PREDICTION OF THE VARIATION OF SWELLING PRESSURE AND 1-D HEAVE OF EXPANSIVE SOILS WITH RESPECT TO SUCTION

Hongyu Tu

Thesis submitted to the
Faculty of Graduate and Postdoctoral Studies
in partial fulfillment of the requirements for the degree of
Master of Applied Science in Civil Engineering

Department of Civil Engineering
Faculty of Engineering
University of Ottawa
Ottawa, Ontario, Canada

Dedicated to my parents,

who gave their endless love, support, and encouragement

ACKNOWLEDGEMENT

This is a great opportunity for me to express my deepest appreciation to **Professor Sai K. Vanapalli**, who is such an energetic and active-thinking genius. As a supervisor, he is always providing helpful advice, constructive criticism and inspiring encouragement. He is not only a supervisor guiding my research, but also a philosopher sharing his academic and life experiences. Without his expertise, understanding, meticulous comments, and generous guidance this dissertation would not be possible.

I owe a deep sense of gratitude to **Dr. Won Taek Oh**, for his collaborative efforts on helping with my first publication, and for giving his precious and kind advice regarding the topic of my research.

I'm highly indebted and thoroughly grateful to **Mr. Zhong Han** and **Mr. Shunchao Qi**, for their immense interest on my research, for providing me with materials, for inspiring and encouraging me upon difficulties, for criticizing and improving my technical writing skills, and for being constant sources of motivation.

I would like to thank **Professor Weilie Zou**, who offered me a precious chance to work on a conference publication together. I owe him a deep sense of appreciation, for sharing his ideas on the topic of my research, and for his kind suggestions on my future career.

I would also like to thank all my office colleagues and other friends, together with whom I have shared both my happiness and sorrows over the last two years as an international graduate student.

I want to express my acknowledgments to my family and my girlfriend, who always provide their love, support, and encouragement. Without their support, this dissertation would not be possible.

Hongyu Tu

ABSTRACT

The one-dimensional (1-D) potential heave (or swell strain) of expansive soil is conventionally estimated using the swelling pressure and swelling index values which are determined from different types of oedometer test results. The swelling pressure of expansive soils is typically measured at saturated condition from oedometer tests. The experimental procedures of oedometer tests are cumbersome as well as time-consuming for use in conventional geotechnical engineering practice and are not capable for estimating heave under different stages of unsaturated conditions. To alleviate these limitations, semi-empirical models are proposed in this thesis to predict the variation of swelling pressure of both compacted and natural expansive soils with respect to soil suction using the soil-water characteristic curve (SWCC) as a tool. An empirical relationship is also suggested for estimating the swelling index from plasticity index values, alleviating the need for conducting oedometer tests. The predicted swelling pressure and estimated swelling index are then used to estimate the variation of 1-D heave with respect to suction for expansive soils by modifying Fredlund (1983) equation. The proposed approach is validated on six different compacted expansive soils from US, and on eight field sites from six countries; namely, Saudi Arabia, Australia, Canada, China, US, and the UK. The proposed simple techniques presented in this thesis are friendly for the practitioners for using when estimating the heave in unsaturated expansive soils.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

NOMENCLATURE

Subscripts

f	Final value
w	Subsequent wetting condition
i	Initial value, or order
max	Maximum value
min	Minimum value
opt	Optimum condition
mea	Measured value
emp	Empirical value
reg	Result of regression analysis
cal	Result of back-calculation
$field$	Result of field investigation
c	Properties of compacted expansive soil
n	Properties of natural expansive soil
p	Predicted value

Abbreviations

Eq.	Equation
Fig.	Figure
d	Day
h	Hour
min	Minute
s or sec	Second
AEV	Air entry value
FS	Free swell
LS	Loaded swell
CVS	Constant volume swell
COLE	Coefficient of linear extensibility
1-D	One dimensional
SWCC	Soil-water characteristic curve
TMI	Thornthwaite moisture index
FSI	free swell index
MBV	Methylene blue value
SP	Surrogate path

Symbols

Δ	Change
P, q (kPa)	Surcharge
P_s (kPa)	Swelling pressure
P_{s0} (kPa)	Intercept on the P_s axis at zero suction value
P_{sf} (kPa)	Swelling pressure measured from FS test
P_{su} (kPa)	Uncorrected swelling pressure measured from CVS test
P_{sc}, P_s', σ'_{cv} (kPa)	Corrected swelling pressure measured from CVS test
P_f, σ_f (kPa)	Final stress state
C_s	Swelling index
C_H	Heave index
C_{SP}	Slope of the surrogate path (SP)
C_c	Compression index
C_i (mol/m^3)	The molar concentration of solute i in dilute solution = n/V
C_τ, C_ψ	Suction Index
C_m	Compressive Index with respect to Matric Suction
$C(\psi)$	Correction factor which is primarily a function of suction corresponding to residual water content
a_m	Coefficient of Compressibility with respect to Matric Suction
a_f, n_f, m_f	SWCC fitting parameters
b_0, b_1, b_2, b_3	Fitting parameters for the diffusion coefficient
I''_{pt}, I''_{pm}, I''_{ps}	Instability Index
I_p	Plasticity index
I_s (%)	Shrinkage index
IL	Liquidity index = $[(w_l - w) / I_p]$
w (%)	Water content
w_l (%)	Liquid limit
w_p (%)	Plastic limit
w_s (%)	Shrinkage limit
w_r (%)	Residual water content
w_{opt} (%)	The optimum water content
θ, θ_w (%)	Volumetric water content
ρ_d (kg / m^3)	Dry density of soil
$\rho_{d,\max}$ (kg / m^3)	The maximum dry density
ρ_w (kg / m^3)	Density of water
ψ (kPa)	Soil suction or total suction
ψ_a or AEV (kPa)	Air-entry value
ψ_r (kPa)	Residual suction

h (pF)	Soil suction or total suction
u_a (kPa)	Pore-air pressre
u_w (kPa)	Pore-water pressure
$(u_a - u_w)$ (kPa)	Matric suction
$(u_a - u_w)_e$ (kPa)	Matric suction equivalent
$(\sigma_y - u_a)$ (kPa)	Net normal stress
π (kPa)	Osmotic suction
R (J/mol/K)	universal (molar) gas constant (i.e., 8.31432 J/mol/K)
R_h	Relative humidity = $\bar{u}_v / \bar{u}_{v0}$
R_1, R_2 (m)	The principal radii of curvature of the interface
R_m (m)	The first or mean radius of curvature = $(R_1^{-1} + R_2^{-1})^{-1}$
t (day)	time
t (℃)	Temperature
T (K)	Kelvin temperature = $(273.16 + t)$
T_s (kPa)	Surface tension
υ_{w0} (m^3 / kg)	Specific volume of water or the inverse of the density = $1 / \rho_w$
ω_v (kg / kmol)	Molecular mass of water vapor
$\bar{u}_v$ (kPa)	Partial pressure of pore-water vapor
$\bar{u}_{v0}$ (kPa)	Saturation pressure of water vapor over a flat surface of pure water at the same temperature
ΔP (kPa)	Pressure difference across the air-water interface
V (m^3)	Volume
n (m^3)	Number of mols of solute dissolved in the volume V of the solution
n	Frequency of the climate change
S	Degree of saturation
y (ft)	Space coordinate for depth
$u(y,t)$ (pF)	The suction as a function of space, y, and time, t
U_e (pF)	The equilibrium soil suction below the depth of active zone
U_0 (pF)	The amplitude of suction variation at the surface
$\Delta U_{\max}$ (pF)	The maximum suction change
α (cm^2/s)	The diffusion coefficient
α	Volume compressibility factor
z_a (M)	Depth of seasonal moisture fluctuation
(dh/dw)	Inverse moisture characteristic
γ_h, C_h	Suction compression index
PE_y	Potential evapotranspiration
DF_y	Deficit of evapotranspiration
R_y	Run off
SP	Swelling potential
S %	Percent swell

ε_s, ε_Q	Swell strain
ε_{cv}	Swell strain determined from the LS test applying a load equal to σ_{ob}
e	Void ratio
e_0, e_i	Initial void ratio
A	Activity = (I_p / c)
ΔH	Heave
H or h_i (m)	Thickness of the i^{th} soil layer
σ_y, σ'_{v0}, P_0 (kPa)	Overburden pressure
σ'_i (kPa)	Inundation pressure during the CVS test
σ_{ob} (kPa)	Confining pressure for the swell test
σ_{ocv} (kPa)	Swelling pressure determined form the CVS pressure for a test specimen initially subjected to σ_{ob}
G_s	Specific gravity
f	Lateral restraint factor
B	Slope of suction versus water content relationship
K_0	Coefficient of lateral earth pressure at rest
β_c	Model parameter for compacted expansive soil
β_n	Model parameter for natural expansive soil
ξ	Scale factor for matric suction equivalent

CHAPTER 1

INTRODUCTION

1.1 Statement of the problem

Expansive soils are widely distributed in arid and semi-arid regions, including some temperate regions of the world. During the last sixty years, several countries which include Australia, Argentina, Burma, China, Cuba, Ethiopia, France, Ghana, Great Britain, India, Iran, Israel, Kenya, Mexico, Morocco, South Africa, Spain, Turkey, United Kingdom, U.S.A. and Venezuela have reported expansive soil problems in various research conferences. Damages to infrastructure constructed with or within expansive soils have been mainly attributed due to their significant changes associated with their volume change behavior. In 1980, Krohn and Slosson estimated that $7 billion are spent each year in the United States as a result of damages to different structures built on expansive soils.

Expansive soils swell upon wetting and shrink upon drying due to seasonal changes (Chen 1975; Ng et al. 2003; Al-Homoud et al. 1995; Groenevelt and Grant 2004; Nwaiwu and Nuhu 2006; Erzin and Erol 2007; Zhan et al. 2007). The volume change of expansive soils with respect to changes in water content (or suction) is predominantly due to the influence of hydrophilic minerals such as the montmorillonite or illite. Once the swelling potential of expansive soils is restrained by surrounding soils or prevented by the overburden pressure or other loads, a counterforce which is commonly referred to as swelling pressure would be generated. The swelling pressure will be imposed on infrastructure such as the foundation slab, highway pavements, and outer wall of basements, tunnels and pipe lines, and consequently results in extensive damages (Fredlund et al. 1995). Moreover, the swell-shrink properties also contribute to instability of expansive soil slopes (Ng et al. 2003; Qi and Vanapalli 2015). Reliable estimation of the swelling pressure as well as the ground heave of expansive soils is important for practicing engineers for avoiding, reducing or alleviating the damages.

Efforts for finding a simple, effective and inexpensive methods for estimating the heave of expansive soils has been of significant interest to various researchers all around world since the 1950's. The methods for ground heave prediction that are available can be classified into three categories; namely: empirical methods, oedometer test-based methods, and suction-based methods (Nelson and Miller 1992; Vanapalli and Lu 2012).

Empirical methods are typically used in the estimation of the volume change indices (e.g., swelling pressure, P_s and swelling index, C_s) or heave (swell strain) directly from soil properties such as the Atterberg limits, dry density, water content, clay fraction, cation exchange capacity and specific surface area. Çimen et al. (2012) and Vanapalli and Lu (2012) provide comprehensive summaries on the empirical equations available from the literature. These equations are typically proposed based on investigations performed on local or limited number of soils and hence are not universally valid for all types of expansive soils.

The oedometer tests are used directly to measure the swelling pressure and volume change indices to estimate ground heave using various prediction equations (e.g., Fredlund 1983; Dhowian 1990a; Nelson and Miller 1992; Nelson et al. 2006). Detailed summaries of these methods are available in Nelson and Miller (1992) and Vanapalli and Lu (2012). Constant volume swell (CVS) test is believed to provide more reliable evaluation of the swelling pressure and volume change indices compared to other oedometer-based tests by applying corrections to the test results. Fredlund (1983), Rao et al. (1988), and Nelson and Miller (1992) have suggested correction procedures to take account of the sample disturbance and equipment compressibility. A detailed discussion about the strengths and limitations of various oedometer tests is presented in the second chapter. The most critical limitation is that the swelling pressure is conventionally measured by inundating the soil sample in an oedometer apparatus until no further tendency of swelling occurs at almost fully saturated condition (Singhal 2011). The expansive soils however in nature are most likely to be in a state of unsaturated condition. For this reason, the ground heave prediction based on the swelling pressure measured from the oedometer tests may contribute to unreliable estimation or over-estimation of the ground heave.

Soil suction based methods (e.g., U.S. Army Corps of Engineering (WES) method or CLOD test methods) were proposed to predict the ground heave of expansive soils. The suction based methods have the distinct advantage of estimating the variation of ground heave with respect to different soil suction. However, these tests do not take into account of effective stress (Nelson and Miller 1992). Furthermore, suction-based methods are based on direct measurement of unsaturated soil properties (i.e. soil suction, suction compression index). As soil suction and suction compression index cannot be reliably measured, these methods have not been widely accepted in engineering practice.

The recent focus of research has been directed towards developing semi-empirical methods using the soil-water characteristic curves (SWCC), which is the relationship between the gravimetric water content and soil suction, as a tool to predict the swelling pressure and heave (Vanapalli et al. 2012; Pedarla et al. 2012). More discussion of this method is discussed later in this chapter and later chapters.

Often heave is estimated by integrating the swell strain over the active zone. The swell strain can either be directly measured or be inferred from volume change parameters measured from oedometer tests or suction tests. Reliable determination on the volume change parameters (e.g., swelling pressure, P_s, and swelling index, C_s) is required for reasonable estimation on the ground heave.

1.2 Research objectives

To alleviate the limitations of the presently used methods for estimation of the heave, the research focus of the study in this paper has been directed towards proposing SWCC-based models for estimating the swelling pressure of expansive soils. Two SWCC-based prediction models are proposed; one for compacted soils, and the other natural expansive soils, for predicting the variation of swelling pressure (P_s) with respect to initial soil suction. Empirical relations between the model parameters and basic soil properties (i.e., plasticity index, clay content, and dry density) are suggested based on the analyses on several sets of data for compacted and natural expansive soils. An empirical relationship between swelling index (C_s) and plasticity index (I_p) is also developed extending the original contribution of Vanapalli and Lu (2012).

In addition, Fredlund (1983) equation which has proposed for estimating the potential heave for expansive soils that are in a state of saturated condition has been modified to estimate ground heave of expansive soils for unsaturated conditions.

These proposed techniques only require basic geotechnical test results and the information of the SWCC. In other words, the proposed techniques are simple and encouraging to implement the mechanics of unsaturated soils into geotechnical engineering practice for expansive soils.

The objectives of the present research are summarized as follows:

- To collect and summarize information including soil properties of expansive soils, the variation of swelling pressure with respect to different initial conditions (i.e., initial suction), and the SWCCs of the expansive soils from all over the world;
- To propose SWCC-based prediction models to predict the variation of the swelling pressure with respect to initial suction;
- To investigate the relationship between the model parameters and basic soil properties (i.e., plasticity index, clay fraction, and dry density);
- To improve an empirical relationship (Vanapalli and Lu 2012) which can be used to predict the swelling index, C_s from plasticity index, I_p;
- To propose an equation for predicting the variation of ground heave of expansive soils for varying degrees of saturation conditions;
- To validate the proposed swelling pressure and ground heave prediction techniques using the information of eight different case studies results from six different countries.

1.3 Background of the study

1.3.1 Literature review

A comprehensive review of the literature has been provided in the second chapter. Several aspects of expansive soil which include the mineralogy, swelling mechanism, soil suction components, volume change parameters, laboratory tests, and methods of ground heave prediction are discussed in this chapter.

1.3.2 Theoretical background

A school of thought has been extended during the last two decades by various investigators using the saturated soil property and the SWCC to predict the unsaturated soil properties. For example, Fredlund et al. (1994) proposed a method to predict the coefficient of permeability using the SWCC; Vanapalli et al. (1996) proposed a SWCC-based semi-empirical method to predict the shear strength; Oh et al. (2009) developed a SWCC-based semi-empirical method to predict the modulus of elasticity; Sheikhtaheri (2014) developed a SWCC-based semi-empirical method to predict the bearing capacity of single model piles; Han and Vanapalli (2015) developed a SWCC-based semi-empirical method to predict the resilient modulus.

The tests results of Zhan (2003), Agus (2005), Pedarla et al. (2012), and Pereira et al. (2012) shows that there is a strong relationship between the swelling pressure and initial soil suction. In addition, a strong relationship has also been observed between the SWCC and the variation of swelling pressure with respect to suction. These results were encouraging to propose a model to predict the variation of swelling pressure with respect to suction in this thesis using the SWCC as a tool along with other empirical equations that were briefly discussed earlier.

1.4 Novelty of the research

The test procedures associated with collecting volume change properties of expansive soil is cumbersome, expensive and time consuming. The swelling pressure data that is typically collected is limited to certain depths, which lack of representativeness to the entire active zone depth in which volume change behavior is typically predominant. The approach presented in the present thesis alleviates these limitations, as the model parameters in the proposed SWCC-based swelling pressure prediction model is dependent to the soil index properties, which are simple to gather. In words, this method can provide variation of the swelling with respect to the depth and the initial soil suction.

Furthermore, in the widely used oedometer test methods for estimating heave, the swelling pressure is conventionally measured by inundating the soil sample until there is no further tendency of swelling under fully saturated condition. However, natural expansive soils are most likely to be in a state of varying unsaturated condition based on the environmental factors. The ground heave estimation based on the oedometer test results provides only potential heave value at saturated conditions, which typically over estimates heave. The novelty of the present research is that the proposed model is capable to predict the variation of swelling pressure with respect to the initial soil suction. This information is useful to estimate the ground heave at different degrees of saturation (i.e. suction values).

1.5 Thesis layout

The present thesis consists of six chapters. These chapters are organized as follows:

This Chapter 1, “Introduction”, provides a general background information and ground heave behavior of expansive soils. The key objectives of the thesis are also highlighted providing details of how they will be addressed in the later chapters of the thesis. The novelty of the thesis is also discussed.

The Chapter 2, “Literature Review”, provides a state-of-the-art review on expansive soils. The focus of the chapter has been directed towards summarizing the background knowledge on the natural of expansive soils, the soil suction components, the volume change parameters, different laboratory tests, and the various methods available in the literature for ground heave estimation.

The Chapter 3, "Prediction of Volume Change Parameters", provides details of two SWCC-based models to predict the swelling pressure for both compacted and natural expansive soils. In addition, an empirical equation modified from the original contribution of Vanapalli and Lu (2012) is suggested to estimate the swelling index from the plasticity index.

The Chapter 4, "Ground Heave Prediction for Unsaturated Expansive Soils", provides an equation that has been developed to estimate the ground heave of unsaturated soil at different degrees of saturation (i.e. suction values) using the information of swelling pressure and swelling index. This equation is modified from the equation that was originally suggested by Fredlund (1983) for saturated soils.

The Chapter 5, "Validation of Proposed Techniques: Case Studies", summarizes the key details of all the case studies information available in the literature with respect ground heave in expansive soils. The proposed swelling pressure and ground heave prediction techniques in this thesis are validated on different case studies by comparing with the measured values. There is a good comparison between measured and predicted values for all the case studies results.

The Chapter 6, "Summary and Conclusions", succinctly summarizes the research undertaken through this thesis highlighting key conclusions. In addition, recommendations for future studies are also provided.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Upon wetting, expansive soils exhibit either ground heave or “swelling pressure”, or a combination of both. Essentially, the “swelling pressure” develops when attempt is made to prevent the expansive soil from swelling. Magnitude of the swelling pressures changes with respect to the volume change (or heave) process. A maximum value of “swelling pressure” can be expected when the soil undergoing wetting and reaches saturation condition under a constant volume condition. Termed as swelling pressure, P_s, this maximum value is defined as one of the volume change parameters for describing the swell potential of expansive soils. Using the information of swelling pressure, P_s, the ground heave can be predicted based on the consolidation-swell curve determined from oedometer tests (Sullivan and McClelland 1969; Fredund 1983; Nelson et al. 1998). In addition to the methodologies based on the determination of swelling pressure using oedometer tests, numerous other methods are available to predict the ground heave based on the constitutive relationships between the void ratio and soil suction. For example, various testing procedures have been suggested in the literature to determine the volume change parameters in terms of soil suction (Lytton 1977a,b; Johnson and Snethen 1978; Fredlund 1979; McKeen 1985). A state-of-the-art review of the testing procedures on the volume change parameters (e.g., swelling pressure, swelling index, suction compression index, and other parameters) and the various ground heave prediction methods are summarized in this chapter. In addition, as the depth of active zone and the depth of crack are of great significance to the magnitude of ground heave, a comprehensive summary on the relevant determination (or prediction) methods are provided in the present chapter. For the sake of completeness, three key aspects related to the natural expansive soil, which include about their origin, clay minerals, and swelling mechanism are also discussed in this chapter.

2.2 Nature of expansive soil

2.2.1 Origin

Donaldson (1969) classified the parent materials associated with the expansive soils into two groups, namely:

(i) The basic igneous rocks, in which the feldspar and pyroxene minerals can be decomposed to form montmorillonite and other secondary minerals;
(ii) The sedimentary rocks that contain montmorillonite as one of constituents break down physically to form expansive soils.

Tourtelot (1973) investigated the paleogeographic condition in the Rocky Mountain and Great Plains regions of the United States. It was believed that the montmorillonite was probably formed from two separate origins; namely, the products of weathering and erosion of the rocks, and the ash generated by the volcanic eruptions.

Snethen et al. (1975) believes that the origin and distribution of expansive materials are generally a function of the geological history, sedimentation, and present local climatic conditions. The following conditions, either individually or in combination, were regarded as the sources of the formation or origin of expansive materials: (a) weathering, (b) diagenetic alteration of pre-existing minerals, and (c) hydrothermal alteration.

As the most important source of clay formation, weathering process is considered following three different mechanisms: (a) inheritance, (b) neo-formation, and (c) transformation (Eberl 1984). These reactions are typically characterized by ion exchange with the surrounding environment and/or layer transformation in which the structure of octahedral, tetrahedral, or fixed interlayer cations is modified (Mitchell and Soga 2005).

2.2.2 Clay minerals

Most soil classification systems arbitrarily define *clay particles* as having an effective diameter of two microns (0.002 mm) or less (Chen 1975). The term *clay mineral* is also difficult to define precisely; Mackenzie (1963) suggested that the clay minerals are those which are typically found in the colloidal fraction in both soils and rocks (Mackenzie and Mitchell 1966).

Snethen et al. (1975) classified the clay minerals as follows:

(i) Two-layer clays consist of one silica tetrahedral layer bonded to one aluminum octahedral layer. Kaolinite is the common mineral, in which the octahedral layer contains mainly aluminum; serpentine consists of a magnesium-rich octahedral layer.
(ii) Three-layer clays have one octahedral layer bonded between two tetrahedral layers; examples of this type are illite, vermiculite, and montmorillonite.
(iii) Mixed-layer clays consist of interstratifications of the two- and three-layer clay minerals previously described. The mixing may be regular or random. Examples of regular mixing include chlorite and montmorillonite-chlorite.

The three most important clay minerals are montmorillonite, illite, and kaolinite, which are crystalline hydrous aluminosilicates (Chen 1975). Generally, kaolinite is considered to be inert with respect to volume change, while montmorillonite contributes to most of the expansive soil problems. The expansion contributed by illite is also not negligible. Table 1.1 presents the typical values of free swell strain for common clay minerals.

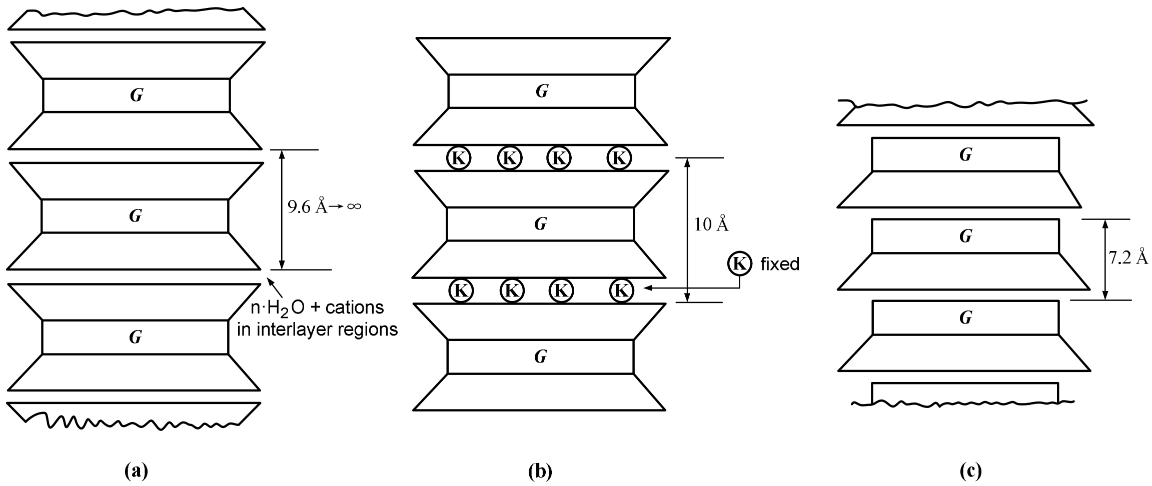

Figure 2.1 Schematic diagram of the structure of montmorillonite, illite, and kaolinite: (a) montmorillonite, (b) illite, and (c) kaolinite (after Mitchell and Soga 2005)

Table 2.1 Typical Values of Free Swell for Common Clay Minerals

Clay Mineral	**Free Swell, %**	
	Grim (1962)	Shamburger et al. (1975)
Na-montmorillonite	1400−2000	1400−1600
Ca-montmorillonite	45−145	65−145
Illite	60−120	60−120
Kaolinite	5−60	5−60

2.2.3 Swelling mechanism

The swelling mechanism of expansive soil has been widely discussed in the literature (Bolt 1956; Grim 1962; Grim 1968; Gillot 1968; Chen 1975; Snethen et al. 1975; Taylor and Smith 1986; Schafer and Singer 1976; Nelson and Miller 1992; Stavridakis 2006). Bolt (1956) conventionally sub-divided swelling processes into (i) mechanical, and (ii) physico-chemical.

Mechanical swelling occurs in response to elastic and time-dependent stress unloading, which in practice can be brought about by digging excavations, tectonic uplift or erosion (Taylor and Smith 1986). The physico-chemical swelling can be classified into crystalline swelling driving by the hydration energy, and osmotic swelling associated with the electrical double layer effects.

Crystalline Swelling

Crystalline swelling is resulted from a short-range hydration that the expansive clay minerals sequentially intercalate one, two, three, or four discrete layers of water between the mineral interlayers (Likos 2004). Newman (1987) and van Olphen (1991) termed this process as "Type I" swelling, which occurs prior to osmotic, or "Type II" swelling associated with longer-range electrical double layer effects.

Crystalline swelling is driven primarily by the energy associated with the initial hydration of exchangeable interlayer cations and hydrogen bonding or charged surface-dipole attraction effects associated with solid-liquid interactions occurring in the immediate vicinity of the clay particle surfaces (Likos 2004). The conceptual model of the sequential crystalline swelling process for montmorillonite is illustrated in Figure 2.2.

When the clay sheet interlayer distance exceeds 4 water molecules (about 10 Å), the forces associated with hydration of cations become insignificant compared with the forces of electrostatic repulsion between adjacent plates. In this case, further swelling is described as "osmotic swelling", and the distance between plates might increase until the plates become completely dissociated (Newman 1987).

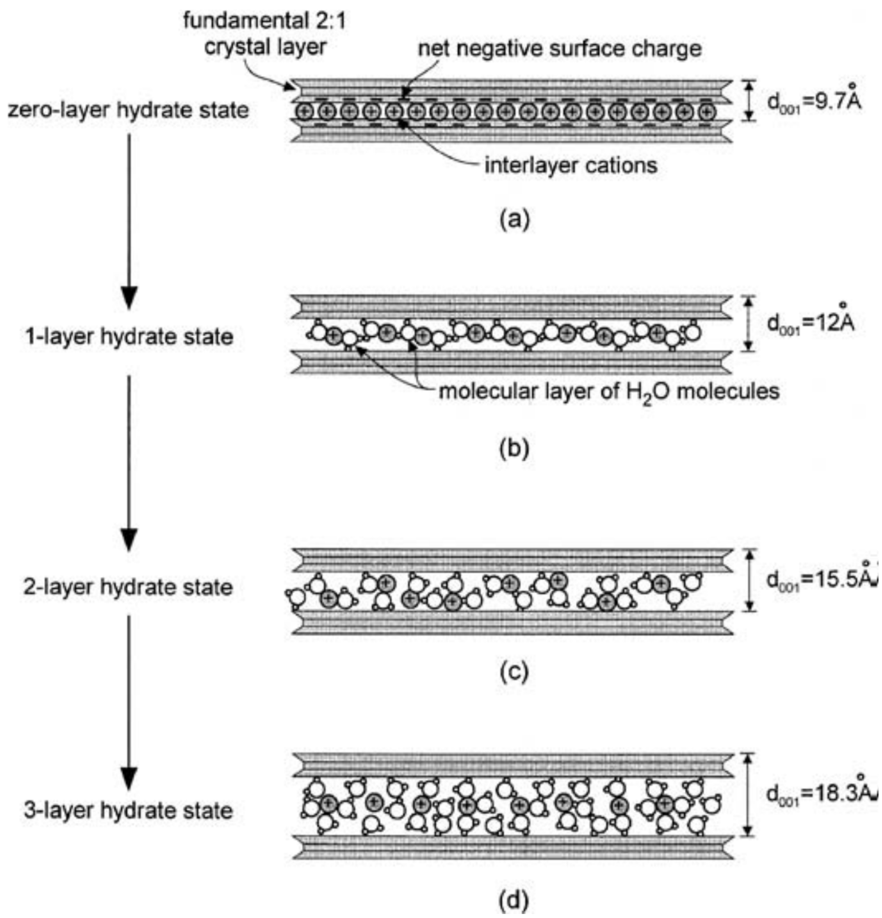

Figure 2.2 Conceptual model of the sequential crystalline swelling process for montmorillonite (from Likos 2004)

Osmotic Swelling

Montmorillonite is composed of a lattice of Al_2O_3 and SiO_2 units; a net negative charge develops on the clay surface when divalent metals such as magnesium substitute in the lattice for aluminum or silicon (Greathouse et al. 1994). In the presence of an aqueous electrolyte solution, the negatively charged mineral surfaces attract cations and polar water, forming a double-layer system (see Figure. 2.3). The overlap of two double layers results in a repulsive force pushing the clay platelets apart; it also causes an excess cation concentration between the platelets, consequently, free water has to be drawn into the system to restore equilibrium. It was believed that the expansion in Na-montmorillonite, and to a lesser extent Ca-montmorillonite, clays is governed by double-layer swelling. Bolt and Miller (1955), Warkentin et al. (1957), Mitchell (1960), Mesri and Olson (1971), Sridharan and Jayadeva (1982), Greathouse et al. (1994), Tripathy et al. (2004) are among those who have contributed to the better understanding of the mechanism controlling the volume change behaviour from the viewpoint of double layer repulsive forces.

Compared with montmorillonite, illite shows an intermediate response regarding the double-layer structure. While the volume change of kaolinite is purely a mechanical unloading phenomenon (Taylor and Smith 1986).

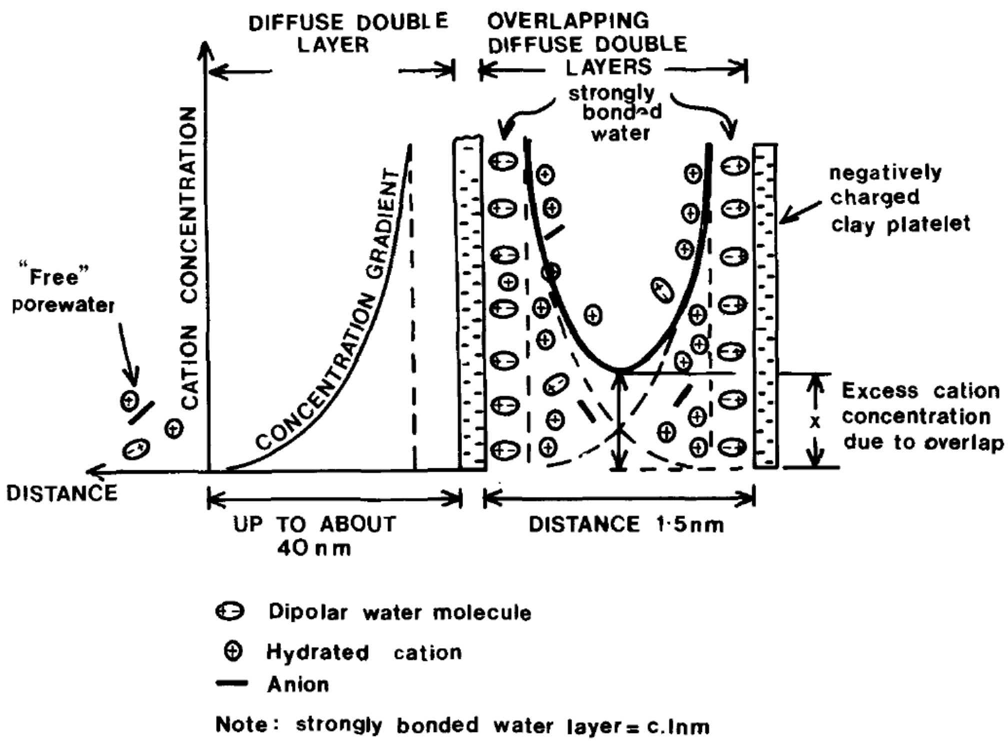

Figure 2.3 Model of double-layer (osmotic) swelling of two clay mineral platelets (from Taylor and Smith 1986)

2.3 Soil suction

The concept of soil suction was originally developed in soil physics in 1900's (Buckingham 1907; Gardner and Widtsoe 1921; Richards 1928). Soil suction is commonly referred to as the free energy state of soil water (Edlefsen and Anderson 1943). This energy of soil water is a result of the affinity that a soil has for water, and can be measured in terms of the pore-water vapor (or relative humidity) (Aitchison 1965; Richards 1965). Termed as total suction, ψ, it can be calculated using Kelvin equation based on thermodynamic principles (Richards 1965).

$$\psi = -\frac{RT}{\upsilon_{w0}\omega_{\upsilon}} \ln\left(\frac{\bar{u}_{\upsilon}}{\bar{u}_{\upsilon 0}}\right) \tag{2.1}$$

where,

ψ = soil suction or total suction, R = universal (molar) gas constant (i.e., 8.31432 J/mol/K), T = absolute temperature [i.e., $T = (273.16 + t^{\circ})$ (K)], t° = temperature (℃), υ_{w0} =specific volume of water or the inverse of the density [i.e., $1 / \rho_w$ (m^3 / kg)], ρ_w = density of water (i.e., 998 kg / m^3 at $t^{\circ} = 20$ ℃), ω_{υ} = molecular mass of water vapor (i.e., 18.016 kg / kmol), $\bar{u}_{\upsilon}$ = partial pressure of pore-water vapor (kPa), $\bar{u}_{\upsilon 0}$ = saturation pressure of water vapor over a flat surface of pure water at the same temperature (kPa), the term $\bar{u}_{\upsilon} / \bar{u}_{\upsilon 0}$ is called relative humidity, R_h.

The total suction has two components which are matric suction associated with the capillary phenomenon on the air-water interface and osmotic suction π associated to the agent dissolved in the soil water.

$$\psi = (u_a - u_w) + \pi \tag{2.2}$$

where, $(u_a - u_w)$ = matric suction, u_a = pore air pressure, and u_w = pore water pressure, π = osmotic suction.

2.3.1 Matric suction

Aitchison (1965) provided quantitative definitions for soil suction and its components. This definition has been quoted by the International Society of Soil Science (Krahn and Fredlund 1971). The matric suction component was defined as below:

"Matric or capillary component of free energy in suction terms, it is the equivalent suction derived from the measurement of the partial pressure of the water vapor in equilibrium with the soil water, relative to the partial pressure of the water vapor in equilibrium with a solution identical in composition with the soil water."

The matric suction component of total suction is associated with the capillary phenomenon on the air-water interface. In some scenarios, the matric suction is also termed as "capillary pressure". Figure 2.4 illustrates the physical model and phenomenon related to capillarity.

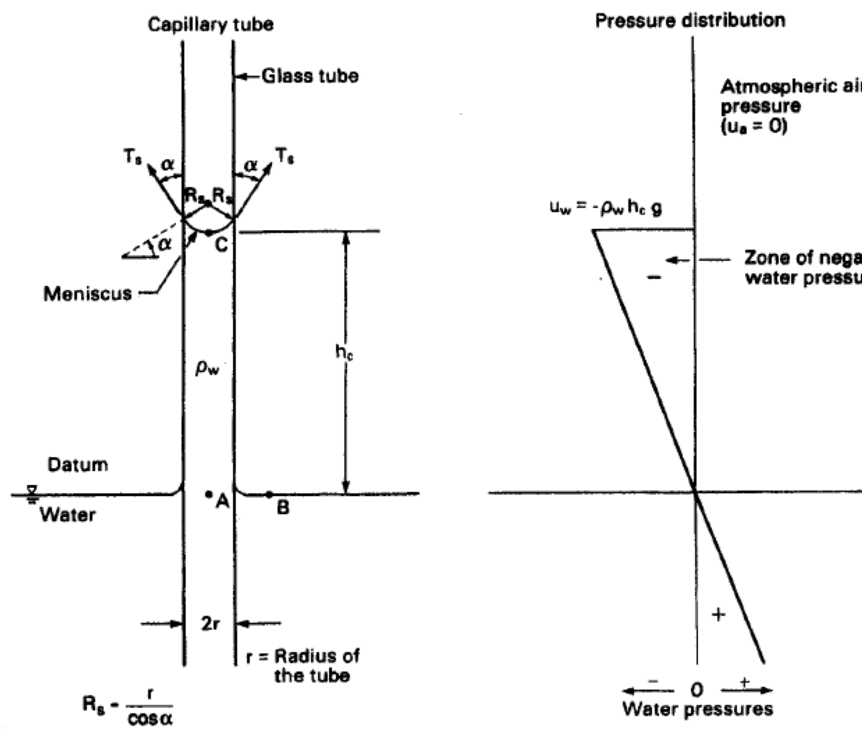

Figure 2.4 Physical model and phenomenon related to capillarity (from Fredlund and Rahardjo 1993)

As it is shown in Figure 2.4, the capillary phenomenon is attributed to the surface tension of water arising from the unbalanced molecular interactions on the air-water interface (or contractile skin). The Young-Laplace equation presents the relationship between the pressure difference, $(u_a - u_w)$ across the air-water interface and the surface tension, T_s acting on the contractile skin.

$$\left(u_a - u_w\right) = T_s\left(\frac{1}{R_1} + \frac{1}{R_2}\right) = T_s\frac{1}{R_m} \tag{2.3}$$

where, R_1 and R_2 = the principal radii of curvature of the interface (Figure 2.5), and $(R_1^{-1} + R_2^{-1})^{-1}$ = the first or mean radius of curvature, R_m.

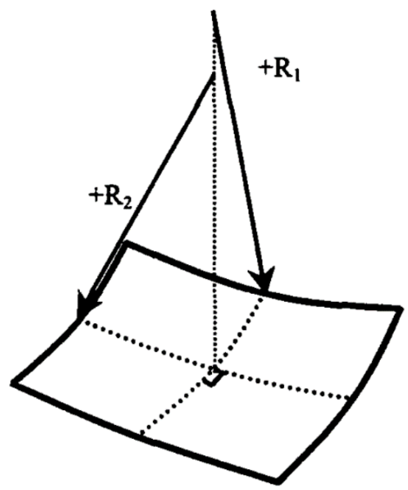

Figure 2.5 Schematic diagram of principal radii of the contractile skin (from Wang and Fredlund 2003)

The terms used for radii in Equation (2.3) are dependent on the geometries for the air-water interface. If the interface is a sphere, the R_m value is equal to $R/2$ (where, R = radius of the sphere); while in the case of a saddle-type meniscus, R_1 is equal to $-R_2$. Figure 2.6 illustrates some simplified geometries for the air-water interface that might form in an unsaturated soil matrix.

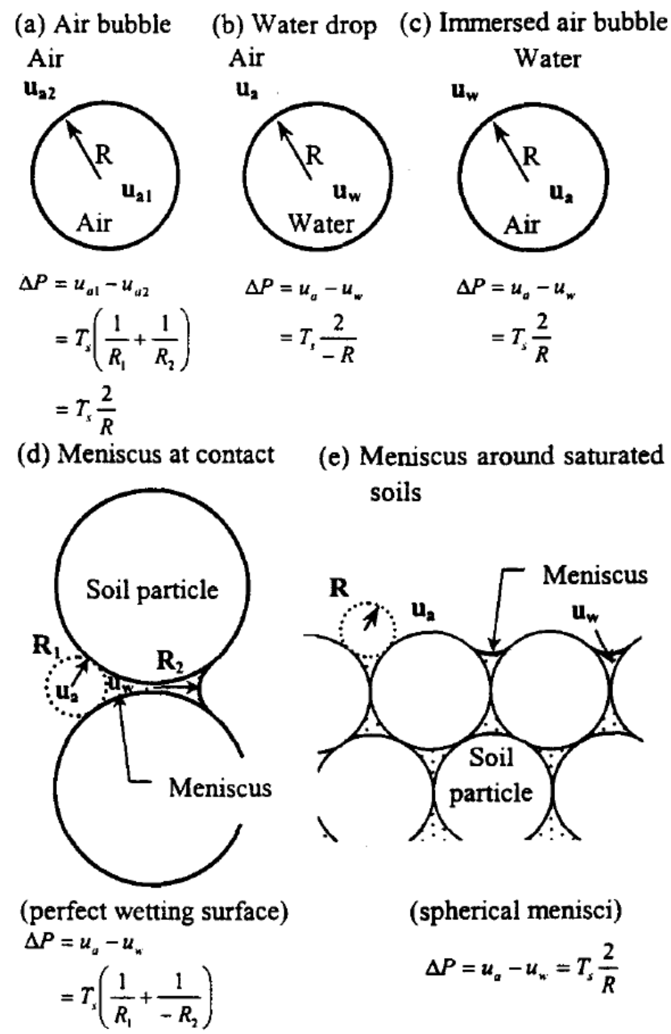

Figure 2.6 diagram of simplified geometry of the air-water interface and associated pressure difference ΔP across the interface based on the Young-Laplace equation (from Wang and Fredlund 2003)

2.3.2 Osmotic suction

Aitchison (1965) provided the definition of osmotic suction component as below:

> "Osmotic (or solute) component of free energy in suction terms, it is the equivalent suction derived from the measurement of the partial pressure of the water vapor in equilibrium with a solution identical in composition with the soil water, relative to the partial pressure of water vapor in equilibrium with free pure water."

The pore-water in soil mass is typically dissolved with salts, contributing a difference between the water vapor pressure over a surface of solvent, $\bar{u}_{vs}$ and that over a surface of pure water, $\bar{u}_{v0}$ ($\bar{u}_{vs} < \bar{u}_{v0}$). The increase of the solution concentration results in a reduction in the $\bar{u}_{vs}$ value, hence leads to a decrease in the relative humidity, R_h. As a consequence, there will be an increase in the total suction. The portion of total suction associated with the dissolved salts is termed as osmotic suction (π).

van't Hoff law describes that substances in dilute solution obey the ideal gas laws. The magnitude of the osmotic pressure (suction) can be computed using the equation given below:

$$\pi = \frac{n}{V}RT = C_i RT \tag{2.4}$$

where R = the gas constant, T = the absolute temperature, n = the number of mols of solute dissolved in the volume V of the solution, and C_i (= n/V) = the molar concentration of solute i in dilute solution (Feher and Ford 1995).

Based on the experimental research, Miller and Nelson (2006) reached a conclusion that the total suction of the NaCl-amended specimens was dominated by the osmotic component, indicating that π has a much higher magnitude than ($u_a - u_w$) at the NaCl concentrations tested. Whereas, it was recognized by many researchers that the presence of osmotic suction has minor influence on the mechanical behavior of unsaturated soils when compared with the matric suction component (van Genuchten 1980, Alonso et al. 1990, Vanapalli et al. 1996, Rassam and Williams 1999, Rampino et al. 2000, Vanapalli and Oh 2010).

Due to the existence of osmotic component, it is inadequate to use the total suction measurements as a proxy for estimating changes in matric suction. Otherwise, it may be problematic when comparing soil properties (e.g., suction compressibility) between different soils, which possesses various magnitude of osmotic suction (Miller and Nelson 2006). Krahn and Fredlund (1972) suggested to assume the osmotic suction as a constant value and to be subtracted from the measured total suction to estimate matric suction.

2.3.3 Soil-water characteristic curve

A soil-water characteristic curve (SWCC) describes the relationship between water content and soil suction for a single soil specimen (Fredlund and Rahardjo 1993). The amount of water in the soil is generally quantified in terms of gravimetric water content, w, volumetric water content, θ, or degree of saturation S. Many alternative terminologies are available in the literature and are widely used for representing the same meaning of soil-water characteristic curve; which include, *water retention curve*, *soil moisture curve*, *soil water retention curve*, *soil water characteristic*, and numerous other terms (Fredlund et al. 2001). Different graphical representations can be used for the SWCC data. In the literature, soil suction has been plotted either on the abscissa or on the ordinate, in either logarithmic or arithmetic scale.

