EXPERIMENTAL AND NUMERICAL MODELING STUDIES FOR INTERPRETING AND ESTIMATING THE p–δ BEHAVIOR OF SINGLE MODEL PILES IN UNSATURATED SANDS

Mohamadjavad Sheikhtaheri, B.Sc.

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Prof. Sai. K. Vanapalli, P.Eng.

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Department of Civil Engineering

University of Ottawa

Ottawa, Ontario

Canada K1N 6N5

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ABSTRACT

The design of pile foundations in conventional geotechnical engineering practice is based on the soil mechanics principles for saturated soils. These approaches are also extended to pile foundations that are placed totally or partially above the ground water table (i.e., vadose zone), where the soil is typically in a state of unsaturated condition. Such approaches lead to unrealistic estimations of the load carrying capacity and the settlement behavior of pile foundations. Some studies were undertaken in recent years to understand the influence of the matric suction towards the bearing capacity of model pile foundations placed in unsaturated fine-grained and coarse-grained soils. The conventional α, β and λ methods were modified to interpret the contribution of shaft carrying capacity of single piles in fine-grained soils (e.g., Vanapalli and Taylan 2011, Vanapalli and Taylan 2012). Also, the conventional β method has been used to understand the contribution of matric suction towards the shaft resistance in unsaturated sands (Vanapalli et al. 2010).

One of the key objectives of the present research study is directed to determine the contribution of matric suction towards the bearing capacity and settlement behavior of model single pile foundations in unsaturated sands. A series of single model pile load tests were performed in a laboratory environment to study the contribution of the matric suction towards the total, shaft, and base bearing capacity of the model piles with three different diameters (i.e., 38.30, 31.75, and 19.25 mm) in two unsaturated sands (i.e., a clean commercial sand and a super fine sand). Hanging column method (i.e., plexi glass water container) was used to control the matric suction values in the compacted sands in the test tank by varying the water table. The results of the testing programs indicate the significant contribution of the matric suction towards the bearing capacity of single model piles (i.e., 2 to 2.5 times of base bearing capacity and 5 times of shaft bearing capacity under unsaturated conditions in comparison with saturated condition). The test results were interpreted successfully by modifying the conventional methods for estimating the pile shaft bearing capacity (i.e., β method) and base bearing capacity (i.e., Terzaghi 1943, Hansen 1970 and Janbu 1976). In addition, semi-empirical methods were

proposed for predicting the bearing capacity of single model piles using the effective shear strength parameters (i.e., c' and ϕ') and the soil-water characteristic curve (SWCC). There is a good agreement between the measured and the predicted bearing capacity of single model piles using the semi-empirical models proposed in this study.

In addition, numerical investigations were undertaken using the commercial finite element analysis program SIGMA/W (Geostudio 2007) to simulate the load-displacement (i.e., p-δ) behavior of the single model piles for the two sands (i.e., clean commercial sand and super fine sand) under saturated and unsaturated conditions. An elastic-perfectly plastic Mohr-Coulomb model that takes into account the influence of the matric suction was used to simulate the load-displacement (i.e., p-δ) behavior. The numerical approach proposed in this thesis is simple and only requires the information of the effective shear strength parameters (i.e., c' and ϕ'), the elastic modulus (i.e., E_{sat}) under saturated conditions, the soil-water characteristic curve (SWCC), and the distribution of the matric suction with respect to depth.

The approaches proposed in this thesis can be extended to determine the in-situ load carrying capacity of single piles and also simulate the load-displacement (i.e., p-δ) behavior. The studies presented in this thesis are promising and encouraging to study their validity in-situ conditions. Such studies will be valuable to implement the mechanics of unsaturated soils into geotechnical engineering practice.

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LIST OF SYMBOLS

SUBSCRIPT

sat	= *saturated condition*
unsat	= *unsaturated condition*
max	= *maximum value*

SYMBOLS

α	= *frictional capacity coefficient*
α, β	= *fitting parameters for elastic modulus*
β	= *frictional capacity coefficient*
γ_d	= *dry unit weight of soil*
γ_{total}	= *total unit weight of soil*
δ	= *settlement of model pile*
δ'	= *effective internal friction angle of pile-soil interface*
Δq	= *increment of applied stress in elastic range*
$\Delta\delta$	= *increment of settlement in elastic range*
θ_r	= *residual volumetric water content*
θ_s	= *volumetric water content at saturation*
θ_w	= *volumetric water content*
Θ	= *normalized volumetric water content*
κ	= *fitting parameter for shear strength*
λ	= *frictional capacity coefficient*
μ	= Poisson's ratio
υ, μ	= *fitting parameters for prediction of undrained shear strength of unsaturated soils*
σ	= *applied stress in shear strength tests*
σ'	= *effective stress*

σ'_v	= *vertical effective stress*
$(\sigma - u_a)$	= *net normal stress*
τ	= *shear strength of soil*
$\tau_{suction}$, τ_{us}	= *contribution of shear strength of unsaturated soils due to matric suction*
τ_z	= *pile-soil shear stress at depth z*
ϕ'	= *effective internal friction angle for saturated condition*
ϕ^b	= *contribution of the matric suction to the angle of internal friction*
χ	= *effective stress parameter*
ψ_{BC}	= *bearing capacity fitting parameter*
a_w	= *normalized area of water*
A_b	= *pile base area*
A_{dw}	= *area of water corresponding to any degree of saturation*
A_{tw}	= *total area of water at 100% saturation*
A_s	= *surface area of the portion of pile embedded in soil*
B, D	= *pile diameter*
c'	= *effective cohesion for saturated condition*
c_u	= *undrained shear strength*
C_c	= *coefficient of curvature*
C_u	= *coefficient of uniformity*
d_c, d_q	= *depth factors*
D_{10}, D_{30}, D_{50}, and D_{60}	= *diameter of soil particle corresponding to passage percentage*
$D_{container}$	= *container diameter*
D_{pile}	= *pile diameter*
e	= *void ratio*
E	= *elastic modulus*
f_s	= *unit skin friction*
G_s	= *specific gravity*
H	= *pressure head*

I_p	= *plasticity index*
K_0	= *coefficient of lateral earth pressure at rest*
K_{py}	= $tan^2(45 + \phi'/2)$
L	= *length of model pile*
N_c, N_q, N_γ, N'_c, N'_q, and N'_γ	= *bearing capacity factors*
P	= *applied load on model pile*
P_a	= *atmospheric pressure (i.e., 101.3 kPa)*
Q_b	= *pile base resistance at the pile toe*
Q_s	= *pile shaft friction resistance*
Q_u	= *single pile ultimate bearing capacity*
$Q_{(ua-uw)}$	= *contribution of matric suction towards pile shaft capacity*
R_s	= *effective radius of maximum pore size of HAE material*
S	= *degree of saturation*
S_r	= *residual degree of saturation*
S_c, S_γ	= *shape factors*
T_s	= *surface tension of air-water interface*
u_a	= *pore-air pressure*
u_w	= *pore-water pressure*
$(u_a - u_w)$	= *matric suction*
$(u_a - u_w)_1$	= *matric suction right below the pile toe*
$(u_a - u_w)_2$	= *matric suction at the depth of stress bulb*
$(u_a - u_w)_{AVR}$	= *average matric suction value*
$(u_a - u_w)_b$	= *air-entry value*
w	= *water content*
w_{opt}	= *optimum water content*
w_r	= *residual water content*

CHAPTER 1

INTRODUCTION

1.1 Statement of the Problem

The shallow and deep foundations that are used for carrying the loads from super structures in many situations are placed above the ground water table (i.e., the vadose zone) where the soils are in a state of unsaturated condition. The influence of matric suction (i.e., capillary stresses) in this vadose zone is typically not taken into account in the conventional design of both the shallow and deep foundations. Pile foundations are typically designed based on principles of saturated soil mechanics assuming the soil is in fully saturated, submerged or dry condition. Such assumptions are used in practice as they are simple and conservative in many scenarios. However, such an approach is unrealistic with respect to estimation of the reliable load versus settlement behavior of pile foundations.

Several studies undertaken during last 50 years highlighted the contribution of matric suction towards the bearing capacity of unsaturated soils (Broms 1964, Steensen-Bach et al. 1987, Oloo et al. 1997). There are some studies reported in the literature taking into account the influence of matric suction in the design of footings (for example, Mohamed and Vanapalli 2006, Oh and Vanapalli 2009) and piles (Vanapalli and Taylan 2011, Vanapalli and Taylan 2012) based on model studies. Focus of these studies was directed towards interpreting the bearing capacity of shallow foundations in unsaturated coarse and fine-grained soils and the shaft capacity of single piles in unsaturated fine-grained soils. In the present study, a theoretical framework for interpreting the bearing capacity of pile foundations in coarse-grained unsaturated soils is provided. In addition, a semi-

empirical model is proposed for predicting the variation of the bearing capacity of pile foundations in unsaturated sands.

1.2 Research Objectives

The focus of the present research program was directed towards developing a framework for interpreting the total and base bearing capacity of a single pile in non-plastic unsaturated soils based on laboratory studies using model piles. In addition, simple semi-empirical models were developed to predict the variation of the carrying capacity of single piles with respect to matric suction. Such techniques would be useful for estimating the load carrying capacity of in-situ piles and encourage implementing the mechanics of unsaturated soils into geotechnical engineering practice.

The various objectives of this research are summarized as follows:

- *To design and construct bearing capacity test systems for the determination of the total, base, and shaft capacity of single model piles in a laboratory environment under both saturated and unsaturated conditions.*
- *To investigate the bearing capacity behavior of single piles in non-plastic unsaturated soils by performing comprehensive model pile load tests.*
- *To propose a semi-empirical equation for predicting the base bearing capacity of single pile foundations in unsaturated soils and estimating the contribution of matric suction.*
- *To develop numerical methods to simulate the load-displacement (i.e., p-δ) behavior of single model piles under both saturated and unsaturated conditions extending the elastic-plastic Mohr-Coulomb model using commercially available software SIGMA/W (i.e., Geoslope product).*

1.3 Scope of the Study

The scope of this thesis can be summarized as below.

1.3.1 Literature review

In recent years, considerable research has been undertaken to understand the engineering behavior of unsaturated soils. A comprehensive theoretical framework for interpreting the mechanical properties of unsaturated soils has been summarized by Fredlund and Rahardjo (1993). Several studies are reported in the literature taking into account the contribution of matric suction towards the bearing capacity in unsaturated soils (Oloo et al. 1997, Douthitt et al. 1998, Miller and Muraleetharan 1998, Georgiadis et al. 2002, Costa el al. 2003, Georgiadis et al. 2003, Mohamad and Vanapalli 2006, Vanapalli et al. 2007, Hamid and Miller 2009, Hossain and Yin 2010, Oh and Vanapalli 2011, Gurpersaud et al. 2013). The focus of the present research program was directed towards developing techniques and models for interpreting and predicting bearing capacity of single pile foundations in unsaturated coarse-grained soils. Comprehensive understanding of the shear strength behavior of unsaturated soils is necessary for proposing such a framework. Due to this reason, the focus of the literature review was directed towards providing relevant background on the shear strength behavior of unsaturated soils.

1.3.2 Theoretical background

Various empirical, semi-empirical, and computational procedures have been developed to estimate or predict the shear strength of unsaturated soils during last 50 years (Vanapalli 2009). The soil-water characteristic curve (SWCC) has been used as a tool along with saturated shear strength parameters (i.e., c' and ϕ') in the prediction of the shear strength of unsaturated soils by several investigators (Vanapalli et al, 1996, Oberg and Sallfors 1997, Khalili and Khabbaz 1998, Bao et al. 1998). Several researchers developed techniques for interpretation and estimation of bearing capacity of model footings and model piles extending the shear strength behavior of unsaturated soils. Semi-empirical models were proposed for estimation and prediction of the variation of bearing capacity of model footings in unsaturated soils (i.e., coarse and fine grained soils) with respect to matric suction (Vanapalli and Mohamed 2007, Oh and Vanapalli 2013). Recently, Vanapalli and Taylan (2012) proposed modifications to the conventional α, β, and λ methods that are typically used to interpret the shaft capacity of single piles in fine-grained saturated soils for unsaturated soils. They developed semi-empirical models for *estimating the variation of the shaft capacity of single piles taking into account the contribution of matric suction for both coarse and fine-grained soils extending the SWCC as a tool.*

The base resistance capacity contribution of single piles is dominant in comparison with shaft resistance capacity, in sandy type of soils (Miura 1983, Yasufuku and Hyde 1995, Ohno and Sawada 1999, Manandhar and Yasufuku 2012). Several methods are available in the literature to estimate the base bearing capacity of single pile foundations in saturated soils using the soil density, pile dimensions and the shear strength properties (Terzaghi 1943, Hansen 1970, Vesic 1973, Meyerhof 1976, Janbu 1976). Determination of independent contribution of pile base capacity of single piles from field tests is difficult. Due to this reason, a semi-empirical technique was proposed to predict the variation of the base bearing capacity of single piles in unsaturated soils with respect to matric suction using the SWCC as a tool. This technique was developed following similar procedures for prediction of the shear strength of unsaturated soils (Vanapalli et al. 1996).

1.3.3 A study for estimating the base bearing capacity of test piles in non-plastic unsaturated soils

A comprehensive experimental program was undertaken at the Geotechnical Laboratory of the University of Ottawa, Canada, to determine the pile base resistance, shaft resistance and total bearing capacity of single model piles in two different sandy soils (i.e., Soil #1: Unimin 7030 sand and Soil #2: Industrial sand) under both saturated and unsaturated conditions. The experimental pile load tests were performed using model piles of three different diameters (i.e., pile base diameter equal to 38.3, 31.75, and 19.25 mm) in specially designed aluminum test tank (i.e., soil container 700 mm in length and 300 mm in diameter). The hanging column method (i.e., plexi-glass water container) was used to achieve different values of matric suction within capillary zone above the water table in the soil container. The matric suction values were measured by placing commercial Tensiometers at different depths in the test tank. Conventional direct shear tests were conducted to determine the shear strength parameters, namely; the effective cohesion, c' and the angle of internal friction, ϕ', of the tested soils and the interface *strength parameters between the pile material and the tested soils. Other soil properties such as the compaction curve, density index, specific gravity and the SWCC were also determined in the laboratory. Based on the experimental results of the single model pile load tests, a semi-empirical equation and a numerical model were proposed for interpretation and estimation of the single pile base bearing capacity in unsaturated soils.*

1.3.4 Modeling the load versus displacement behaviors of a single pile in non-plastic unsaturated soils

Performing pile-load tests is costly, difficult and time consuming. Due to this reason, numerical modeling studies are undertaken in recent years for estimation of pile load-displacement (i.e., p-δ) behaviors (Mohamedzein et al. 1999, Georgiadis et al. 2003, Muraleetharan et al. 2009, Muraleetharan and Ravichandran 2009, Ravichandran 2009, Krishnapillai and Ravichandran 2012, Taylan et al. 2012, Taylan 2013). Numerical modeling was performed using the commercial finite element analysis program SIGMA/W (Geostudio 2007), which is a product of Geo-SLOPE (Krahn 2007). Elastic-perfectly plastic Mohr-Coulomb model was used to perform the finite element analysis. Comparison between results of numerical modeling and pile load tests from experimental results was provided (i.e., p-δ behavior of model piles).

1.4 Novelty of Research Study

Pile foundations are commonly used in conventional engineering practice to transfer the loads from heavy structures such as bridges, highways, embankments and multi-storied structures to the underlying soil safely and without stability or settlement problems. In many cases the entire or a portion of the pile length is located above the ground water table in unsaturated soil zone (i.e., vadose zone). The conventional design methods ignore the contribution of matric suction towards the bearing capacity of pile foundations, as discussed earlier.

Comprehensive experimental program is carried out to measure the base, shaft, and total bearing capacity of single model piles under both saturated and unsaturated conditions in *sands. Results of the experimental program were used to provide a framework for interpretation of the bearing capacity of single pile foundations in unsaturated sands. Three conventional methods (Terzaghi 1943, Hansen 1970, Janbu 1976) are modified to relate the variation of pile base bearing capacity to matric suction. A semi-empirical model is also proposed to predict the variation of bearing capacity of the single pile with respect to matric suction. The required parameters for using this model include the conventional saturated shear strength parameters (i.e., c' and ϕ') and the SWCC.*

One of the key features of this research study is the numerical modeling performed finite element analysis extending elastic-perfectly plastic Mohr-Coulomb model using commercially available SIGMA/W software program. The proposed FEA is more advantageous compared to in-situ or laboratory pile load tests, as it is more cost effective and less time consuming.

1.5 Thesis Layout

The presented thesis consists of eight chapters. The chapters are organized as follows:

The First Chapter, "Introduction", provides *the statement of the problem, objectives of the thesis research, scope, and novelty of the research study.*

The Second Chapter, "Literature Review", provides a brief review of the mechanics of *saturated and unsaturated soils. The focus is directed towards summarizing the shear strength behavior of unsaturated soils.*

The Third Chapter, "Theoretical Background", provides a brief summary of bearing *capacity of single piles in saturated and unsaturated soils. The limitations of the currently used single pile bearing capacity equations are discussed. A semi-empirical equation for predicting the variation of base bearing capacity of single pile with respect to matric suction is presented.*

The Fourth Chapter, "Equipment and Methodology", describes the details of th*e equipment and design detail of the testing setup used in the experimental study to* *measure the effect of matric suction on single piles bearing capacity. The methodology employed to collect all the necessary information related to the research program is detailed.*

The Fifth Chapter, "Presentation of Results", provides the results for the model pile load *tests, shear strength measurement tests, and matric suction and SWCC measurement tests.*

The Sixth Chapter, "Interpretation of the Model Pile Bearing capacity Test Results", *discusses the interpretation of the test results using the proposed semi-empirical method for the estimation of the base and total bearing capacity of single piles in saturated and unsaturated soils. Also, the measured shaft resistance of the single model piles is compared to predicted values extending modified β method.*

The Seventh Chapter, "Finite Element Modeling the Load versus Displacement *Behavior*s of Single Model Piles in Unsaturated Sands" presents brief theoretical *background of numerical modeling technique of elastic-perfectly plastic Mohr-Coulomb model extended for unsaturated soils. Finite element analysis (FEA) is performed to simulate the load displacement (i.e., p-δ) behavior of single model piles extending the elastic-plastic Mohr-Coulomb model in SIGMA/W program. Comprehensive comparison between the FEA results and measured values of pile load test are provided.*

The Eight Chapter, "*Summary and Conclusions*", presents the summary and conclusion*s of the research program and also provides some recommendations for future studies.*

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter provides a brief review of the basics related to the mechanics unsaturated soils. Also, the shear strength behavior of unsaturated soils in terms of the stress state variables; namely, net normal stress ($\sigma - u_a$) and matric suction ($u_a - u_w$) is discussed. The use of soil-water characteristic curve (SWCC) as a tool along with the shear strength parameters for predicting the shear strength behavior of unsaturated soils is highlighted.

One of the key objectives of the research study is to interpret the bearing capacity of single pile foundations in non-plastic unsaturated soils taking into account the influence of matric suction. In addition, a semi-empirical method is proposed to predict the variation of the load capacity of single piles with respect to matric suction. The background information related to the shear strength behavior of unsaturated soils is necessary to address these objectives. Due to this reason, focus of this chapter is directed towards summarizing the shear strength behavior of unsaturated soils.

2.2 Phase Properties in Saturated and Unsaturated Soils

In conventional saturated soil mechanics, soil is defined as a two-phase system consisting of liquid (i.e., water) and soil (i.e., solid particles). Typically, soils are in a state of unsaturated condition constituting of three phases including solid (soil particles), liquid (water) and gas (air). Air-water interface (i.e., contractile skin) independent properties with defined bounding surfaces, is considered as a fourth phase in unsaturated soils (Davies and Rideal 1963). Fredlund and Morgenstern (1977) extended the air-water interface (i.e., contractile skin) as the fourth phase in the rational interpretation of unsaturated soils behavior as it qualifies as an independent phase. A typical element of *unsaturated soil with a combination of different phases is shown in Figure 2.1. Two of the solid phases are in equilibrium under the applied stresses (i.e., soil particles and contractile skin) and two phases that flow under applied pressures (i.e., the air and water) (Fredlund and Morgenstern, 1976).*

The proportions of existing phases significantly influence the mechanical behavior of the soil. An external pressure or stress regime applied to a soil system, would be balanced by the components of pressure and stress emerging from phases (Murray and Sivakumar 2010). The state of stress in the soil arising from different phases is necessary in the rational interpretation of the mechanical behavior of soil.

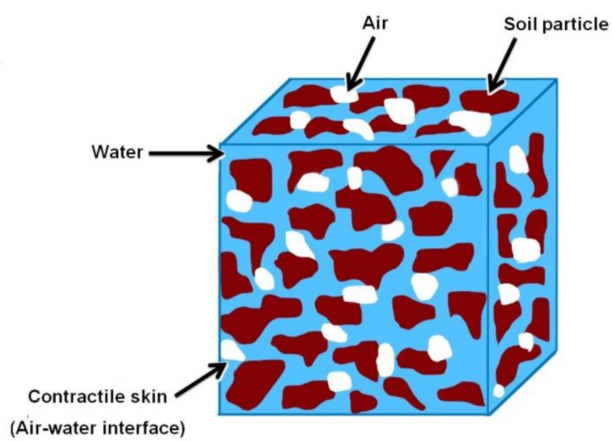


Figure 2.1 *An element of unsaturated soil with a continuous air phase (modified after Fredlund 1979)*

2.3 Stress State Variables and Effective Stress Concept

In general, the soil mechanics can be divided into two different categories; namely, saturated soil mechanics and unsaturated soil mechanics as summarized in Figure 2.2.

Effective stress concept was introduced by Karl Terzaghi (1936) for explaining the mechanical behavior of saturated soils. He expressed the effective stress, σ', in a mathematical form as the difference between the total stress, σ, and the pore-water pressure, u_w.

$$\sigma' = \sigma - u_w \tag{2.1}$$

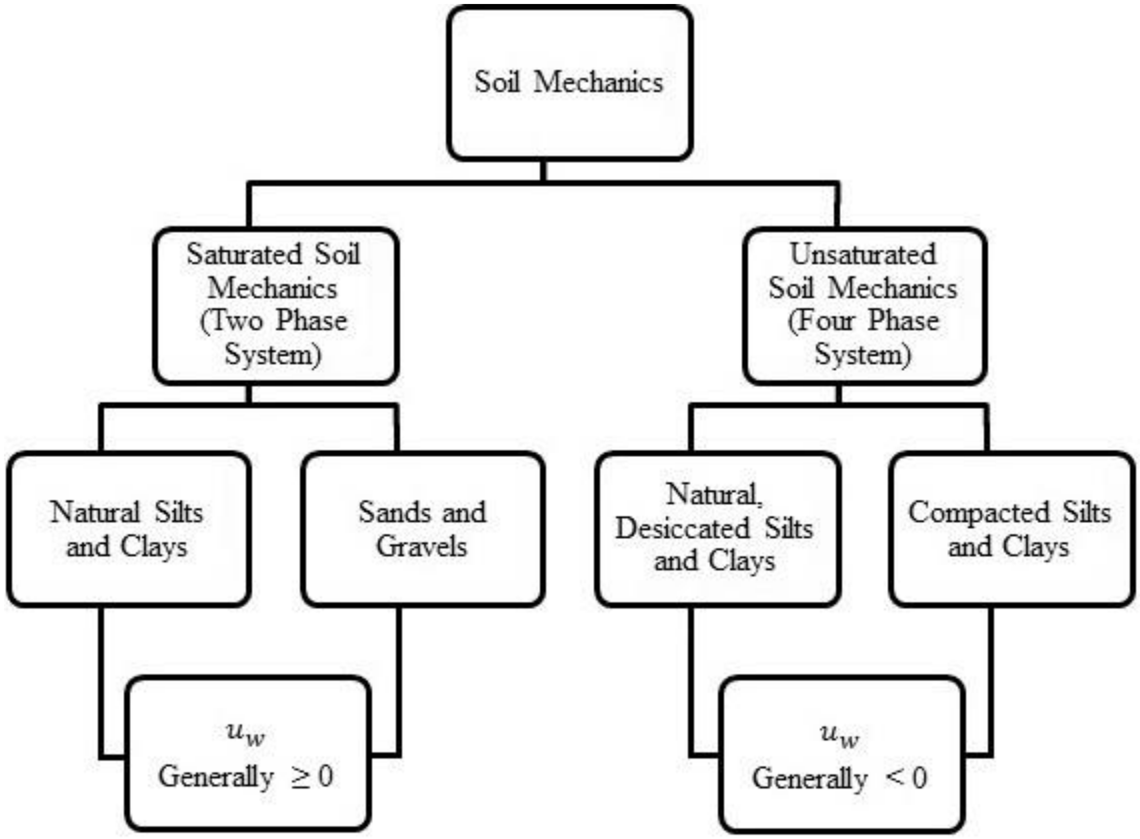


Figure 2.2 *Categories of soil mechanics (modified after Fredlund 1979)*

The mechanical behavior of saturated and dry soils can be described using only one independent single valued effective stress (i.e., ($\sigma - u_w$)) as the soil has two phases (i.e., soil and water in saturated soils or soil and air in dry soils). All the measurable changes of stress in the soil (i.e., compression, distortion and shearing resistance) are due to the change in the effective stress. Figure 2.3 illustrates the effective stress concept for the saturated soils proposed by Terzaghi (1936).

The mechanical behavior of unsaturated soils is more complex than saturated soils. In order to interpret the mechanical behavior of unsaturated soils by describing the state of stress, three widely recognized approaches have been developed. (i) using the modified effective stress approach proposed by Bishop (1959), (ii) the independent stress state approach, proposed by Fredlund and Morgenstern (1977), and (iii) modified stress *variable approach which is mainly adopted by a number of investigators for stress-strain analyses (Lu and Likos, 2006).*

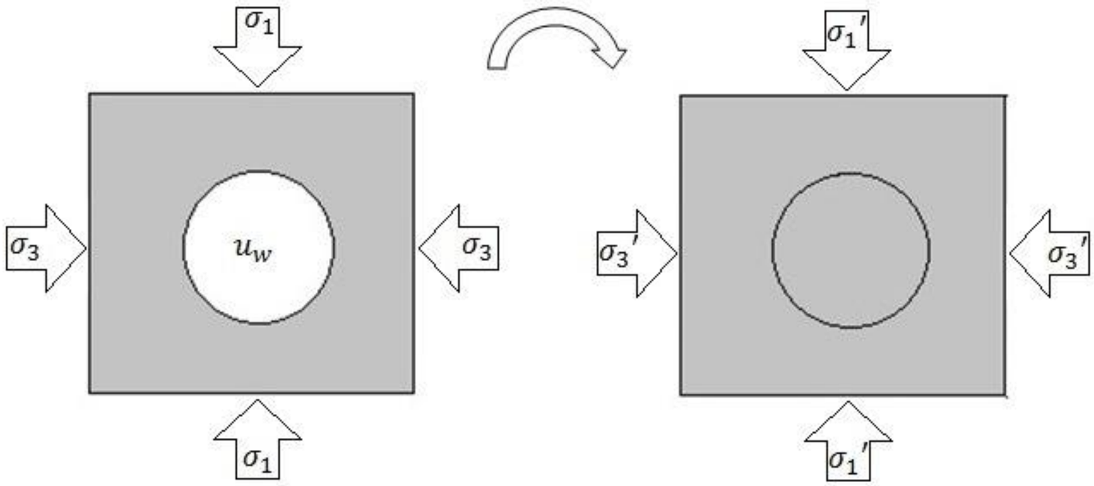


Figure 2.3 *Graphical representation of effective stress concept for saturated soils (Nuth and Laloui, 2007)*

Bishop (1959) proposed a single valued effective stress factor, χ, to modify the Terzaghi`s classic effective stress Eq. 2.1 for unsaturated soils as follows:

$$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) \qquad \textbf{(2.2)}$$

where, σ' = effective stress, σ = total stress, u_a = pore-air pressure, u_w = pore-water pressure, (σ - u_a) = net normal stress, ($u_a - u_w$) = matric suction, χ = effective stress parameter which is highly dependent on the degree of saturation and assumed to change between zero and unity as a function of degree of saturation (i.e., zero for a dry soil and unity for a saturated soil). The relationship between the χ and the degree of saturation, S, was derived from series of experiments on cohesionless silt (Donald, 1961) and compacted soils (Blight, 1961).

The use of single effective stress equation in the interpretation of mechanical (i.e., shear strength and volume change) behavior of unsaturated soils was recognized in the works of Bishop and Donald (1961), Jennings and Burland (1962), Bishop and Blight (1963) and Burland (1964). As finding a single effective stress equation faced difficulties, efforts *gradually were directed towards extending the two independent stress state variables framework to describe the mechanical behavior of unsaturated soils. Significant contribution to the use of two independent stress state variables in the literature was contributed through the research works of Coleman (1962), Matyas and Radhakrishna (1968), Fredlund and Morgenstren (1977). These works laid framework to rationally interpret the mechanical behavior of unsaturated soils.*

Fredlund and Morgenstern (1977) proposed two independent stress tensors for unsaturated soils by performing stress analysis of unsaturated soils based on phase continuum mechanics. They considered the unsaturated soil as a multiple phase soil system (i.e., four phases) and proposed three possible combination of stress variables to be used as stress state variables for unsaturated soils. The proposed combination of stress variables are: (i) $(\sigma - u_a)$ *and* $(u_a - u_w)$, *(ii)* $(\sigma - u_w)$ *and* $(u_a - u_w)$ *and (iii)* $(\sigma - u_a)$ *and* $(\sigma - u_w)$, *where,* σ *= total normal stress,* u_a *= pore-air pressure, and* u_w *= pore-water pressure. The proposed stress state variables were supported by conducting null experiments (i.e.,* $\Delta\sigma = \Delta u_a = \Delta u_w$*) (Fredlund, 1973). To separate the effect of total stress changes and pore water pressure changes, using the third combination of stress variables (i.e.,* $(\sigma - u_a)$ *and* $(u_a - u_w)$*) is more satisfactory. In most practical problems the pore-air pressure is atmospheric and the total stress is* $(\sigma - u_a)$*). Choosing this set of independent stress variables is advantageous both from conceptual and analytical aspects (Fredlund, 1979). The shear strength and volume change behavior of unsaturated soils can be formulated using the two independent tensors of stress state variables as below:*

$$\begin{bmatrix} (\sigma_x - u_a) & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & (\sigma_y - u_a) & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & (\sigma_z - u_a) \end{bmatrix} \textit{and} \begin{bmatrix} (u_a - u_w) & 0 & 0 \\ 0 & (u_a - u_w) & 0 \\ 0 & 0 & (u_a - u_w) \end{bmatrix} \quad \textbf{(2.3)}$$

In recent years, several studies were undertaken by various researchers to propose alternative approaches for stress-strain analyses in the form of modified stress variables (for example, Karube 1988, Alonso et al. 1990, Gallipoli et al. 2003).

2.4 Soil Water Characteristic Curve (*SWCC*)

During the last 20 years, several researchers have extended the soil-water characteristic curve (SWCC) as a tool for interpretation and prediction of engineering behavior of unsaturated soils (for example, Fredlund et al. 1994, Aubertin et al. 1995, Vanapalli et al. 1996, Leong and Rahardjo 1997, Oberg and Sallfors 1997, Bao et al. 1998, Barbour 1998, Khalili and Khabbaz 1998). Determining the unsaturated soil properties from experimental studies is costly and time consuming. Due to this reason, interest towards using the SWCC as a tool for interpretation and prediction of unsaturated soil properties is growing (Vanapalli et al. 2004).

The SWCC can be defined as a relationship between the gravimetric water content, w, volumetric water content, θ, or degree of saturation, S, and soil suction. The mechanical behavior of unsaturated soils can be derived from the SWCC studying the distribution of soil, water, and air phases changes due to drying or wetting (i.e., as the degree of saturation changes). Figure 2.4 illustrates a SWCC for a desaturation cycle from degree of saturation equals unity to zero. The curve includes a wide range of suction from 0 to 1,000,000 kPa (i.e., from saturated condition to total dry condition). The SWCC commonly can be divided into three main zones as boundary effect zone, transition zone, and residual zone of unsaturation.

Two key features of the SWCC are the air-entry value (AEV) or bubbling pressure, $(u_a - u_w)_b$, *and the residual degree of saturation,* S_r. *Each of the two key features can be used for interpretation of the mechanical behavior of unsaturated soils (Barbour, 1998). In the boundary effect zone the soil is in state of saturated and no reduction in area of water would happen, due to the reason that the water phase in this zone is continuous and all the soil pores are filled with water. The AEV indicates the suction value at which air begins to enter into the largest pores of the soil and beyond this point the soil starts to desaturate in the transition zone (Vanapalli et al., 1999). Furthermore, increasing the suction value, leads to reduction in the water content or degree of saturation. As the applied suction increases the water menisci area in contact with soil particles or aggregates starts to decrease and the water phase would be no longer be continuous as the air enters into the* *soil pores. As the suction reaches the residual degree of saturation, S, the influence of suction to further drain of the liquid phase in soil triggers to diminish in the residual zone of unsaturation. When the water content reaches a residual value, w_r, even large increases in suction values will no longer change the water content in the residual zone of unsaturation. The SWCC is a conceptual, interpretative, and predictive model that can be used for understanding, interpreting and predicting the shear strength behavior of unsaturated soils (Barbour, 1998).*

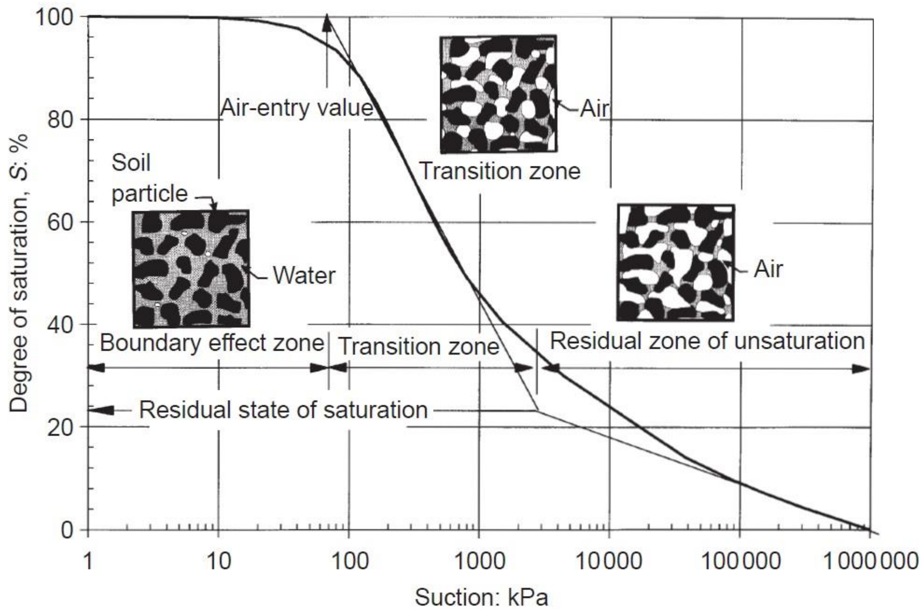


Figure 2.4 *Soil-water characteristic curve over the entire suction range of 0 to 1,000,000 kPa (modified after Vanapalli et al., 1999)*

2.5 Shear Strength of Unsaturated Soils

Shear strength of soil is an essential property required for proposing a proper solution for several geotechnical practical problems such as the bearing capacity of foundations (i.e., either shallow or deep foundations), stability of slopes in cuts or fills, and the earth pressure. Extending the Mohr-Coulomb failure criteria and the effective stress concept, shear strength of saturated soils can be presented as:

$$\tau = c' + (\sigma - u_w) \tan \phi' \qquad \textbf{(2.4)}$$

where, τ = shear stress, c' = effective cohesion, $(\sigma - u_w)$ = effective normal stress, σ = total normal stress, u_w = pore-water pressure, ϕ' = effective angle of internal friction for a saturated soil.

