid limit and clay fraction			

Table A. 10 Predicted B.C. for the tested sand using Oloo et al. (1997) model for Botkin Silt (Wet of optimum)

SOIL TYPE	Botkin silt		
B	0.03	m	
L	0.03	m	
Shear Failure		LSF	
c'	2.5	kPa	
$0.67\ c'$	1.70	kPa	
γ_{sat}	20.78	kN/m^3	
γ	19.14	kN/m^3	
ϕ'	28.0	*degrees*	
ϕ^*	20	*degrees*	$\tan\phi^* = 0.67 \tan\phi'$
ϕ^b	7.74	*degrees*	
N_{c_o}	14.6	From Table 5.2	
N_{q_o}	6.2	From Table 5.2	
N_{γ_o}	5.2	From Table 5.2	
s_c	1.425	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	

Measured matric suction (u_a-u_w), (kPa)	Oloo et al. (1997) Eq.[5.11] B.C., (kPa)	Experimental B.C., (kPa)
0	36	-
25	225	-
50	415	-
119	937	600
160	1247	-
178	1383	-

Soil (E) Glacial Till (Wet of optimum)

Table A. 11 Predicted B.C. for the tested sand using present model for Glacial Till (Wet of optimum)

SOIL TYPE	Glacial Till		
B	0.03	m	
L	0.03	m	
Shear Failure	LSF		
c'	16.9	kPa	
$0.67\, c'$	11.32	kPa	
γ_{sat}	20.58	kN/m^3	
γ	18.79	kN/m^3	
$(u_a-u_w)_b$	40.0	(kPa)	Air Entry Value
ψ	7.50	-	
ϕ'	21.9	*degrees*	
ϕ^*	15	*degrees*	$\tan \phi^* = 0.67 \tan \phi'$
N_c	12.86	From Table 5.1	
N_q	4.45	From Table 5.1	
N_γ	1.52	From Table 5.1	
ξ_c	1.346	Eq. [5.5]	
ξ_γ	0.6	Eq. [5.7]	
Measured SWRC		**Proposed Eq. [5.22]**	**Experimental**
S, (%)	**(u_a-u_w), (kPa)**	**B.C., (kPa)**	**B.C., (kPa)**
100	**0**	**196**	-
96.6	**20**	**349**	-
94.15	**50**	**490**	-
92.2	**80**	**589**	-
91	**96**	**625**	525
86	**200**	**777**	-

Table A. 12 Predicted B.C. for the tested sand using Oloo et al. (1997) model for Glacial Till (wet of optimum)

SOIL TYPE	Glacial Till		
B	0.03	m	
L	0.03	m	
Shear Failure		LSF	
c'	16.9	kPa	
$0.67c'$	11.32	kPa	
γ_{sat}	20.58	kN/m^3	
γ	18.79	kN/m^3	
ϕ'	21.9	*degrees*	
ϕ^*	15	*degrees*	$\tan\phi^* = 0.67 \tan\phi'$
ϕ^b	13.17	*degrees*	
N_{c_o}	11	From Table 5.2	
N_{q_o}	4	From Table 5.2	
N_{γ_o}	2.7	From Table 5.2	
s_c	1.364	Eq. [5.12]	
s_γ	0.6	Eq. [5.13]	

Measured matric suction (u_a-u_w), (kPa)	Oloo et al. (1997) Eq.[5.11] B.C., (kPa)	Experimental B.C., (kPa)
0	170	-
76	437	-
96	507	525
114	570	-
193	847	-

APPENDIX B

B 1. PUBLISHED PAPERS BASED ON THIS RESEARCH PROGRAM

B.1.1 Laboratory Investigations for the Measurement of the Bearing Capacity of Unsaturated Soils

The 59th Canadian Geotechnical Conference will be held from 1st – 4th October 2006, Vancouver, British Columbia, Canada.

B.1.2 Bearing Capacity of Model Footings in Unsaturated Soils

The 2nd International Conference of Mechanics of Unsaturated Soils will be held from 7th to 9th March 2007 at the Bauhaus University, Weimar, Germany.

LABORATORY INVESTIGATIONS FOR THE MEASUREMENT OF THE BEARING CAPACITY OF AN UNSATURATED COARSE-GRAINED SOIL

Fathi M. O. MOHAMED, Dept. of Civil Engineering, University of Ottawa, Ottawa, ON, Canada
Sai K. VANAPALLI, Dept. of Civil Engineering, University of Ottawa, Ottawa, ON, Canada

ABSTRACT
The bearing capacity of a surface footing on saturated and unsaturated, compacted coarse-grained soil was measured in a specially designed equipment in a laboratory environment using square shaped model footings of two different sizes. The soil suctions in the compacted, unsaturated coarse-grained soil were measured using commercial Tensiometers located in the vicinity of the expected stress bulb below the model footings. The measured bearing capacity of the footings increases almost linearly with matric suction up to the air-entry value of the soil. The trends of results of the variation of bearing capacity with respect to suction are similar to that of the shear strength of unsaturated soils. The bearing capacity of the tested coarse-grained soil under unsaturated conditions was found to be approximately 5 to 7 times higher than the bearing capacity of the soil under fully saturated conditions. The results of this experimental program suggest the conventional bearing capacity theory used in the engineering practice is highly conservative when it is applied for unsaturated soils.

RÉSUMÉ
La portance d'une semelle de surface sur un sol à gros grains compacté, saturé et non-saturé, a été mesurée à l'aide d'équipement de laboratoire conçu à cet effet et utilisant deux semelles quarrées de taille différente. Les mesures de la succion du sol dans le sol à gros grains compacté non-saturé ont été effectuées à l'aide de tensiomètres commerciaux situés à proximité du bulbe de contraintes prévu sous les semelles. La portance mesurée des semelles augmente presque linéairement avec la succion jusqu'à la valeur d'entrée d'air. Les tendances des résultats de la variation de la portance en fonction de la succion sont semblables à celle de la résistance au cisaillement pour des sols non-saturés. La portance du sol à gros grains testés dans des conditions non-saturées s'est avérée être approximativement de 5 à 7 fois plus élevée que la portance du sol dans des conditions entièrement saturées. Les résultats de ce programme expérimental suggèrent que la théorie conventionnelle de portance utilisée dans la pratique soit fortement conservatrice quand elle est appliquée aux sols non-saturés.

1. INTRODUCTION

In many arid and semi-arid regions, shallow foundations are usually located above the ground water table where the soil is typically in a state of unsaturated condition. There is limited laboratory and field data available in the literature with respect to the bearing capacity of unsaturated soils (Miller and Muraleetharan 1998, and Costa et al. 2003). The contribution of capillary stresses (i.e., matric suction) is usually ignored in the conventional bearing capacity analysis. The bearing capacity of unsaturated soils can be a governing parameter in the design of shallow foundations in dense or compacted soils (i.e., coarse-grained soils) if the settlement is negligible (Bowles 1996). Therefore, a framework for interpreting the bearing capacity of unsaturated soils will be useful in engineering practice.

2. BACKGROUND

The bearing capacity theory has been a subject of interest for many researchers during the last century. Prandtl (1921) was one of the pioneering investigators who studied the bearing capacity of soils. In that study, a strip footing was loaded until the footing penetrated in to the soil. The applied stress, which caused the failure, was defined as the ultimate bearing capacity of a soil. Several techniques and empirical procedures followed this study provide a comprehensive understanding of the bearing capacity of shallow foundations (Terzaghi 1943, Terzaghi and Peck 1948, Meyerhof 1956 and 1965, Vesić 1963 and 1973, Lawrence 1968, and Bolton and Lau 1993). However, all these procedures were based on the assumption that the soil is in a state of fully saturated condition. Several studies have shown that conventional procedures for determination of the bearing capacity of shallow foundations usually provide conservative estimations (De Beer and Ladanyi 1961, Vesić 1973, and Silvestri 2003). This is particularly true for soils that are in a state of unsaturated condition.

Considerable research has been undertaken during the last 50 years to interpret the engineering behavior of unsaturated soils using independent stress state variables (Bishop and Blight 1963, and Fredlund and Morgenstern 1977). The capillary stress or matric suction, $(u_a - u_w)$ which is one of the key stress state variables used in the interpretation of unsaturated soils behaviour is defined as the difference between the air pressure, u_a and water pressure, u_w. Matric suction has a significant influence on the engineering behavior of unsaturated soils including the bearing capacity (Fredlund and Rahardjo 1993, and Oloo et al. 1997). Steensen-Bach et al. 1987 comment that ignoring the influence of capillary stresses in the bearing capacity of unsaturated soils would be equivalent to disregarding the influence of reinforcement in the design of reinforced concrete.

