uOttawa

L'Université canadienne
Canada's university

FACULTÉ DES ÉTUDES SUPÉRIEURES ET POSTOCTORALES

uOttawa

L'Université canadienne
Canada's university

FACULTY OF GRADUATE AND POSDOCTORAL STUDIES

Lu Lu
AUTEUR DE LA THÈSE / AUTHOR OF THESIS

M.A.Sc. (Civil Engineering)
GRADE / DEGREE

Department of Civil Engineering
FACULTÉ, ÉCOLE, DÉPARTEMENT / FACULTY, SCHOOL, DEPARTMENT

A Simple Technique for Estimating the 1-D Heave of Natural Expansive Soils

TITRE DE LA THÈSE / TITLE OF THESIS

Sai Vanapalli
DIRECTEUR (DIRECTRICE) DE LA THÈSE / THESIS SUPERVISOR

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

Paul Simms

Mamadou Fall

Ioan Nistor

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

This thesis is dedicated to my parents.

父母之爱是一首田园诗，幽远纯净，和雅清淡；

父母之爱是一幅山水画，洗去铅华雕饰，留下清新自然；

父母之爱是一首深情的歌，婉转悠扬，轻吟浅唱；

父母之爱是一阵和煦的风，吹去朔雪纷飞，带来春光无限。

父母之爱是一生相伴的盈盈笑语，

父母之爱是漂泊天涯时的缕缕思念。

A SIMPLE TECHNIQUE FOR ESTIMATING THE 1-D HEAVE OF NATURAL EXPANSIVE SOILS

by

Lu Lu

A Thesis
Submitted under the Supervision of
Dr. Sai K. Vanapalli

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

Department of Civil Engineering
University of Ottawa
Ottawa, Canada K1N 6N5

January 2010

Your file Votre référence
ISBN: 978-0-494-73831-3
Our file Notre référence
ISBN: 978-0-494-73831-3

ABSTRACT

Expansive soils are considered to be a nightmare to the geotechnical engineers. As the losses to the infrastructure constructed on expansive soils is estimated to be in billions of dollars world wide annually, several researchers and practitioners from various regions of the world have made significant contributions to better our present understanding of these problematic soils. One of the topics that attracted interest is related to the 1-D heave prediction or estimation methods for expansive soils. Early research studies on this topic were focused on developing empirical relationships which are not universally valid. Current techniques use soil suction methods and oedometer test methods to predict or estimate the 1-D heave; however, the various soil parameters required in these techniques can only be obtained from time consuming laboratory or in-situ tests that are expensive and difficult to be performed by conventional geotechnical engineers.

In the present study, a simple technique is proposed to estimate the 1-D heave in expansive soils which requires only the information of plasticity index, I_p and variation in water content with respect to depth in the active zone. This technique is developed using the results of 5 case studies published in the literature. In addition to these case studies, 8 other case studies results of 1-D heave in expansive soils are summarized. The data of 13 of the case studies is collected from various regions of the world which include Australia, Canada, Chile, Saudi Arabia, Sudan, United States and Yugoslavia. Comparisons are provided between the measured and estimated 1-D heave for all the case studies have been using the proposed simple technique. There is a reasonably good comparison between the measured heave and the estimated heave for 8 of the 13 case studies results (i.e., less than 30%). The estimated heave is more than the measured heave for all the case studies; which is conservative from an engineering practice perspective. The reasons associated with the discrepancies between the measured and predicted 1-D heave values for the remainder of the five case studies are

also discussed.

The proposed simple technique is encouraging for the practicing geotechnical engineers in the estimation of 1-D heave in expansive soils.

ACKNOWLEDGEMENTS

The author wishes to express her sincere appreciation to her supervisor, who is also a philosopher, Dr. Sai K. Vanapalli. She is grateful to his guidance, encouragement, invaluable advice and suggestions throughout this research program.

Special thanks are due to her colleague Won Taek Oh. He is not only a colleague, but also a respected friend. Without his assistance, her task would have been much more difficult.

The author expresses her thanks to her colleagues Fathi Mohamed, Nil Taylan, Rui Sun and the other graduate students in geotechnical engineering program at the University of Ottawa for their assistance.

Also, thanks are due to the faculty and staff in Civil Engineering Department for providing support to her graduate study and thesis experiment.

Above all, the author is thankful to Guanyin Bodhisattva for providing this rare opportunity and experience.

This thesis is dedicated to my parents.