A typical Mohr-Coulomb failure envelope for saturated soils is shown in Figure 2.5.

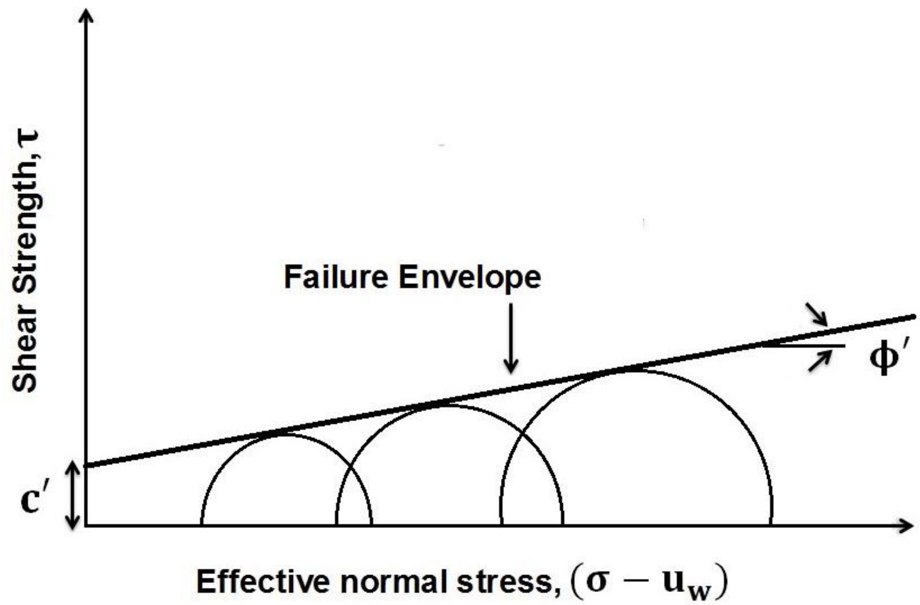


Figure 2.5 *Mohr-Coulomb failure envelope for a saturated soil*

2.5.1 Linear shear strength behavior of unsaturated soils

Fredlund and Morgenstern (1977) expressed that the stress state variables for unsaturated soils can be selected using any two sets of three stress state variable; namely, $(\sigma - u_a)$, $(\sigma - u_w)$, and $(u_a - u_w)$ as discussed earlier in this chapter. An equation for estimating the shear strength of unsaturated soils in terms of independent stress state variables was suggested by Fredlund et al. (1978).

$$\tau = c' + (\sigma_n - u_a)\tan\phi' + (u_a - u_w)\tan\phi^b \qquad \textbf{(2.5)}$$

where, τ = shear stress of an unsaturated soil, c' = effective cohesion, $(\sigma_n - u_a)$ = independent contributions of the net normal stress, $(u_a - u_w)$ = contribution of matric suction, ϕ' = effective internal friction angle under saturated condition due to net normal stress, ϕ^b = contribution of the matric suction to the angle of internal friction.

The contribution of the shear strength of unsaturated soils due to the matric suction, $\tau_{suction}$, can be presented as an independent component of the shear strength of unsaturated soils.

$$\tau_{suction} = (u_a - u_w)\tan\phi^b \qquad \textbf{(2.6)}$$

The corresponding matric suction, $(u_a - u_w)$, at soil saturation is equal to zero. Due to this reason, the failure envelope can be plotted on the shear strength, τ, with respect to the net normal stress, $(\sigma - u_a)$. The matric suction, $(u_a - u_w)$, is plotted as a third dimension to interpret the failure envelope. A planar surface that can be referred to as extended Mohr-Coulomb failure envelope can be defined using Eq. 2.5. The variation of shear strength, τ, with respect to the independent stress state variables can be plotted as shown in Figure 2.6.

Fredlund et al. (1978) analyzed three sets of shear strength test data presented by Bishop et al. (1960) using the proposed equation (Eq. 2.5) for interpreting the shear strength of unsaturated soils (Figure 2.7). The three triaxial tests were performed by keeping the water content constant (i.e., CW Tests) while the pore-air and pore-water pressures were measured. The analysis suggested that failure envelope with respect to the independent stress state variables is planar.

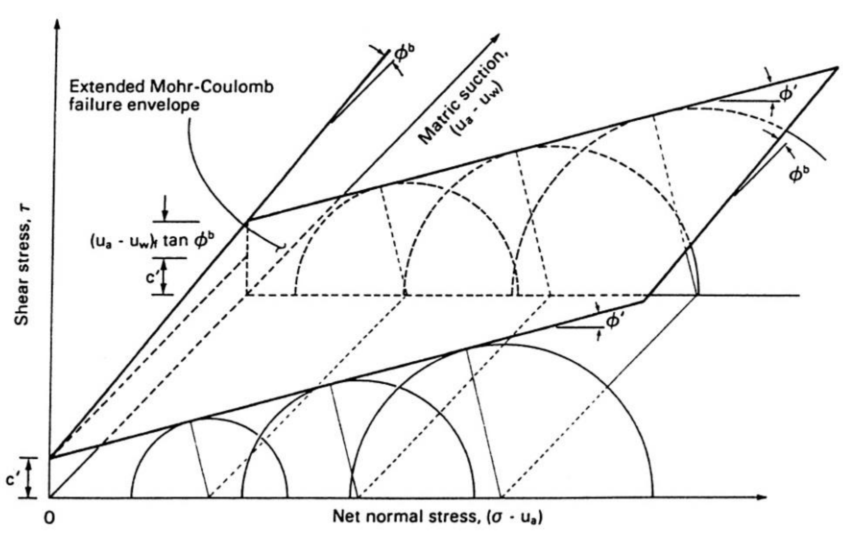


Figure 2.6 *Extended Mohr-Coulomb failure envelope (Gasmo et al., 1999)*

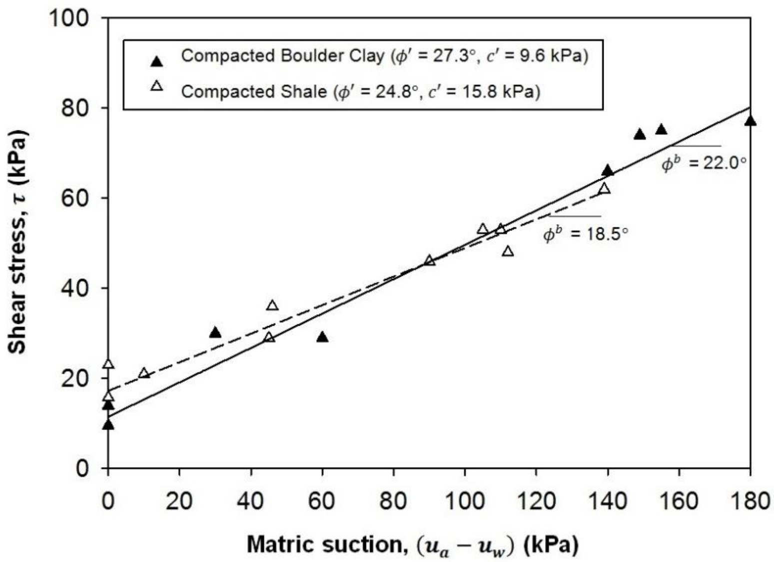


Figure 2.7 *Failure envelope on the shear stress, τ, with respect to matric suction, $(u_a - u_w)$, for two compacted soils (Fredlund et al., 1987 data from Bishop et al., 1960)*

Ho and Fredlund (1982) reanalyzed the results of consolidated drained and constant water content tests on unsaturated Dhanauri clay presented by Satija (1978) and also the results of consolidated drained (CD) direct shear and triaxial tests on unsaturated Madrid gray clay published by Escario (1980). The results of the analysis supported a planar type of failure envelope for shear strength of unsaturated soils. The experiment results for this analysis however used were performed the low matric suction range (i.e., 0 to 200 kPa).

2.5.2 Non-linear shear strength behavior of unsaturated soils

Fredlund et al. (1987) reanalyzed the experiment results published by Satija (1978) assuming a curved failure envelope. Results of this analysis indicated non-linearity in the shear strength behavior of unsaturated soils over a large suction range (i.e., 0 to 400 kPa) (Figure 2.8).

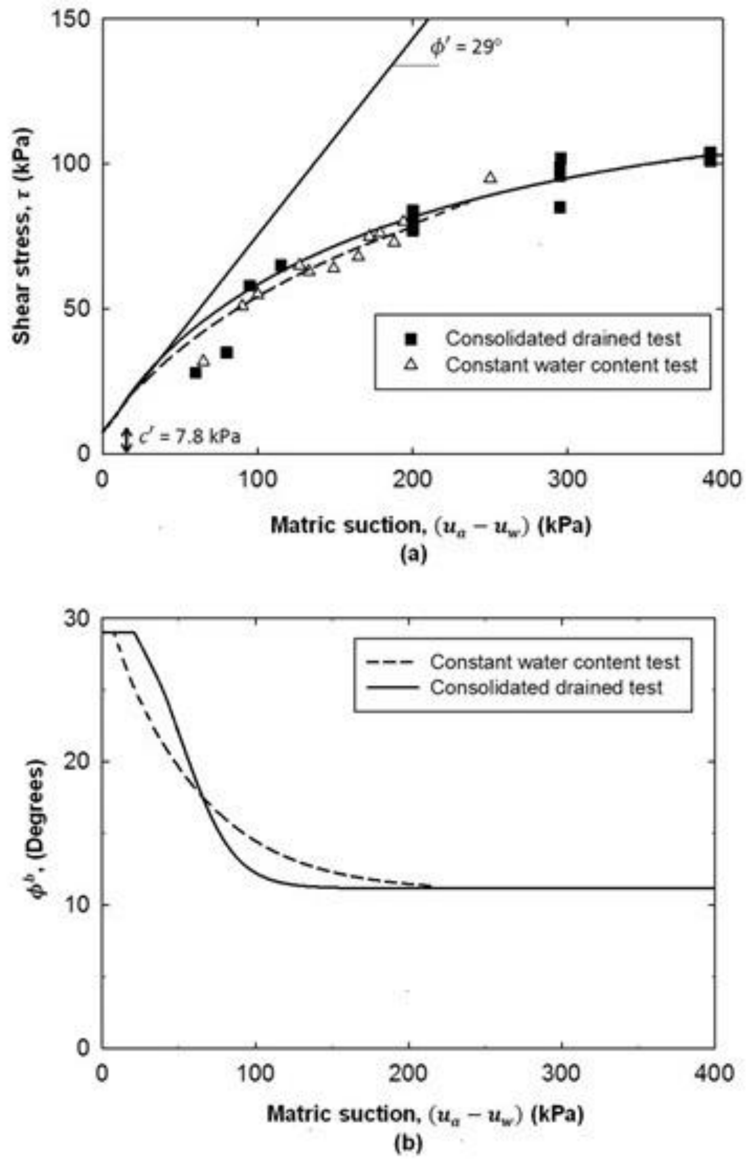


Figure 2.8 *Non linearity in the failure envelope with respect to the matric suction, (u_a - u_w):(a) curved failure envelopes for compacted Dhanauri clay at low density (data from Satija 1978), (b) corresponding ϕ^b values (Fredlund et al., 1987)*

Up to air-entry value, $(u_a - u_w)_b$, the contribution of angle of internal friction due to matric suction, ϕ^b, is equal to the angle of internal friction, ϕ′ (i.e., $\phi^b = \phi'$). As the soil starts to desaturate, water is drained from the soil pores, beyond the AEV, $(u_a - u_w)_b$. Any further increase in matric suction is not as influential as an increase in the net normal stress. As Figure 2.9 indicates, beyond the AEV, $(u_a - u_w)_b$, the contribution of the angle of internal *friction due to matric suction, ϕ^b, tends to decrease to a value lower than the angle of internal friction, ϕ'.*

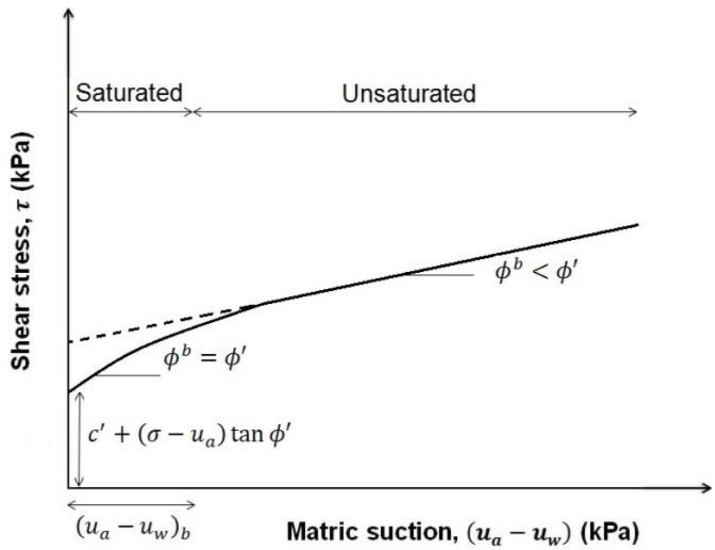


Figure 2.9 *Non-linearity in the failure envelope on the shear stress, τ, with respect to matric suction, $(u_a - u_w)$ (modified after Fredlund et al., 1987)*

2.5.3 Predicting of shear strength of unsaturated soils using the *SWCC* and saturated shear strength parameters

Experimental studies to investigate the shear strength behavior of unsaturated soils are costly and time consuming as discussed earlier. Due to this reason, various investigators have proposed estimation procedures for predicting the shear strength behavior of unsaturated soils. The SWCC has been used as a tool in many of these investigations to predict the shear strength of unsaturated soils (Fredlund et al. 1996, Vanapalli et al. 1996, Oberg and Sallfors 1997, Bao et al. 1998, Khalili and Khabbaz 1998, Xu and Sun 2002, Tekinsoy et al. 2004, Xu 2004).

The rate of change in the shear strength of unsaturated soils for various matric suction values (i.e., different unsaturated conditions) is greatly dependent on contact area of water with soil particles within pores (Vanapalli et al., 1996). A relationship between the shear strength of unsaturated soils and the SWCC can be defined for different stages of *desaturation. The SWCC can be divided into three main zones (i.e., boundary effect zone, transition effect zone, and residual zone of unsaturation) for interpreting the shear strength. In each of these zones the shear strength behavior is different (Figure 2.10 a and b). Up to the AEV, there is a linear increase in shear strength. In other words, the contribution of the angle of internal friction due to the matric suction is equal to contribution of the effective angle of internal friction due to net normal stress (i.e., ϕ^b = ϕ'). The rate of change in shear strength with respect to matric suction is higher in boundary effect zone than the other two zones (i.e., transition zone and residual zone of unsaturation). In these zones, the net contribution of suction starts decreasing as the wetted contact of area decreases. Further increase in matric suction is not as effective as an increase in net normal stress and the ϕ^b tend to drop to a lower value than the ϕ' (i.e., $\phi^b < \phi'$).*

The shear strength of unsaturated soils may rise, drop, or remain constant, in residual zone of unsaturation based on soil type and the amount of drainage that can occur from the soil pores. In sands at residual zone of unsaturation, the water content can be quite low and soil particles may not be affected by the suction. Due to this reason, even high values of matric suction will not produce significant increase in the shear strength.

In order to predict the shear strength of unsaturated soils, Lamborn (1986) proposed an equation extending a micromechanics model based on irreversible thermodynamics to the energy versus volume relationship for multiphase system material as below:

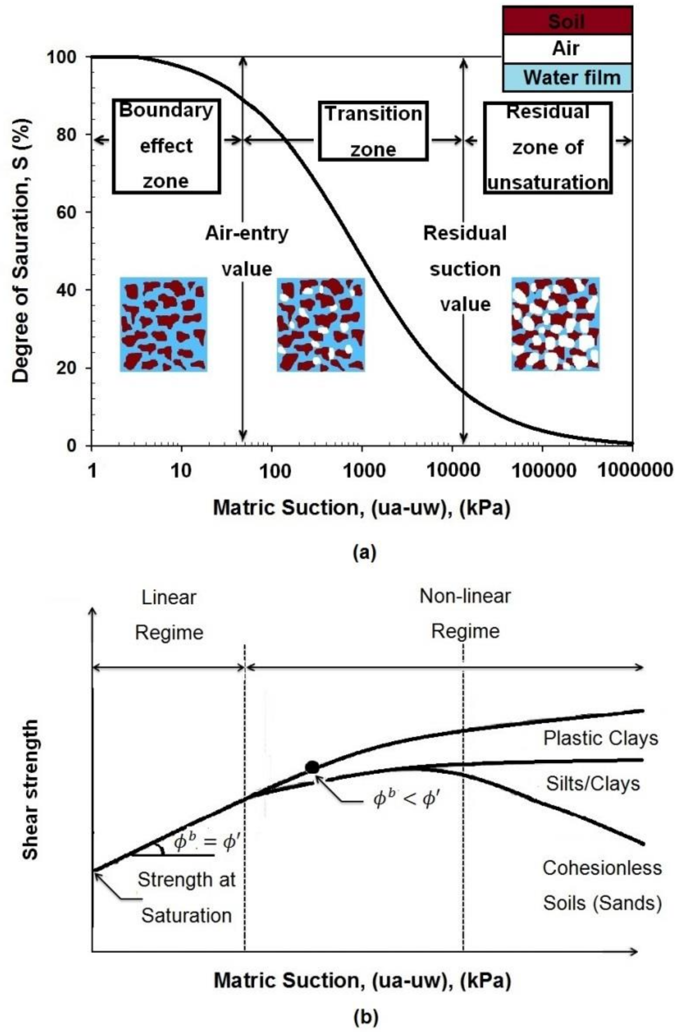


Figure 2.10 *The relationship between the SWCC and shear strength of unsaturated soils (modified after Vanapalli, 2009)*

$$\tau = [c' + (\sigma_n - u_a)\tan\phi'] + (u_a - u_w)\theta_w(\tan\phi') \qquad \textbf{(2.7)}$$

where, θ_w = volumetric water content which is defined as the ratio of volume of water to the total volume of the soil. The volumetric water content is a nonlinear function of soil matric suction (i.e., volumetric water content decreases as the soil suction increases). As the volumetric water content, θ_w, equals to unity the ϕ^b would be equal to ϕ' (Vanapalli and Fredlund, 1999).

Vanapalli et al. (1996) and Fredlund et al. (1996) proposed an approach to predict the non-linear variation shear strength with respect to suction for unsaturated soils using the SWCC and the effective shear strength parameters. Normalized area of water, a_w, is a controlling parameter to determine the rate of contribution of matric suction towards the shear strength. The normalized area of water, a_w, is defined as below:

$$a_w = \frac{A_{dw}}{A_{tw}} \quad \textbf{(2.8)}$$

where, A_{tw} = the total area of water at 100% saturation and A_{dw} = the area of water corresponding to any degree of saturation. The normalized area of water, a_w, is representing directly the water volume in the soil, which differs from unity at saturation to zero as the soil is completely dry. The normalized volumetric water content, Θ, of the soil with respect to matric suction, and the normalized area of water is represented using the following relationship:

$$a_w = (\Theta^{\kappa}) \quad \textbf{(2.9)}$$

where, κ = fitting parameter. A relationship between the fitting parameter, κ, and plasticity index, I_p, was suggested by Vanapalli and Fredlund (2000). This relationship was modified by Garven and Vanapalli (2006) based on extensive investigation of the available shear strength data in the literature as given below:

$$\kappa = -0.0016 \cdot I_p^2 + 0.0975 \cdot I_p + 1 \quad \textbf{(2.10)}$$

Vanapalli et al. (1996) expressed the contribution of shear strength due to the matric suction, τ_{us}, as a function of normalized area of water, a_w, as below:

$$\tau_{us} = (u_a - u_w)(a_w \tan \phi') \quad \textbf{(2.11)}$$

By substituting Eq. (2.9) in Eq. (2.11) the contribution of shear strength due to the matric suction can be expressed, τ_{us}, as below:

$$\tau_{us} = (u_a - u_w)[(\Theta^{\kappa})(\tan \phi')] \quad \textbf{(2.12)}$$

The normalized water content, Θ is equal to degree of saturation, S, the Eq. (2.12) can be written as:

$$\tau_{us} = (u_a - u_w)[(S^{\kappa})(tan\,\phi')] \quad \textbf{(2.13)}$$

Shear strength of unsaturated soils at any given matric suction value can be predicted by considering the contribution of shear strength due to the matric suction, τ_{us}, (i.e., derived from the SWCC) and the contribution of shear strength due to the net normal stress (i.e., the saturated shear strength when the pore-air pressure, u_a, is equal to the pore-water pressure, u_w) as proposed by Vanapalli et al. (1996).

$$\tau = [c' + (\sigma_n - u_a)\tan\phi'] + [(u_a - u_w)S^{\kappa}\tan\phi'] \quad \textbf{(2.14)}$$

Vanapalli et al. (1996) proposed another method to predict the shear strength of unsaturated soils using SWCC with no need of using fitting parameter, κ. The equation uses the residual volumetric water content, which can be estimated from the SWCC.

$$\tau = [c' + (\sigma_n - u_a)\tan\phi'] + (u_a - u_w)[\tan\phi'\left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)] \quad \textbf{(2.15)}$$

where, θ = volumetric water content at any matric suction, θ_r = residual volumetric water content, and θ_s = volumetric water content at saturation. In order to use Eq. (2.15) the residual volumetric water content, θ_r, has to be estimated using the SWCC.

The nonlinear shear strength behavior of unsaturated soils with respect to matric suction is rigorously explained in Vanapalli (2009) by differentiating Eq. (2.13) with respect to matric suction. In case of sandy soils the plastic index is equal to zero (i.e., non-plastic soils) as a result the fitting parameter, κ, equals to unity is required for the shear strength prediction. The reduction in the shear strength of sandy soils for matric suction values close to residual zone of unsaturation can be justified as the degree of saturation, S, is relatively small and the value of $d(S^{\kappa})/d(u_a - u_w)$ is negative (Vanapalli, 2009).

$$\tan\phi^b = \frac{d\tau_{unat}}{d(u_a - u_w)}$$

$$= [(S^{\kappa}) + (u_a - u_w)\frac{d(S^{\kappa})}{d(u_a - u_w)}]\tan\phi' \quad \textbf{(2.16)}$$

2.6 Summary

During the past 50 years considerable research has been performed to understand the engineering behavior of unsaturated soils. Efforts have been directed towards developing experimental methods and theoretical frameworks to investigate the shear strength behavior of unsaturated soils. The focus of this chapter is directed towards providing literature review of the shear strength behavior and how it can be predicted using the SWCC as a tool.

The main purpose of this research is to develop a framework for interpretation and estimation of carrying capacity of single pile foundations. Shear strength behavior of unsaturated soils is a key property for understanding the single pile foundations bearing capacity. The theoretical background provided in this chapter is used as a tool for proposing the semi-empirical methods for predicting the single pile base capacity in unsaturated sandy soils. Details of the procedures for developing the semi-empirical methods are summarized in Chapter 3.

CHAPTER 3

THEORETICAL BACKGROUND

3.1 Introduction

Design of pile foundations in conventional engineering practice is based on a combination of empirical methods and past experience. Theoretical methods for design of pile foundations were not well received in the literature. The pioneering geotechnical engineers Terzaghi and Peck (1967) stated that the "…theoretical refinements in dealing with pile problems are completely out of place and can be safely ignored…". In spite of this doubtful evaluation, in the last few decades efforts have been directed towards relying on the pile design procedures from empiricism to theoretical based methods. Estimation of axial capacity is the first major step in the design of pile foundations is relied on empirical relationships (Poulos, 1989).

Douglas (1989) studies support the need for conducting the pile load tests to overcome the limitations in the estimation procedures among the empirical, theoretical and numerical computer modeling studies. More recently, Randolph (2003) stated that the axial pile capacity in many soil types may be difficult to be estimated more accurately than about $\pm$30%. As a result, pile load tests should be performed early during the construction phase to improve the final design.

In-situ pile testing is however costly and time consuming; due to this reason, researchers focused to develop reliable empirical, analytical or numerical techniques to estimate the bearing capacity pile foundations. Studying the load versus displacement behavior of piles in different soil types has been of interest for researchers in the past half a century (Vesic 1963, Tavenas 1971, Hanna and Tan 1973, Tan and Hanna 1974, Beringen et al. 1979, Olson and Dennis 1982, Yazdanbod et al. 1984, Briaud and Tucker 1988, Kraft 1991, Randolph et al. 1994, and Igoe et al. 2011).

The design of single pile foundations in many cases is based on conventional soil mechanics assuming the soil is in a state of saturated condition, in spite of the entire length or part of the pile located in an unsaturated soil zone (i.e., above the ground water table). This design approach is also extended in regions where the soil never reaches saturated conditions over their entire design life (i.e., arid and semi-arid regions). In other words, the influence of capillary stresses towards the contribution of base and shaft resistance is conventionally not taken into consideration in the design of single pile foundations. Several studies are reported in the literature that take into account the influence of matric suction on the bearing capacity in unsaturated soils during the test studies (Oloo et al. 1997, Douthitt et al. 1998, Miller and Muraleetharan 1998, Georgiadis et al. 2002, Costa el al. 2003, Georgiadis et al. 2003, Mohamad and Vanapalli 2006, Vanapalli et al. 2007, Hamid and Miller 2009, Hossain and Yin 2010, Oh and Vanapalli 2011, Gurpersaud et al. 2013). The key objectives of the research study as discussed in the earlier chapters is to propose simple semi-empirical techniques and numerical models for interpreting and predicting the base bearing capacity of single pile foundations in unsaturated sands taking into account the contribution of matric suction.

This chapter provides a theoretical background for the conventional single pile foundations bearing capacity estimation procedures in saturated soils. The conventional methods that are used in practice are modified to take into account the effect of matric suction on single pile foundations carrying capacity in unsaturated soils..

3.2 Pile Foundations

Pile foundations are commonly used in engineering practice to carry the loads from heavy structures such as multi-storied buildings, bridges, highways, embankments, to the underlying soil safely without stability or settlement problems. Piles are used in situations when the bearing capacity of soil is low, non-availability of proper bearing stratum at shallow depth, where shallow foundations are not practical or economical. Extensive growth of offshore energy resources and development of high-rise structures, highlight the need for using pile foundations with higher capacities and deeper penetrations

(Chandrasekaran et al. 1978, Bowles 1996, Katzenbach et al. 2000, Overy 2007, Madabhushi et al. 2010, Doherty and Gavin 2011)

Pile foundations can be classified by different criteria such as pile material (i.e., steel, reinforced concrete piles, or wood), method of installation (i.e., driven, jacked or bored piles), and load carrying mechanism of the pile. Based on the load carrying mechanism, piles can be categorized as below:

• End bearing piles: pile end resistance plays significant role in this group to transfer the *load of superstructure among the water or weaker soils to strong stratum.*

• Friction piles: vertical distribution of the superstructure *load to the lower stratum by means of pile shaft friction which is sometimes called as floating piles.*

• Compaction piles: rather than load carrying approach, piles can be used to compact the *soil. Through using these piles the loose, granular soil would be become denser. Normally a steel tube is derived into the ground which replaces the tubular volume by forming a sand pile from granular materials.*

• Tension piles: in case of superstructures which are subjected to lateral loads such as *wind, wave, and earthquake, these pile can be utilized to neutralize the pull-out forces.*

Single piles ultimate bearing capacity arises from the combined contribution of the shaft and base resistance. The ultimate pile base resistance can be calculated using the conventional methods developed for estimating the surface footing bearing capacity. These methods are extensions of the rigid plasticity theory approaches for surface footings (Chandrasekaran et al. 1978, Gui and Muhunthan, 2006). The most common methods for predicting the pile base capacity in geotechnical engineering practice are Terzaghi (1943), Hansen (1970), Janbu (1976), Meyerhof (1976), Vesic (1977), and Coyle and Castello (1981).

Figure 3.1 illustrates different pile types based on pile load carrying mechanism.

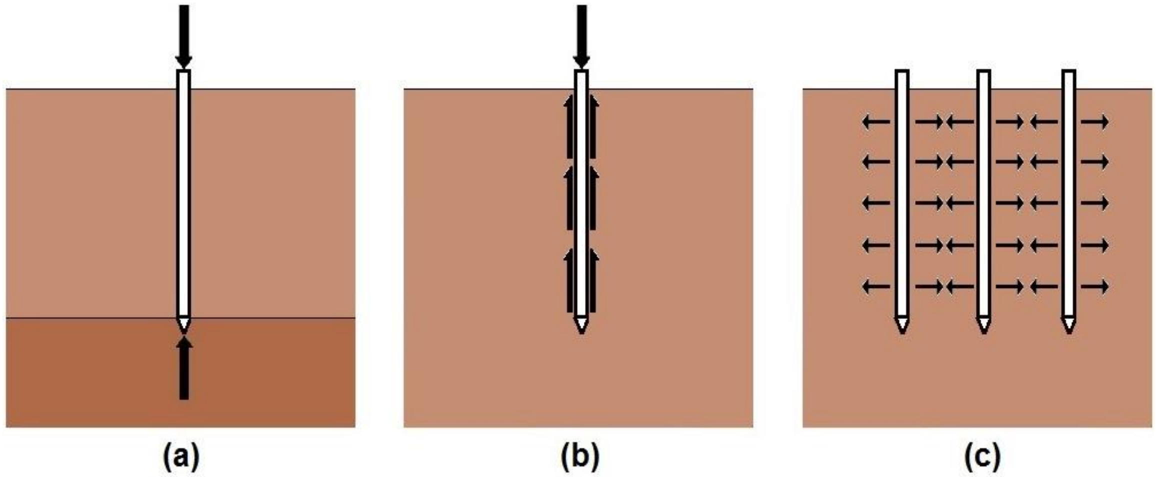


Figure 3.1 *Typical pile configuration based on pile load carrying capacity (a) end bearing pile, (b) friction pile, (c) compaction pile (modified after Madabhushi et al., 2010)*

3.2.1 Piles in sand

Estimating the axial capacity of piles driven into sand is one of the most intriguing challenges in the foundation design due to high levels of uncertainty (Randolph et al., 1994). It is well known in the literature that the pile base bearing capacity contribution of single piles is dominant in sandy type of soils in comparison with the shaft carrying capacity (Miura 1983, Yasufuku and Hyde 1995, Ohno and Sawada 1999, Manandhar and Yasufuku 2012). Determination of the independent contribution of base resistance of single pile from field tests is difficult. Due to this reason, it will be valuable to provide an interpretation technique and also semi-empirical procedure for predicting the base resistance of single pile in unsaturated sands.

3.2.2 Piles in unsaturated soils

In many situations, a portion or in some cases the entire length of the pile may be located above the ground water table where the soil is in a state of unsaturated condition. Such scenarios are commonly encountered in arid and semi-arid regions of the world. The pile capacity however is estimated using the conventional methods extending principles of saturated soil mechanics. In other words, the contribution of capillary stresses (i.e., matric suction) towards the pile load capacity is typically ignored.

In past few years, some studies have been conducted to understand the influence of matric suction on bearing capacity of both shallow and deep foundations in unsaturated soils (Oloo et al. 1997, Douthitt et al. 1998, Miller and Muraleetharan 1998, Georgiadis et al. 2002, Costa el al. 2003, Georgiadis et al. 2003, Hamid 2005, Mohamad and Vanapalli 2006, Rojas et al. 2007, Vanapalli et al. 2007, Hamid and Miller 2009, Oh and Vanapalli 2009, Hossain and Yin 2010, Vanapalli and Taylan 2011, Oh and Vanapalli 2012, Vanapalli and Taylan 2012, Gurpersaud et al. 2013, Oh and Vanapalli 2013). Some of these studies focused on the investigation of single model pile shaft resistance. However to the best knowledge of the author, no studies are reported in the literature to propose estimation methods for single model pile base resistance with respect to contribution of matric suction in unsaturated sands.

In the present study, the conventional methods for estimation of the single pile base capacity (Terzaghi 1943, Hansen 1970, and Janbu 1976) are modified in order to take into account the contribution of matric suction in unsaturated conditions. The form of equation is presented such that, there is a smooth transition between the modified and conventional methods when the matric suction value is set to zero.

3.3 Pile Bearing Capacity

The single piles ultimate carrying capacity can be estimated from the combined contribution of the shaft and base resistance. The axial load capacity of a single pile is typically expressed using the relationship below:

$$Q_u = Q_s + Q_b \tag{3.1}$$

where, Q_u = single pile ultimate bearing capacity, Q_s = shaft friction resistance of the pile, Q_b = pile base resistance at the pile toe.

3.3.1 Pile shaft capacity

The pile shaft resistance is fully mobilized along the length of the pile-soil interface. The pile shaft capacity is commonly can be estimated as:

$$Q_s = f_s \times A_s \quad \textbf{(3.2)}$$

where, f_s = unit skin friction, A_s = surface area of the portion of the pile embedded in soil.

The unit skin friction, f_s, is determined based on the laws of mechanics considering friction between solid surfaces. The pile shaft capacity is commonly evaluated integrating pile-soil shear stress, τ_z, at depth z over the surface area of shaft along the embedded pile length (Vesic, 1973). The shaft resistance of single pile foundations in sands is calculated by empirical methods which extend the back calculated parameters from load test databases (for example, Gavin and Lehane 2003, Randolph 2003). The common methods for estimating the pile shaft capacity are namely, α method (Skempton 1959), β method (Burland 1973), and λ method (Vijayvergiya and Focht 1972).

The α method is based on total stress analysis (TSA) to estimate the shaft carrying capacity of single pile foundations in saturated fine-grained soils under undrained loading conditions. The single pile shaft capacity is related to the undrained soil shear strength, c_u, using a dimensionless parameter, which is referred to as adhesion factor, α. The general form of this method is stated as below:

$$Q_s = f_s \times A_s = \sum_{i=1}^{i=n} \alpha c_u \pi d L \quad \textbf{(3.3)}$$

where, d = pile diameter, L = pile length.

Oh and Vanapalli (2009) suggested a relationship for determining the undrained shear strength of unsaturated soils, $c_{u(unsat)}$. This relationship is useful to estimate the $c_{u(unsat)}$, taking account of the influence of matric suction. The variation of $c_{u(unsat)}$ with suction can be determined using the SWCC and the undrained shear strength of saturated soils, $c_{u(sat)}$. The proposed method for estimating the unsaturated undrained shear strength was used by Vanapalli and Taylan (2011) for modifying the α method as below:

$$Q_{f(us)} = \alpha c_{u(unsat)} \pi d L$$

$$= \alpha c_{u(sat)} \left[1 + \frac{(u_a - u_w)}{({}^{Pa}/_{101.3})} (S^{\nu}/\mu)\right] \pi d L \quad \textbf{(3.4)}$$

where, $c_{u(sat)}$, $c_{u(unsat)}$ = shear strength under saturated and unsaturated conditions respectively, v, μ = fitting parameters function of plasticity index, I_p, P_a = atmospheric pressure (i.e., 101.3 kPa).

For the single piles which are loaded at a slow rate, the loading condition can be assumed as drained condition. Burland (1973) proposed the β method as an effective stress analysis (ESA) method for the drained condition based on following assumptions:

(i) The effective cohesion decreases to zero as a result of the remolding soil beside the pile during the installation process.

(ii) As the excess pore pressures dissipated as a result of volume displacement, the effective stress on pile surface is considered to be at least equal to horizontal effective stress.

(iii) The major shear deformation during pile loading is assumed to be restricted to a thin zone around the pile shaft. In this thin zone, drainage occurs at a relatively faster rate during loading.

The following general equation is used to estimate the shaft frictional resistance:

$$f_s = c' + K_{\circ}\sigma_v' tan\phi' \quad \textbf{(3.5)}$$

where, c' = effective cohesion, K_0 = mean lateral earth coefficient at rest, σ'_v = vertical effective stress along the pile length, ϕ' = effective internal angle of friction.