Limited research work was carried out to study the influence of matric suction on the bearing capacity of unsaturated soils (Broms 1964, Steensen-Bach et al. 1987, Fredlund and Rahardjo 1993, Miller and Muraleetharan 1998, Oloo et al. 1997, and Costa et al. 2003). Oloo et al. 1997 presented an approach to interpret the bearing capacity of two fine-grained unsaturated soils based on laboratory model studies. However, there are limited studies reported in the literature with respect to interpretation of the bearing capacity of unsaturated coarse-grained soils (Steensen-Bach et al. 1987, and Nabil 1985 and Costa et al. 2003).

In this paper, the bearing capacity of a surface footing on saturated and unsaturated, compacted coarse-grained soil was measured using the University of Ottawa Bearing Capacity Equipment (UOBCE) that was specially designed and built for this research program at the University of Ottawa student workshop. The bearing capacity was measured in a controlled laboratory environment using two square shaped model footings of different sizes (i.e., 100 mm x 100 mm and 150 mm x 150 mm). The bearing capacity of the tested coarse-grained soil under unsaturated conditions was found to be approximately 5 to 7 times higher than the bearing capacity of the same soil under fully saturated conditions. The results of this experimental program suggest the conventional bearing capacity theory which is used in the engineering practice is highly conservative when it is applied for unsaturated soils. The results of this research program also demonstrate that there is a strong relationship between the soil-water retention curve (SWRC) and the bearing capacity similar to the shear strength behaviour of unsaturated soils.

3. EQUIPMENT DETAILS AND METHODOLOGY

3.1 General

Figure 1 shows the details of the University of Ottawa Bearing Capacity Equipment (UOBCE) that was specially designed to serve the objectives of the proposed testing program. The key objective of the research presented in this paper was to measure the bearing capacity of a compacted coarse-grained soil under saturated and unsaturated conditions. This equipment has special provisions to simulate fully saturated and unsaturated conditions in the bearing capacity tank. The water table level in the tank can be adjusted to the desired level using drainage valves. While the water level in the tank can be measured using the piezometers, the capillary tension (i.e., matric suction) in the unsaturated compacted coarse-grained soil was measured using commercial Tensiometers.

3.2 Details of equipment

All the key features of this equipment are summarized in this section:

(i) The loading frame for this equipment was constructed using an aluminum C-channel (150 mm web x 50 mm flange with 8 mm in thickness). The frame is 2450 mm in height and 1450 mm wide. Four C-channels (with the same section as the loading frame) were used to support the test tank. Two more channels were fastened on top of the frame of the loading machine such that they can offer resistance to the reaction loads. One more C-channel was placed underneath the loading machine to facilitate the placement of the loading arm and the load cell.

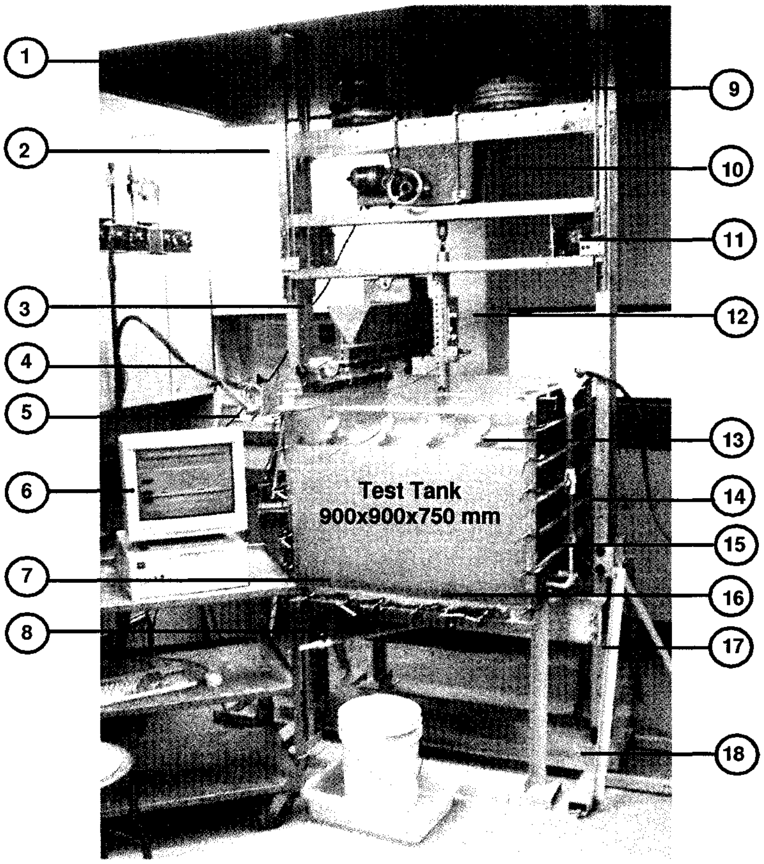
(ii) An electrically operated and mechanically controlled loading system was used to load the model footings. A maximum load of 14 kN can be applied using this system.

(iii) The test tank dimensions are 900 mm x 900 mm in plan and 750 mm in depth with provisions for collecting the required data which include load, deformation, water level in the tank, and matric suction below the surface footing.

(iv) The test tank was constructed using 6 mm thick aluminum sheets. Stiffeners were added along the horizontal direction to prevent any lateral bending or bulging that may occur during the loading of the footing. The clear distance between the model footing edge and the sides of the tank was equal to four times the width of the footings used in the study to avoid the influence of boundary effects in the stress bulb zone. The depth of the tank was deeper than the expected depth of the stress bulb below the model footing. Considerably larger depth was used to allow the water table to be raised and lowered to simulate different suction values using drainage valves without any interference from the instrumentation used in the study.

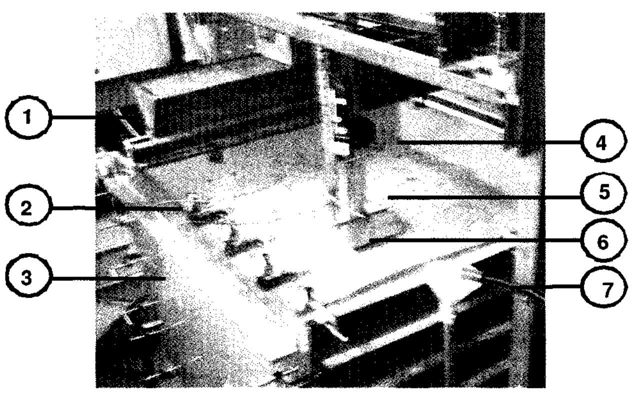
(v) Aluminum metal model square footings with the dimensions of 100 mm x 100 mm or 150 mm x 150 mm dimensions were used in this research program. The thickness of the footing was equal to 50 mm. The bottom surface of the model footing was corrugated to introduce roughness in the footing. The footings were placed on the surface of the soil in the test tank and subjected to vertical static loading using an adjustable-loading machine with a constant loading rate of 1.2 mm/min.

(vi) The soil was placed in the tank through the use of a V- shaped hopper which had a capacity to hold 25 kg



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

Figure 1. The University of Ottawa Bearing Capacity Equipment (UOBCE)



1. Hopper	**2 .Tensiometer**
3. Transparent window	**4. Load cell**
5. LVDT	**6. Model footing**
7. Water supply source	

Figure 2. Top view of the UOBCE

of soil. The hopper movement can be controlled with the aid of a rotating drum (Figure 1) which was operated using an electrical motor. The movement of the hopper was monitored in the vertical direction using a side crank and cables on four rollers at the top of the frame.

(vii) The maximum density index, D_r value that can be achieved in the tank with respect to the height of free fall of the soil has been determined using the V-hopper. A free height of fall of 1 m was found to provide maximum density index to the sand in the test tank. For this reason, the height of free fall of soil was fixed at 1 m from the hopper. The soil was placed in the tank in 50 mm lifts. This hopper can also be moved horizontally from left to right and in reverse directions using a side motor with a horizontal chain.

(viii) The front face of the test tank constituted of a transparent acrylic plate. The acrylic plate acted as a window to allow observation of the water table level and marking the thickness of the soil layers during the installation and to empty the soil quickly from the tank after the completion of each test. The acrylic window which is 25 mm thick is fastened with C-clamps to the tank as shown in Figure 1, item 15.