Figure 2.7 summarizes some of the key features on the drying curve of the SWCC:

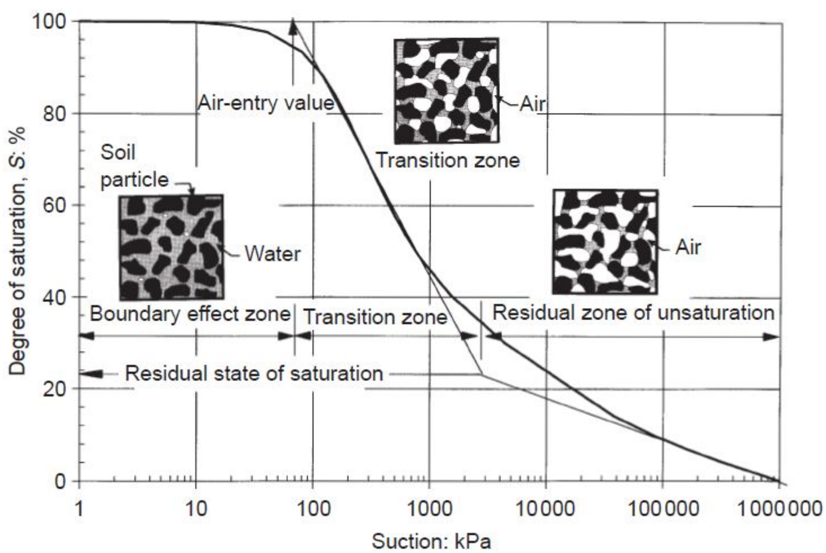

Figure 2.7 Typical unimodal soil-water characteristic curve (from Vanapalli et al. 1999)

Air-entry value (ψ_a):

The air-entry value of the soil (i.e., bubbling pressure) is the matric suction where air starts to enter the largest pores in the soil (McWhorter and Sunada 1977; Corey 1994; Fredlund and Xing 1994).

Residual water content (w_r):

The residual water content is the water content where a large suction change is required to remove additional water from the soil (McWhorter and Sunada 1977; Corey 1994; Fredlund and Xing 1994), the suction value corresponding to the residual water content is referred as *residual suction* (ψ_r).

Boundary effect zone:

The boundary effect zone is the zone located within the suction range of 0 to ψ_a. In the boundary effect zone, the soil is essentially in a state of saturated condition (Vanapalli et al. 1996).

Transition zone:

The transition zone is the zone located within the suction range of ψ_a to ψ_r. In the transition zone, the water content in the soil starts to reduce significantly with increasing suction and the amount of water at the soil particle or aggregate contacts reduces as desaturation continues (Vanapalli et al. 1996).

Residual zone:

The residual zone is initiated at a suction value greater than ψ_r, and extends up to 10^6 kPa. In this zone, large increases in suction lead to a relatively small change in water content (or degree of saturation). The amount of water loss in the liquid phase in this stage is small, since that the water menisci is small (Vanapalli et al. 1996).

2.3.3.1 Hysteresis effect

The SWCC measured from desorption process is termed as *drying curve*; likewise, the SWCC measured from an adsorption process is termed as *wetting curve*. As it is depicted in Figure 2.8, soil exhibits a hysteresis during the drying and wetting cycles. The drying curve of SWCC is typically located over the wetting curve.

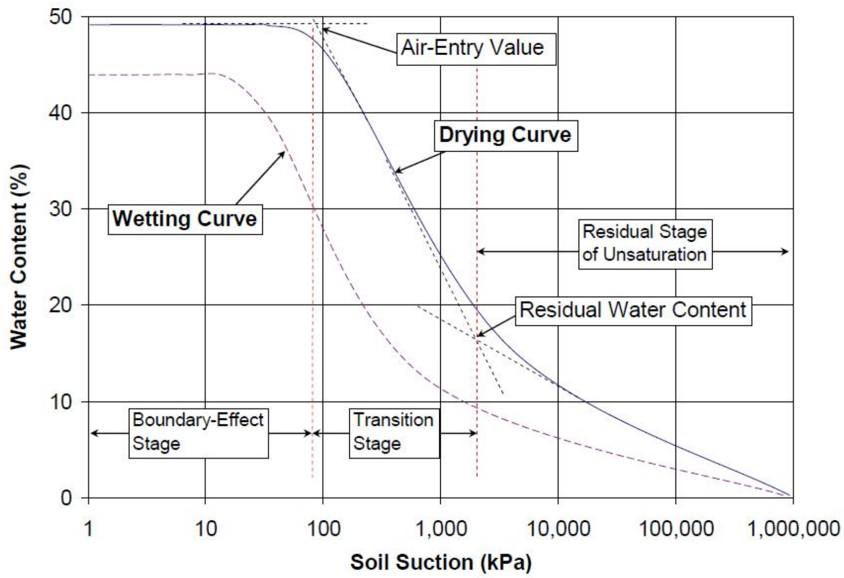

Figure 2.8 Typical drying and wetting curves of the SWCC

The non-uniform pore-size distribution in the soil results in different height of the capillary rise during the drying and wetting processes; the contact angle at an advancing interface during the wetting process is different from that at a drying interface during the drying process (Bear 1979); in addition, entrapped air are presented in the soil (Fredlund and Rahardjo 1993). The above factors are considered as the major causes of the hysteresis in the SWCC behavior.

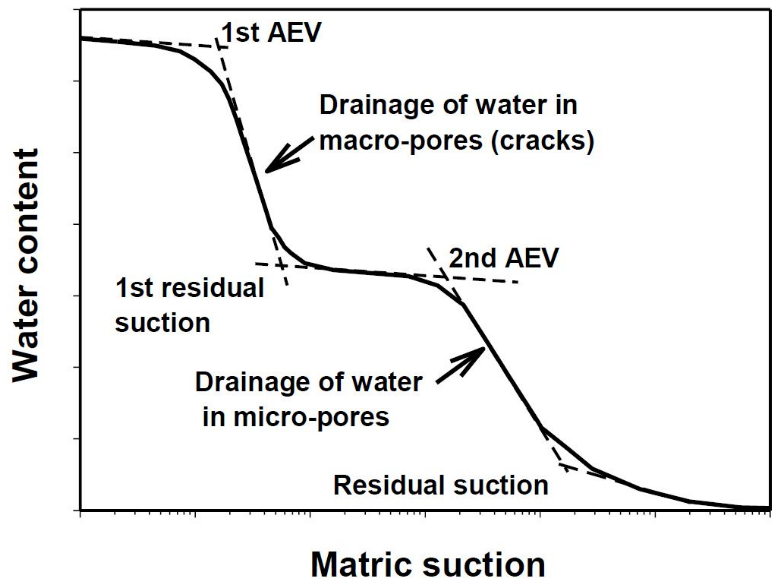

Figure 2.9 Typical bimodal SWCC (from Qi and Vanapalli 2015)

2.3.3.2 Fitting equations

In the literature, many mathematical models have been proposed to best-fit the experimental soil-water characteristic data (e.g., Gardner 1958; Brooks and Corey 1964; Mualem 1976; van Genuchten 1980; Fredlund and Xing 1994; Leong and Rahardjo 1997). In addition, some empirical approaches have been proposed to predict the SWCCs based on the pore size distribution and the volume-mass relationships (e.g., Gupta and Larson 1979; Fredlund et al. 1997). These models, however, are only applicable for representing the unimodal SWCCs (i.e., with two bending curves only), which are associated with soil samples with one level of pore size distribution. When two or more levels of pore size distribution exist in the soil, the corresponding SWCCs can be bimodal or multimodal (Zhang and Chen 2005). Bimodal SWCCs are typically observed in structured soils such as the aggregated loam or soils with cracks (Smettem and Kirby 1990; Wilson et al. 1992; Durner 1994; Mallants et al. 1997; Abbaszadeh et al. 2010; Elkady 2014). A typical shape of bimodal SWCC is shown in Figure 2.9.

Numerous SWCC fitting equations are proposed by different researchers. Table 2.2 provides a comprehensive summary of these equations from the literature.

Table 2.2 Equations Used to Best-Fit SWCC Data (expanded from Fredlund et al. 2013)

Reference	Equation	Description	Natural of SWCC	
Gardner (1958)	$\Theta_d = \dfrac{1}{1+a_g\psi^{n_g}}$ where, $\Theta_d = \dfrac{w(\psi)}{w_s}$	a_g = fitting parameter which is a function of air-entry value of the soil n_g = fitting parameter which is a function of rate of water extraction from soil once air-entry value of soil has been exceeded	unimodal	(2.5)
Brooks and Corey (1964)	$w(\psi) = w_s$ or $\Theta_n = 1$ for $\psi \le \psi_{\text{aev}}$ $\Theta_n = \left[\dfrac{\psi}{\psi_{\text{aev}}}\right]^{-\lambda_{bc}} \quad \psi > \psi_{\text{aev}}$ where, $\Theta_n = \dfrac{w(\psi)-w_r}{w_s - w_r}$	ψ_{aev} = air-entry value of soil λ_{bc} = pore size distribution index w_r = residual water content located through trial-and-error process that yields straight line on semi log plot of degree of saturation versus suction	unimodal	(2.6)
Brutsaert (1967)	$\Theta_n = \dfrac{1}{1+[\psi/a_b]^{n_b}}$ where, $\Theta_n = \dfrac{w(\psi)-w_r}{w_s - w_r}$	a_b = fitting parameter which is a function of air-entry value of soil n_b = fitting parameter which is a function of rate of water extraction from soil once air-entry value has been exceeded	unimodal	(2.7)
Laliberte (1969)	$\Theta_n = \dfrac{1}{2}\text{erfc}\left[a_l - \dfrac{b_l}{c_l + (\psi/\psi_{\text{aev}})}\right]$ where,	a_l, b_l, c_l = parameters assumed to be unique functions of pore-size distribution index λ	unimodal	(2.8)

	$\Theta_n = \dfrac{w(\psi) - w_r}{w_s - w_r}$			
Campbell (1974)	$w = w_s$, $\psi < \psi_{aev}$ $w = w_s \left[\dfrac{\psi}{\psi_{aev}} \right]^{-1/b_c}$, $\psi \geq \psi_{aev}$	ψ_{aev} = air-entry value of soil b_c = fitting parameter	unimodal	(2.9)
van Genuchten (1980)	$\Theta_n = \dfrac{1}{[1 + (a_{vg}\psi)^{n_{vg}}]^{m_{vg}}}$ where, $\Theta_n = \dfrac{w(\psi) - w_r}{w_s - w_r}$	a_{vg}, a_{vm}, a_{vb} = fitting parameters primarily related to inverse of air-entry value (units equal to 1/kPa) n_{vg}, n_{vm}, n_{vb} = fitting parameters primarily related to rate of water extraction from soil once air-entry value has been exceeded m_{vg}, m_{vm}, m_{vb} = fitting parameters that are primarily related to residual water content conditions	unimodal	(2.10)
van Genuchten (1980) – Mualem (1976)	$\Theta_n = \dfrac{1}{[1 + (a_{vm}\psi)^{n_{vm}}]^{m_{vm}}}$ where, $m_{vm} = 1 - \dfrac{1}{n_{vm}}$		unimodal	(2.11)
van Genuchten (1980) – Burdine (1953)	$\Theta_n = \dfrac{1}{[1 + (a_{vb}\psi)^{n_{vb}}]^{m_{vb}}}$ where, $m_{vb} = 1 - \dfrac{2}{n_{vb}}$		unimodal	(2.12)
McKee and Bumb (1984)	$\Theta_n = \exp\left[\dfrac{a_{m1} - \psi}{n_{m1}} \right]$ where,	a_{m1} = curve-fitting parameter n_{m1} = curve-fitting parameter	unimodal	(2.13)

(Boltzmann distribution)	$\Theta_n = \dfrac{w(\psi) - w_r}{w_s - w_r}$			
McKee and Bumb (1984) (Fermi distribution)	$\Theta_n = \dfrac{1}{1 + \exp[(\psi - a_{m2})/n_{m2}]}$ where, $\Theta_n = \dfrac{w(\psi) - w_r}{w_s - w_r}$	a_{m2} = curve-fitting parameter n_{m2} = curve-fitting parameter	unimodal	(2.14)
Fredlund and Xing (1994)	$w(\psi) = C(\psi)\dfrac{w_s}{\{\ln[e + (\psi / a_f)^{n_f}]\}^{m_f}}$ where, $C(\psi) = 1 - \dfrac{\ln(1 + \psi / \psi_r)}{\ln[1 + (10^6 / \psi_r)]}$ $\Theta_n = \dfrac{w(\psi)}{w_s}$	a_f = fitting parameter which is primarily a function of air-entry value of soil n_f = fitting parameter which is primarily a function of rate of water extraction from soil once air-entry value has been exceeded m_f = fitting parameter which is primarily a function of residual water content $C(\psi)$ = correction factor which is primarily a function of suction corresponding to residual water content.	unimodal	(2.15)
Pereira and Fredlund (2000)	$w(\psi) = w_r + \dfrac{w_s - w_r}{[1 + (\psi / a_p)^{n_p}]^{m_p}}$	a_p = fitting parameter which is primarily a function of air-entry value of soil n_p = fitting parameter which is primarily a function of rate of water extraction from soil, once air-entry value has been exceeded m_p = fitting parameter which is primarily a function of residual water content	unimodal	(2.16)

Burger and Shackelford (2001)	$\theta = \begin{cases} C(\psi)\dfrac{\theta_j}{\left\{\ln\left[e+\left(\dfrac{\psi}{a'}\right)^{n'}\right]\right\}^{m'}}; & \psi_j < \psi \\ C(\psi)\dfrac{\theta_s}{\left\{\ln\left[e+\left(\dfrac{\psi}{a}\right)^{n}\right]\right\}^{m}}; & \psi \le \psi_j \end{cases}$ where, $C(\psi) = \begin{cases} C(\psi) = 1-\dfrac{\ln\left[1+\left(\dfrac{\psi}{\psi'_r}\right)\right]}{\ln\left[1+\left(\dfrac{10^6}{\psi'_r}\right)\right]}; & \psi_j < \psi \\ C(\psi) = 1-\dfrac{\ln\left[1+\left(\dfrac{\psi}{\psi_r}\right)\right]}{\ln\left[1+\left(\dfrac{10^6}{\psi_r}\right)\right]}; & \psi \le \psi_j \end{cases}$	θ = volumetric water content θ_j = the junction volumetric water content θ_r = the residual volumetric water content θ_s = the saturated volumetric water content ψ = soil suction ψ_j = the soil suction at the junction point ψ_r and ψ'_r = the residual soil suctions for macroscopic, microscopic portions of data e = base of natural logarithm a, m, n = the fitting parameters for macroscopic portion of data a', m', n' = the fitting parameters for microscopic portion of data.	bimodal	(2.17)
de F. N. Gitirana Jr. and Fredlund (2004)	$S = \dfrac{S_1 - S_2}{1+\left(\psi/\sqrt{\psi_{b1}\psi_{res1}}\right)^{d1}} + \dfrac{S_2 - S_3}{1+\left(\psi/\sqrt{\psi_{res1}\psi_{b2}}\right)^{d2}}$ $+\dfrac{S_3 - S_4}{1+\left(\psi/\sqrt{\psi_{b2}\psi_{res2}}\right)^{d2}} + S_4$ $S_i = \dfrac{\tan\theta_i\left(1+r_i^2\right)\ln\left(\psi/\psi_i^a\right)}{\left(1-r_i^2\tan^2\theta_i\right)} + (-1)^i$ $\times\dfrac{\left(1+\tan^2\theta_i\right)}{\left(1-r_i^2\tan^2\theta_i\right)}\sqrt{r_i^2\ln^2\left(\psi/\psi_i^a\right)+\dfrac{a^2\left(1+\tan^2\theta_i\right)}{\left(1-r_i^2\tan^2\theta_i\right)}} + S_i^a$	i = 1, 2, 3, 4 $\theta_i = -(\lambda_{i-1}+\lambda_i)/2$, is hyperbolas rotation angles $r_i = \tan[(\lambda_{i-1}-\lambda_i)/2]$, is aperture angles tangents, $\lambda_0 = 0$, $\lambda_i = \arctan\{(S^a_i - S^a_{i+1})/[\ln(\psi^a_{i+1}/\psi^a_i)]\}$ = desaturation slopes, $S^a_1 = 1$, $S^a_2 = S_{res1}$, $S^a_3 = S_b$, $S^a_4 = S_{res2}$, $S^a_5 = 0$, $\psi^a_1 = \psi_b$, $\psi^a_2 = \psi_{res1}$, $\psi^a_3 = \psi_{b2}$, $\psi^a_4 = \psi_{res2}$, $\psi^a_5 = 10^6$, $d_j = 2\exp[1/\ln(\psi^a_{j+1}/\psi^a_j)]$ = weighting factors, j = 1, 2, 3.	bimodal	(2.18)

Pham and Fredlund (2005)	$\begin{cases} w_1(\psi) = w_u - S_1 \log(\psi) & 1 \le \psi < \psi_{\text{aev}} \\ w_2(\psi) = w_{\text{aev}} - S_2 \log\left(\dfrac{\psi}{\psi_{\text{aev}}}\right) & \psi_{\text{aev}} \le \psi < \psi_r \\ w_3(\psi) = S_3 \log\left(\dfrac{10^6}{\psi}\right) & \psi_r \le \psi < 10^6 \text{ kPa} \end{cases}$	S_1, S_2, S_3 = slope of straight line portions of SWCC within each of three zones w_u = water content at 1 kPa w_{aev} = water content at air-entry value w_1,w_2,w_3 = water content in line segments 1, 2, and 3, respectively.	unimodal	(2.19)
Zhang and Chen (2005)	$\theta(\psi) = p_l n_{pl}\left[1 - \dfrac{\ln\left(1+\dfrac{\psi}{\psi_{rl}}\right)}{\ln\left(1+\dfrac{10^6}{\psi_{rl}}\right)}\right]\left\{\dfrac{1}{\ln\left[e+\left(\dfrac{\psi}{a_l}\right)^{nl}\right]}\right\}^{ml}$ $+ p_s n_{ps}\left[1 - \dfrac{\ln\left(1+\dfrac{\psi}{\psi_{rs}}\right)}{\ln\left(1+\dfrac{10^6}{\psi_{rs}}\right)}\right]\left\{\dfrac{1}{\ln\left[e+\left(\dfrac{\psi}{a_s}\right)^{ns}\right]}\right\}^{ms}$ $\theta_s = \theta_{sl} + \theta_{ss}$; $\theta_{sl} = p_l\, n_{pl}$; $\theta_{ss} = p_s\, n_{ps}$;	e = the base of natural logarithm ψ_r = soil suction in the residual condition a, m, n = three parameters of the SWCC function subscripts l and s represent the large-pore series component and the small-pore series component, respectively.	bimodal	(2.20)

Satyanaga et al. (2013)	$\theta_w = \left(1-\frac{\ln\left(1+\frac{\psi}{\psi_r}\right)}{\ln\left(1+\frac{10^6}{\psi_r}\right)}\right)\left[\begin{array}{l}\theta_r \\ +\left(\theta_{s1}-\theta_{s2}\right)\left(1-erfc\frac{\ln\left(\frac{\psi_{a1}-\psi}{\psi_{a1}-\psi_{m1}}\right)}{s1}\right) \\ +\left(\theta_{s2}-\theta_r\right)\left(1-erfc\frac{\ln\left(\frac{\psi_{a1}-\psi}{\psi_{a1}-\psi_{m1}}\right)}{s1}\right)\end{array}\right]$ $s = \left(\theta_{s2}\right)\exp\sqrt{\frac{\sum_{i=1}^{n}\left(\ln\frac{\psi_i}{\mu}\right)}{n}}$ $\mu = \sqrt[n]{\psi_1\psi_2\cdots\psi_n}$ $erfc = \int_{-\infty}^{X}\frac{1}{\sqrt{2\pi}}\exp\left(-\frac{x^2}{2}\right)dx$	θ_w = the calculated volumetric water content θ_s = the saturated volumetric water content (measured in the laboratory), ψ = the matric suction under consideration ψ_a = the air-entry value of soil ψ_m = the matric suction at the inflection point of SWCC θ_r = the residual volumetric water content ψ_r = the matric suction corresponding to residual volumetric water content s = the geometric standard deviation of SWCC μ = the geometric mean of matric suction *erfc* = the complimentary error function	bimodal	(2.21)

Li et al. (2014)	$w(\psi)=\lambda\left(\frac{w_s}{\lambda+1}-w_r\right)\frac{\sqrt{\psi_t\psi_a}^{\,n/\log(\psi_t/\psi_a)}}{\psi^{n/\log(\psi_t/\psi_a)}+\sqrt{\psi_t\psi_a}^{\,n/\log(\psi_t/\psi_a)}}$ $+\left(\frac{w_s}{\lambda+1}-w_r\right)\frac{(l\psi_t)^m}{\psi^m+(l\psi_t)^m}+\lambda w_r$ $\times\frac{\sqrt{\psi_r\psi_{a2}}^{\,n/\log(\psi_r/\psi_{a2})}}{\psi^{n/\log(\psi_r/\psi_{a2})}+\sqrt{\psi_r\psi_{a2}}^{\,n/\log(\psi_t/\psi_{a2})}}+w_r\frac{(l\psi_r)^m}{\psi^m+(l\psi_r)^m}$	λ = the ratio of the mass content of bulk water to that of water lens, it can be calculated as the ratio *$(w_s$ - $w_r)/w_r$*, w_s is saturated water content, w_r is residual water content (i.e. mass content of water-lens stored in micro-pores of soil) w_b = mass content of bulky water ψ_a = air-entry value of a unimodal SWCC, or the first air-entry value of a bimodal or multimodal SWCC ψ_r = suction corresponding to the residual water content of a unimodal SWCC, or the second residual suction of a bimodal SWCC w_{b1} = mass content of bulky water stored in macro-pores w_{l1} = mass content of water lens stored in macro-pores w_{b2} = the mass content of bulky water stored in micro-pores ψ_{a2} = the air entry value for water stored in micro-pores ψ_t = the residual suction for water stored in macro-pores.	bimodal	(2.22)

2.4 Active zone and crack propagation

2.4.1 Active zone

The term "active zone" may have different meaning for different scenarios. Nelson et al. (2001) provided four definitions for clarity:

Active zone is the zone of soil that contributes to soil expansion at any particular time.

Zone of seasonal moisture fluctuation is the zone in which water content changes due to climatic changes at the ground surface.

Depth of wetting is the depth that water contents have reached owing to the introduction of water from external sources.

Depth of potential heave is the depth at which the overburden vertical stress equals or exceeds the swelling pressure of the soil.

The depth of potential heave can be easily obtained from the information of vertical soil stress; however, it results in a maximum estimate on the depth of active zone. As a consequence, the computation of ground heave based on this depth of active zone may contribute a considerably over-estimated value (Azam et al. 2013).

Since the movement of expansive soil is essentially due to the reaction of the variation of moisture content, *zone of seasonal moisture fluctuation* and *depth of wetting* are particularly important as they are used to estimate heave by integrating the strain produced over the zone in which water contents change (Nelson and Miller 1992; Nelson et al. 2001; Walsh et al. 2009; Jones and Jefferson 2012).

The *zone of seasonal moisture fluctuation* is the active zone corresponding to the influence of climatic changes, the depth of this zone are mostly discussed in the scenario that an open expansive soil ground surface is subjected to climatic factors, such as evapo-transpiration and temperature fluctuation.

On the other hand, the introduction of a soil cover or slab-on-grade significantly reduces the evaporation from the soil. For such a scenario, the depth of active zone is not predominately governed by the climatic changes, which are somehow isolated from the soil, but by the moisture migration process beneath the soil cover. Once steady condition is achieved, the depth to which the wetting front is advanced is regarded as the *depth of wetting*. The value of this parameter is essentially influenced by external factors such as irrigation system, long-lasting extensive rainfall infiltration, leaking of water pipe, or seepage from ponds or ditches (e.g., Ng et al. 2003; Yoshida et al. 1983).

The depth of active zone can be determined using four approaches: (i) long-term monitoring on water content or soil suction profiles, (ii) mathematical-physical approaches, (iii) empirical determinations and (iv) numerical simulations.

2.4.1.1 In-situ monitoring

The information of in-situ monitoring on the water content profile or soil suction profile can be used to determine the depth of active zone. Numerous investigations have been performed by different researchers to monitor the in-situ water content and soil suction. Various techniques (such as boring, filter paper, tensiometer, psychrometer, and etc.) are applicable over the monitoring period (Sweeney 1982; Nelson et al. 1994; Cameron 2001; Zhang et al. 2000; Jaksa et al. 2002; Ng et al. 2003; Fityus et al. 2004). The in-situ data of soil suction and water content have been illustrated in Figure 2.10 as examples.

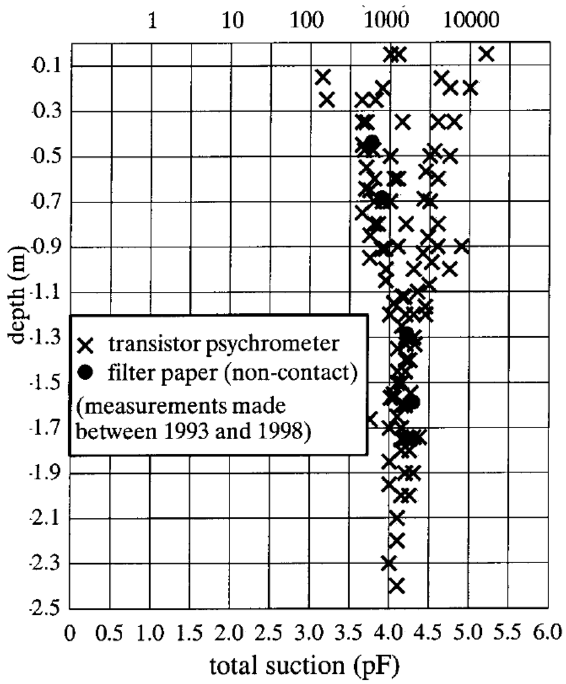

Figure 2.10 Profiles of total soil suction; (b) profiles of gravimetric water content (from Fityus et al. 2004)

Shi and Xu (1990) presented the field data over the period from 1975 to 1984, and suggested the depth of active zone as a location at which the amplitude of the variation of water content equals 1% of the amplitude of the variation of water content at the ground surface. PTI (2004) suggested the depth of zone as the location of the equilibrium moisture content, in uniform soil characterized by 0.027 pF (0.106 kPa) suction change per ft or to other conditions such as a cemented layer or water table. Considering the disunity of the soil strata, Nelson and Miller (1992) suggested to plot the variation of either water divided by plasticity index (w / I_p) or liquidity index [($w_l - w$) / I_p], rather than water content, with respect to depth (Figure 2.11).

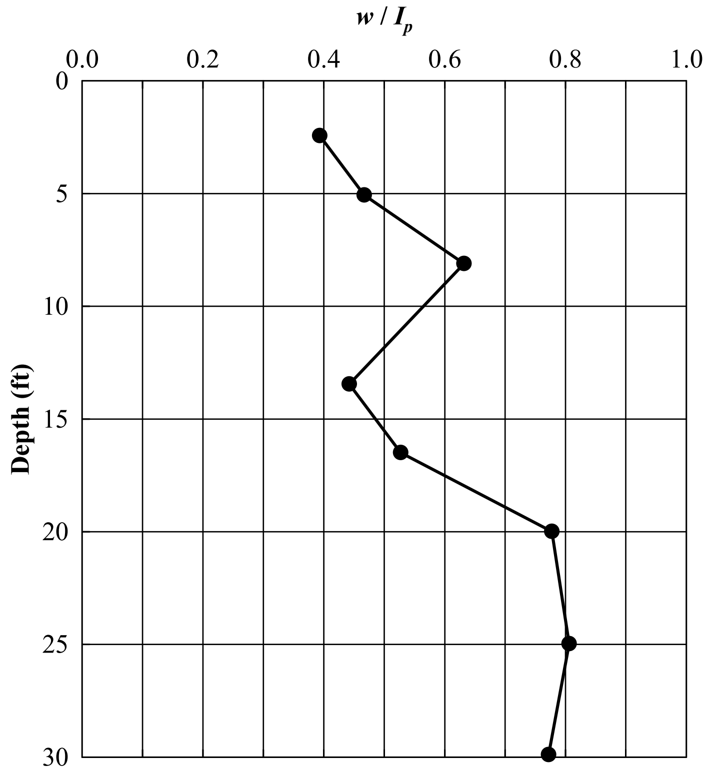

Figure 2.11 Estimation of active zone based on basic soil properties (after Nelson and Miller 1992)

2.4.1.2 Semi-empirical approach

Mitchell (1979) derived an analytical solution of a one-dimensional differential equation which explains the relation between the value of suction and the climatic changes:

$$u(y,t) = U_e - U_0 \exp\left\{-\left[\left(\frac{n\pi}{\alpha}\right)^{0.5}\right]y\right\}\cos\left\{2n\pi t - \left[\left(\frac{n\pi}{\alpha}\right)^{0.5}\right]y\right\} \tag{2.23}$$

where, $u(y,t)$ is the suction as a function of space, y, and time, t [pF (kPa)]; U_e is the equilibrium soil suction below the depth of active zone, U_0 is the amplitude of suction variation at the surface [pF (kPa)], n is the frequency of climate changing, α is the diffusion coefficient (cm^2/s), y is the space coordinate for depth [m (ft)], t is the time coordinate (days).

Note that Equation (2.23) was proposed based on certain assumptions: a) the soil suction at the ground surface presents sinusoidal variations, and b) the amplitude of soil suction exponentially decreases with depth. The essential characteristics of the results of Equation (2.23) are shown in Figure 2.12.

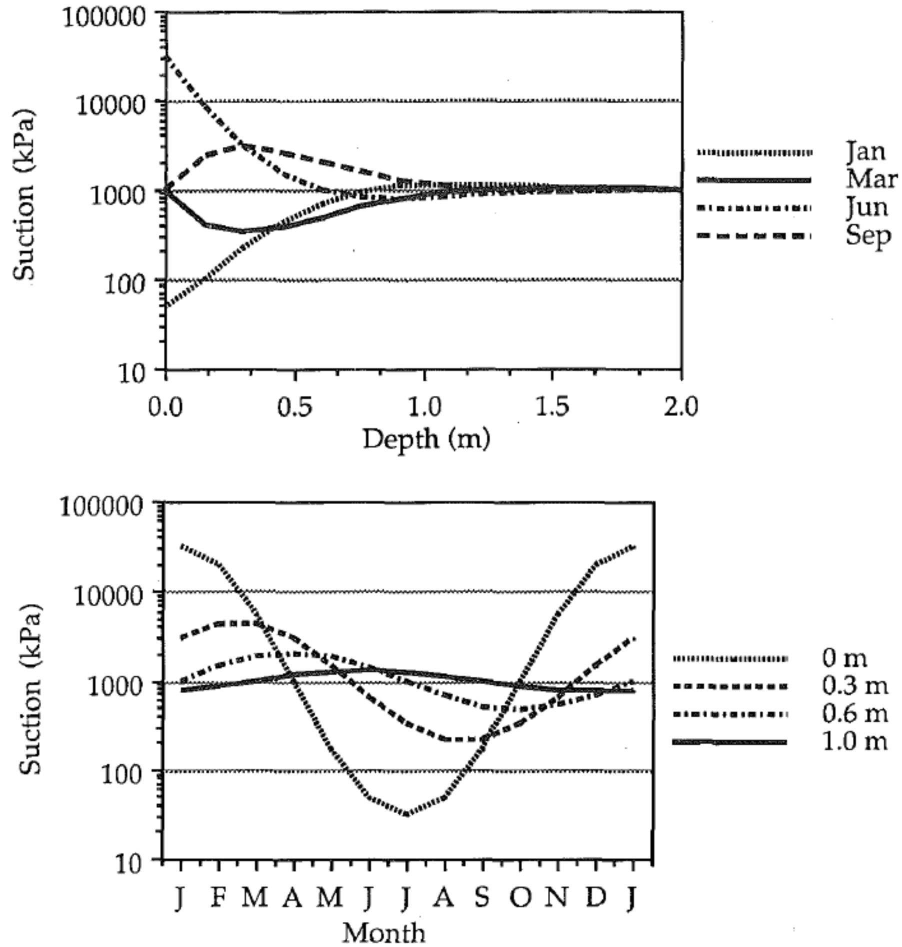

Figure 2.12 Examples of calculated suction variation: (a) calculated suction variation with depth; (b) calculated suction variation with time (from McKeen et al. 1990)

Considering the extremes (or envelope) of the suction profile, McKeen and Johnson (1990) derived an equation for estimating the zone of seasonal moisture fluctuation:

$$z_a = \frac{\ln(\frac{2U_0}{\Delta U_{\max}})}{\sqrt{\frac{n\pi}{\alpha}}} \tag{2.24}$$

where z_a is the depth of seasonal moisture fluctuation, $\Delta U_{\max}$ is the maximum suction change, below which movement is considered insignificant (i.e., allowable suction change).

The values of four parameters (i.e., U_0, $\Delta U_{\max}$, n, α) are needed to estimate the depth of active zone using Equation (2.24). Analysing the database established for Dallas/Ft. Worth locations (McKeen 1981, 1985), McKeen and Johnson (1990) suggested an allowable suction change, $\Delta U_{\max}$=0.4 pF. The $\Delta U_{\max}$ parameter of this value was applied to estimate the depth of active zone at a Chinese location (Ankang, Shanxi); the estimate shows a good agreement with the observation (Pan et al. 2006). Briaud et al. (2003) suggested to adopt 10% of the amplitude of suction change at the ground surface (i.e., $\Delta U_{\max} = 0.1\times 2U_0$). It is believed that the U_0 and $\Delta U_{\max}$ parameters can also be achieved from the long-term in-situ monitoring data (Briaud et al. 2003).

Rigorous determination of the frequency of climate changes, n, and the diffusion coefficient, α need long-term observations and complex testing procedures. Thornthwaite moisture index (TMI) along with other soil parameters such as inverse moisture characteristic (dh/dw) and suction compression index (γ_h) were suggested to be used for empirical determination of the n and α values (McKeen and Johnson 1990; Lytton 1994).

Thornthwaite moisture index was originally defined by Thornthwaite (1948) to characterize the cyclic nature of climatic wetting and drying of soil. The TMI value for year y can be computed form local climatic data using Equation (2.25).

$$TMI = \frac{100(R_y) - 60(DF_y)}{PE_y} \tag{2.25}$$

where (PE_y) is potential evapotranspiration, (DF_y) is deficit, (R_y) is run off.

High positive TMI values indicate net drainage with no significant desiccation (i.e., wet or humid zone), whereas high negative TMI values, the reverse (i.e., arid zone) (Russam and Coleman 1961). Aitchison and Richards (1965) found a rough correlation between the upper limit of the equilibrium of total soil suction under covered areas and this index.

Lytton (1994) recommended the frequency number, $n = 1$ when the TMI value is less than −30, to take the frequency number, $n = 2$, for the TMI values greater than −30 (Wray et al. 2005).

Besides, McKeen and Johnson (1990) proposed a multiple linear relation between diffusion coefficient, α and three soil parameters (TMI, dh/dw, and γ_h). This empirical relation (Equation 2.26) was validated on 6 sets of data from different locations in US (McKeen and Johnson 1990).

$$\alpha = b_0 + b_1(TMI) + b_2\left(\frac{dh}{dw}\right) + b_3(\gamma_h) \tag{2.26}$$

where, b_0, b_1, b_2, and b_3 are fitting parameters.

2.4.1.3 Numerical simulation

Several commercial software packages are available to model the moisture flow in soils, including SoilCover (Unsaturated Soils Group 1996), HYDRUS (2D/3D) (Šimůnek et al. 2012), VADOSE/W (Geo-Slope 2014), SVFlux (SoilVision 2009) and FlexPDE (PDE Solutions 2015).

These software packages have been utilized by several researchers to determine the depth of active zone. For example, SVFlux and FlexPDE were used by Vu et al. (2007) to analyse the soil suction variation in Regina clay; VADOSE/W was used by Overton et al. (2006) to determine the depth of wetting; and HYDRUS package was used by Twarakavi et al. (2008) to estimate the interaction between groundwater and vadose zone (i.e., depth of wetting).

2.4.2 Desiccation cracking

The performance of expansive soil is significantly influenced by the existence of cracks. Such influence is mostly imposed on two aspects: swell potential and water flow (depth of wetting). Both the swelling pressure and percent swell of the soil will reduce due to the introducing of cracks; saturated hydraulic conductivity (k_{sat}) for cracks is dramatically higher than that for the intact soil, the unsaturated hydraulic conductivity of the soil however is not substantially influenced by the cracks since it heals during the wetting process (Abbaszadeh 2011).

Irwin (1958) classified cracks into to three types; which includes opening mode (mode I), sliding mode (mode II), and tearing mode (mode III). The mode I crack (Figure 2.13) are mostly discussed for understanding the crack propagation in desiccated soils.

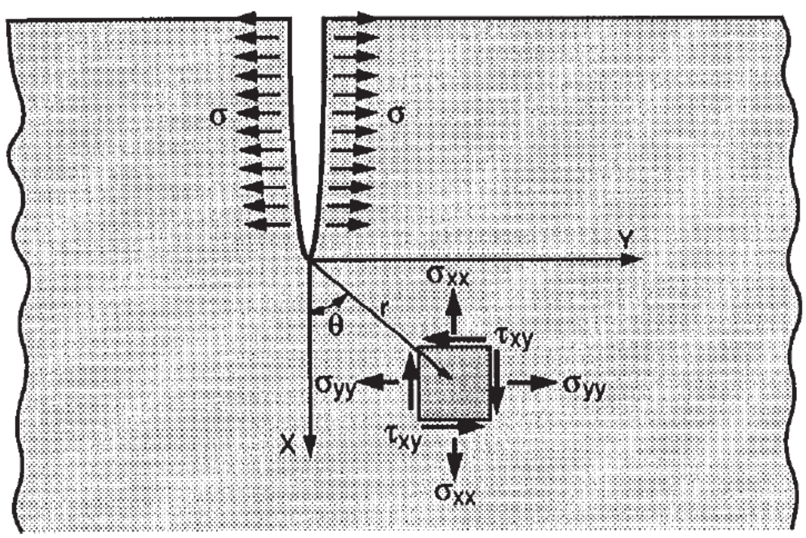

Figure 2.13 Stresses at crack tip for the opening mode (mode I) (from Konrad and Ayad 1996)

Lee et al. (1988) proposed a finite-element model of cracking propagation in brittle soils based on the theory of linear elasticity fracture mechanics (LEFM) and the mode I propagation fracture toughness parameter (or stress intensity factors, K_I) criteria.

Morris et al. (1992) proposed three respective theoretical solutions for predicting the crack depth of desiccated soils, (1) based on linear elasticity, (2) based on LEFM, and (3) associating cracking to shear failure. These three solutions consider that the cracking subjected on unsaturated soil is related to the changes in matric suction, ($u_a - u_w$) and net normal stress, ($\sigma - u_a$). The Morris (1992) methods are not applicable for predicting the propagation of primary cracks that is believed to be initiated in saturated condition. However, it can be used to predict the propagation of secondary cracks resulted from drying and wetting processes.

Abu-Hedjleh and Znidarcic (1995) presented another desiccation theory for soft fine-grained waste soils. This model considers consolidation under one-dimensional compression; desiccation under one-dimensional shrinkage; propagation of vertical cracks; and desiccation under three-dimensional shrinkage. The analysis of crack propagation was performed by using the cracking function approach, which considers that soil starts cracking at a given depth only when the soil void ratio e at this depth reaches a critical value e_{cr}.

Konrad and Ayad (1996) proposed a framework for predicting the crack depth of the primary cracks as well as the average spacing between the primary cracks. The theory of LEFM and the fictitious stress superposition concept introduced by Lachenbruch (1961) constituted the basis of the proposed prediction model. Two crucial soil parameters, namely, the tensile strength, σ_t, and the critical fracture toughness K_{Ic} are needed in this approach.

Since that the desiccation cracks provide water channels for moisture propagation in expansive soils, the above approaches sometimes can be alternatively used to estimate the depth of wetting during short-term infiltration (e.g., rainfall). For example, Yao et al. (2005) computed the ground heave of a Guangxi expansive soil site based on an assumption that the depth of active zone (depth of wetting) is equivalent to the crack depth.

2.5 Volume change parameters and determination

2.5.1 Oedometer swell tests

The one-dimensional consolidation apparatus, which is known as oedometer, has been widely used for testing expansive soils (Holtz and Gibbs 1952; Jennings and Knight 1957; Lambe and Whitman 1959; Skempton 1961; Gizienski and Lee 1965; Hardy 1965; Noble 1966; Matyas 1969; Fredlund 1969; Jennings et al. 1973; Chen 1975; Porter and Nelson 1980; Shankar et al. 1982; Kumar 1984; Justo et al. 1984; Sridharan et al. 1986; Rao et al. 1988; Fredlund 1995). Different testing methods for conducting oedometer swell tests are available in the literature. Among which, three type of testing methods; namely, free swell (FS) tests, loaded swell (LS) tests, and constant volume swell (CVS) tests are typically used for determining relevant volume change parameters (free swell strain, swelling pressure, swelling index).

2.5.1.1 Relevant swell characteristics

Oedometer swell test can be used to determine some swell properties for expansive soils, such as the percent swell (swell strain), swelling pressure, and swelling index. These parameters can be used to identify the swelling potential of expansive soils, and also grounded the application of oedometer test-based methods for predicting the ground heave.

Some relevant terminologies are summarized below to avoid possible confusions:

Swelling Potential, *SP*, is a term describing the swelling capacity of an expansive soil when absorbing moisture. It either can be quantified from the results of oedometer tests (percent swell, swelling pressure) or can be empirically qualified based on soil index properties.

Percent Swell, *S* %, being synonymous with *Free Swell %* or *Free Swell Strain*, it is commonly determined from oedometer swell test following free-swell procedures that allow a laterally confined sample on soaking to swell under a certain amount of token load (e.g., 1 kPa or 7 kPa) (Seed et al. 1962; ISRM 1989; ASTM 2003). Obviously, the result is affected by the state of the sample, the token load, and the initial water content (or initial soil suction).

Swelling Pressure, P_s will develop if an attempt is made to stop the swelling of an expansive soil (Shuai 1996). Various types of oedometer swell tests simulating different confining conditions and loading sequences can be used to determine the value of swelling pressure from the test results (e–log p plot). The swelling pressure is considered as one of the volume change parameters of expansive soil.

Swelling Index, C_s, is the slope of the rebound curve of void ratio versus the logarithm of the net normal stress (e–log p) plot obtained from oedometer tests.

2.5.1.2 Free swell test

Since the free swell (hereinafter referred to as FS) test was first introduced by Jennings and Knight (1957), it was modified by several researchers (Jennings et al. 1973; Kumar 1984; Schreiner and Burland 1991) to investigate the swell behavior of expansive soils.

The performing procedures of FS test has been summarized in the Method A of ASTM-D4546 (1996, 2003), which suggest to inundate the soil specimen with water and allow it to swell freely in the vertical direction under a token load of at least 1 kPa. After the soil specimen stops from swelling, the magnitude of percent swell can be measured. The vertical stress is then applied in increments to gradually consolidate the soil specimen. The stress required to consolidate the soil specimen to its initial volume (or void ratio) (see Figure 2.14) is defined as swelling pressure (Hardy 1965; Sridharan et al. 1986; ASTM 1996, 2003). Since that the determination of swelling pressure using the FS test involves free swell and load back procedures, this magnitude of swelling pressure is sometimes termed as “load-back swelling pressure”.

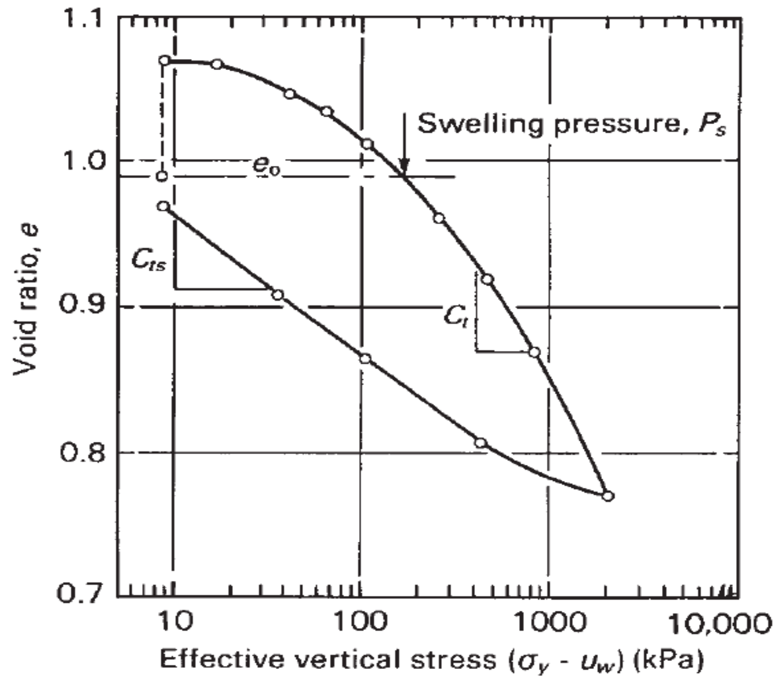

Figure 2.14 Typical free-swell oedometer test results (from Fredlund et al. 2012)

After the applied stress reaching the value that is equivalent to the "load-back swelling pressure", keep increasing the applied stress to continue the consolidation process until reaching a certain value of void ratio. The applied stress is then gradually removed; the measured data results in a rebound (swelling) curve, from which the slope (i.e., swelling index, C_s) can be determined (see Figure 2. 14).

The stress path of FS test in Figure 2.15 shows that the loading and wetting sequences is different form the actual condition of the field. This also constitutes the major drawback of the FS test (Brackley 1975a; Justo et al. 1984; El Sayed and Rabba 1986).