The shaft capacity of single piles in unsaturated soils, $Q_{f(us)}$, can be estimated using a general expression:

$$Q_{f(us)} = Q_f + Q_{(u_a - u_w)} \quad \textbf{(3.6)}$$

The contribution of shaft capacity of single piles due to the matric suction, $Q_{(ua-uw)}$, can be estimated using the model proposed by Vanapalli et al. (1996) for prediction of shear strength of unsaturated soils under the drained loading conditions:

$$\tau = [c' + (\sigma_n - u_a)\tan\phi'] + [(u_a - u_w)S^{\kappa}\tan\phi'] \quad \textbf{(3.7)}$$

where, (σ_n - u_a) = net normal stress, ϕ' = effective internal fiction, (u_a - u_w) = matric suction, S = degree of saturation, κ = fitting parameter used to provide a proper fit between the measured shear strength and estimated values.

The contribution of matric suction towards the shear strength, τ_{us}, is represented by the second part of Eq. (3.7):

$$\tau_{us} = (u_a - u_w)[(S^{\kappa})(tan\,\phi')] \qquad \textbf{(3.8)}$$

The contribution of matric suction towards the pile shaft capacity, $Q_{(ua-uw)}$, can be estimated by extending the approach proposed by Hamid and Miller (2009), as the shaft resistance is related to the shear strength.

$$Q_{(u_a-u_w)} = \tau_{us}A_s$$

$$= [(u_a - u_w)(S^{\kappa})(tan\phi')]\pi dL \qquad \textbf{(3.9)}$$

Vanapalli and Taylan (2011) proposed the ultimate shaft capacity of single piles under unsaturated conditions. By substituting the Eq. (3.9) in Eq. (3.6) the modified β method for estimation of single pile shaft capacity in unsaturated soils can be derived:

$$Q_{f(us)} = [\beta\sigma_v' + (u_a - u_w)(S^{\kappa})(tan\phi')]\pi dL \qquad \textbf{(3.10)}$$

where, κ = fitting parameter which can be estimated using a relationship suggested by Vanapalli and Fredlund (2000) between the fitting parameter, κ and plasticity index, I_p. This relationship was modified by Garven and Vanapalli (2006) as below:

$$\kappa = -0.0016I_p^2 + 0.0975I_p + 1 \qquad \textbf{(3.11)}$$

Combination of the total (i.e., undrained) and effective (i.e., drained) stress approach (i.e., λ method), was developed by Vijayvergiya and Focht (1972), to estimate the shaft capacity of piles which are driven into fine-grained soils. The total shaft capacity of single piles is calculated using the following relationship:

$$Q_s = [\lambda(\sigma_v' + 2c_u)]\pi dL \qquad \textbf{(3.12)}$$

where, λ = frictional capacity coefficient which is dependent on the total pile embedment length. Based on 42 pile load test data analysed by Vijayvergiya and Focht (1972), the normal range for λ for pile penetration of 0 to 70 m was estimated in a range of 0.12 to 0.5.

Vanapalli and Taylan (2011) modified form the λ method for unsaturated soilss to take into account of the contribution of matric suction towards the shaft capacity of single piles as given below:

$$Q_{s(us)} = \lambda[\sigma_v' + 2\alpha c_{u(sat)}\left[1 + \frac{(u_a - u_w)}{(^{Pa}/_{101.3})}(S^v/\mu)\right]]\pi dL \qquad \textbf{(3.13)}$$

3.3.2 Pile base bearing capacity

The theoretical approach to analyze and estimate the static pile bearing capacity was investigated by several investigators (Caquot 1934, Buisman 1935 and Terzaghi 1943) extending the approaches of Prandtl (1920) and Reissner (1924). Their work was mainly based on failure mechanism for single pile foundations which has established a benchmark for future works (Chandrasekaran et al., 1978). Following the same approach, several different solutions were proposed by various researchers (De Beer 1945, Meyerhof 1951, Hansen 1970, Janbu 1976, Vesic 1977, Coyle and Castello 1981). In this section some of the conventional methods proposed for estimation of single pile base capacity are briefly reviewed.

3.3.2.1 Terzaghi (1943)

Terzaghi (1943) proposed a method for determining the bearing capacity of shallow foundation which can be extended for estimation of the pile base resistance. Figure 3.2 illustrates the proposed bearing capacity failure pattern around the pile tip. The soil above the pile base is assumed as an equivalent surcharge, q. The shear strength of the overburden soil is ignored and its weight is only considered. This failure mechanism indicates the downward movement of the volume I and consequently displacement of soil outward and upward (i.e., volume II, III, II', and III') with the failure surfaces ending at *the pile tip level.*

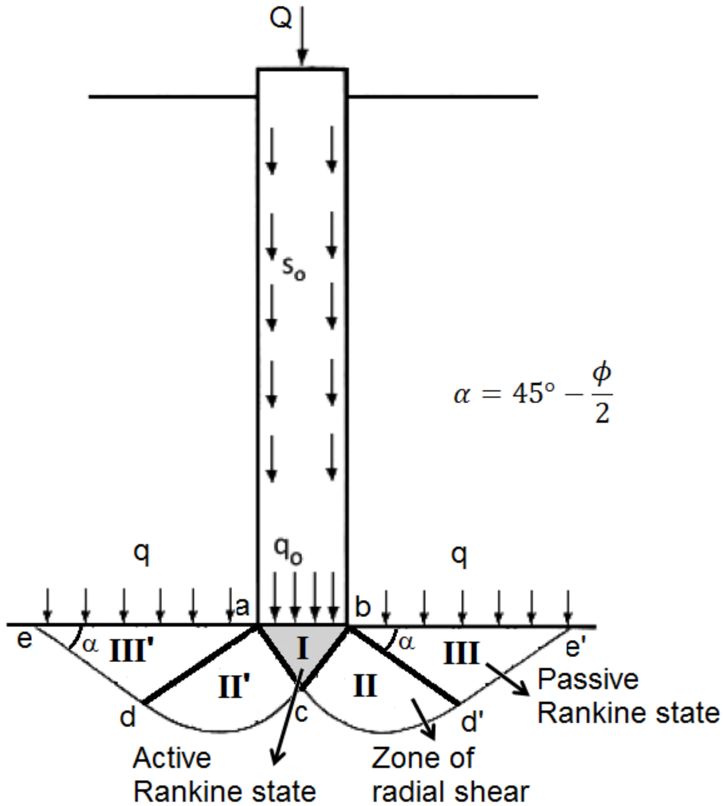


Figure 3.2 *Bearing capacity failure pattern around the pile tip assumed by Terzaghi (1943)*

The soil mass is divided by two planes into three zones with different shear patterns. The plane ad inclines toward the left at an angle of α (i.e., α = 45° - ϕ/2) to the horizontal line and the other plane ac toward the right at an angle of 45° + ϕ/2. The zone (I) indicate the active Rankine state and also zones (III) and (III') represent the passive Rankine state. *The two active and passive Rankine zones are divided by a zone of radial shear.*

The general form of Terzaghi (1943) method for estimating the base bearing capacity of single piles is a superposition of influence of soil cohesion, c', overburden pressure, q, and the soil unit weight, γ, which is calculated based on limit equilibrium (Zhu and Michalowski, 2005) and given as below:

$$Q_b = A_b(c'N_c s_c + \bar{q}N_q + \tfrac{1}{2}\gamma B N_\gamma s_\gamma) \quad \textbf{(3.14)}$$

where, A_b = pile base area, c' = soil cohesion, $\overline{q}$ = surcharge load, γ = total unit weight of soil, B = pile diameter, N_c, N_q, and N_γ = bearing capacity factor, s_c and s_γ = shape factor.

The bearing capacity factors can be estimated using the following relationships:

$$N_q = \frac{a}{\cos^2(45+\frac{\phi'}{2})} \quad \textbf{(3.15)}$$

$$N_c = (N_q - 1)\cot\phi' \quad \textbf{(3.16)}$$

$$N_\gamma = \frac{\tan\phi'}{2}\left(\frac{K_{py}}{\cos^2\phi'} - 1\right) \quad \textbf{(3.17)}$$

where, a = a coefficient related to the internal angle of friction, $K_{py} = \tan^2(45 + \phi'/2)$

Figure 3.3 expresses the relationship between the bearing capacity factors and the angle of internal friction angle, ϕ'. The bearing capacity factors N_c and N_q have been calculated using analytical method assuming the soil weightless by various investigators (Terzaghi 1943, Meyerhof 1951, Sokolovskii 1963, Vesic 1973, Chen 1975, Bolton and Lau 1993). These studies estimate the bearing capacity factors N_c and N_q with small differences and approximately the same. However, there is a large scatter in estimated values of the bearing capacity factor N_γ by different researchers, which highlights the theoretical uncertainty associated with this parameter (Ukritchon et al., 2003).

The shape factors used in Terzaghi (1943) equation are defined in Table 3.1. These shape factors were proposed based on empirical or semi-empirical considerations using the test data of Golder et al. (1941). These shape factors are introduced as shape modifiers to convert the bearing capacity factors from plain strain to axisymmetric conditions.

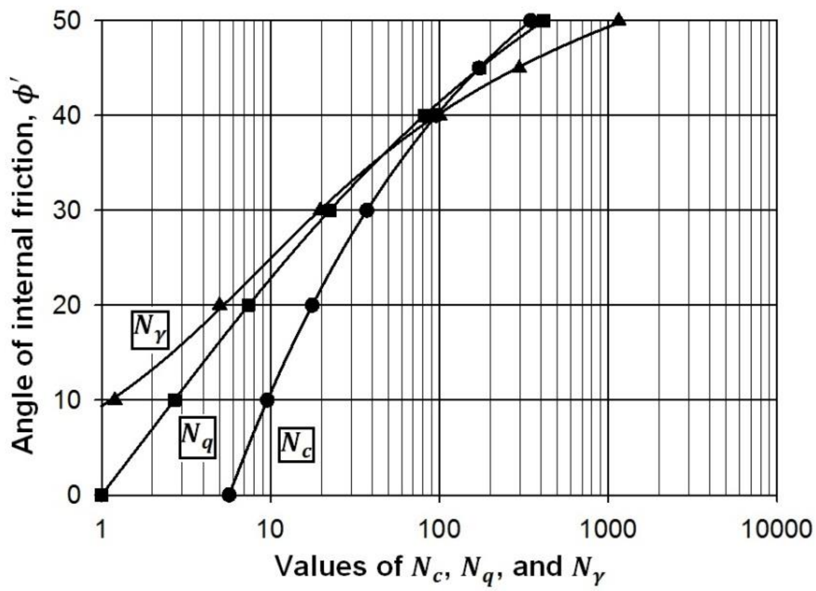


Figure 3.3 *Bearing capacity factors (data from Bowles 1996)*

Table 3.1 *Terzaghi (1943) shape factors for various foundations*

Shape Factor	Strip foundation	Round foundation	Square foundation
S_c	*1.0*	*1.3*	*1.3*
S_γ	*1.0*	*0.6*	*0.8*

Terzaghi (1943) did not take into account the contribution of matric suction towards the bearing capacity of soils; hence, using the conventional method will be conservative for soils that are in a state of unsaturated conditions.

3.3.2.2 Hansen (1970)

The proposed method is an extension of the Meyerhof (1951) work on the effect of footing base on bearing capacity. This method allows any D/B (i.e., embedment depth to foundation diameter ratio) and consequently can be used for both shallow and deep foundations. Hansen (1970) proposed that all the loads applying on the foundation are combined into one resultant with two components, V, which is normal to the base of the *foundation and H, which is in the base. The intersection of these two components is called load center. The general form of the proposed method is as given below:*

$$Q_b = A_b(cN_c d_c + \eta \bar{q} N'_q d_q + \frac{1}{2}\gamma' B N_\gamma) \quad \textbf{(3.18)}$$

The bearing capacity factors used in this method can be estimated using the equations as below:

$$N_q = e^{\pi \tan \phi'} \tan^2(45 + \frac{\phi'}{2}) \quad \textbf{(3.19)}$$

$$N_c = \left(N_q - 1\right) \cot \phi' \quad \textbf{(3.20)}$$

$$N_\gamma = 1.5 \left(N_q - 1\right) \tan(1.4\phi') \quad \textbf{(3.21)}$$

The relationship between the bearing capacity factors and the angle of internal friction is shown in Figure 3.4.

In order to calculate the depth factor (i.e., d_c and d_q) Hansen proposed the following equations:

$$\frac{D}{B} > 1 \begin{cases} d_c = 1 + 0.4\, tan^{-1} \frac{D}{B} \\ d_q = 1 + 2\, tan\, \phi' \,(1 - sin\, \phi')^2\, tan^{-1} \frac{D}{B} \end{cases} \quad \textbf{(3.22)}$$

where: D = pile embedment depth, B = foundation diameter.

Using the Hansen (1970) method will be conservative for unsaturated soils.

3.3.2.3 Janbu (1976)

The failure mechanism proposed by Terzaghi (1943) leads to conservative results as the assumed mechanism is not consistent with the actual ground movement in practice (Meyerhof, 1948). The height of the failure surface for deep foundation will not end at the pile base level. Estimating the height of the failure surface with respect to pile base level which indicates the level where the shearing strength of the soil is mobilized becomes uncertain. In an attempt to alleviate this uncertainty, Janbu (1976) extended the *previous analysis of plastic equilibrium of a surface footing to deep foundations. Figure 3.5 illustrates the proposed failure mechanism. The zone of plastic equilibrium increases as a function of foundation diameter from pile base level up to a maximum height (i.e., 6-10 pile diameter). The central zone ABC below the pile base remains in an elastic state of equilibrium and acts as a part of the foundation. Two other zones are generated at the ultimate bearing capacity, namely; a radial shear zone, BCD, inclines toward the right at an angle of the ψ (i.e., ψ varies from 60º in soft compressible to 105º in dense soils) and a mixed shear zone, BDE, where the shear changes between the limits of radial and plane shear. Janbu proposed the following equation for single pile base resistance estimation.*

$$Q_b = A_b(cN'_c d_c + \eta\bar{q}N'_q d_q + \frac{1}{2}\gamma' BN'_\gamma) \quad \textbf{(3.23)}$$

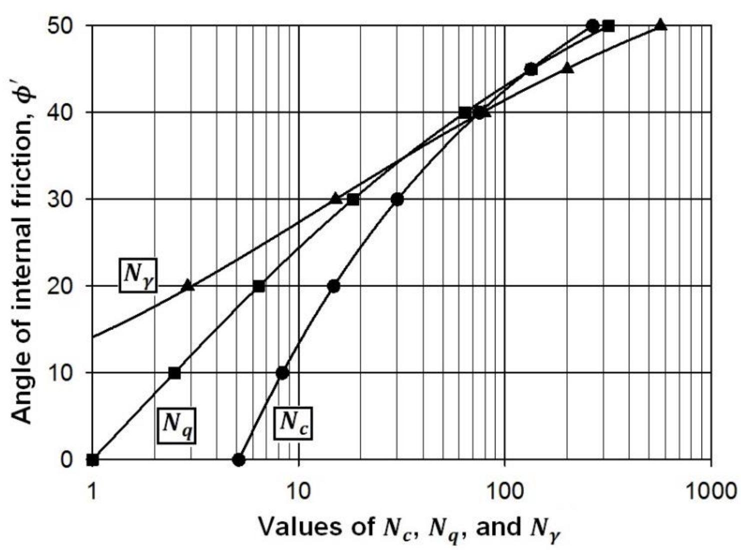


Figure 3.4 *Bearing capacity factors (data from Bowles 1996)*

The bearing capacity equation is of the same form as the Terzaghi (1943) equation; however, the bearing capacity factors N'_q and N'_c are calculated using recommended ψ values for different types of soil. The bearing capacity factor N'_γ is same as Hansen (1970) method. The variation of bearing capacity factors versus the angle of internal friction, ϕ', is shown in Figure 3.6.

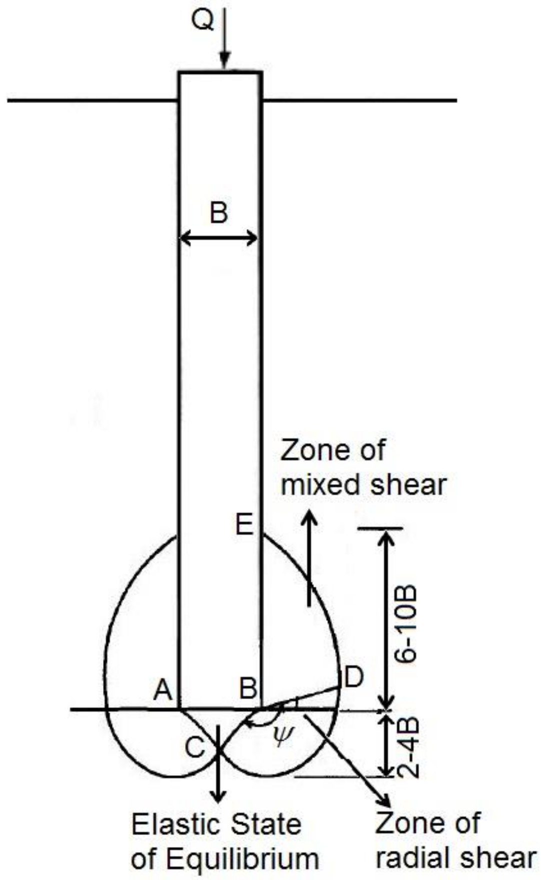


Figure 3.5 *Bearing capacity failure pattern around the pile tip assumed by Janbu (1976)*

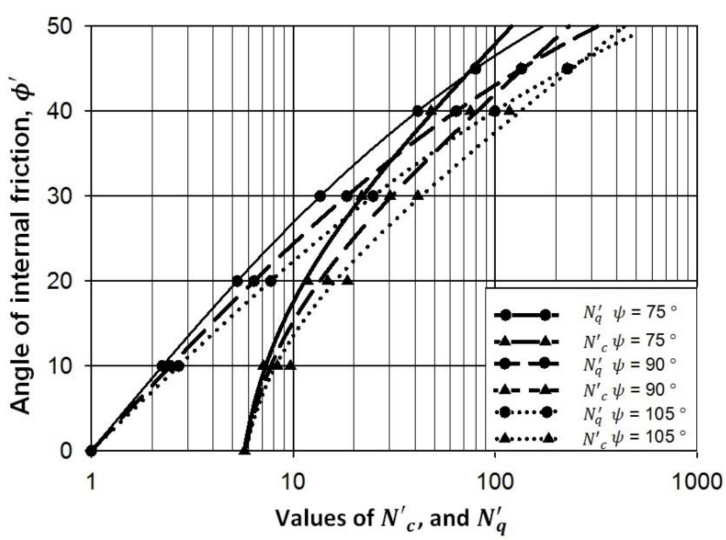


Figure 3.6 *Bearing capacity factors (data from Bowles 1996)*

3.4 Modified Single Pile Base Bearing Capacity in Unsaturated Soils

In the present research study, the selected conventional methods (Terzaghi 1943, Hansen 1970, and Janbu 1976) are modified for estimating the model pile base capacity for unsaturated soils.

In order to modify the Terzaghi (1943) pile bearing capacity equation, shear strength contribution due to influence of matric suction can be added to the effective cohesion. Several investigators including Vanapalli and Mohamed (2007) and Vanapalli et al. (2007) have extended the approach as given below:

$$q_b = [c' + (u_a - u_w)\,tan\,\phi^b]N_C + qN_q + 0.5B\gamma N_\gamma \tag{3.24}$$

The contribution of shear strength with respect to matric suction, tan ϕ^b, can be estimated using the proposed relationship by Vanapalli et al. (1996) using the SWCC and by substituting the shear strength contribution due to matric suction term as tan $\phi^b = S^\kappa$ tan ϕ'; where S is the degree of saturation. The fitting parameter, κ, is used to take into account the non-linear behavior of the shear strength of unsaturated soils. Vanapalli and Mohamed (2007) suggested using the same approach for interpreting the bearing capacity in unsaturated soils using a bearing capacity parameter as ψ_{BC}:

$$q_b = \left[c' + (u_a - u_w)S^{\psi_{BC}}tan\phi'\right]N_c + qN_q + 0.5B\gamma N_\gamma \tag{3.25}$$

$(u_a - u_w)S^{\psi BC}tan\phi'$, is the bearing capacity contribution due to matric suction. To overcome the limitation of using a bearing capacity fitting parameter, ψ_{BC}, which is dependent on experimental results, Vanapalli and Mohamed (2007) proposed a relationship between the bearing capacity parameter, ψ_{BC}, and plasticity index, I_p, as given below:

$$\psi_{BC} = -0.0031\left(I_p\right)^2 + 0.3988\left(I_p\right) + 1 \tag{3.26}$$

In case of coarse-grained soils the plasticity index is equal to zero; consequently the bearing capacity parameter becomes equal to one. Taking into account the contribution of *matric suction up to air-entry value,$(u_a - u_w)_b$, and beyond this point, Eq. 3.22 can be modified as below:*

$$Q_b = A_b([c' + (u_a - u_w)_b(1 - S^{\psi_{BC}})\tan\phi' + (u_a - u_w)_{AVR} S^{\psi_{BC}} \tan\phi']N_c s_c + \sigma'_{vb} N_q + \frac{1}{2} B\gamma N_\gamma s_\gamma \quad \textbf{(3.27)}$$

where, σ'_{vb} = the vertical effective stress at the depth of pile base, $(u_a - u_w)_{AVR}$ = the average matric suction in depth of the stress bulb, which can be estimated by considering average value of matric suction immediately below the pile tip and the matric suction at the depth of stress bulb (i.e., 1.5 pile diameter) which was proposed by Vanapalli and Mohamed (2007) :

$$(u_a - u_w)_{AVR} = \frac{1}{2}[(u_a - u_w)_1 + (u_a - u_w)_2] \quad \textbf{(3.28)}$$

where, $(u_a - u_w)_1$ = the matric suction right below the pile toe and $(u_a - u_w)_2$ = the matric suction at the depth of stress bulb (see Figure 3.7).

The philosophy of taking into account the shear strength contribution due to matric suction for modifying the Terzaghi (1943) method can be extended for the Hansen (1970) and Janbu (1976) methods (i.e., Eq. 18 and 24) for unsaturated conditions as below:

$$Q_b = A_b([c' + (u_a - u_w)_b(1 - S^{\psi_{BC}})\tan\phi' + (u_a - u_w)_{AVR} S^{\psi_{BC}} \tan\phi']N'_c d_C + \sigma'_{vb} N'_q d_q + \frac{1}{2} B\gamma N_\gamma) \quad \textbf{(3.29)}$$

The proposed modified methods (i.e., Eq. 3.27 and 3.29) can be used to interpret and estimate the pile base bearing capacity under both saturated and unsaturated conditions.

3.5 Summary

In conventional engineering practice, the influence of matric suction towards the single pile bearing capacity is typically ignored. The conventional methods proposed by Terzaghi (1943), Hansen (1970), Janbu (1976) were modified such that they can be used for interpreting the load carrying capacity of single piles in unsaturated soilss. The *modified equations take the conventional methods form by setting the matric suction value to zero. In addition, semi-empirical methods are proposed for predicting the variation of the load carrying capacity of single piles. The conventional saturated shear strength parameters (i.e., c' and ϕ') and information from SWCC (i.e., air entry value, average matric suction value below the pile base, and the corresponding degree of saturation for each matric suction value) are required for using the proposed semi-empirical equations.*

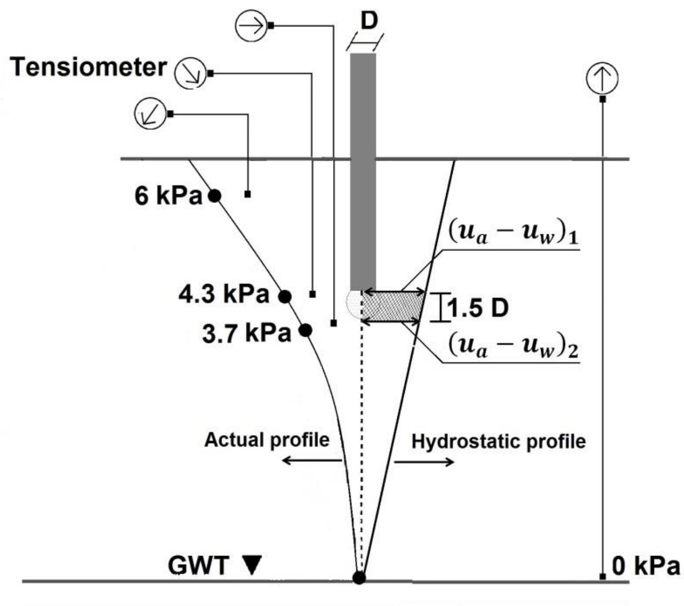


Figure 3.7 *Schematic to demonstrate the procedure used for determining the average matric suction below the pile base (Modified after Vanapalli and Mohamed 2007)*

CHAPTER 4

EQUIPMENT AND METHODOLOGY

4.1 Introduction

This chapter provides details of equipment used for measuring the bearing capacity (i.e., total, shaft, and end bearing capacity) along with $p - \delta$ behavior of single mode pile foundation under axial loading for two compacted non-plastic soils under both saturated and unsaturated conditions. The purpose of the testing program was to measure the effect of matric suction on the total, shaft, and end bearing capacity of single model piles in unsaturated coarse-grained soils. The soil water characteristic curve (SWCC) of the two compacted non-plastic soils (i.e., Soil #1: Unimin 7030 sand and Soil #2: Industrial sand) were measured using the Tempe cell apparatus. In order to measure the variation of the matric suction of the soil profile in the test tank, commercial Tensiometers were placed at different depths in the test tank above the water table. A varying suction profile was achieved in the test tank by varying the ground water table depth using the hanging column technique. The effective shear strength parameters (i.e., c' and ϕ') and the soil-pile interface strength parameters (i.e., c'_a and δ') required to estimate the single model pile bearing capacity were measured using direct shear apparatus. The key details of the test equipment used in this testing program are briefly summarized in this chapter.

4.2 Model Pile Load Test Program

In order to determine the bearing capacity of single model pile foundations, a series of model pile load tests were conducted. Model piles were located in a specially designed test tank and loaded (i.e., in compression) under both saturated and unsaturated conditions. The key features of the model pile load test program equipment are presented in this chapter. Figure 4.1 shows the details of the model pile load test setup along with a *soil container and other facilities. All the bearing capacity test system accessories are described in following sections.*

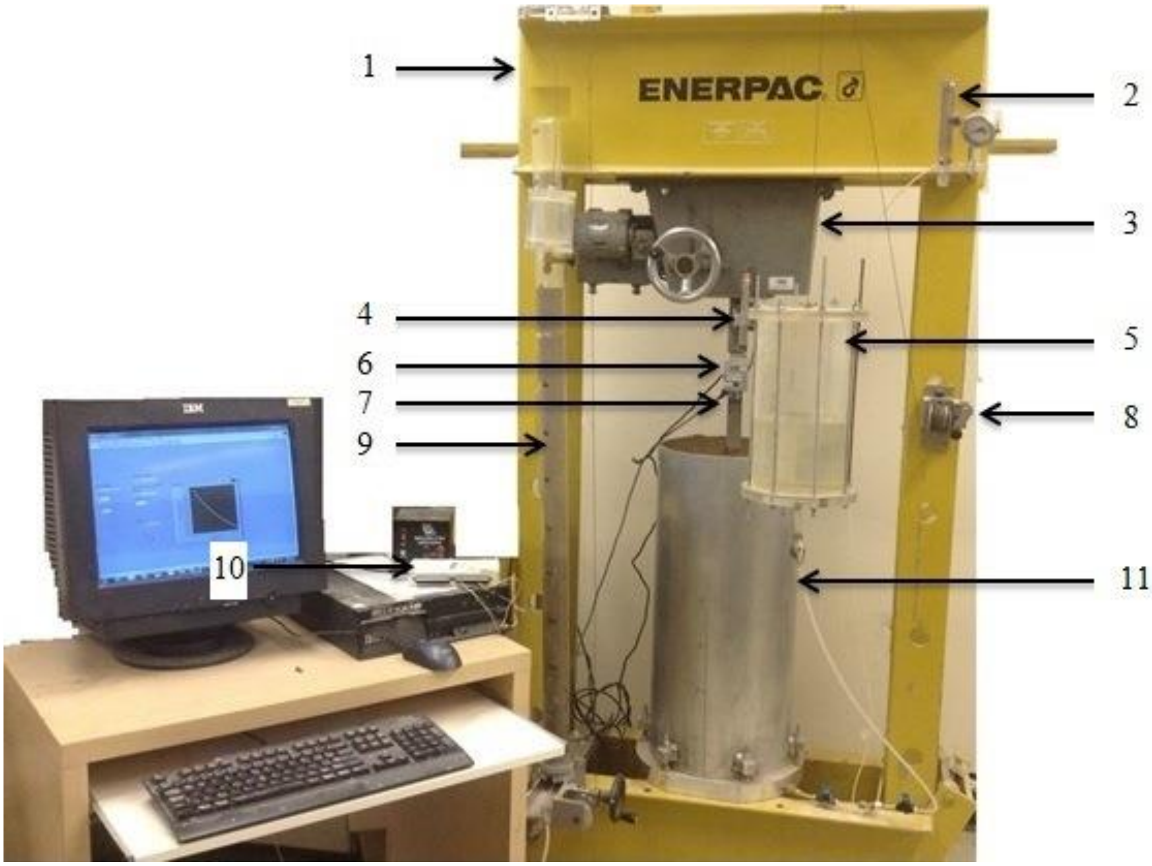


Figure 4.1 *Bearing capacity test system : 1. Loading frame, 2. Tensiometer, 3. Loading Machine, 4. Displacement transducer, 5. Water container, 6. Load cell, 7. Model pile, 8. Pulley system, 9. Suction profile set, 10. Data acquisition system, 11. Soil container*

4.2.1 Loading frame and loading machine

The test set up was located on an ENERPAC loading frame. The loading frame is a steel H-frame press made by ENERPAC Hydraulic Technology which is 1930 mm in height and 1030 mm in width and provides loading capacity of 50 tons. An electrically operated and mechanically controlled loading machine is attached to the top of the loading frame by bolts to load the model pile into the soil. The maximum loading capacity of the loading machine is approximately 15 kN.

4.2.2 Water reservoir

In order to adjust the water level in the soil container, a plexy-glass cylindrical container 300 mm in height and 150 mm in diameter is used as a water reservoir with drainage valve connections to the base of the soil container (see Figure 4.1). The water container facilitates water drainage into and out of the soil container and is useful in saturating or desaturating the soil compacted in the test tank. The water level in the soil container can be adjusted by sliding the water container upward and downward with a pulley system. The water container unit acts as a hanging water column to achieve different capillary stress values above the water table in the soil profile of the soil container. In addition to valves which were used to control the water drainage into and out of the soil container, a piezometer was designed and installed to measure the water level in the soil container.

4.2.3 Soil container

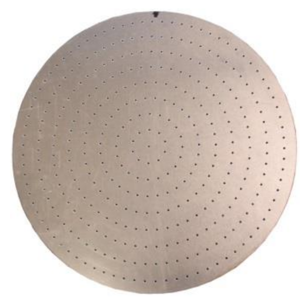
A soil container was used as a test tank to determine the bearing capacity of single model piles. The soil container was fabricated from an aluminum tube of 300 mm in diameter, 700 mm in height and 8 mm in thickness. The soil container consists of three elements; base plate, lid, and geotextile (Figure 4.2). The base plate with circular grooves enables the gradual drainage of water into and out of soil. A stainless steel lid 260 mm in diameter and 1.25 mm in thickness with small holes 2 mm in diameter covers the base plate. A geotextile was placed on the lid to prevent the soil particles from clogging circular grooves of the base plate and valves. The soil container facilitates the water table to be lowered to a depth of 700 mm below the soil surface. Different matric suction (i.e., capillary stresses) values were achieved by varying the depth of the water table below the single model pile base. By performing the single model pile load tests in the soil container the load displacement ($p - \delta$) behavior of the model piles was determined.

4.2.4 Suction profile set

The suction profile set designed by Li (2008) used to pre-calibrate the relationship between the matric suction (i.e., capillary stresses) and the soil depth in the soil container by varying the water table below the soil surface (Figure 4.3). The matric suction was measured using two commercial Tensiometers located within the soil profile in different *depths in the suction profile set. The estimated time to reach the equilibrium condition in the suction profile set for the two sands studied in this research program was 24 to 48 hours.*

Fully assembled soil container

(a) Base Plate



(b) Lid

(c) Geotextile

Figure 4.2 *Various components of the soil container (Vanapalli et al., 2011)*

The unit consists of a transparent plexi-glass soil column 50 mm in diameter and 1500 mm in height, a water container, thin plastic tube, valves and a pulley block. The depth of the ground water table in the unit was adjusted to the desirable depth by sliding upward and downward the water container using the pulley system.

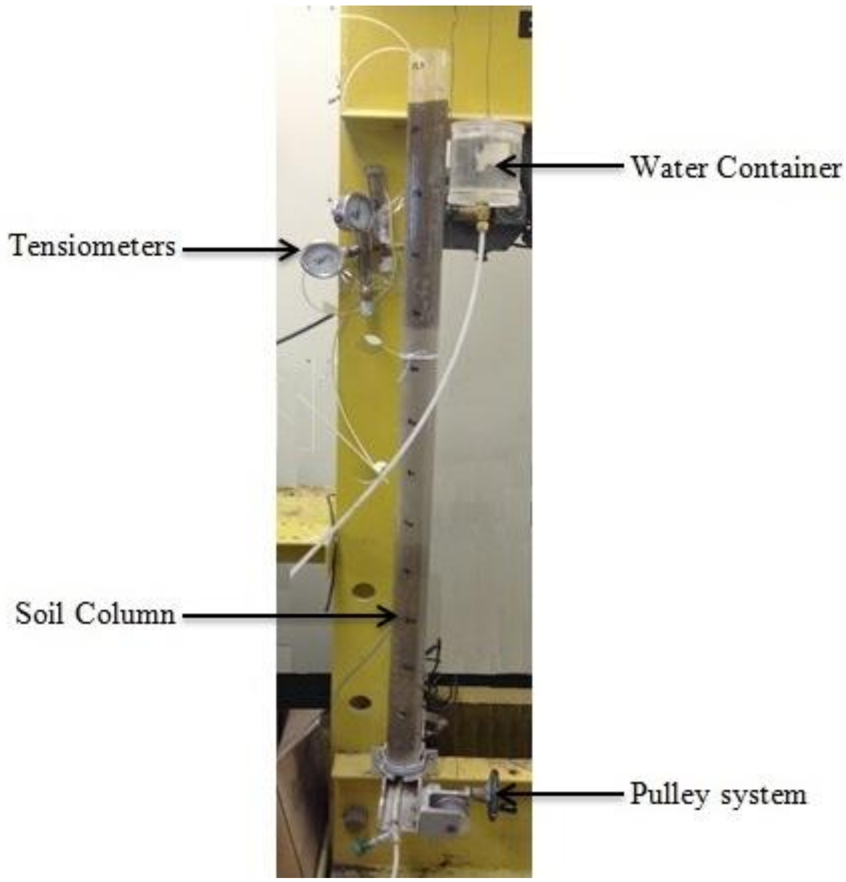


Figure 4.3 *Schematic diagram of suction profile set*

4.2.5 Single model piles

Three cylindrical solid stainless steel model piles with base diameter equal to 38.3, 31.75, and 19.25 mm and 350 mm long were used in the testing program (Figure 4.4). The steel used in this study is cold finished round tube steel with surface roughness of 12 μm (i.e., average roughness, R_a) The objective of using three different pile diameters in the study is to understand the influence of model pile diameter on the pile bearing capacity behavior. The model pile diameters were decided based on two guidelines suggested by Bolton et al. (1999) to prevent the boundary and scale effects for performing the model pile capacity estimation tests. The two guidelines are (i) the ratio of soil container diameter to pile diameter (i.e., $D_{container}/D_{pile}$ should be greater than 8 to avoid boundary effects; and also, (ii) the pile diameter should be greater than 20 × D_{50} size of the soil such that the scale effect is negligible. Table 4.1 summarizes various model pile and soil *container dimensions used to study the load-displacement (i.e., p-δ) behavior of model piles. The depth of the soil container was deeper than the expected depth of the stress bulb (i.e., 1.5 to 2 D_{pile}) below the model pile (i.e., approximately 15 to 25 D_{pile}).*

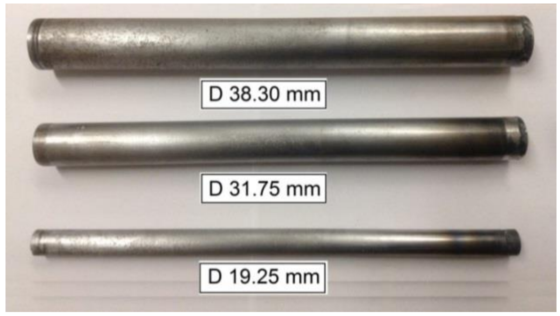


Figure 4.4 *Three different model piles with varying base diameters*

Table 4.1 *Dimension characteristics of model pile tests (Taylan, 2013)*

Reference	*Soil container diameter (mm)*	*Model pile diameter (mm)*	*Soil container to model pile diameter ratio*
Vesic (1963, 1964)	*2450*	*50.8-171.5*	*15-50*
Hanna and Tan (1973)	*610 x 610*	*15.7-38*	*16-39*
Tan and Hanna (1974)			
Das et al. (1977)	*475 x 610*	*25.4*	*23*
Chaudhuri and Symons (1983)	*1100*	*25.4-50*	*22-43*
Kerisel (1964)	*6400*	*40-320*	*20-160*
Robinsky and Morrison (1964)	*501 x 711*	*20.5-37.5*	*24-13*
Shin et al. (1993)	*457 x 457*	*25.4*	*18*
Al-Mhaidib (2001)	*450*	*30*	*15*
Mayoral et al. (2005)	*510*	*51*	*10*
Nanda and Patra (2011)	*750*	*32*	*23.5*

Numerical modeling was performed using the SIGMA/W software on the stress bulb formation (i.e., due to the loading) below the pile base. In the present numerical modeling, the model pile was loaded by increments of vertical displacement on its top using a force-displacement boundary condition (i.e., displacement rate -0.0025 m/s). Result of the study shows that the stress bulb is not influenced by the boundary conditions of the soils container used in the testing program. The numerical analysis confirms that the selected soil container dimensions (i.e., diameter and height of the soil container) used to study the load-displacement (i.e., p-δ) behavior of model piles are appropriate and are not interfering with the developed stress bulb (see Figure 4.5). Details of the performed numerical study are discussed in Chapter 7.