父母之爱是一首田园诗，幽远纯净，和雅清淡；

父母之爱是一幅山水画，洗去铅华雕饰，留下清新自然；

父母之爱是一首深情的歌，婉转悠扬，轻吟浅唱；

父母之爱是一阵和煦的风，吹去朔雪纷飞，带来春光无限。

父母之爱是一生相伴的盈盈笑语，

父母之爱是漂泊天涯时的缕缕思念。

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

List of Symbols

Symbol	Term	Unit
A	Area	m^2
α	Compressibility index	-
a_t	Coefficient of compressibility with respect to a change in net normal stress	-
a_m	Coefficient of compressibility with respect to a change in matric suction	-
b_t	Coefficient of water content change with respect to a change in net normal stress	-
b_m	Coefficient of water content with respect to a change in matric suction	-
c	Clay content	%
C_c	Compression index	-
C_s	Corrected swelling index	-
C_w	Suction modulus ratio	-
C_ρ	Heave index	-
C_H	Heave index	-
C_τ	Suction index	-
C_ψ	Suction index	-
D	Thickness of nonexpansive layer	m
D_h	Suction index with respect to moisture content	-
E	Modulus of linear deformation	-
E_w	Water volumetric modulus	-
e	Void ratio	-
e_0	Initial void ratio	-
e_f	Final void ratio	-
F	Correction factor for degree of expansiveness	-
f_i	Factor to include the effects of the lateral confinement	-
G_s	Specific gravity	-
H	Thickness of the soil layer	m
H_i	thickness of the i_{th} layer	m
ΔH	Total heave	mm
H_w	Water volumetric modulus	-
h_i	Initial matric suction	kPa
h_f	Final matric suction	kPa
h_0	Matric suction without surcharge pressure	kPa
I_P	Plasticity index	%
I_s	Shrinkage index	%

I_{pt}	Instability index of the soil	-
I_v	Volume index	-
K	Correction parameter	-
L	Length	m
ln	Natural logarithm	-
log	Logarithm base 10	-
M	Mass	g
m_1'	Coefficient of compressibility with respect to net normal stress	-
m_2'	Coefficient of compressibility with respect to matric suction	-
P_f	Final stress state	kPa
P_i	Initial stress state	kPa
P'_s	Corrected swelling pressure	kPa
SP	Swelling potential	%
S_f	Final assumed degree of saturation	%
S_i	Initial degree of saturation	%
ΔS	Change in degree of saturation	%
T	Temperature	°C
t	time	h
u	Pore pressure	kPa
Δu	Change in suction	kPa
u_a	Pore-air pressure	kPa
u_w	Pore-water pressure	kPa
u_{wf}	Final pore-water pressure	kPa
V	Volume	m^3
V_i	Initial volume	m^3
ΔV	Volume change	m^3
W_L	Liquid limit	%
W_P	Plastic limit	%
W_S	Shrinkage limit	%
w	Water content	%
w_i	Initial water content	%
w_f	Final water content	%
w_{0i}	Average initial water content	%
w_{0f}	Average final water content	%
Δw	Water content change	%
z	Depth	m
ρ	Density	kg/m^3
ρ_d	Dry density	kg/m^3
ρ_w	Density of water	kg/m^3
γ_d	Dry unit weight	kN/m^3
γ_w	Unit weight of water	kN/m^3

γ_h	Suction compression index	kPa
σ	Total normal stress	kPa
σ'	Effective normal stress	kPa
σ'_{cv}	Swelling pressure from constant volume swell test	kPa
σ'_f	vertical stress at the midpoint of the soil layer	kPa
σ_v	vertical stress at the midpoint of the soil layer	kPa
$\sigma_1\ \sigma_2\ \sigma_3$	Principal stresses (major, intermediate and minor)	kPa
σ_{oct}	Average stress or octahedral normal stress	kPa
σ_{mean}	Average total normal stress	kPa
σ_y	Total overburden pressure	kPa
$\Delta\sigma_y$	Change in total stress	kPa
τ	Shear stress	kPa
τ_{oct}	Octahedral shear stress	kPa
τ_{mf}	Final matric suction	kPa
ε	Linear strain	-
ε_v	Volumetric strain	-
ψ_i	Initial suction	kPa
ψ_f	Final suction	kPa

CHAPTER 1

INTRODUCTION

1.1 General Information

Several parameters such as clay mineralogy, stress history, temperature, humidity and environmental conditions (i.e., wet-dry and freeze-thaw cycles) influence the engineering behavior of natural expansive soils. The natural water content of the expansive soils is dependent on all of the above parameters including the soil suction. One of the key problems associated with expansive soils is the swell pressures which contribute to the heave movements resulting in the lifting of the structures in both lateral and vertical directions due to the changes in water content (Nelson and Miller 1992, Fredlund and Rahardjo 1993).