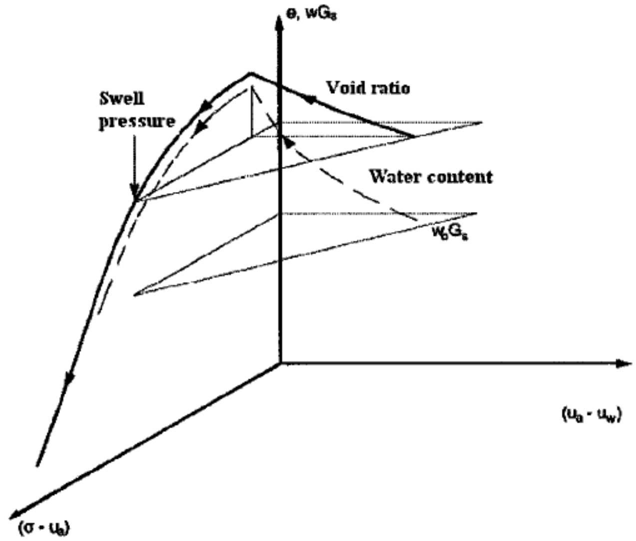

Figure 2.15 Three-dimensional stress path of free-swell oedometer test (from Fredlund et al. 1995)

2.5.1.3 Loaded swell test

The loaded swell (hereinafter referred to as LS) test is also known as the swell under load test. This oedometer test method is first recorded by Holtz and Gibbs (1952) and then modified by many researchers to investigate the swelling pressure of expansive soils (Skempton 1961; Gizienski and Lee 1965; Noble 1966; Matyas 1969). Referred as the Method A in standard ASTM-D4546 (2008), the detailed procedures for conducting LS test are as follows:

(i) Prepare four or more identical soil specimens assembled in oedometers;
(ii) Apply different level of stresses on respective oedometer units, then give them access to free water and allow free swell;
(iii) Record the swell, final water content, and dry densities;
(iv) The minimum vertical stress required for preventing swell is termed as swelling pressure, and the swell strain corresponding to a near zero stress of 1 kPa is termed as percent swell; the magnitude of these parameters can be interpreted from the test results (see Figure 2.16).

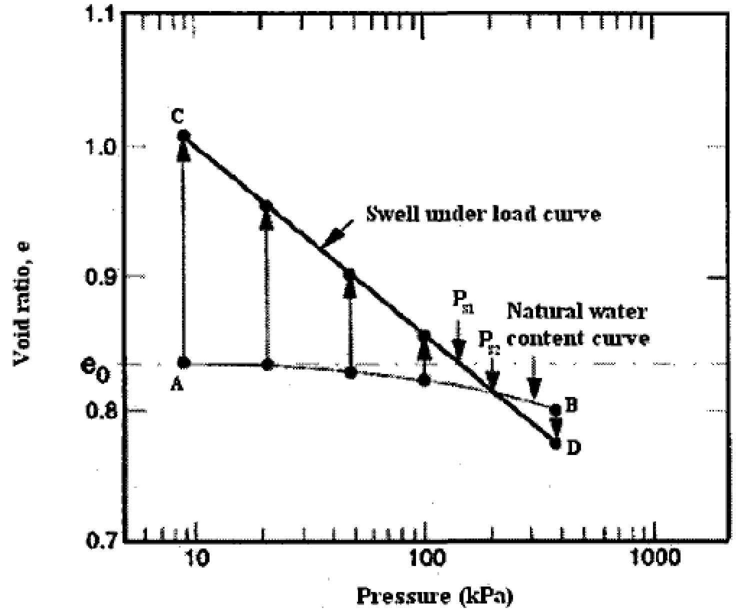

Figure 2.16 Typical loaded swell oedometer test results (from Shuai 1996)

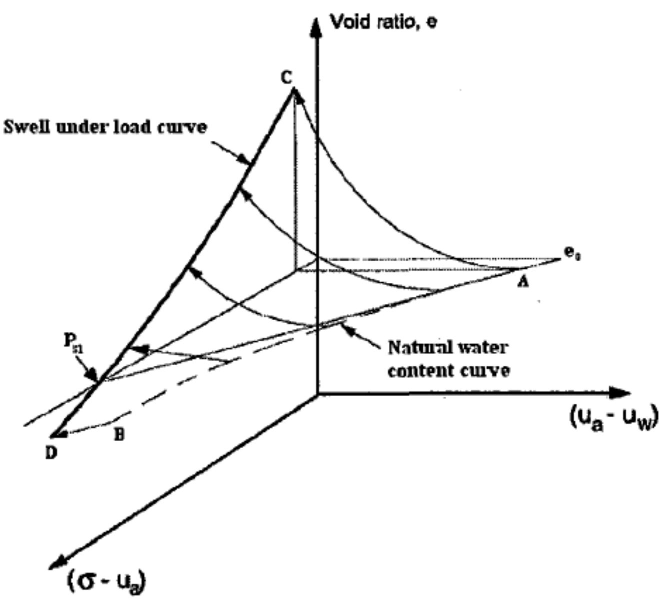

Figure 2.17 Three-dimensional stress path of loaded swell oedometer test (from Shuai 1996)

The stress path of the LS test is shown in Figure 2.17. The loading and wetting sequence followed in LS test is most likely encountered in the field condition. This constitutes the main advantage of LS test. However, identical compacted soil specimens are difficult to be prepared. It is even more so for obtaining undisturbed natural expansive soils.

2.5.1.4 Constant volume swell test

Constant volume swell (hereinafter referred to as CVS) tests have been suggested by several researchers (Frydman and Calabresi 1987; ISRM 1989; ASTM 1996; Thompson et al. 2006; Singhal et al. 2011). In the CVS test, a soil specimen is subjected to a token load and immersed in water. The volume of the soil specimen is maintained constant by varying the load on the soil specimen. The applied stress is supposed to increase until the soil specimen has no further tendency for swelling. The value of the stress at this point is conventionally regarded as the magnitude of swelling pressure (see Figure 2.18). Afterwards, increments and decrements of stress are sequentially applied to determine the consolidation and rebound curves; on which the compression index and swelling index can be achieved. The stress path of CVS test is depicted in Figure 2.19.

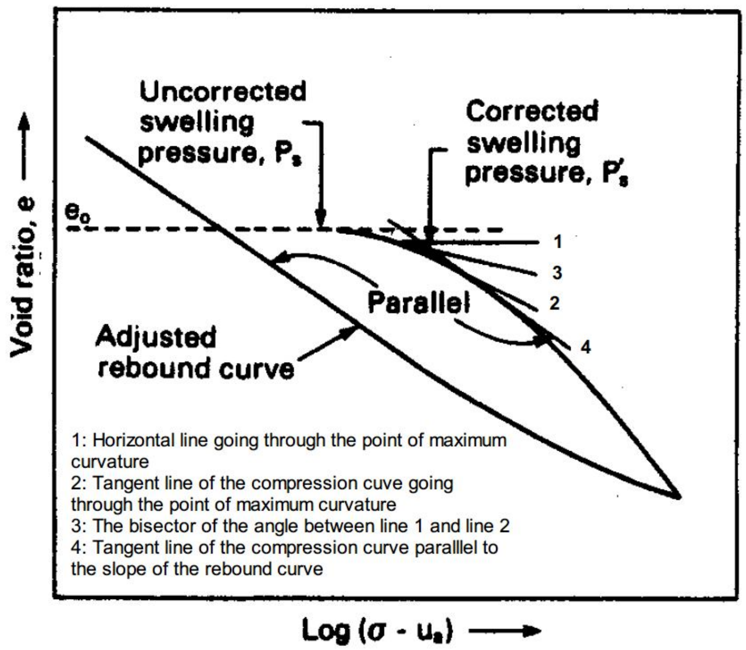

Figure 2.18 Typical loaded swell oedometer test results and correction procedures (from Fredlund and Rahardjo 1993)

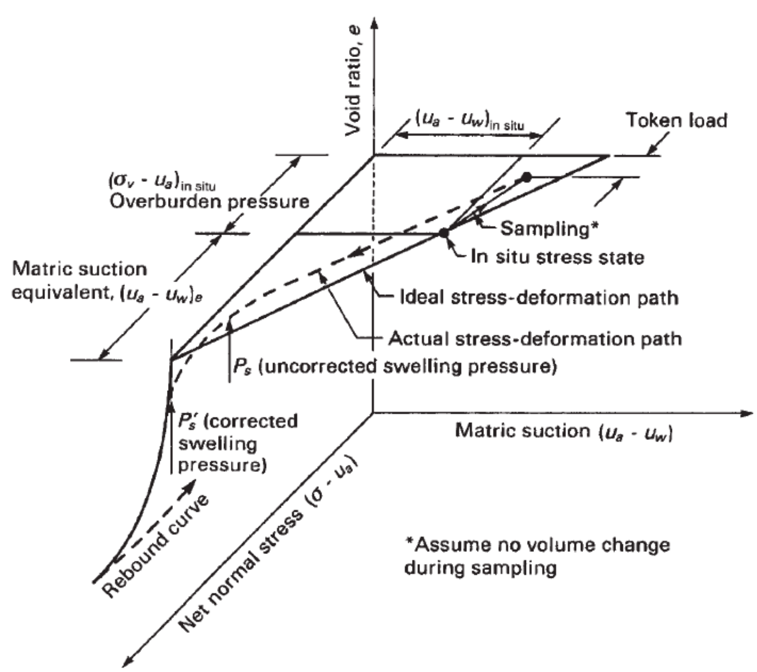

Figure 2.19 Three-dimensional ideal and actual stress paths of constant volume swell oedometer test (from Fredlund et al. 2012)

Since the CVS test does not involve volume change and does not incorporate hysteresis into the estimation of the swelling pressure, it was recommended as the best method to investigate the swelling behavior of expansive soils (Porter and Nelson 1980; Fredlund 1983). Even though, the CVS test has certain limitations, such as the difficulty associated with the maintenance of an absolute constant volume during operation, the deformation of apparatus and especially, the influence of sample disturbance.

The result of CVS test is found to be rather sensitive to even small changes of the soil specimen volume (Barber 1956; Dawson 1956; DuBose 1956; Seed et al. 1962). The stress control can be individual dependent during the maintenance of a constant volume. Even more, the compressibility of the apparatus may contribute some inevitable volume changes of the soil specimen that also influences the results of CVS test results. Fredlund (1969) suggested that the compressibility of the oedometer apparatus should be determined prior to the performance of a CVS test.

The significance of sampling disturbance has been recognized by many researchers (Fredlund et al. 1980; Nelson and Miller 1992). The deviation between actual stress path and ideal stress path is illustrated in Figure 2.19. To compensate the influence attributed to the sampling disturbance, Fredlund and Rahardjo (1993) and Nelson and Miller (1992) suggested two respective correction procedures to obtain the “corrected swelling pressure” (see Figure 2.18 and Figure 2.20).

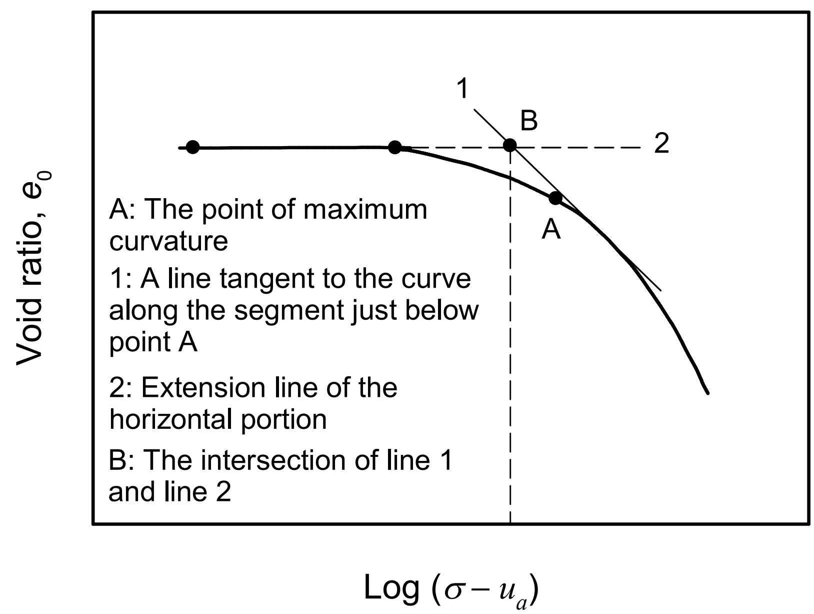

Figure 2.20 Correction procedures suggested by Nelson and Miller (1992)

2.5.1.5 Factors influencing the swelling pressure measurement

Several factors have significant influence on the determination of swelling pressure. These factors includes test methods, surcharge load, initial water content (or soil suction), dry density, and soil structure (Noble 1966; Rao et al. 2004; Attom et al. 2006; Pedarla et al. 2012; Pedarla 2013).

Sridharan et al. (1986) suggested that for identical soil specimens, FS test results in the greatest value of swelling pressure, LS test results in the lowest value of swelling pressure, while the result of CVS test is located in between the two values (see Figure 2.21). Relevant data available in the literature has been summarized in Table 2.3, presenting greatest values measured from FS tests, lower values for corrected swelling pressure measured from CVS tests, and lowest values for uncorrected swelling pressure measured from CVS tests. However, the experimental results recorded in Feng et al. (1998) disagree with the above sequence (see Figure 2.22).

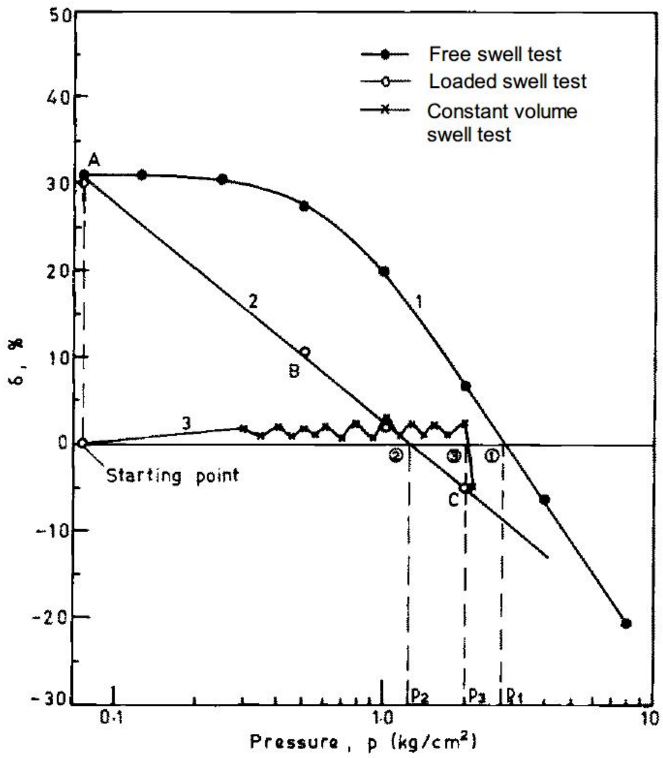

Figure 2.21 Results of different oedometer tests (after Sridharan et al. 1986)

Table 2.3 Swelling pressures measured from different types of tests

Sources		FS Tests	CVS Tests		$\frac{P_{sf}}{P_{su}}$	$\frac{P_{sf}}{P_{sc}}$	$\frac{P_{sc}}{P_{su}}$
		P_{sf} (kPa)	P_{su} (kPa)	P_{sc} (kPa)			
Gilchrist (1963) from Frdlund & Rahardjo (1993)		353	145	263	2.44	1.34	1.81
Abduljauwad et al. (1998)		3100	520	800	5.96	3.88	1.54
Azam and Wilson (2006)		–	150	320	–	–	2.13
Nagaraj et al. (2009)	Sample 1	172	160	–	1.07	–	–
	2	316	231	–	1.37	–	–
	3	939	636	–	1.48	–	–
Shuai (1996)	Sample 1	320	151	–	2.12	–	–
	2	320	250	300	1.28	1.07	1.20
	3	320	265	–	1.21	–	–
Fredlund (1969)	Sample 1	–	306	442	–	–	1.44
	2	–	249	335	–	–	1.35
	3	–	67	81	–	–	1.21

Note: P_{sf} is measured from FS test, P_{su} is uncorrected swelling pressure measured from CVS test, and P_{sc} is corrected swelling pressure measured from CVS test.

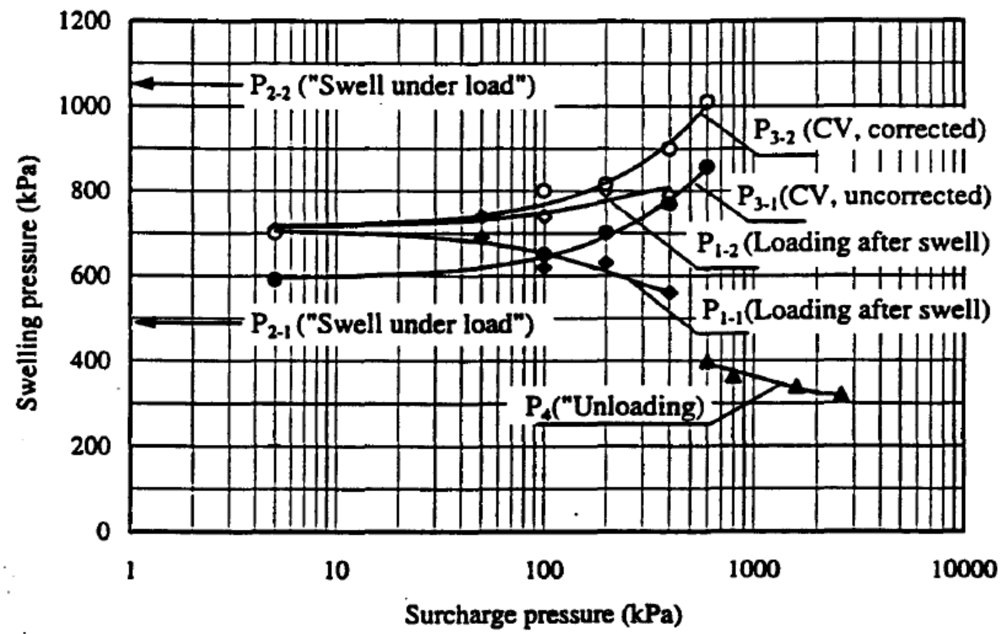

Figure 2.22 The relationship between swelling pressure and surcharge pressure for different methods (after Feng et al. 1998)

Feng et al. (1998) also discussed the relation between swelling pressure and surcharge load. Typically, the measured magnitude of swelling pressure increases with increased surcharge pressure (Figure 2.22).

The influence of initial water content and soil suction on the swelling pressure has been well investigated by numerous researchers (e.g., Holtz and Gibbs 1952; Noble 1966; Kassif et al. 1973; Justo et al. 1987; Abduljauwad et al. 1993; Rao et al. 2004). Essentially, the initially drier soil generates greater amount of swelling pressure. The tendency of swelling pressure with respect to the variation of the initial water content and soil suction are illustrated in Figure 2.23 and Figure 2.24, respectively.

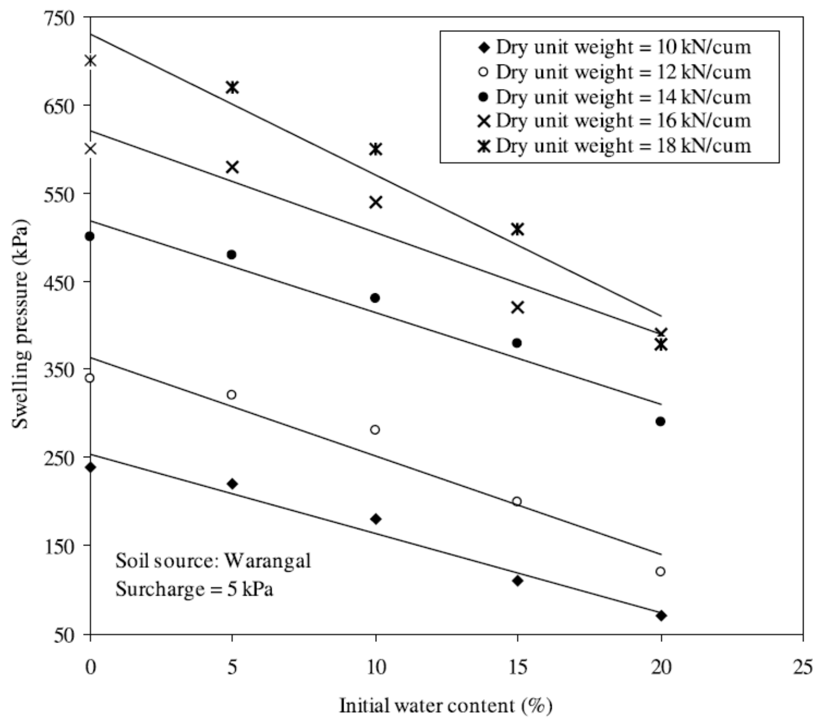

Figure 2.23 Variation of swelling pressure with respect to initial water content (from Rao et al. 2004)

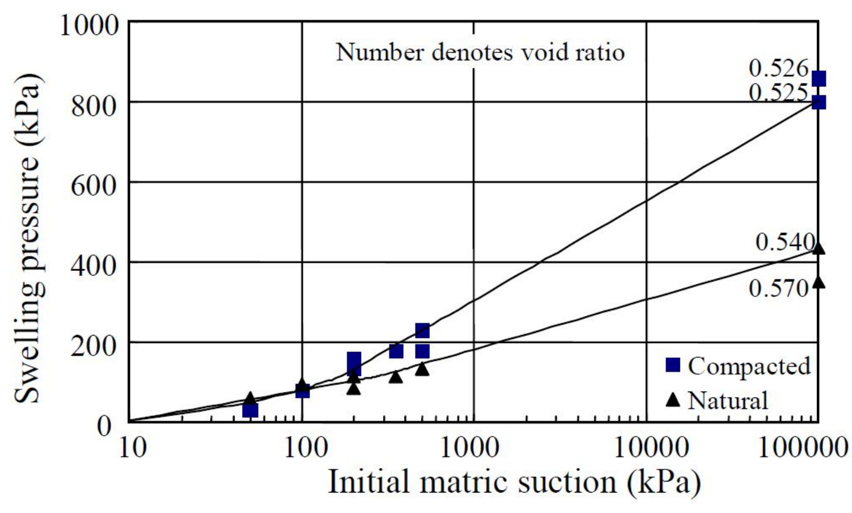

Figure 2.24 Variation of swelling pressure with respect to initial suction (from Zhan et al. 2007)

Chen (1975), Rao et al. (2004), and Pedarla (2013) investigated the relationship between load-back swelling pressure and initial dry density. As shown in Figure 2.23, at a same initial water content, the load-back swelling pressure increases with an increase in the initial dry density. The reason cited for such performance of swelling pressure is the increased interaction between clay particles due to closer packing. It also manifests the influence of the soil structure on the magnitude of swelling pressure.

2.5.2 Suction indices and determination

The soil indices that relate suction changes and volume changes are summarised in Table 2.4. These indices were defined in many different ways, and can be obtained from either direct measurement or indirect speculation. For example, the suction compression index, can either determined from a combination of CLOD test and suction measurement, or from an empirical relationship (Equation 2.27) (McKeen 1992).

$$C_h = -0.02673\left(\frac{\Delta h}{\Delta w}\right) - 0.38704 \tag{27}$$

where, Δh = change in total suction, Δw = change in water content.

Table 2.4 Volume change indices with respect to suction changes (after Hamberg 1985 and Nelson and Miller 1992)

Reference	Terminology	Symbol	Definition
Fredluns (1979)	Coefficient of Compressibility with respect to Matric Suction	a_m	Slope of void ratio versus matric suction $a_m = \Delta e/\Delta(u_a{-}u_w)$
Fredluns (1979) Sherry (1982) Nelson and Miller (1992)	Compressive Index with respect to Matric Suction	C_m	Slope of void ratio versus log matric suction $C_m = \Delta e/\Delta \log (u_a{-}u_w)$
Johnson (1977, 1979) Johnson and Snethen (1978)	Suction Index	C_τ or C_ψ	Slope of void ratio versus log matric suction, approximated by $C_\tau = \alpha G_s / 100B$ where, α = compressibility factor ($0<\alpha<1$) B = slope of suction / water content relationship
Lytton (1977a) McKeen (1981) McKeen (1985)	Suction Compression Index	γ_h	Slope of volumetric strain versus log total suction: $\gamma_h = (\Delta V/V)/\Delta\log h = [\Delta e / (1+e_f)]/\Delta\log h$ $\Delta V/V$= volumetric strain CLOD test determination: $\gamma_h = (\Delta V/V_d) \cdot f / \Delta \log h$ V_d = dry volume $\Delta V/V_d$ = volumetric strain with respect to dry volume f = lateral strain factor
McKeen (1985)	Suction Compression Index	C_h	Slope of volumetric strain versus log total suction: $C_h = (\Delta V/V_{nat})/\Delta \log h$ V_{nat} = natural volume $\Delta V/V_{nat}$ = volumetric strain with respect to natural volume CLOD test determination: $C_h = (\Delta V/V_{nat}) \cdot f /\Delta \log h$ f = lateral strain factor
Aitchison and Martin (1973)	Instability Index	I''_{pt}	I''_{pt} = slope of vertical strain versus log total suction
Fargher et al. (1979)	Instability Index	I''_{pm} I''_{ps}	$I''_{pt} = \varepsilon_v /\Delta \log h$ I''_{pm} = slope of vertical strain versus log matric suction $I''_{pm} = \varepsilon_v /\Delta \log (u_a{-}u_w)$ I''_{ps} = slope of vertical strain versus log solute (osmotic) suction $I''_{ps} = \varepsilon_v /\Delta \log \pi$
Grossman et al. (1968) (U.S.D.A Soil Conservative Service)	Coefficient of Linear Extensibility	COLE	Value of linear strain corresponding to suction change from 33 kPa (2.53 pF) to oven dry: COLE = $\Delta L / \Delta L_D = (\gamma_D /\gamma_w)^{1/3} - 1$ where $\Delta L/\Delta L_D$ = linear strain relative to dry dimensions γ_D = bulk density of overn dry sample γ_w = bulk density of sample at 33 kPa

2.5.2.1 Suction measurement

As presented in early sections, the total suction has two components including matric suction and osmotic suction. The soil suction can be determined from either direct methods or indirect methods. Table 2.5 provides a summary on different methods for suction measurement, the suction components to be measured, as well as the range of suction measurement. Among these methods, psychrometer, Tensiometer, axis-translation techniques, filter paper, thermal conductivity sensor, and pore fluid squeezing technique are commonly used.

Table 2.5 Summary of suction measurement methods (after Pan et al. 2010)

	Suction component	**Technique (Method)**	**Suction range (kPa)**	**Equilibrium time**
Direct methods	Matric suction	Axis-transition technique	0−1500	Hours
		Tensiometer		Hours
		Suction probe		minutes
Indirect methods	Matric suction	Time domain reflectometry	0−1500	Hours
		Electrical conductivity sensor	50−1500	6−50 hours
		Thermal conductivity sensor	0−1500	hours − days
		Contact filter paper	All	7 – 14 days
	Osmotic suction	Pore fluid squeezing technique	0−1500	Days
	Total suction	Psychrometer technique	100−10000	1 h
		Relative humidity sensor	100−8000	hours−days
		Chilled-mirror hygrometer	150−30000	10 minutes
		Non-contact filter paper	All	7 – 14 days

2.5.2.2 COLE and clod tests

Coefficient of Linear Extensibility (COLE) test was originally proposed by the Soil Conservative Service (Grossman et al. 1968). The principle of COLE test can be used for determining the original suction compression index term, γ_h proposed by Lytton (1977a). Based on the concepts of COLE test, McKeen (1985) formulated the clod test method to determine a modified suction compression index term, C_h, introducing a lateral strain factor, f.

The detailed procedures for conducting the clod test have been reported in the literature (McKeen 1981; McKeen 1985; Hamberg 1985; Perko 2000), and are summarized below:

1. Obtain specimens (clods) of undisturbed representative soil samples. The clods may be irregularly shaped. Take necessary precautions to maintain field moisture content of the specimens.
2. Separate the specimens into two halves. Perform a filter paper suction test on one half and reserve the other for immediate clod testing.
3. Measure the weight of each specimen (W1). Sampling weight should be between approximately 70 and 140 g.
4. Tie a fine wire around each sample and attach a tag for handling and identification. Measure the weight of each tagged specimen (W2).
5. Coat the specimens with liquid resin similar to 1 part DOW Saran© F310 dissolved in 7 parts methyl ethyl ketone or acetone. The coating procedure should be done under a fume hood as follows: dip in resin, dry five minutes in air. Weight each coated specimen immediately after air drying (W3).
6. Set a container of water on an electronic balance and "tare out" its weight. The container should be large enough to facilitate specimen submersion without touching the sides. Momentarily submerge each specimen under water and record its buoyant force (W5).
7. Oven dry the specimens for 48 hours at 105 °C. Weight the oven-dried specimens in air and submerged under water to determine W7 and W8, respectively.

2.5.2.3 Australian Standard tests

The ratio of vertical strain to suction change was experimentally observed and defined as instability index, I_{pt} (Aitchison and Woodburn 1969; Aitchison 1970; and Lytton and Woodburn 1973). This index is equivalent to the suction index, C_τ (Johnson and Snethen 1978), and can be determined from shrink swell test. The Australian Standard AS 2870–1996 suggested three methods for estimation of instability index. They are namely; the shrink-swell test (AS 1289.7.1.1), loaded shrinkage test (AS 1289.7.1.2), and core shrinkage test (AS 1289.7.1.3).

2.6 Heave prediction methods

2.6.1 Empirical determination

Soil classification parameters such as Atteberg limits, plasticity index, clay fraction, activity, dry density, and initial water content are used to empirically predict the swelling behavior of expansive soils. Empirical relations between percent swell, S %, swelling pressure, p_s and the soil parameters are summarized in Table 2.6.

Since only routine soil parameters are needed, the empirical methods were received well in conventional engineering practise. These relationships, however, are most likely developed based on limited local data. As a consequence, they are not universally valid for all types of expansive soils.

Table 2.6 Summary of the empirical methods (after Nelson and Miller 1992; Rao et al. 2004; Çimen et al. 2012; and Vanapalli et al. 2012; and Adem 2015)

Reference	Empirical relationships for swelling potential and swelling pressure
Seed et al. (1962)	S % = 0.00216 $I_p^{2.44}$ for undisturbed S % = 0.0036 $I_p^{2.44}$ for disturbed soils S % = 3.6×10^{-5} $A^{2.44}$ $c^{3.44}$ I_p = plasticity index c = clay content A = activity = (I_p / c)
van der Merve (1964)	$\Delta H = Fe^{-0.377D} (e^{-0.377H} - 1)$ H = volume change ΔH = total heave F = correction factor for degree of expansiveness D = the thickness of non-expansive layer
Ranganatham & Satyanarayana (1965)	S % = 0.000413 $I_s^{2.67}$ I_s = shrinkage index, ($w_l - w_s$) w_l = liquid limit w_s = shrinkage limit

Komornik and David (1969)	Log $p_s = -2.132 + 0.0208w_l + 0.000665\gamma_d - 0.0269w_i$
Nayak & Christensen (1971)	$S\,\% = 0.00229\ I_p^{2.67}\ (1.45\ c)\ /\ w_i + 6.38$ p_s (psi) $= [(3.58 \cdot 10^{-2})\ I_p^{1.12}\ c^2\ /\ w_i^2] + 3.79$ w_i = initial water content p_s = the swelling pressure
Vijayvergiya & Ghazzaly (1973)	$S\,\% = (0.44\ w_l - w_i + 5.5)\ /\ 12$ Log (S %) $= 0.0526\ \gamma_d + 0.033\ w_l - 6.8$ γ_d = dry unit weight
Schneider & Poor (1974)	Log (S %)$= 0.9\ (I_p\ /\ w_i) - 1.19$
McCormack & Wilding (1975)	$S\,\% = 7.5 - 0.8w_i + 0.203c$
Brackley (1975b)	$S\,\% = (5.3 - (147e\ /\ I_p) - \mathrm{Log}\ P)\times(0.525I_p + 4.1 - 0.85w_i)$ P = surcgarge
O'Neil & Ghazzally (1977)	$S\,\% = 2.77 + 0.131w_l - 0.27w_n$ w_n = natural water content
Chen (1975)	$S\,\% = 0.2558\ e^{0.00838\ Ip}$
Johnson (1978)	$S\,\% = 23.82 + 0.7346I_p - 0.1458H - 1.7w_0 + (0.0025I_p)w_0 - (0.00884I_p)H$ $S\,\% = -9.18 + 1.5546I_p + 0.08424H + 0.1w_0 - (0.0432I_p)w_0 - (0.01215I_p)H$
Weston (1980)	$S\,\% = 0.00411\ w_{lw}^{4.17}\ \sigma_v^{-3.86}\ w_i^{-2.33}$ w_{lw} = weighted liquid limit
Bandyopadhyay (1981)	$S\,\% = 0.00114\ A^{2.559}c^{3.44}$
Picornell & Lytton (1984)	$\Delta H = \sum[f_i\ (\Delta v\ /\ v)_i]\ H$ H = the stratum thickness $(\Delta v/v)_i$ = volumetric strain f_i = factor to include the effects of the lateral confinements
Dhowian (1990a)	$\Delta H = (S\,\%)\ H$
Basma (1993)	$S\,\% = 0.00064I_p^{1.37}c^{1.37}$
Çokça (2002)	$S\,\% = -121.807 + 12.1696$ MBV $+ 27.6579$ Log ψ_i MBV = methylene blue value
Erguler & Ulusay (2003)	$p_s = -227.27 + 2.14w_i + 1.54w_l + 72.49\gamma_d$
Rao et al. (2004)	$S\,\% = 4.24\gamma_{di} - 0.47w_i - 0.14q_i - 0.06$FSI $- 55$ γ_{di} = dry unit weight q_i = initial surcharge FSI = free swell index
Erzin & Erol (2004)	Log $p_s = -4.812 + 0.01405I_p + 2.394\gamma_d - 0.0163w_i$

	$\text{Log } p_s = -5.020 + 0.01383I_p + 2.356\gamma_d$
Sabtan (2005)	$S\,\% = 1.0 + 0.06(c + I_p - w_i)$ $p_s = 135.0 + 2.0(c + I_p - w_i)$
Azam (2007)	$S\,\% = 0.6\, I_p^{1.188}$
Yilmaz (2009)	$S\,\% = 2.0981e^{-1.7169\, IL}$ IL = liquidity index = $[(w_l - w) / I_p]$
Türköz and Tosun (2011)	$S\,\% = -57.865 + 37.076\, \rho_d + 0.524\text{MBV} + \varepsilon$ ε = mean-zero Gaussian random error term
Çimen et al. (2012)	$(S\,\%)_1 = (0.3139\gamma_d^{0.3552} - 0.1177\, w_i^{0.4470})I_p^{0.9626}$ $(\text{Log } p_s)_1 = 0.0276I_p - 365.2118\gamma_d^{-2.4616} - 0.0320w_i + 2.2292$ $(S\,\%)_2 = (0.4768\gamma_d^{\,0.3888} - 0.0033\, w_i^{1.6045})I_p^{0.7224}$ $(\text{Log } p_s)_2 = 0.0239I_p - 1285.3723\gamma_d^{\,-3.2768} - 0.0396w_i + 2.3238$ $(S\,\%)$ = mean $(\text{SP}_1, \text{SP}_2)$ $\text{Log } p_s$ = mean $[(\text{Log } p_s)_1, (\text{Log } p_s)_2]$
Zumrawi (2013)	$S\,\% = 24.5\, (q^{-0.26})\, (I_p\, c)^{1.26}\, [F_i - 7.1\, (q^{0.22})\, (I_p\, c)^{1.26}]$ q = surcharge F_i = initial state factor

2.6.2 Oedometer test-based methods

As presented in the early sections, oedometer tests can be used to determine the volume change indices as well as initial and final void ratio. On one hand, the obtained data of swell strain, S % or void ratio, e (initial and final) can be directly used for ground heave estimation, on the other hand, the information of swelling pressure, P_s and swelling index, C_s facilitate the ground heave estimation based on certain assumptions on the stress path.

Extensive research has been undertaken to estimate the ground heave based on the results of different types of oedometer tests. The oedometer test-based heave prediction methods are summarized in Table 2.7.

Typically, the soil specimens are fully wetted in the oedometer apparatus. As a consequence, these methods essentially are only useful for estimating the potential heave (which is the maximum amount of heave) of expansive soil.

Table 2.7 The oedometer test-based heave prediction methods (modified from Vanapalli & Lu 2012 and Singhal 2010)

Name of the Method	Country of Origin	Reference
Double Oedometer method	South Africa	Jennings & Knight (1957)
Volumemeter method	South Africa	De Bruijn (1961)
Salas & Serratosa method	Spain	Burland (1962)
Sampson, Shuster & Budge method	Colorado, U.S.	Sampson et al. (1965)
Noble method	Canada	Noble (1966)
Sullivan & McClelland method	U.S.	Sullivan & McClelland (1969)
Komornik, Wiseman & Ben-Yacob method	Israel	Komornik et al. (1969)
Holtz method	U.S.	Holtz (1970)
Navy method (Direct method)	U.S.	NAVFAC (1971)
Wong & Yong method	U.K.	Wong & Yong (1973)
U.S.B.R. method	U.S.	Gibbs (1973)
Simple Oedometer method	South Africa	Jennings et al. (1973)
PVR Method (Texas Highway Department)	U.S.	Smith (1973)
Mississippi method	U.S.	Teng et al. 1972 Teng et al. 1973 Teng & Clisby (1975)
Controlled strain test	U.S.	Porter & Nelson (1980)
Fredlund, Hasan & Filson method	Canada	Fredlund et al. (1980)
Sridharan, Rao & Sivapullaiah method	India	Sridharan et al. (1986)
Erol, Dhowian & Youssef methof	Saudi Arabia	Erol et al. (1987)
Shankar, Ratnam & Rao method	India	Shanker et al. (1987)
Heave index method	U.S. (Denver)	Nelson et al. (1998)
Al-Shamrani & Al-Mhaidib method	Saudi Arabia	Al-Shamrani & Al-Mhaidib (1999)
Basma, Al-Homoud & Malkawi method	Jordan	Basma et al. (2000)
Subba Rao & Tripathy method	India	Subba Rao & Tripathy (2003)

The oedometer test-based methods estimate the magnitude of the potential heave by applying the consolidation theory in reverse. The heaving process of expansive soil can be regarded as a release of swell potential which is quantified by the magnitude of swelling pressure, namely the initial stress state. When the soil stops from heaving, the final stress state can be quantified by the magnitude of overburden pressure. Based on certain simplifications on the stress path, the relation between volume change and stress state condition can be described using volume change parameter such as swelling index, C_s, heave index C_H, slope of surrogate path (SP), C_{SP} (Fredlund 1983; Nelson et al. 2006; and Singhal 2011).

2.6.2.1 Fredlund (1983) method

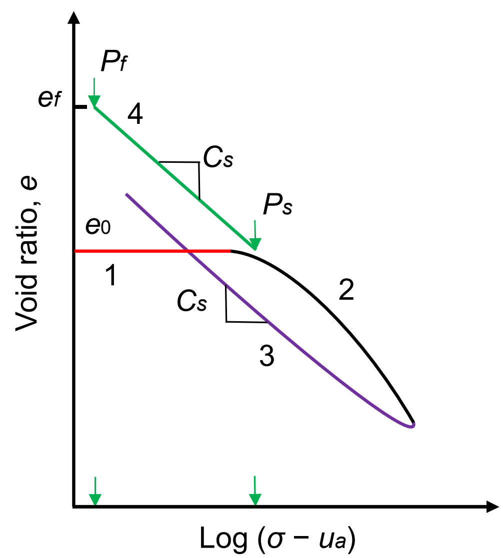

Figure 2.25 Schematic of heave estimation using Fredlund (1983) method

Fredlund (1983) assumed that the stress path (line 4 in Figure 2.25) from initial stress state to final stress state is parallel to the rebound curve of consolidation test, on which the slope is termed as swelling index, C_s. Based on such understanding, Fredlund (1983) proposed an equation to estimate the ground heave of expansive soil. To compensate the influence of sampling disturbance, the corrected swelling pressure P_s' was used in Equation (2.28). The detailed correction procedures for swelling pressure determination have been presented in early section.

$$\Delta H = C_s \frac{H}{1+e_0} \log\left(\frac{P_f}{P_s'}\right) \tag{2.28}$$

where, H = the thickness of soil layer, C_s = swelling index, e_0 = initial void ratio, P_s' = corrected swelling pressure, P_f (= $\sigma_y + \Delta\sigma_y - u_{wf}$) = final stress state, (σ_y is total overburden pressure, $\Delta\sigma_y$ is the change in total stress, u_{wf} is pore-water pressure), and ΔH = the heave of the soil layer.

2.6.2.2 Nelson et al. (2006) method

Instead of using C_s to describe the void ratio versus stress relationship, Nelson et al. (2006) suggested to use heave index, C_H to describe the swell strain versus stress relationship. The determination of C_H involves two different oedometer test (CVS and LS) on two identical soil samples, while the preparation of identical soil sample is generally not practical (Figure 2.26).

$$C_H = \frac{S\%}{\log\left[\dfrac{\sigma'_{cv}}{(\sigma'_i)_A}\right]} \tag{2.29}$$

$$\Delta H = \sum h_i \, C_H \log\left(\frac{\sigma'_{cv}}{\sigma'_{v0}}\right) \tag{2.30}$$

where, C_H = heave index, S % = vertical swell strain, σ'_{cv} = constant volume swelling pressure, σ'_i = inundation pressure during the CVS test, σ'_{v0} = overburden pressure, ΔH = the heave of the soil layer, h_i = the thickness of i^{th} layer, i = the number of the soil layers.

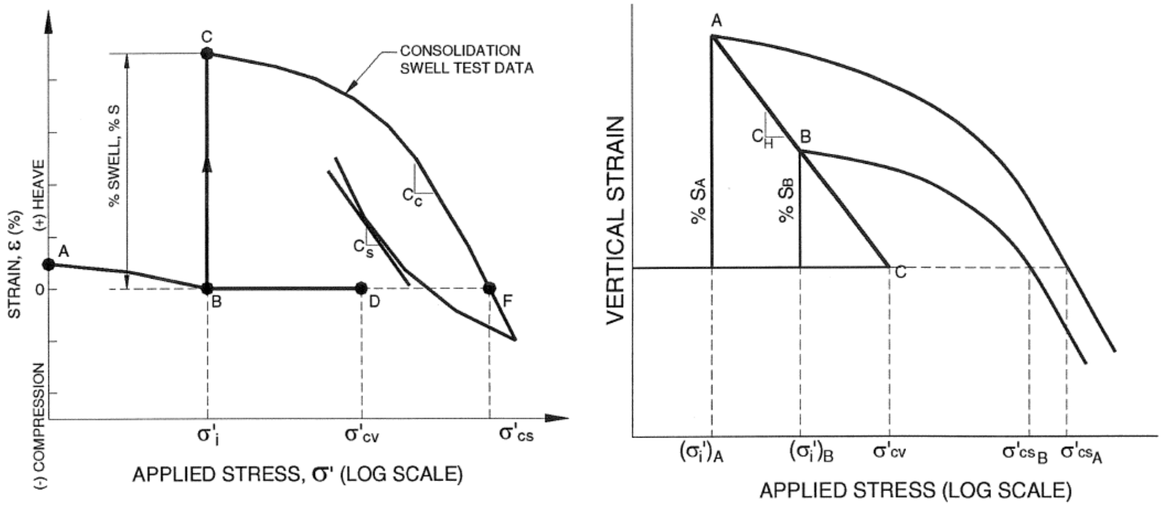

Figure 2.26 Determination of heave index, C_H using CVS and LS tests (from Nelson et al. 2006)

2.6.2.3 Singhal (2011) method

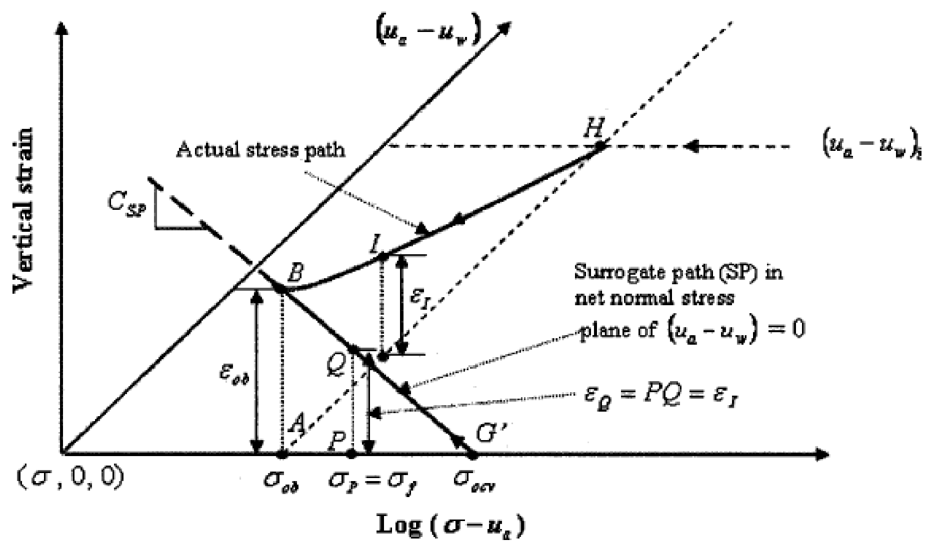

Figure 2.27 Strain-based "equivalence" of reduction of suction from $(u_a - u_w)_i$ to zero (path HB) to reduction in net normal stress from σ_{ocv} to σ_{ob} (along path G′B, the SP)

Similar to the method proposed by Nelson et al. (2006), Singhal (2011) proposed a heave prediction utilizing a loaded swell (LS) test and a constant volume swell (CVS) test. Both the test specimens are confined at overburden stress (σ_{ob}) corresponding to the depth of the sample taken from the field. When the maximum swell occurs, the matric suction was assumed to approach zero (fully saturated condition). In this case, instead of actual three dimensional stress path, the surrogate path (SP) located on the $\varepsilon - \log p$ plot can be used to determine the maximum potential heave, if the initial and final stress state are known (Figure 2.27). As per Singhal (2011), the initial stress state can be considered to be equivalent to the constant swelling pressure as it is measured, while the final stress state needs to be estimated based on the information of matric suction.

$$C_{SP} = \frac{\varepsilon_{ob} - 0}{\log\left[\dfrac{\sigma_{ocv}}{\sigma_{ob}}\right]} \tag{2.31}$$

where, C_{SP} = slope of the surrogate path (SP), ε_{cv} = swell strain determined from the LS test applying a load equal to σ_{ob}, σ_{ob} = confining pressure for the swell test, which is the overburden stress for the sample, σ_{ocv} = swelling pressure determined form the CVS pressure for a test specimen initially subjected to σ_{ob}.

$$\varepsilon_{Q} = C_{SP} \log\left[\frac{\sigma_{ocv}}{\sigma_{f}}\right] \tag{2.32}$$

where, ε_Q = swell strain, σ_f = final stress state.

$$\sigma_f = \sigma_{ob} + R_w(\sigma_{ocv} - \sigma_{ob}) \tag{2.33}$$

where,

$$R_w = \frac{(u_a - u_w)_f}{(u_a - u_w)_i} \tag{2.34}$$

where, $(u_a - u_w)_i$, $(u_a - u_w)_f$ = initial and final matric suction

2.6.3 Suction-based methods

Soil response to suction changes can be predicted in much the same manner as soil response to saturated effective stress changes. The relationship between void ratio and matric suction (a_m, C_m, C_τ, C_h, γ_h, and I_{pt}) is analogous to the compression index or suction index determined by oedometer tests (Nelson and Miller 1992).