(ix) Linearly Variable Displacement Transducer (LVDT) was attached to the vertical loading arm and the tip of the LVDT was placed directly on the surface of the model footing. A load cell capable of measuring 15 kN was mounted on the loading arm. Both the LVDT and the load cell were connected to a data acquisition system (DAS) as shown in Figure 1.

(x) A piezometer was used to monitor the level of the water table in the tank, which is on left side of the test tank (item 5 in Figure 1). Commercial Tensiometers were installed after saturating the ceramic tips and located at different depths as shown in Figure 2.

(xi) The base of the equipment was connected to the loading frame using steel angles to ensure the stability during the loading of the footing. No movement or sway of the loading frame was observed during the loading process. A 50 mm thick layer of clean aggregate was laid on the base area of the test tank and a thin geotextile sheet was placed on top of the aggregate to function as a porous barrier between soil and the aggregate. The objective of this layer is to facilitate free and gradual movement of water in the test tank in order to achieve uniform saturation or de-saturation conditions as desired by the testing requirements. Drainage pipes with valves connected to the bottom of the test tank were used in monitoring the water level. A water supply pipe of 20 mm in diameter was used to control the amount of water supplied to the tank. This main water supply pipe branches in to 4 smaller pipes of 12.5 mm which facilitate to saturate the soil gradually and uniformly from the base of the tank to the surface (i.e., the saturation was progressed from the bottom to the top of the soil surface). Both saturation and desaturation conditions were achieved successfully in the tank using this system.

4. MATERIAL DESCRIPTION AND PROPERTIES

Dry sieve analysis was performed using ASTM standard test method D422-63 (ASTM 1997).

Table 1. Soil description and properties

Property	Description or Value
Specific gravity, G_s	2.65
D_{60} (mm)	0.22
D_{30} (mm)	0.18
D_{10} (mm)	0.12
Coefficient of uniformity, C_u	1.83
Coefficient of curvature, C_c	1.23
Average dry density of the compacted soil in the tank, kN/m³	16.05
Void ratio, e	0.62 - 0.64
Unified soil classification system	SP

The grain size distribution of the soil from this analysis is shown in Figure 3. The soil was classified in accordance with the USCS as poorly graded sand (SP). The soil used in the study had 5% of silt. Table 1 summarizes the properties of the soil.

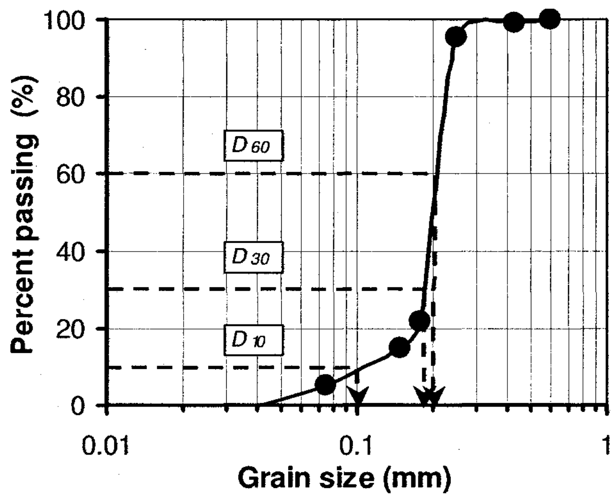


Figure 3. Grain size distribution of the sand used in the study

5. TEST PROCEDURE

5.1 General

The failure mode of the bearing capacity of soils is dependent on the density index, D_r value (i.e., relative density). The density index should be 70% for a surface footing to fail under general shear failure mode (Vesic 1973 and Das 2004).

The highest density index, D_r value that was achieved using the hopper as discussed earlier in this paper for the soil used in the study was 55%. For this reason, the soil was further compacted using a 5 kg hand compactor. The average density index value that was achieved in this study was equal to 64%. The density index was carefully controlled for all the tests to ensure identical density index values. The density index values were verified by collecting soil samples in Aluminum cups (with small perforations) placed at different levels in the tank. These cups were also used to measure the water content values. The data collected using this technique is tabulated in Table 2.

A number of experiments were performed to measure the bearing capacity on this compacted coarse-grained soil under different conditions. The first series of tests performed under fully saturated condition (i.e., zero suction), and the second series of tests were conducted under unsaturated conditions for three different average suction values (i.e., 2 kPa, 4 kPa and 6 kPa). A minimum of three tests were conducted and average values are reported for the measured bearing capacity.

5.1.1 Tests under fully saturated condition

The water table was slowly raised from the base of the tank through the bottom aggregate layer. This technique facilitated escape of air from bottom to the surface layers of the soil in the tank gradually to ensure a fully saturated condition. The adjustments of the water level were inspected periodically in the piezometers. The supplier valve was closed once the water level reached the soil surface in the tank. The bearing capacity of the saturated soil was measured by loading the footing gradually at a rate of 1.2 mm/min. All Tensiometers were indicating zero suction values after saturation and during the testing period.

5.1.2 Tests under unsaturated conditions

The soil in the tank was first saturated as detailed in the previous section. The water table was then lowered down (using drainage valves) to different levels of depth from the soil surface to achieve different suction values. Equilibrium conditions with respect to suction value in the stress bulb zone (i.e., depth of 1.5B) were typically achieved in a time period of 24 hours. The bearing capacity of the coarse-grained compacted soil using this technique was measured under different average suction values (i.e., 2, 4 and 6 kPa). While the suction values were measured using the Tensiometers, the gravimetric water contents were determined approximately at the same levels collecting soil specimens in small aluminum cups. These cups with perforations were embedded in the unsaturated soil below footing and close to each ceramic-tip of the Tensiometers. Figure 4 shows the cross section of the tank in a schematic form and provides the details of the placement of Tensiometers and the aluminum cups at different depths (labeled as C_{1-1}, C_{2-1}, C_{3-1} and C_{4-1} ...etc). This figure also shows the variation of suction with respect to depth in the tank. Table 2 summarizes the data of a set of results in which the average suction value in the stress bulb zone was 6 kPa.

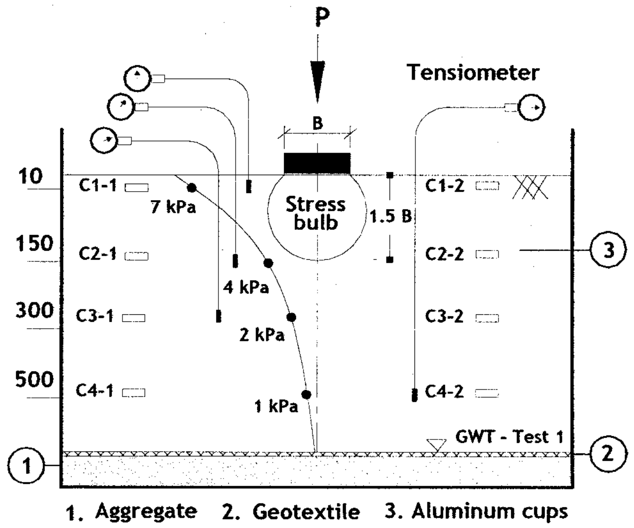
Table 2. Typical data from the test tank for an average suction value of 6 kPa in the stress bulb zone.

D[1] (mm)	γ_t (kN/m³)	γ_d (kN/m³)	e	w (%)	S (%)	AVR $(u_a - u_w)$ (kPa)
10	18.17	15.94	0.63	14.0	58	6
150	18.75	15.85	0.64	18.3	76	4
300	19.27	16.07	0.62	20.0	86	2
500	19.40	15.74	0.63	23.2	98	1
600	19.74	15.95	0.63	23.8	100	0

[1] AVR: Average

where:

D : depth from the soil surface of tank (mm)
γ_t : total unit weight (kN/m³)
γ_d : dry unit weight (kN/m³)
e : void ratio
w : water content (%)
S : degree of saturation (%)
$(u_a - u_w)$: matric suction (kPa)



(Note: all dimensions in mm)

Figure 4. Schematic showing the cross-section details of the test tank

5.2 Bearing capacity tests

Figure 5 presents the typical stress versus settlement for the tests using 100 mm x 100 mm model footing in the laboratory. The suction values for each test represent an average value of capillary stresses in the proximity of the stress bulb that constitutes a depth of 1.5 times the width of the square footing (Poulos and Davis 1974). The failure mechanism is close to the general shear failure mode with clear well defined failure loads.

From the experimental results it can be seen that the contribution of the matric suction to the bearing capacity is substantial.