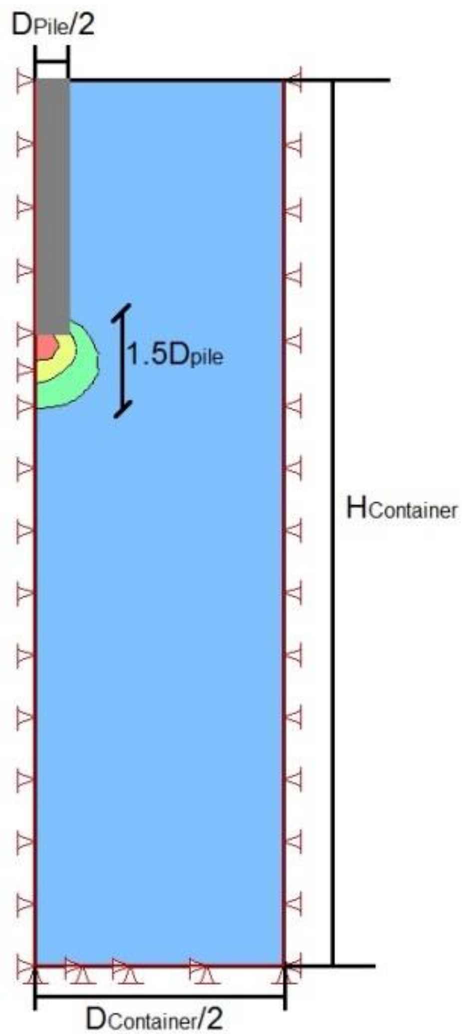
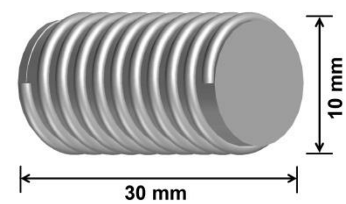


Figure 4.5 *Stress bulb formation below the pile base*

To perform the single model pile base resistance estimation test, the model pile was located inside a hollow sleeve which is 45 mm in diameter (Figure 4.6 (a)). By placing the pile inside the hollow sleeve, any contact between the model pile shaft and soil particles during the load test was prevented (Figure 4.7). On the other hand, the contribution of the model pile shaft resistance towards the total bearing capacity can be reliably determined by placing a pile base below the model pile toe (Figure 4.6 (b)). The pile base is a hollow cylinder that is 45 mm in diameter and 40 mm in height and 1 mm in thickness with holes in the bottom part to allow free movement of water from the pile base during pile loading. The pile base provides a gap for the model pile shaft to penetrate into the soil in the test tank and eliminates any contact between the model pile toe and soil container (model pile toe penetrates into the hollow space in the pile base). This technique helps in eliminating the contribution of model pile base resistance during the load test (Figure 4.8). In order to determine the model pile total bearing capacity, neither the sleeve nor the pile base was employed. The model pile was connected to the loading machine via the detachable screw connection. (Figure 4.6 (c)).

(a)

(b)



(c)

Figure 4.6 *(a) Sleeve (b) Pile base (c) Detachable screw connection*

4.2.6 Load displacement measurement system

The corresponding vertical displacement of the model pile was measured by a Linearly Variable Displacement Transducer (LVDT). The lightweight LVDT is fixed next to the model pile on a magnetic holder. The maximum displacement capacity of the used LVDT in the research program is 50 mm. In order to measure the applied load to the model pile form the loading machine, a lightweight ARTECH S beam load cell 20210, with loading capacity of 500 lb was used. The corresponding applied load and vertical displacement of the model pile can be measured and recorded in the form of output voltage which was *recorded by the Data Acquisition System (DAS) NI-USB 6210 a product of National Instrument Corporation.*

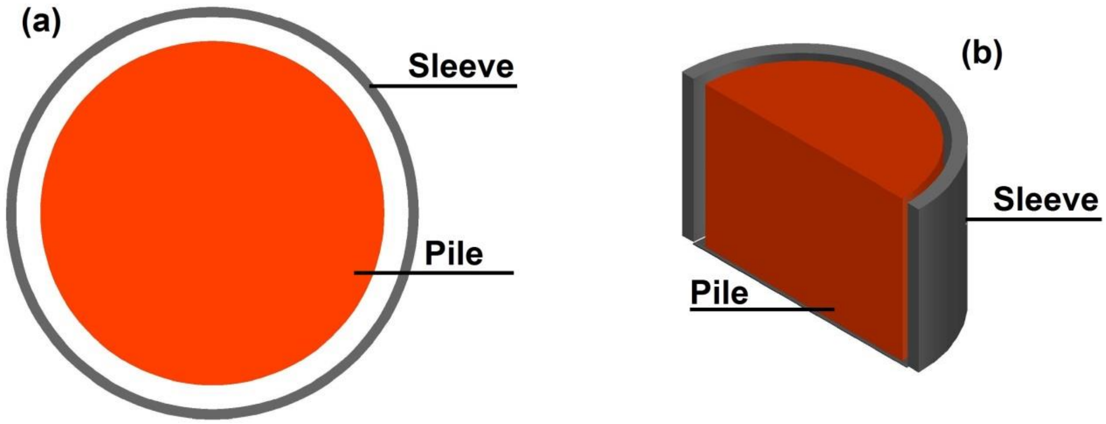


Figure 4.7 *(a) Pile-sleeve plane view* *(b) Pile-sleeve cross section view*

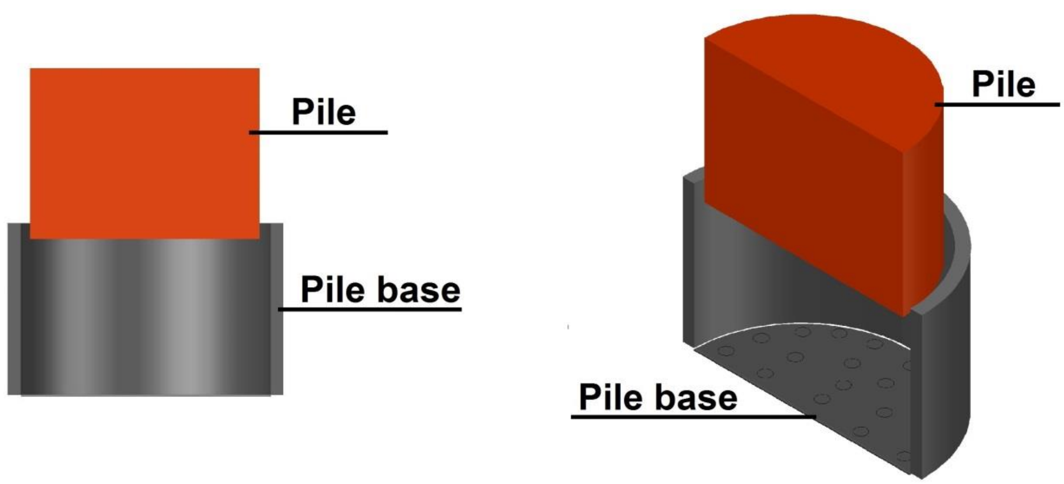


Figure 4.8 *Pile-pile base cross section view*

4.2.7 Single model pile load test procedure

The total, shaft, and base (i.e., end) bearing capacity of the single model pile were measured separately in the two selected soils performing series of pile load test. The properties of the two tested soils are presented in Table 4.2. The required procedures for measuring the bearing capacity of the single model pile are detailed in following sections. The reported test results were the average values of the three tests results). The variation of each set of test results (i.e., base, shaft, or total capacity) was less than ± 5 to 7 %.

Table 4.2 *Tested soils properties*

Soil Property	*Unimin 7030 sand*	*Industrial sand*
Soil friction angle, ϕ' (°)	35.3	40.3
Effective cohesion, c' (kPa)	2.6	3.3
Soil-steel interface friction, (δ')	24.2	33.1
Specific gravity, G_s	2.65	2.69
Optimum water content, w_{opt} (%)	14.6	15.2
Maximum dry unit weight, $\gamma_{dry(max)}$, (kN/m^3)	16.8	17.7
Total unit weight, γ_{total}, (kN/m^3)	18.6	19.9
Saturated unit weight, γ_{sat}, (kN/m^3)	20.4	20.8
Void Ratio, e	0.63	0.56
D_{60}, (mm)	0.22	0.29
D_{30}, (mm)	0.18	0.17
D_{10}, (mm)	0.12	0.1
Coefficient of uniformity, C_u	1.83	2.9
Coefficient of curvature, C_c	1.23	0.99

4.2.7.1 Measuring the total bearing capacity of single model pile

The procedures used for measuring the total bearing capacity of the model pile in the present study can be summarized as below:

i. *The soil was compacted in five layers (i.e., 100 mm height of each layer) using a 1 kg hammer to achieve uniform density condition over the entire depth of the soil in the test tank. The average density index, D_r, was 65% at optimum moisture content obtained from the compaction test.*

ii. *The model pile was connected to the loading machine via the detachable screw connection (Figure 4.6 (c)).*

iii. *The soil around the pile was placed in two compacted layers (i.e., 100 mm height of each layer). No discontinuity between the pile and soil was allowed* *during the installation procedure to assure measuring the total bearing capacity (i.e., arising from both shaft and base resistance).*

iv. *The load was applied to the single model pile head through mechanically controlled and electrically operated loading machine.*

v. *The water table level was monitored periodically using the installed piezometer.*

vi. *The applied load and displacement were measured using the load cell and LVDT.*

During the pile installation procedure, care was taken to assure the model pile is always vertical. In present research program, the single model pile was penetrated to a depth of 20 mm. The single model pile was loaded at a rate of 0.7 mm/min. The selected loading rate for the present study was similar to strain rates used by other investigators for testing sands (i.e., 1.0 mm/min from Vanapalli and Taylan 2011 and 1.2 mm/min from Mohamad and Vanapalli 2006).

The model pile installation procedure is illustrated in Figure 4.9.

Figure 4.9 *Pile installations for determination of single model pile total bearing capacity*

4.2.7.2 Measuring the base capacity of single model pile

In order to measure the single model pile base bearing capacity the single model pile surrounded by the hollow sleeve (Figure 4.7) was connected to the loading machine. The same procedure detailed earlier for the total bearing capacity measurement was followed. The hollow sleeve was used to eliminate any contact between the pile shaft and surrounding soil; such a technique facilitates eliminating the contribution of pile shaft resistance and measures only the pile base capacity (Figure 4.10)

Figure 4.10 *Pile installation for determination of single model pile base bearing capacity*

4.2.7.3 Measuring the shaft capacity of single model pile

The procedure discussed for total and base bearing capacity measurement were followed to determine the single model pile shaft capacity. The model pile was placed in the pile base to introduce a gap between the model pile base and the soil particles (Figure 4.8). A thin flexible plastic film had been provided to cover the pile base to prevent any soil particles to fall and enter into the pile base during the pile installation and model pile load test. An aperture with diameter equal to model pile diameter was made in the flexible plastic film to accommodate model pile penetration into the pile base through the flexible plastic film. This technique facilitates measurement of only the model pile shaft capacity and preventing any contact between the pile toe and soil particles (Figure 4.11).

4.2.8 Model pile load test under saturated and unsaturated conditions

The single model pile load test was conducted under both saturated (i.e., matric suction equals to 0 kPa) and unsaturated (i.e., matric suction value equals to 2 and 4 kPa) conditions.

In order to perform the test under the saturated condition, the oven dried soil was mixed with a water content equal to the OMC (i.e., 14.6% for Soil #1 and 15.2% for Soil #2) and compacted in layers of 100 mm in the test tank. The compacted soil was saturated in the test tank from the bottom by gradually allowing the water from the hanging water column. Using this technique facilitates air to escape the soil through the top surface. The required time to reach the equilibrium condition in the test tank was estimated 24 to 48 hours, by measuring the matric suction in the test tank with Tensiometers. The piezometer readings (i.e., measurement of the water table in the test tank) were used to back calculate the soil suction assuming the hydrostatic variation of matric suction, which were consistent with Tensiometers readings.

Figure 4.11 *Pile installation for determination of single model pile shaft capacity*

To perform the test under the unsaturated condition, the compacted soil in the test tank was saturated prior to the test following the procedures detailed earlier. After saturating the soil, the water table in the test tank was lowered to the depths of 450 and 650 mm *corresponding to the average matric suction values 2 and 4 kPa (i.e., within the stress bulb) respectively by adjusting the height of the plexi-glass water container (i.e., hanging column method). The water was allowed to drain out freely from the test tank using the drainage valves. The required time to achieve the equilibrium matric suction value in the stress bulb beneath the pile base (i.e., depth of 1.5 pile diameter below the pile base) was estimated 24 to 48 hours after lowering the water table. The matric suction was measured using the installed Tensiometers. Figure 4.12 shows a schematic of the test setup of soil container for unsaturated condition with an average matric suction value of 2 kPa (i.e., (1.7+2.3)/2)) in the stress bulb zone beneath the pile base.*

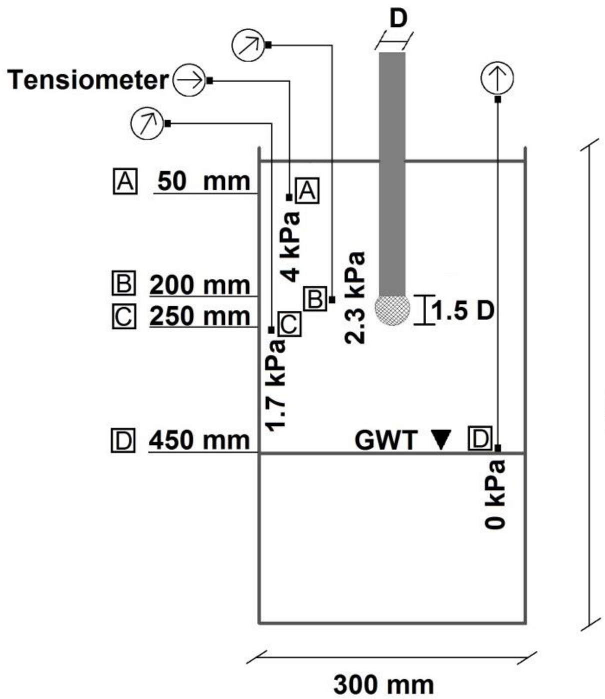


Figure 4.12 *Schematic showing cross-section details of the test setup under unsaturated condition (i.e., matric suction value equals to 2 kPa)*

4.3 Direct Shear Apparatus

The effective shear strength parameters of saturated soil (i.e., c' and ϕ') and the soil-pile interface strength parameters (i.e., c'_a and δ') were determined using the conventional direct shear test apparatus following the ASTM Standard D3080/3080M (2012). These *parameters are required for estimating the bearing capacity of single model piles in saturated and unsaturated conditions. The apparatus employed in the present study is El 26-2112 a product of ELE International. In order to facilitate applying a wide range of shear stress to the specimen, the ELE direct shear apparatus has been designed to apply a maximum shear stress of 2800 kPa for the specimen with size of 60 mm × 60 mm. Figure 4.13 presents the general assembly of the ELE direct shear apparatus. In order to determine the interface strength parameters between the pile material and the tested soils (i.e., c'_a and δ'), an interface plate with same dimension as the direct shear box (i.e., 60 mm × 60 mm) was specially designed and built at the University of Ottawa machine shop with same material and roughness as the model pile. The test was conducted by placing the interface plate into the lower part of the direct shear box and fastening the lower and upper parts of the direct shear box while the soil sample was compacted in one layer. Figure 4.14 indicates the upper part of the direct shear box and the interface plate located in the lower part of the shear box.*

Figure 4.13 *EL 26-2112 Direct shear apparatus (ELE International)*

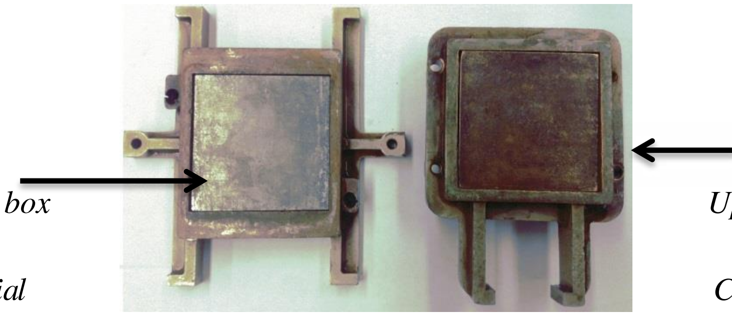


Figure 4.14 *Direct shear box interface test*

4.4 Tempe Cell Apparatus for Measuring the SWCC

The SWCC for the coarse-grained soils was determined using the Tempe cell apparatus extending the axis translation technique (Power and Vanapalli, 2010).

4.4.1 Axis translation technique

The matric suction is defined as the difference between the pore-air pressure, u_a, and the pore water pressure, u_w. Pore-air pressure is typically atmospheric in unsaturated soils (i.e., $u_a = 0$) and the pore water pressure is negative with respect to atmospheric pressure. In order to measure the matric suction of an unsaturated soil specimen in laboratory environment with matric suction value higher than the atmospheric pressure (i.e., greater than 101.3 kPa) without cavitation problem, the axis translation technique is commonly used (Hilf, 1956). The axis translation technique facilitates measuring the matric suction in unsaturated soil specimen in a controlled environment by elevating the origin of reference for atmospheric air pressure and negative water pressure to a condition of positive air pressure and pore water pressure. (Figure 4.15)

4.4.2 Tempe cell apparatus

Tempe cell apparatus is typically used for measuring the SWCC of coarse and fine-grained soils for the matric suction range from 0 to 500 kPa extending the axis translation *technique. The Tempe cell includes a saturated high air entry disk (HAED) to separate the air and water phases in a closed vessel. The SWCC was measured by measuring the water expelled from the soil sample under an applied matric suction value following the ASTM Standard D6836-02 (2002). Matric suction in the range from 0 to 20 kPa was used for this research program.*

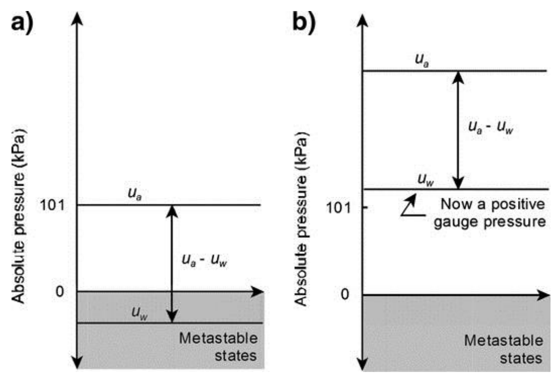
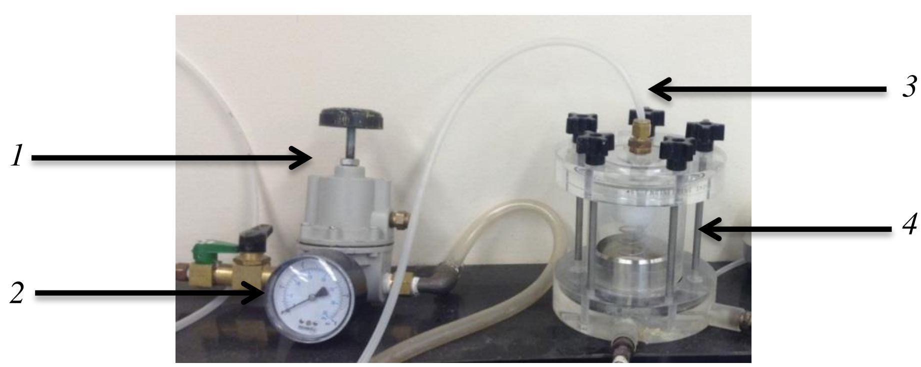


Figure 4.15 *Use of the axis translation technique to avoid cavitation problem (a) atmospheric conditions (b) axis translation (Marinho et al. 2008)*

The two sands were tested desaturated to low degrees of saturation when the matric suction value equal or greater than 15 kPa was applied. The general assembly of the Tempe cell is shown in Figure 4.16.



1. Regulator 2.Sensitive pressure gauge

3. Pressure supplier 4. Tempe cell

Figure 4.16 *Tempe cell general assembly*

4.5 Tensiometers

Matric suction in unsaturated soils is conventionally measured by commercial Tensiometers (Figure 4.17) in range of 0 to 100 kPa without cavitation problems.

4.5.1 Tensiometer structure

The commercial Tensiometer used in this study is model 2100F product of Soilmoisture Equipment Corp. in California, USA. The Tensiometer consists of a plastic tube, a porous ceramic cup sensor, vacuum dial gauge and a thin neoprene tube that transfer the negative pore water pressure in the soil from the sensor to the dial gauge.

4.5.2 Principle and methodology

The Soilmoisture Equipment commercial Tensiometer 2100F works based on the properties of the high-air entry (HAE) ceramic cup. The HAE materials are characterized by microscopic pores of relatively uniform size and size distribution. As the HAE material (i.e., ceramic cup) gets saturated with water, air cannot pass through the material as a result of the ability of the contractile skin resistance towards the air flow. Physically, the HAE material acts as a membrane between the two air and water phases.

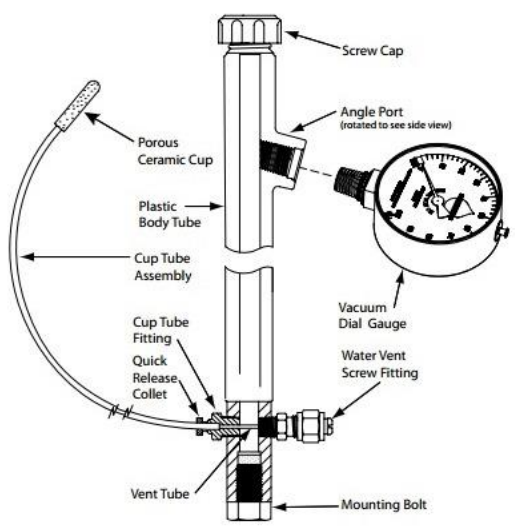


Figure 4.17 *A schematic of commercial Tensiometer (Soilmoisture Equipment Corp. 2009)*

There is an inversely proportional relationship between the maximum pore size of the HAE material and the maximum sustainable difference between the air pressure, u_a, above the HAE material (i.e., ceramic cup) and the pore water pressure, u_w, within or below the ceramic cup. This relationship can be expressed using the Young-Laplace Eq. (4.1).

$$(u_a - u_w)_b = \frac{2T_s}{R_s} \tag{4.1}$$

where, $(u_a - u_w)_b$ = the air entry value, T_s = the surface tension of the air water interface, R_s = the effective radius of the maximum pore size of the HAE material (i.e., ceramic cup).

CHAPTER 5

PRESENTATION OF TEST RESULTS

5.1 General

Specially designed single model piles described in earlier chapter were loaded in two compacted sands (i.e., Soil #1: Unimin 7030 sand and Soil # 2: Industrial sand) under both saturated and unsaturated conditions to determine their p-δ behavior in a specially designed test tank with other accessories. In this chapter, the model pile test results are presented in two sections. In the first section, the test results related to the conventional soil properties and the soil water characteristic curve (SWCC) are presented. In the second section, the test results related to the p-δ behavior of model single piles are presented.

5.2 Soil Properties Tests

The basic soil properties of the two selected sands, namely, Soil #1: Unimin 7030 sand and Soil #2: Industrial sand, were determined through a series of tests.

5.2.1 Sieve analysis and specific gravity tests

In order to determine the grain size distribution of the two selected soils, representative samples were collected from the whole batch of sands. The soil samples were air-dried for 24 hours and three sieve analysis tests were conducted on each soil type following the ASTM D422 (1994) procedures.

The average grain size distributions of the two selected soils are shown in Figure 5.1. The key parameters derived from these tests are summarized in Table 5.1.

Table 5.1 *Properties of the selected soils for the study*

Soil Property	*Soil #1*	*Soil #2*
Void Ratio, e	*0.63*	*0.56*
D_{60}, *(mm)*	*0.22*	*0.29*
D_{30}, *(mm)*	*0.18*	*0.17*
D_{10}, *(mm)*	*0.12*	*0.1*
Coefficient of uniformity, C_u	*1.83*	*2.9*
Coefficient of curvature, C_c	*1.23*	*0.99*
Specific gravity, G_s	*2.65*	*2.69*

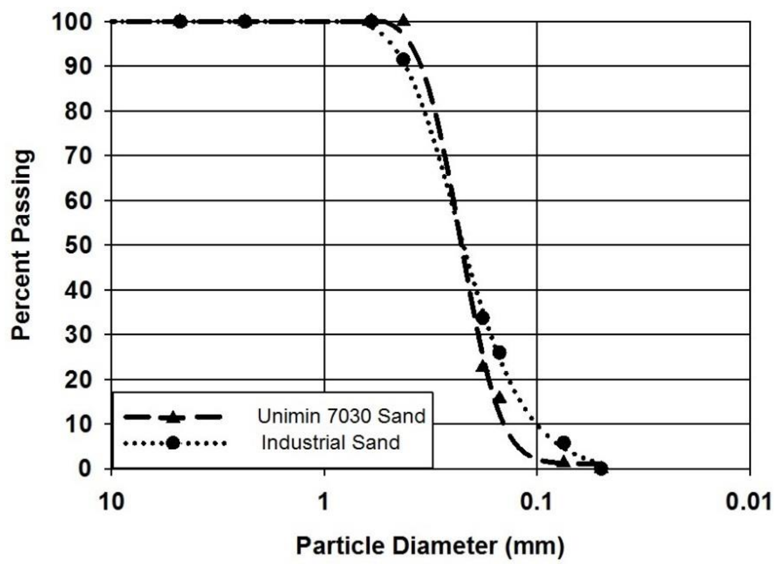


Figure 5.1 *Grain size distribution of the selected soils in the study*

The specific gravity of the selected soils was measured using the ASTM D854-10 (1994).

5.2.2 Compaction test

The compaction tests were performed following the ASTM D698-12 (1999) to determine the variation of the dry unit weight, γ_d*, with respect to moisture content, w. Figure 5.2 (a) and (b) show the test results on the two soils.*

In order to compact the soil in the bearing capacity test tank at the optimum moisture content, a 1 kg hammer was used. The maximum dry unit weight, γ_{dmax}, achieved for Soil #1 and Soil #2 is 16.8 kN/m^3 and 17.7 kN/m^3 respectively. Also, the optimum water content, $w_{(OMC)}$, was measured for soils are 14.6% and 15.2 %, respectively.

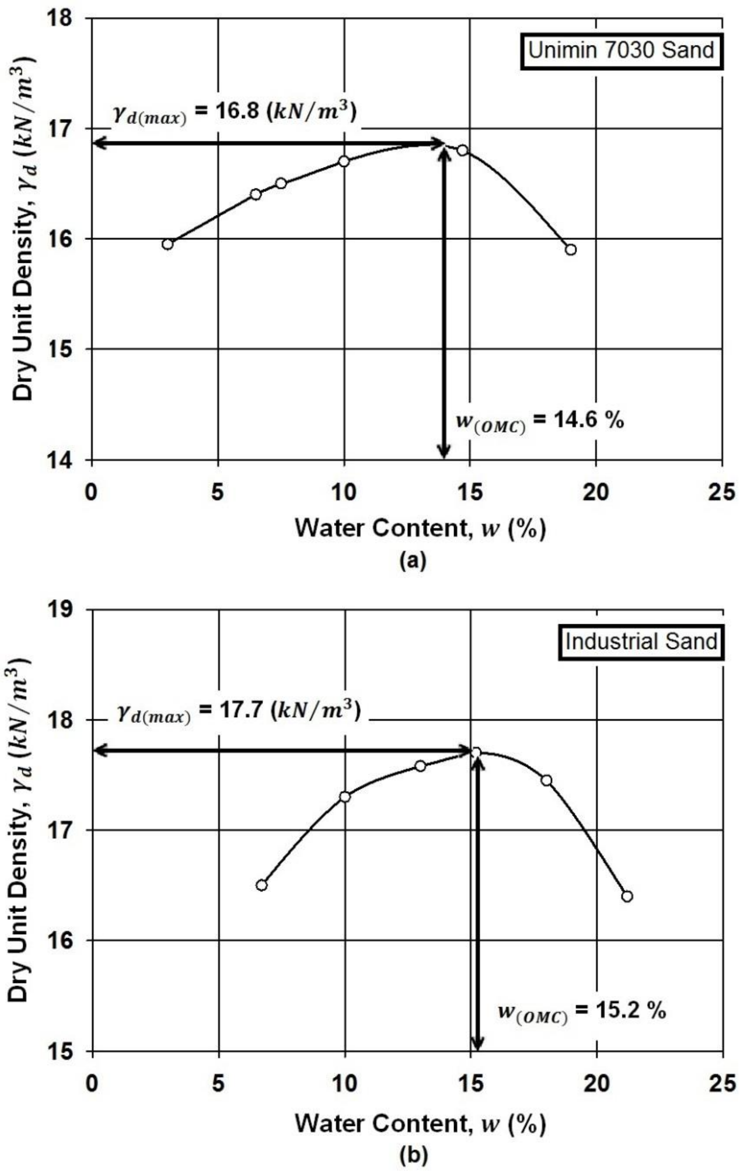


Figure 5.2 *Compaction curves for (a) Soil #1 and for (b) Soil #2.*

5.2.3 Direct shear test

The saturated shear strength parameters, namely; the effective cohesion, c', and the angle of internal friction, ϕ', of the two sands and also the soil-pile interface strength parameters (i.e., c'_a and δ') were determined using the conventional direct shear test apparatus. These results are required in the interpretation of the bearing capacity of single model piles in both saturated and unsaturated conditions. Figure 5.3 (a) and (b) show the relationship between the shear strength and the normal stress of the two studied soils. The measured shear strength parameters for Soil #1 were c' = 2.6 kPa, ϕ' = 35.3° and for Soil #2 were c' = 3.3 kPa, ϕ' = 40.3°, respectively. The measured soil-pile interface strength parameters for Soil #1 were c'_a = 0 kPa, δ' = 24.2° and for Soil #2 were c'_a = 0 kPa, δ' = 33.1° respectively.

5.2.4 Soil-water characteristic curve estimation test

Soil water characteristic curve (SWCC) for the two selected sands was measured following the procedures detailed in Chapter 4 (see section 4.4). The measured SWCC from the Tempe cell was compared with one-point prediction technique proposed by Vanapalli and Catana (2005). The one-point prediction technique establishes a relationship between the parameters (a, n, m) of the Fredlund and Xing (1994) SWCC estimation model and soil properties to determine the SWCC. The parameter can be derived from simple soil properties such as grain-size distribution curve and the volume mass properties, along with one measured data point of suction versus water content. The correlated parameters are given as below:

$$a = \frac{1.33}{(d_e)^{0.86}} \tag{5.1}$$

$$n = \frac{7.78}{[C_u \times e]^{1.14}} \tag{5.2}$$

$$m = x \tag{5.3}$$

where, C_u = coefficient of uniformity, e = void ratio, x = adjustable variable for the estimated SWCC to match the one measured data point, d_e = dominant particle-size *diameter from Vukovic and Soro (1992). Figure 5.4 illustrates the comparison between the measured and the predicted SWCC.*

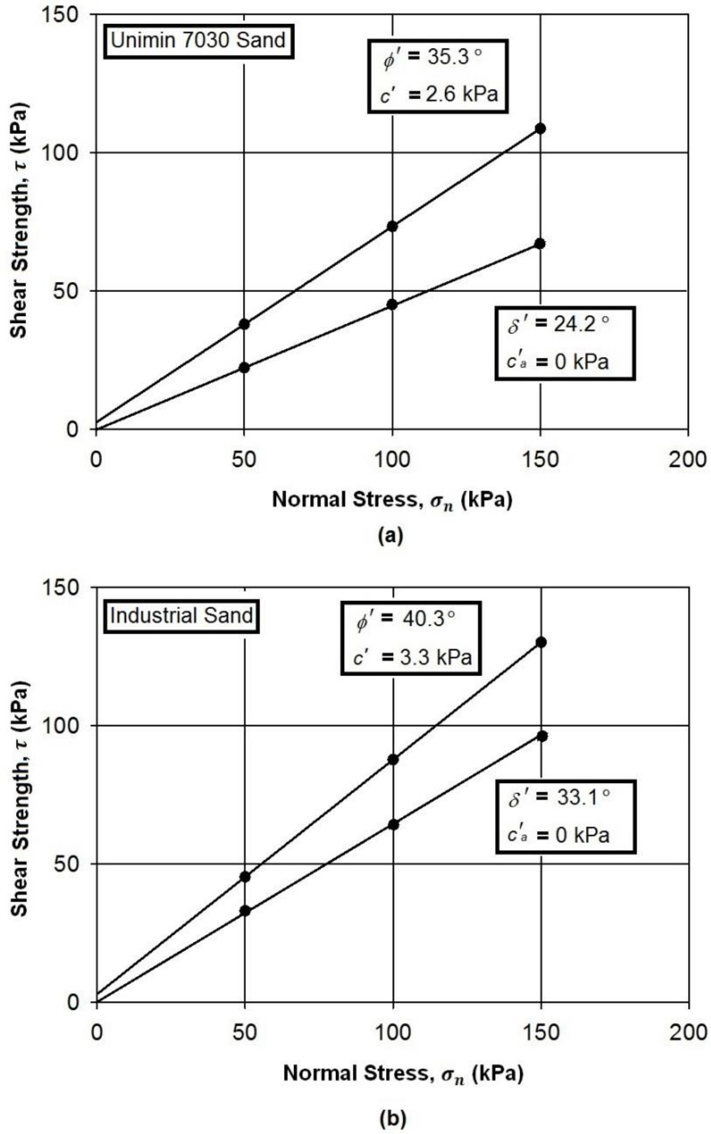


Figure 5.3 *Direct shear test results for (a) Soil #1 and for (b) Soil #2.*

5.3 Model Pile Load Tests

The total, shaft, and base bearing capacities of single model piles were determined performing a series of model pile load tests using three model piles with different diameters (i.e., pile diameter 38.30mm, 31.75mm, and 19.25 mm) in two sandy soils (i.e., *Soil #1: Unimin 7030 sand, and Soil #2: Industrial sand) under both saturated and unsaturated conditions. Details of the testing program are presented in Chapter 4 (see section 4.2). Figure 5.5 provides details of different model pile load tests performed in this study.*

5.3.1 Model pile load tests under saturated condition

Several single model pile load tests were performed under saturated condition. Tensiometers installed in the testing facility during the pile installation procedures and loading indicated zero suction reading which suggests the soil is in a state of saturated condition. The applied load versus displacement results from the model pile load tests (i.e., total, shaft, and base bearing capacity measurement tests) under saturated condition are presented in Figure 5.6 through Figure 5.11.