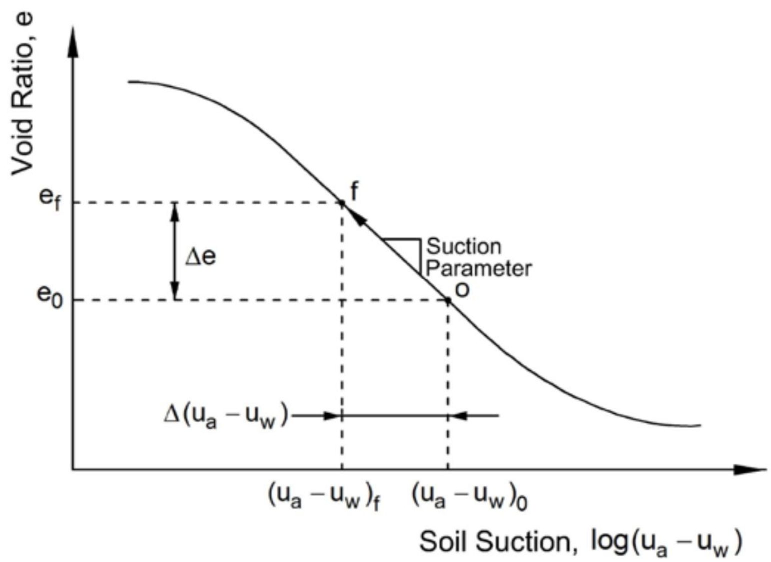

Figure 2.28 Idealized void ratio versus logarithm of suction relationship for a representative sample (modified after Hamberg 1985)

Figure 2.28 illustrates the idealized relationship between void ratio and suction for representative soil sample. The key information required for application of suction-based ground heave prediction methods is reliable determination of the suction parameters (see Figure 2.28). As presented in early sections (see Table 2.4), these suction parameters have been defined by different researchers (Aitchison & Martin 1973; Fargher et al. 1979; Johnson 1977; Lytton 1977a; Johnson and Snethen 1978; Fredlund 1979; McKeen 1981; Sherry 1982; McKeen 1985; Nelson and Miller 1992). The corresponding suction-based ground heave prediction methods are summarized in Table 2.8.

Table 2.8 Suction-based methods for predicting ground heave (after Adem 2015)

Equation		Reference
$\Delta H = \sum_{\text{layers}} \frac{H}{3} \frac{(w_f - w_i) G_s}{(100 + w_i G_s)}$ where, ΔH = soil heave H = soil layer thickness w_i = initial water content (measured) w_f = final water content (estimated in terms of the equilibrium matric suction) G_s = specific gravity	(2.35)	Richards (1967)
$\delta = \sum_{i=1}^{n} I_{pt} (\Delta u)(\Delta z_n)$ where, δ = vertical shrinkage or heave I_{pt} = instability index Δu = soil suction change Δz_i = thickness of i^{th} soil layer n = total number of soil layers considered	(2.36)	Aitchison (1973)
$S_f = \sum_{i=1}^{n} f_i \left(\frac{\Delta V}{V} \right)_i \Delta z_i$ $\left(\frac{\Delta V}{V} \right)_i = -\gamma_h \log\left(\frac{h_f}{h_i} \right) - \gamma_\sigma \log\left(\frac{\sigma_f}{\sigma_i} \right)$ where, S_f = surface displacement f_i = lateral confine factor $(\Delta V/V)_i$ = average volume strain Δz_i = thickness of i^{th} soil layer n = total number of soil layers considered h_i, h_f = initial and final water potentials σ_f = applied octahedral normal stress σ_i = octahedral normal stress above which overburden pressure restricts volumetric expansion γ_h = matric suction compression index γ_σ = mean principal stress compression index	(2.37)	Lytton (1977b)

$$\Delta H = H\frac{C_\tau}{1+e_0}\log\frac{h_0}{h_f+\alpha\sigma_f}$$ where, ΔH = soil heave H = soil layer thickness C_τ = suction index e_0 = initial void ratio h_0 = matric suction without surcharge pressure $\log h_0 = A-(Bw_0)$ h_f = final matric suction σ_f = final applied pressure (overburden plus external load) α = compressibility index	(2.38)	Johnson & Snethen (1978)
$$\Delta H = \sum_{i=1}^{n}\frac{H_i}{1+e_0}\left[C_t\Delta\log(\sigma-u_a)+C_m\Delta\log(u_a-u_w)\right]$$ where, ΔH = soil heave H_i = thickness of the i^{th} soil layer n = total number of soil layers considered e_i = void ratio of the i^{th} soil layer C_t = compressive index with respect to total stress C_m = compressive index with respect to matric suction $(\sigma - u_a)$ = total stress $(u_a - u_w)$ = matric suction	(2.39)	Fredlund (1979)
$$\Delta H = \sum_{i=1}^{n}\left(I_{pt}\ \Delta u\ H_i\right)$$ where, ΔH = vertical surface movement I_{pt} = instability index Δu = soil suction change H_i = soil layer thickness over which I_{pt} can be taken as constant n = number of layers to depth of the active zone Δw = moisture content change $\Delta\varepsilon_v$ = change in vertical strain	(2.40)	Mitchell & Avalle (1984)

$\Delta H = \sum_{i=1}^{n} \frac{H_i}{1+e_0} \left(C_h \Delta \log h \right)_i$ where, ΔH = soil heave H_i = thickness of the i^{th} soil layer e_0 = initial void ratio C_h = suction index with respect to void ratio h = soil suction n = number of layers to depth of the active zone	(2.41)	Hamberg (1985)
$\Delta H = H \gamma_h \left(\Delta \mathrm{pF} - \Delta \mathrm{pP} \right)$ where, ΔH = shrinkage or swell over vertical increment H = vertical increment over which shrink or swell is occurring γ_h = suction compression index ΔpF = change in soil suction over vertical increment ΔpP = change in soil overburden over vertical increment	(2.42)	Wray (1984)
$\Delta H = H \frac{C_\psi}{1+e_0} \log \frac{\psi_i}{\psi_f}$ where, ΔH = soil heave H = soil layer thickness C_ψ = suction index =(α G_s) / (100 B) e_0 = initial void ratio ψ_i, ψ_f = initial and final suction α = volume compressibility factor G_s = specific gravity of solid particles B = slope of suction versus water content relationship	(2.43)	Dhowian (1990a)
$\Delta H = C_h \ \Delta u \ \Delta t \ f \ s$ where, ΔH = surface heave C_h = suction compression index =(−0.02673) (Δu / Δw) – 0.38704 Δu = suction change Δt = soil layer thickness f = lateral restraint factor = (1 + 2K_0) / 3	(2.44)	McKeen (1992)

s = reduction factor to account for overburden
= 1 − 0.01(%SP)
Δw = moisture content change
K_0 = coefficient of lateral earth pressure at rest
SP = swell pressure applied to the soil due to overburden pressure

(2.45) Cover & Lytton (2001)

$$S_f = \sum_{i=1}^{n} f_i \left(\frac{\Delta V}{V}\right)_i \Delta z_i$$

$$\left(\frac{\Delta V}{V}\right)_i = -\gamma_h \log\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log\left(\frac{\sigma_f}{\sigma_i}\right)$$

$$\gamma_h = \frac{\gamma(\text{swellingcase}) + \gamma(\text{shrinkagecase})}{2}$$

$$\gamma(\text{swelling case}) = \left(\frac{\text{COLE}}{100} + 1\right)^3 - 1$$

$$\gamma(\text{shrinkage case}) = 1 - \frac{1}{\left(\frac{\text{COLE}}{100} + 1\right)^3}$$

where,
S_f = surface displacement
f_i = lateral confine factor
$(\Delta V/V)_i$ = average volume strain
Δz_i = thickness of i^{th} soil layer
n = total number of soil layers considered
h_i, h_f = initial and final water potentials
σ_f = applied octahedral normal stress
σ_i = octahedral normal stress above which overburden pressure restricts volumetric expansion
γ_h = matric suction compression index
γ_σ = mean principal stress compression index
COLE = coefficient of linear extensibility

$$\Delta H = \sum_{i=1}^{n} f\left(\frac{\Delta V}{V}\right)_i \Delta z_i$$ $$\left(\frac{\Delta V}{V}\right)_{i,\ swelling} = -\gamma_h \log\left(\frac{h_f}{h_i}\right) - \gamma_\sigma \log\left(\frac{\sigma_f}{\sigma_i}\right)$$ $$\left(\frac{\Delta V}{V}\right)_{i,\ shrinkage} = -\gamma_h \log\left(\frac{h_f}{h_i}\right) + \gamma_\sigma \log\left(\frac{\sigma_f}{\sigma_i}\right)$$ where, ΔH = surface displacement f = crack fabric factor = 0.67 – 0.44 ΔpF, 1/3 ≤ f ≤ 1.0 $(\Delta V/V)_i$ = volume strain Δz_i = the i^{th} depth increment n = number of depth increment h_i, h_f = initial and final values of matric suction σ_i, σ_f = initial and final values of mean principal stress γ_h = matric suction compression index γ_σ = mean principal stress compression index = C_c / (1 + e_0) ΔpF = change of suction C_c = compression index e_0 = void ration	(2.46)	Lytton et al. (2004)

Different from most of the oedometer test-based methods designating the measured swell potential (swelling pressure) and overburden pressure as initial and final stress state condition, the final condition of expansive soil in suction-based methods can be quantified by any suction value corresponding to specific amount of void ratio (see Figure 2.28). In other words, rather than predicting the maximum potential heave, the suction-based methods can be used to predict the ground heave of unsaturated expansive soils during the wetting process. It is believed that the suction-based methods for ground heave prediction provide better characterization of expansive soil behavior and more reliable estimates of anticipated volume change under field conditions than the oedometer methods (Snethen 1980).

2.7 Summary

Extensive research has been undertaken on expansive soils because of the enormous cost associated with their problems. The present chapter provides a comprehensive review on the literature, emphasizing the background knowledge related to the ground heave estimation. Various topics related to expansive soils behavior, which include the clay mineralogy, swelling mechanism, soil suction and its components, depth of active zone, depth of cracks, volume change indices and testing methodologies, and methods for ground heave prediction are discussed.

CHAPTER 3

PREDICTION OF VOLUME CHANGE PARAMETERS

3.1 Introduction

Information of both the swelling pressure and heave are required in the design and construction of foundations in expansive soils. Due to this reason, over the last six decades, extensive studies have been undertaken to reasonably estimate or determine swelling pressure and heave. The methodologies available in the literature for the estimation of heave can be divided into three categories; namely, (i) empirical methods, (ii) soil suction methods, and (iii) oedometer methods.

Typically, empirical methods predict the volume change indices (e.g., swelling pressure, swelling index and etc.) or heave (swell strain) directly from soil index properties (e.g., Atterberg limits, dry density, water content, clay fraction, cation exchange capacity, specific surface area). Çimen et al. (2012) and Vanapalli and Lu (2012) provide comprehensive summaries on the empirical equations available in the literature. However, since these empirical equations are typically developed from investigations on local or limited number of soils, they are not universally valid for all types of expansive soils.

The swelling pressure and heave estimated based on soil suction are considered to be most reasonable and reliable estimates although they are time-consuming in estimating soil suction values. Two typical soil suction methods are U.S. Army Corps of Engineering (WES) (Johnson and Snethen 1978) method and CLOD method (McKeen 1981). The results of these two methods considered to be reliable since the sample disturbance used for these tests can be minimized. However, suction methods typically do not consider the effect of in-situ effective stress (Nelson and Miller 2012).

The oedometer swelling tests, measuring the swelling pressure and swelling index, have been commonly used to predict the heave of expansive soils. Three different types of experimental procedures (i.e., free swell, FS: loaded swell, LS; and constant volume swell, CVS) are available for conducting these tests. More details of these tests and their procedures have been summarized in Chapter 2. There is evidence suggesting that the measurement of swelling pressure is strongly sensitive to the testing procedures followed (Fredlund 1969; Shuai 1996; Azam and Wilson 2006; Nagaraj et al. 2009). The constant volume swell (CVS) test provides a swelling pressure value that falls in between the free swell (FS) and load swell (LS) testing procedures (Sridharan et al. 1986). In addition, for alleviating the influence of sample disturbance, Fredlund and Rahardjo (1993) and Nelson and Miller (1992) suggested two respective graphical correction procedures to reliablly estimate the swelling pressure from the CVS tests. However, as the swelling pressure values that are plotted on the semi-logarithm scale are rather sensitive with respect to the correction procedures. Moreover, the maintenance of a constant volume state during the saturation process of the expansive soil samples could be rather cumbersome and time consuming. Especially, the methodologies based on oedometer test results in an overestimated ground heave, as these tests are proceeded until the samples reaching fully saturated condition.

To overcome the disadvantages of the above heave prediction methods, simple techniques for predicting the 1-D heave of expansive soils have been proposed in the present chapter. These techniques includes swelling pressure prediction models, swelling index estimation, and equation for the ground heave of the expansive soil exhibiting partially wetting process.

Two SWCC-based models for predicting the variation of the swelling pressure (P_s); one for the compacted soils and the other for natural expansive soils. Both the models are capable of predicting the variation of swelling pressure with respect to initial soil suction (ψ_i). A simple relationship between swelling index (C_s) and plasticity index (I_p), is also developed based on the original contribution of Vanapalli et al. (2010). In addition, the Fredlund (1983) equation, which is conventionally used for estimating the heave of expansive soils that eventually reach a fully saturated condition, has been modified such that the equation can be used for estimating the heave of expansive soils exhibiting partially saturated process (i.e. for various values of suction). The proposed techniques are validated by providing comparisons between the predicted and measured swelling pressure and heave (or swell strain) for both of compacted expansive specimens and natural expansive soil sites.

3.2 Background

The SWCC, which is defined as the relationship between the moisture content and the soil suction has been used as a tool to predict unsaturated soil properties (e.g., coefficient of permeability, shear strength, modulus of elasticity, bearing capacity, resilient modulus, and shear modulus). Several SWCC-based prediction models (for example, Fredlund et al. 1994; Vanapalli et al. 1996; Simms and Yanful 2001; Thu et al. 2007; Oh et al. 2009; Alonso et al. 2013, Han and Vanapalli, 2015) have been proposed the last two decades for predicting or estimating unsaturated soil properties. These studies are found to be valuable for practicing engineers to implement mechanics of unsaturated soils in engineering practice.

This philosophy is also promising for predicting the variation of swelling pressure of expansive soils. For example, Pedarla et al. (2012) conducted a series of CVS tests and soil suction tests on two different compacted expansive soils (i.e., Texas and Oklahoma clay). The SWCCs of the soils were then compared with the variation of swelling pressure with respect to the initial soil suction for both Texas and Oklahoma clays. As it is shown in Figure 3.1, there is a strong relationship between the SWCCs and the swelling pressure variation with respect to initial soil suction values. The major features of this correlation can be described as below:

a) Swelling pressure increases with increasing soil suction.
b) Swelling pressure versus soil suction relationship is nonlinear and is typically an S-shape plot across three distinct zones; namely, boundary effect zone, transition zone and residual zone.
c) Swelling pressure is relatively constant or gently increases for the suction values less than the air-entry value, ψ_a (i.e., in the boundary effect zone).

d) Swelling pressure increases at a rapid rate in the transition zone (i.e., from ψ_a to ψ_r) with respect to soil suction.
e) Rate of swelling pressure increment in the residual zone (i.e., when the suction values is greater than ψ_r) decreases with respect to soil suction.

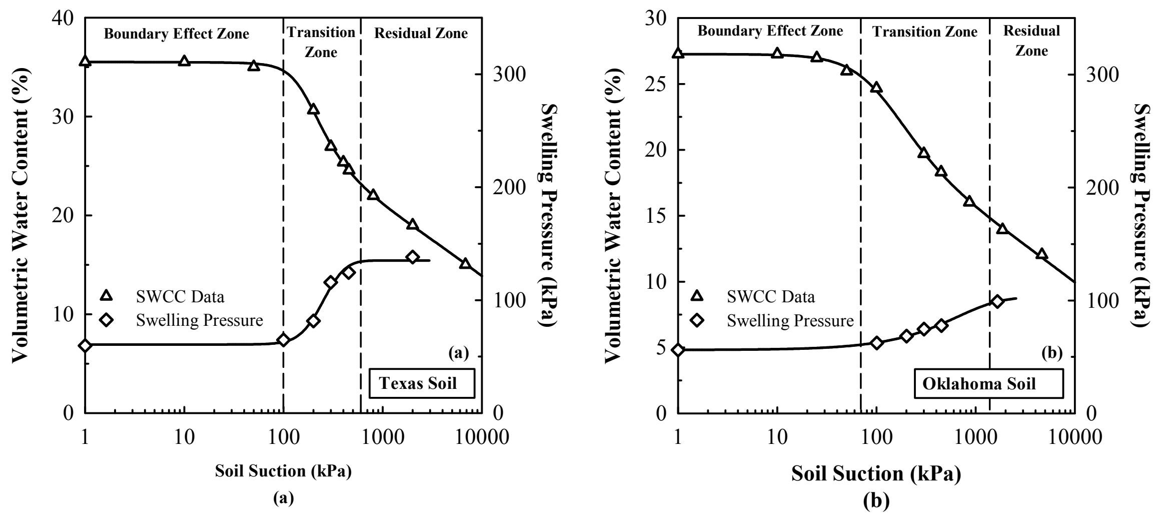

Figure 3.1 Typical relationship between the SWCC and the swelling pressure variation with respect to soil suction: (a) Texas expansive soil, (b) Oklahoma expansive soil

These observations suggest that the swelling pressure of expansive soil can be predicted from the SWCC using a semi-empirical model and reasonable fitting parameters.

Vanapalli et al. (2012) proposed a model to predict the variation of swelling pressure with respect to suction (Equation 3.1) using the SWCC as a tool along with a fitting parameter, *a* based on the experiment results for sand-bentonite mixtures.

$$P_s = \left(\frac{S}{100}\right)^a \cdot \psi \tag{3.1}$$

where, S = degree of saturation, and ψ = soil suction.

The validity of Equation (3.1) was checked with experimental data from Agus (2005). The results showed that there is a strong relationship between the fitting parameter, *a* and dry density, ρ_d.

Since this model (Equation 3.1) was limited to the swelling behavior of sand-bentonite mixtures, the focus of the present study has been extended to the swelling behavior of both compacted and natural expansive soils, which are of interest in geotechnical engineering practice in the design of pavements, foundations and retaining structures associated with expansive soils.

3.3 Swelling pressure predicting models for compacted expansive soils

3.3.1 Proposed prediction model

In the present study, a semi-empirical model is proposed to estimate the variation of swelling pressure with respect to soil suction using the SWCC as a tool for compacted expansive soils by modifying the equation (Equation 3.1) proposed by Vanapalli et al. (2012).

$$P_{sc} = P_{s0} + \beta_c \cdot \psi_i \cdot \left(\frac{S}{100} \right)^2 \tag{3.2}$$

where, P_{sc} = swelling pressure of compacted expansive soil specimen, P_{s0} = intercept on the P_s axis at zero suction value, β_c = model parameter for compacted expansive soil, and ψ_i = initial soil suction.

A constant value of 2 is found suitable for use as the exponent value over the base (S / 100), in the proposed model (Equation 3.2). The same value of 2 has been used in different semi-empirical models to predict the variation of modulus of elasticity (Oh et al. 2009; Vanapalli and Oh 2010; Adem and Vanapalli 2014) and resilient modulus (Han and Vanapalli 2015) with respect to soil suction, for fine grained soils including expansive soils.

3.3.2 Model parameter, P_{s0} for compacted expansive soils

Blight (1965), Kassiff and Shalom (1971) and Gens and Alonso (1992) studies show expansive soils exhibit swelling pressure even after reaching saturated condition. Pedarla et al. (2012) results also show similar trends (see Figure 3.1). In addition, there is evidence that denser soils swell more than loose soils after reaching saturated condition (Kassiff and Shalom 1971).

To accommodate this characteristic of the swelling behavior of expansive soils, parameter P_{s0} has been proposed to represent the contribution of swelling pressure value at zero suction value (i.e., intercept on the P_s axis at zero suction value). The limited testing results (Pedarla et al. 2012) shown in Figure 3.1 suggest using P_{s0} = 55 kPa in Equation (3.2) to provide estimates of swelling pressure for compacted expansive soils in the present research study. Further investigations are required to obtain a better understanding on the relation between P_{s0} value and influencing factors such as the method of compaction, degree of compaction, dry density and mineralogy of the expansive soils.

3.3.3 Model parameter, β_c for compacted expansive soils

A database of the swelling pressures measured at various initial soil suctions has been established for compacted expansive soils from the literature (Khattab et al. 2002; Zhan et al. 2007; Pedarla et al. 2012; Pedarla 2013). The gathered information is then categorized into two groups; namely, the swelling pressures measured from constant volume tests (hereinafter referred to as CVS group), and the swelling pressures measured from free swell then load back tests (hereinafter referred to as FS group). The soil properties of these two groups of soils are summarized in Table 3.1.

Table 3.1 Properties of the compacted expansive soils

Soil ID	Soil Type	G_s	w_{opt} (%)	$\rho_{d,\max}$ (Mg/m^3)	w_l (%)	I_p (%)	c (%)	A	USCS	Remarks
C1	Texas	2.72	17	1.64	55	37	92	0.40	CH	Constant Volume Swell tests
C2	Oklahoma	2.83	24	1.59	41	21	90	0.23	CL	
C3	French Clay	2.70	32	1.35	115	50	80	0.63	CH	
C4	Burleson	2.72	19	1.63	55	37	52	0.52	CH	Free Swell Test
C5	Colorado	2.70	19	1.65	63	42	46	0.91	CH	
C6	Grayson	2.73	24	1.46	75	49	55	0.89	CH	
C7	San Antonio	2.79	22	1.61	67	43	52	0.83	CH	
C8	San Diego	2.72	17	1.74	42	28	23	1.22	CL	
C9	Zaoyang	2.67	20.5	1.66	50.5	31	39	0.79	CH	

Note: w_{opt} is the optimum water content, $\rho_{d,\max}$ is the maximum dry density, G_s is specific gravity, w_l is liquid limit, I_p is plasticity index, c is clay fraction, and A is Activity (= I_p / c)

The values of β_c for different expansive soils can be determined from regression analyses or back-calculations from information of the measured data. Since the variation of swelling pressure with respect to initial soil suction were measured for soil specimens C1, C2, C3, and C9 (see Figure 3.2 and Figure 3.3f), regression analyses is required in the present study to determine the β_c value for each of the variation curves (see Figure 3.2 and Figure 3.3f). The β_c value determined from regression analyses are labeled as $\beta_{c,\text{reg}}$.

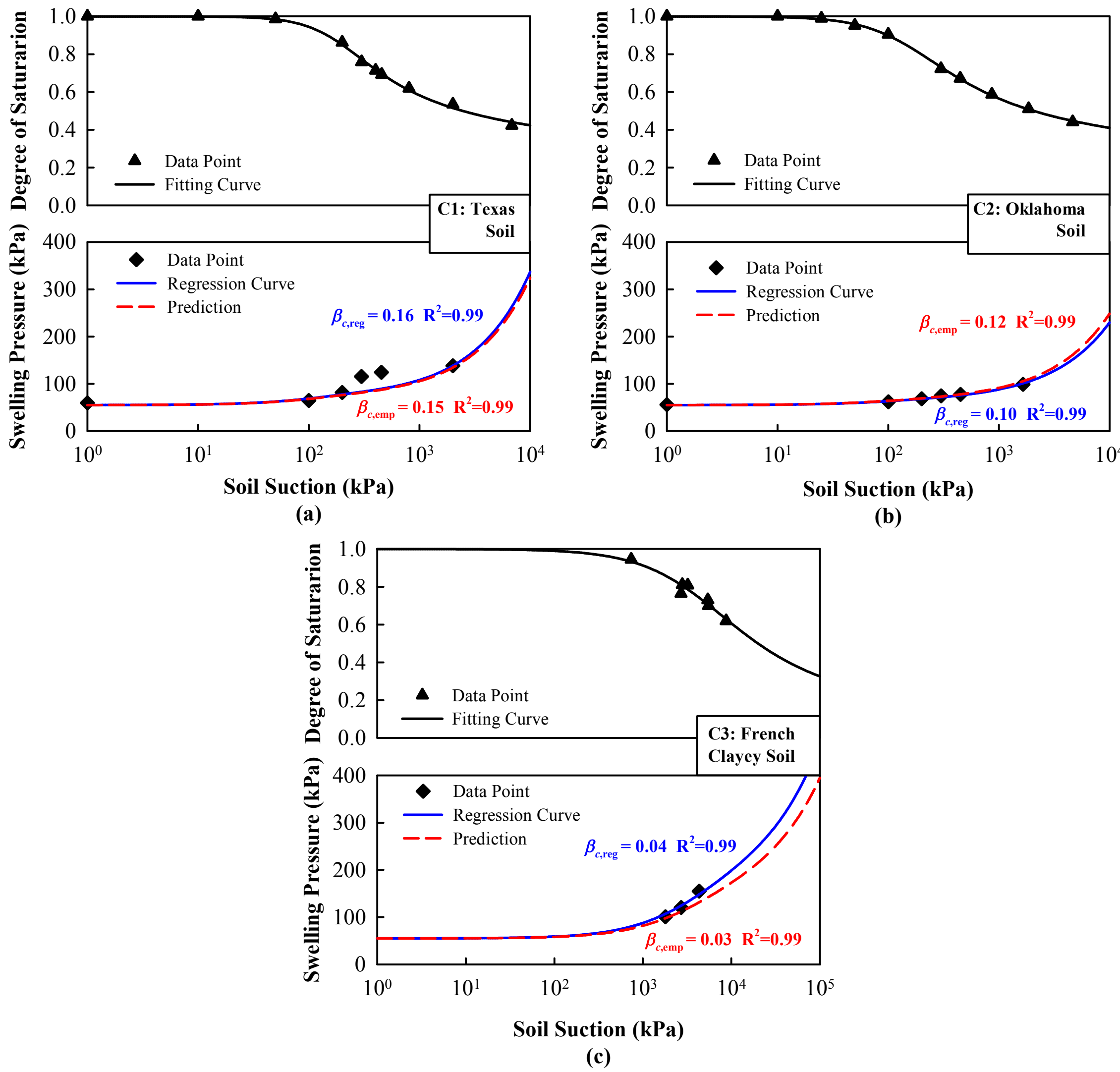

Figure 3.2 Data of compacted soils in CVS group: (a) Texas expansive soil (b) Oklahoma expansive soil (c) French Clayey soil

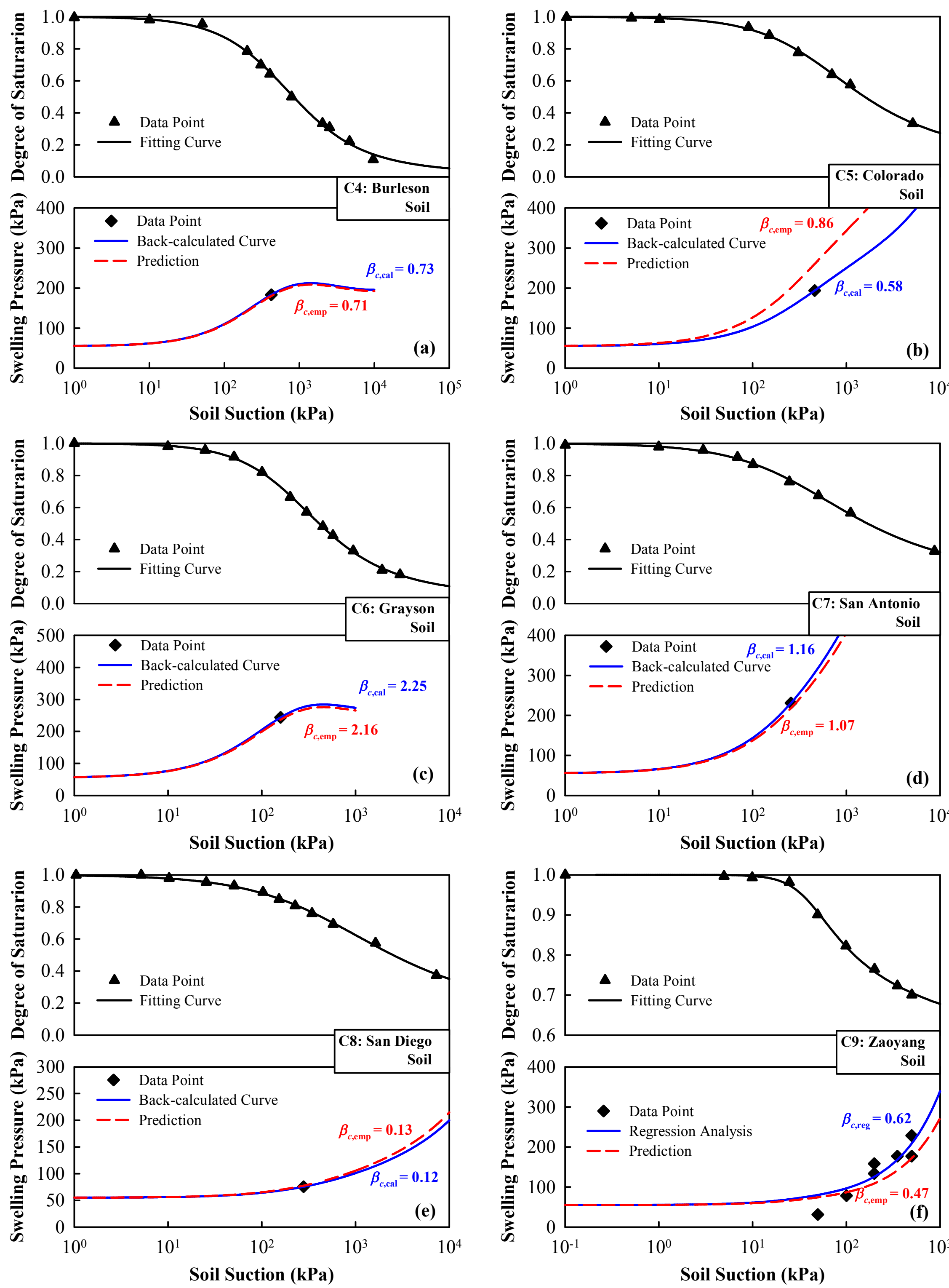

Figure 3.3 Data of compacted soils in FS group: (a) Burleson soil, (b) Colorado soil, (c) Grayson soil, (d) San Antonio soil, (e) San Diego soil, (f) Zaoyang soil

For remainder of the compacted soil specimens (C4 to C8), the swelling pressure was measured at a specific suction value (see Figure 3.3a to Figure 3.3e). The β_c value for these specimens has been directly back-calculated from the swell pressure test results and are referred to as $\beta_{c,\mathrm{cal}}$.

The β_c values ($\beta_{c,\mathrm{reg}}$ and $\beta_{c,\mathrm{cal}}$) determined for different compacted soils (C1 to C9) are summarized in Table 3.2.

Table 3.2 Fitting parameter, β_c for the compacted expansive soils

Soil ID	Soil Type	$\rho_{d,\max}$ (Mg/m^3)	I_p (%)	c (%)	$\beta_{c,\mathrm{reg}}$ or $\beta_{c,\mathrm{cal}}$	$\beta_{c,\mathrm{emp}}$		Remarks
C1	Texas	1.64	37	92	0.16	0.15	Eq. (3.3)	Constant Volume Swell tests
C2	Oklahoma	1.59	21	90	0.10	0.12		
C3	French Clay	1.35	50	80	0.04	0.03		
C4	Burleson	1.63	37	52	0.73	0.71	Eq. (3.4)	Free Swell Test
C5	Colorado	1.65	42	46	0.58	0.86		
C6	Grayson	1.46	49	55	2.25	2.16		
C7	San Antonio	1.61	43	52	1.16	1.07		
C8	San Diego	1.74	28	23	0.18	0.13		
C9	Zaoyang	1.66	31	39	0.62	0.47		

The correlations between the "back-calculated" β_c (i.e., $\beta_{c,\mathrm{reg}}$ and $\beta_{c,\mathrm{cal}}$) values and the soil properties (i.e., maximum dry density, $\rho_{d,\max}$, and plasticity index, I_p) are summarized in Figure 3.4. For the CVS group (specimen C1 to C3), $\beta_{c,\mathrm{reg}}$ increases with increasing $\rho_{d,\max}$ (see Figure 3.4a); however, the behavior of FS group specimens (C4 to C9) shows opposite trend (see Figure 3.4b). This contrary behavior of β_c with respect to $\rho_{d,\max}$ can be attributed to the different compaction methods (Vanapalli et al. 2012). Such a behavior may also be due to the use of different testing procedures (e.g., CVS, FS) for determination of the swelling pressures. In addition, the $\beta_{c,\mathrm{cal}}$ values of FS group show a strong relationship and increase with an increasing in the I_p value (see Figure 3.4b).

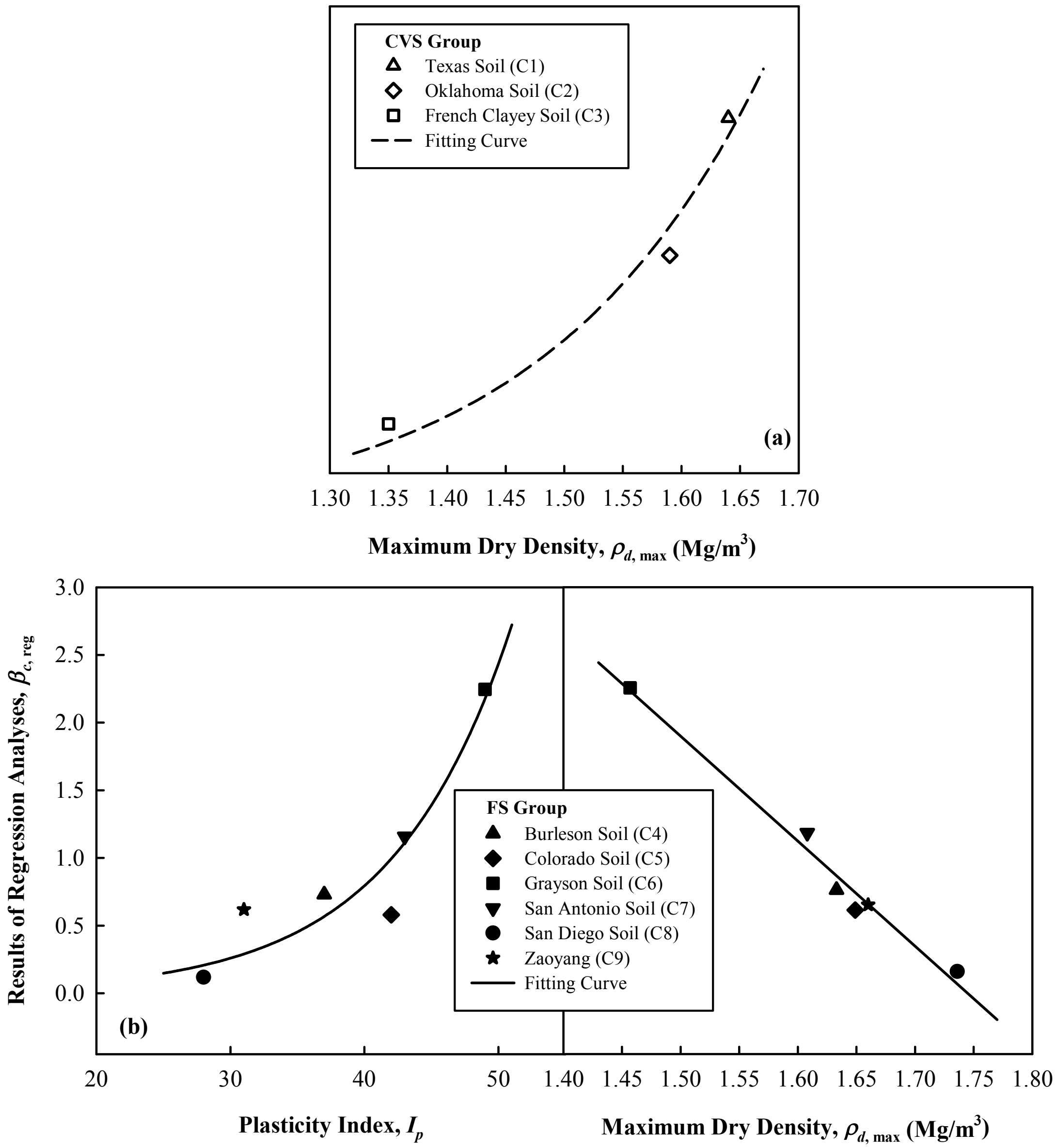

Figure 3.4 The relationships between β_c value and soil properties: (a) CVS group, (b) FS group

Two different empirical equations for CVS and FS tests (Equation 3.3 and Equation 3.4, respectively) are suggested for estimating the β_c values. These equations can be used in Equation 3.2 to predict the swelling pressure. The comparison between the "back-calculated" β_c (i.e., $\beta_{c,\text{reg}}$ and $\beta_{c,\text{cal}}$) and the empirical β_c (i.e., $\beta_{c,\text{emp}}$ calculated with Equation 3.3 or Equation 3.4) shows a good agreement between each other (see Figure 3.2, Figure

3.3, and Figure3.5). The validity of the proposed equations however needs to be checked for other compacted expansive soils, which is discussed in later sections.

$$\beta_{c1} = 0.25\mathrm{e}^{5.306\rho_{d,\max}} / 10000 \qquad \text{(for CVS test)} \tag{3.3}$$

$$\beta_{c2} = \left(0.011\mathrm{e}^{0.107 I_p} - 7.872\rho_{d,\max} + 13.706\right) / 2 \qquad \text{(for FS test)} \tag{3.4}$$

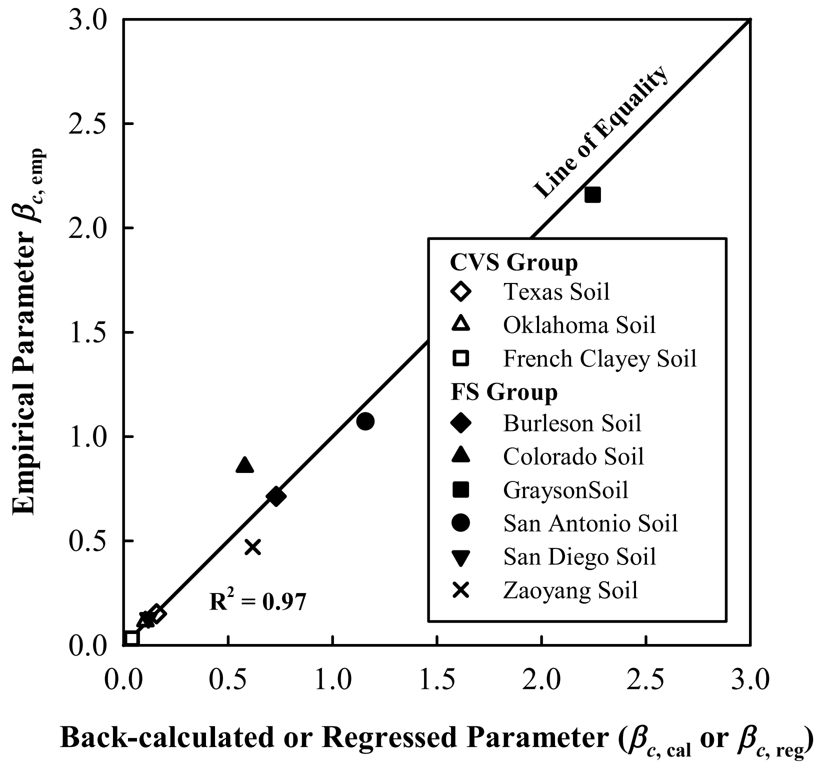

Figure 3.5 Comparison of parameter β_c obtained from back-calculation or regression analyses and that calculated with empirical equation for compacted soils (Equation 3.3 and 3.4)

3.4 Swelling pressure predicting models for natural expansive soils

3.4.1 Proposed prediction model

Fredlund et al. (1980) studies shows that the "corrected" swelling pressure, P_s' equals to the sum of the in-situ net normal stress, $(\sigma_y - u_a)_{field}$ and the "matric suction equivalent",

$(u_a - u_w)_e$. Several research studies in recent years have been dedicated to better understand the correlation between the "matric suction equivalent" and the in-situ soil suction (Vu 2002; Singhal 2010; Singhal et al. 2014). For example, a scale factor, ξ was proposed by Vu (2002) to describe the relationship between the "matric suction equivalent" and the in-situ soil suction (see Equation 3.5). However, no relationship was proposed to estimate the ξ value.

$$P_s' = \left(\sigma_y - u_a\right)_{field} + \xi \cdot \psi_{field} \tag{3.5}$$

where, P_s' = corrected swelling pressure, $(\sigma_y - u_a)_{field}$ = *in-situ* overburden pressure, ψ_{field} = *in-situ* soil suction.

Based on this understanding, a SWCC-based prediction model (Equation 3.6), which is similar in form of the proposed model for compacted soils (Equation 3.2), has been proposed for natural soils.

$$P_{sn} = \left(\sigma_y - u_a\right)_{field} + \beta_n \cdot \psi_{field} \cdot \left(\frac{S}{100}\right)^2 \tag{3.6}$$

where, P_{sn} = swelling pressure of natural expansive soil, β_n = model parameter for natural expansive soil.

3.4.2 Model parameter, β_n for natural expansive soil

In the present paper, investigation on twelve sets of data has been performed to obtain an understanding on the correlation between the model parameter, β_n and the soil index properties. All of the data needed for the present study on natural expansive soils was collected from the literature (Singhal 2010; Pereira et al. 2012). The basic soil properties of these undisturbed expansive soils (Guabirotua materials, Prescott clays and San Antonio clays) are summarized in Table 3.3.

Table 3.3 Properties of the natural expansive soils

Soil ID	Soil Type	G_s	ρ_{dn} (Mg/m^3)	w_l (%)	I_p (%)	c (%)
	Guabirotuba Material					
N1	Sample 1	2.68	1.25	86.0	54.5	58.0
N2	Sample 2	2.67	1.17	100	55.5	75.0
N3	Sample 3	2.69	1.22	81.0	39.4	78.0
N4	Sample 4	2.67	1.04	83.0	47.6	67.0
N5	Sample 5	2.65	1.50	42.0	18.1	30.0
N6	Prescott Clay	2.81	1.66	75	47	45
	San Antonio Clay (high I_p)					
N7	Sample1	2.72	1.70	65	47	50
N8	Sample2	2.72	1.66	69	51	52
	San Antonio Clay (low I_p)					
N9	Sample 1	2.72	1.91	51	34	40
N10	Sample 2	2.72	1.80	52	37	60
N11	Sample 3	2.72	1.94	47	32	43
N12	Sample 4	2.72	1.76	48	33	60

The swelling pressures of Guabirotuba materials were measured from conventional CVS testing procedures suggested by ISRM (1989), without correction for sample disturbance. The overburden pressure applied on the soil specimens during the CVS tests was considered to be 25 kPa since a seating load of the same value was applied. While the swelling pressures of Prescott clays and San Antonio clays were measured from a modified CVS test which was referred to as “overburden constant volume” (OCV) swell test by Singhal et al. (2011; 2014). The key feature of the OCV swell test is that the specimen has to be loaded to field overburden stress prior to wetting. Detailed information on the applied overburden pressure during the OCV tests is available in the literature (Singhal et al. 2011; 2014).