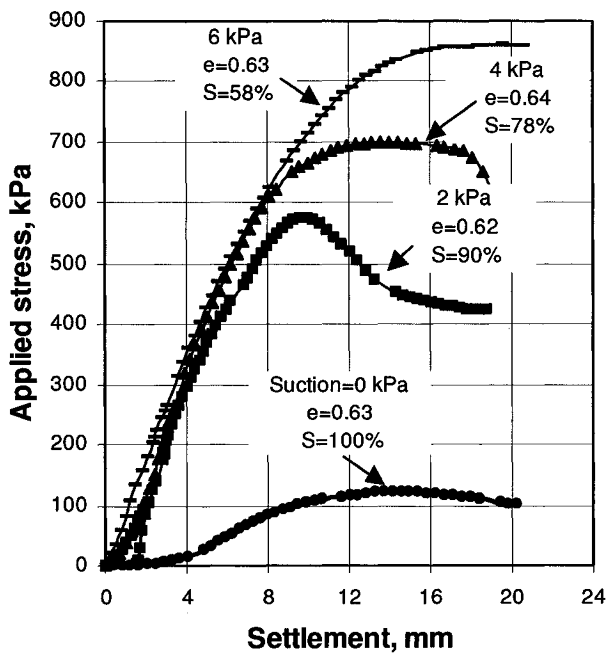


Figure 5. Stress versus settlement relationship for 100 mm x 100 mm model footing

6. DETERMINATION OF THE SOIL WATER RETENTION CURVE (SWRC)

The SWRC relationship is plotted using three different methods. Figure 6 shows the soil-water retention curve (drying curve) plotted as a relationship between the degree of saturation, *S* and the matric suction, $(u_a - u_w)$ using three different methods. The objective of the determination of SWRC was to understand its relationship with the bearing capacity of unsaturated soils similar to the shear strength of unsaturated soils.

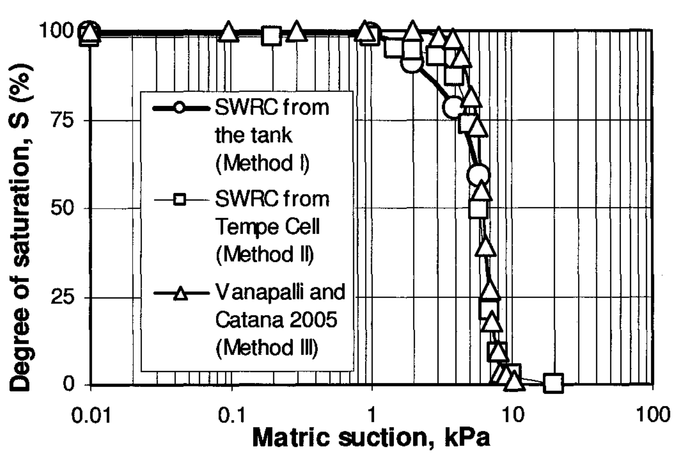


Figure 6. Measured and predicted SWRCs for the tested soil

The first method for obtaining the SWRC was from the direct measurements from the test tank. In order to obtain the SWRC relationship from the test tank, the water content measurements were determined from the test tank using small cups as detailed in section 5.3 (using the first method). As detailed earlier, Tensiometers were used for measuring the matric suction. The gravimetric water content values were determined at four different average suction values (i.e., 1, 2, 4 and 6 kPa) after attaining equilibrium matric suction values in the tank. The saturated water content was also measured from soil specimens in aluminum cups (without perforations) under the water table level when all the Tensiometer readings were reporting zero suction values.

The second method is the direct measurement of the SWRC using the Tempe cell apparatus in the laboratory (Figure 7). The setup details for measuring the SWRC in are shown in this figure. A pressure gauge with a sensitivity of measuring values of 0.2 kPa is connected to a pressure regulator. The drained water from the Tempe cell for different values of suction was collected in a bottle and its mass measured directly using an electronic scale.

The third method was a one-point prediction method following the procedures from Vanapalli and Catana (2005). This procedure can be used to estimate the SWRC for coarse-grained soils using parameters derived from the grain size distribution and volume mass properties along with one measured point of matric suction versus gravimetric water content. For estimating the SWRC, the data point of water content and suction was 4 kPa and 18.3% respectively was used.

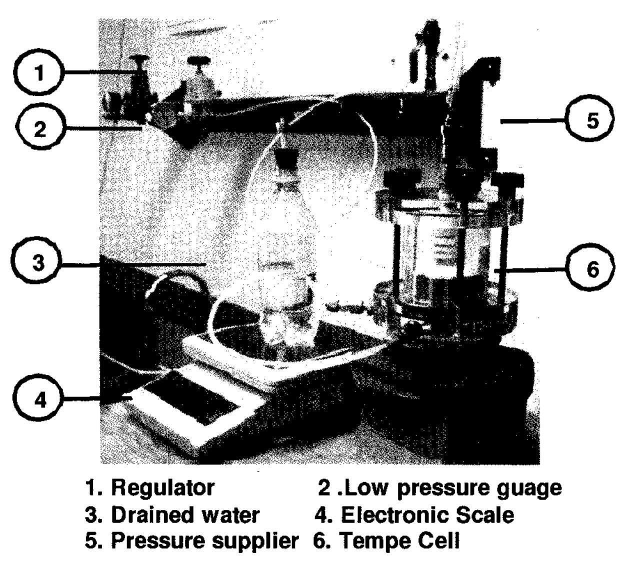


Figure 7. Tempe cell apparatus for measuring the SWRC

There is a good comparison between the SWRCs using all the three different methods. The air-entry value is approximately in the range of 2.5 to 3 kPa from all the three methods. There is a steep transition zone in the suction range of 3 to 10 kPa (Figure 6). Such a behavior is consistent with the nature of the poorly graded sand used in the research study.

7. DISCUSSIONS OF TEST RESULTS

The relationship between the applied stress and settlement of typical experimental results for a 100 mm x 100 mm model footing is shown in Figure 5. It can be observed that the ultimate bearing capacity for all model footings in unsaturated condition is higher than the bearing capacity of the same model footing in fully saturated condition. The SWRC (plotted on an arithmetic scale) and variation of the bearing capacity with respect to matric suction for two model footings of different sizes (i.e., 100 mm x 100 mm and 150 mm x150 mm) is shown in Figure 8. This relationship demonstrates that there is a significant increase in the bearing capacity of the model footing due to the contribution of suction in the range 0 to 6 kPa (i.e., the analysis is based on the average suction value in the proximity of the stress bulb) for the tested coarse-grained soil. The results also suggest the bearing capacity approximately increases linearly with matric suction up to the air-entry value. There is a non-linear increase in the bearing capacity with respect to matric suction beyond the air-entry value. The trends of the results of the bearing capacity of an unsaturated soil are similar to the shear strength behavior of unsaturated soils. For this reason, it will be useful to propose prediction procedures for estimating the bearing capacity of unsaturated soils along similar lines as the shear strength of unsaturated soils using the SWRC and the effective shear strength parameters (i.e., c' and ϕ').

The bearing capacity for the compacted unsaturated coarse-grained soil measured in this study was observed to be 5 to 7 times higher than the ultimate bearing capacity of the same soil under saturated condition.

The results of this study are consistent with the observations of Steensen-Bach et al. (1987) who reported the bearing capacity of a coarse grained unsaturated soil to be 4 to 6 times higher than the bearing capacity of the same soil under saturated conditions. Nabil (1985) studies also show that the measured bearing capacity of moist coarse grained was 3 to 5 times higher than the bearing capacity determined using Terzaghi's equation (1943) and 2 to 3.5 times higher than Meyerhof's equation (1956).

8. SUMMARY AND CONCLUSIONS

In this paper, an experimental program has been carried out to determine the bearing capacity of a saturated and unsaturated compacted coarse-grained soil using specially designed equipment in the laboratory environment. The experimental studies demonstrate that the matric suction has a significant influence on the bearing capacity of the tested compacted, coarse-grained unsaturated soil.

The bearing capacity of the soil under unsaturated conditions increases almost linearly up to the air-entry value. There is a non-linear increase in the bearing capacity beyond the air-entry value. Despite the fact that the tested soil is a coarse-grained soil, the experimental results suggest a significant contribution of the matric suction towards the bearing capacity.