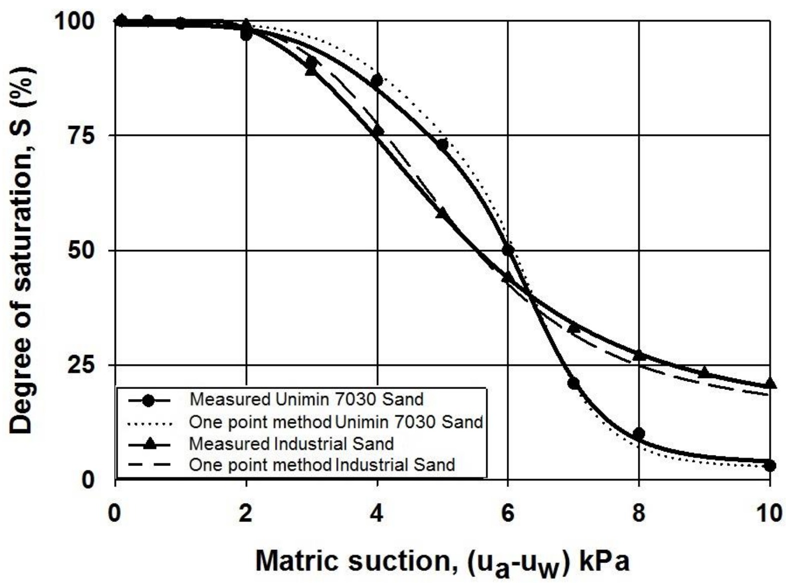


Figure 5.4 *Comparison between the measured and predicted SWCC for Soil #1 and for Soil #2.*

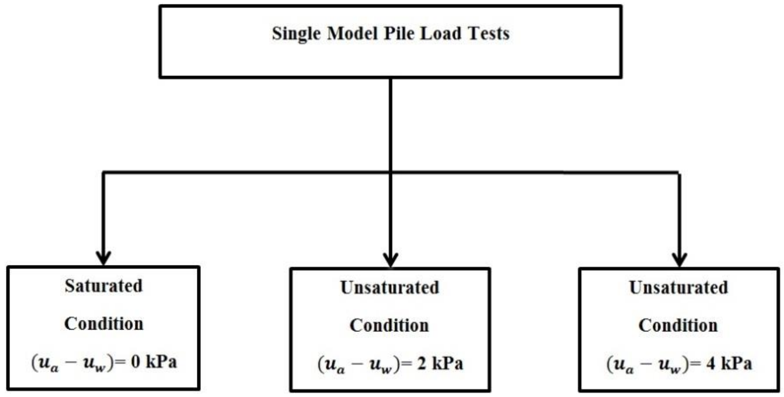


Figure 5.5 *Single model pile load tests details*

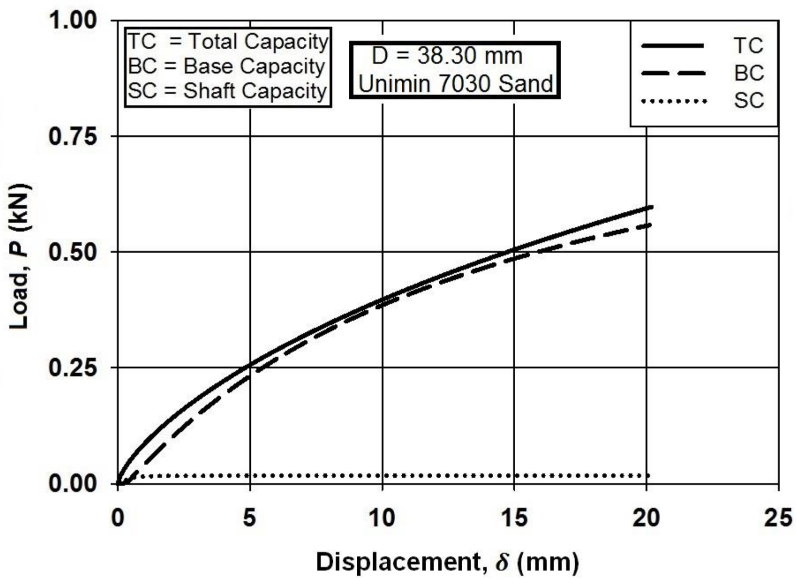


Figure 5.6 *Measured load-displacement of single model pile D38.30 mm in Unimin 7030 sand under saturated condition.*

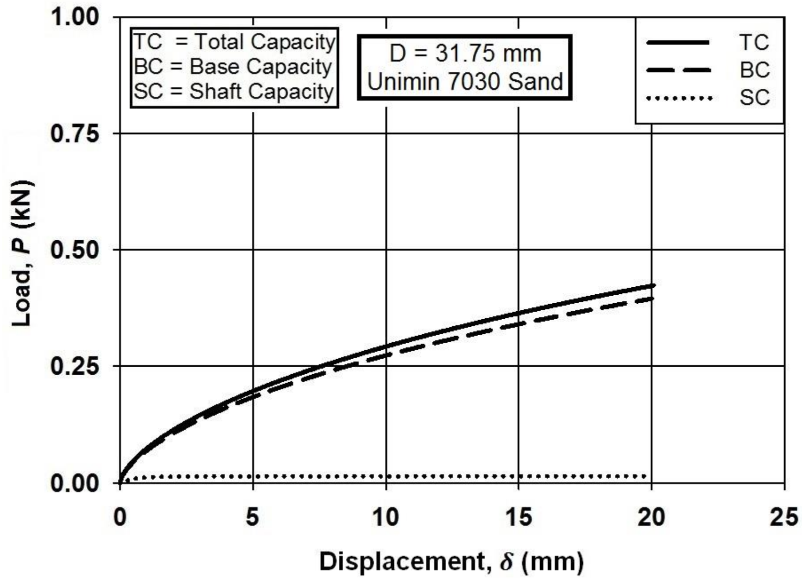


Figure 5.7 *Measured load-displacement of single model pile D31.75 mm in Unimin 7030 sand under saturated condition.*

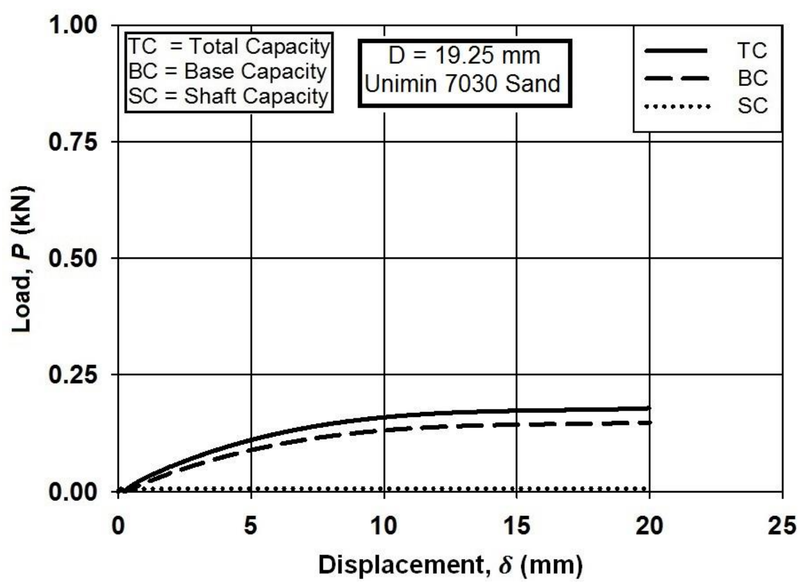


Figure 5.8 *Measured load-displacement of single model pile D19.25 mm in Unimin 7030 sand under saturated condition.*

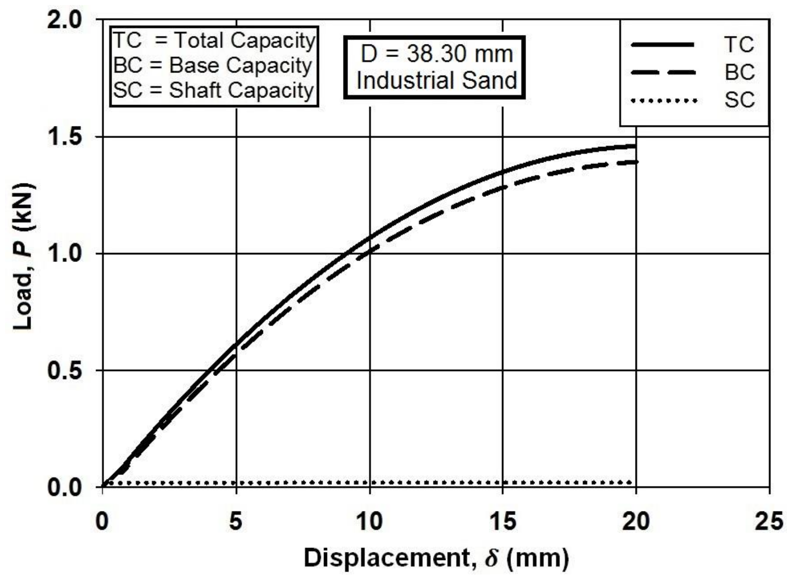


Figure 5.9 *Measured load-displacement of single model pile D38.30 mm in Industrial sand under saturated condition.*

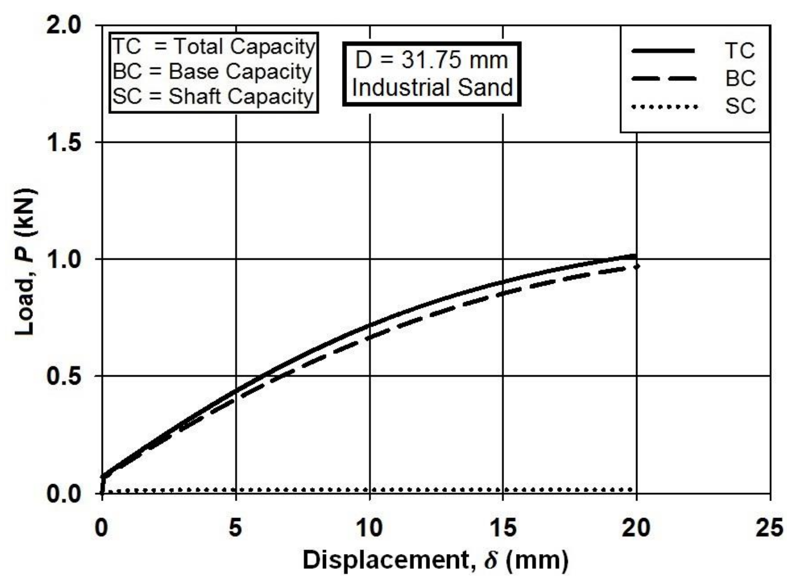


Figure 5.10 *Measured load-displacement of single model pile D31.75 mm in Industrial sand under saturated condition.*

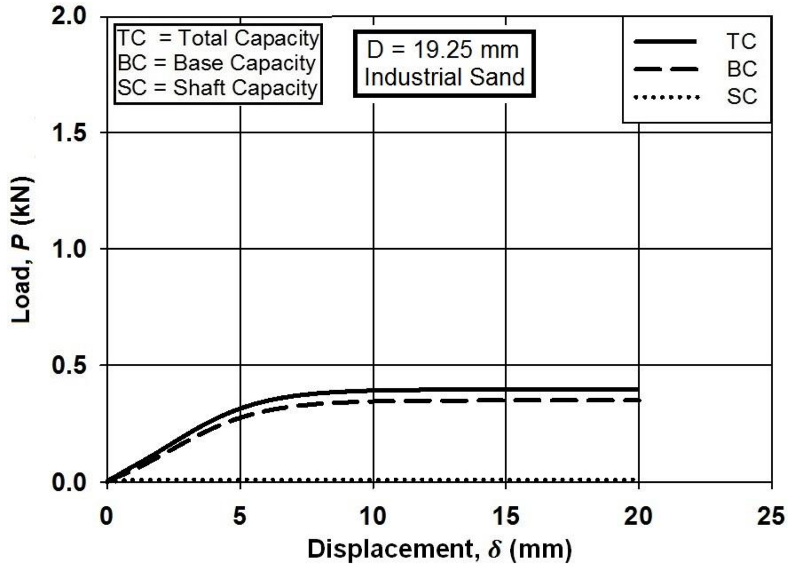


Figure 5.11 *Measured load-displacement of single model pile D19.25 mm in Industrial sand under saturated condition.*

As shown in Figure 5.6 through Figure 5.11, the contribution of the pile base capacity towards the pile total capacity is dominant in comparison with the pile shaft capacity (i.e., approximately 95% or higher of the pile total capacity arises from the pile base capacity). The measured results from the present study are in good agreement with those reported in the literature (Miura 1983, Yasufuku and Hyde 1995, Ohno and Sawada 1999, Manandhar and Yasufuku 2012).

5.3.2 Model pile load tests under unsaturated condition

The soil in the bearing capacity test tank was assured to be in state of unsaturated condition following the test procedures discussed in Chapter 4 (see section 4.2). The variation of the matric suction in the stress bulb zone (i.e., a zone with a depth of 1.5 times of the pile diameter below the pile toe) was measured using the Tensiometers installed in the test tank. A series of model pile load tests were performed under the different stages of unsaturated conditions (i.e., average matric suction values 2 and 4 kPa). Results of these tests are presented in the following sections. The reported results were the average values of the several test results (i.e., minimum of three sets). The variation of each set of test results (i.e., base, shaft, or total capacity) was less than ± 5 to 7 %.

5.3.2.1 Model pile load tests under average matric suction of 2 kPa

An average matric suction value of 2 kPa was achieved within the stress bulb by lowering the water table approximately 450 mm below the soil surface using the hanging column method (i.e., plexi-glass water container). The variation of the matric suction in the test tank with depth is presented in Figure 5.12.

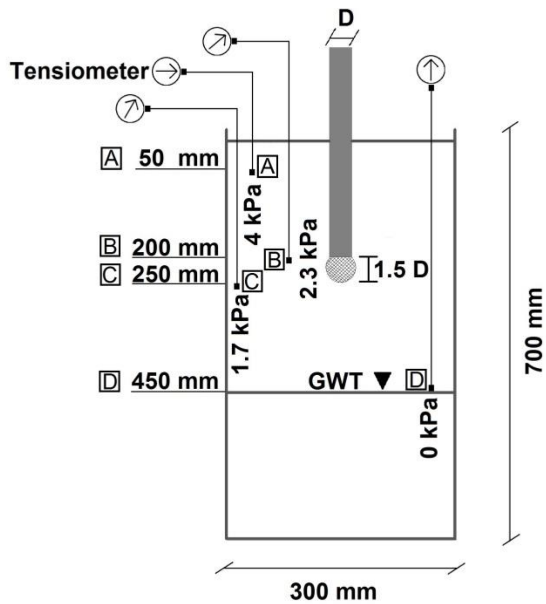


Figure 5.12 *Variation of matric suction with respect to soil depth in the test tank under unsaturated condition 2 kPa matric suction.*

The axial load versus displacement results from the model pile load tests (i.e., total, shaft, and base bearing capacity measurement tests) under unsaturated condition with average matric suction of 2 kPa are presented in Figure 5.13 through Figure 5.18.

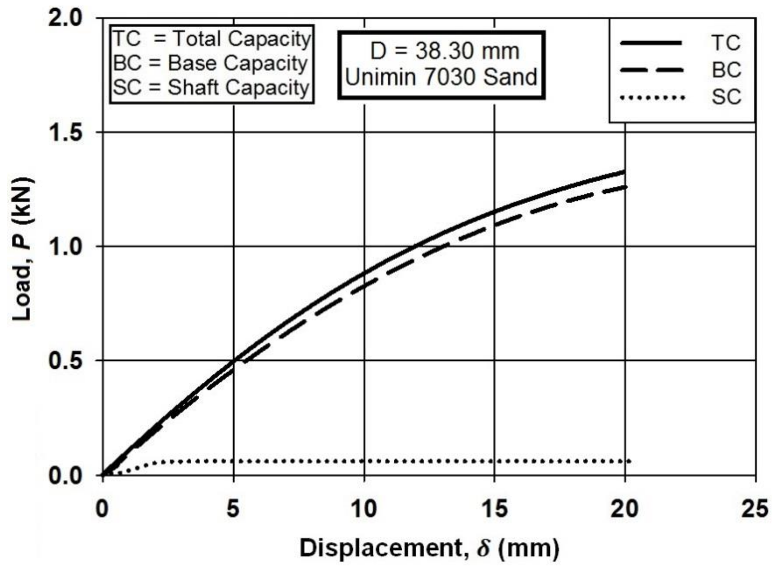


Figure 5.13 *Measured load-displacement of single model pile D38.30 mm in Unimin 7030 sand under unsaturated condition 2 kPa matric suction.*

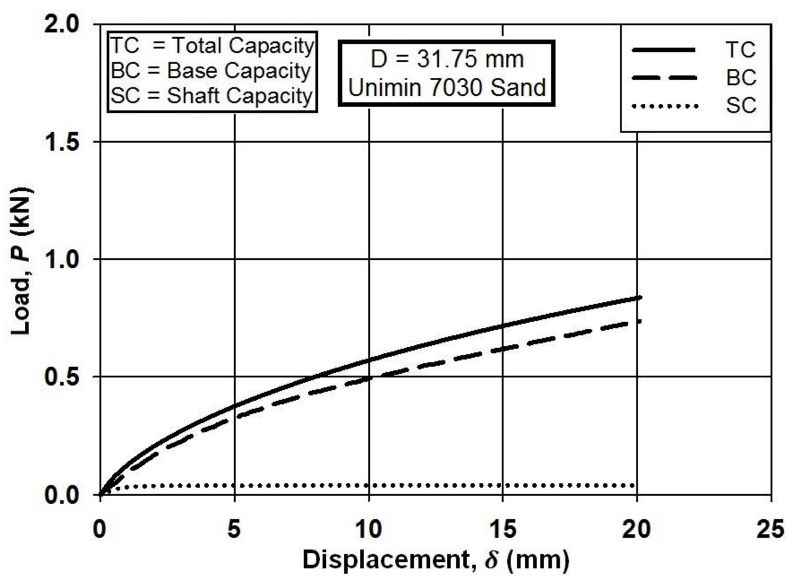


Figure 5.14 *Measured load-displacement of single model pile D31.75 mm in Unimin 7030 sand under unsaturated condition 2 kPa matric suction.*

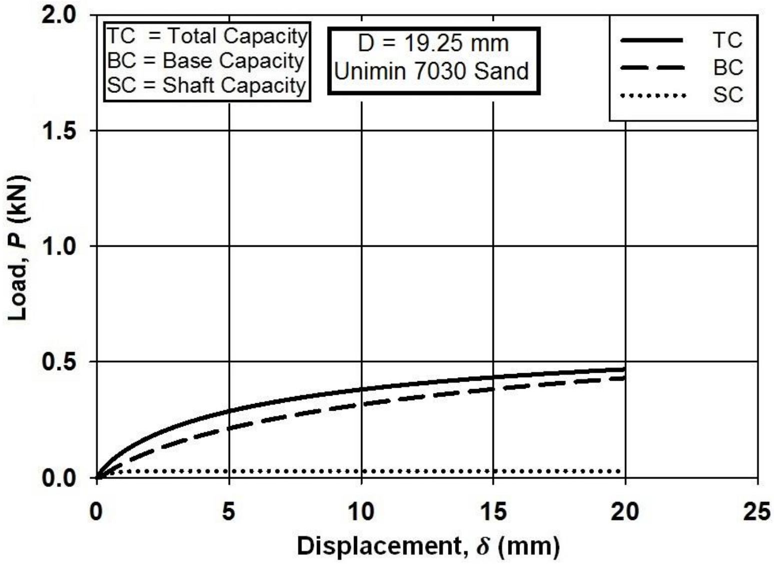


Figure 5.15 *Measured load-displacement of single model pile D19.25 mm in Unimin 7030 sand under unsaturated condition 2 kPa matric suction.*

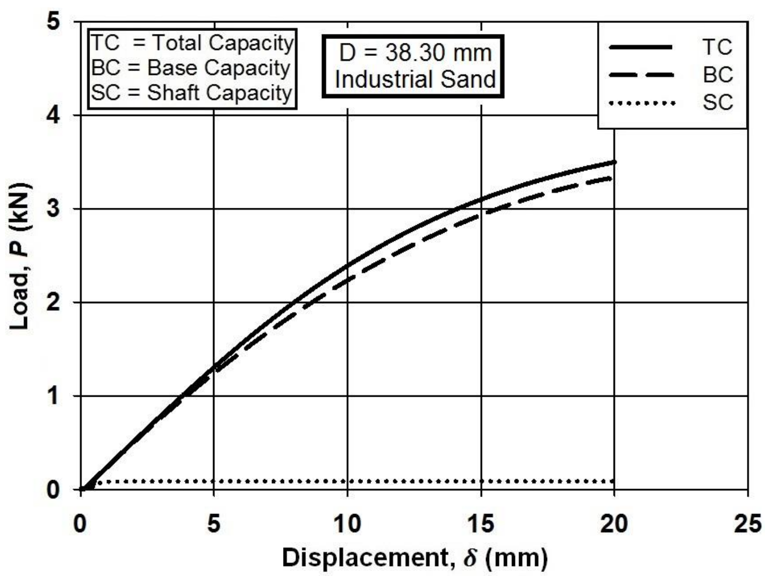


Figure 5.16 *Measured load-displacement of single model pile D38.30 mm in Industrial sand under unsaturated condition 2 kPa matric suction.*

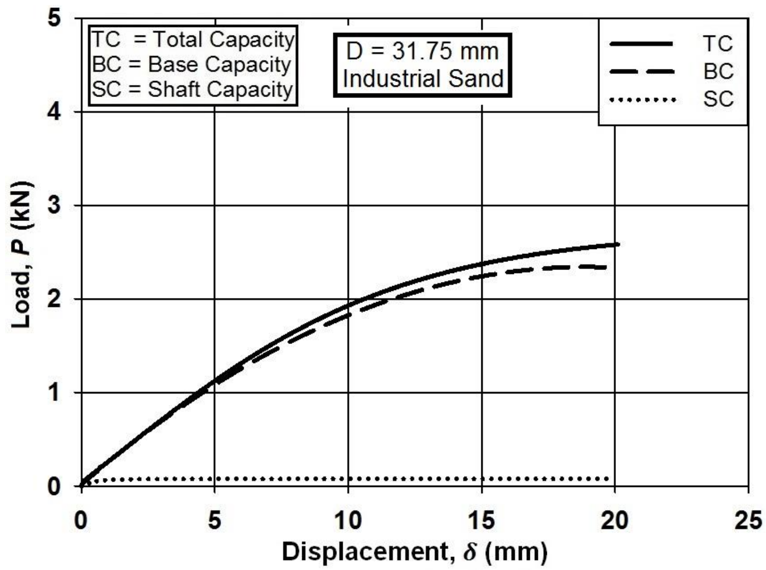


Figure 5.17 *Measured load-displacement of single model pile D31.75 mm in Industrial sand under unsaturated condition 2 kPa matric suction.*

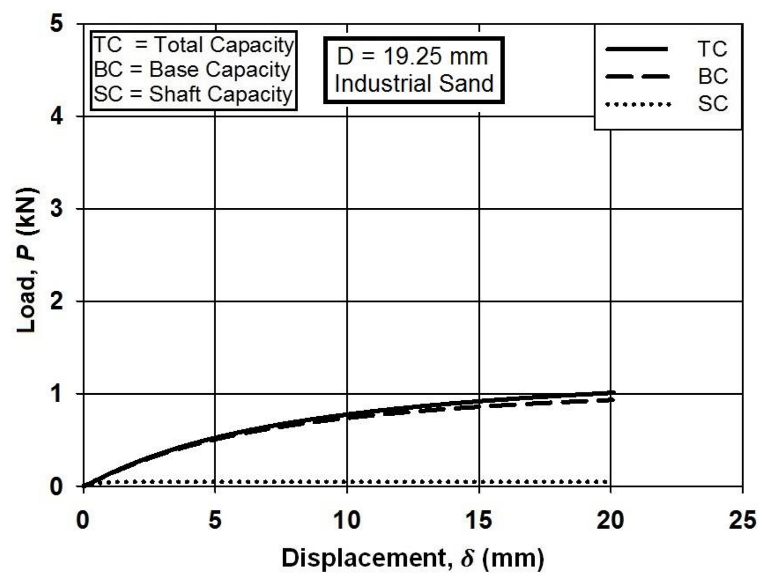


Figure 5.18 *Measured load-displacement of single model pile D19.25 mm in Industrial sand under unsaturated condition 2 kPa matric suction.*

5.3.2.2 Model pile load tests under unsaturated condition average matric suction of 4 kPa

An average matric suction value of 4 kPa was achieved within the stress bulb by lowering the water table approximately 650 mm below the soil surface by sliding down the plexi-glass water container (i.e., hanging water column). The variation of matric suction in the soil profile of the test tank is presented in Figure.5.19. The applied load versus displacement results from the model pile load tests (i.e., total, shaft, and base bearing capacity measurement tests) under unsaturated condition with average matric suction of 4 kPa are presented in Figure 5.20 through Figure 5.25.

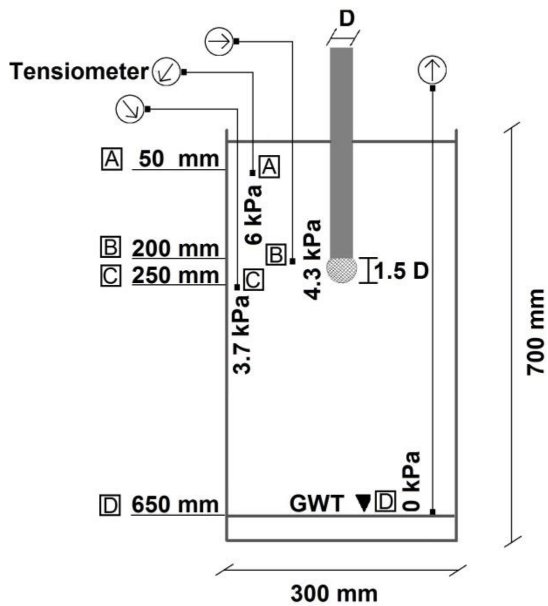


Figure 5.19 *Variation of matric suction with respect to soil depth in the test tank under unsaturated condition 4 kPa matric suction.*

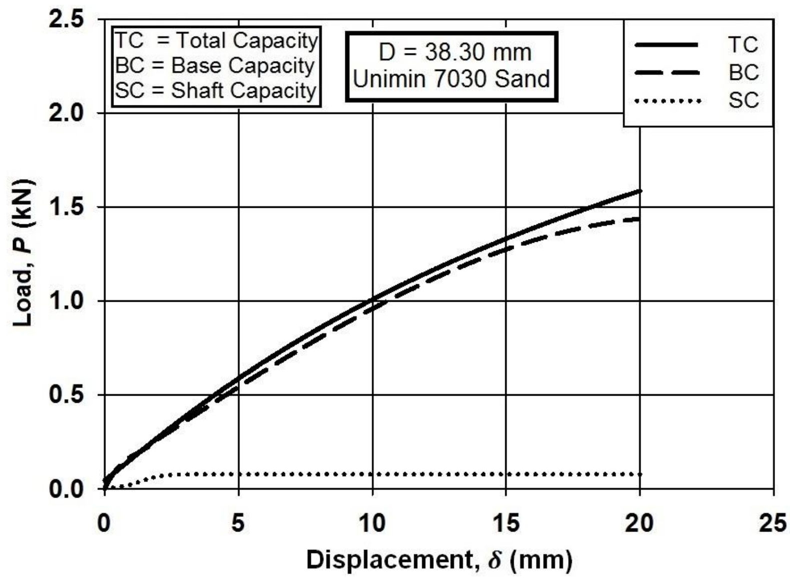


Figure 5.20 *Measured load-displacement of single model pile D38.30 mm in Unimin 7030 sand under unsaturated condition 4 kPa matric suction.*

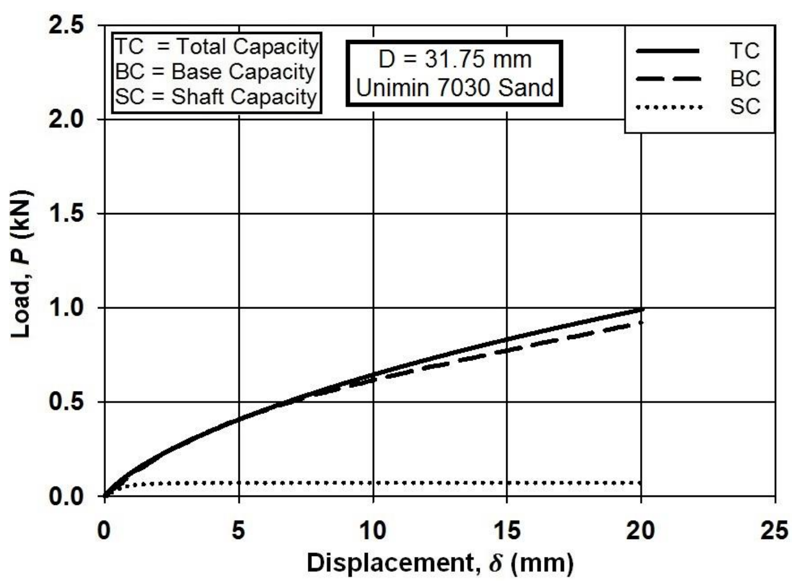


Figure 5.21 *Measured load-displacement of single model pile D31.75 mm in Unimin 7030 sand under unsaturated condition 4 kPa matric suction.*

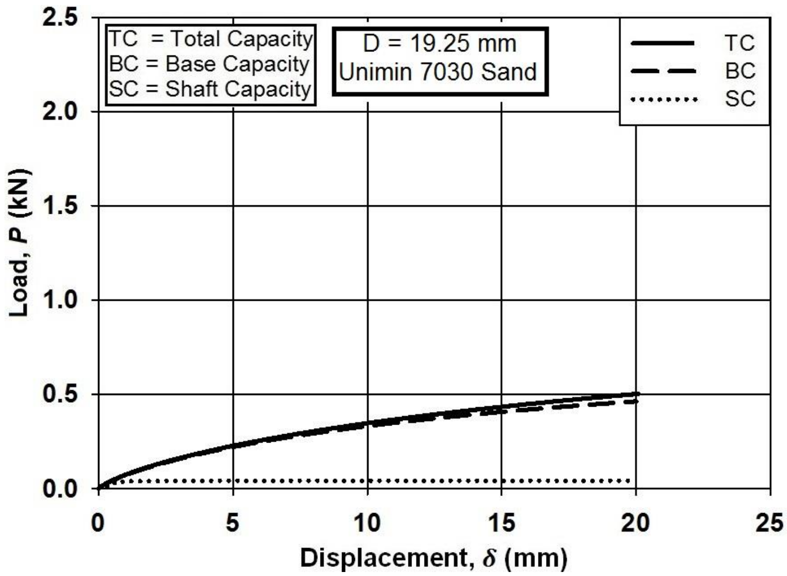


Figure 5.22 *Measured load-displacement of single model pile D19.25 mm in Unimin 7030 sand under unsaturated condition 4 kPa matric suction.*

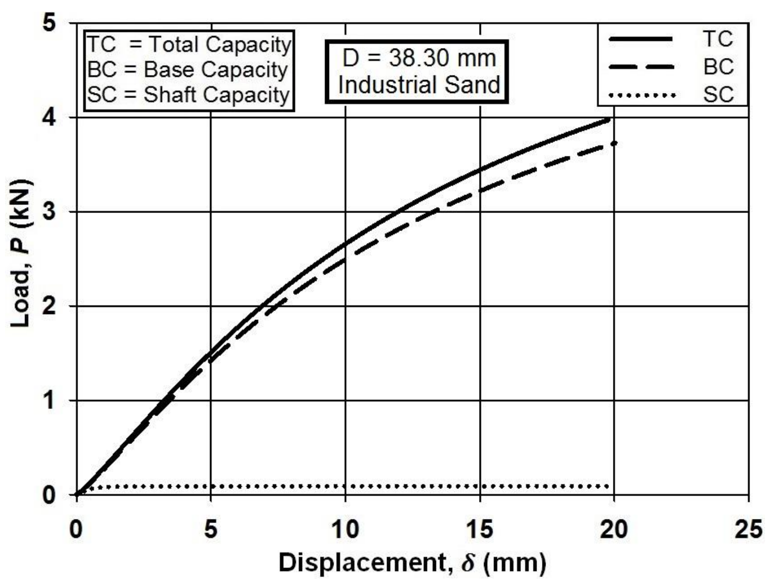


Figure 5.23 *Measured load-displacement of single model pile D38.30 mm in Industrial sand under unsaturated condition 4 kPa matric suction.*

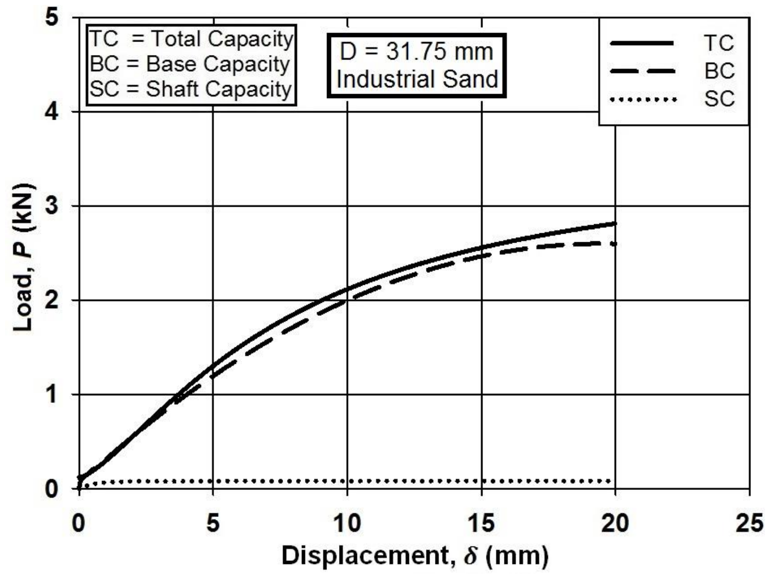


Figure 5.24 *Measured load-displacement of single model pile D31.75 mm in Industrial sand under unsaturated condition 4 kPa matric suction.*

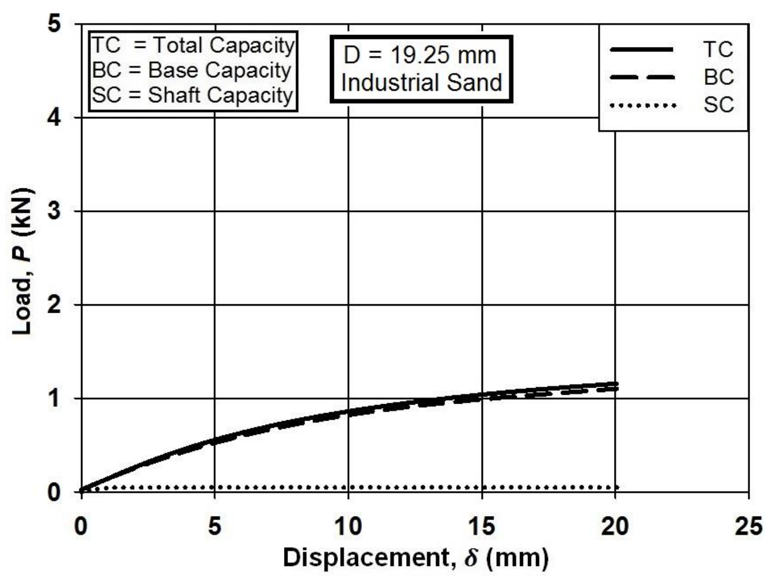


Figure 5.25 *Measured load-displacement of single model pile D19.25 mm in Industrial sand under unsaturated condition 4 kPa matric suction.*

The results of the single model pile load tests show a significant increase in the bearing capacity (i.e., total, shaft, or base) under unsaturated conditions in comparison with *saturated condition. As the matric suction value increased from 0 kPa (i.e., saturated condition) to 4 kPa (i.e., unsaturated condition), the bearing capacities increased significantly. The model pile bearing capacity under unsaturated conditions is approximately 2 to 2.5 times higher in comparison to saturated condition (Figure 5.26). These test results highlight the need for developing methods to estimate, interpret, and predict the p-δ behavior of pile foundations in unsaturated soils.*

5.4 Summary

In this chapter, the results of various tests conducted to determine the soil properties and the single model pile p-δ behavior under both saturated and unsaturated soils are presented. The experimental studies show that bearing capacities of the single model piles in two selected soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) to be approximately 2 to 2.5 times higher than the single model pile bearing capacity under saturated condition. These studies suggest the need for proposing reliable methods for estimating or predicting the bearing capacity of pile foundations under unsaturated conditions.