Similar to the investigations conducted on compacted expansive soils, the β_n value for natural soils has been obtained from regression analysis ($\beta_{n,\mathrm{reg}}$) or from back-calculations ($\beta_{n,\mathrm{cal}}$). The swelling pressure variation with respect to initial soil suction was measured for each of Guabirotua materials (i.e., specimen N1 to N5). Hence, regression analyses are required to "back-calculate" a representative β_n value ($\beta_{n,\mathrm{reg}}$) for each of the curves (see Figure 3.6a to Figure 3.6e). For the other natural soil specimens (N6 to N12), β_n values ($\beta_{n,\mathrm{cal}}$) are directly back-calculated from the measured swelling pressure value at a specific suction value(see Figure 3.6f). Table 3.4 summarizes the β_n value for all the twelve expansive soils.

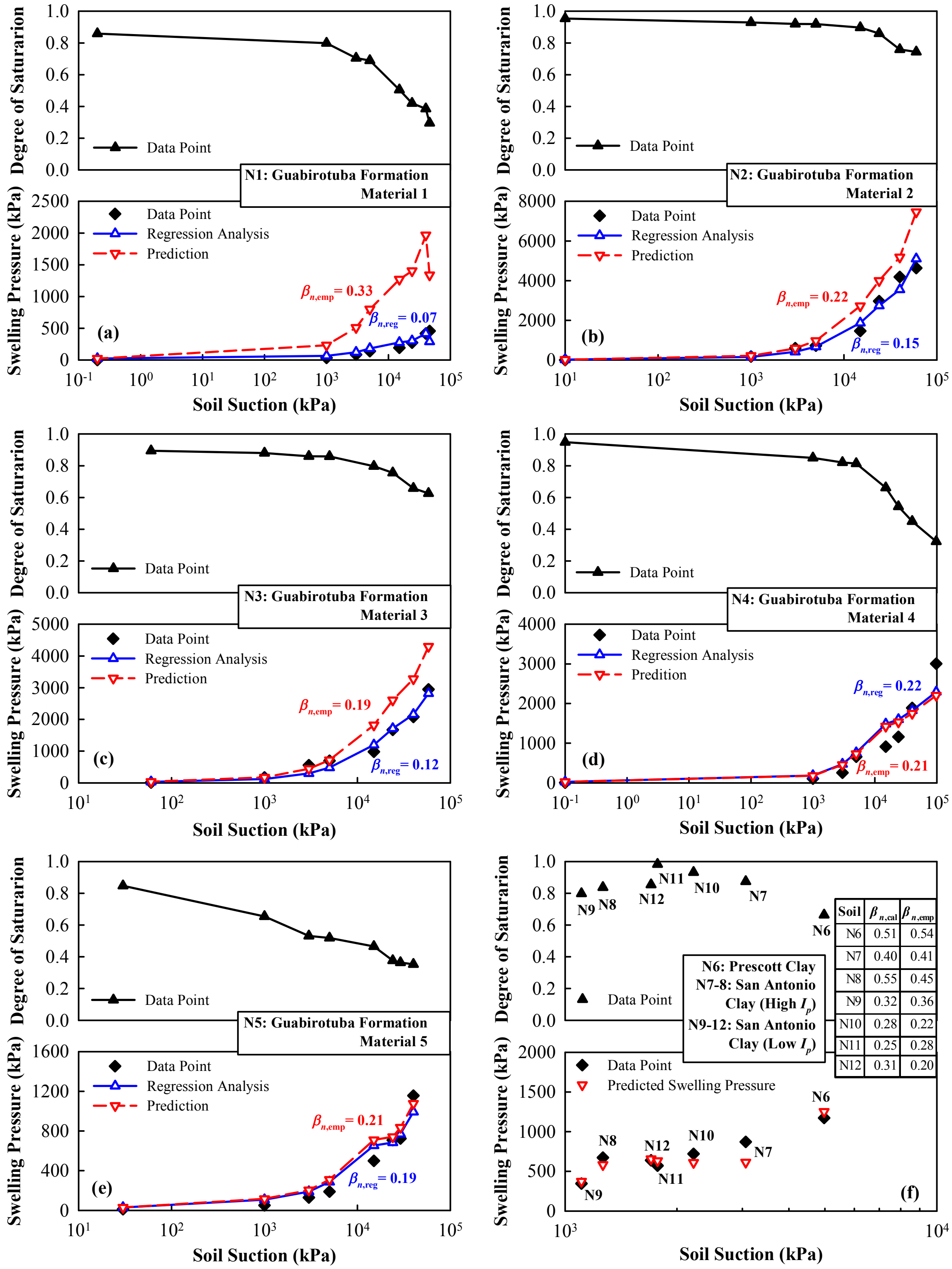

Figure 3.6 Data of undisturbed natural soils: (a) Guabirotuba Formation material 1, (b) Guabirotuba Formation material 2, (c) Guabirotuba Formation material 3, (d) Guabirotuba

Formation material 4, (e) Guabirotuba Formation material 5, (f) Prescott and San Antonio expansive soils

Table 3.4 Fitting parameter, β_n for the natural expansive soils

Soil ID	Soil Type	ρ_{dn} (Mg/m^3)	I_p (%)	c (%)	$\beta_{n,\mathrm{reg}}$ or $\beta_{n,\mathrm{cal}}$	$\beta_{n,\mathrm{emp}}$ Eq. (3.7)	Remarks
	Guabirotuba Material						The β_n values of soil N1-N5 are obtained from regression analyses
N1	Sample 1	1.25	54.5	58.0	0.07	0.33	
N2	Sample 2	1.17	55.5	75.0	0.15	0.22	
N3	Sample 3	1.22	39.4	78.0	0.12	0.19	
N4	Sample 4	1.04	47.6	67.0	0.22	0.21	
N5	Sample 5	1.50	18.1	30.0	0.19	0.21	
N6	Prescott Clay	1.66	47	45	0.51	0.54	The β_n values of soil N6-N12 are obtained from back-calculation
	San Antonio Clay (high I_p)						
N7	Sample1	1.70	47	50	0.40	0.41	
N8	Sample2	1.66	51	52	0.55	0.45	
	San Antonio Clay (low I_p)						
N9	Sample 1	1.91	34	40	0.32	0.36	
N10	Sample 2	1.80	37	60	0.28	0.22	
N11	Sample 3	1.94	32	43	0.25	0.28	
N12	Sample 4	1.76	33	60	0.31	0.20	

There is evidence that the plasticity index, I_p, in-situ dry density, ρ_{dn}, and activity, A (= I_p / c) are strongly related to the swell behavior of expansive soils (for example, Seed et al. 1962; Chen 1975; Erzin and Erol 2007). The correlations between the “back-calculated” model parameters ($\beta_{n,\mathrm{reg}}$ and $\beta_{n,\mathrm{cal}}$) and soil properties (I_p, ρ_{dn} and A) are investigated. Considering these three soil parameters as indicators, an empirical equation (Equation 3.7) is established to estimate the β_n value. The comparison between the “back-calculated” β_n (i.e., $\beta_{n,\mathrm{reg}}$ and $\beta_{n,\mathrm{cal}}$) and the empirical β_n (i.e., $\beta_{n,\mathrm{emp}}$ calculated from Equation 3.7) shows a good agreement (see Figure 3.6 and Figure 3.7).

$$\beta_n = 0.096A^{4.467}\left(2.375\rho_{dn} - 0.017I_p\right) + 0.178 \tag{3.7}$$

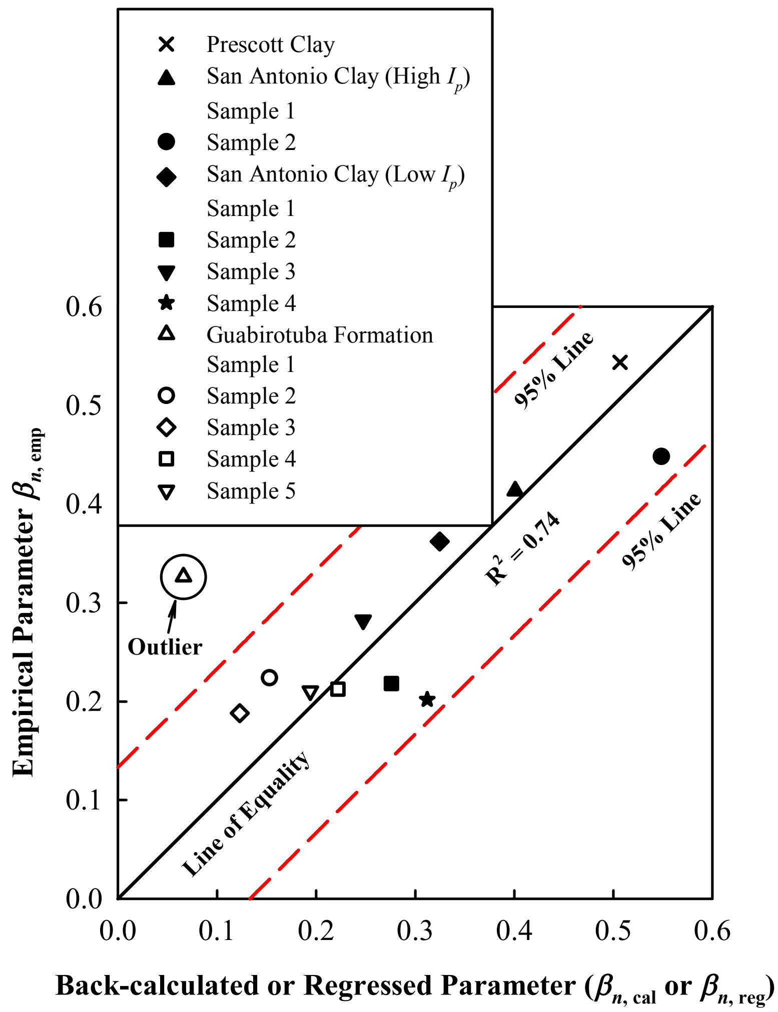

Figure 3.7 Comparison of parameter β_n obtained from back-calculation or regression analyses and that calculated with empirical equation for natural soils (Equation 3.7)

3.5 Empirical equations for predicting swelling index

In addition to the predicted swelling pressure, P_{sp}, reasonable measurement or estimation on swelling index, C_s is required to reliably predict the heave of expansive soils.

Vanapalli and Lu (2012) suggested that the swelling index, C_s increases with increasing plasticity, I_p, and proposed a relationship between C_s and I_p (Equation 3.8) using the published experimental results from the literature. However, Equation (3.8) is limited for I_p values lower than 65. In the present study, a modified relationship (Equation 3.9), is proposed after analyzing more sets data for I_p values within the range of 65 to 100. The C_s – I_p relationship presents an S-shape curve, where the increasing rate of C_s with respect to increasing I_p trends to slow down in high I_p zone (Figure 3.8).

$$C_s = 0.019\,\mathrm{e}^{0.0343 I_p} \tag{3.8}$$

$$C_s = 0.188/[1+\mathrm{e}^{(0.0343-I_p)/15.636}] \tag{3.9}$$

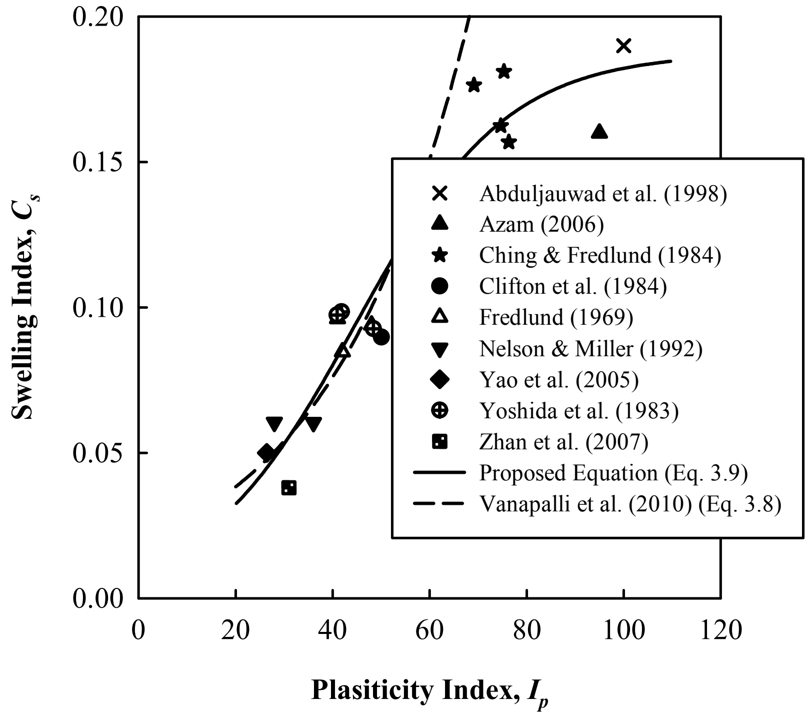

Figure 3.8 Empirical relationships between swelling index, C_s and plasticity index, I_p for expansive soil

3.6 Summary of the proposed techniques

In this chapter, several simple techniques are proposed for predicting/estimating the volume change parameters of expansive soils, including:

(a) Semi-empirical swelling pressure (P_s) prediction models for both compacted and natural expansive soils;
(b) Empirical relationships for estimating the model parameters from basic soil index properties;
(c) Empirical relationships for estimating the swelling index (C_s) from I_p value.

These techniques facilitate the prediction of swelling pressure and ground heave, of which the results can be used to estimate the magnitude of ground heave using the equations proposed from the $e-p$ relation of the consolidation-rebound tests, for example, Fredlund (1983) Equation.

CHAPTER 4

GROUND HEAVE PREDICTION FOR UNSATURATED EXPANSIVE SOILS

4.1 Introduction

Common to all ground heave prediction methods is a need of determination on the initial and final stress states. The oedometer test-based methods consider the swelling pressure as initial stress state, and typically the overburden pressure as final stress state. Since that the swelling pressure is measured at the point where no further swelling is expected for the inundated soil sample (i.e., the soil sample is almost fully saturated) in the oedometer apparatus, the previous assumptions essentially limit the prediction methods to the worst case of scenario (fully saturation). While natural expansive soils are typically in a state of unsaturated condition and rarely reach fully saturated conditions. The heave in expansive soils increases gradually with a decrease in suction (i.e., an increase in degree of saturation) associated with snow or rainfall infiltration. For this reason, there is a need of an approach for estimating the heave during the partially wetting process (see Figure 4.1). An approach is proposed in the present chapter, by modifying the equation proposed by Fredlund (1983) to estimate the variation of heave with respect to different initial suction.

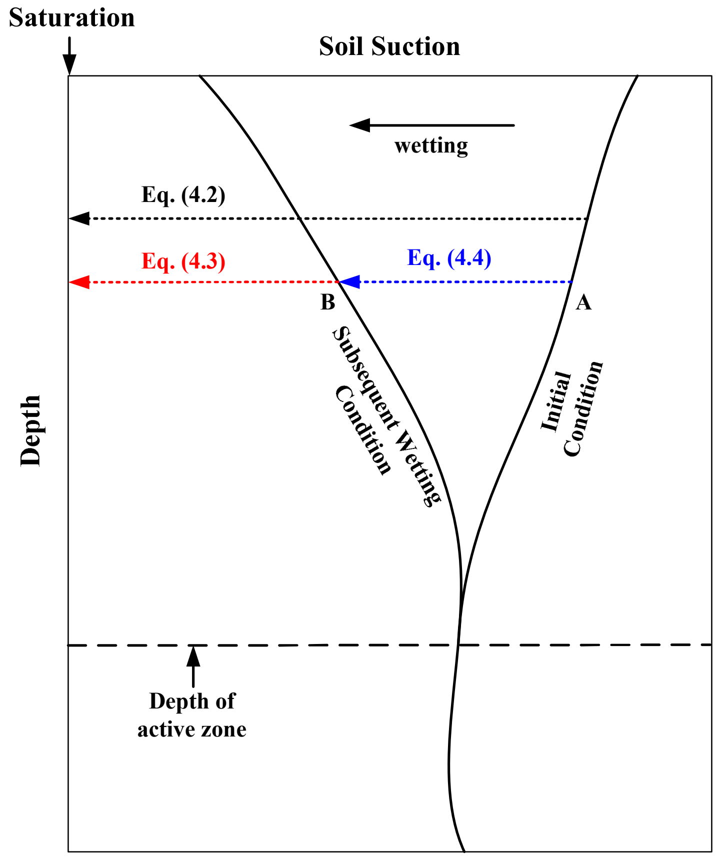

Figure 4.1 Different scenarios of ground heave

4.2 Fredlund (1983) Equation

Fredlund (1983) proposed an equation (Equation 4.1) for estimating the heave in expansive soils by incorporating the corrected swelling pressure, P_s'. The correction procedures on the swelling pressure suggested by Fredlund (1983) are modified from Casagrande's empirical construction (Casagrande 1936) for determining the preconsolidation pressure. The accuracy of ground heave estimation based on Equation (4.1) is largely dependent on appropriate assumptions with respect to the final stress state of the soil. For arriving at conservative estimation of the heave, the expansive soils are mostly assumed to reach a fully saturated condition (i.e. final pore-water pressure, u_{wf} is assumed to be zero or small positive values). Due to this reason, the predicted value is only useful for the estimation of the maximum potential heave of the expansive soil.

$$\Delta h = h \frac{C_s}{1+e_0} \log\left(\frac{P_f}{P_s'}\right) \tag{4.1}$$

where, h is the thickness of soil layer, C_s is swelling index, e_0 is initial void ratio, P_s' is corrected swelling pressure, P_f ($= \sigma_y + \Delta\sigma_y - u_{wf}$) is final stress state, (σ_y is total overburden pressure, $\Delta\sigma_y$ is the change in total stress, u_{wf} is pore-water pressure), and Δh is the heave of the soil layer.

There are some limitations to estimate the heave using Equation (4.1):

1) The testing procedures for obtaining swelling pressure and swelling index are rather cumbersome and also time-consuming;

2) The P_s' value of undisturbed specimen is less reproducible, since the P_s' value is estimated on a logarithm scale and sensitive to the correction procedures;

3) Assumptions are required with respect to the final stress state, which is rather subjective.

4.3 Modified equation

To achieve the objective on predicting the ground heave during partially wetting process, Equation (4.1) can be modified as below:

$$\Delta h_i = h \frac{C_s}{1+e_i} \log\left(\frac{P_0}{P_{si}}\right) \tag{4.2}$$

$$\Delta h_w = h \frac{C_s}{1+e_w} \log\left(\frac{P_0}{P_{sw}}\right) \tag{4.3}$$

$$\Delta h = \Delta h_i - \Delta h_w = C_s h \left[\frac{\log(P_0/P_{si})}{1+e_i} - \frac{\log(P_0/P_{sw})}{1+e_w} \right] \tag{4.4}$$

where, e_i is void ratio of initial condition, e_w is void ratio of subsequent wetting condition, P_{si} is swelling pressure of the soil at initial condition, P_{sw} is swelling pressure of the soil at subsequent wetting condition, P_0 (= σ_y + $\Delta\sigma_y$) is the overburden pressure, Δh_i is the maximum potential heave at initial condition, and Δh_w is the maximum potential heave at a subsequent wetting condition, Δh is the heave in the expansive soil from initial condition to the subsequent wetting condition.

As shown in Figure 4.1, Equation (4.2) predicts the maximum potential heave of the soil at initial condition; Equation (4.3) predicts the maximum potential heave of the soil at subsequent wetting condition. The difference (Equation 4.4) between these two estimates is the ground heave due to gradual wetting (say for example, from Point A to Point B). However, it is challenging to trace the variation of void ratio during the wetting process. The void ratio of subsequent unsaturated condition can be estimated by solving the equation set (i.e., Equation 4.4 and 4.5).

$$\Delta h = \frac{e_i - e_w}{1 + e_i} h \tag{4.5}$$

The proposed approach (Equation 4.4, 4.5) can be used along with the swelling pressure prediction models presented in Chapter 3 to predict the variation of the ground heave as the wetting process continues.

CHAPTER 5

VALIDATION OF PROPOSED TECHNIQUES FOR ESIMATING THE SWELL PRESSURE AND THE HEAVE IN EXPANSIVE SOILS FROM CASE STUDIES

5.1 Introduction

In Chapter 3 and Chapter 4, simple techniques have been proposed:

(i) to predict variation of swelling pressure with respect to suction using the SWCC as a tool (i.e. Equation 3.2 and 3.6);

(ii) to estimate the model parameters of the proposed swelling pressure prediction models (i.e. Equation 3.3, 3.4 and 3.7);

(iii) to estimate the swelling index, C_s from plasticity index, I_p (i.e. Equation 3.9); and

(iv) to estimate the heave (or swell strain) in unsaturated expansive soils (i.e. Equation 4.4 and 4.5).

In this chapter, the validity of the proposed simple techniques is verified on the information of several case studies.

5.2 Case studies on compacted soils

Comparisons are provided between the estimated heave (or swell strain) and the measured heave (or swell strain) for six sets of data on compacted soils. Table 5.1 summarizes the soil properties, initial conditions, and Free Swell (FS) test results of the compacted soils, which were originally investigated by Pedarla et al. (2012) and Lin and Cerato (2012a; 2012b).

Table 5.1 Properties of the six types of compacted expansive soils

Soil Type	Basic Soil Property						Free Swell Test		
	G_s	w_{opt} (Mg/m^3)	$\rho_{d,\max}$ (Mg/m^3)	w_l (%)	I_p (%)	USCS	Initial Condition	Token Load (kPa)	$\Delta H/H$ (%)
Texas	2.72	17	1.64	55	37	CH	OMC-MDD	1	9.5
Oklahoma	2.83	24	1.59	41	21	CL	OMC-MDD	1	5.2
Carnisaw	2.68	26.2	1.65	59	27	MH	OMC-MDD	1	2.3
Hollywood	2.78	20.6	1.70	54	34	CH	OMC-MDD	1	5.6
Heiden	2.77	24.2	1.58	70	49	CH	OMC-MDD	1	9.3
Eagle Ford	2.71	27.1	1.45	92	57	CH	OMC-MDD	1	12.7

The swell strains of the six expansive soils are re-evaluated using Equation (4.1), where the swelling index, C_s and the swelling pressure, P_s are required. The C_s values are estimated from I_p using Equation (3.9). The swelling pressures are estimated using the proposed prediction model (Equation 3.2). Since two equations (i.e. Equation 3.3 and 3.4) have been suggested in the present research to determine the model parameter, β_c, two different estimations on P_s and on swell strain, ε_s afterwards are obtained. The detailed procedures of re-evaluation on the swell strains are presented in Table 5.2.

Table 5.2 Prediction on swelling pressure and swell strain of the compacted expansive soils

				Estimation 1			Estimation 2		
Soil Type	[1]ψ_i (kPa)	[2]S (/100)	C_s	β_c	P_s (kPa)	$\Delta H/H$ (%)	β_c	P_s (kPa)	$\Delta H/H$ (%)
			Eq. (3.9)	Eq. (3.3)	Eq. (3.2)	Eq. (4.1)	Eq. (3.4)	Eq. (3.2)	Eq. (4.1)
Texas	2000	0.52	0.0718	0.15	135	8.8	0.69	420	11.0
Oklahoma	1650	0.52	0.0342	0.12	106	3.7	0.65	341	4.7
Carnisaw	469	0.90	0.0462	0.16	116	5.6	0.44	224	6.4
Hollywood	758	0.81	0.0635	0.21	159	8.2	0.35	230	8.8
Heiden	1120	0.80	0.1073	0.11	135	12.5	1.67	1260	18.4
Eagle Ford	1126	0.75	0.1296	0.05	90	12.9	3.60	2363	22.8

[1] the initial suction equal to the soil suction at OMC-MDD condition, which has been measured for each soil
[2] the S values are obtained from the respective SWCC

Figure 5.1 provides comparisons between the estimated swell strain, $\varepsilon_{s,\mathrm{emp}}$ and the measured swell strain, $\varepsilon_{s,\mathrm{mea}}$. The values of $\varepsilon_{s,\mathrm{emp}}$ calculated with $\beta_{c,\mathrm{emp}}$ estimated from Equation (3.3) provide good comparison with the measured values. While, the calculated swell strains using $\beta_{c,\mathrm{emp}}$ estimated from Equation (3.4) are approximately 1.63 times greater than the measured values (see Figure 5.1).

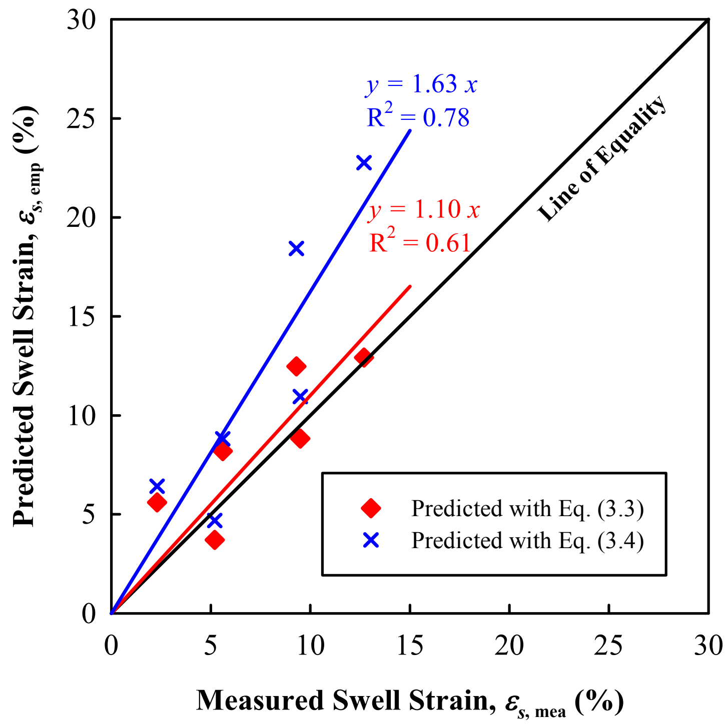

Figure 5.1 Comparison between the estimated swell strain and the measured swell strain

These differences may be attributed to the fact that the two different empirical equations (Equation 3.3 and 3.4) are respectively proposed based on the results obtained from different testing procedures (namely, CVS and FS). The empirical equation (Equation 3.3) developed from CVS test results provide reasonable swell strain estimation in comparison to the empirical equation (Equation 3.4) developed from FS test results. Such a performance of the proposed equations is reasonable as CVS tests better simulate the swelling process of expansive soils while the swelling pressures measured from FS tests are always greater than that measured from CVS tests (Sridharan et al. 1986; Fredlund and Rahardjo 1993; Nelson and Miller 1992).

5.3 Case studies on natural sites

Several published case studies information from various regions of the world have been investigated to check the validity of the proposed techniques for predicting the swelling pressure and heave of natural expansive soils. These case studies include the information of eight different natural expansive soil sites located at six different countries (i.e., Australia, Canada, China, Saudi Arabia, USA, and the UK). A general description of these field sites is presented in Table 5.3. The required information such as the SWCCs, soil suction profiles (or water content profiles), and basic soil properties of the respective sites are summarized from the published literature.

Table 5.3 Information of the eight field sites

Case study	Location	Predominate soil	USCS	Test type	References
A	Chattenden, Kent, UK	London clay	CH	Tree removal	Crilly et al. 1992; Crilly and Driscoll 2000
B	Al-Qatif, Eastern Province, Saudi Arabia	Al-Qatif clay	CH	Oedometer Tests Artificial infiltration	Abduljauwad et al. 1998; Azam 2006; Azam and Wilson 2006
C	Newcastle, New South Wales, Australia	Maryland clay	CH	Oedometer Tests Natural infiltration	Fityus et al. 2004
D	Adelaide, South Australia, Australia	Keswick clay	CH	Tree removal	Richards et al. 1983
E	Arlington, Texas, US	Arlington clay	CL-CH	Artificial infiltration	Briaud et al. 2003
F	Al-Ghat, Riyadh, Saudi Arabia	Al-Ghat shale	CH	Oedometer Tests Artificial infiltration	Al-Shamrani and Dhowian 2003
G	Zaoyang, Hubei, China	Zaoyang expansive soil	CL-CH	Oedometer Tests Artificial infiltration	Ng et al. 2003
H	Regina, Saskatchewan, Canada	Regina clay	CH	Oedometer Tests Natural infiltration	Azam et al. 2013

5.3.1 Case Study A: London clay site, UK

The investigated site is located at approximately 45 km east of central London. The site is underlain by London clay to a depth of at least 44 m below natural ground water table (Crilly et al. 1992). The soil properties of this site are, specific gravity, $G_s = 2.75$, plasticity index, $I_p = 63$, and clay fraction, $c = 62\%$ (Crilly et al. 1992). Both the drying and wetting SWCCs of the London clay have been fitted with the Fredlund and Xing (1994) equation, as presented in Figure 5.2.

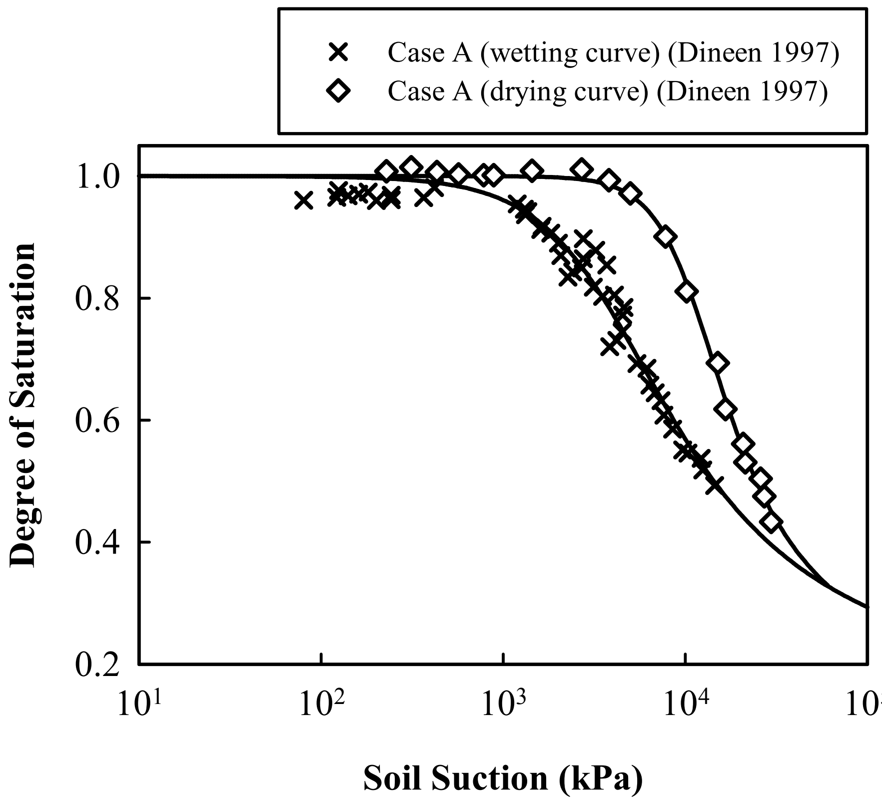

Figure 5.2 The SWCCs used for Case Study A

The removal of trees on the site led to changes in the water contents and contributed to vertical movements of the ground. Crilly and Driscoll (2000) presented the results of long-term field monitoring on the open ground within the investigated area. These field tests were categorized into two groups which are referred to as Group A (away from trees), and Group B (in areas of fallen trees) with respect to the location where the test was conducted.

The monitoring data of water content, soil suction, and ground heave are summarized in Figure 5.3 and Figure 5.4, for Group A and Group B, respectively.

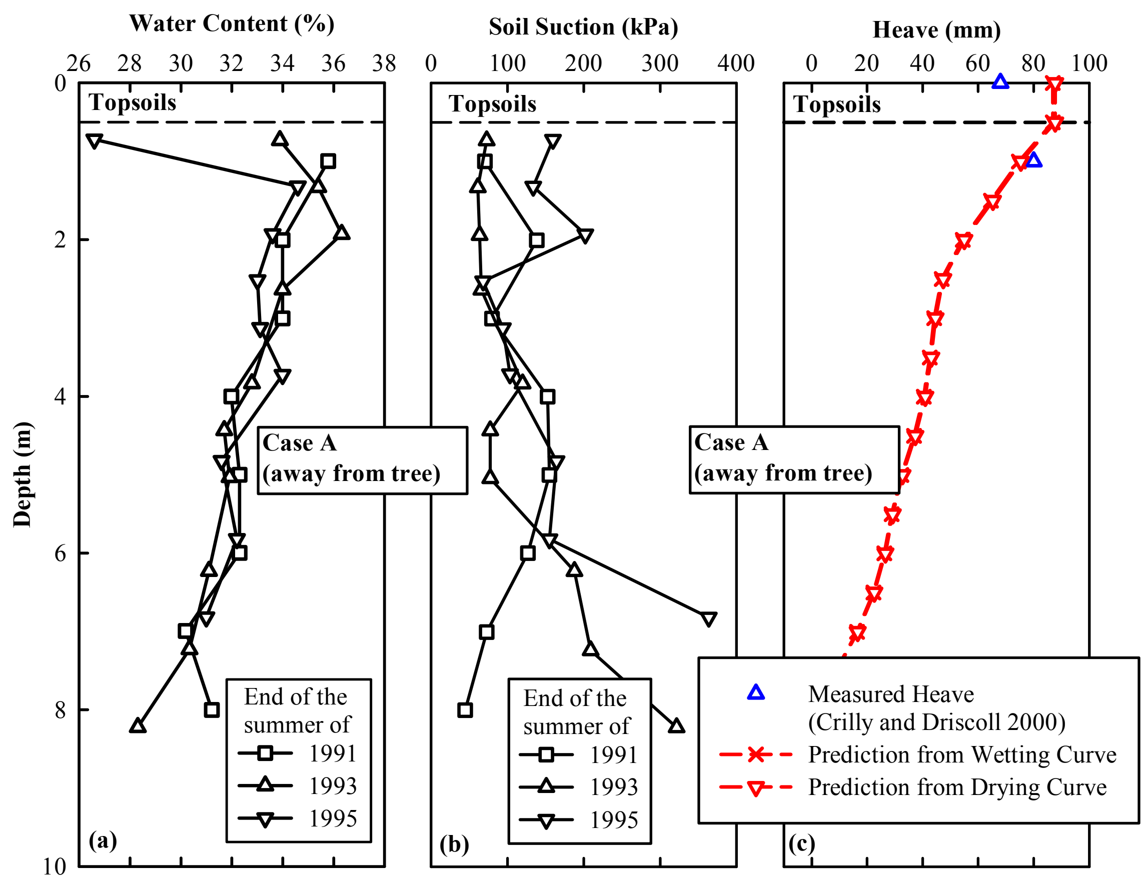

Figure 5.3 Information of London clay site (Group A, away from tree): (a) water content, (b) soil suction and (c) ground heave (from Crilly and Driscoll 2000)

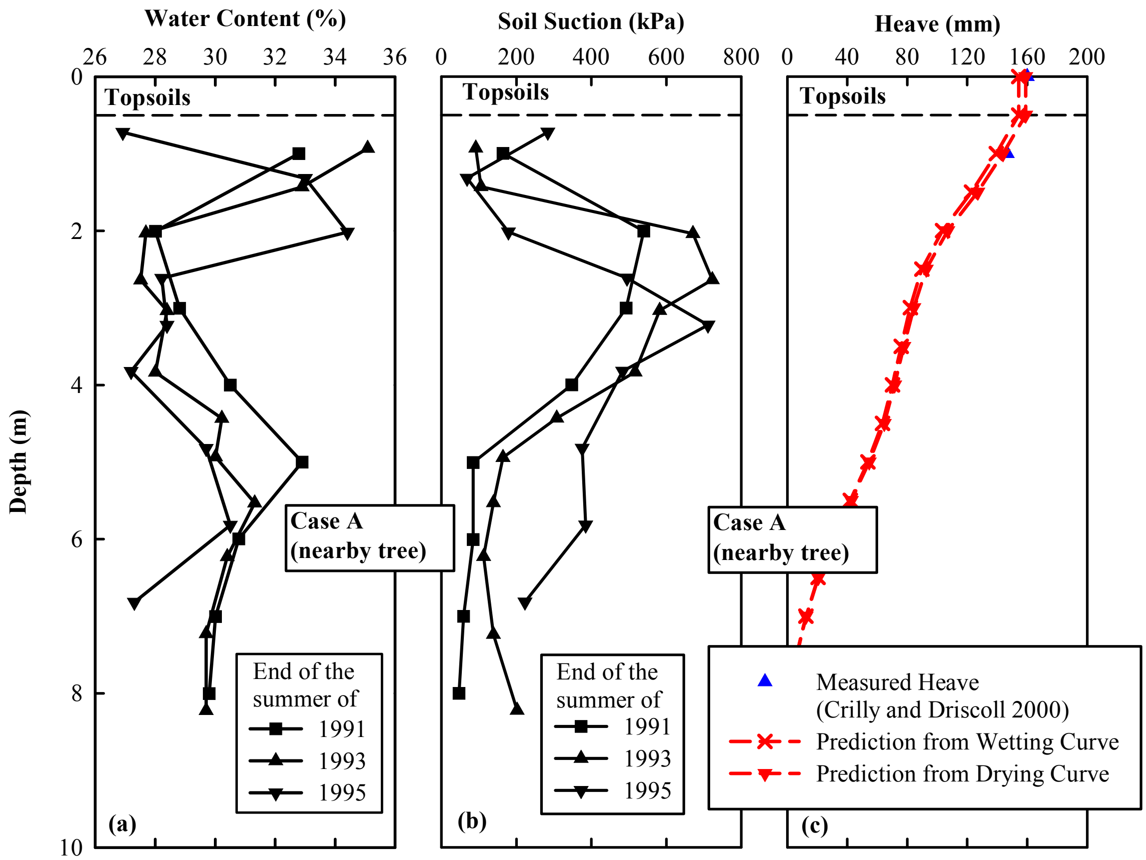

Figure 5.4 Information of London clay site (Group B, nearby tree): (a) water content, (b) soil suction and (c) ground heave (from Crilly and Driscoll 2000)

The detailed calculation procedures of the ground heave estimation, with application of the proposed techniques, are presented in the Appendix Table A.1. The estimated and measured heave variation shows a good agreement in the shallow depth (see Figure 5.3c and Figure 5.4c). From these figures it can be observed that the heave estimation using the drying curve of SWCC is slightly higher in comparison to that using the wetting curve. The deviation between these two estimations is due to the effect of hysteresis, which typically results in a higher value of degree of saturation, S on the drying curve, than the wetting curve, for the same suction value. However, in the present case study, the difference between the two estimations is negligible, since that the effect of hysteresis for the in-situ soil suction variation from 50 to 700 kPa is insignificant (see Figure 5.2). The assumptions used in the analysis of this case study are summarized below:

(i) The upper 0.5 m-thick layer is considered to be non-expansive top soil; such an assumption has been made because typically the top layer is with organic material due to vegetation effects; in addition, there is no investigation results of the soil properties within this depth;

(ii) The variation of ρ_{dn} and e_i needed for the estimation of ground heave are back-calculated from mass-volume relationship information (i.e. density, water content, degree of saturation, specific gravity);

(iii) The heave was estimated within the top 8 m of the soil layer as the suction profiles and water content data are only available for this zone; in addition, it is believed that the heave below this zone will be insignificant due to influence of the overburden pressure.

5.3.2 Case Study B: Al-Qatif clay, Saudi Arabia

Expansive clays in the eastern Saudi Arabia such as the Al-Qatif clay are typically fissured and contain high quantities of anhydrous calcium sulfate (Azam and Wilson 2006). The swelling pressure of undisturbed Al-Qatif clay has been measured by several investigators (Abduljauwad et al. 1998; Azam 2006; Azam and Wilson 2006).

SWCCs

The drying curve of SWCC was established by using axis translation technique and filter-paper method (Elkady et al. 2013a). The measured data has been fitted with Fredlund and Xing (1994) equation (see Figure 5.5).

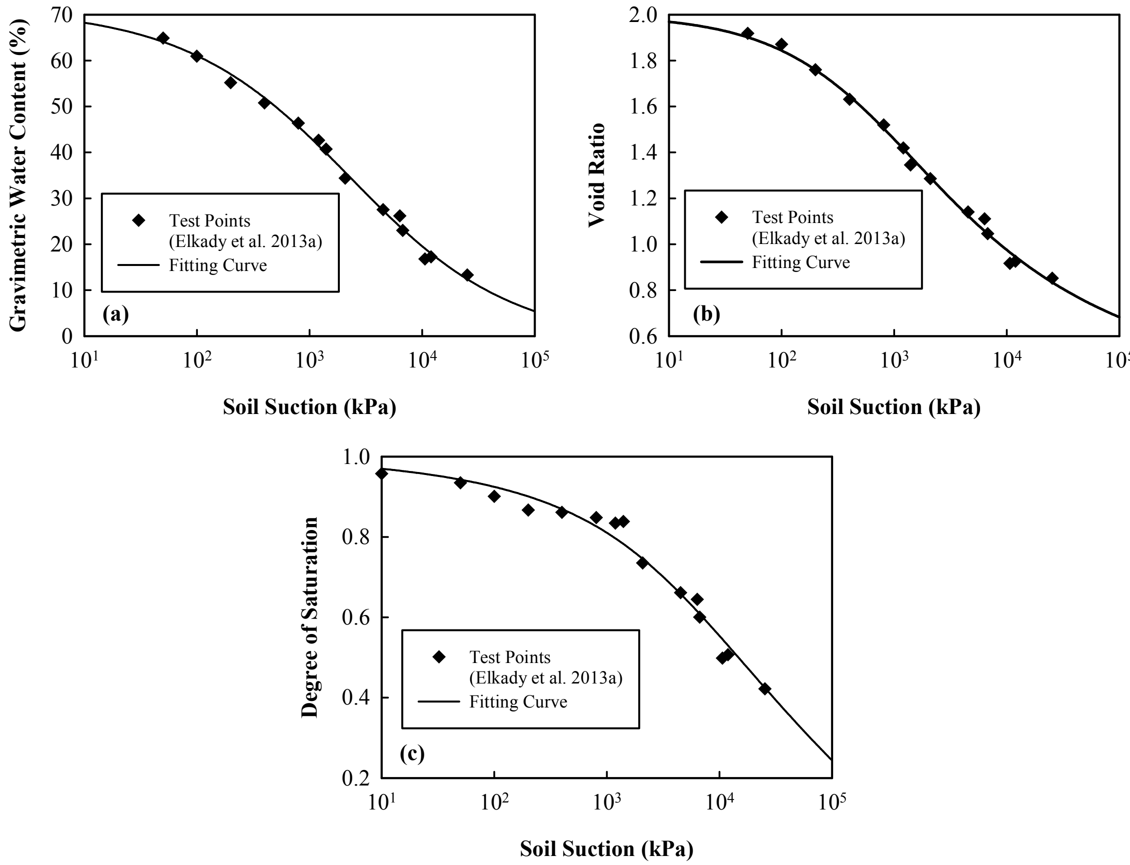

Figure 5.5 (a) SWCC in terms of gravimetric water content, (b) void ratio versus soil suction content relationship, (c) SWCC in terms of degree of saturation

Laboratory Test Results

Azam (2006) and Azam and Wilson (2006) measured swelling pressures using the CVS tests as per standard ASTM (D4546-96) on undisturbed block samples collected from two different sites (i.e. Site 1 and 2). Both conventional circular rings (Height × Diameter = 19.5 mm × 70 mm) as well as large-scale circular and square moulds (Height × Diameter or Side = 85 mm × 300 mm) were respectively used in the oedometer tests to determine the swelling pressures. The large-scale specimens effectively capture the fissures and also alleviate the size effects when measuring the swell pressures. As expected, the swelling pressures measured with smaller size conventional rings were greater than that measured with large-size moulds.

The soil properties as well as the predicted swelling pressures are summarized in Table 5.4. For the soil samples collected from Site 1, the predicted swelling pressure, P_{sp} is close to the value measured with conventional oedometer (using small-size moulds). However, for the soil samples collected from Site 2, the predicted swelling pressures, P_{sp} is less than the measured values as large-size moulds have been used.