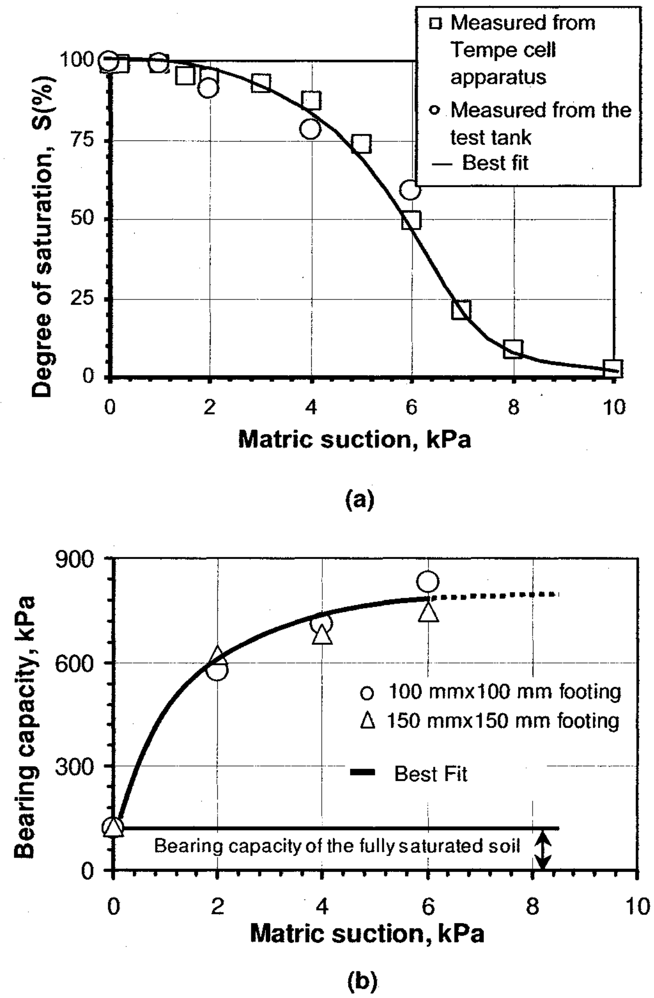


Figure 8. (a) Measured SWRCs from test tank and Tempe cell and (b) Variation of bearing capacity with respect to matric suction

The measured bearing capacity of the compacted coarse-grained soil under unsaturated conditions was found to be 5 to 7 times higher than the bearing capacity under fully saturated conditions. In addition, these results show that there is a strong relationship between the SWRC and the bearing capacity of the tested coarse-grained soil.

The results of this experimental program suggest the conventional bearing capacity theory used in the engineering practice is highly conservative when it is applied for unsaturated soils.

9. ACKNOWLEDGMENTS

The first author thanks the General Public Committee of Higher Education in Libya (G.P.C.H.E) for providing the financial support during this research. The authors would also like to acknowledge the funding received from National Sciences and Engineering Research of Canada (NSERC) for this project. Thanks also go to John Perrins, senior engineer in the university of Ottawa, Faculty of Engineering Student Workshop and Julio Ángel Infante Sedano, graduate student at the University of Ottawa for their assistance.

References

ASTM (1997). Standard classification of soils for engineering purposes (unified classification system, D2487-93). In 1997 Annual Book of ASTM Standards, Vol. 04.08. American Society for Testing and Materials (ASTM) Philadelphia, PA. : 217 – 227.

Bishop, A. W. and Blight, G. E. (1963). Some aspects of effecive stress in saturated and unsaturated soils. *Geotechnique*, Vol. 13, No. 3: 177 – 197.

Bolton, M. D. and Lau, C. K. (1993). Vertical Bearing Capacity Factors for Circular Footings on Mohr-Coulomb Soil. *Canadian Geotechnical Journal*, 30: 1024-1033.

Bowles, J. E. (1996). *Foundation Analysis and Design*. 5th ed. McGaw – Hill, NY, USA.

Broms, B. B., (1964). The Effect of Degree of Saturation on the Bearing Capacity of Flexible Pavements. *Highway Research Recoed*, No. 71: 1 – 14.

Costa, Y.D, Cintra J. C. And Zornberg J. G. (2003). Influence of Matric Suction on the Results of Plate Load Tests Performed on a Lateritic Soil Deposit. *Geotechnical Testing Journal*, Vol. 26, No. 2: 219 – 226.

Das, B. M. (2004). *Principles of Foundation Engineering*. 5th ed., Brooks, CA, USA.

De Beer, E. E. and Ladanyi, B. (1961). Etude expérimental de la capacité portante du sable sous des foundations circulares établies en surface. *Proceeding of the 5th International Conference on Soil Mechanics and Foundation Engineering*, Paris, France, Vol. 1: 577– 581.

De Beer, E. E. (1965). Bearing Capacity and Settlement of Shallow Foundations on Sand. *Proceedings of a Symposium on Bearing Capacity and Settlement of Foundations*, Duck University, Duhram: 15 – 32.

Fredlund, D. G. and Morgenstern, N. R., (1977). Stress State Variables for Unsaturated Soils. *Journal of the Geotechnical and Engineering Division*, ASCE, 103(GT5):447 – 466.

Fredlund, D. G. and Rahardjo H. (1993). *Soil Mechanics for Unsaturated Soils*, 1st ed., Wiley, New York.

Lawrence, A. L. (1968). Theoretical Bearing Capacity of very Shallow Footings. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 94, No. (SM6): 1347 – 1357.

Meyerhof, G. G. (1956). Penetration Tests and Bearing Capacity of Cohesionless Soils. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 82, No. (SM1): Paper: 866.

Meyerhof, G. G. (1965). Shallow Foundations. *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. 91, No. (SM2): 21 – 31.

Miller, G. A. and Muraleetharan, K. K. (1998). In Situ Testing in Unsaturated Soil. *Proceedings of the 2nd International Conference on Unsaturated Soils*, Beijing, China, Vol. 1: 416 – 421.

Nabil, F. I. (1985). Allowable Pressure from Loading Tests on Kuwaiti Soils. *Canadian Geotechnical Journal*, 22: 151 – 157.

Oloo, S.Y. Fredlund, D.G., and Gan, J.K-M. (1997). Bearing Capacity of Unpaved Roads. *Canadian Geotechnical Journal*, 34: 398 – 407.

Poulos, H. D. and Davis, E. H. (1974). *Elastic Solutions for Soil and Rock Mechanics*, John Wiley and Sons, NY, USA.

Prandtl, L. (1921). Eindringunsfestigkeit und Festigkeit von Schneiden. Z. Angew. Math. Mech. 1(4): 15 – 20.

Steensen-Bach J.O., Foged, N. and Steenfelt, J.S. (1987). Capillary Induced Stresses – Fact or Fiction? *9th ECSMFE, Groundwater Effects in Geotechnical Engineering*, Dublin: 83 – 89.

Silvestri, V. (2003). A Limit Equilibrium Solution for Bearing Capacity of Strip Foundations on Sand. *Canadian Geotechnical Journal*, 40: 351 – 361.

Terzaghi, K. (1943). *Theoretical Soil Mechanics*, John Wiley and Sons. New York, NY, USA.

Terzaghi, K. and Peck, R. B. (1948). *Soil Mechanics in Engineering Practice*, Asia Publishing House.

Vanapalli, S. K. and Catana, M. C. (2005). Estimation of the Soil Water Characteristic Curve of Course-grained Soil Using one Point Measurement and Simple Properties. *Proceedings of an International Symposium on Advanced Experimental Unsaturated Soil Mechanics*: 401 – 410.

Vesić, A. S. (1963). Bearing Capacity of Deep Foundations in Sand. *Highway Research Record*, No. 39: 112 – 153.

Vesić, A. S. (1973). Analysis of Ultimate Loads of Shallow Foundations. *Journal of the Soil Mechanics and Foundation Division*, ASCE, Vol. 99, No. (SM1): 45 – 73.

Bearing Capacity of Model Footings in Unsaturated Soils

S. K. Vanapalli[1] and F. M. O. Mohamed[2]

[1] Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada
vanapall@eng.uottawa.ca
[2] Department of Civil Engineering, University of Ottawa, Ottawa, Ontario, Canada
fmoha035@uottawa.ca

Summary. A simple technique is proposed to predict the variation of the bearing capacity of an unsaturated soil with respect to matric suction. This technique is based on extending conventional bearing capacity theory proposed by Terzaghi. The proposed equation in this paper is presented as a functional relationship such that the variation of the bearing capacity of an unsaturated soil with respect to matric suction can be predicted. This technique is developed extending the concepts for predicting the shear strength of unsaturated soils proposed by Vanapalli et al. (1996). Using the approach presented in this paper, the bearing capacity of an unsaturated soil can be predicted using the saturated shear strength parameters, c' and ϕ' and the soil-water retention curve (SWRC).