Expansive soils are typically found in a state of unsaturated condition. The soil suction varies from a value of zero when it is in a state of saturated condition to 1,000,000 kPa when it is dry. This behavior is observed irrespective of the type of soil (Vanapalli et al. 1999). Typically, all coarse-grained soils, non-plastic soils and many fine-grained soils with low plasticity exhibit insignificant changes with respect to volume change behavior over the entire suction range of 0 to 10^6 kPa. However, the volume change behavior along with the flow and shear strength behavior of the expansive soils are significantly influenced over the entire suction range of the expansive soils. Due to this reason, expansive soils are considered to be a nightmare to the geotechnical engineers. The losses associated with expansive soils in the U.S.A are far greater than other natural hazards such as the earthquakes and floods (Jones and Holtz 1973).

Many regions of the world have extensive deposits of expansive soils; they are particularly wide spread in semi-arid and arid regions of the world where the evapo-transpiration typically exceeds the precipitation (Dregne 1976; Fredlund and

Rahardjo 1993). Some of the countries which have these deposits are Argentina, Australia, Burma, Canada, China, Cuba, Ethiopia, Ghana, India, Israel, Iran, Mexico, Morocco, Rhodesia, South Africa, Spain, Turkey, United States, and Venezuela (Chen 1975). These soils cause damages to structures, pavements, pipelines, slopes, irrigation channels and other constructions and contribute to significant financial losses. The properties of expansive soils are continuously changing with changes in water content and suction and hence it is a challenge to predict their behavior.

Expansive soils along with collapsible, residual, and compacted soils are considered as problematic soils as conventional soil mechanics cannot be applied for these soils. The geotechnical engineers in several situations prefer to avoid problematic soils; particularly expansive soils. However, with the population growth and with many regions of the world having vast deposits, geotechnical engineers have little choice but to deal with these soils.

Early research studies during the period of 1950's and early 1960's were based on limited studies that focused on developing empirical relationships (Seed et al. 1962, Chen 1975). Such empirical relationships were not found to be useful for explaining the expansive soils behavior of other regions (Brackley 1975, Burland 1984). Two symposiums of this period (South Africa, 1957; Colorado, 1959) and several series of specialized conferences which includes seven international conferences on expansive soils (Texas A&M, College Station, Texas, 1965; Texas A&M, College Station, Texas, 1969; Haifa, Israel, 1973; Denver, Colorado, 1980; Adelaide, Australia, 1984; New Delhi, India, 1987; Dallas, Texas, 1992) have significantly contributed to better understand the expansive soils behavior.

Several research studies have shown that unsaturated expansive soils can be better interpreted in terms of stress state variables (Fredlund and Morgenstern 1977, Alonso et al. 1990, Fredlund and Rahardjo 1993). The focus of research in the area of expansive soils has been directed by most researchers in the world extending the mechanics of unsaturated soils since the First Conference on Unsaturated Soils, which was held in Paris in 1995. Since then, four more international conferences on unsaturated soils held in Beijing, China (1998), Recife, Barzil (2002), Carefree, USA (2006) have also significantly contributed to understand various aspects of expansive soils.

The design of foundations in expansive soils is one of the greatest challenges facing geotechnical engineers (Mitchell 1984, Nelson and Miller 1992). This is because the expansive soils typically have high swell pressures and contribute to intolerable heave of foundations and often affects critical safety aspects. Therefore, the design of foundations on expansive soils should include analyses of expected heave and consequences of foundation movement over the design life of the structure (Nelson et al., 2003). Practicing engineers rely on using a variety of design, construction, and stabilization techniques to reduce losses associated with expansive soils. However, the success of the techniques or procedures will mainly depend on key information such as the reliable determination or estimation of the swelling pressure and the soil heave. The most valuable information available in this research area include about case study results from Australia, Canada, Saudi Arabia, Sudan and the USA, which summarize the 1-D heave and swelling pressure of natural expansive soils (Fredlund 1969, Yoshida et al. 1983, Beal 1984, Mitchell and Avalle 1984, Hamberg and Nelson 1984, Ching and Fredlund 1984, Cameron and Walsh 1984, Clifton et al. 1984, Osman and Sharief 1987, Dhowian et al. 1987, Maksimovic and Tonkovic 1987, Retamal et al. 1987, Cameron 1989, Snethen and Huang 1992, Nelson and Miller 1992, Li et al. 1995, Fityus and Smith1998, Vu and Fredlund 2004, Delaney et al. 2005). Several millions of research dollars may have been invested into these studies to develop reliable techniques for estimation of 1-D heave of expansive soils.