Table 5.4 Case study of Al-Qatif clay based on laboratory tests

<table>
<tr><th colspan="3">Measured and Predicted Soil Properties</th><th>Site 1
(Azam & Wilson, 2006)</th><th>Site 2
(Azam, 2006)</th></tr>
<tr><td colspan="3">Water Content, w (%)</td><td>28</td><td>42</td></tr>
<tr><td colspan="3">Specific Gravity, G_s</td><td>2.85</td><td>2.75</td></tr>
<tr><td colspan="3">Dry Density, ρ_d (Mg/m^3)</td><td>1.37</td><td>1.10</td></tr>
<tr><td colspan="3">Clay Fraction, c (%)</td><td>70−75</td><td>70−75</td></tr>
<tr><td colspan="3">Initial Void Ratio, e_0</td><td>1.1</td><td>1.5</td></tr>
<tr><td colspan="3">Degree of Saturation, S</td><td>0.75</td><td>0.77</td></tr>
<tr><td colspan="3">Liquid Limit, w_l (%)</td><td>115</td><td>150</td></tr>
<tr><td colspan="3">Plasticity Index, I_p (%)</td><td>60</td><td>95</td></tr>
<tr><td colspan="3">Activity, A</td><td>0.857</td><td>1.310</td></tr>
<tr><td colspan="3">[1]Initial Soil Suction, ψ_i</td><td>1989.75</td><td>1614.24</td></tr>
<tr><td colspan="3">[2]Overburden Pressure, (σ_y - u_a) (kPa)</td><td>7</td><td>7</td></tr>
<tr><td colspan="3">Parameter, β_n</td><td>0.286</td><td>0.498</td></tr>
<tr><td colspan="3">Predicted P_s (kPa)</td><td>327</td><td>484</td></tr>
<tr><td rowspan="3">Measured P_s (kPa)</td><td colspan="2">Conventional sample</td><td>320</td><td>555</td></tr>
<tr><td rowspan="2">Large-Scale Sample</td><td>Circular</td><td>245</td><td>360</td></tr>
<tr><td>Square</td><td>–</td><td>217</td></tr>
</table>

[1] The initial soil suction of each soil sample is estimated from the SWCC based on known degree of saturation

[2] The overburden pressures are taken as the token load applied during the testing procedures

Field Study Results

The clay content of Al-Qatif clay ranges from 70 to 75%, and the natural dry density, ρ_{dn} ranges around 1.2 Mg/m3 (Abduljauwad et al. 1998; Azam 2006; Elkady et al. 2013b). Abduljauwad et al. (1998) conducted a series of tests to determine the variations of soil index properties, swelling pressure and soil suction with respect to the depth at a location. The swelling pressures were measured from CVS tests on the undisturbed soil samples collected from different depths. The Atterberg limits of these soil samples were also determined from laboratory tests; Figure 5.6a summarizes all these test results. The initial soil suction profile and the soil suction profile after conducting in-situ infiltration test are summarized in Figure 5.6b.

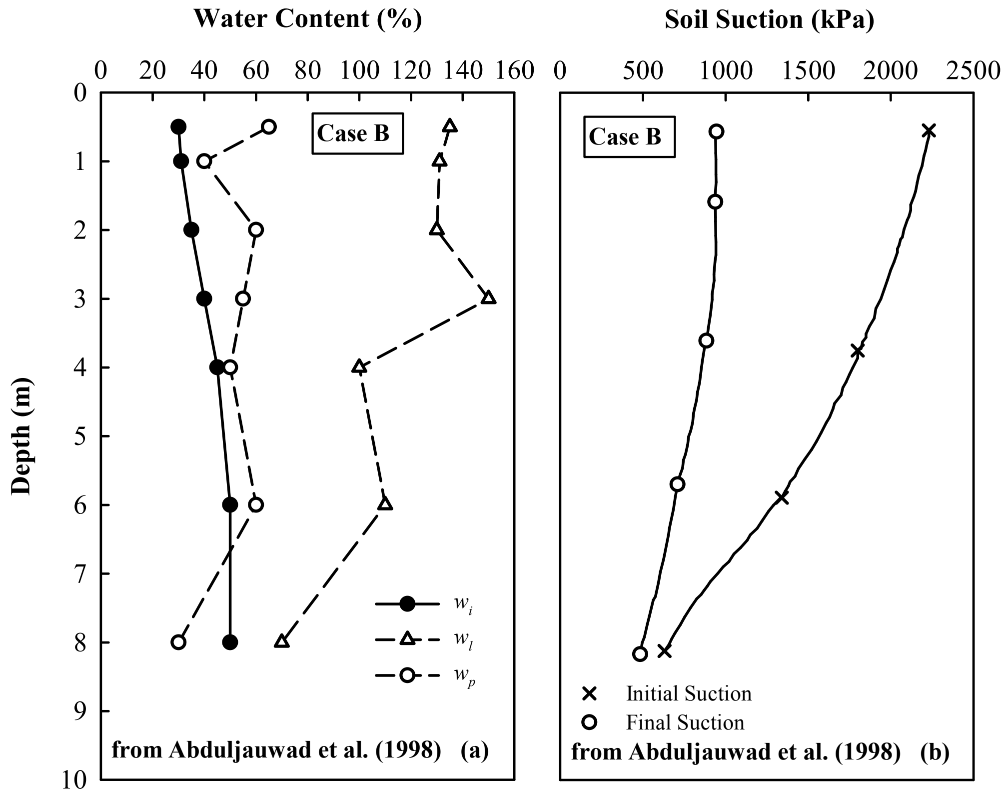

Figure 5.6 Measured and predicted soil properties: (a) water content and Atterberg limits, (b) suction profiles

The information summarized in the earlier paragraph is employed to estimate the variation of swelling pressure with respect to depth using the proposed swelling pressure prediction model (Equation 3.6) and the empirical equation (Equation 3.7) for estimating the model parameter, β_n. The detailed calculation procedures are presented in the Appendix Table A.2. It can be observed from Figure 5.7 that the swelling pressure prediction is over-estimated within the top and bottom layers while under-estimated within the middle layers. It is believed that this discrepancy is mainly due to the non-uniform distribution of soil properties. In other words, the SWCCs used in the present study may not fully represent the real situation of the entire layers of the investigated site. Nevertheless, the predicted swelling pressure profile provides a conservative prediction within the top 2m of the soil.

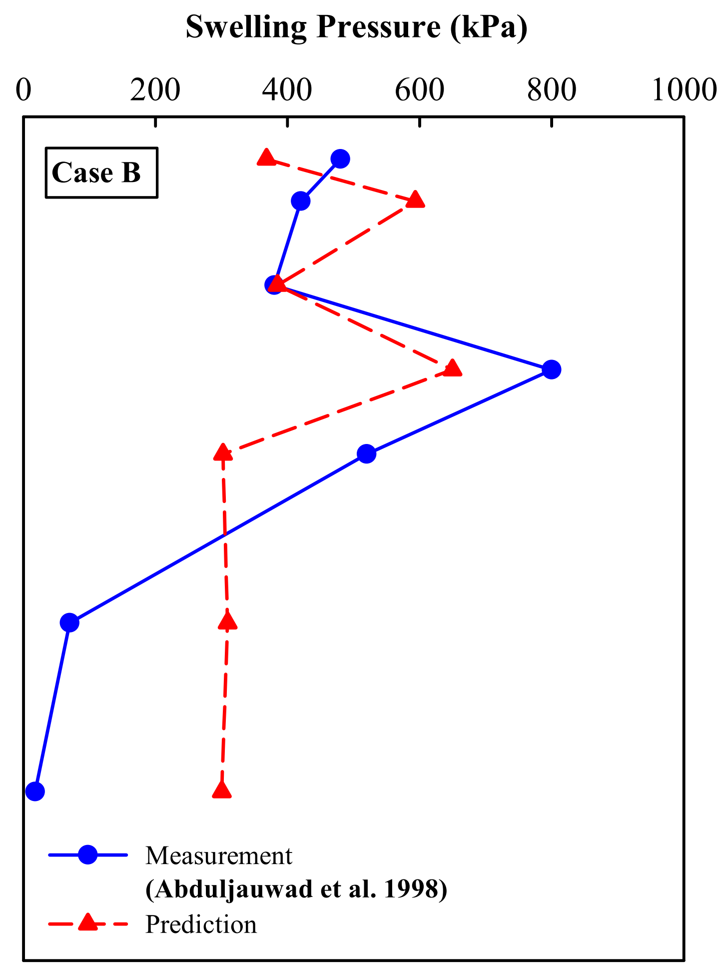

Figure 5.7 Measured and predicted swelling pressure

5.3.3 Case Study C: Maryland clay site, Australia

The Maryland site lies 10 km west of the city of Newcastle, Australia. Long-term field measurements on soil suction and heave lasted from 1993 to 2000 (Fityus et al. 2004). The variation of basic soil properties (Atterberg limits, natural dry density and clay fraction) with respect to depth is also available in the literature. The SWCCs of Maryland clay are shown in Figure 5.8. Two different values of net normal stress (i.e., 10 kPa and 400 kPa) were employed to measure the SWCCs (Li et al. 2007). In the present study, the SWCC measured under net normal stress of 10 kPa is utilized since it better represents the in-situ condition.

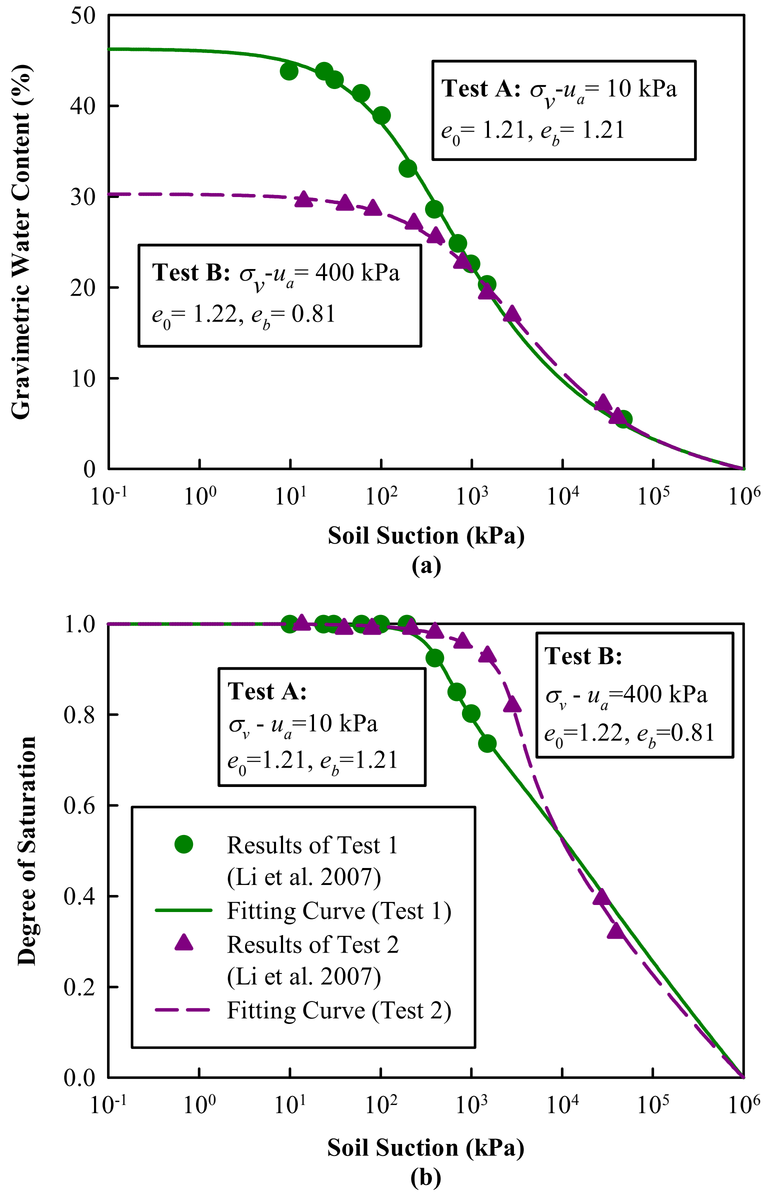

Figure 5.8 SWCCs of Maryland clay, (a) degree of saturation, (b) gravimetric water content

Two sets of CVS tests were conducted on soil specimens with different initial water contents. The measured swelling pressures are compared with that predicted using the proposed techniques. Results summarized in Figure 5.9 shows that, for w_i less than 20 %, the predicted variation is approximately in between the two measured curves.

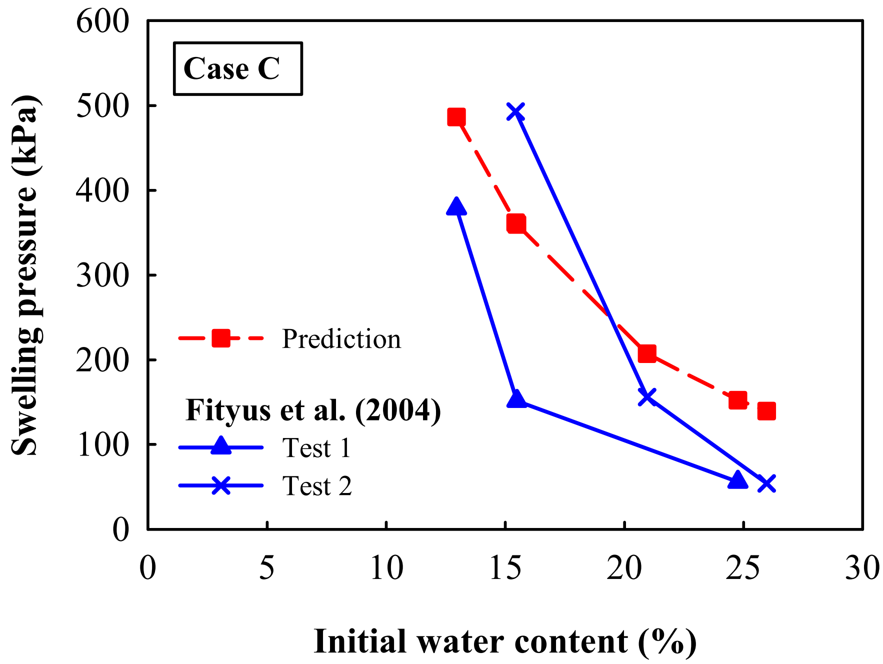

Figure 5.9 Measured and predicted swelling pressure of Maryland clay

The suction profile summarized in Figure 5.10 shows that the expansive soil at the site was in a state of unsaturated condition during the entire period of monitoring from 1993 to 2000. In this case, the modified equation (i.e. Equation 4.4) that facilitates the prediction of heave of unsaturated expansive soils is used to estimate the ground heave. In the present case study, the envelope of the suction profile may not truly represent the initial and final conditions of the investigated site. However, the suction envelope can be used to approximately estimate the maximum ground heave within the monitoring period. In addition, the depth of active zone can be estimated as about 2 m from the suction profile, since that the soil suction below this depth maintains a relative constant value over the entire monitoring duration.

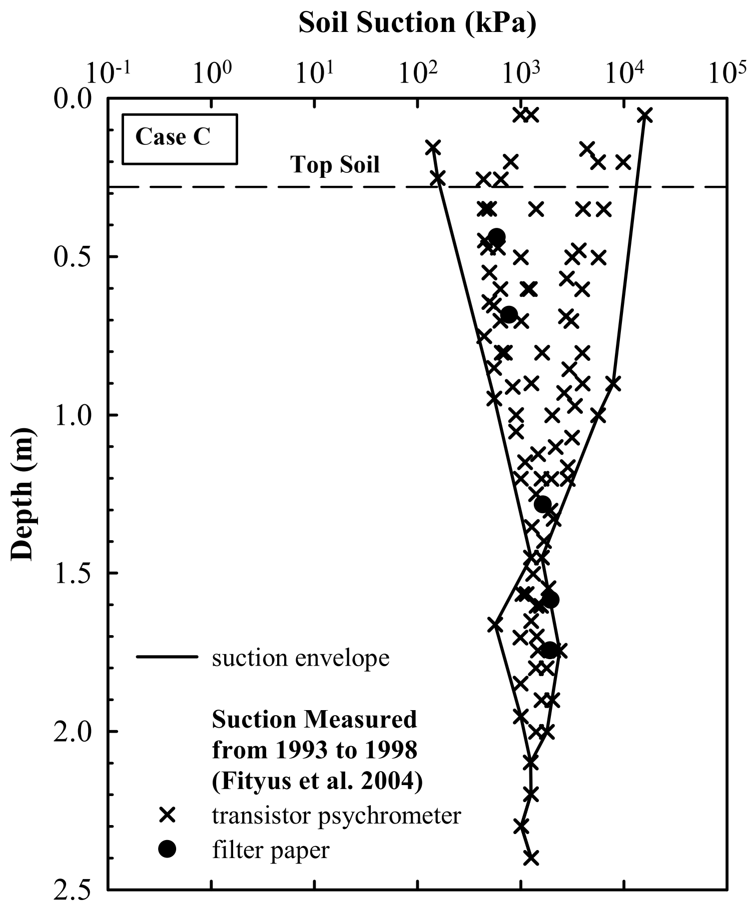

Figure 5.10 Suction profile of Maryland clay site

It has been recorded in the literature that the topsoil consists of 0.3 m thickness silty clay, which is believed to be non-expansive in nature. For this reason, the ground heave was re-evaluated within the depth of active zone (2 m) neglecting the top soil layer (0 to 0.3 m). From the suction envelope, respective values can be determined for both initial dry condition and subsequent wet condition; hence, the swelling pressures of the soil at both conditions can be predicted using the proposed model (Equation 3.6). Based on the soil property (i.e., I_p, C_s, and ρ_{dn}) profiles (Figure 5.11), the model parameter, β_n can be determined for each layer using Equation (3.7). The swelling index, C_s values are estimated from plasticity index, I_p using Equation (3.9). The void ratio of the soil at dry condition, e_i is back-calculated from relevant soil parameters. Using the information above, the heave associated with gradual wetting can be estimated using Equation (4.4) and Equation (4.5). The detailed calculation procedures are presented in the Appendix Table A.3.

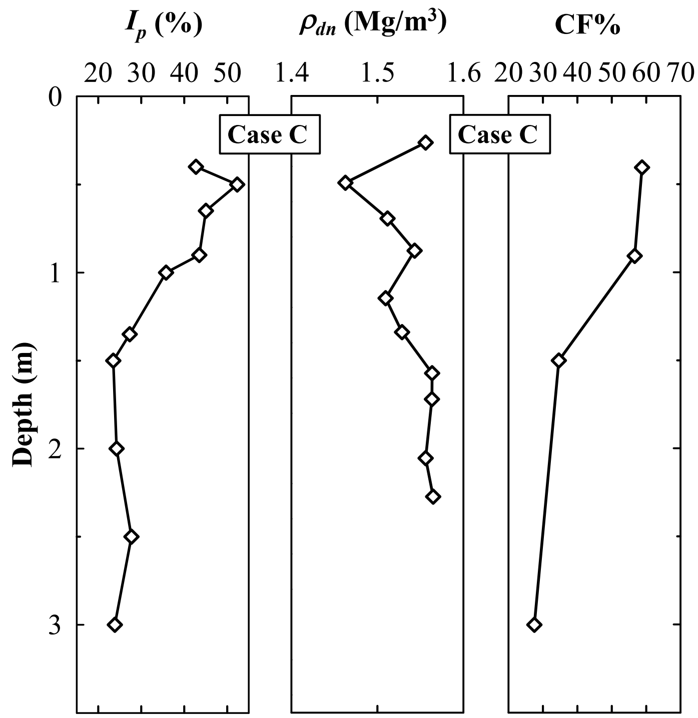

Figure 5.11 Profiles of soil properties of Maryland clay (after Fityus et al. 2004)

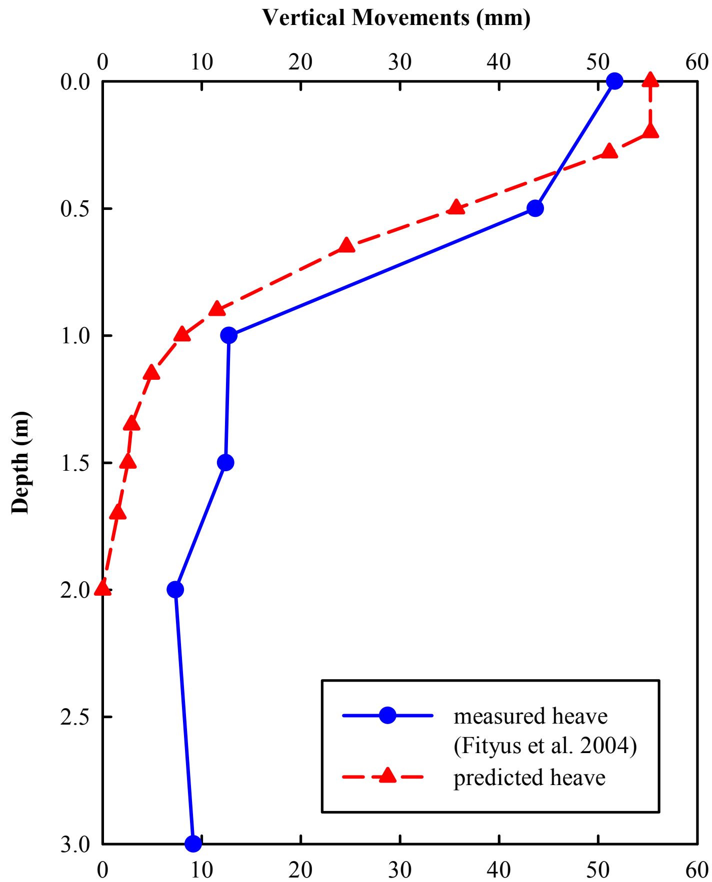

Figure 5.12 Measured and predicted heave of Maryland clay site

It is believed that the ground heave below the depth of active zone should not be expected. However, 10 mm heave was measured within the 2 m to 3 m depth zone (see Figure 5.12). The heave below 2 m depth may be attributed to either the errors during the long-term monitoring or the change of water content (or suction) in deeper stratum layers associated with the ground water flow. Figure 5.12 summarizes the measured and predicted ground movements. Good agreement can be observed between the measured and predicted ground heave with respect to depth if the previously discussed 10 mm measured heave below 2 m depth is ignored. Nevertheless, the proposed techniques provide a conservative estimation (55 mm) of total ground heave when compared to the measured value (51 mm).

5.3.4 Case Study D: Keswick clay site, Australia

Richards et al. (1983) investigated differential deformation of the National Art Gallery of South Australia at Adelaide. The problem was induced by the heave of a local expansive soil which is known as Keswick clay (or Pleistocene clay). The average plasticity index, I_p and natural dry density, ρ_{dn} of this clay are 47.6 (ranges from 25 to 63) and 1.46 Mg/m3 (ranges from 1.27 to 1.92 Mg/m3) respectively (Sheard and Bowman 1994; Jaksa 1995). The clay fraction (c %) at the investigated area was found to be about 40% within the depth from 0.30 to 2.0 m (SoilInfo 2014). The SWCCs of Keswick clay from Jones et al. (2009) is shown in Figure 5.13.

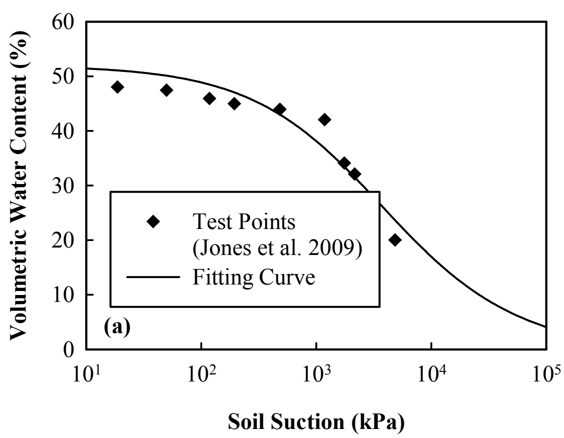

Figure 5.13 The SWCC of Keswick clay (a) in terms of volumetric water content, and (b) in terms of degree of saturation

The information of soil suction and ground heave are reported in Richards et al. (1983). The variations of soil suction with respect to the depth were measured at two different spots; namely, Spot A (which is inside the building) and Spot B (which is outside the building and adjacent to nearby trees) (see Figure 5.14). Differential deformation around the building was triggered by the removal of trees. The heave inside the building was minor; however, the heave on the roadway from where trees were removed was measured to be around 50 mm.

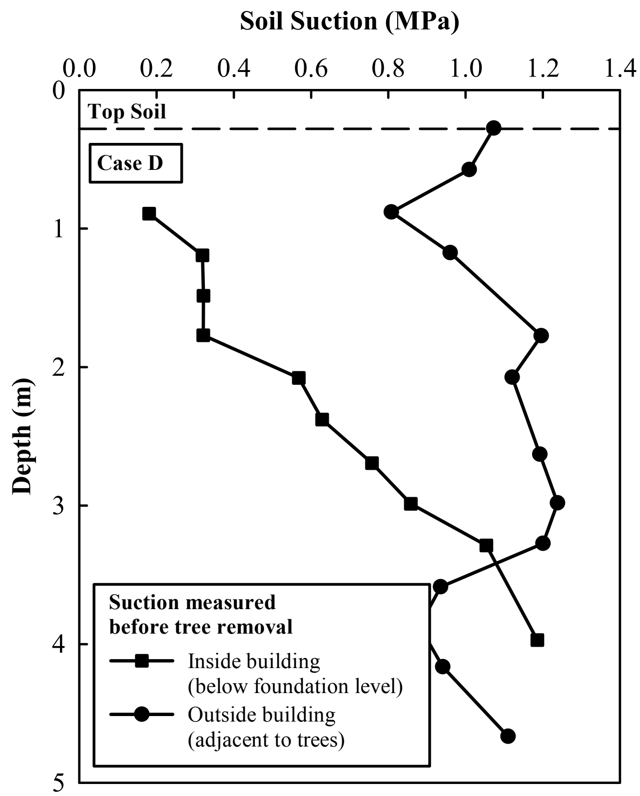

Figure 5.14 Suction profiles of Keswick clay site (after Richards et al. 1983)

The heave of roadway is estimated using the proposed techniques in the present paper. Detailed calculation procedures are presented in the Appendix Table A.4. The estimated heave is found to be 56 mm, which is slightly higher in comparison to the measured value of 50 mm at natural ground surface (see Figure 5.15). The assumptions made at arriving the estimated heave in this case study include: (i) the top 0.2 m soils, without suction data (see Figure 5.14) is considered to be non-expansive; (ii) the suction profile of the soil beneath the building is assumed to be the subsequent wet condition of the soil outside the building, since after the removal of the trees, the water content of the soil outside the building gradually reaches an equilibrium condition with that of the soil beneath the building, and (iii) the heave was calculated up to a depth of 3.4 m where the initial and subsequent suction profiles intersect (see Figure 5.14).

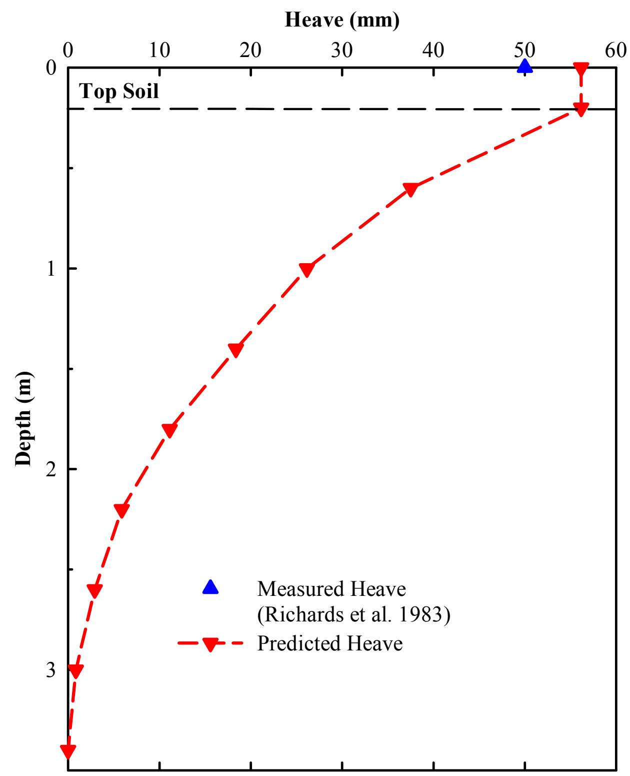

Figure 5.15 Measured and predicted heave of Keswick clay site

5.3.5 Case Study E: Arlington clay site, US

Expansive soils are widely distributed in state of Texas of the U.S. The present case study focuses on evaluating the ground heave of a field site in Arlington, Texas over the period from July 1999 to March 2001. The Arlington site is underlain with two types of soils, which are the dark gray silty clay within the top layers and the brown silty clay within the bottom layers (see Figure 5.16a). The SWCC in terms of gravimetric water content and the water content-void ratio-soil suction relationship have been measured by Zhang and Briaud (2010). The data points of the SWCC (Figure 5.17) are fitted with the Fredlund and Xing (1994) equation.

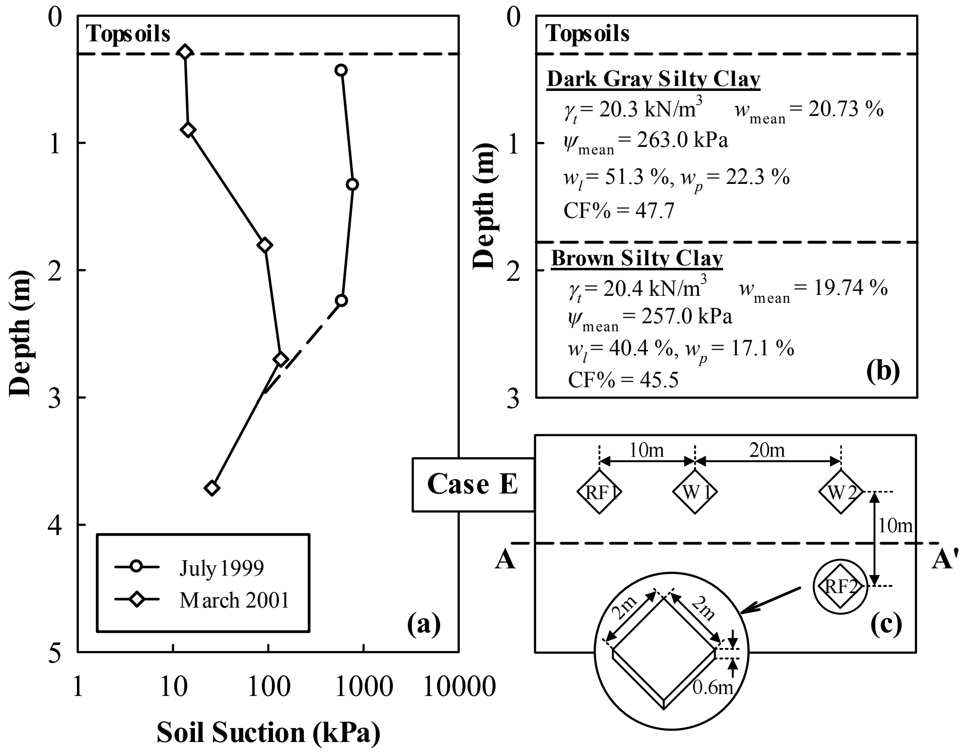

Figure 5.16 Information on Arlington clay site: (a) soil suction, (b) soil profile and properties, (c) location of the footings (modified from Briaud et al. 2003)

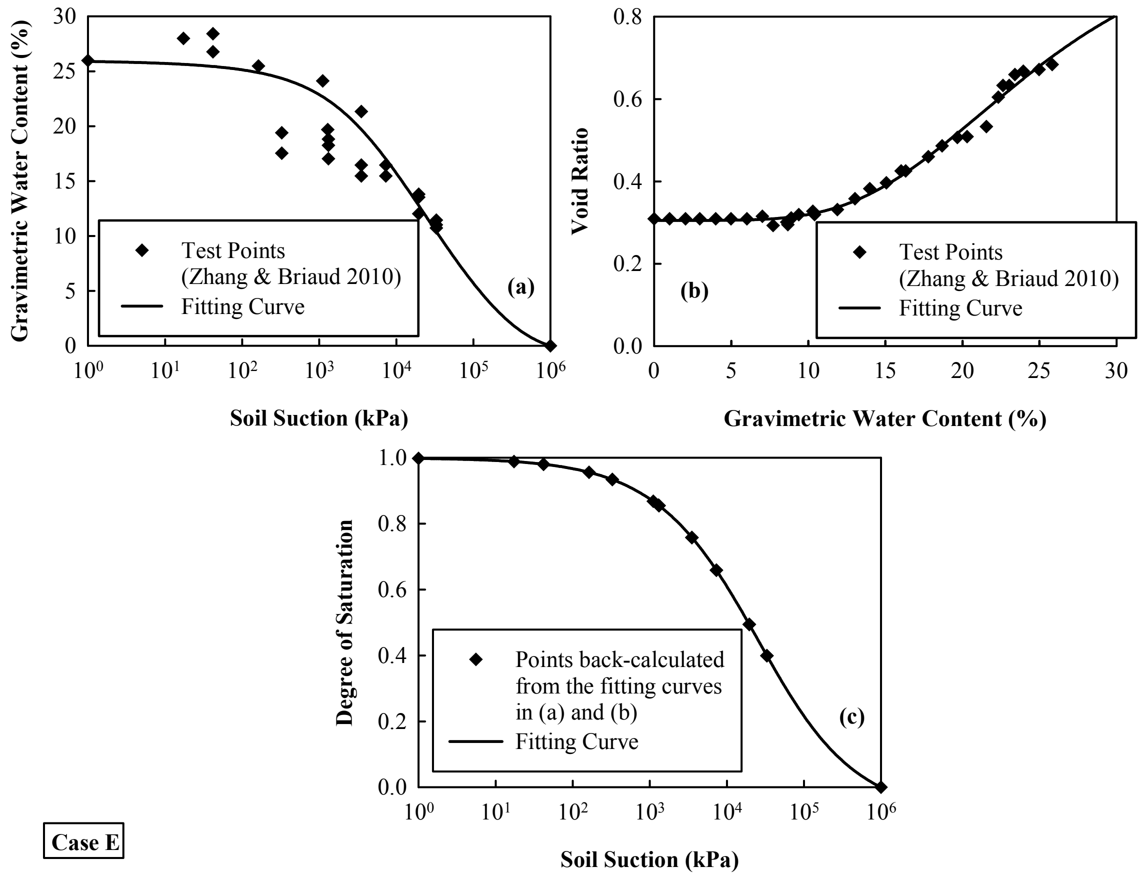

Figure 5.17 (a) SWCC in terms of gravimetric water content, (b) void ratio versus gravimetric water content relationship, (c) SWCC in terms of degree of saturation

Briaud et al. (2003) performed a series of field tests at the site over a period of two years. The site arrangement is shown in Figure 5.16c, where it can be observed that four areas were outlined; namely, RF1, RF2, W1 and W2. A 0.6 m thickness 2 m × 2 m square footing was constructed at each of these areas. The present case study focuses only on the field performance of the RF1 area from July 1999 to March 2001. The information on the soil stratigraphy and the average soil index properties are shown in Figure 5.16b. In addition, long-term performance of the RF1 area regarding the variation of soil suction with respect to depth (see Figure 5.16a) and the heave at the ground surface (see Figure 5.18) were recorded. The suction data indicates that the depth of active zone is approximately 3 m. This value also agrees with the result estimated by Briaud et al. (2003), using the empirical approaches developed by Mitchell (1979) and McKeen and Johnson (1990).

The estimated heave at the ground surface was 37 mm in comparison to the measured value of 30 mm using the proposed method (see Figure 5.18). The detailed calculation procedures are presented in the Appendix Table A.5.

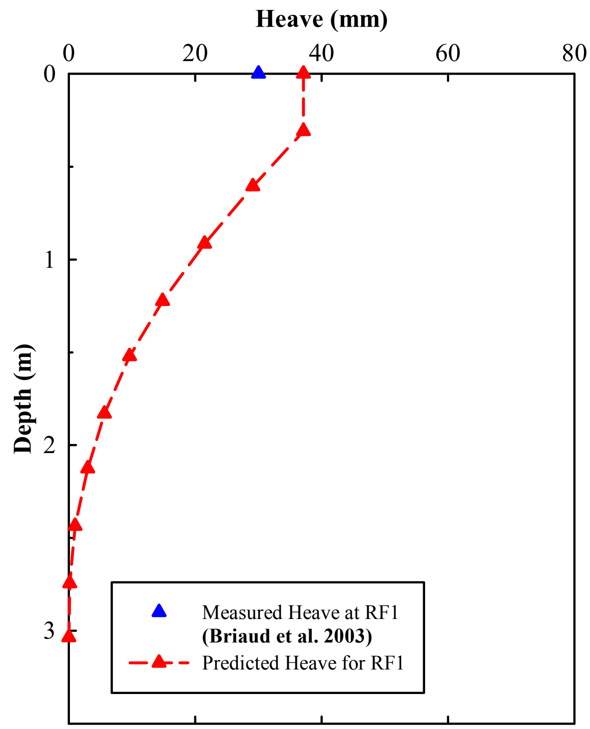

Figure 5.18 Measured and predicted heave at Arlington site

5.3.6 Case Study F: Al-Ghat shale site, Saudi Arabia

The present case is based on the field records of a site at the town of Al-Ghat, 270 km northwest of Riyadh, capital of Saudi Arabia (Al-Shamrani and Dhowian 2003). The Al-Ghat shale has an average specific gravity, plasticity index, clay fraction and dry density of 2.78, 35%, 72% and 1.89 Mg/m3, respectively (Al-Shamrani and Dhowian 2003). The SWCC in terms of degree of saturation (Figure 5.19) were obtained from the data of gravimetric water content-void ratio-soil suction relationship measured by Elkady (2014).

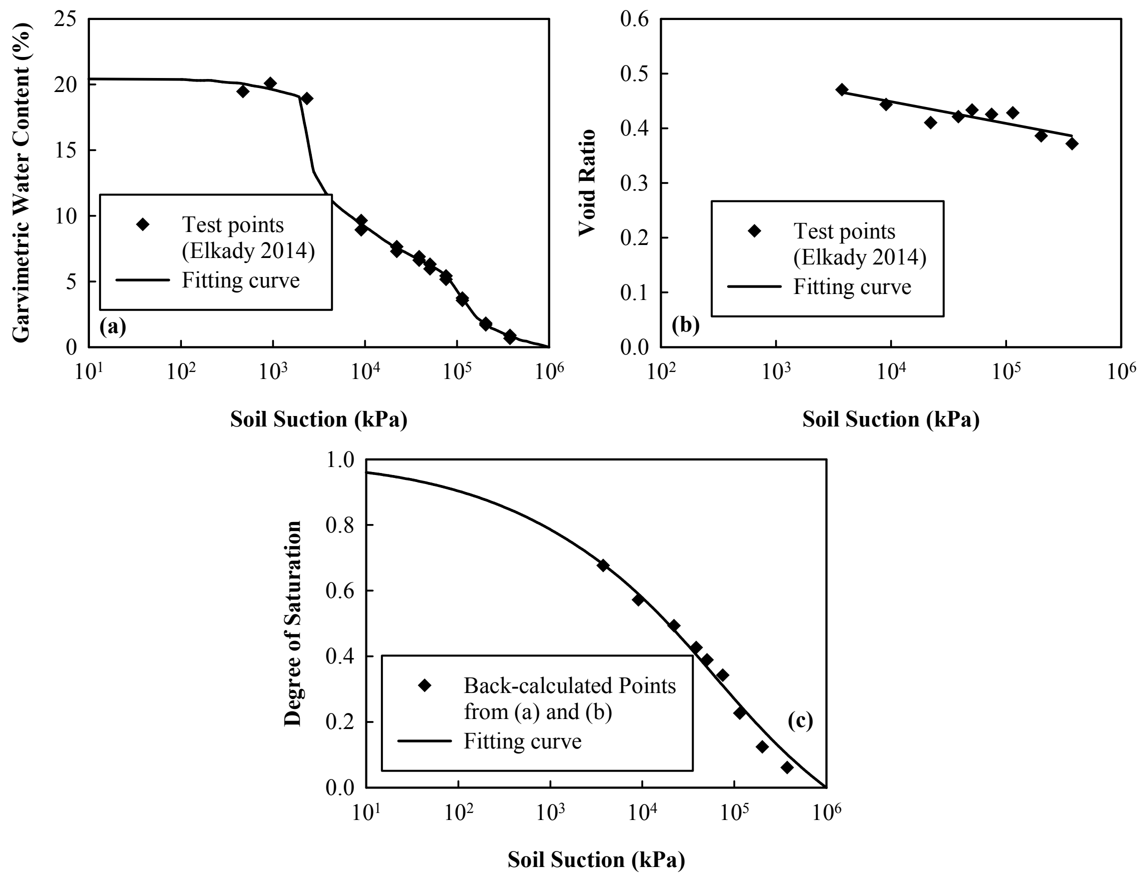

Figure 5.19 (a) SWCC in terms of gravimetric water content, (b) void ratio versus gravimetric water content relationship, (c) SWCC in terms of degree of saturation

Al-Shamrani and Dhowian (2003) performed a series of field tests on an experimental station covering an area of 20 m×20 m, the top soils were removed to expose the expansive materials before conducting these tests. An infiltration system was established to artificially saturate the investigated site. Artificial infiltration contributed to swelling of the expansive soil at the site. The information of water content, soil suction profiles from the published literature is summarized (see Figure 5.20).

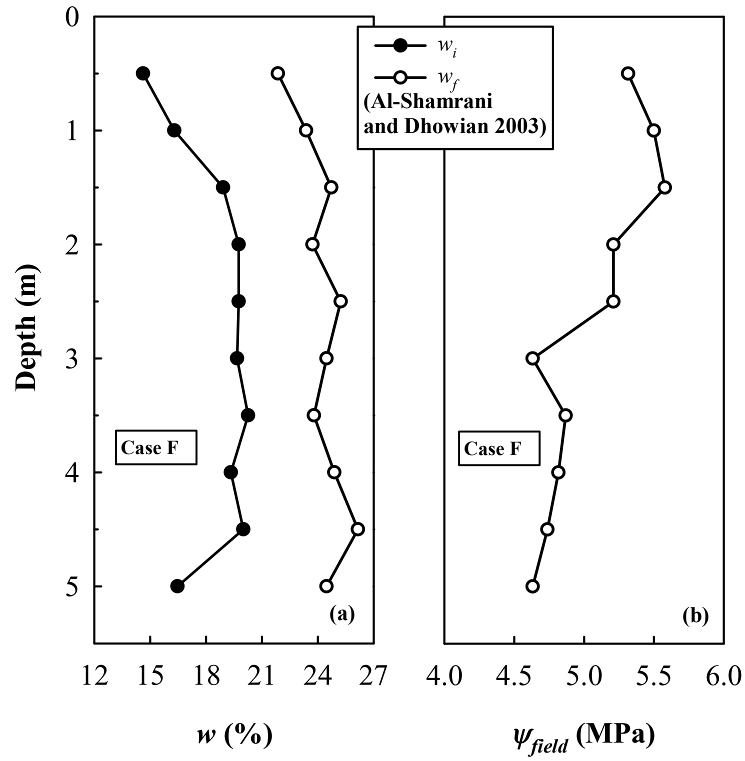

Figure 5.20 Water content and suction profiles of Al-Ghat site: (a) water content profiles, (b) initial soil suction profile

In addition to these field tests, Al-Shamrani and Dhowian (2003) conducted many laboratory tests including oedometer tests, triaxial tests, and suction tests on undisturbed and compacted soil specimens to predict the heave using different prediction methods proposed by different researchers (Johnson and Snethen 1978; Lytton 1977b; Mitchell 1980; McKeen 1992; Hamberg and Nelson 1984; Richards 1967; Dhowian 1990a; Fityus and Smith 1998). For comparison purposes, the proposed techniques are also used to predict the heave. The predicted variations of swelling pressure and heave with respect to depth are shown in Figure 5.21. The detailed calculation procedures are presented in the Appendix Table A.6.

It can be observed from Figure 5.21b, that the heave predicted using the proposed method is close to the measured values. Some of the prediction methods based on comprehensive testing results are also reasonable (Al-Shamrani and Dhowian 2003). However, the laboratory procedures for obtaining relevant parameters (e.g., swelling pressure, P_s;

swelling index, C_s; lateral restraint factor, f_p; suction compression index, C_ψ; moisture index, C_w) are rather cumbersome and time-consuming.

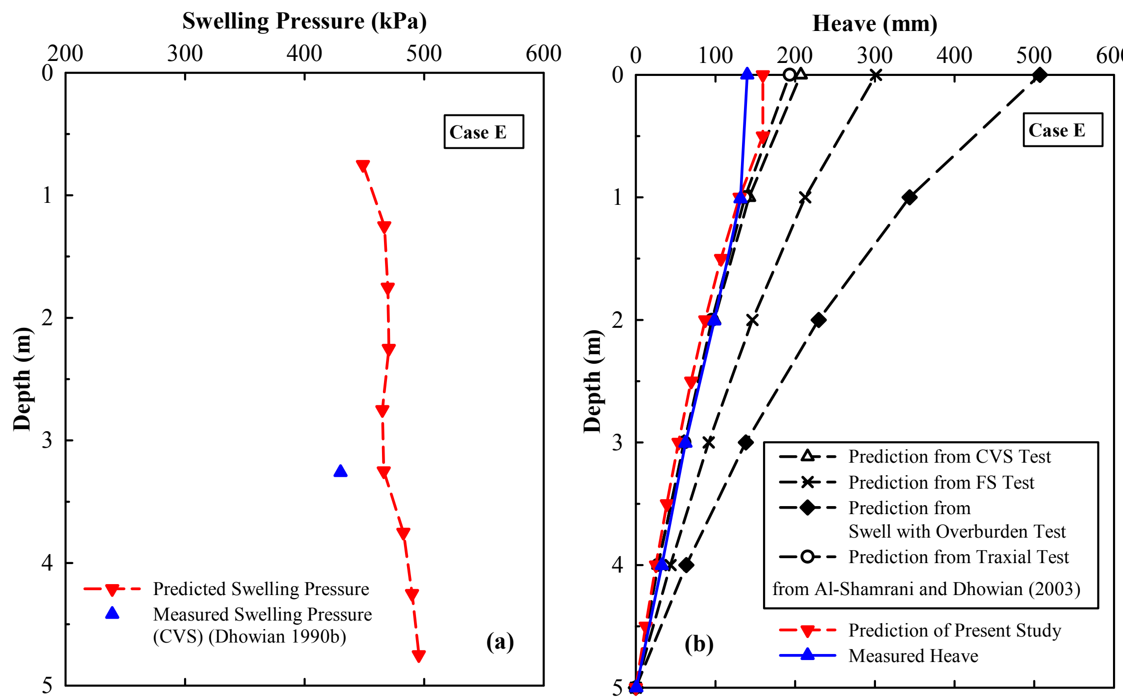

Figure 5.21 (a) Predicted and measured swelling pressure, (b) predicted and measured ground heave

5.3.7 Case Study G: Zaoyang expansive soil site, China

The focus of present case study is directed towards estimating the ground heave of a natural expansive soil slope at Zaoyang, China. This site was located about 230 km north-west of Wuhan and about 100 km south of the intake canal for the South-to-North Water Transfer Project (SNWTP) in Nanyang, Henan, China (Ng et al. 2003). The predominant stratum was a yellow-brown clay, of which the predominant clay minerals are illite and montmorillonite. It was reported that abundance cracks were developed in the surface. The SWCCs (Figure 5.22) of Zaoyang expansive soil, soil profiles and soil properties of the investigated site has been recorded in the literature (Ng et al. 2003 and Zhan et al. 2007).