Key words: Bearing capacity, unsaturated soils, shear strength, matric suction, soil-water retention curve, model footings

Introduction

The bearing capacity is one of the key parameters required in the design of shallow foundations. Several approaches are available in the literature for determination of the bearing capacity of soils based on the saturated shear strength parameters (Terzaghi, 1943; Meyerhof, 1951). However, in several situations, shallow foundations are located above the ground water table where the soil is typically in a state of unsaturated condition. Nevertheless, the bearing capacity of soils is commonly determined assuming fully saturated conditions ignoring the influence of capillary stresses or the matric suction. Due to this reason, estimation of the bearing capacity of shallow foundations using the conventional approaches may not be reliable leading to uneconomical designs.

Foundation designs for unsaturated soils are complex and require not only the soil-structure interaction but also a fundamental understanding of soil behaviour that comprises the combined role of suction and cementation (Schnaid et al., 1995). Several researchers carried out investigations on the bearing capacity of unsaturated soils (Broms, 1963; Steensen-Bach et al., 1987; Oloo, 1994; Miller and Muraleetharan, 1998; Costa et al., 2003; Mohamed and Vanapalli, 2006). All these studies have shown significant contribution of matric suction to the bearing capacity of unsaturated soils. However, limited theoretical research work is reported in the literature with respect to the interpretation of the bearing capacity of unsaturated soils (Fredlund and Rahardjo, 1993; Oloo et al., 1997).

Based on the results presented in this study, a semi-empirical equation is proposed to predict the variation of the bearing capacity of unsaturated soils using the saturated shear strength parameters, c' and ϕ' and the SWRC. The equation presented in this paper is developed extending the concepts for predicting the shear strength of unsaturated soils proposed by Vanapalli et al. (1996). The proposed equation is also extended for other model footings studies reported in the literature that includes both coarse-grained and fine-grained soils. The studies presented in this paper show that there is a good comparison between the measured and predicted bearing capacity of model footings.

Test Equipment

Figure 1 shows the details of equipment specially designed at the University of Ottawa for determining the bearing capacity of coarse-grained soils using model footings. All the key features of this equipment are summarized in Mohamed and Vanapalli (2006). This equipment has special provisions to achieve fully saturated and unsaturated conditions of the compacted sand in the test tank. While the water table level in the test tank can be adjusted to the desired level using drainage valves, the capillary tension (i.e., matric suction) variation with respect to depth in the unsaturated soil zone below the model footing can be measured using commercial Tensiometers.

Properties of the Tested Soil

The properties of the soil used in this study were determined in the geotechnical laboratory of the University of Ottawa. The soil was classified using *USCS* as poorly graded sand (SP). The average void ratio after compaction was 0.63 and the dry unit weight was $16.02\,\mathrm{kN/m^3}$. The internal friction angle was 35.3° from direct shear tests. Bolton (1986) studies on sands show that the effective stress and soil density affect the rate of dilatancy of soils and thereby influence the internal friction angle, ϕ'. As the dilatancy of sands has

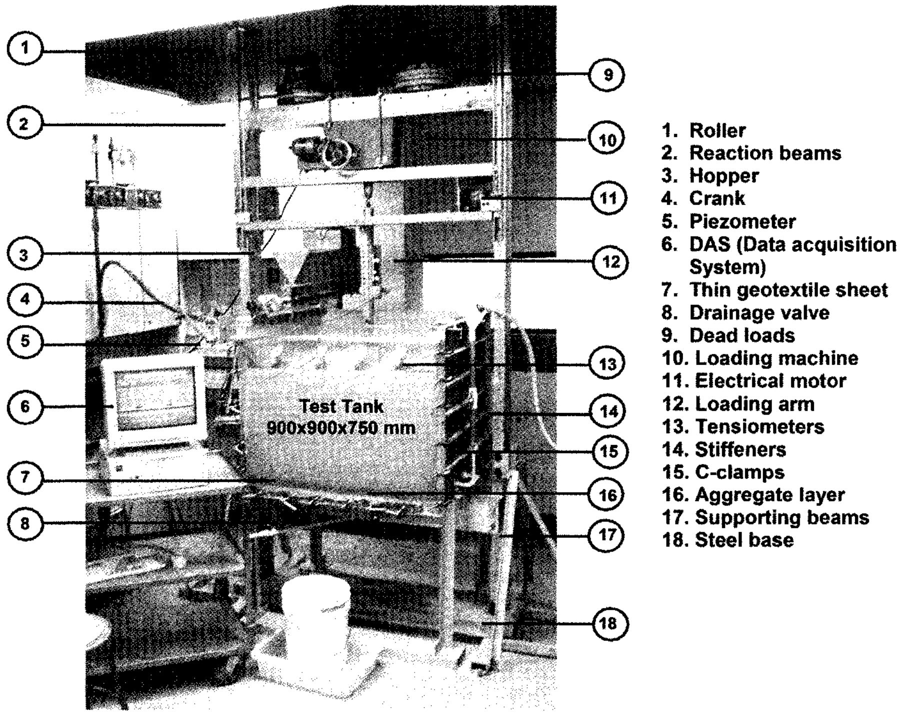


Fig. 1. University of Ottawa Bearing Capacity Equipment (UOBCE)

a significant effect on the ϕ' value, it was suggested that the measured ϕ' values can be modified as $\phi' = (\phi' + 0.8$ of dilatancy angle). Some investigators have found that better comparisons can be provided between the measured and computed bearing capacity values by using ϕ' values about 10 to 15% higher than the measured values (Steensen-Bach et al., 1987). Therefore, ϕ' value which is 10% higher than the measured value (i.e., 39°) was used in this study.

Typical Experimental Results

The relationship between the applied stress and settlement of typical experimental results on a 100 mm × 100 mm square model footing is shown in Fig. 2. This relationship demonstrates that there is a significant increase in the bearing capacity of the model footing due to the contribution of matric suction in the range 0 to 6 kPa for the tested compacted, coarse-grained soil. The analysis presented in this paper is based on the average matric suction value in the proximity of the stress bulb. The procedure used for the determination of the average suction value is detailed using Fig. 3.

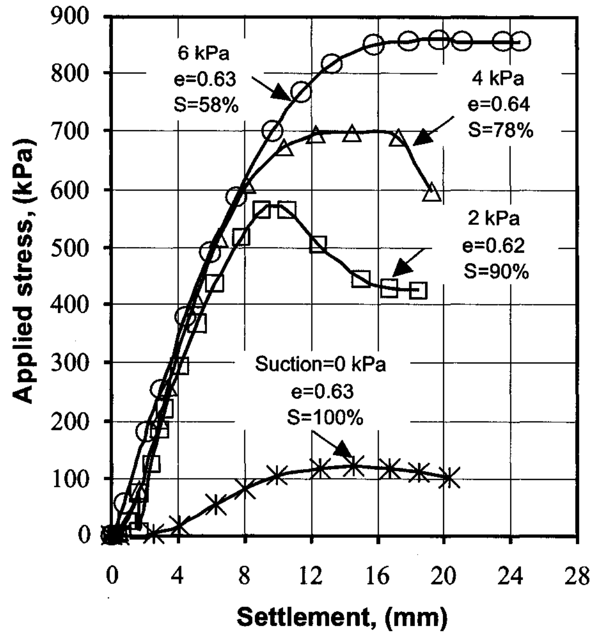


Fig. 2. The relationship between the applied stress versus settlement for 100 mm × 100 mm square model footing

The variation of the matric suction with respect to depth underneath the model footing is non-linear as shown in Fig. 3 (left hand side). The variation of matric suction in the capillary zone above the ground water table (GWT) is typically hydrostatic (see right hand side of Fig. 3) for coarse-grained soils. The experimental results supported this generally observed behavior and indirectly demonstrated that the Tensiometers performed well. As the significant soil stresses are typically distributed over a depth of $1.5B$ (Poulos and Davis 1974 and Chen 1999) the matric suction value is considered as the average value of $(u_a - u_w)_1$ (matric suction close to the surface of the footing) and $(u_a - u_w)_2$ (matric suction value at the bottom of the stress bulb) as shown in Fig. 3.