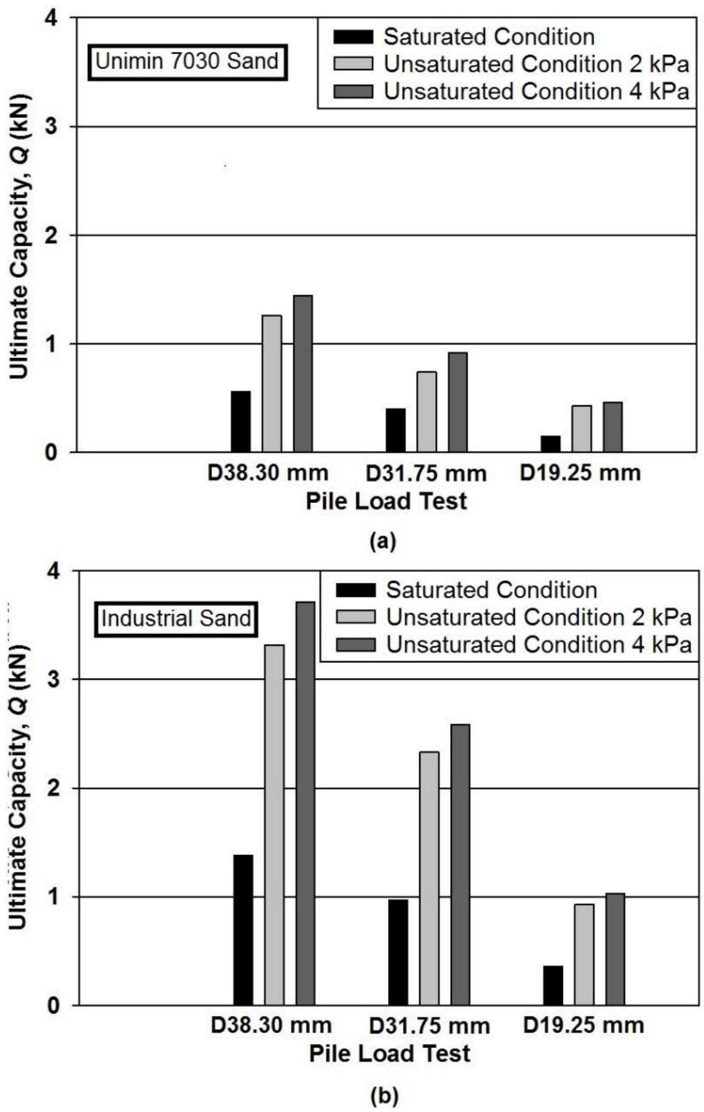


Figure 5.26 *Comparison between the results of saturated and unsaturated (i.e., 2 and 4 kPa) pile load test for (a) Soil #1 and for (b) Soil #2.*

CHAPTER 6

INTERPRETATION OF THE MODEL PILE BEARING CAPACITY TEST RESULTS USING A SEMI-EMPIRICAL TECHNIQUE

6.1 Introduction

Single model pile load tests were performed in a specially designed tank in the laboratory following the procedures detailed in Chapter 4 to study the load-displacement behavior (i.e., p-δ) of single model piles with different diameters (i.e., 38.30, 31.75, and 19.25 mm) under both saturated and unsaturated conditions for two different sandy soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) in this thesis. The load-displacement behavior (i.e., p-δ) of the single model piles are presented in Chapter 5.

The ultimate bearing capacity of a single pile arises from the combination of the shaft and base resistance. The pile shaft capacity is predicted using the modifed β method (Vanapalli and Taylan 2012) which is an effective stress approach method suitbale for interpretation of pile shaft capacity in sandy soils.

In this chapter, three conventional methods proposed by various investigators; namely Terzaghi (1943), Hansen (1970), and Janbu (1976) are modified for interpretaion and prediction of the single pile base bearing capcity in unsaturated soils. In addition, semi-empirical equations are proposed for predicting the pile base capacity the relationship between the variation of matric suction and pile base bearing capacity. When the matric suction value is set to zero, the modified methods are consistent with the conventional methods that are used for soils in saturated condition. The proposed semi-empirical methods are consistent with the techniques for predicting the shear strength and the bearing capacity of unsaturated soils proposed by Vanapalli et al. (1996) and Vanapalli and Mohamed (2007) respectively. The pile base bearing capacity is predicted extending *the proposed modified equations using the saturated shear strength parameters (i.e., c' and ϕ') and the soil water characteristic curve (SWCC). Comprehensive comparisons are also provided between the measured and the predicted bearing capacity of the single model piles in this chapter.*

6.2 Model Pile Base Capacity Test Results Interpretation

The single model pile base capacity was measured in the testing program using a hollow sleeve following the test procedures detailed in Chapter 4. The results of the model pile base capacity test are presented in Chapter 5 and interpretation of these results is provided in this chapter.

6.2.1 Model pile failure criteria

The required settlement to reach the ultimate pile bearing capacity is based on practice guidelines. Lee and Salgado (2000) stated that the size effects in calibration of the model pile load test are insignificant for small deformations; due to this reason, the loading of model piles should be limited to a settlement level of interest in practice (i.e., 10% of pile diameter). In the present study, the model pile was, however, penetrated to a depth beyond 10% pile diameter (i.e., 20 mm) to study the p-δ behavior. However, the settlement level equal to 10% of the pile diameter is assumed to be the failure criteria in the present study and the corresponding applied load is calculated from the p-δ curves. (see Figure 6.1).

Lee et al. (2003) studied 36 calibration chamber load tests on closed- and open- ended model piles in sand. In addition, full-scale field load tests in sand were performed to provide a size effect factor that can be used for interpreting the model pile load test results and calculating the equivalent field pile base resistance. Based on these studies the equivalent field pile base resistance can be calculated by dividing the model pile base resistance by the size effect factor. Lee et al. (2003) recommended using the size effect factor equal to 0.4 for close-ended piles. The corrected pile base resistance values, using *the size effect factor (i.e., model pile base resistance divided by the size effect factor equal to 0.4 for closed end model piles) are used in the analysis section.*

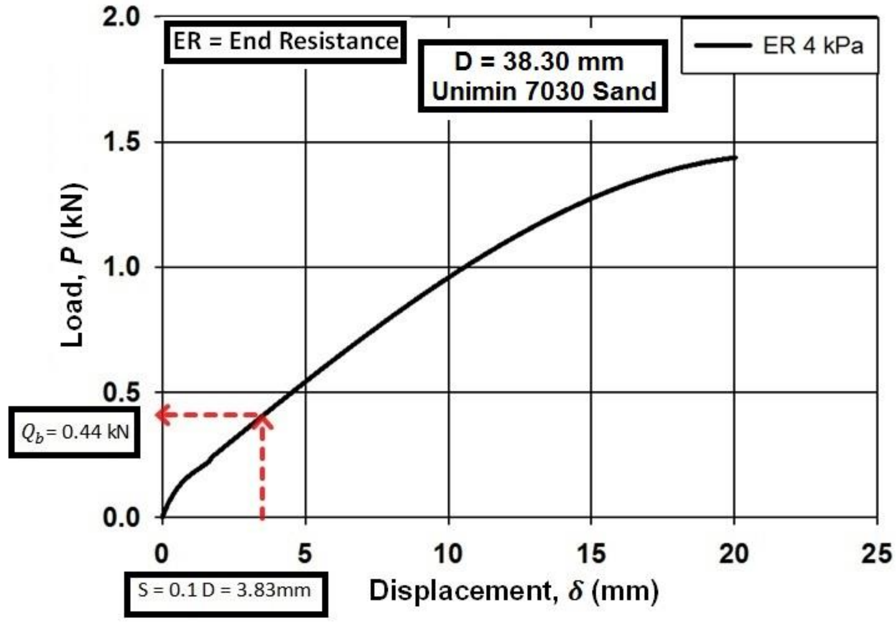


Figure 6.1 *Failure criteria provided by Lee and Salgado (2000)*

6.2.2 Estimation of single model pile base capacity by the modified methods

The pile base capacity can be estimated using the conventional methods proposed for prediction of shallow foundation carrying capacity (Gui and Muhunthan, 2006). Terzaghi (1943) proposed a method for estimating the shallow foundation bearing capacity extending the failure mechanism suggested by Prandtl (1921) as given below:

$$Q_b = A_b(c'N_c s_c + \bar{q}N_q + \tfrac{1}{2}\gamma B N_\gamma s_\gamma) \quad \textbf{(6.1)}$$

where, A_b = pile base area, c' = soil cohesion, $\bar{q}$ *= surcharge load, γ = total unit weight of soil, B = pile diameter, N_c, N_q, and N_γ = bearing capacity factor, s_c and s_γ = shape factor.*

The failure mechanism indicated the stress distribution below the foundation base, as three zones of soil volume (i.e., zone I, II, and III). As shown in Figure 6.2, the soil *volume in zone (I) moves downward and the other two zones displace outward and upward. The failure surfaces end at the pile tip level.*

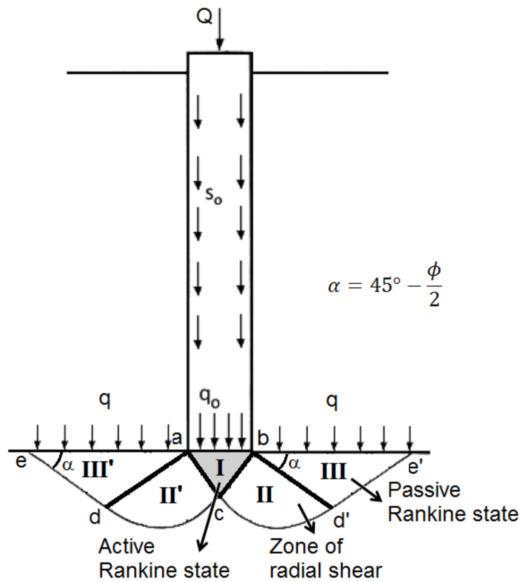


Figure 6.2 *Bearing capacity failure pattern around the pile tip assumed by Terzaghi (1943)*

Some research studies were reported in the literature for estimating the bearing capacity of shallow foundations in unsaturated soils (Oloo 1994, Vanapalli and Mohamed 2007, Vanapalli et al. 2007, Vanapalli and Oh 2010, Oh and Vanapalli 2013). The Terzaghi (1943) method is modified for estimating the bearing capacity of single pile foundation. Details of the modified form were presented in Chapter 3. The general form of the proposed modified method is given below:

$$Q_b = A_b([c' + (u_a - u_w)_b(1 - S^{\psi_{BC}})\tan\phi' + (u_a - u_w)_{AVR}S^{\psi_{BC}}\tan\phi']N_c s_c + \sigma'_{vb}N_q + \frac{1}{2}B\gamma N_\gamma s_\gamma \quad \textbf{(6.2)}$$

where, Q_b = ultimate pile base capacity, c' = effective cohesion, $(u_a - u_w)_b$ = air-entry value, S = degree of saturation, ψ_{BC} = bearing capacity parameter, ϕ' = internal angle of friction, $(u_a - u_w)_{AVR}$ = the average matric suction at depth of the stress bulb (i.e., 1.5 pile diameter below the pile base), σ'_{vb} = the vertical effective stress at the depth of pile base, *B = pile base diameter, γ = total unit weight of soil, N_c, N_q, and N_γ = bearing capacity factor, s_c and s_γ = shape factor, A_b = pile base area.*

Hansen (1970) proposed a method for estimation of the bearing capacity of shallow foundations extending the Meyerhof (1951) work. This method is capable of predicting the bearing capacity based on D/B values (i.e., embedment depth to foundation diameter). Due to this reason, this method can be used for both shallow and deep foundations. Janbu (1976) developed a method for prediction of pile base bearing capacity by introducing the ψ angle (i.e., ψ varies from 75º to 105º) in failure pattern around the pile base (see Figure 6.3).

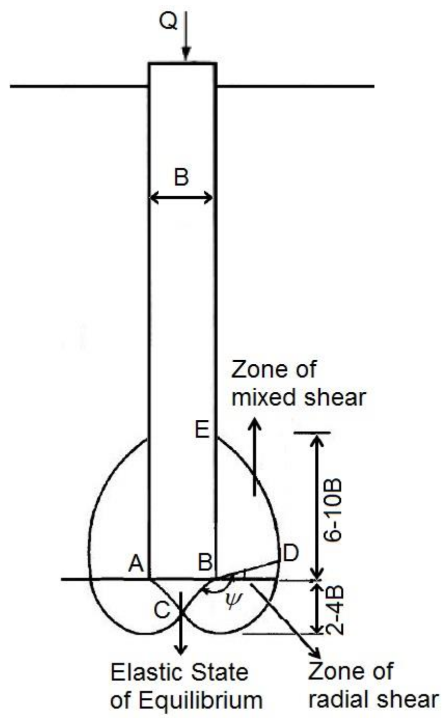


Figure 6.3 *Bearing capacity failure pattern around the pile tip assumed by Janbu (1976)*

The Hansen (1970) and Janbu (1976) methods were modified extending the same philosophy used for predicting the shear strength and bearing capacity of unsaturated soils proposed by Vanapalli et al. (1996) and Vanapalli and Mohamed (2007), as shown below:

$$Q_b = A_b([c' + (u_a - u_w)_b(1 - S^{\psi_{BC}})\tan\phi' + (u_a - u_w)_{AVR}S^{\psi_{BC}}\tan\phi']N'_c d_C + \sigma'_{vb}N'_q d_q + \frac{1}{2}B\gamma N_\gamma) \quad \textbf{(6.3)}$$

where, N'_c, N'_q, and N_γ = bearing capacity factor, d_c and d_q = depth factor.

The required bearing capacity, shape and depth factors used in Eq. 6.2 and Eq. 6.3 are listed in Appendix. The bearing capacity parameter, ψ_{BC}, can be defined by a relationship proposed by Vanapalli and Mohamaed (2007) with plasticity index, I_p. In case of coarse-grained soils (i.e., two selected soils for the experimental program) the plasticity index is equal to zero; due to this reason, the bearing capacity parameter becomes equal to unity.

The average matric suction at the depth of stress bulb, $(u_a - u_w)_{AVR}$, and the air-entry value, $(u_a - u_w)_b$, are two important parameters required to interpret and predict the pile base capacity. In following sections, procedures for determining these two parameters are detailed.

6.2.3 Estimation of matric suction variation

The matric suction variation and the average matric suction value in the stress bulb below the model pile base should be determined to predict the pile bearing capacity under unsaturated conditions. In order to determine the average matric suction within the stress bulb (i.e., 1.5 pile diameter), Vanapalli and Mohamed (2007) and Oh and Vanapalli (2011) proposed a procedure. The average matric suction is defined as the matric suction value at the centroid of distribution profile (see Figure 6.4). The matric suction values, $(u_a - u_w)$, in the testing program were measured by placing commercial Tensiomters at different depth in the test tank. The matric suction readings from Tensiometers exhibited hydrostatic variation assumption. Due to this reason, hydrostatic variation of matric suction with respect to depth can be assumed for performing the bearing capacity analysis.

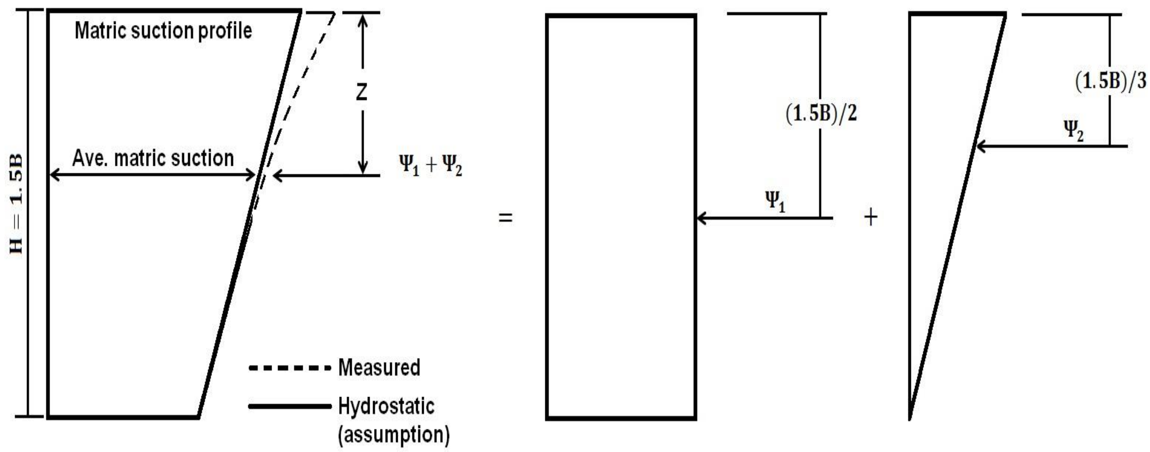


Figure 6.4 *Procedure for estimation of average matric suction below the pile base (Oh and Vanapalli 2011)*

6.2.4 Estimation of air-entry values of the soils from the *SWCC*

Estimating the air-entry value (AEV) of the selected soils from the SWCC is required for predicting the single model pile base capacity using the proposed modified methods. The AEV of each soil can be estimated as the intersection between the tangent of the SWCC in the boundary effect zone and tangent of the SWCC from the transition zone as detailed in Vanapalli et al. (1999). The estimated AEV for the Soil #1 Unimin 7030 sand and Soil #2 Industrial sand are presented as 3.0 and 2.4 kPa respectively (see Figure 6.5).

6.2.5 Effect of dilatancy

Houlsby (1991) studied the influence of dilatancy on mechanical behavior of sands. Based on these studies, the angle of friction is dependent on the angle of dilation. The observed angle of friction arises from a combination of the angle of dilatancy, ψ, and the angle of friction, ϕ', at constant volume. Steensen-Bach et al. (1987) suggested using 10 to 15% higher value of friction angle to provide a better comparison between the measured and estimated bearing capacity of sands. In the present study, the internal friction angle is used as 10% higher than the measured value from direct shear test (i.e., for Unimin 7030 sand $\phi' = 35.3° + 3.53° \approx 39°$, and for Industrial sand $\phi' = 40.3° + 4.03° \approx 44°$). Similar assumptions were used by Oh and Vanapalli (2012) and Vanapalli and Mohamed (2012).

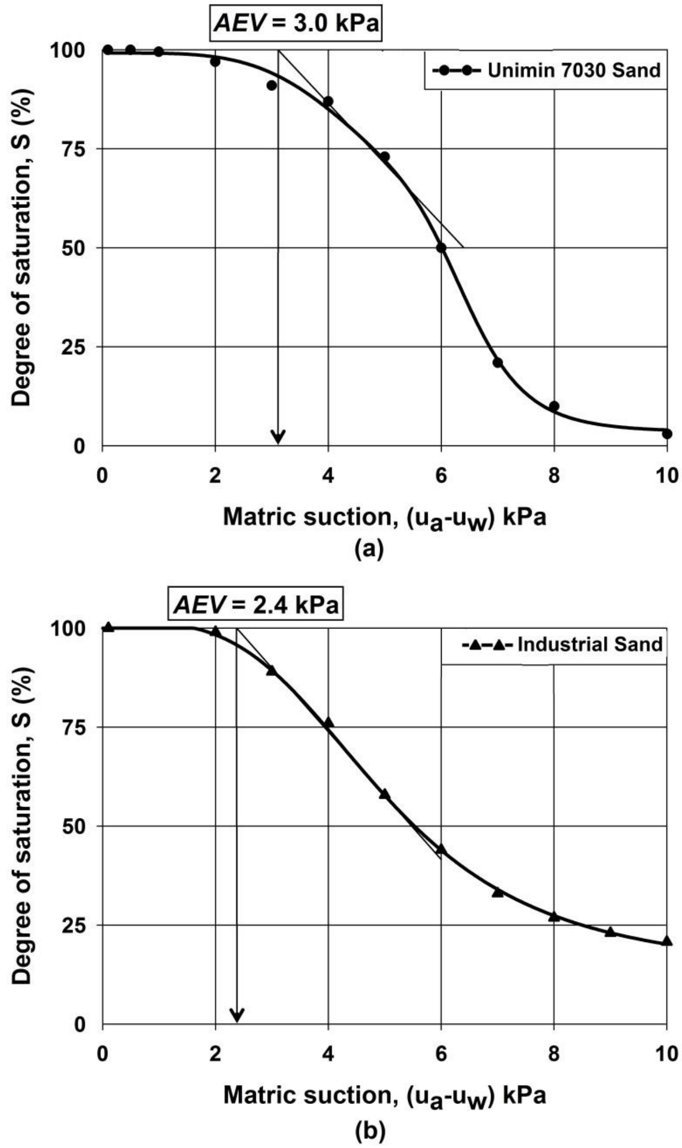


Figure 6.5 *AEV from SWCC of (a) Unimin 7030 sand and (b) Industrial sand*

6.3 Model Pile Base Capacity Test Results Analysis

In this section, the model pile base bearing capacity results are analyzed using the three proposed modified methods (i.e., Terzaghi 1943, Hansen 1970, and Janbu 1976) are compared with the model pile test results. Comparison between the measured and predicted results is presented through the Table 6.1 to 6.6 and Figure 6.6 to 6.11.

Table 6.1 *Pile base resistance of Pile D38.30 mm in Unimin 7030 sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	**-**	**kPa**	**kN**	**kN**	**kN**	**kN**
0	*1.00*	*3.0*	*0.535*	*0.512*	*0.524*	*0.460*
2	*0.98*	*3.0*	*0.893*	*0.857*	*0.877*	*0.910*
4	*0.84*	*3.0*	*1.085*	*1.034*	*1.057*	*1.100*

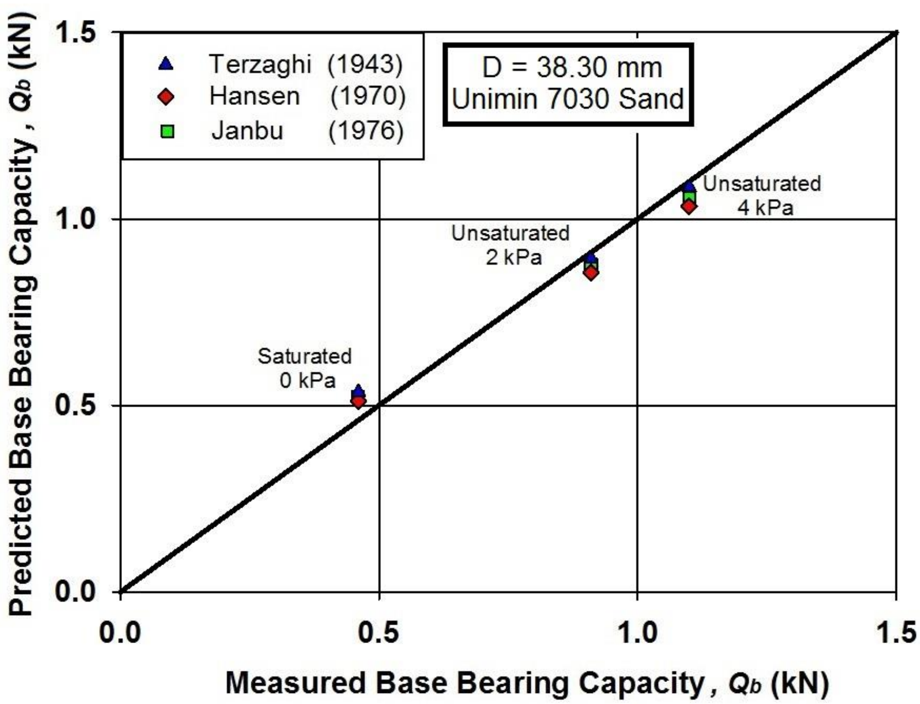


Figure 6.6 *Comparison between the measured and predicted model pile base capacity for Pile D38.30 mm in Unimin 7030 sand*

Table 6.2 *Pile base resistance of Pile D31.75 mm in Unimin 7030 sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	**-**	**kPa**	**kN**	**kN**	**kN**	**kN**
0	*1.00*	*3.0*	*0.366*	*0.353*	*0.362*	*0.350*
2	*0.98*	*3.0*	*0.611*	*0.587*	*0.603*	*0.580*
4	*0.84*	*3.0*	*0.743*	*0.709*	*0.725*	*0.750*

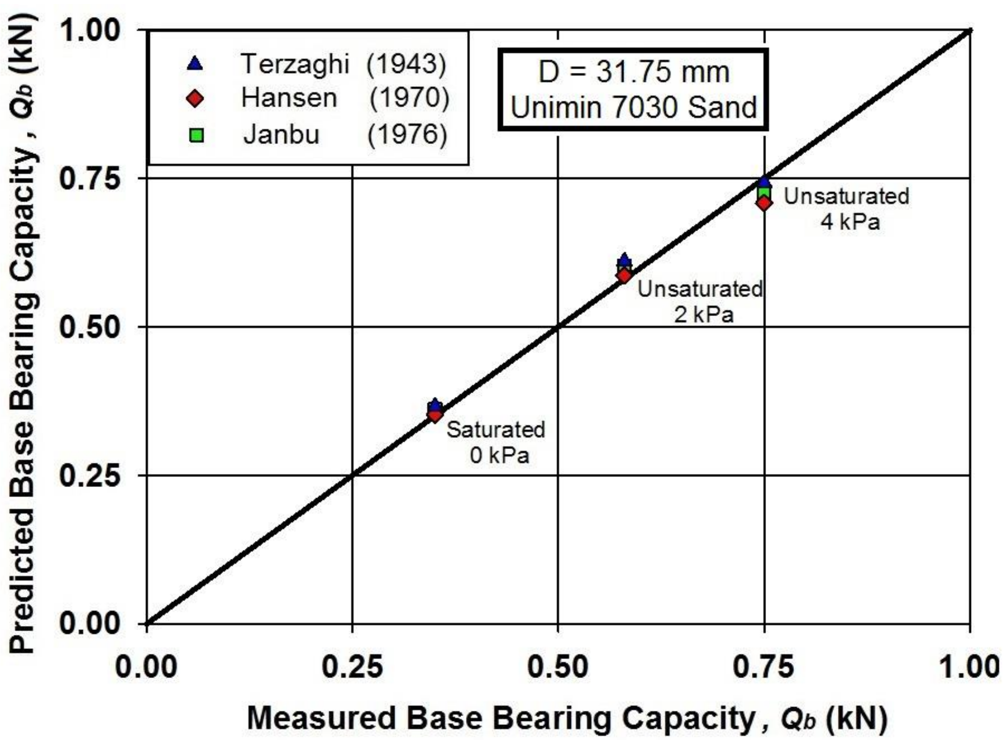


Figure 6.7 *Comparison between the measured and predicted model pile base capacity for Pile D31.75 mm in Unimin 7030 sand*

Table 6.3 *Pile base resistance of Pile D19.25 mm in Unimin 7030 sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	**-**	**kPa**	**kN**	**kN**	**kN**	**kN**
0	*1.00*	*3.0*	*0.133*	*0.130*	*0.133*	*0.120*
2	*0.98*	*3.0*	*0.223*	*0.214*	*0.219*	*0.270*
4	*0.84*	*3.0*	*0.271*	*0.259*	*0.265*	*0.290*

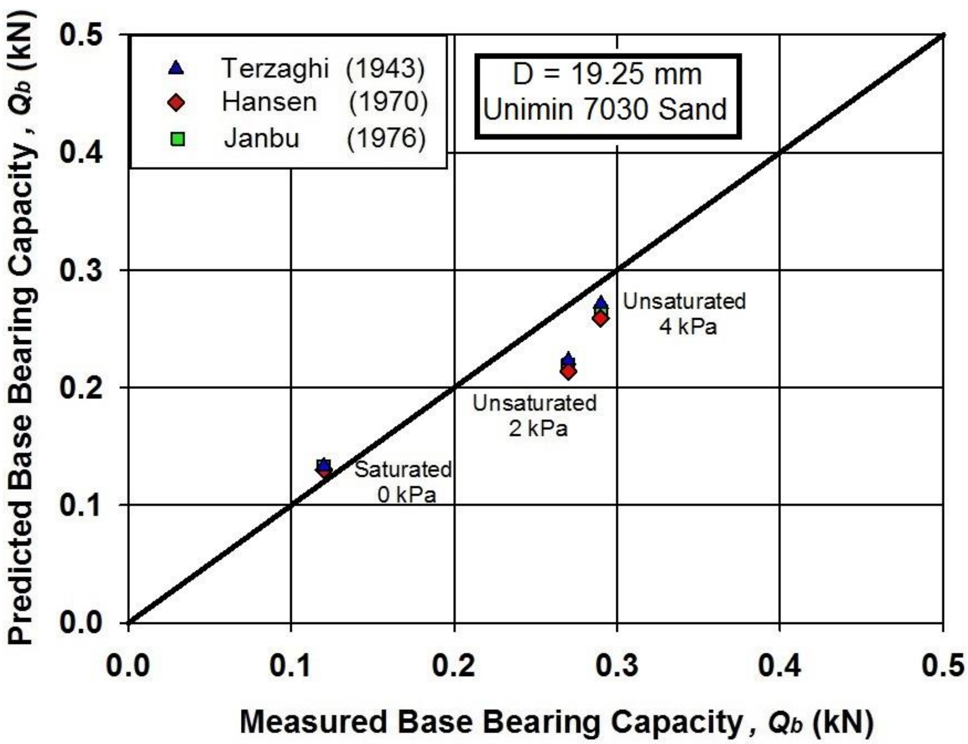


Figure 6.8 *Comparison between the measured and predicted model pile base capacity for Pile D19.25 mm in Unimin 7030 sand*

Table 6.4 *Pile base resistance of Pile D38.30 mm in Industrial sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	**-**	**kPa**	**kN**	**kN**	**kN**	**kN**
0	*1.00*	*2.4*	*1.402*	*1.381*	*1.265*	*1.130*
2	*0.98*	*2.4*	*2.347*	*2.317*	*2.128*	*2.430*
4	*0.74*	*2.4*	*2.763*	*2.717*	*2.491*	*2.810*

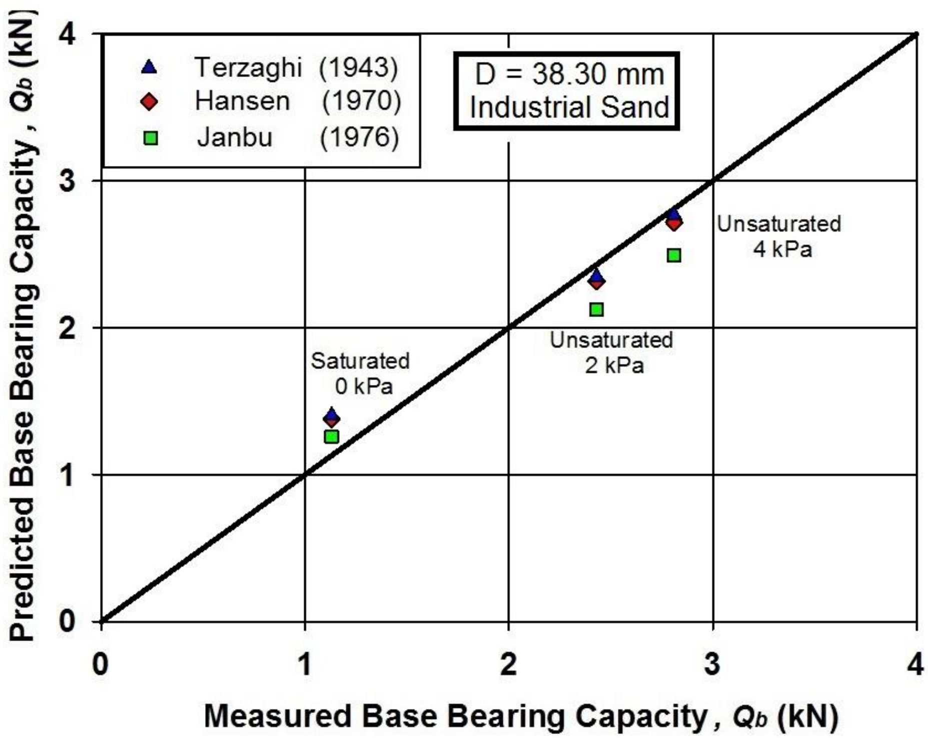


Figure 6.9 *Comparison between the measured and predicted model pile base capacity for Pile D38.30 mm in Industrial sand*

Table 6.5 *Pile base resistance of Pile D31.75 mm in Industrial sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	**-**	**kPa**	**kN**	**kN**	**kN**	**kN**
0	*1.00*	*2.4*	*0.957*	*0.951*	*0.872*	*0.850*
2	*0.98*	*2.4*	*1.602*	*1.585*	*1.466*	*1.820*
4	*0.74*	*2.4*	*1.888*	*1.888*	*1.745*	*2.050*

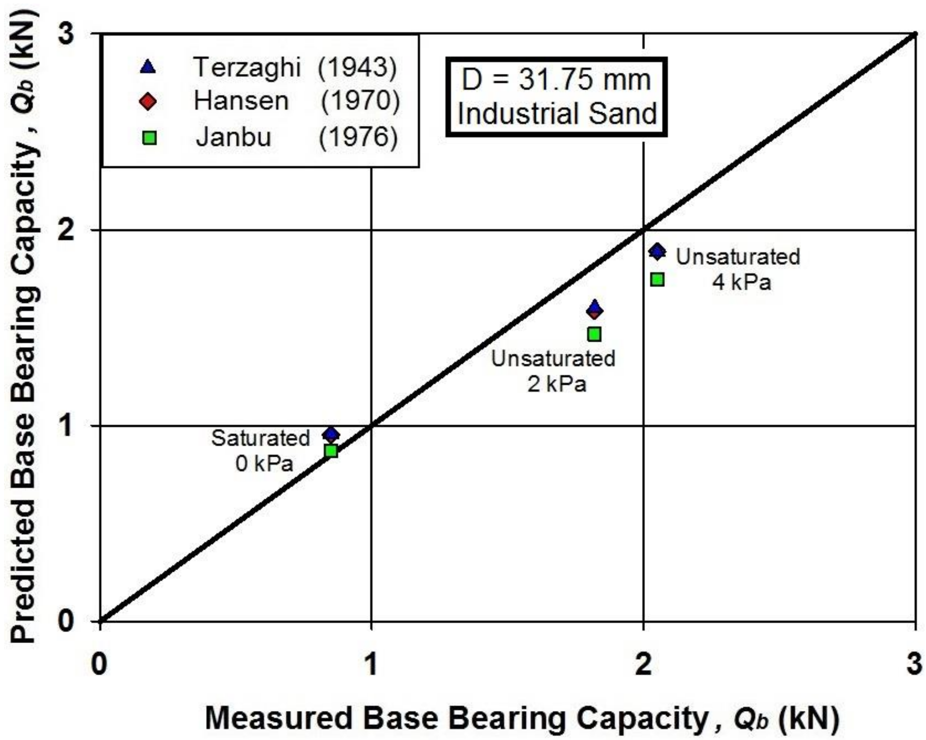


Figure 6.10 *Comparison between the measured and predicted model pile base capacity for Pile D31.75 mm in Industrial sand*

Table 6.6 *Pile base resistance of Pile D19.25 mm in Industrial sand*

$(u_a - u_w)_{AVR}$	S	$(u_a - u_w)_b$	Q_b Cal. Terzaghi (1943)	Q_b Cal. Hansen (1970)	Q_b Cal. Janbu (1976)	Q_b Meas.
kPa	-	kPa	kN	kN	kN	kN
0	*1.00*	*2.4*	*0.348*	*0.349*	*0.319*	*0.300*
2	*0.98*	*2.4*	*0.582*	*0.576*	*0.535*	*0.620*
4	*0.74*	*2.4*	*0.687*	*0.677*	*0.630*	*0.700*

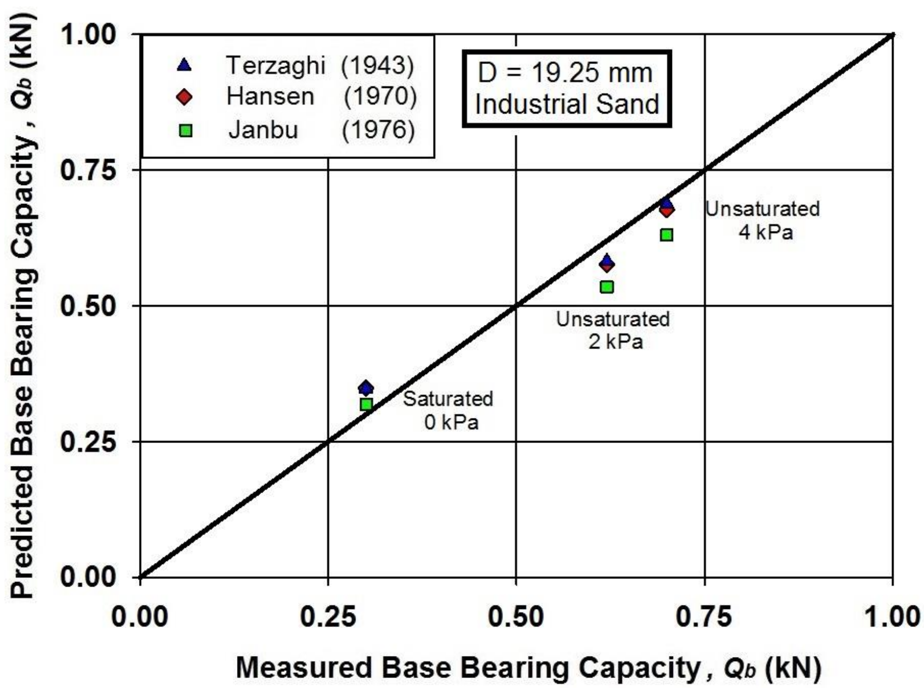


Figure 6.11 *Comparison between the measured and predicted model pile base capacity for Pile D19.25 mm in Industrial sand*

The contribution of pile base capacity in sandy type of soils is greater than 95% of the total pile capacity (see section 5.3) and the matric suction contribution is greater than 55% (i.e., 50-60 %) of the pile base capacity for two matric suction values (i.e., 2 and 4 kPa) used in the experimental program. Results from Figure 6.6 to 6.11 show the ultimate *pile base capacity in unsaturated sands is significantly higher (i.e., approximately 2 to 2.5 times) in comparison with the saturated condition. The test results demonstrate that the conventional methods used in engineering practice are conservative when they are extended for soils that are in a state of unsaturated condition. In other words, these results highlight the need for introducing modification to the conventional methods to take into account the influence of matric suction.*

Results of the analysis on two selected soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) using the modified semi-empirical methods (Terzaghi 1943, Hansen 1970, and Janbu 1976) are summarized in Table 6.1 through 6.6. The results of the predicted pile base capacity using the proposed modified methods are in agreement with experimental results (i.e., approximately less than 10% difference between the measured and predicted values).