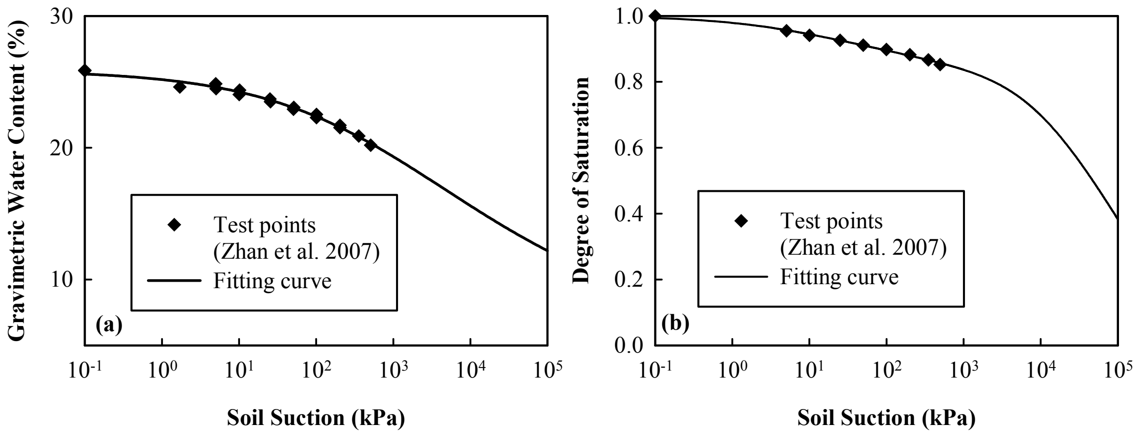

Figure 5.22 SWCCs of Zaoyang expansive soil: (a) in terms of gravimetric water content, (b) in terms of degree of saturation

Ng et al. (2003) performed two artificial rainfall infiltration tests during the one-month monitoring period (from August 13 to September 12, 2001). Prior to the rainfall stimulation, three rows of instrumentation (R1, R2, R3) were implemented at different positions of the slope to measure the water content, soil suction, and ground movements (horizontal and vertical). Figure 5.23 presents the suction variations with respect to depth at different stages during the infiltration test. The suction profile indicates that the depth of active zone was about 3.5 m, since that the changes on suction value below this depth is negligible. Furthermore, the soil suction varies around zero by the end of the monitoring period; namely, a fully saturated condition was achieved within active zone depth.

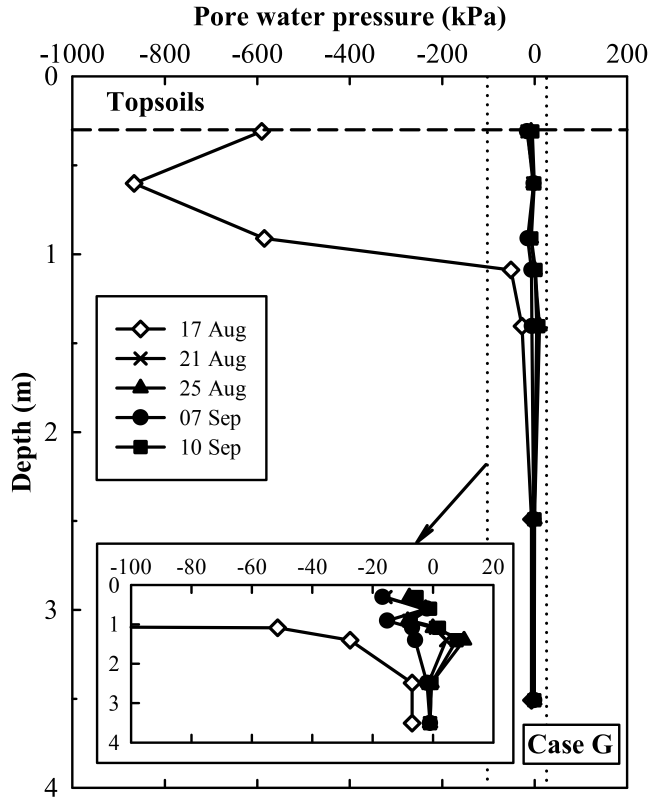

Figure 5.23 Soil suction measurement of the Zaoyang expansive soil site (after Ng et al. 2003)

Since that there was limited investigation on the water content and soil suction within the top 0.3 m depth, these soils are assumed to be non-expansive in the present case study. Such an assumption is reasonable, considering the highly developed fissures on the surface layers. The variation of swelling pressure and heave with respect to depth was estimated using the proposed techniques following similar procedures discussed for previous case studies. The ground heave is estimated to be 33 mm, which demonstrates a reasonable comparison with the measured value of 31 mm (see Figure 5.24). The detailed calculation procedures are presented in the Appendix Table A.7.

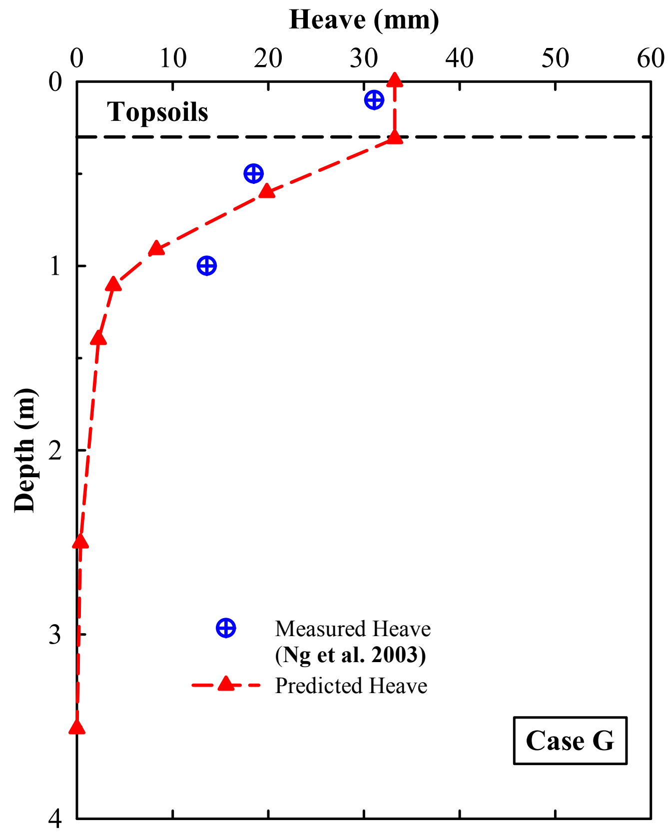

Figure 5.24 The predicted and measured ground heave at Zaoyang site

5.3.8 Case Study H: Regina clay site, Canada

A glacio-lacustrine geology and a semi-arid climate resulted in the development of an expansive soil deposit in Regina, Saskatchewan (Ito and Azam 2009). Information on the geotechnical properties of Regina clay as well as ground deformation monitoring of field sites has been established by Yoshida et al. (1983), Azam et al. (2013) and Shah (2011).

The present case study is established based on the information available in the literature, including the SWCCs of typical Regina clay (Figure 5.25) and the variation of soil index properties with respect depth (Figure 5.26). The initial soil suctions were back-calculated from the SWCC with known volumetric water content values of the soil at various depths (see Figure 27a,b). The variation of swelling pressure with respect to depth (Figure 27c) was estimated following similar procedures as discussed in earlier sections. The predicted swelling pressure at the depth of 1.2 m shows a good agreement with the value of an undisturbed sample measured from the CVS test.

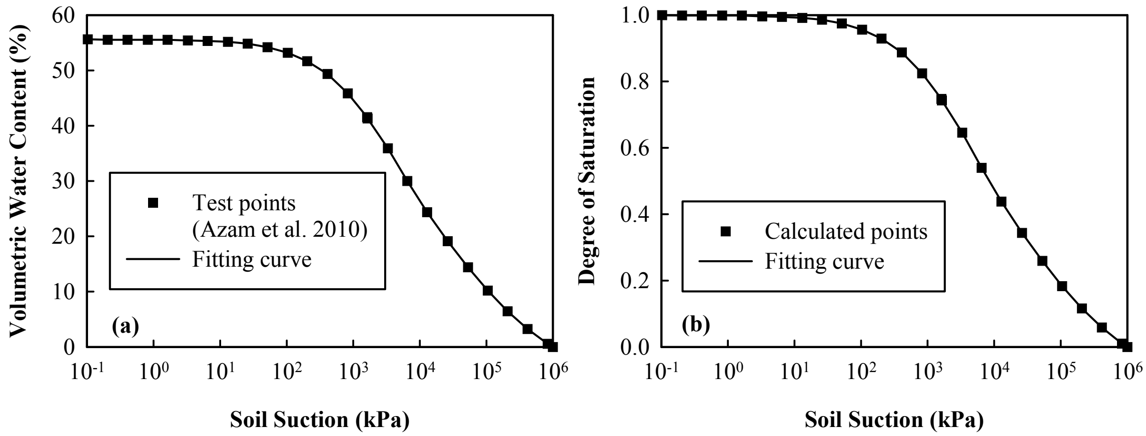

Figure 5.25 SWCCs of Regina clay: (a) in terms of gravimetric water content, (b) in terms of degree of saturation

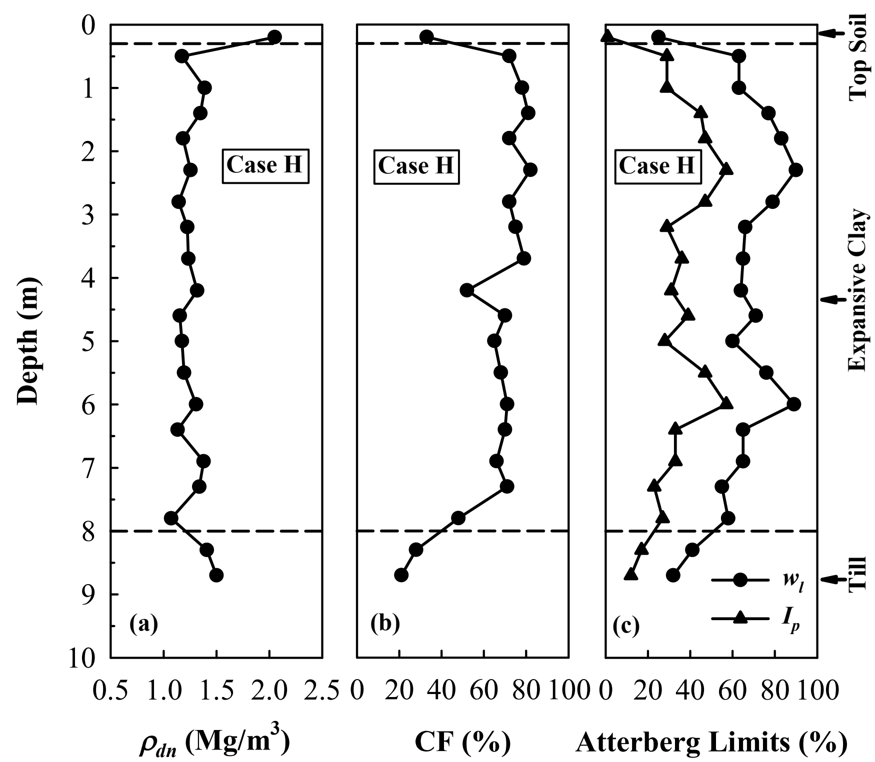

Figure 5.26 Soil profile of Regina site: (a) natural dry density, (b) clay fraction (c) Atterberg limits (modified from Azam et al. 2013)

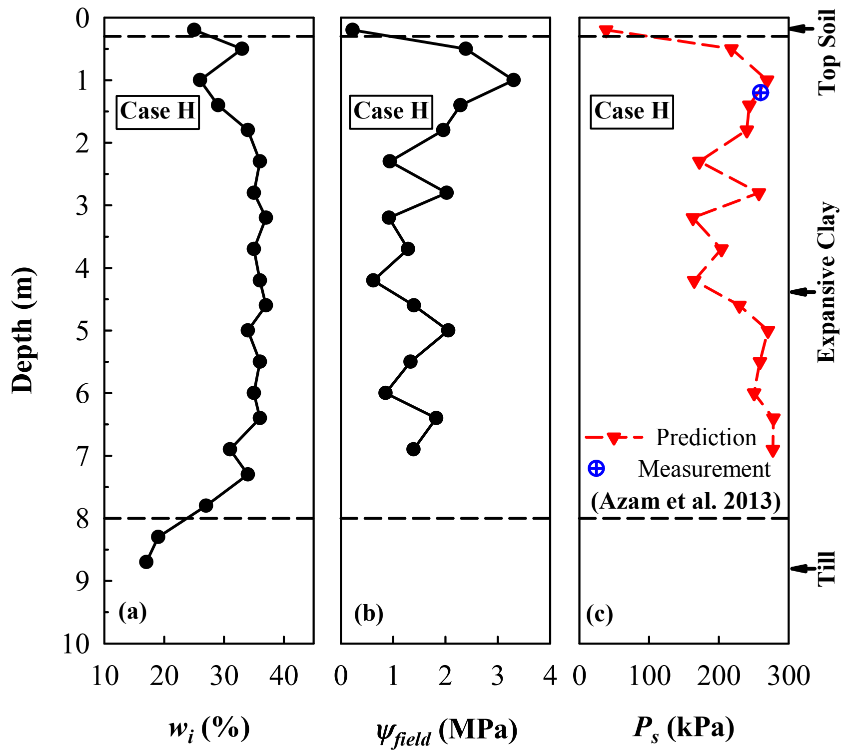

Figure 5.27 Initial condition and swelling pressure variation of Regina site: (a) initial water content, (b) *in-situ* soil suction, (c) measured and predicted swelling pressure

Unlike most of the previous case studies, there was no record on the final state of the soil suction (or water content) for the Regina clay site. In this case, heave value is estimated extending conservative assumption that the final state of the soil within active zone reaches a fully saturated condition. In this case, Equation (4.2) should be applied for the estimation of the ground heave. Since the depth of active zone cannot be determined from the information provided in present case, it is estimated using an empirical equation (Equation 2.24) proposed by McKeen and Johnson (1990), to be 3.26 meters. The value of $2U_0$ was determined from a suction profile recorded by Vu et al. (2007); the value of ΔU_{max} was suggested to be $0.1 \times 2U_0$ by Briaud et al. (2003); the value of n was conservatively chosen as 0.5 cycle/year; the moisture diffusion coefficient α is normally in the range of 10^{-7} to 10^{-9} m^2/s, hence the upper range value of 10^{-7} m^2/s was used to be conservative. Detailed procedure of the estimation on the depth of active zone is presented in Table 5.5.

Table 5.5 Estimation of active zone depth for Regina site

Parameter	U_0 (pF)	$2U_0$ (pF)	ΔU_{max} (pF)	n (cycle/year)	A (cm^2/sec)	z (m)
Values	2.33	4.67	0.47	0.5	10^{-3}	3.3

Using the above information, the ground heave variation can be estimated (see Figure 5.28). Detailed calculation procedures are presented in the Appendix Table A.8. The estimated ground heave is compared with the measured data (Shah 2011), which indicates that a considerable amount of heave was contributed by the soil below 2 m depth. Figure 5.28 demonstrates that the heave is overestimated above 1.5 m depth, while is underestimated below 1.5 m depth. The differences between the estimated and measured heave can be attributed to four sources; (i) the assumption used for the SWCC, (ii) the assumption used for active zone depth; (iii) the assumption on final state of the soil; and (iv) the differences between the site where the data of the soil was measured and the sites where the ground heaves were monitored. In spite of some errors that may have contributed in the prediction of ground heave based on the limited information, the proposed technique is reasonable for estimating the swell characteristics (swelling pressure and ground heave) of the Regina clay.

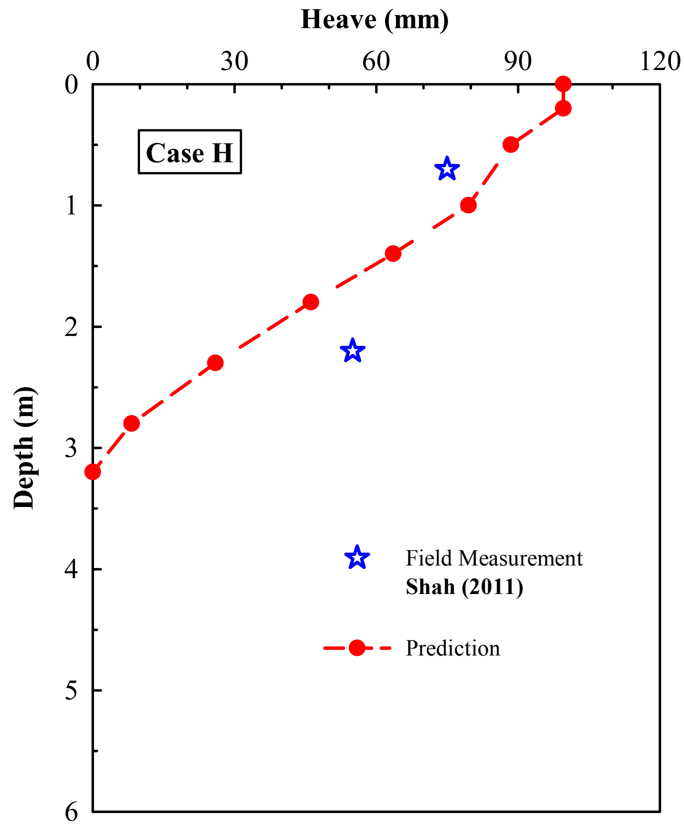

Figure 5.28 The predicted and measured ground heave for Regina site

5.3.9 Discussion on the predictions of the heave at natural expansive soil sites

The proposed ground heave prediction techniques for natural expansive soils are validated with the information of eight field case studies. For each of these case studies, the proposed techniques have been utilized to estimate the variations of heave and/or swelling pressure with respect to depth. The predicted and measured ground heaves for all the case studies are summarized in Figure 5.29. The estimation at all depths shows a good comparison with the corresponding measurements (R^2 = 0.96). Also, conservative values have been estimated for ground surface in most of the cases. Some minor deviations between the measured and estimated heave at the ground surface may be attributed to the using of the drying SWCCs.

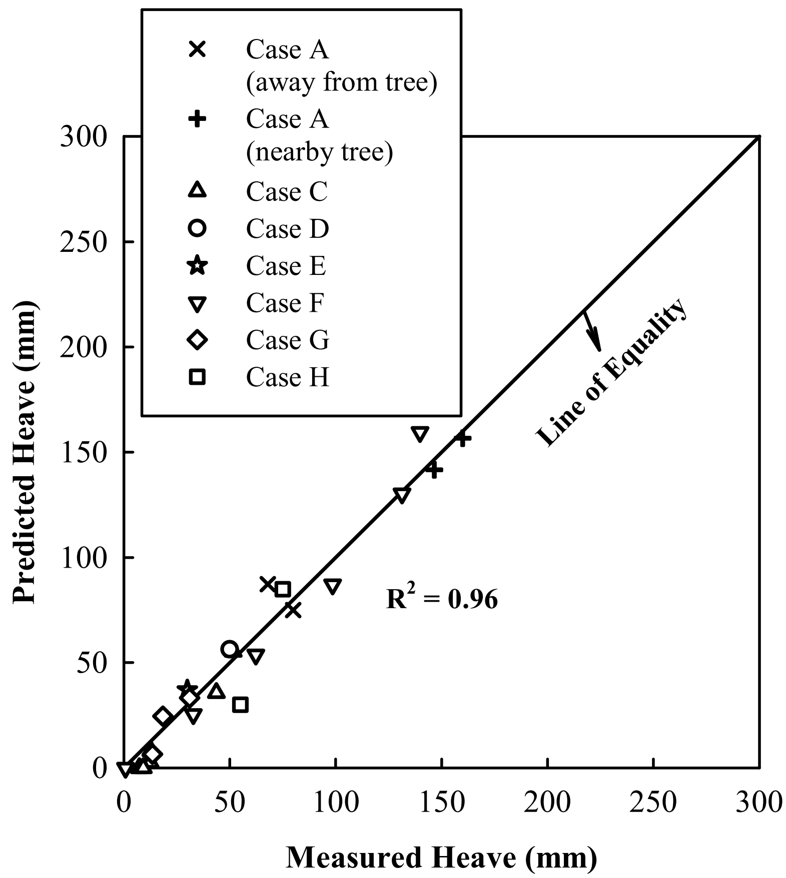

Figure 5.29 Comparison between measured and predicted heaves

CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

The 1-D heave of expansive soils is typically estimated based on the results of either oedometer swelling tests or suction tests. However, the procedures associated with these tests are tedious, time-consuming and expensive. Besides, the swelling pressure measured from oedometer swelling tests may result in over estimation of ground heave, since the soil specimen is submerged until the specimen attains a fully saturated condition. The methods based on suction tests neglects the influence of overburden pressure which leads to erroneous estimates of in-situ ground heave.

In this thesis, SWCC based swelling pressure predicting models (Equation 3.2 and 3.6) for both the compacted and natural expansive soils are proposed. The model parameters required for using these models can be estimated from basic soil properties using proposed equations (Equation 3.3, 3.4 and 3.7). In addition, the swelling index can be estimated form plasticity index, I_p, based on an empirical relationship (Equation 3.9). The predicted/estimated volume change indices (P_s and C_s) can be employed to estimate the ground heave of unsaturated expansive soils using a modified heave calculation equation (Equation 4.4).

A database from the literature that includes the information of six compacted expansive soil samples and eight natural expansive soil sites from different regions of the world have been used to validate the proposed heave estimating techniques.

6.2 Conclusions

- The semi-empirical models (Equation 3.2 and 3.6) for predicting swelling pressure are proposed based on the S – shape variation of swelling pressure with respect to the change of initial soil suction.
- The model parameters in the swelling prediction models are correlated to the soil index properties; based on the analyses of relevant data, Equation (3.3) and Equation (3.4) have been suggested to estimate the model parameters for Equation (3.2) (for compacted soils); Equation (3.7) has been suggested to estimate the model parameters for Equation (3.6) (for natural soils).
- Analyses of compacted soil samples suggests that, the Equation (3.3) provides more reasonable estimates when compared to Equation (3.4); this is because that Equation (3.3) is proposed based on the results of constant volume swelling (CVS) tests while Equation (3.4) is proposed based on the results of free swell (FS) tests. These results are consistent with the observations of other investigators who suggested that CVS tests provide reasonable swell pressure values in comparison to FS tests.
- There is an S – shape relationship between the C_s value and I_p value. The original contribution of Vanapalli and Lu (2012) was modified to propose Equation (3.9). This equation facilitates the estimation of C_s values for expansive soils with different I_p values.
- Conventional oedometer-test based ground heave prediction methods are only capable for estimating the potential heave of expansive soils, since they are based on an assumption that the soil reaches fully saturated condition. However, natural expansive soils rarely reach fully saturated conditions. For this reason, Equation (4.4) is proposed to estimate the ground heave of expansive soils as it gradually gets wetter and attains different degrees of saturation (i.e. different suction values).
- The drying curve SWCC, which is less representative for estimating heave in comparison to the wetting curve of SWCC, however, has been used as a tool to estimate the heave. The drying SWCC is used for two reasons; (i) it can be easily measured in

comparison to the wetting SWCC; (ii) drying SWCC conservatively estimates the ground heave in comparison to the wetting SWCC.

- For case study analyses on field sites, the prediction on both swelling pressure and heave (or swell strain) shows a reasonably good agreement with measured values.

6.3 Strengths and Limitations of the Proposed Research

Common to all prediction methods, the techniques proposed in the present research study are based on certain assumptions and simplifications. The strengths and limitations are summarized below:

- The proposed techniques are simple for use in practice for estimating the swelling pressure and ground heave; only basic soil properties are needed to determine the relevant parameters required in the proposed prediction models (or equations);
- The model parameter P_{s0} in Equation (3.2) is assumed to be a constant value (55 kPa) in the present study. More investigations are necessary to check the validity of this value or another method has to be proposed for estimating P_{s0};
- Both the Fredlund (1983) equation (Equation 4.1) and modified Fredlund (1983) equation (Equation 4.4) proposed in this thesis are based on an assumption that the swelling index C_s is equivalent to a constant value (the slope of the rebound curve); while actually the magnitude of the C_s value varies with respect to different initial conditions (e.g., e_i and e_w in Equation 4.2 and 4.3).

6.4 Future work

- Further investigations are required to obtain a better understanding on the relation between P_{s0} value and influencing factors such as the method of compaction, degree of compaction, dry density and mineralogy of the expansive soils.
- Uncertainty analysis for the proposed equations should be undertaken such that the practitioners can better understand the reliability of the proposed approaches presented in the thesis.

- The drying curve of SWCCs are mostly used in the present study, while the swelling of expansive soil essentially occurs along the wetting path. As measuring the wetting SWCC is cumbersome, there is a need of an approach for predicting the wetting curve of the SWCC from the drying curve taking relevant soil index properties as indicators;
- In the present research study, limited discussion is provided with respect to the influence of soil cracks on the swelling behavior of expansive soils. A more rigorous approach of using the bimodal SWCC-based prediction model is required for more reasonable estimation of the swelling pressure and the heave.
- The proposed techniques can be used to predict the variation of swelling pressure and potential heave with respect to different initial soil suction values. Incorporating with the commercial software (e.g., VADOSE/W) that simulates the variation of suction (or water content) with respect to environmental changes, the proposed techniques can be used to predict the evolution process of ground heave taking account of the influence of the environmental factors.
- The proposed swelling pressure and ground heave prediction technologies can be used to develop the design methodologies for the infrastructure such as the shallow foundations, pile foundations, and retaining walls that are constructed in expansive soil problems, by integrating the concepts of unsaturated soil mechanics.

REFERENCES

Abbaszadeh, M. M., Houston, S., Zapata, C., Houston, W., Welfert, B., and Walsh, K. (2010). “Laboratory determination of soil-water characteristic curves for cracked soil.” In: Alonso E, Gens A (eds.) *Proceedings of the 5th International Conference on Unsaturated Soils*, Barcelona, Spain, 6–9 September 2010. Taylor & Francis Group, London, 409-415.

Abbaszadeh, M. (2011). *The effect of cracks on unsaturated flow and volume change properties of expansive clays and impacts on foundation performance*, Doctoral dissertation, Arizona State University.

Abduljauwad, S. N. (1993). “Study on the performance of calcareous expansive clays.” *Bulletin of the Association of Engineering Geologists*, XXX(4): 481-498.

Abduljauwad, S. N., Al-Sulaimani, G. J., Basunbul, I. A., and Al-Buraim, I. (1998). “Laboratory and field studies of response of structures to heave of expansive clay.” *Géotechnique*, 48(1): 103-121.

Abu-Hejleh, A. N., and Znidarcic, D. (1995). “Desiccation theory for soft cohesive soils.” *Journal of Geotechnical Engineering*, ASCE, 121(6): 493–502.

Adem, H. H. (2015). *Modulus of elasticity based method for estimating the vertical movement of natural unsaturated expansive soils*. Doctoral dissertation, University of Ottawa.

Adem, H. H., and Vanapalli, S. K. (2014) “Elasticity moduli of expansive soils from dimensional analysis.” *Geotechnical Research*, 1(2): 60-72.

Aitchison, G. D. (1965). *Discussion in Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering*, Montreal, QC, 318–321.

Aitchison, G. D. (1970). “A statement of the problems the engineer faces with expansive soils.” *Proceedings of the 2nd International Research and Engineering Conference on*

Expansive Clay Soils, Texas A & M University, College Station, 33-51.

Aitchison, G. D. (1973). "The quantitative description of the stress-deformation behavior of expansive soils." *Proceedings of the 3rd International Conference on Expansive Soils*, Haifa, Israel, 79–82.

Aitchison, G. D., and Martin, R. (1973). "A membrane oedometer for complex stress-path studies in expansive clays." *Proceedings of 3rd International Conference on Expansive Soils*, Haifa, Israel, Vol 2, 83-88.

Aitchison, G. D., and Richards, B. G. (1965). "A broad-scale study of moisture conditions in pavement subgrades throughout Australia." In *Moisture equilibria and moisture changes in soil beneath covered areas*, 184-232, Sydney, Butterworths.

Aitchison, G. D., and Woodburn, J. A. (1969). "Soil suction in foundation design." *Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 2, 1-9.

Agus, S. S. (2005). *An experimental study on hydro-mechanical characteristics of a polymer-enhanced bentonite sand mixtures*. Ph.D. Thesis, University of Weimar, Germany.

Al-Homoud, A. S., Basma, A. A., Husein Malkawi, A. I., and Al Bashabsheh, M. A. (1995). "Cyclic swelling behavior of clays." *Journal of geotechnical engineering*, 121(7): 562-565.

Alonso, E. E., Gens, A., and Josa, A. (1990). "A constitutive model for partially saturated soils." *Géotechnique*, 40(3): 405-430.

Alonso, E. E., Pinyol, N. M., and Gens, A. (2013). "Compacted soil behavior: initial state, structure and constitutive modelling." *Géotechnique*, 63(6): 463-478.

Al-Shamrani, M. A., and Dhowian, A. W. (2003). "Experimental study of lateral restraint effects on the potential heave of expansive soils." *Engineering Geology*, 69(1): 63-81.

AS 2870. (1996). *The Australian standard for the design and construction of residential slabs and footings*. Standard Australia.

ASTM (1996). "Standard test methods for one-dimensional swell or settlement potential of cohesive soils." *Annual Book of ASTM Standards, D4546-96*, American Society of Testing Material.

ASTM (2003). "Standard test methods for one-dimensional swell or settlement potential of cohesive soils." *Annual Book of ASTM Standards, D4546-96*, American Society of Testing Material.

ASTM (2008). "Standard test methods for one-dimensional swell or settlement potential of cohesive soils." *Annual Book of ASTM Standards, D4546-96*, American Society of Testing Material.

Attom, M. F., Abu-Zreig, M. M., and Obaidat, M. T. (2006). "Effect of remolding techniques on soil swelling and shear strength properties." In *Expansive Soils: Recent Advances in Characterization and Treatment*, (Al-Rawas, A. A., and Goosen, M F. A. Ed.), Taylor & Francis Group, London, UK.

Azam, S. (2006). "Large-scale odometer for assessing swelling and consolidation behaviour of Al-Qatif clay." *Expansive soils: Recent advances in characterization and treatment*, Balkema Publishers-Taylor and Francis, The Netherlands, 85-99.

Azam, S. (2007). "Study on the swelling behaviour of blended clay-sand soils." *Geotechnical and Geological Engineering*, 25(3): 369–381.

Azam, S., Shah, I., Raghunandan M. E., and Ito M. (2013). "Study on swelling properties of an expansive soil deposit in Saskatchewan, Canada." *Bulletin of Engineering Geology*, 72(1): 25-35.

Azam, S., and Wilson, G. W. (2006). "Volume change behavior of a fissured expansive clay containing anhydrous calcium sulfate." *Proceeding of the 4th Int. Conf. on Unsaturated Soils*, April 2-6, 2006, Carefree, Arizona, Vol. 1, 906-915.

Bandyopadhyay, S. S. (1981). "Prediction of swelling potential for natural soils." *Journal of Geotechnical Engineering*, 107(5): 658–691.

Barber, E. S. (1956). "A discussion on engineering properties of expansive clays." *Transactions*, ASCE, 121: 669-673.

Basma, A. A. (1993). "Prediction of expansion degree for natural compacted clays." *Geotechnical Testing Journal*, 16(6), 542-542.

Basma, A. A., Al-Homoud, A. S., and Malkawi, A. I. (2000). "Swelling-shrinkage behavior of natural expansive clays." *Applied Clay Science*, 11(2), 211-227.

Bear, J. (1979). *Hydraulic of Groundwater*. McGraw-Hill, New York.

Blight, G. E. (1965). "The time-rate of heave of structures on expansive clays." In *Moisture equilibria and moisture changes in soils beneath covered areas*, G.D. Aitchison (eds), Butterworths, Sydney, 78-87.

Bolt, G. H. (1956). "Physico-chemical analysis of the compressibility of pure clays." *Géotechnique*, 6(2): 86–93.

Bolt, G. H., and Miller, R. D. (1955). "Compression studies of illite suspensions." *Soil Science Society of America Proceedings*, 19: 285–288.

Brackley, I. J. A. (1975a). "Swell under load." *Proceedings of 6th African Regional Conference of Soil Mechanics and Foundation Engineering*. Lagos, 65-70.

Brackley, I. J. A. (1975b). "A model of unsaturated clay structure and its application to swell behaviour." *Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*, Durban, South Africa, pp. C26–C34.

Briaud, J., Zhang, X., and Moon, S. (2003). "Shrink test–water content method for shrink and swell predictions." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 129(7): 590–600.

Brooks, R. H., and Corey, A. T. (1964). “Hydraulic properties of porous media.” *Colorado State University Hydrology Paper*, No. 3, Fort Collins, CO.

Brutsaert, W. (1967). “Some methods of calculating unsaturated permeability.” *Transactions of ASABE*, Vol. 10, 400-404.

Buckingham, E. (1907). “Studies of the Movement of Soil Moisture.” *U.S.D.A. Bur. of Soils, Bulletin No. 38*.

Burdine, N. T. (1953). “Relative permeability calculations from pore size distribution data.” *Transactions of the Metallurgical Society of AIME*, Vol. 198, 71–78.

Burger, C. A., and Shackelford, C. D. (2001). “Soil-water characteristic curves and dual porosity of sand-diatomaceous earth mixtures.” *Journal of Geotechnical and Environmental Engineering*, ASCE, 127(9): 790-800.

Burland, J. B. (1962). “The estimation of field effective stresses and the prediction of total heave using a revised method of analyzing the double oedometer test.” *The Civil Engineer in South Africa, Transactions of South African Institute of Civil Engineering*, July 1962.

Cameron, D. A., (2001), “The extent of soil desiccation near trees in a semi-arid environment.” *Geotechnical and Geological Engineering*, Kluwer Academic Publishers, 19(3-4): 357–370.

Campbell, G. S. (1974). “A simple method for determining unsaturated conductivity from moisture retention data.” *Soil Science Journal*, 117(6): 311-314.

Casagrande, A. (1936). “The determination of the preconsolidation load and its practical significance.” Disscusion D-34, *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering*, Cambridge, Massachusetts, US, vol. 3, 60-64.

Chen, F. H. (1975). *Foundations on Expansive Soils*. Elsevier Scientific Pub. Co., Amsterdam, Netherlands.

Ching, R. K. H., and Fredlund, D. G. (1984). "A small Saskatchewan town copes with swelling clay problems." *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, Australia, 306-310.

Çimen, Ö., Keskin, S. N., and Yıldırım, H. (2012). "Prediction of swelling potential and pressure in compacted clay." *Arabian Journal of Science and Engineering*, 37(6): 1535-1546.

Clifton, A. W., Yoshida, R. T., and Fredlund, D. G. (1984). "Performance of dark hill, Regina, Canada, constructed on a highly swelling clay." *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, Australia, 197-201.

Çokça E. (2002). "Relationship between methylene blue value, initial soil suction and swell percent of expansive soils." *Turkish Journal of Engineering and Environmental Sciences*, 26(6), 521-530.

Corey, A. T. (1994). "Mechanics of immiscible fluids in porous media." *Water Resources Publications*, Highlands Ranch, Colorado.

Cover, A. P., and Lytton, R. L. (2001). "Estimating soil swelling behaviour using soil classification properties." *Proceedings of Geo-Institute Shallow Foundation and Soil Properties Committee Sessions at the ASCE 2001 Civil Engineering Conference*, Houston, Texas, 44–63.

Crilly, M. S., Driscoll, R. M. C., and Chandler, R. J. (1992). "Seasonal ground and water movement observations from an expansive clay site in the UK." *Proceedings of the 7th International Conference on Expansive Soils*, Dallas, 1, 313-318.

Crilly, M. S., and Driscoll, R. M. C. (2000). "The behaviour of lightly loaded piles in swelling ground and implications for their design." *Proceedings of the ICE-Geotechnical Engineering*, 143(1): 3-16.

Dawson, R. F. (1956). "A discussion on engineering properties of expansive clays." *Transactions*, ASCE, 121: 664-666.

De Bruijn, C. M. A. (1961). "Swelling characteristics of a transported soil profile at Leeuhof Vereeniging (Transvaal)." *Proceedings of the 5th International Conference on Soil Mechanics Foundation Engineering*, Paris, Vol. 1, pp. 43–49.

Dhowian, A. W. (1990a). "Simplified heave prediction model for expansive shale." *Geotechnical Testing Journal*, ASTM, 13(4): 323-333.

Dhowian, A. W. (1990b). "Field performance of expansive shale formation." *Engineering Sciences*, 2(1): 165-182.

Dineen, K. (1997). *The influence of soil suction on compressibility and swelling*. Ph.D. Thesis, University of London (Imperial College), London, UK.

Donaldson, G. W. (1969) "The occurrence of problems of heave and the factors affecting its nature." *Proceedings of the 2nd International Research and Engineering Conference on Expansive Clay Soils*, Texas A&M University, College Station, Tex., August 1969, 25-36.

DuBose, L. A. (1956). "A discussion on engineering properties of expansive clays." *Transactions*, ASCE, 121: 674-676.

Durner, W. (1994). "Hydraulic conductivity estimation for soils with heterogeneous pore structure." *Water Resource Research*, 30(2): 211-223.

Eberl, D. D., (1984). "Clay mineral formation and transportation in rocks and soils." *Philosophical Transactions of the Royal Society of London, Series A*, 311: 241-259.

Edlefsen, N. E., and Anderson, A. B. C. (1943). *Thermodynamics of soil moisture*. University of California.

Elkady, T. Y., Al-Mahbashi, A. M., and Al-Refeai, T. O. (2013a). "Stress-dependent soil-water characteristic curves of lime-treated expansive clay." *Journal of Materials in Civil Engineering*, ASCE, doi: 10.1061/(ASCE)MT.1943-5533.0000995.

Elkady, T. Y., Al-Mahbashi, A. M., Al-Refeai, T. O., and Al Shamrani, M. (2013b).

“Evaluation of soil water characteristic curves of sand-clay mixtures.” *International Journal of GEOMATE*, 4(2): 528-532.

Elkady, T. Y. (2014). “Unsaturated characteristics of undisturbed expansive shale from Saudi Arabia.” *Arabian Journal of Geosciences*, 7(5): 2031-2040.

El Sayed, S. T., and Rabba, S. A. (1986). “Factors affecting behavior of expansive soils in the laboratory and field-a review.” *Journal of Geotechnical and Geoenvironmental Engineering*, 17(1): 89-107.

Erguler, Z. A., and Ulusay, R. (2003). “A simple test and predictive models for assessing swell potential of Ankara (Turkey) Clay.” *Engineering Geology*, 67(3), 331-352.

Erol, A. O., Dhowian, A., and Youssef, A. (1987). “Assessment of oedometer methods for heave prediction.” *Proceedings of 6th International Conference of Expansive Soils*. New Delhi, India, 99-103.

Erzin, Y., and Erol, O. (2007). “Swell pressure prediction by suction methods.” *Engineering Geology*, 92(3): 133-145.

Fargher, P. J., Woodburn, J. A., and Selby, J. (1979). *Footings and Foundations for Small Buildings in Arid Climates: With Special Reference to South Australia.* The Institution of Engineers, South Australian Division, Australia.

Feher, J. J., and Ford, G. D. (1995). “A simple student laboratory on osmotic flow, osmotic pressure, and the reflection coefficient.” *Advances on Physiology Education*, 13(1): 10-20.

Feng, M., Gan, K. M., and Fredlund, D. G. (1998). “A laboratory study of swelling pressure using various test methods.” *Proceedings of the 2nd International Conference on Unsaturated Soils*, Beijing, China, International Academic Publishers, 350-355.

Fityus, S. G., and Smith, D. W. (1998). “A simple model for the prediction of free surface movements on swelling clay profiles.” *Proceedings of the 2nd International Conference on Unsaturated Soils*, Beijing, China, 473-478.

Fityus, S. G., Smith, D. W., and Allman M. A. (2004). "Expansive soil test site near Newcastle." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130(7): 686-695.

Fredlund, D. G. (1969). "Consolidometer test procedural factors affecting swell properties." *Proceedings of the 2nd International Conference on Expansive Clay Soils*, 1969, College Station, Texas, 435-456.

Fredlund, D.G. (1979). "Appropriate concepts and technology for unsaturated soils." *2nd Canadian Geotechnical Colloquium*, *Canadian Geotechnical Journal*, 16(1): 121–139.

Fredlund, D. G., Hasan, J. U., and Filson, H. (1980). "The Prediction of total heave." *Proceedings of the 4th International Conference on Expansive Soils*, Denver, Colorado, 1-17.

Fredlund, D. G. (1983). "Prediction of ground movements in swelling clays." *Proceedings of the 31st Annual Soil Mechanics and Found Engineering Conference*, 1983, University of Minnesota, Minneapolis.

Fredlund, D. G. (1995). "The prediction of heave in expansive soils." *Proceedings of the Canada–Kenya Symposium on Unsaturated Soil Behavior and Applications*, University of Nairobi, Nairobi, Kenya (pp. 105-119).

Fredlund, D. G., and Rahardjo, H. (1993). *Soil mechanics for unsaturated soils*, John Wiley & Sons, New York.

Fredlund, D. G., Rahardjo, H., and Fredlund, M. D. (2012). *Unsaturated soil mechanics in engineering practice*. John Wiley & Sons, New York.

Fredlund, D. G., Rahardjo, H., Leong, E. C., and Ng, C. W. W. (2001). "Suggestions and recommendations for the interpretation of soil-water characteristic curves." *14th Southeast Asian Geotechnical Conference*, Hong Kong, China, 503–508.

Fredlund, D. G., and Xing, A. (1994). "Equations for the soil-water characteristic curve." *Canadian Geotechnical Journal*, 31(4): 521-532.

Fredlund, D. G., Xing, A., and Huang, S. (1994). "Predicting the permeability function for unsaturated soils using the soil-water characteristic curve." *Canadian Geotechnical Journal*, 31(4): 533-546.

Fredlund, M. D., Fredlund, D. G., and Wilson, G. W. (1997). "Prediction of the soil-water characteristic curve from grain-size distribution and volume-mass properties." *Proceedings of the 3rd Brazilian Symposium on Unsaturated Soils, NSAT '97* , Rio de Janeiro, Brazil, Vol. 1, 13–23.

Frydman, S., and Calabresi, G. (1987). "Suggested standard for one dimensional testing." *Proceedings of 6th International Conference on Expansive Soils*. New Delhi, India, 91-98.

Gardner, W. R. (1958). "Some steady state solutions of the unsaturated moisture flow equation with application to evaporation from a water-table." *Soil Science Journal*, 85(4): 228–232.

Gardner, W. and Widtsoe, J. A. (1921). "The Movement of Soil Moisture." *Soil Science*, 11: 215–232.

Gens, A., and Alonso, E. E. (1992). "A framework for the behaviour of unsaturated expansive clays." *Canadian Geotechnical Journal*, 29(6), 1013-1032.

Geo-Slope. (2014). *Vadose Zone Modeling with VADOSE/W: An Engineering Methodology*. 3rd Edition. Geo-Slope, Calgary, Alberta.

Gibbs, H. J. (1973). "Use of a consolidometer for measuring expansion potential of soils." *Proceedings of Workshop on Expansive Clays and Shales in Highway Design and Construction*. University of Wyoming, Laramie, 206-213.

Gilchrist, H. G. (1963). *A study of volume change of a highly plastic clay*. M.Sc. thesis, University of Saskatchewan, Saskatoon, Saskatchewan, Canada.

Gillot, J. G. (1968) *Clay in Engineering Geology*, Elsevier Press, London.

de F. N. Gitirana Jr., G., and Fredlund, D. G. (2004). "Soil-water characteristic curve equation with independent parameters." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 130(2): 209-212.

Gizienski, S. F., and Lee, L. J. (1965). "Comparison of laboratory swell tests to small scale field tests." *Proceedings of 1st International Conference on Expansive Soils*, Texas A&M University, College Station, 108-119.

Greathouse, J. A., Feller, S. E., and McQuarrie, D. A. (1994). "The modified Gouy-Chapman theory: comparisons between electrical double layer models of clay swelling." *Langmuir*, 10(7), 2125-2130.

Grim R. E. (1962). *Applied Clay Mineralogy*, McGraw-Hill, New York, NY.

Grim, R. E. (1968). *Clay Mineralogy*, McGraw-Hill, New York, NY.

Groenevelt, P. H., and Grant, C. D. (2004). "A new model for the soil-water retention curve that solves the problem of residual water contents." *European Journal of Soil Science*, 55(3): 479-485.

Grossman, R. B., Brasher, B. R., Franzmeier, D. P., and Walker, J. L. (1968). "Linear extensibility as calculated from natural-clod bulk density measurements." *Soil Science Society of America Journal*, 32(4), 570-573.