The Measured and the Predicted SWRC

Figure 4 shows the soil-water retention curve (drying curve) plotted as a relationship between the degree of saturation, S and the matric suction, $(u_a - u_w)$ using three different methods. Two direct methods were used for measuring the SWRC's. The first method constituted the measurement of the SWRC directly from the test tank. The SWRC was also measured using the Tempe cell in the laboratory, which formed the second method. More details of the procedures used in the determination the SWRC are available in Mohamed and

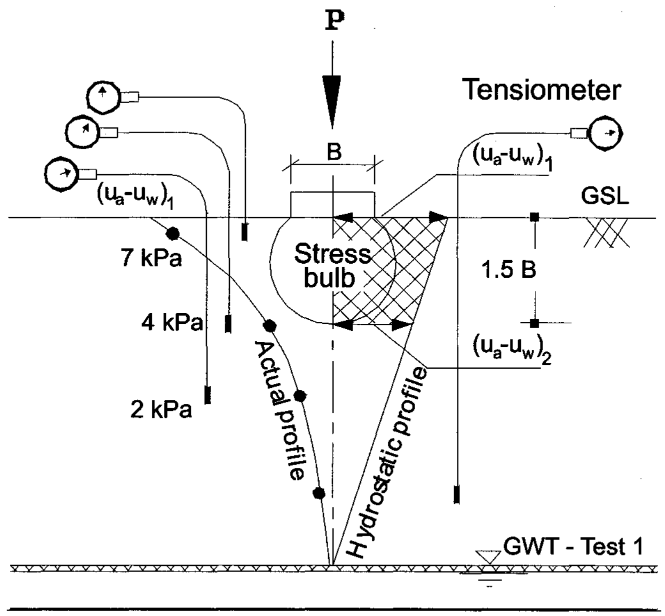


Fig. 3. Schematic to demonstrate the procedure used for determining the average matric suction below the footing

Vanapalli (2006). The third method for estimation of the SWRC was based on the procedure summarized in Vanapalli and Catana (2005). This procedure uses one measured point (i.e., water content and matric suction) along with data obtained from the grain size distribution curve. Figure 4 shows that there is a good agreement between the SWRC's using all the three methods. The objective of the determination of the SWRC was to understand its relationship with the bearing capacity of unsaturated soils similar to the shear strength of unsaturated soils.

Bearing Capacity of Unsaturated Soils

Terzaghi (1943) proposed an equation for computing the bearing capacity of shallow strip footings extending Prandtl (1921) assumptions for the soil failure mechanism. This equation is valid for strip footings resting in a homogenous soil and subjected to vertical loading.

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

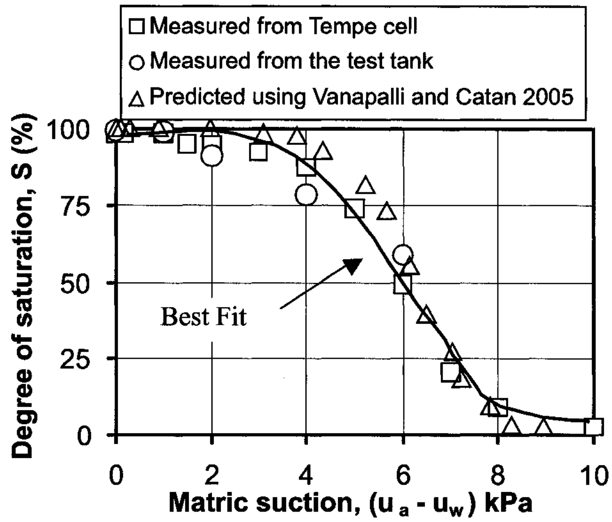


Fig. 4. Measured SWRC from the Tempe cell and test tank along with the estimated SWRC

where:
q_u = ultimate bearing capacity, kPa
q = overburden pressure, kPa
c' = effective cohesion, kPa
N_c, N_q, N_γ = bearing capacity factors due to cohesion, surcharge and unit weight, respectively
γ = soil unit weight, kN/m^3
B = footing width, m

Equation (1) can be written as given below for interpreting the bearing capacity of surface footings taking account of the influence of the shear strength contribution due to matric suction for unsaturated soils:

$$q_u = [c' + (u_a - u_w)\tan\phi^b]N_c + 0.5B\gamma N_\gamma. \tag{2}$$

Vanapalli et al. (1996) proposed a relationship for estimating the shear strength contribution with respect to matric suction, $\tan\phi^b$ using the SWRC and the saturated shear strength parameters as $\tan\phi^b = S^\kappa \tan\phi'$, where S is the degree of saturation. This term, $S^\kappa \tan\phi'$, takes account of the non-linear variation of the shear strength of unsaturated soils using a fitting parameter, κ. Extending the concepts of shear strength of unsaturated soils proposed by Vanapalli et al. (1996), equation (2) can be used to predict the bearing capacity of unsaturated soils which desaturate on application of matric suction

$$q_u = [c' + (u_a - u_w)S^\psi \tan\phi']N_c\xi_c + 0.5\gamma BN_\gamma\xi_\gamma \tag{3}$$

where:
ψ = bearing capacity fitting parameter
ξ_c, ξ_γ = shape factors due to cohesion and unit weight (from Vesić 1973)

The fitting parameter, ψ used in equation (3) is referred as a bearing capacity fitting parameter in the remainder of the paper. The bearing capacity contribution due to matric suction can be obtained from a part of equation (3), which is equal to $(u_a - u_w)S^\psi \tan\phi'$. This is similar to predicting the shear strength contribution due to matric suction using the expression $[(u_a - u_w)S^\kappa \tan\phi']$ from Vanapalli et al. (1996). Similar to prediction of the shear strength of unsaturated soils, the contribution of matric suction to the bearing capacity can be determined using the SWRC.

The philosophy of using the fitting parameter, ψ in the bearing capacity of unsaturated soils is similar to using the fitting parameter, κ for predicting the shear strength of unsaturated soils. Due to this reason, the limitation of using equation (3) in the prediction of the bearing capacity of unsaturated soils is similar to using the equation proposed by Vanapalli et al. (1996) for the prediction of the shear strength of unsaturated soils. In other words, equation (3) can only be used for predicting the bearing capacity when the experimental results are available. To alleviate such a limitation for the shear strength of unsaturated soils, Vanapalli and Fredlund (2000) and Garven and Vanapalli (2006) provided a relationship between κ versus plasticity index, I_P for predicting the shear strength of unsaturated soils. Such a relationship will be useful to obtain fitting parameter value from the plasticity index, I_P of the soil.

In the present study, the same philosophy has been extended to propose a relationship between the bearing capacity fitting parameter, ψ and plasticity index, I_P such that the bearing capacity of unsaturated soils can be predicted without the experimental results.

Equation 3 will take the form as equation 4 for interpreting the experimental results of the square model footing by including Vesić (1973) shape factors. Close observation of this equation shows that the equation has been modified to take account of the air-entry value, $(u_a - u_w)_b$. Up to the air-entry value, the contribution of matric suction to the bearing capacity is equal to $(u_a - u_w)_b(\tan\phi' - S^\psi \tan\phi')$. The form of this term is similar to $(u_a - u_w)_b(\tan\phi' - \tan\phi^b)$, which was derived by Oloo (1994). The bearing capacity contribution due to matric suction $\tan\phi^b$ in this term has been replaced with the term $S^\psi \tan\phi'$ (see equation 4):

$$q_u = [c' + (u_a - u_w)_b(1 - S^\psi)\tan\phi' + (u_a - u_w)_{\mathrm{AVR}} S^\psi \tan\phi'] \times N_c\left[1.0 + \left(\frac{N_q}{N_c}\right)\left(\frac{B}{L}\right)\right] + 0.5B\gamma N_\gamma\left[1.0 - 0.4\left(\frac{B}{L}\right)\right] \quad (4)$$

where
$(u_a - u_w)_{\mathrm{AVR}} = \frac{1}{2}[(u_a - u_w)_1 + (u_a - u_w)_2]$ as defined in Fig. 3,
$(u_a - u_w)_b$ = air entry value, kPa.

The bearing capacity factors due to cohesion, N_c, surcharge, N_q and unit weight, N_γ were developed by several researchers (Terzaghi, 1943; Meyerhof, 1951; Vesić, 1973; Kumbhokjar, 1993). The bearing capacity factors of N_c and N_q proposed by most of the investigators are approximately the same. For this reason, the bearing capacity factors, N_c and N_q originally proposed by Terzaghi (1943) using the limit equilibrium method were used in the analysis. There is no general consensus with respect to the bearing capacity factor due to unit weight, N_γ. The N_γ values proposed by various investigators are significantly different (Terzaghi, 1943; Meyerhof, 1951; Vesić, 1973). Kumbhokjar (1993) has undertaken an extensive study and proposed N_γ values based on numerical analysis which are relatively higher in comparison to other N_γ values reported in the earlier literature. This study also shows that bearing capacity computations provide better comparisons with the measured bearing capacity values using the proposed N_γ values. For this reason, the bearing capacity factor, N_γ values proposed by Kumbhokjar (1993) are used in this study.