The Terzaghi modified method predicts closest values to measured ones in comparison with the other two modified methods. As a result, the modified Terzaghi's methods is *chosen to study the governing relationship between the SWCC and the model pile base bearing capacity and the variation of shear strength of unsaturated sand with respect to matric suction. Figure 6.12 and 6.13 show the relationship between the SWCC and the variation of model pile base bearing capacity, and the shear strength behavior of unsaturated sands. As expected, the model pile base bearing capacity changes with respect to matric suction in the same manner as the shear strength of unsaturated sands. As it can be seen in Figures 6.12 and 6.13, in the boundary effect zone the shear strength and model pile base bearing capacity increase linearly (i.e., the ϕ^b is equal to ϕ'). In the transition zone the slope of raise in shear strength and model pile base bearing capacity tends to decrease (i.e., $\phi^b < \phi'$). The reduction in pile base bearing capacity for matric suction values close to residual zone of unsaturation can be justified by the reduction that takes place in shear strength of unsaturated sands (i.e., $d(S^\kappa)/d(u_a - u_w)$ is negative) in this zone.*

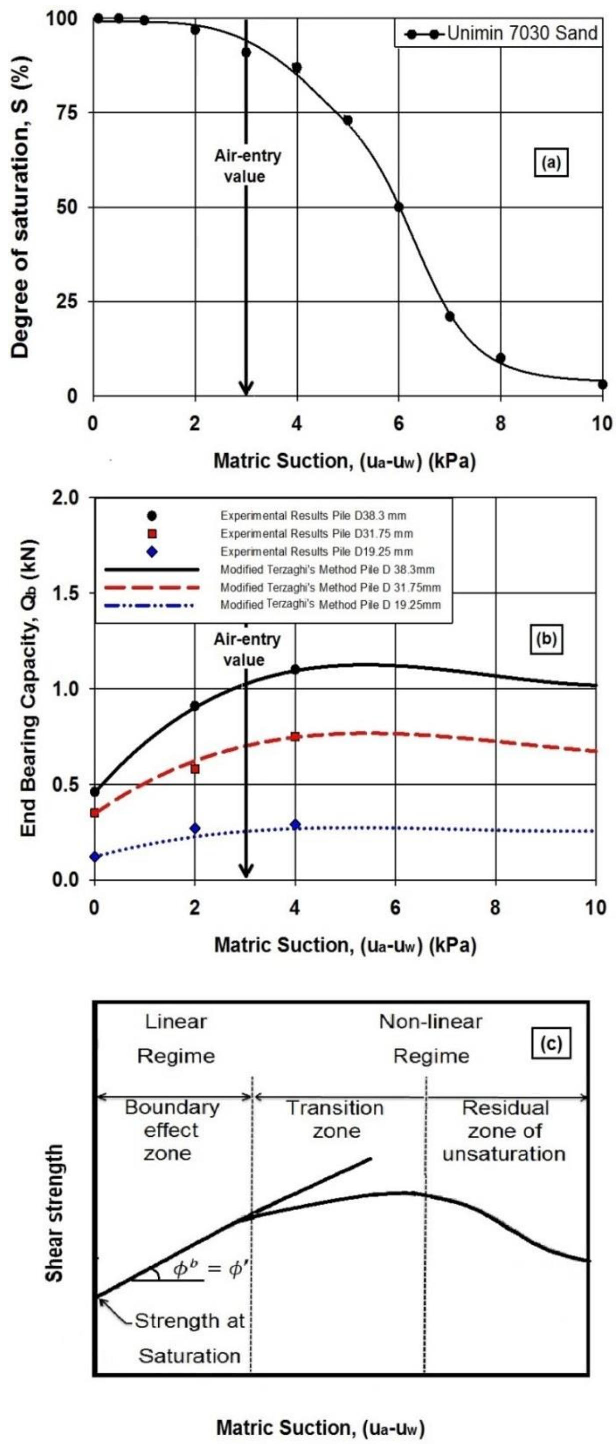


Figure 6.12 *(a) Unimin 7030 sand SWCC (b) Comparison between the measured and* predicted model pile end bearing capacity using modified Terzaghi's method (c) shear *strength behavior of sandy soils with respect to matric suction (modified after Vanapalli, 2009)*

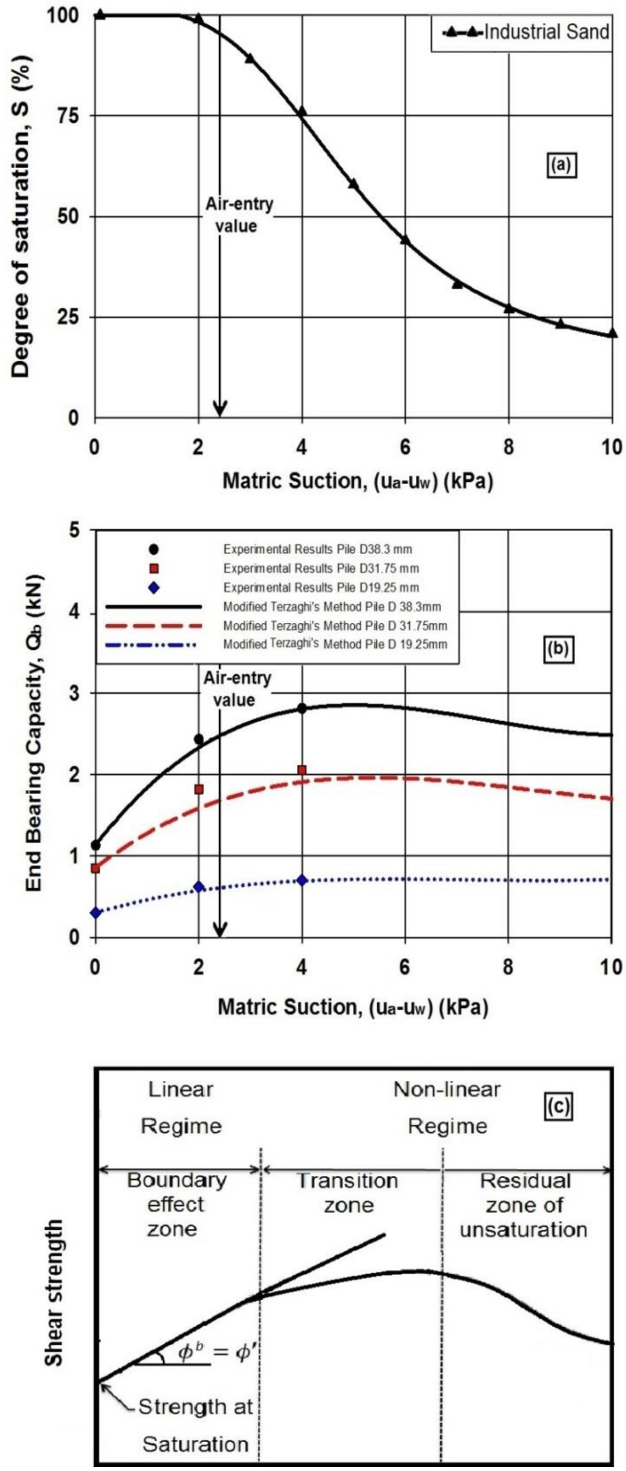


Figure 6.13 *(a) Industrial sand SWCC (b) Comparison between the measured and* predicted model pile end bearing capacity using modified Terzaghi's method (c) shear *strength behavior of sandy soils with respect to matric suction (modified after Vanapalli, 2009)*

6.4 Model Pile Shaft Capacity Test Results Interpretation

The single model pile shaft capacity was measured in the testing program using the pile base to prevent any contact between the pile toe and soil particles and consequently neglecting contribution of model pile base bearing capacity, following the test procedures detailed in Chapter 4. The results of the model pile shaft capacity tests are presented in Chapter 5 and interpretation of these results is provided in this chapter.

6.4.1 Model pile failure criteria

The pile failure criteria used in engineering practice for estimation of pile shaft capacity is typically assumed as the settlement equal to 10% of the pile diameter (Yasufuku, 1997).

6.4.2 Estimation of single model pile base capacity by the modified β method

The model pile shaft capacity under both saturated and unsaturated conditions can be predicted using the modified β method. The conventional β method is applicable for the single model piles loaded at slow rate which can be assumed as drained condition. Recently, Vanapalli et al. (2010) proposed modified β method extending the conventional β method along with lines of predicting the shear strength of unsaturated sandy type of soils. Some research studies extended the modified β method for prediction of the shaft capacity of single piles and bearing capacity of soil nails (Vanapalli et al. 2010, Vanapalli and Taylan 2011, Vanapalli and Taylan 2012, Gurpersaud et al. 2013). Details of the modified β method are presented in Chapter 3 (see section 3.3.1). The general equation of this modified method is summarized:

$$Q_{f(us)} = Q_f + Q_{(u_a - u_w)}$$

$$Q_{f(us)} = [\beta \sigma'_v + (u_a - u_w)(S^{\kappa})(tan\delta')]A_s \tag{6.3}$$

where, β = bearing capacity coefficient, σ'_v = vertical effective stress along the pile length, $(u_a - u_w)$ = matric suction, S = degree of saturation, κ = fitting parameter used to provide *a proper fit between the measured shear strength and estimated values, δ' = effective angle of interface friction along the soil/pile, A_s = pile shaft area.*

In case of sandy type of soils, β value can be estimated using various recommendations using the internal friction angle of sand, ϕ'. For soils with a friction angle ϕ', in range of 28° to 37°, Meyerhof (1976) proposed the β values of 0.44 to 1.2. Also, API (1984) recommends the β value should be computed using the correlation of $\beta = 0.7\tan(\phi'-5°)$.

6.5 Model Pile Shaft Capacity Test Results Analysis

In this section results of analysis on model pile shaft capacity using the modified β method are compared with the model pile test results. The β value used in the present study is assumed through the API (1984) recommended correlation (i.e., for Unimin 7030 sand β = 0.47, and for Industrial sand β = 0.57). Result of analysis and comparison between the predicted and measures model pile shaft capacity are summarized in Table 6.7 to 6.12 and Figure 6.14 to 6.19.

Table 6.7 *Pile shaft resistance of Pile D38.30 mm in Unimin 7030 sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.47	*1.00*	*1*	*0*	*0.0118*	*0.0172*
0.47	*0.98*	*1*	*2*	*0.0665*	*0.0606*
0.47	*0.84*	*1*	*4*	*0.0689*	*0.0768*

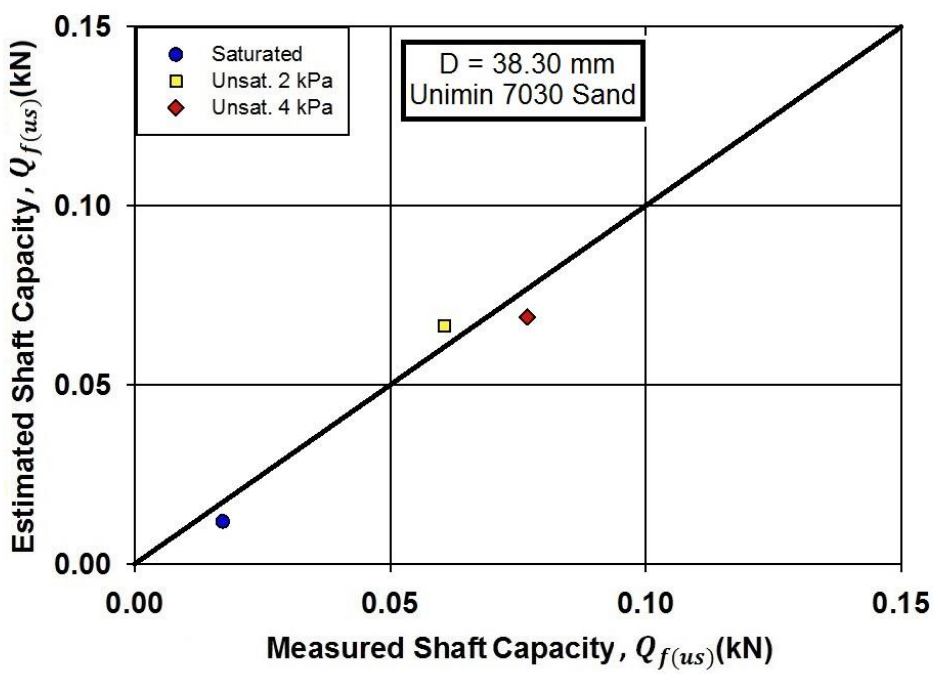


Figure 6.14 *Comparison between the measured and predicted model pile shaft capacity for Pile D38.30 mm in Unimin 7030 sand*

Table 6.8 *Pile shaft resistance of Pile D31.75 mm in Unimin 7030 sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.47	*1.00*	*1*	*0*	*0.0098*	*0.0139*
0.47	*0.98*	*1*	*2*	*0.0551*	*0.0380*
0.47	*0.84*	*1*	*4*	*0.0571*	*0.0680*

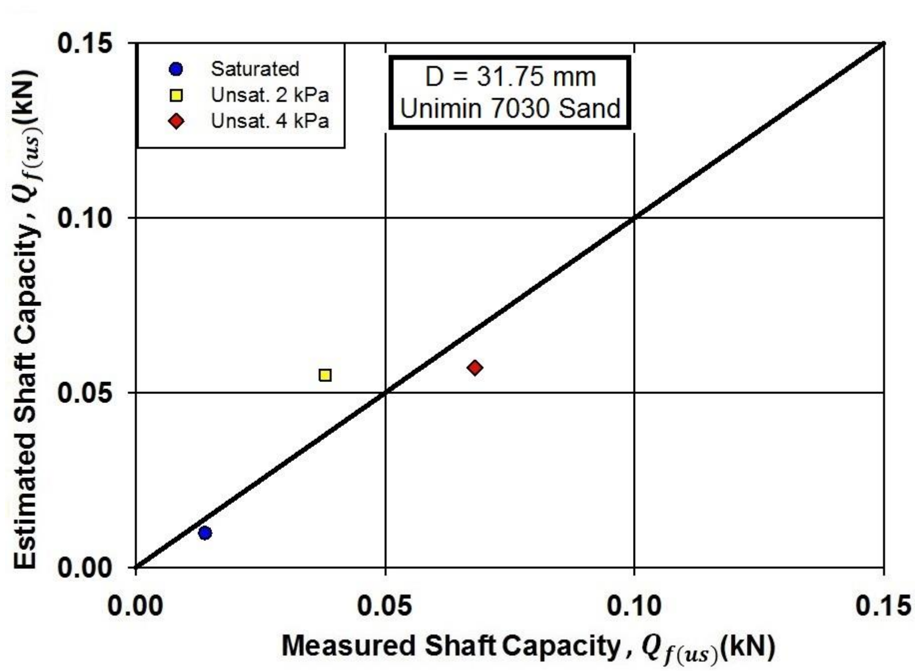


Figure 6.15 *Comparison between the measured and predicted model pile shaft capacity for Pile D31.75 mm in Unimin 7030 sand*

Table 6.9 *Pile shaft resistance of Pile D19.25 mm in Unimin 7030 sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.47	*1.00*	*1*	*0*	*0.0059*	*0.0059*
0.47	*0.98*	*1*	*2*	*0.0334*	*0.0274*
0.47	*0.84*	*1*	*4*	*0.0346*	*0.0389*

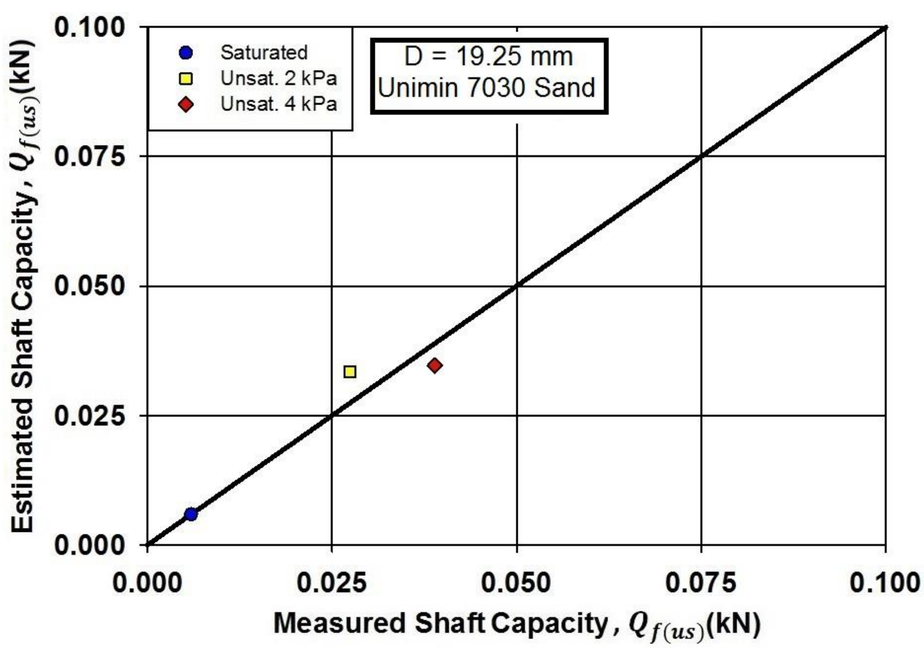


Figure 6.16 *Comparison between the measured and predicted model pile shaft capacity for Pile D19.25 mm in Unimin 7030 sand*

Table 6.10 *Pile shaft resistance of Pile D38.30 mm in Industrial sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.57	*1.00*	*1*	*0*	*0.0148*	*0.0182*
0.57	*0.98*	*1*	*2*	*0.0751*	*0.0863*
0.57	*0.74*	*1*	*4*	*0.0785*	*0.0884*

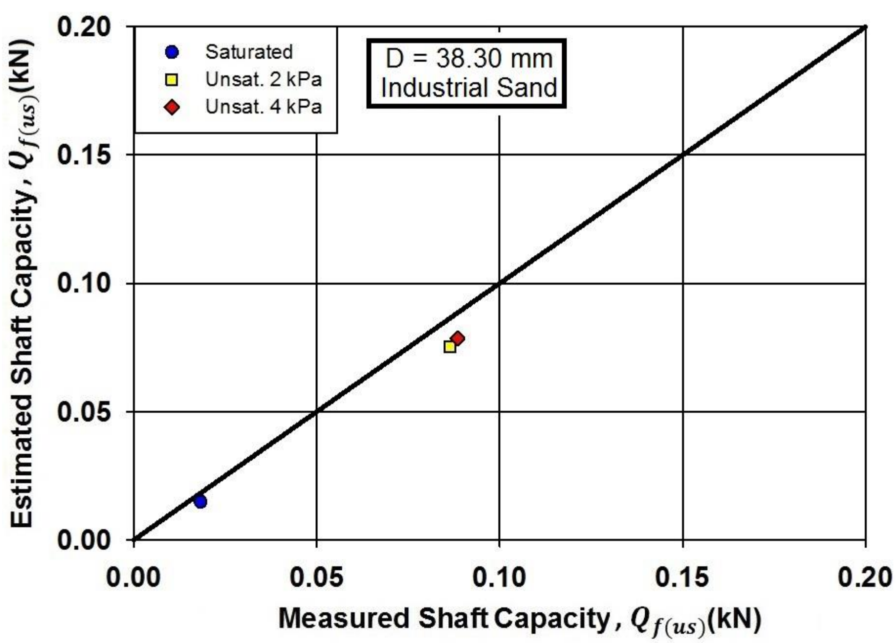


Figure 6.17 *Comparison between the measured and predicted model pile shaft capacity for Pile D38.30 mm in Industrial sand*

Table 6.11 *Pile shaft resistance of Pile D31.75 mm in Industrial sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.57	*1.00*	*1*	*0*	*0.0123*	*0.0145*
0.57	*0.98*	*1*	*2*	*0.0622*	*0.0754*
0.57	*0.74*	*1*	*4*	*0.0650*	*0.0779*

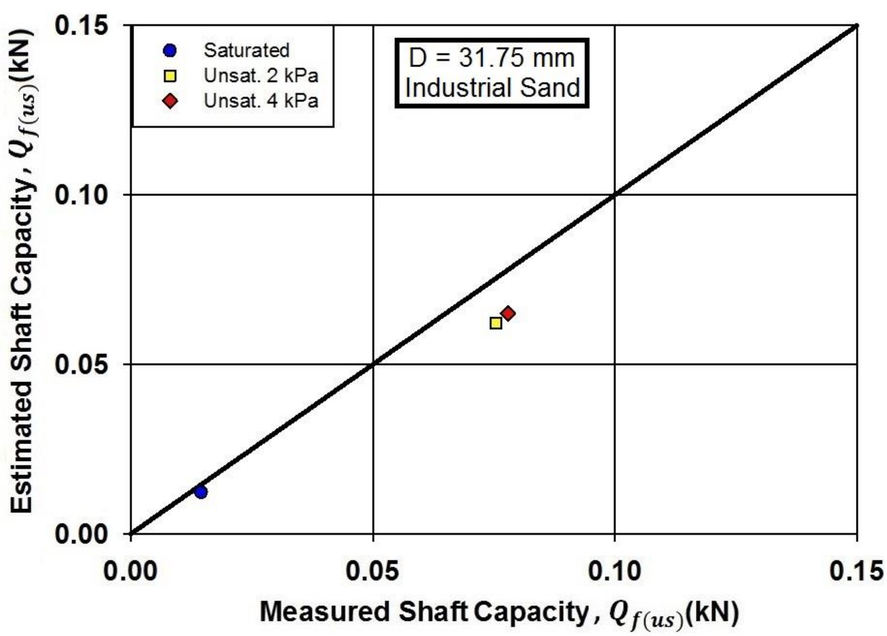


Figure 6.18 *Comparison between the measured and predicted model pile shaft capacity for Pile D31.75 mm in Industrial sand*

Table 6.12 *Pile shaft resistance of Pile D19.25 mm in Industrial sand*

β	S	κ	$(u_a - u_w)_{AVR}$	$Q_{f(us)}$ Cal.	$Q_{f(us)}$ Meas.
-	-	-	*kPa*	**kN**	**kN**
0.57	*1.00*	*1*	*0*	*0.0074*	*0.0086*
0.57	*0.98*	*1*	*2*	*0.0377*	*0.0461*
0.57	*0.74*	*1*	*4*	*0.0394*	*0.0468*

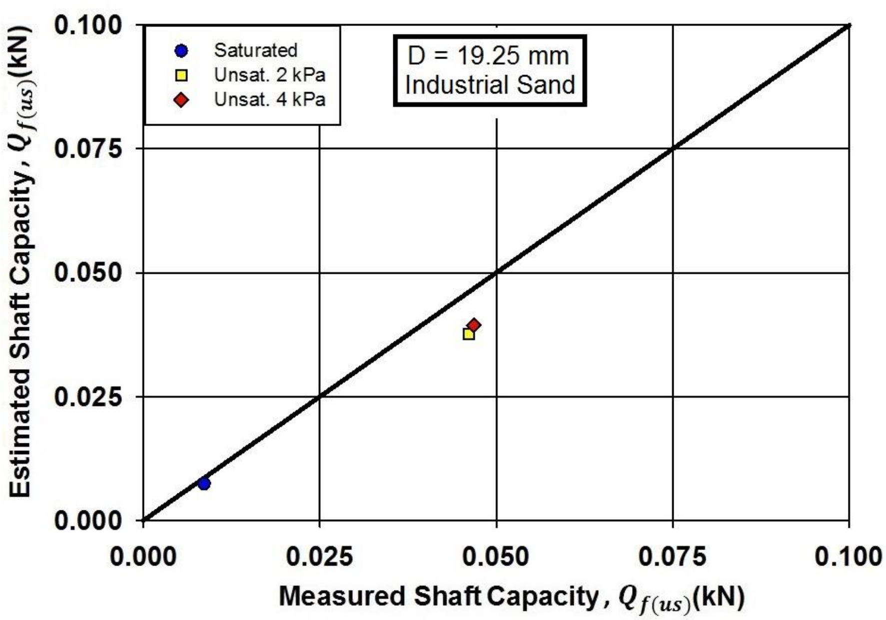


Figure 6.19 *Comparison between the measured and predicted model pile shaft capacity for Pile D19.25 mm in Industrial sand*

Results summarized in Figure 6.14 to 6.19 show the ultimate pile shaft capacity in unsaturated sands is significantly higher (i.e., approximately 5 times) in comparison to saturated condition. In present study, the modified β is used for analysis of the pile shaft test results. As the contribution of pile shaft capacity in sandy type of soils is less than 5% of the total pile capacity (see section 5.3), precise measurement of pile shaft capacity (i.e., in comparison with pile base capacity) is challenging. Results of the analysis on two *selected soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) predicted the pile shaft capacity using the modified β method are in agreement with experimental results (i.e., approximately 20% difference between the measured and predicted values).*

6.6 Summary

In this chapter, the results of the single model pile load tests conducted under both saturated and unsaturated (i.e., matric suction value equals to 2 and 4 kPa) conditions on two selected soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) with three model pile with varying diameters (i.e., 38.30, 31.75, and 19.25 mm) were presented and analyzed to study the contribution of matric suction towards the pile bearing capacity (i.e., both pile base and shaft capacity).

The experimental model pile base capacity under unsaturated conditions was found to be approximately 2 to 2.5 times higher than the pile base capacity under saturated condition. Analysis of these test results encourage to modify the conventional methods for interpretation and prediction of pile base capacity. Three semi-empirical modified equations (Terzaghi 1943, Hansen 1970, and Janbu 1976) were proposed to predict the model pile base bearing capacity under unsaturated conditions taking into account the effect of matric suction. These proposed methods were used to interpret the results of the model pile load tests. The modified methods found to be suitable to predict the variation of pile base capacity with respect to matric suction (i.e., approximately less than 10% difference between the predicted and measured values).

The modified β method was used to interpret the results of model pile shaft capacity load tests. The experimental model pile shaft capacity under unsaturated condition was found to be approximately 5 times higher than the shaft capacity of model pile under saturated condition.

CHAPTER 7

FINITE ELEMENT MODELING OF THE LOAD VERSUS DISPLACEMENT BEHAVIOR OF THE SINGLE MODEL PILES IN SATURATED AND UNDERSATURATED SANDS

7.1 Introduction

The bearing capacity of pile foundations (i.e., p-δ behavior) can be reliably determined from in-situ tests. The in-situ determination of pile bearing capacity is however time consuming, needs trained professional and heavy equipment. Therefore, in many scenarios it is expensive. All these reasons contribute to estimating the pile bearing capacity and settlement behavior using numerical methods.

Experimental results of single model pile loading tests performed on two different unsaturated sandy soils in a laboratory environment were presented and interpreted in Chapter 5 and 6, respectively. These studies show the significant contribution from the matric suction towards the model pile bearing capacity. These experimental results highlight the need for extending the mechanics of unsaturated soils taking into account the effect of matric suction. Several researchers in recent years studied the influence of the matric suction on the p-δ behavior of single piles extending numerical techniques (Mohamedzein et al. 1999, Georgiadis et al. 2003, Muraleethran et al. 2009, Muraleethran and Ravichandran 2009, Ravichandran 2009, Krishnapillai and Ravichandran 2012, Taylan et al. 2012, Taylan 2013).

In this chapter, the single model pile test results presented in Chapter 5 were simulated using the commercial finite element software, SIGMA/W (GEO-SLOPE). The elastic-perfectly plastic constitutive model extending Mohr-Coulomb yield criterion was used in the numerical analysis (Chen and Zhang, 1991). The key tests used in the numerical *analysis include the conventional shear strength parameters (i.e., c' and ϕ'), the elastic modulus under saturated (i.e., E_{sat}) and unsaturated conditions (i.e., E_{unsat}), the soil-water characteristic curve (SWCC), and the variation of pore-water pressure with respect to depth are required. Numerical modeling approach used by Taylan et al. 2012 is extended in the numerical analysis in the present study.*

The SIGMA/W software (GEO-SLOPE) used in this study, is designed for modeling the soil mechanical behavior and limitation in modeling the non-soil material (i.e., steel) should be taken into account. However, the availability and simplicity of using this software justified modeling the model pile load test in this study.

7.2 Background

The volumetric behavior of unsaturated soils is dependent on initial and final stress, suction values, and the path from initial to final state. Irreversible volumetric deformation may occur as the suction value changes in unsaturated soils (i.e., wetting and drying cycles) (Alonso et al. 1990). Elastic-plastic models are used for modeling the variation of volumetric deformation of unsaturated soils due to swelling-shrinkage or net normal stress, (Alonso et al. 1990, Wheeler and Sivakumar 1995, Cui and Delage 1996, Rampino et al. 2000, Gallipoli et al. 2003). As such, the finite element analysis (here after referred as FEA) performed using the elastic-perfectly plastic framework. The Mohr-Coulomb yield criterion that is conventionally used to describe the soil behavior is extended in the present study. The elastic-perfectly plastic framework provides realistic simulation of the soil behavior (Chen and Zhang, 1991).

In order to formulate the elastic-perfectly behavior of the soil a yield function is required. The yield function defines the boundary between the elastic and plastic behavior of the soil (Griffiths and Lane, 1999). In the present study, the yield function (F) of elastic-perfectly plastic model is extended as a scalar function of stress components or stress invariants {σ} and state parameters {k}.

$$F(\{\sigma\},\{k\}) = 0 \qquad \textbf{(7.1)}$$

Extending this approach, the variation of state parameters {k} with plastic straining can be determined using a hardening/softening rule. Using a hardening/softening rule (i.e., taking into account the effect of dilatancy) has two important drawbacks, (i) providing unrealistic plastic volumetric strains in comparison with real soil behavior, and (ii) soil at yield surface will dilate repeatedly. However, in practice soils at yield surface, reach a constant volume condition (i.e., zero incremental plastic volumetric strains) (Potts and Zdravkovic 1999). Due to this reason, for the studied soils, the Mohr-Coulomb failure model is assumed as perfectly plastic and no hardening/softening rules were extended. In other words, the state parameter {k} is assumed to be constant and independent of plastic strain and the plastic volumetric strain behavior. In other words, the angle of dilatancy (ψ) is neglected (i.e., the dialatancy angle, ψ, is not required in the model).

The soil is assumed is assumed to be isotropic and elastic and the normal and shear stress at all nodes within the mesh using the Young's modulus (*E) and the* Poisson's ratio (*μ) is estimated. The generated stresses are compared with the Mohr-Coulomb failure criterion. If the stress state at any particular point lies within the Mohr-Coulomb failure envelope, the model assumes the material at the point is within the elastic domain. If the stress state falls outside the failure envelope, the model assumes the material at the point is yielding and the governing plastic behavior would generate the stresses using the shear strength parameters (i.e., c' and ϕ'). The shear strength behavior of unsaturated soils is simulated using the conventional saturated shear strength parameters along with SWCC. Extending this approach for modeling the unsaturated soils behavior reduces the number of required tests for determining the modeling parameters.*

7.3 Modeling of the *p-δ* Behavior of Single Model Piles in Unsaturated Sand using Elastic- Perfectly Plastic Mohr-Coulomb Model in *SIGMA/W*

The single model pile load tests undertaken in a laboratory environment were modeled using the SIGMA/W finite element software extending the elastic-perfectly plastic Mohr-Coulomb model for both saturated and unsaturated conditions.