Gupta, S. C., and Larson, W. E. (1979). "Estimating soil-water retention characteristics from particle size distribution, organic matter percent, and bulk density." *Water Resources Research Journal*, 15(6): 1633–1635.

Hamberg, D. J. (1985). *A Simplified Method Ior Predicting Heave in Expansive Soils*. Masters Thesis, Colorado State University, Fort Collins, Colorado.

Hamberg, D. J., and Nelson, J. D. (1984). "Prediction of floor slab heave." *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, South Australia, 137-140.

Han, Z., and Vanapalli, S. K. (2015). “Model for predicting the resilient modulus of unsaturated subgrade soil using the soil-water characteristic curve.” *Canadian Geotechnical Journal*, available online, doi: 10.1139/cgj-2014-0339.

Hardy, R. M. (1965). “Identification and performance of swelling soil type.” *Canadian Geotechnical Journal*, 2(2): 141-153.

Holtz, W.G. (1970). “Suggested method for test for one-dimensional expansion and uplift pressure of clay soils.” *Special Procedures for Testing Soil and Rock for Engineering Purposes*, ASTM STP 479.

Holtz, W. G. and Gibbs, H. J. (1956). “Engineering properties of expansive clays.” *ASCE Centennial Convocation*, Denver, Colorado.

Irwin, G. R. (1958). “Fracture.” In *Handbuch der physik*. (Flugge S. Ed.) Vol. 6. Springer-Verlag, Berlin, Germany.

ISRM (1989). “Commission on swelling rock and working group on swelling-rock of the commission on testing methods (suggested methods for laboratory testing of argillaceous swelling rocks).” *International Journal of Rock Mechanics, Mining Science and Geomechanics Abstracts*, 26(5): 415-426.

Ito, M., and Azam, S. (2009). “Engineering characteristics of a glacio-lacustrine clay deposit in a semi-arid climate.” *Bulletin of Engineering of Geology and the Environment*, 68(4): 551-557.

Jaksa, M. B. (1995). *The influence of spatial variability on the geotechnical design properties of a stiff, overconsolidated clay*. Ph.D. Thesis, University of Adelaide, Adelaide, Australia.

Jaksa, M. B., Kaggwa, W. S., Woodburn, J. A., and Sinclair, R. (2002). “Influence of large gums trees on the soil suction profile in expansive clays.” *Australian Geomechanics*, 71(1): 23-33.

Jennings, J. E. B. and Knight, K. (1957). “The prediction of total heave from double oedometer test." *Proceedings of Symposium on Expansive Clays*, South African Institution of Civil Engineers, Johannesburg, 13–19.

Jennings, J. E., Firth, R. A., Ralph, T. K., and Nagar, N. (1973). "An improved method for predicting heave using the oedometer test." *Proceedings of International Conference of Expansive Soils*, Haifa, Israel, 149-154.

Johnson, L. D. (1977). *Evaluation of laboratory suction tests for prediction of heave in foundation soils*. Technical Report S-77-7, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss.

Johnson, L. D. (1978). *Predicting potential heave and heave with time in swelling foundation soils*. US Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss. Technical report S-78-7.

Johnson, L. D. (1979). *Overview for Design of Foundations on Expansive Soils*. (No. WES/MP/GL-79-21), U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, Miss.

Johnson, L. D., and Snethen, D. R. (1978). "Prediction of potential heave of swelling soils." *Geotechnical Testing Journal*, ASTM, 1(3): 117-124.

Jones, L. D., and Jefferson, I. (2012). "Expansive soils." *ICE Manual of Geotechnical Engineering* (Burland J. ed.), Vol 1, 413-441

Jones, E. W., Koh, Y. H., Tiver, B. J., and Wong, M. A. H. (2009). "Modelling the behaviour of unsaturated, saline clay for geotechnical design." School of Civil, Environmental and Mining Engineering, University of Adelaide.

Justo, J. L., Delgado, A., and Ruiz, J. (1984). "The influence of stress-path in the collapse-swelling of soils at the laboratory." *Proceedings of 5th International Conference of Expansive Soils.* Adelaide, Australia, 67-71.

Kassiff, G., and Shalom, A. B. (1971). "Experimental relationship between swell pressure and suction." *Géotechnique*, 21(3): 249-255.

Khattab, S. A. A., Al-Mukhtar, M., and Fleureau, J. M. (2002). "Effect of initial suction on the swelling pressure and porosity of lime stabilized clays." *Proceedings of the 3rd International Conference on Unsaturated Soils, UNSAT 2002*, 10-13 March 2002, Recide, Brazil, Vol. 2, 605-609.

Komornik, J. and David, A. (1969). "Prediction of swelling potential for compacted clays." *Journal of the Soil Mechanics and Foundation Engineering Division*, ASCE, 95 (1), 209–225.

Komornik, A., Wiseman, G., and Ben-Yaacob, Y. (1969). "Studies of in-situ moisture and swelling potential profiles." *Proceedings of 2nd International Conference of Expansive Soils*, Texas, 348-361.

Konrad, J. M., and Ayad, R. (1997). "A idealized framework for the analysis of cohesive soils undergoing desiccation." *Canadian Geotechnical Journal*, 34(4), 477-488.

Krahn, J., and Fredlund, D. G. (1972). "On total matric and osmotic suction." *Soil Science*, 114(5): 339–348.

Kumar, N., (1984). "Review of common design and construction practices on expansive soils in Colorado." *Proceedings of 5th International Conference of Expansive Soils.* Adelaide, Australia, 165-169.

Lachenbruch, A. H. (1961). "Depth and spacing of tension cracks." *Journal of Geophysical Research*, 66(12): 4273–4292.

Laliberte, G. E. (1969). "A mathematical function for describing capillary pressure-desaturation data." *Bulletin of the International Association Hydrological Sciences*, 14(2): 131–149.

Lambe, T. W. and Whitman, R. V. (1959). "The role of effective stress in the behavior of expansive soils." *Quarterly of the Colorado School of Mines*, 54(4): 33–66.

Lee, F. H., Lo, K. W., and Lee, S. L. (1988). "Tension crack development in soils." *Journal of Geotechnical Engineering*, ASCE, 114(8): 915–929.

Leong, E. C., and Rahardjo, H. (1997). "A review on soil-water characteristic curve equations." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 123(12): 1106–1117.

Li, X., Li, J. H., and Zhang, L. M. (2014). "Predicting bimodal soil–water characteristic curves and permeability functions using physically based parameters." *Computers and Geotechnics*, 57: 85–96.

Li, J., Sun, D., Sheng, D. C., Sloan, S., and Fredlund, D. G. (2007). "Preliminary study on soil water characteristics of Maryland clay." *Proceedings of the 3rd Asian Conference on Unsaturated Soils*, Nanjing, China, 569-574.

Lin, B., and Cerato, A. B. (2012a). "Prediction of expansive soil swelling based on four micro-scale properties." *Bulletin of Engineering Geology and the Environment*, 71(1): 71-78.

Lin, B., and Cerato, A. B. (2012b). "Investigation on soil-water characteristic curves of untreated and stabilized highly clayey expansive soils." *Geotechnical and Geological Engineering*, 30(4): 803-812.

Likos, W. J. (2004). "Measurement of Crystalline Swelling in Expansive Clay." *Geotechnical Testing Journal*, 27(6): 540-546.

Lytton, R. L., (1977a). "The characterization of expansive soils in engineering." *Symposium on Water Movement and Equilibrium in Swelling Soils*, American Geophysical Union, San Francisco, CA.

Lytton, R. L. (1977b). "Foundations in expansive soils." *Numerical Methods in Geotechnical Engineering*, Christian J. T. (eds.), McGraw-Hill.

Lytton, R. L. (1994). "Prediction of movement in expansive clays." *Proceedings of Vertical and Horizontal Deformations of Foundations and Embankments*, *Geotechnical Special Publication No. 40*, A. T. Yeung and G. Y. Felio, eds., Vol. 2, ASCE, New York, 1827–1845.

Lytton, R. L., and Woodburn, J. A. (1973). "Design and performance of mat foundations on expansive clay." *Proceedings of the 3rd International Conference on Expansive Soils*, Society of Soil Mechanics and Foundation Engineering Haifa, Israel, Vol. 1, 301–307.

Lytton, R. L., Aubeny, C., and Bulut, R. (2004). *Design procedure for pavements on expansive soils*. Report No. FHWA/TX-05/0-4518-1. Texas Department of Transportation, Austin, Texas.

Mackenzie, R. C. (1963). "De natura lutorum." *Clays and Clay Minerals*, 11: 11-28.

Mackenzie, R. C., and Mitchell, B. D. (1966). "Clay mineralogy." *Earth-Science Reviews*, 2: 47-91.

Mallants, D., Tseng, P. H., Toride, N., Timmerman, A., and Feyen, J. (1997). "Evaluation of multimodal hydraulic functions in characterizing a heterogeneous field soil." *Journal of Hydrology*, 195(1): 172-199.

Matyas, E. L., (1969). "Some properties of two expansive clays from Western Canada." *Proceedings of 2nd International Conference on Expansive Clay Soils*, Texas A&M University, College Station, 263-278.

McCormack, D. E., and Wilding, L. P. (1975). "Soil properties influencing swelling in Canfield and Geeburg soils." *Soil Science Society of America Journal*, 39(3): 496–502.

McKeen, R. G. (1981). *Design of Airport Pavements for Expansive Soils*. Federal Aviation Agency, U.S. Department of Transportation, Washington.

McKeen, R. G. (1985). *Validation of procedures for pavement design on expansive soils (No. NMERI-WA5-4 (5.02))*. New Mexico Engineering Research Institute, Albuquerque.

McKeen, R. G., and Johnson, L. D. (1990). "Climate-controlled soil design parameters for mat foundations." *Journal of Geotechnical engineering*, 116(7): 1073-1094.

McKeen, R. G. (1992). “A model for predicting expansive soil behavior.” *Proceedings of the 7th International Conference on Expansive Soils*, Dallas, Texas, 1-6.

McKee, C. R., and Bumb, A. C. (1984). “The importance of unsaturated flow parameters in designing a hazardous waste site.” *Hazardous Waste and Environmental Emergencies: Hazardous Materials Control Research Institute National Conference*, Houston, TX, Silver Spring, MD, 50-58.

McWhorter, D. B., and Sunada, D. K. (1977). “Ground-water hydrology and hydraulics.” *Water Resources Publications*, Highlands Ranch, Colorado.

Mesri, G., and Olsen, R. E. (1971). “Consolidation characteristics of montmorillonite.” *Géotechnique*, 21(4): 341–352.

Miller, D. J., and Nelson, J. D. (2006). “Osmotic suction in unsaturated soil mechanics.” *Unsaturated Soils 2006*, ASCE, 1382-1393.

Mitchell, J. K. (1960). “The application of colloidal theory to compressibility of clays.” *Proceedings of the Seminar on Interparticle Forces in Clay–Water–Electrolyte Systems*, Vol. 2, Commonwealth Scientific and Industrial Research Organisation, Melbourne, Australia, 92–97.

Mitchell, P. W. (1979). *The structural analysis of footings on expansive soils*. Research Report No.1, Kenneth W. G. Smith and Associates, Adelaide, South Australia, Australia.

Mitchell, P.W. (1980). “The concepts defining the rate of swell of expansive soils.” *Proceedings of the 4th International Conference on Expansive Soils*, Denver, Colorado, Vol. 1, 106-116.

Mitchell, P. W., and Avalle, D. L. (1984). “A technique to predict expansive soils movement.” *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, Australia, 124–130.

Mitchell, J. K., and Soga, K. (2005). *Fundamentals of Soil Behavior*. 3rd Edition. John

Wiley & Sons, Inc., New Jersey.

Morris, P. H., Graham, J., and Williams, D. J. (1992). “Cracking in drying soils.” *Canadian Geotechnical Journal*, 29: 263–277.

Mualem, Y. (1976). “A new model for predicting hydraulic conductivity of unsaturated porous media.” *Water Resources Research*, 12: 513–522.

Nagaraj, H. B., Munnas, M., and Sriharan, A. (2009). “Critical evaluation of determining swelling pressure by swell-load method and constant volume method.” *Geotechnical Testing Journal*, ASTM, 32(4): 1-10.

NAVFAC. (1971). *Soil Mechanics, Foundation and Earth Structures*. Design Manual, NAVFACDM-7, Naval Facilities Engineering Command, Department of the Navy, Bureau of yards and Docks, Washington, DC.

Nayak, N. V., and Christensen, R. W. (1971). “Swelling characteristics of compacted expansive soils.” *Clays and Clay Minerals*, 19(4), 251-261.

Nelson, J. D., Durkee, D. B., Reichler, J. D., and Miller, D. J. (1994). *Moisture Movement and Heave beneath Simulated Foundation Slabs on Expansive Soils*. prepared for US Army Corps of Engrs. by Civil Engr. Dept., Colorado State University.

Nelson, J.D., Durkee, D.B., and Bonner, J.P. (1998). “Prediction of free field heave using oedometer test data.” *Proceedings of 46th Annual Geotechnical Engineering Conference*, University of Minnesota, St. Paul, MN.

Nelson, J. D., and Miller, D. J. (1992). *Expansive Soils: Problems and Practice in Foundation and Pavement Engineering*. John Wiley and Sons, Inc., New York.

Nelson, J. D., Overton, D. D., and Durkee, D. B. (2001). “Depth of wetting and the active zone.” *Geotechnical Special Publication*, 95-109.

Nelson, J. D., Reichler, D. K., and Cumbers, J. M. (2006). “Parameters for heave prediction by oedometer tests.” *Unsaturated Soils 2006*, ASCE, 951-961.

Ng, C. W. W., Zhan, L. T., Fredlund, D. G., and Gong, B. W. (2003). "Performance of an unsaturated expansive soil slope subjected to artificial rainfall infiltration." *Géotechnique*, 53(2): 143-157.

Newman, A. C. (1987). *Chemistry of clays and clay minerals*. Longman Scientific and Technical, Harlow, Essex, England.

Noble, C. A. (1966). "Swelling measurements and prediction of heave for a lacustrine clay." *Canadian Geotechnical Journal*, 3(1): 32-41.

Nwaiwu, C. M. O., and Nuhu, I. (2006). "Evaluation and prediction of the swelling characteristics of Nigerian black clays." *Geotechnical and Geological Engineering*, 24(1): 45-56.

Oh, W. T., Vanapalli, S. K., and Puppala, A. J. (2009). "Semi-empirical model for the prediction of modulus of elasticity for unsaturated soils." *Canadian Geotechnical Journal*, 46(8): 903-914.

O'Neil, M. W., and Ghazzally, O. I. (1977). "Swell potential related to building performance." *Journal of the Geotechnical Engineering Division*, ASCE, 103(12): 1363–1379.

Overton, D. D., Chao, K. C., and Nelson, J. D. (2006). "Time rate of heave prediction for expansive soils." *Proceedings of the ASCE Conference GeoCongress 2006*, Atlanta, Georgia, United States, 162-168.

Pan, H., Qing, Y., and Pei-yong, L. (2010). "Direct and indirect measurement of soil suction in the laboratory." *Electronic Journal of Geotechnical Engineering*, 15(3): 1-14.

Pan, Z., Xie, Y., Yang, X., and Li, Z. (2006). "Depth identification of the active and cracking zones in expansive soils from in-situ suction measurement." Journal of Engineering Geology, 14(2): 206-211 (in Chinese).

PDE Solutions Inc. (2015). *FlexPDE 6.37 User Manual*. PDE Solutions Inc., Antioch, CA.

Pedarla, A., Puppala, A. J., Hoyos, L. R., Vanapalli, S. K., and Zapata, C. (2012). "SWRC modelling framework for evaluating volume change behavior of expansive Soils." *Proceeding of E-UNSAT 2012*, June, 2012, Napoli, Italy, 221-228.

Pedarla, A. (2013). *SWCC and clay mineralogy based models for realistic simulation of swell behavior of expansive soils*. Ph.D. Thesis, University of Texas at Arlington, Arlington, US.

Pereira, J. H. F., and Fredlund, D. G. (2000). "Volume change behaviour of a residual soil of gneiss compacted at metastable-structured conditions." *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 126, 907-916.

Pereira, E. M., Pejon, O. J., and Zuquette, L. V. (2012). "Swelling tests on material from the Guabirotuba Formation, Parana, Brazil, with suction control." *Quarterly Journal of Engineering Geology and Hydrogeology*, 45(1): 71-78.

Perko, H. A., Thompson, R. W., and Nelson, J. D. (2000). "Suction compression index based on CLOD test results." *Advances in Unsaturated Geotechnics*, 393-408.

Pham, H. Q., and Fredlund, D. G. (2005). "A volume-mass constitutive model for unsaturated soils." *Proceedings of the 58th Canadian Geotechnical Conference*, Saskatoon, SK, Vol. 2, 173-181.

Picornell, M., and Lytton, R. L. (1984). "Modeling the heave of a heavily loaded foundation." *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, Australia, 104-108.

Porter, A. A., and Nelson, J. D. (1980). "Strain controlled testing of expansive soils." *Proceedings of 4th International Conference of Expansive Soils*, Denver, Colorado, 34-44.

PTI. (2004). *Design and construction of post-tensioned slabs-on-ground*. PTI, Phoenix, AZ.

Qi, S., and Vanapalli, S. K. (2015). "Numerical study on expansive soil slope stability considering the cracks and coupling effects." *Proceedings of the 6th Asia-Pacific Conference on Unsaturated Soils*, Guilin, China.

Rampino, C., Mancuso, C., and Vinale, F. (2000). "Experimental behaviour and modeling of an unsaturated compacted soil." *Canadian Geotechnical Journal*, 37(4): 748–763.

Ranganatham, B. V., and Satyanarayana, B. (1965). "A rational method of predicting swelling potential for compacted expansive clays." *Proceedings of the 6th International Conference on Soils Mechanics and Foundation Engineering*, Montreal, 92-96.

Rao, A. S., Phanikumar, B. R., and Sharma, R. S. (2004). "Prediction of swelling characteristics of remoulded and compacted expansive soils using free swell index." *Quarterly Journal of Engineering Geology and Hydrogeology*, 37(3), 217-226.

Rao, R. R., Rahardjo, H., and Fredlund, D. G. (1988). "Closed-form heave solutions for expansive soils." *Journal of Geotechnical Engineering*, 114(5), 573-588.

Rassam, D. W., and Williams, D. J. (1999). "A relationship describing the shear strength of unsaturated soils." *Canadian Geotechnical Journal*, 36(2): 363–368.

Richards, B. G. (1965). "Measurement of the free energy of soil moisture by the psychrometric technique using thermistors." *Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas* (Aitchison, G. D., ed.), A Symposium in Print, Butterworths, Sydney, 39–46.

Richards, B. G. (1967). "Moisture flow and equilibria in unsaturated soils for shallow foundations." *Permeability and Capillarity of Soils*, *STP 417*, ASTM, Philadelphia, PA, 4-34.

Richards, B. G., Peter, P., and Emerson, W. W. (1983). "The effects of vegetation on the swelling and shrinking of soils in Australia." *Géotechnique*, 33(2): 127-139.

Richards, L. A. (1928). "The usefulness of capillary potential to soil moisture and plant investigators." *Journal of Agricultural Research*, 37 (12): 719–742.

Russam, K., and Coleman, J. D. (1961). "The effect of climatic factors on subgrade moisture condition." *Géotechnique*, 3(1), 22-28.

Sabtan, A. A. (2005). "Geotechnical properties of expansive clay shale in Tabuk, Saudi Arabia." *Journal of Asian Earth Sciences*, 25(5), 747-757.

Sampson, E., Shuster, R. L., and Budge, W. D. (1965). "A method of determining swell potential of an expansive clay." *Proceedings of 1st International Conference of Expansive Soils*. Texas, 255-275.

Satyanaga, A., Rahardjo H, Leong, E. C., and Wang, J. Y. (2013). "Water characteristic curve of soil with bimodal grain-size distribution." *Computers and Geotechnics*, 48: 51-61.

Schafer, W. M., Singer, M. J. (1976). "Influence of physical and mineralogical properties on swelling of soils in Yolo County, California." *Soil Science Society of America Journal*, 40(4): 557–562.

Schneider, G. L., and Poor, A. R. (1974). *The prediction of soil heaves and swell pressures developed by expansive clay*. Construction Research Centre, CRC Publications, Tex. Research Report TR-9-74.

Schreiner, H. D., and Burland, J. B. (1991). "A comparison of the three swell test procedures." *Proceedings of 10th Regional Conference for Africa on Soil Mechanics and Foundation Engineering*. Maseru, Lesotho, pp. 259-266.

Seed, B., Woodward, R. J., and Lundgren, R. (1962). "Prediction of swelling potential for compacted clays." *Journal of the Soil Mechanics and Foundations Division*, ASCE, 88: 53-87.

Šimůnek, J., van Genuchten, M. Th., and Šejna, M. (2012). *The HYDRUS software package for simulating the two- and three- dimensional movement of water, heat, and multiple solutes in variably-saturated porous media*. Version 2.0, User Manual.

Shah, I. (2011). *The performance of a steel basement house in Regina*. Report submitted to the Environmental Systems Engineering, Faculty of Engineering and Applied Science, University of Regina, SK, Canada.

Shamburger, J. H., Patrick, D. M., and Lutten, R. J. (1975). Survey of Problem Areas and Current Practices (Report FHWA-RD-75-61), In *Design and Construction of Compacted Shale Embankments*, U.S. Department of Transportation, Vol. 1, 292.

Shankar, N. B., Rao, A. S., and Swamy, A. S. R. (1982). "Swelling behaviors of undisturbed and remolded samples of Black Cotton Clay." *Indian Geotechnical Journal*, 12(2): 152-159.

Sheard, M. J., and Bowman, G. M. (1994). "Soils, stratigraphy and engineering geology of near surface materials of the Adelaide Plains." *S. A. Department of Mines and Energy Report Book*, September 1994, Adelaide, Australia.

Sheikhtaheri, M. (2014). *Experimental and numerical modeling studies for interpreting and estimating the p–δ behavior of single model piles in unsaturated sands*. Doctoral dissertation, University of Ottawa.

Sherry, G . P (1982). *Constitutive relationship for unsaturated uranium mill tailings*. Masters Thesis, Colorado State University, Fort Collins, CO.

Shi, Q., and Xu, N. (1990) "Active layer of expansive soil and its behaviour." *Proceedings of the 1st Chinese Symposium on Expansive Soils*, Chengdu, China (in Chinese).

Shuai, F. (1996). *Simulation of swelling pressure measurements on expansive soils*. Ph.D. Thesis, University of Saskatchewan, Saskatoon, Canada.

Simms, P. H., and Yanful, E. K. (2001). "Measurements and estimation of pore shrinkage and pore distribution in a clayey till during soil-water characteristic curve tests." *Canadian Geotechnical Journal*, 38(4): 741-754.

Singhal, S. (2010). *Expansive soil behavior: property measurement techniques and heave prediction methods*. Ph.D. Thesis, Arizona State University, Phoenix, US.

Singhal, S., Houston, S., and Houston, W. N. (2011). “Effects of testing procedures on the laboratory determination of swell pressure of expansive soils.” *Geotechnical Testing Journal*, ASTM, 34(5): 476-488.

Singhal, S., Houston, S., and Houston, W. N. (2014). “Swell Pressure, Matric Suction, and Matric Suction Equivalent for Undisturbed Expansive Clays.” *Canadian Geotechnical Journal*, DOI. 10.1139/cgj-2014-0159.

Skempton, A. W. (1961). “Effective stress in soils, concrete and rocks.” *Proceedings of the Conference on Pore Pressure and Suction in Soils*, Butterworths, London, UK, 4–16.

Smettem, K. R. J., Kirby C. (1990). “Measuring the hydraulic properties of a stable aggregated soil.” *Journal of Hydrology*, 117(1): 1-13.

Smith, A. W. (1973). “Method for determining the potential vertical rise, PVR.” *Proceedings of Workshop on Expansive Clays and Shales in Highway Design and Construction*. University of Wyoming, Laramie, 189-205.

Snethen, D. R. (1980). “Characterization of expansive soils using soil suction data.” *Proceedings of the 4th International Conference on Expansive Soils*. Denver, Colorado, 54–75.

Snethen, D.R., Townsend, F.C., Johnson, L.D., Patrick, D.M., and Vedros, P. J. (1975). *A review of engineering experiences with expansive soils in highway subgrades*. Interim Report (No. FHWA-RD-75-48), Army Engineer Waterways Experiment Station, Vicksburg, MS.

SoilInfo v0.3.3 (2014). 34°55′13″S, 138°38′42″E, ISRIC-World Soil Information [Accessed 28 December 2014].

SoilVision Systems Ltd. (2009). *SVFlux User’s Manual*. SoilVision Systems Ltd., Saskatoon, SK.

Sridharan, A., and Jayadeva, M. S. (1982). “Double layer theory and compressibility of clays." *Géotechnique*, 32(2): 133–144.

Sridharan, A., Rao, A. S., and Sivapullaiah, P. V. (1986). "Swelling pressure of clays." *Geotechnical Testing Journal*, ASTM, 9(1): 24-31.

Stavridakis, E. I. (2006). "A solution to the problem of predicting the suitability of silty-clayey materials for cement-stabilization." *Geotechnical and Geological Engineering*, 24(2): 379-398.

Subba Rao, K.S., and Tripathy, S. (2003). "Effect of aging on swelling and swell-shrink behavior of a compacted expansive soil." ASTM, *Geotechnical Testing Journal*, 26(1): 36-46.

Sullivan, R. A., and McClelland, B. (1969). "Predicting heave of buildings on unsaturated clay." *Proceedings of the Second International Research and Engineering Conference on Expansive Clay soils*, Texas A&M University Press. College Station, TX, 404–420.

Sweeney, D. J. (1982). "Some in situ soil suction measurements in Hong Kong's residual soil slopes." *Proceedings of the 7th Southeast Asian Geotechnical Conference*, Hong Kong, 91-106.

Taylor, R. K., and Smith T. (1986). "The engineering geology of clay minerals: swelling, shrinking and mudrock breakdown." *Clay Minerals*, 21(3): 235-260.

Teng, T. C. P., and Clisby, M. B. (1975). "Experimental work for active clays in Mississippi." *Transportation Engineering Journal*, 101(1): 77-95.

Teng, T. C. P., Mattox, R. M., and Clisby, M. B. (1972). *A study of active clays as related to highway design*. Mississippi State Highway Department, Jackson.

Teng, T. C. P., Mattox, R. M., and Clisby, M. B. (1973). "Mississippi experimental work on active clays." In: D.R. Lamb and S.J. Hanna (Editors), *Proceedings of Workshop on Expansive Clays and Shales in Highway Design and Construction*, 1-27.

Thompson, R. W., Perko, H. A. and Raethamel, W. D. (2006). "Comparison of constant volume swell pressure and oedometer load-back pressure." *Proceedings of 4th International Conference of Unsaturated Soils*. Carefree, Arizona, 1382-1393.

Thornthwaite, C. W. (1948). "An approach toward a rational classification of climate." *Geographical Review*, 38(1), 54-94.

Thu, T. M., Rahardjo, H., and Leong, E. C. (2007). "Elastoplastic model for unsaturated soil with incorporation of the soil-water characteristic curve." *Canadian Geotechnical Journal*, 44(1): 66-77.

Tourtelot, H. A. (1973). "Geologic origin and distribution of swelling clays." *Proceedings of Workshop on Expansive Clay and Shale in Highway Design and Construction*, I.

Tripathy, S., Sridharan, A., and Schanz, T. (2004). "Swelling pressures of compacted bentonites from diffuse double layer theory." *Canadian Geotechnical Journal*, 41(3), 437-450.

Türköz, M., and Tosun, H. (2011). "The use of methylene blue test for predicting swell parameters of natural clay soils." *Scientific Research and Essays*, 6(8): 1780–1792.

Twarakavi, N. K. C., Šimůnek, J., and Seo, S. (2008). "Evaluating interactions between groundwater and vadose zone using the HYDRUS-based flow package for MODFLOW." *Vadose Zone Journal*, 7(2): 757-768.

Unsaturated Soils Group. (1996). *Soilcover User's Manual Version 3.0*. Unsaturated Soils Group, Department of Civil Engineering, University of Saskatchewan, Saskatoon, SK.

van der Merwe, D. H. (1964). "The prediction of heave from the plasticity index and percentage clay fraction of soils" *Civil Engineers in South Africa*, 6:337–42.

van Genuchten, M. T. (1980). "A closed-form equation for predicting the hydraulic conductivity of unsaturated soils." *Soil Science Society of America Journal*, 44(5), 892-898.

van Olphen, H. (1991). *An Introduction to Clay Colloid Chemistry*, 2nd Ed. Malaba, Florida, Krieger.

Vanapalli, S. K., Fredlund, D. G., Pufahl, D. E., and Clifton, A. W. (1996). "Model for the prediction of shear strength with respect to soil suction." *Canadian Geotechnical Journal*, 33(3): 379-392.

Vanapalli, S. K., Fredlund, D. G., and Pufahl, D. E. (1999). "The influence of soil structure and stress history on the soil–water characteristics of a compacted till." *Géotechnique*, 49(2), 143-159.

Vanapalli, S. K., Lu, L., and Oh, W. T. (2010). "Estimation of swelling pressure and 1-D heave in expansive soils." *Proceeding of the 5th International Conference on Unsaturated Soils*, September 6-8, 2010, Barcelona, Spain, E.Alonso, A.Gens (eds), CRC Press (pubs), Vol. 2, 1201-1207.

Vanapalli, S. K., Lu, L., and Oh, W. T. (2010). "A model for predicting the modulus of elasticity of unsaturated soils using the soil-water characteristic curve." *International Journal of Geotechnical Engineering*, 4(4): 425-433.

Vanapalli, S. K., and Lu, L. (2012). "A state-of-the art review of 1-D heave prediction methods for expansive soils." *International Journal of Geotechnical Engineering*, 6(1): 15-41.

Vanapalli, S. K., Lu, L., Sedano, J. I., and Oh, W. T. (2012). "Swelling characteristics of sand-bentonite mixtures." In *Unsaturated Soils: Research and Applications*, Springer Berlin Heidelberg, 77-84.

Vanapalli, S. K., and Oh, W. T. (2010). "A model for predicting the modulus of elasticity of unsaturated soils using the soil-water characteristic curve." *International Journal of Geotechnical Engineering*, 4(4): 425–433.

Vijayvergiya, V. N., and Ghazzaly, O. I. (1973). "Prediction of swelling potential for natural clays." *Proceedings of the 3rd International Conference on Expansive Clay Soils*, Haifa, Israel, Vol 1, 227–234.

Vu, H. Q. (2002). *Uncoupled and coupled solutions of volume change problems in expansive soils*. Ph.D. Thesis, University of Saskatchewan, Saskatoon, Canada.

Vu, H. Q., Hu, Y., and Fredlund, D. G. (2007). “Analysis of soil suction changes in expansive Regina clay.” *Proceedings of GeoOttawa2007*, Ottawa, Canada.

Walsh, K. D., Colby, C. A., Houston, W. N., and Houston, S. L. (2009). “Method for evaluation of depth of wetting in residential areas.” *Journal of Geotechnical and Geoenvironmental Engineering*, 135(2), 169-176.

Wang, Y. H., and Freeland, D. G. (2003). “Towards a better understanding of the role of the contractile skin.” *Proceedings of the 2nd Asian Conference on Unsaturated Soils*, April 2003, 15-17.

Warkentin, B. P., Bolt, G. M., and Miller, R. D. (1957). “Swelling pressure of montmorillonite.” *Soil Science Society of America Proceedings*, 21: 495–497.

Weston, D. J. (1980). “Expansive roadbed treatment for Southern Africa.” *Proceedings of the 5th International Conference on Expansive Soils*, Denver, CO., ASCE, 339–360.

Wilson, G. V., Jardine, P. M., and Gwo J. P. (1992). “Modeling the hydraulic properties of a multi-region soil.” *Soil Science Society of America Journal*, 56(6): 1731-1737.

Wong, H. Y., and Yong, R. M. (1973). “A study of swelling and swelling force during unsaturated flow in expansive soils.” *Proceedings of 3rd International Conference of Expansive Soils*, Haifa, Israel, 143-151.

Wray, W. K. (1984). “The principle of soil suction and its geotechnical engineering applications.” *Proceedings of the 5th International Conference on Expansive Soils*, Adelaide, Australia, 114–118.

Wray, W. K., El-Garhy, B. M., and Youssef, A. A. (2005). “Three-dimensional model for moisture and volume changes prediction in expansive soils.” *Journal of Geotechnical and Geoenvironmental Engineering*, 131(3), 311-324.

Yao, Hai-lin., Cheng, P., and Wu, Wan-ping. (2005). "Prediction of ground heaves in expansive soils based on consolidation tests." *Chinese Journal of Rock Mechanics and Engineering*, 24(21): 3911-3915 (in Chinese).

Yilmaz, I. (2009). "Swell potential and shear strength estimation of clays." *Applied Clay Science*, 46: 376–384.

Yoshida, R. T., Fredlund, D. G., and Hamilton J. J. (1983). "The prediction of total heave of a slab-on-grade floor on Regina clay." *Canadian Geotechnical Journal* 20(1): 69–81.

Zhan, L. T. (2003). *Field and laboratory study of an unsaturated expansive soil associated with rain-induced slope instability*. Doctoral dissertation, The Hong Kong University of Science and Technology

Zhan, L. T., Chen, P., and Ng, C. W. W. (2007). "Effect of suction change on water content and total volume of an expansive clay." *Journal of Zhejiang University (SCIENCE A)*, 8(5): 699-706.

Zhang, X., and Briaud, J. (2010). "Coupled water content method for shrink and swell prediction." *International Journal of Pavement Engineering*, 11(1): 13–23.

Zhang, L., and Chen, Q. (2005). "Predicting bimodal soil-water characteristic curves." *Journal of Geotechnical and Geoenvironmental Engineering*, 131(5): 666-670.

Zhang, J., Jiao, J. J., and Yang, J. (2000). "In situ rainfall infiltration studies at a hillside in Hubei Province, China." *Engineering Geology*, 57(1): 31-38.

Zumrawi, M. (2013). "Swelling potential of compacted expansive soils." *International Journal of Engineering Research and Technology*, 2(3): 1-6.

APPENDIX

Table A.1 Detailed procedures for estimating the swelling pressure and the heave of London clay site (Case Study A)

							Wetting Curve-Based Estimation				**Drying Curve-Based Estimation**			
Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	P_{si} (kPa)	P_{sw} (kPa)	e_w	Δh (mm)	P_{si} (kPa)	P_{sw} (kPa)	e_w	Δh (mm)
						Eq. (3.9)	Eq. (3.6)	Eq. (3.6)	Eq. (4.4), (4.5)	Eq. (4.4), (4.5)	Eq. (3.6)	Eq. (3.6)	Eq. (4.4), (4.5)	Eq. (4.4), (4.5)
1	0.5	27.6	20.7	15.7	0.764	0.144	139	63	0.816	-14.85	141	63	0.813	-15.06
2	0.5	29.6	19.9	25.9	0.818	0.144	152	59	0.878	-16.63	154	60	0.874	-16.89
3	0.5	28.8	20.0	35.8	0.803	0.144	259	88	0.873	-19.34	268	88	0.863	-20.00
4	0.5	27.5	20.1	45.9	0.773	0.144	343	163	0.821	-13.69	360	166	0.806	-14.44
5	0.5	27.7	19.6	55.8	0.780	0.144	363	240	0.808	-7.69	382	246	0.790	-8.27
6	0.5	27.8	19.6	65.6	0.783	0.144	355	255	0.804	-6.11	371	261	0.788	-6.56
7	0.5	27.5	19.8	75.5	0.770	0.144	319	232	0.791	-5.85	330	236	0.779	-6.19
8	0.5	28.0	19.9	85.4	0.779	0.144	283	196	0.803	-6.70	289	197	0.794	-6.95
9	0.5	29.1	19.7	95.3	0.808	0.144	267	153	0.843	-9.72	271	154	0.836	-9.98
10	0.5	29.9	19.5	105.1	0.829	0.144	268	136	0.872	-11.65	271	136	0.865	-11.92
11	0.5	29.9	19.4	114.8	0.829	0.144	276	142	0.871	-11.46	280	142	0.864	-11.72
12	0.5	28.8	19.6	124.6	0.797	0.144	267	149	0.834	-10.26	270	149	0.829	-10.46
13	0.5	27.5	20.0	134.5	0.760	0.144	242	154	0.788	-8.07	243	154	0.785	-8.18
14	0.5	27.0	20.1	144.5	0.746	0.144	226	160	0.768	-6.22	226	160	0.765	-6.27
15	0.5	27.0	20.1	154.5	0.745	0.144	235	167	0.766	-6.07	235	167	0.764	-6.12
										-154.32				-159.00

Note: e_i is back-calculated, e_w and Δh are determined by solving equation set (i.e., Equation 4.4 and 4.5).

Table A.2 Detailed procedures for estimating the swelling pressure of Al-Qatif clay site (Case Study B)

Depth (m)	w_i (%)	ψ_i (kPa)	†S (/100)	$\rho_{d,\max}$ (Mg/m^3)	w_l (%)	I_p (%)	c (%)	A	β_n	P_s (kPa)
									Eq. 7	Eq. 6
0.5	30	940	0.82	1.2	135	70	75	0.93	0.295	368
1.0	31	942	0.82	1.2	131	91	75	1.21	0.475	593
2.0	35	940	0.82	1.2	130	70	75	0.93	0.295	385
3.0	40	920	0.82	1.2	150	95	75	1.27	0.519	650
4.0	45	850	0.82	1.2	100	50	75	0.67	0.209	302
6.0	50	685	0.84	1.2	110	50	75	0.67	0.209	309
8.0	50	496	0.86	1.2	70	40	75	0.53	0.191	300

† The degree of saturation is estimated from the SWCC based on known soil suction values

Table A.3 Detailed procedures for estimating the swelling pressure and the heave of Maryland clay site (Case Study C)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	ψ_w (kPa)	P_{sw} (kPa)	e_w	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)		Eq. (3.6)	Eq. (4.4), (4.5)	Eq. (4.4), (4.5)
1	0.08	8.7	22.28	5.35	0.470	0.0593	0.20	13234.1	649.6	164.8	37.3	0.677	-4.14
2	0.22	9.0	21.67	8.62	0.477	0.0875	0.24	12196.5	763.1	204.1	56.4	0.709	-15.44
3	0.15	9.5	21.32	12.61	0.490	0.1064	0.30	10518.9	867.6	284.7	91.7	0.741	-11.08
4	0.25	10.2	21.90	16.94	0.505	0.0931	0.27	8906.3	713.8	415.7	112.1	0.695	-13.06
5	0.10	11.4	22.10	20.79	0.531	0.0794	0.25	6787.1	567.5	552.5	126.8	0.688	-3.51
6	0.15	13.1	21.84	23.53	0.571	0.0634	0.23	4684.1	428.4	682.2	136.5	0.688	-3.10
7	0.20	15.5	21.85	27.35	0.625	0.0526	0.23	2928.0	328.3	907.4	164.2	0.713	-2.00
8	0.15	18.7	22.16	31.20	0.692	0.0427	0.23	1717.9	243.2	1178.7	195.6	0.744	-0.37
9	0.20	17.8	22.40	35.10	0.672	0.0391	0.24	2004.0	277.2	821.8	168.6	0.714	-1.03
10	0.30	17.7	22.44	40.71	0.671	0.0399	0.26	2033.4	301.3	839.1	184.9	0.707	-1.54
													-55.27

Table A.4 Detailed procedures for estimating the swelling pressure and the heave of Keswick clay site (Case Study D)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	ψ_w (kPa)	P_{sw} (kPa)	e_w	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)		Eq. (3.6)	Eq. (4.4), (4.5)	Eq. (4.4), (4.5)
1	0.4	25.8	18.1	7.7	0.977	0.1032	0.68	1044.8	380.5	74.7	52.9	1.069	-18.68
2	0.4	26.4	18.1	15.3	0.977	0.1032	0.68	937.6	364.0	338.3	109.7	1.033	-11.37
3	0.4	26.3	18.1	22.9	0.977	0.1032	0.68	958.2	375.6	273.2	165.6	1.015	-7.77
4	0.4	25.4	18.0	30.4	0.977	0.1032	0.68	1119.9	417.3	321.5	193.8	1.013	-7.26
5	0.4	25.2	18.0	37.9	0.977	0.1032	0.68	1164.8	433.5	458.2	249.5	1.003	-5.23
6	0.4	25.2	18.0	45.3	0.977	0.1032	0.68	1163.9	440.5	655.7	322.4	0.991	-2.96
7	0.4	25.0	17.9	52.6	0.977	0.1032	0.68	1213.4	457.1	793.5	367.2	0.987	-2.08
8	0.4	25.2	18.0	59.9	0.977	0.1032	0.68	1166.0	455.0	971.8	416.7	0.981	-0.84
													-56.18

Table A.5 Detailed procedures for estimating the swelling pressure and the heave of Arlington clay site (Case Study E)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	ψ_w (kPa)	P_{sw} (kPa)	e_w	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)		Eq. (3.6)	Eq. (4.4), (4.5)	Eq. (4.4), (4.5)
1	0.30	20.7	20.3	21.3	0.347	0.0508	0.216	581.5	124.8	13.6	24.2	0.383	-8.01
2	0.31	20.7	20.3	27.4	0.363	0.0508	0.216	635.3	139.4	14.3	30.5	0.396	-7.62
3	0.31	20.7	20.3	33.7	0.439	0.0508	0.216	698.4	155.3	21.6	38.2	0.470	-6.66
4	0.30	20.7	20.3	39.9	0.551	0.0508	0.216	724.3	165.4	40.3	48.2	0.579	-5.22
5	0.31	20.2	20.4	46.0	0.480	0.0446	0.206	692.0	160.9	72.6	60.1	0.499	-3.99
6	0.30	19.7	20.4	52.4	0.496	0.0385	0.197	640.7	155.4	98.6	70.6	0.509	-2.63
7	0.31	19.7	20.4	58.5	0.472	0.0385	0.197	488.2	139.5	111.6	79.0	0.481	-2.00
8	0.31	19.7	20.4	64.8	0.472	0.0385	0.197	263.0	110.9	121.4	87.1	0.476	-0.85
9	0.29	19.7	20.4	71.0	0.472	0.0385	0.197	120.7	93.1	100.2	89.5	0.472	-0.13
													-37.11

Table A.6 Detailed procedures for estimating the swelling pressure and the heave of Al-Ghat shale site (Case Study F)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)	Eq. (4.2)
1	0.5	15.5	21.4	15.4	0.666	0.0662	0.193	5407.9	448.8	-29.1
2	0.5	17.6	22.9	26.4	0.762	0.0662	0.193	5539.5	467.1	-23.4
3	0.5	19.3	23.0	37.9	0.833	0.0662	0.193	5394.7	470.7	-19.8
4	0.5	19.7	23.0	49.4	0.847	0.0662	0.193	5210.5	471.9	-17.6
5	0.5	19.7	23.1	61.0	0.838	0.0662	0.193	4921.1	467.0	-15.9
6	0.5	20.0	23.0	72.5	0.844	0.0662	0.193	4750.0	468.6	-14.6
7	0.5	19.8	23.0	84.0	0.839	0.0662	0.193	4842.1	485.5	-13.7
8	0.5	19.7	23.2	95.5	0.832	0.0662	0.193	4776.3	493.2	-12.9
9	0.5	18.2	23.2	107.1	0.769	0.0662	0.193	4684.2	499.4	-12.5
										-159.5

Table A.7 Detailed procedures for estimating the swelling pressure and the heave of Zaoyang expansive soil site (Case Study G)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)	Eq. (4.2)
1	0.29	17.6	18.1	8.2	0.559	0.0557	0.288	728.6	156.3	-13.38
2	0.31	18.9	18.3	13.7	0.599	0.0557	0.288	725.7	161.4	-11.53
3	0.20	20.1	18.5	18.3	0.609	0.0557	0.288	318.1	85.5	-4.52
4	0.29	21.2	18.7	22.9	0.618	0.0557	0.288	39.0	32.6	-1.55
5	1.11	21.9	18.8	36.0	0.623	0.0557	0.288	16.2	40.3	-1.88
6	1.01	21.9	18.8	55.8	0.607	0.0557	0.288	3.8	57.1	-0.36
										-33.22

Table A.8 Detailed procedures for estimating the swelling pressure and the heave of Regina clay site (Case Study H)

Layer	Thickness (m)	w_i (%)	γ (kN/m^3)	P_f (kPa)	e_i	C_s	β_n	ψ_i (kPa)	P_{si} (kPa)	Δh (mm)
						Eq. (3.9)	Eq. (3.7)		Eq. (3.6)	Eq. (4.2)
1	0.3	29.0	16.2	7.5	1.077	0.0508	0.1815	2909.4	241.9	-11.09
2	0.5	29.5	16.2	13.9	1.114	0.0508	0.1815	2846.5	243.7	-8.96
3	0.4	27.5	17.1	21.4	0.979	0.0731	0.1881	2796.3	256.5	-15.94
4	0.4	31.5	16.3	28.1	1.113	0.0984	0.2008	2124.5	242.0	-17.43
5	0.5	35.0	16.1	35.4	1.191	0.1155	0.2114	1448.4	206.1	-20.18
6	0.5	35.5	15.9	43.4	1.261	0.1155	0.2107	1480.8	214.7	-17.74
7	0.4	36.0	15.8	50.5	1.288	0.0761	0.1933	1471.0	210.2	-8.24
8	0.5	36.0	16.4	57.8	1.204	0.0599	0.1830	1101.6	183.4	-5.45
										-105.05