Comparison between the Measured and Predicted Bearing Capacity of Unsaturated Soils

Equation (4) is used in the prediction procedure to provide comparisons with the measured values of bearing capacity for typical model square footings. The model footings were subjected to static vertical loads in the UOBCE both under saturated and unsaturated conditions. Different values of capillary suction (i.e., matric suction) were achieved by varying the water table level in the test tank (Fig. 1).

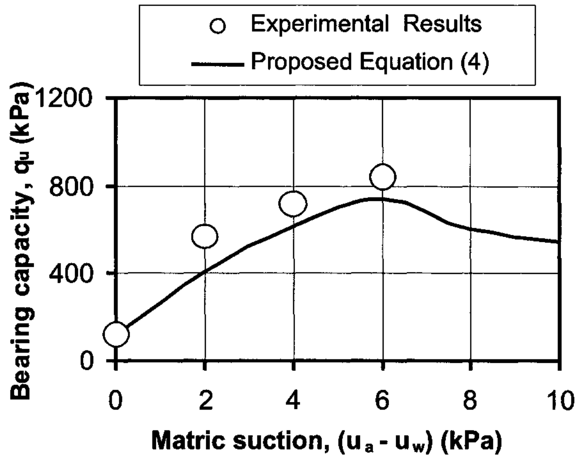


Fig. 5. Comparison between the measured and predicted bearing capacity versus matric suction for 100 mm × 100 mm footing

Figure 5 shows comparisons between the measured and predicted values of the variation of bearing capacity with respect to matric suction for the compacted coarse-grained sand studied. There is a reasonably good comparison between the measured and predicted bearing capacity values. However, the predicted values are slightly lower than the measured bearing capacity values. These differences may be attributed to the assumption used in the procedure for the estimation of the average matric suction values below the footing, which typically results in the lower contribution of matric suction compared to the actual value. Similar trends were observed for the measured and predicted values of bearing capacity for four other soils data from the literature. For this reason, the proposed prediction procedure may be summarized to be conservative as the measured bearing capacity is slightly higher than the predicted bearing capacity.

All the above observations are derived from the results of model footings tested in a laboratory environment. More details of the analysis and comparisons between the measured and predicted bearing capacity of the other four soils could not be provided in this paper due to space limitations.

The bearing capacity fitting parameter, ψ value was equal to 1 for all the three sandy soils studied and higher values were required for the other fine-grained soils. Based on results of the study undertaken through this research program on five soils, a relationship was developed between the bearing capacity fitting parameter, ψ and the plasticity index, I_P (Fig. 6 and equation (5))

$$\psi = 1.0 + 0.34 I_P - 0.0031 I_P^2. \tag{5}$$

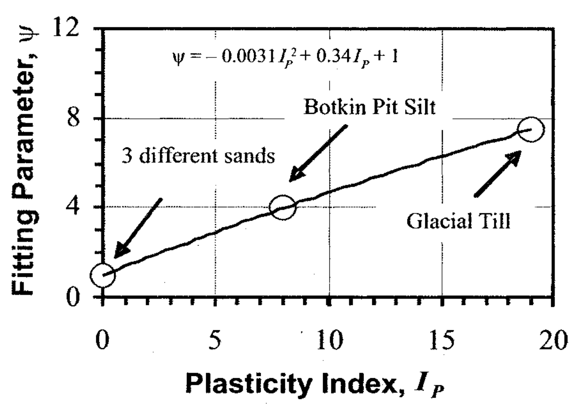


Fig. 6. Relationship between the bearing capacity fitting parameter, ψ and plasticity index, I_p

Conclusions

In this paper, a simple technique is proposed for predicting the bearing capacity of unsaturated soils using the saturated shear strength parameters, c' and ϕ' and the SWRC. The results of the study suggest that there is a good comparison between the measured and predicted bearing capacity values. The framework is based on studies undertaken on model footings and shows considerable promise for extending it to field studies.

Acknowledgements

The authors thank the General Public Committee of Higher Education in Libya (G.P.C.H.E) for providing the financial support to F.M.O. Mohamed during this research program. The authors would like to acknowledge the funding received from National Sciences and Engineering Research of Canada (NSERC) for this project. Thanks also go to C. Catana, graduate student from the Department of Civil Engineering, University of Ottawa for his assistance.

References

Bolton MD (1986) The Strength and Dilatancy of Sands, Geotechnique 36(1):65–78

Broms BB (1963) The Effect of Degree of Saturation on the Bearing Capacity of Flexible Pavements, Highway Research Record 71:1–14

Chen FH (1999) Soil Engineering, Testing, Design and Remediation. CRC Press LLC, N.W., Boca Raton, Florida, USA

Costa YD, Cintra JC, Zornberg JG (2003) Influence of Matric Suction on the Results of Plate Load Tests Performed on a Lateritic Soil Deposit. Geotechnical Testing Journal, 26(2):219–226

Fredlund DG, Rahardjo H (1993) Soil Mechanics for Unsaturated Soils, 1st Ed. Wiley, New York

Garven E, Vanapalli SK (2006) Evaluation of Empirical Procedures for Predicting the Shear Strength of Unsaturated Soils. Proceedings of the Fourth International Conference on Unsaturated Soils, Carefree, Arizona, American Society of Civil Engineers Geotechnical Special Publication 147(2):2570–2581

Kumbhokjar AS (1993) Numerical Evaluation of Terzaghi's N_γ. Journal of Geotechnical Engineering, American Society of Civil Engineers 1999(3):598–607

Meyerhof GG (1951) The Ultimate Bearing Capacity of Foundations, Geotechnique 2:301–332

Miller GA, Muraleetharan KK (1998) In Situ Testing in Unsaturated Soil. Proceedings of the Second International Conference on Unsaturated Soils, Beijing, China, Vol. 1:416–421

Mohamed FMO, Vanapalli SK (2006) Laboratory Investigations for the Measurement of the Bearing Capacity of an Unsaturated Coarse-Grained Soil. 59^{th} Canadian Geotechnical Conference, BC, Vancouver (in print)

Oloo SY (1994) A Bearing Capacity Approach to the Design of Low Volume Traffic Roads. Ph.D. Thesis, University of Saskatchewan, Canada

Oloo SY, Fredlund DG, Gan JK-M (1997) Bearing Capacity of Unpaved Roads. Canadian Geotechnical Journal 34:398–407

Poulos HD, Davis EH (1974) Elastic Solutions for Soil and Rock Mechanics, John Wiley and Sons, New York

Prandtl L (1921) Eindringunsfestigkeit und Festigkeit von Schneiden. Z. Angew. Math. Mech. 1(4):15–20

Schnaid F, Consoli NC, Cudmani RO, Militistsky J (1995) Load-settlement response of shallow foundations in structured unsaturated soils. Proceedings of the First International Conference of Unsaturated Soils, Paris, France 999–1004

Steensen-Bach JO, Foged N, Steenfelt JS (1987) Capillary Induced Stresses – Fact or Fiction? Nineth ECSMFE, Groundwater Effects in Geotechnical Engineering, Dublin 83–89

Terzaghi K (1943) Theoretical Soil Mechanics. John Wiley and Sons, New York

Vanapalli SK, Fredlund DG, Pufahal DE, Clifton AW (1996) Model for the Prediction of Shear Strength with respect to Soil Suction. Canadian Geotechnical Journal 33:379–392

Vanapalli SK, Fredlund DG (2000) Comparison of Empirical Procedures to Predict the Shear Strength of Unsaturated Soils Using the Soil-Water Characteristic Curve, Geo-Denver 2000. American Society of Civil Engineers, Geotechnical Special Publication 99:195–209

Vanapalli SK, Catana MC (2005) Estimation of the Soil–Water Characteristic Curve of Coarse-Grained Soils Using One Point Measurement and Simple Properties. Proceedings of an International Symposium on Advanced Experimental Unsaturated Soil Mechanics 401–410

Vesić AS (1973) Analysis of Ultimate Loads of Shallow Foundations. Journal of the Soil Mechanics and Foundation Division, ASCE 99(SM1):45–73