7.3.1 Estimation of soil properties used in *FEA*

The pile load tests were performed using different pile diameters (i.e., pile diameter equal to 38.30, 31.75, and 19.25 mm) for two selected soils (i.e., Soil #1 Unimin 7030 sand and Soil #2 Industrial sand) under both saturated and unsaturated (i.e., matric suction value equal to 2 and 4 kPa) conditions. The FEA was performed using the SIGMA/W software program for these two soils to estimate the p-δ behavior of the single model piles.

The positive and negative pore-water pressure variations are assumed to change hydrostatically with depth. This assumption was extended based on the matric suction values, (u_a - u_w), measured by placing commercial Tensiometers at different depths in the test tank. The measured (i.e., Tensiometer's reading) and estimated (i.e., hydrostatic *variation assumption) matric suction values which were close to each other. The pile dimensions, groundwater table conditions, soil properties, and the matric suction values defined in FEA were same as detailed in experimental program in Chapter 4.*

The SWCC information of the two selected soils measured using the Tempe cell and reported in Chapter 5 was used for the FEA in SIGMA/W. The measured SWCC was compared and supported with one-point prediction technique proposed by Vanapalli and Catana (2005) as detailed earlier in Chapter 5 (see section 5.2.4 for more details). In the SIGMA/W software the SWCC function can be defined as a relationship between the volumetric water content, θ, and the matric suction, (u_a - u_w).

The unsaturated shear strength behavior of the two selected soils for conducting the FEA can be predicted using the SWCC and the saturated shear strength parameters (i.e., c' and ϕ'), using a semi-empirical equation proposed by Vanapalli et al. (1996), as below:

$$\tau_{unsat} = [c' + (\sigma_n - u_a)\tan\phi'] + (u_a - u_w)[\tan\phi' \left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)] \quad \textbf{(7.2)}$$

where, τ_{unsat} = shear strength of unsaturated soil, c' = effective cohesion, (σ_n - u_a) = net normal stress, ϕ' = effective internal friction angle under saturated condition, (u_a - u_w) = matric suction, θ = volumetric water content at any suction, θ_s = the volumetric water *content at saturation, θ_r = the residual volumetric water content. The θ_r can be gained from the SWCC.*

The effective cohesion, c', for saturated condition and the SWCC function are required in the SIGMA/W software to estimate the variation of the total cohesion, c, using the following relationship:

$$c = c' + (u_a - u_w)[\tan\phi'\left(\frac{\theta - \theta_r}{\theta_s - \theta_r}\right)] \quad \textbf{(7.3)}$$

The pore-water pressure variation with respect to depth is defined in SIGMA/W by setting an initial water table, h_{sat}, and a maximum negative pressure head, h_{max}, assuming the hydrostatic variation of positive and negative pore-water pressure above and below the ground water table. If the height of the unsaturated soil layer is higher than the defined maximum pressure head (i.e., $h_{max} < h_{unsat}$), the negative pore-water pressure is constant up to the ground surface beyond the maximum negative pressure head. On the other hand, if the height of unsaturated soil layer is lower than the defined maximum negative pressure head (i.e., $h_{max} > h_{unsat}$) the negative pore-water pressure increase hydrostatically up to ground surface (Figure 7.1).

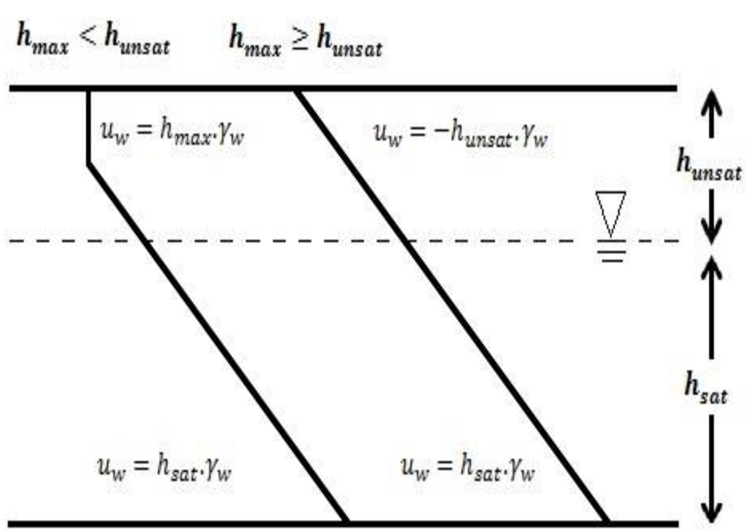


Figure 7.1 *Pore-water pressure variation with respect to depth in SIGMA/W (modified after Oh and Vanapalli, 2011).*

The initial modulus of elasticity for saturated condition, E_{sat}, is calculated using the load-displacement curves measured in pile load tests for different pile diameters for the two sands. Oh and Vanapalli (2011) proposed a method for estimating the initial tangent modulus of elasticity, E_i, using the model footing load test results extending the method proposed by Timoshenko and Goodier (1951).

$$E_i = \frac{\Delta q}{(\Delta\delta/1.5B)} \tag{7.4}$$

where, Δq = increment of applied stress in elastic range, $\Delta\delta$ = increment of settlement in elastic range, B = width or diameter of model footing. 1.5B in Eq. (7.4) indicates the depth at which the stress below the foundation is predominant (i.e., stress bulb zone).

Extending the technique proposed by Oh and Vanapalli (2011) and the results of single model pile load tests, the initial tangent modulus of elasticity for saturated condition for the two selected soils was estimated (see Table 7.1)

The variation of the elastic modulus of unsaturated soils, E_{unsat}, with respect to matric suction can be predicted using the SWCC and the saturated modulus of elasticity, E_{sat}, by a semi-empirical method proposed method by Oh et al. 2009.

$$E_{unsat} = E_{sat}\left[1 + \alpha \frac{(u_a - u_w)}{P_a/101.3} S^{\beta}\right] \tag{7.5}$$

where, E_{sat}, E_{unsat} = elastic modulus under saturated and unsaturated conditions, respectively, $(u_a - u_w)$ = matric suction, S = degree of saturation, α, β = fitting parameters, and P_a = atmospheric pressure (i.e., 101.3 kPa).

The trends of variation of the elastic modulus and the bearing capacity of unsaturated sands with respect to matric suction are similar. As the matric suction increases up to air-entry value, the elastic modulus, E, linearly increases, and beyond the air-entry value, E increases non-linearly up to residual matric suction. The contribution of the matric suction towards E, then tends to decrease as matric suction approaches residual suction. The proposed value of β equal to unity can be used for coarse-grained soils with plasticity index, I_p, equal to zero (Oh et al., 2009). There is a relationship between the fitting *parameter, α and the model pile diameter. The fitting parameter, α increases non-linearly with increasing diameter. Oh et al. (2009) suggested that the contribution of matric suction towards the modulus of elasticity decreases with ratio of foundation size to soil particle. As the foundation diameter is relatively small in comparison with soil particle size, the contribution of the load applied on the model foundation carried by the individual soil particles is dominant in comparison with the friction arising at the contact points of the soil particles. They suggested that α value should be selected in the range of 1.5 to 2.*

Table 7.1 *Estimated E_{sat} for the two selected soils*

Pile diameter (mm)	$\boldsymbol{E_{i(sat)}}$ *for Soil #1 (kPa)*	$\boldsymbol{E_{i(sat)}}$ *for Soil#2 (kPa)*
38.30	*3500*	*7500*
31.75	*3000*	*6500*
19.25	*2000*	*4000*

The saturated and unsaturated moduli of elasticity (i.e., E_{sat} and E_{unsat}) of the two soils were estimated using Eq. 7.4 and 7.5 respectively. The unsaturated modulus of elasticity (E_{unsat}) is a function of matric suction (i.e., Eq. 7.5) and the variation of matric suction with depth in the soil was assumed to be hydrostatic, as discussed earlier in this chapter. In other words, the unsaturated modulus of elasticity can be defined in SIGMA/W as a function of depth. This approach defines the saturated modulus of elasticity, E_{sat}, constant up to the GWT and for unsaturated conditions the unsaturated modulus of elasticity, E_{unsat}, varies with respect to depth above the GWT.

7.4 Model parameters

In order to model the p-δ behavior of single model pile in SIGMA/W, axisymmetric analysis was conducted using quadrilateral and triangular elements with single nodes considering appropriate boundary conditions. The single model pile was circular, *installed vertically in the test soil tank and was axially loaded vertically, thus, axi-symmetric analysis is appropriate (Potts and Zdravkovic, 1999). Figure 7.2 presents the key details and stress-strain boundary conditions used in modeling with respect to actual test setup in testing program detailed earlier in Chapter 4. The vertical boundary of the model test setup was assumed to be fixed-X due to the confinement of the soil from any movement along x-axis (Figure 7.2 (a)). Also, the base boundary of the model test setup was fixed-XY to restrict the soil from moving along the x and y-axis (Figure 7.2 (b)). The pile was loaded by increments of vertical displacement on its top using a force-displacement boundary condition (i.e., displacement rate -0.0025 m/s) (Figure 7.2 (c)). The saturation condition was simulated by drawing the initial water table (Figure 7.2 (d)). The soil is modeled as elastic-perfectly plastic material and the pile as linear elastic material with relatively high modulus of elasticity compared to the soil. The model parameters used for this study in SIGMA/W software using elastic-plastic Mohr-Coulomb model are summarized in Table 7.2. The* Poisson's *ratio (μ) for the two soils and the pile material (i.e., steel) was 0.334 and 0.15, respectively. The coefficient of earth pressure at rest (K_0) was selected for the two soils studied as equal to 1. The* Poisson's ratio *(μ) and the coefficient of earth pressure at rest (K_0) for the present study were similar to the values used by other investigators for performing the FEA (Potts and Zdravkovic 1999, Oh and Vanapalli 2011, Taylan et al. 2012). The model pile load and displacement (i.e., p-δ) in the FEA is measured at the pile top, where the load is applied to the model pile to take into account both shaft and base resistance contributions.*

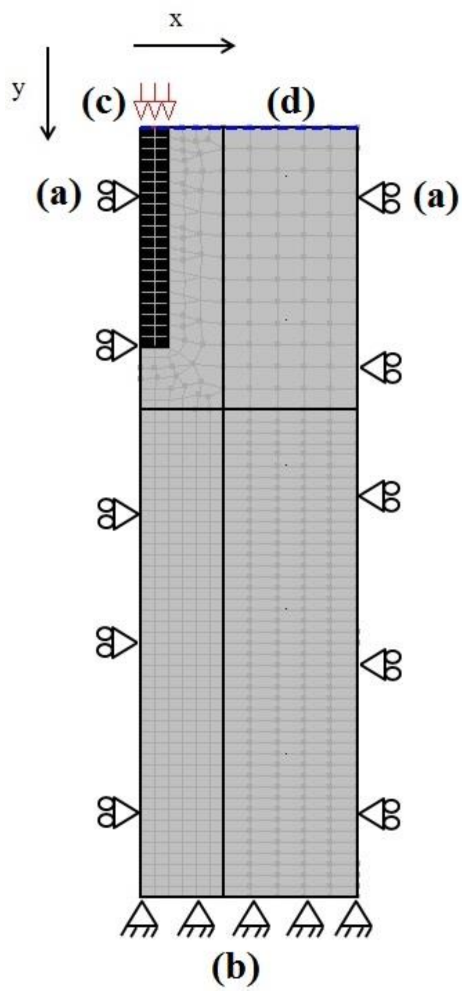


Figure 7.2 *Finite element modeling in SIGMA/W, (a) fixed-X, (b) fixed-XY, (c) loading, (d) initial water table.*

7.5 Finite element analysis results

The FEA results of the single model pile load tests using the SIGMA/W software are compared with the results of the model pile load tests performed in laboratory environment. The load displacement (i.e., $p-\delta$*) behavior of the single model piles for both saturated and unsaturated conditions for two sands studied is presented in Figure 7.3 to 7.8.*

Table 7.2 *Model parameters used in SIGMA/W for the FEA*

Model Parameters	*Soil #1*	*Soil#2*	*Pile*
Model	*Elastic-Plastic*	*Elastic-Plastic*	*Linear-Elastic*
Effective cohesion, c' (kPa)	*2.60*	*3.30*	-
Effective internal friction angle, ϕ' *(°)*	*35.30*	*40.30*	-
Total unit weight, γ *(kN/m*^{3}*)*	*18.6*	*19.9*	*24*
Saturated modulus of elasticity, $E_{i(sat)}$ *(kPa)*	*3500*	*7500*	20×10^6
Coefficient of earth pressure at rest, K_0	*1*	*1*	-
Poisson's ratio, μ	*0.334*	*0.334*	*0.15*

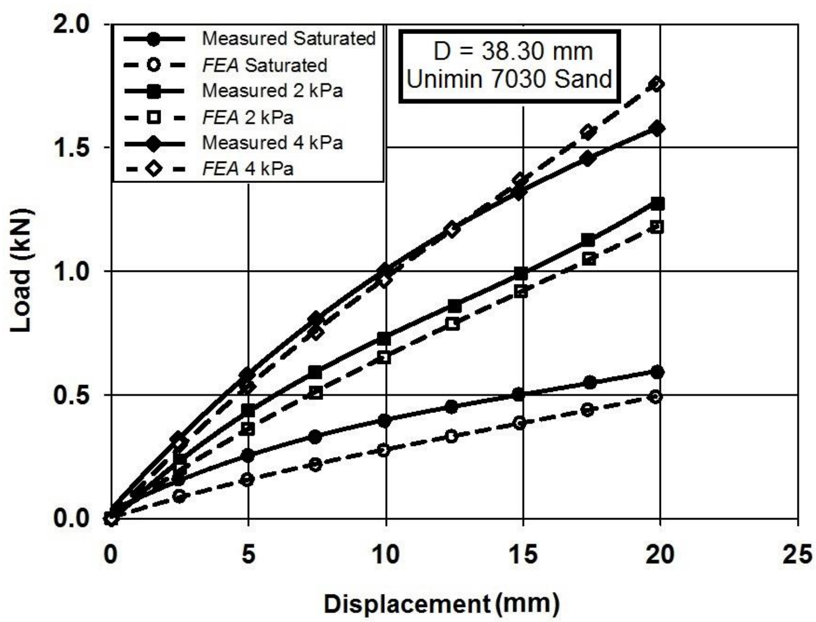


Figure 7.3 *Comparison between the measured and FEA model pile total capacity for Pile D38.30 mm in Unimin 7030 sand*

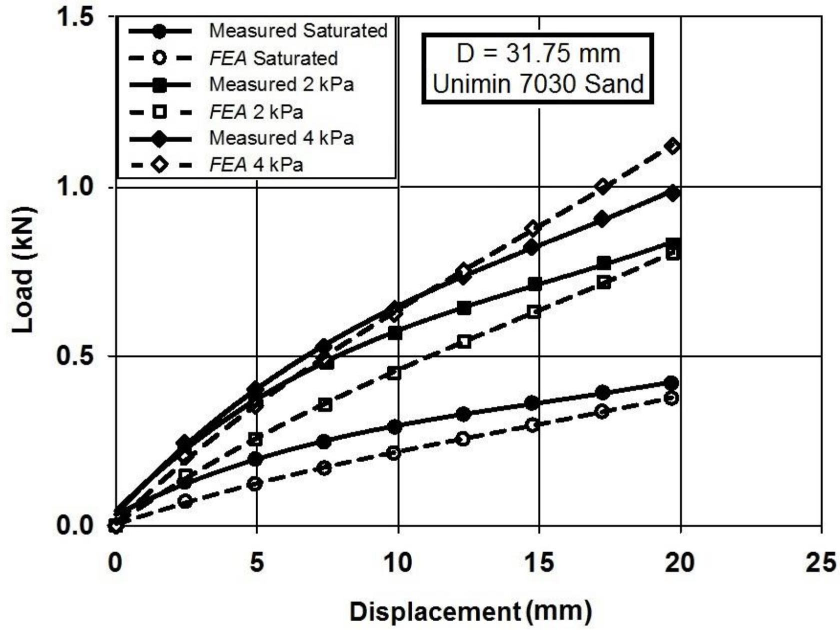


Figure 7.4 *Comparison between the measured and FEA model pile total capacity for Pile D31.75 mm in Unimin 7030 sand*

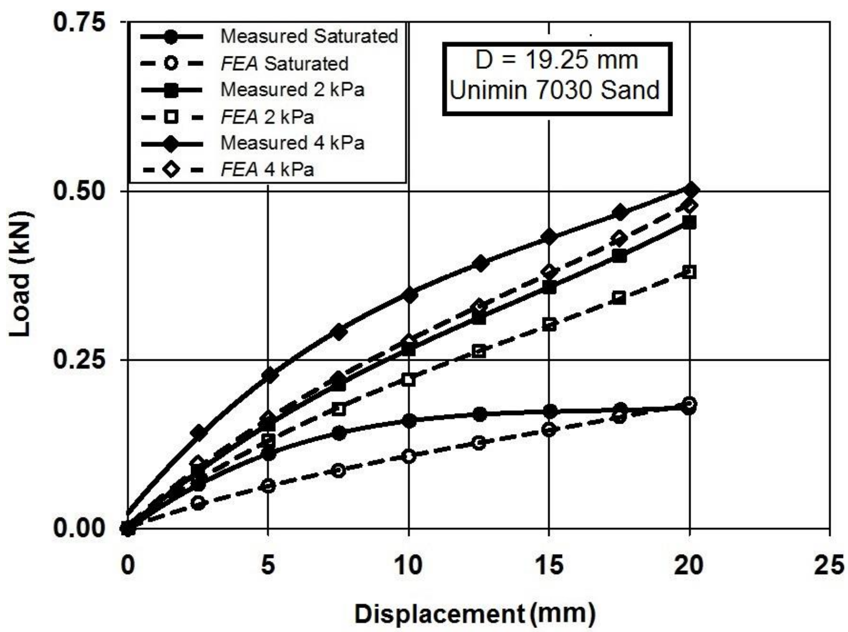


Figure 7.5 *Comparison between the measured and FEA model pile total capacity for Pile D19.25 mm in Unimin 7030 sand*

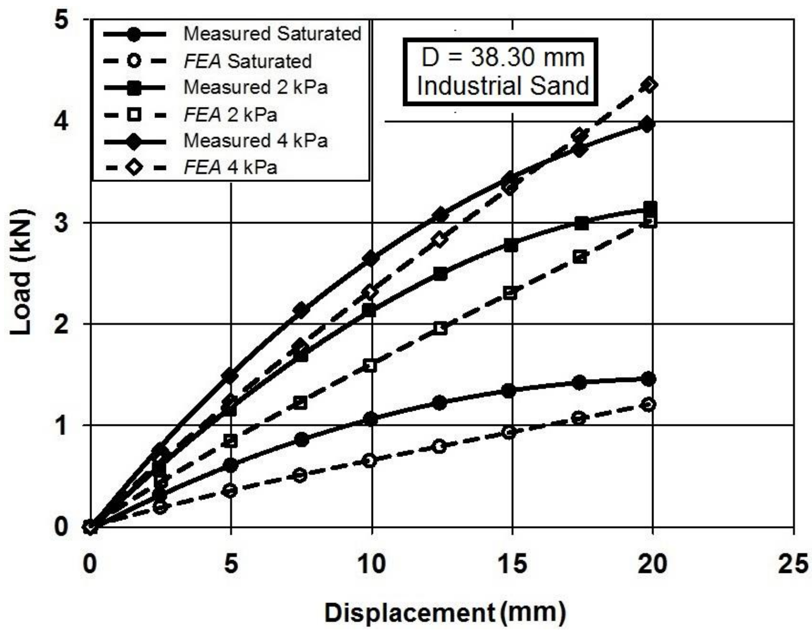


Figure 7.6 *Comparison between the measured and FEA model pile total capacity for Pile D38.30 mm in Industrial sand*

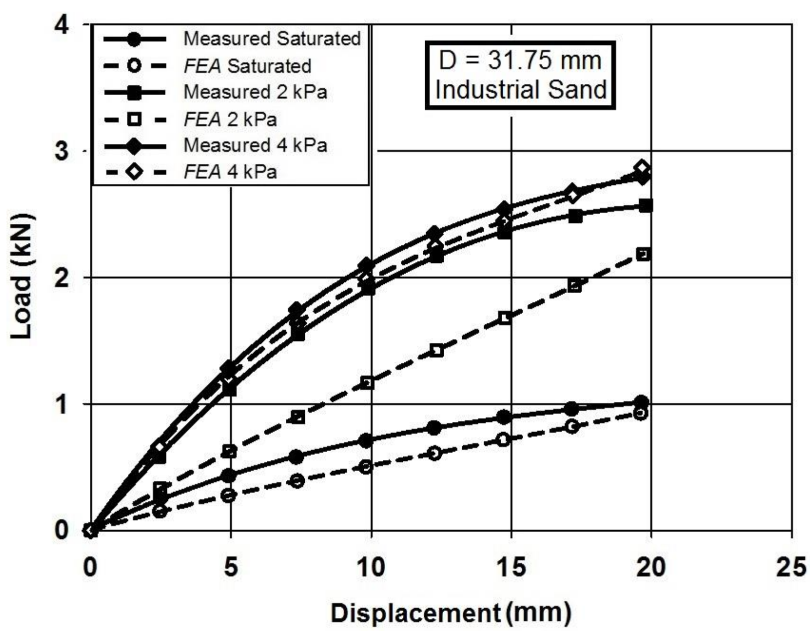


Figure 7.7 *Comparison between the measured and FEA model pile total capacity for Pile D31.75 mm in Industrial sand*

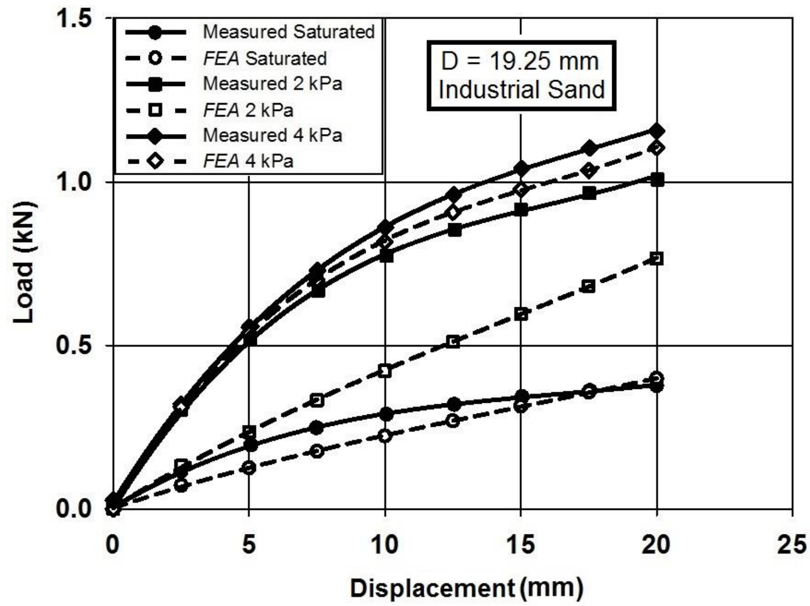


Figure 7.8 *Comparison between the measured and FEA model pile total capacity for Pile D19.25 mm in Industrial sand*

FEA results indicate significant influence of the matric suction on the pile bearing capacity in unsaturated sands (i.e., approximately 3 times higher values) in comparison with saturated condition. The estimated bearing capacity values obtained from FEA are close to the measured values (i.e., in average 10 to 15% difference between the measured and estimated values). In other words, there is a reasonable agreement between the results of FEA and the model pile load tests under both saturated and unsaturated conditions. This agreement validates the feasibility of using the elastic-perfectly plastic Mohr-Coulomb model in the FEA of single model piles behavior in unsaturated conditions. As detailer earlier in this chapter, the elastic plastic model can be used for FEA of model piles under both saturated and unsaturated conditions using the conventional shear strength parameters (i.e., c' and ϕ') along with the SWCC in the SIGMA/W software. The dilatancy effect (i.e., the dilatant plastic volumetric strain behavior which is dependent on the angle of dilatancy, ψ) is neglected in this study, as the Mohr-Coulomb failure model is assumed to be perfectly plastic, and no hardening/softening rules are required. Extending this approach for unsaturated soils may reduce the required number of tests for determining the parameters for the FEA (i.e., determination of the angle of dilatancy, ψ,s *is not required). In addition, the proposed FEA is more advantageous compared to in-situ or laboratory pile load tests, due to the fact that it is more cost effective and less time consuming.*

7.6 Summary

The load-displacement (i.e., p-δ) behavior can be extended for prediction of the bearing capacity of pile foundations using the effective shear strength parameters (i.e., c' and ϕ') along with the SWCC. Experimental studies presented in this thesis indicate significant contribution of matric suction towards the bearing capacity of single pile foundations located in unsaturated sandy soils.

In the present chapter, an elastic-perfectly plastic Mohr-Coulomb model is extended to predict the load-displacement behavior (i.e., p-δ) of single model piles under both saturated and unsaturated condition for two selected sandy soils (i.e., Soil #1 Unimin 7030 sand, and Soil #2 Industrial sand) for different pile diameters (i.e., pile diameter equals to 38.30, 31.75, and 19.25 mm) using the commercially available FEA software program SIGMA/W. The theoretical background for performing FEA using the extended numerical model is provided. The required parameters for the model include the effective shear strength parameters (i.e., c' and ϕ'), the soil-water characteristic curve (SWCC), the saturated modulus of elasticity (i.e., E_{sat}) (i.e., obtained from the load-displacement curves), the variation of elastic modulus for unsaturated conditions with respect to depth, and the variation of pore-water pressure along the soil profile. Results of the FEA were compared with measured values from the single model pile load tests. A reasonable agreement was observed between the load displacement behavior of single model pile under both saturated and unsaturated conditions using the numerical modeling technique (i.e., the FEA in the SIGMA/W software) and load testing program. This acceptable agreement encourages using the elastic-perfectly plastic Mohr-Coulomb model extended for FEA using the SIGMA/W software as a useful tool to predict the load-displacement behavior of single pile foundations for both saturated and unsaturated soils.

CHAPTER 8

SUMMARY AND CONCLUSIONS

8.1 Introduction

Bearing capacity of both the shallow and deep foundation in engineering practice are based on conventional principles of soil mechanics assuming the soil to be in a saturated, submerged or dry condition. However, recent research studies suggest that there is a significant contribution of matric suction towards the bearing capacity of both shallow and deep foundations in both unsaturated fine and coarse-grained soils (for example, Douthitt et al. 1998, Georgiadis et al. 2002, Georgiadis et al. 2003, Mohamed and Vanapalli 2006, Ravichandran and Shada 2010, Vanapalli and Taylan 2012, Oh and Vanapalli 2013). In many cases, a portion or the entire length of pile foundations is placed above the ground water table where the soil is in a state of unsaturated condition (i.e., capillary zone). Such scenarios are common in arid and semi-arid regions of the world. Due to this reason, there has been significant interest in recent years to understand the contribution of matric suction towards the bearing capacity in unsaturated soils (for example, Douthitt et al. 1998, Georgiadis et al. 2003, Vanapalli and Taylan 2012). There is limited number of studies in this direction to convince the geotechnical engineers to use them with confidence in the conventional design of pile foundations. Most of the research in this direction is still in its infancy.

The main focus of this thesis research is directed towards two key objectives. The first objective was to develop reliable interpretation techniques by modifying the presently used methods in the literature and also extend them into the form of semi-empirical models to predict the contribution of matric suction of the base and shaft capacity of model single piles in sands under both saturated and unsaturated conditions. The second objective was directed towards developing a numerical modeling technique using finite element analysis to simulate the load displacement (i.e., p-δ) behavior of single model *piles using an elastic-perfectly plastic Mohr-Coulomb model taking into account influence of matric suction in unsaturated sandy soils.*

Several conclusions derived from the studies undertaken in this thesis are summarized in the following sections.

8.2 Estimation of the Bearing Capacity of Single Model Piles in Unsaturated Sandy Soils

- *Comprehensive experimental program was planned and conducted in the geotechnical laboratory to investigate the variation of base, shaft and total bearing capacity of single model piles in two sandy soils with respect to matric suction under both saturated and unsaturated conditions. The single model piles used in this study program were stainless steel cylindrical piles with different diameters (i.e., 38.30, 31.75, and 19.25 mm). The base, shaft and total bearing capacity of stainless steel piles in two different sandy soils (i.e., Soil #1 Unimin 7030 sand, Soil #2 Industrial sand) were measured independently using specially designed equipment. The model pile load tests were performed under three different average matric suction values (i.e., 0, 2, and 4 kPa) by varying the matric suction in the aluminum test tank with the compacted sand using hanging column technique (i.e., plexi-glass water container).*
- *The bearing capacity of single model piles under unsaturated conditions is significantly higher (i.e., approximately 2 to 2.5 times for base capacity and 5 times for shaft capacity) in comparison with saturated conditions for both the sands studies in this research program due to contribution of matric suction. These experimental results highlight the need for modification of presently used conventional design procedures for single piles as they are highly conservative for use in sandy soils that are typically in a state of unsaturated condition.*
- *Three conventional methods; Terzaghi (1943), Hansen (1970), and Janbu (1976), were modified to interpret the influence of matric suction on the base bearing capacity of single piles. In addition, semi-empirical models were developed for*

predicting the variation of the load carrying capacity of single piles with respect to suction using the effective shear strength parameters (i.e., c' and ϕ') and the soil-water characteristic curve (SWCC). Similar procedures that were used for prediction of the shear strength (Vanapalli et al. 1996), bearing capacity of shallow foundations in coarse-grained soils (Vanapalli and Mohamed 2007) and fine-grained soils (Oh and Vanapalli 2013) and pile foundations in fine-grained soils (Vanapalli and Taylan, 2012) were used in the present research pogrom for developing the semi-empirical models. Comparisons between the measured load carrying capacity of single and the predicted values using proposed semi-empirical models are within acceptable agreement.

The studies proposed in this thesis are simple and are based on methods used in conventional engineering practice for estimating the bearing capacity of unsaturated sandy soils.

8.3 Modeling the Load versus Displacement Behaviors of Single Model Piles in Unsaturated Coarse-grained Soils

- *The load-displacement (i.e., p-δ) behavior of single model piles in both saturated and unsaturated sands was simulated using a simple numerical model extending an elastic-perfectly plastic Mohr-Coulomb model in commercially available finite element program (i.e., SIGMA/W, Geostudio 2007). The FEA was performed using the effective shear strength parameters (i.e., c' and ϕ') and the SWCC. The effect of matric suction towards the bearing capacity of single model piles was taking into account conducting the FEA.*
- *The required parameters for performing the FEA include the SWCC (i.e., measured for both soils in laboratory), the effective shear strength parameters (i.e., c' and ϕ' obtained from direct shear test), the saturated modulus of elasticity (i.e., E_{sat} obtained from the load-displacement curves), the variation of elastic modulus for unsaturated conditions (i.e., calculated using a method proposed by Oh et al.,*

2009), and the variation of pore-water pressure with respect to depth (i.e., assuming hydrostatic change in pore-water pressure).

- *The irreversible volumetric deformation that may occur in unsaturated soils as the matric suction value changes (i.e., wetting and drying cycles) can be reliably interpreted using elastic-plastic models (Alonso et al. 1990, Wheeler and Sivakumar 1995, Cui and Delage 1996, Rampino et al. 2000, Gallipoli et al. 2003). Taking into account the effect of dilatancy in the soil behavior modeling (i.e., using hardening-softening rules) may lead to unrealistic plastic volumetric strains in comparison with real soil behavior. In addition, as the soil yields it will dilate repeatedly; however, in real soil behavior, a constant volume condition will occur after reaching the failure surface (Potts and Zdravkovic 1999). Due to this reason, the soil behavior is assumed to be perfectly plastic as a first approximation in the present study extending no hardening/softening rule in the modeling. Therefore, the effect of dilatancy is neglected (i.e., determination of the dilatancy angle, ψ, is not required in this model). This approach is simple and alleviates the need for several tests for determining the model parameters. More research is however necessary in future studies to understand the influence of effects of dilatancy in the numerical modelling simulations.*
- *Comparisons between the $p - \delta$ behavior of single model piles located in coarse-grained soils under both saturated and unsaturated conditions obtained from the FEA and pile load tests were in acceptable agreement.*
- *The proposed numerical method, which is a first step towards simulating the load displacement behavior is promising and can be extended with modifications based on more rigorous studies to reliably predict the p-δ behavior of in-situ single piles.*

8.4 Recommendations and Suggestions for Future Studies

The recommendations and suggestion offered for future research studies are summarized in this section.

- *The proposed techniques and models in the present thesis are developed based on limited experimental results of two sandy soils. More research studies on different type of sandy soils are necessary using large scale equipment (i.e., soil test tank, model pile) to better understand the boundary effect limitations in the research studies.*
- *In-situ pile load tests should be carried out in unsaturated sandy soils to validate the proposed techniques and models taking into account the contribution of matric suction towards the bearing capacity of single piles. Such studies can be encouraging for the practicing engineers to use the mechanics of unsaturated soils principles in practice for estimating the bearing capacity of single piles.*
- *In the present research study, the bearing capacity and displacement behavior of the single model piles was measured and interpreted for axial loading conditions only. However, in engineering practice other scenarios such as tension and lateral loading are common. More studies are necessary in this direction to better understand the behavior of single piles.*
- *Similar studies should be extended with the present research objective to understand the behavior load carrying capacity and displacement of pile groups in unsaturated soils. Such studies are of more interest for the practicing engineers.*

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APPENDIX

BEARING CAPACITY CALCULATIONS

Table A.1 *Required information for estimating the base and shaft capacity of single model pile in Unimin 7030 sand*

Soil Property		Unimin 7030 sand
Soil friction angle, ϕ' (◦)	From Lab. test	35.3
Soil friction angle, ϕ' (◦)	Modified	39.0
Effective cohesion, c' (kPa)		2.6
Soil-steel interface friction, δ' (◦)		24.2
Saturated unit weight, γ_{sat} (kN/m^3)		20.4
Total unit weight, γ_{total} (kN/m^3)		18.6
Air-entry value, $(u_a - u_w)_b$ (kPa)		3.0
Fitting parameter, ψ		1
β value		0.47
Fitting parameter, κ		1

Table A.2 *Required information for estimating the base and shaft capacity of single model pile in Industrial sand*

Soil Property		*Industrial sand*
Soil friction angle, ϕ' (◦)	*From Lab. test*	*40.3*
Soil friction angle, ϕ' (◦)	*Modified*	*44.0*
Effective cohesion, c' (kPa)		*3.3*
Soil-steel interface friction, δ' (◦)		*33.1*
Saturated unit weight, γ_{sat} (kN/m^3)		*20.8*
Total unit weight, γ_{total} (kN/m^3)		*19.9*
Air-entry value, $(u_a - u_w)_b$ (kPa)		*2.4*
Fitting parameter, ψ		*1*
β value		*0.57*
Fitting parameter, κ		*1*

Table A.3 *Bearing capacity and shape factors were used in Terzaghi (1943) method for analysis*

Soil Type	N_c	N_q	N_γ	s_c	s_γ
Unimin 7030 sand	*88.0*	*73.0*	*89.0*	*1.3*	*0.6*
Industrial sand	*180.0*	*183.0*	*339.0*	*1.3*	*0.6*

Table A.4 *Bearing capacity and depth factors were used in Hansen (1970) method for analysis*

Soil Type	N'_c	N'_q	N_γ	d_c	d_q
Unimin 7030 sand	*68.0*	*56.0*	*68.0*	*1.55*	*1.30*
Industrial sand	*145.0*	*150.5*	*232.0*	*1.55*	*1.25*

Table 0.5 *Bearing capacity and depth factors were used in Janbu (1976) method for analysis*

Soil Type	N'_c	N'_q	N_γ	d_c	d_q
Unimin 7030 sand	*69.50*	*58.0*	*68.0*	*1.55*	*1.30*
Industrial sand	*134.0*	*135.0*	*232.0*	*1.55*	*1.25*