1.2 Problem Statement

Estimation of the potential heave of expansive soils is required for taking appropriate design and construction measures to minimize the anticipated damages to the structures. Currently, the techniques commonly used for estimating or predicting the 1-D heave can be divided into three categories: (i) empirical methods; (ii) soil suction methods and (iii) oedometer test methods. However, there are limitations to use these techniques: i) these techniques are not universally valid as they are proposed using only limited soils data collected locally; ii) they do not use the stress state variables approach that provides a rational basis for interpretation; iii) the various soil parameters required in these techniques can only be obtained from time consuming laboratory or in-situ tests that are expensive and difficult to be performed by conventional geotechnical engineers.

1.3 Objective of Present Research

The key objective of the research presented in this thesis is to provide a simple technique to estimate the swelling pressure and the 1-D heave of expansive soils, which requires only the information of plasticity index, I_p and the variation in natural water content with respect to depth in the active zone of natural expansive soils. An attempt is made in this thesis to reanalyze the data of the valuable case studies reported in the literature to propose a consistent but simple framework that alleviates the limitations of the presently used techniques in the estimation of swelling pressure and 1-D heave.

1.4 Thesis Organization

This thesis is organized in six separate chapters. This chapter, Introduction provides details of the research problem addressed in this thesis. Chapter 2 provides a general background and reviews the various factors that influence the swelling behaviour of expansive soils. Several techniques available in the literature for estimating the swelling pressure and the heave of expansive soils are discussed in the next Chapter 3 along with their limitations. Chapter 4 summarizes the details of a simple method proposed for estimating 1-D heave of natural expansive soils. The proposed technique was tested on 13 case studies results published in the literature and summarized as Chapter 5. This Chapter also provides comparisons between measured heave and estimated heave by using this method. Chapter 6 summarises the conclusions of the research presented from this research program and provides suggestions for future research studies.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction

Expansive soils contribute to significant damages to structures constructed on, in or with and contribute to significant financial losses. More effective and economical design of structures would be possible if the swelling pressure and ground heave of expansive soil can be reliably estimated or predicted. These soils are typically in a state of unsaturated condition and heave as the degree of saturation increases due to an increase in the water content. Several techniques available in the literature for estimating the swelling pressure and the heave of expansive soils are discussed in the next Chapter 3 along with their limitations.

This chapter provides a general background and reviews the various factors that influence the swelling behaviour of expansive soils. In addition, some key concepts related to unsaturated soils mechanics are presented. The background information summarized in this Chapter forms as foundation to various other details presented and discussed in later chapters of the thesis.

2.2 General Background

The solid phase of soils in both saturated and unsaturated soils typically consists of various amounts of crystalline clay and non-clay minerals, non-crystalline clay material, organic matter, and precipitated salts (Das 2008). The minerals are commonly formed by atoms of elements such as oxygen, silicon, hydrogen, and aluminum, organized in various crystalline forms. These elements combine with other elements such as calcium, sodium, potassium, magnesium, and carbon and comprise over 99% of the solid mass of soils (Spangler and Handy 1982).

The solid phase consists of some or different fractions of boulders, gravel, sand, silt and clay particles. It is well known that despite its size (i.e., less than 0.002 mm) the clay fraction has an important role in controlling the engineering behaviour of a soil (Mitchell 1993). In other words, coefficient of permeability, shear strength and volume change behaviour of soils are significantly influenced by the clay fraction.

The structure of a soil is the combined effect of fabric, composition, and interparticle forces in both saturated and unsaturated soils including expansive soils. The structure of soils is also used to account for differences between the properties of natural (structured) and remolded (destructured) soils (Leroueil and Vaughan, 1990).

Unsaturated soils can be either naturally occurring soils which are commonly found in arid and semi-arid regions or artificially created (man-made) soils such as those formed by compaction. Collapsible soils, residual soils, lateritic soils, weakly cemented soils and expansive soils are typical examples of natural unsaturated soils. The unsaturated soils contain both air and liquid phases in the soil pores and their engineering behaviour is significantly different from conventional saturated soils.

2.2.1 Various Phases in Unsaturated Soils

A soil that is in a state of unsaturated condition consists of four different phases, namely; solid phase, water phase, air phase and contractile skin (Fredlund and Morgenstern 1977) (Figure 2.1). The properties of a soil depend mainly on the properties of its solid and water phases and the interaction between them.

Fredlund and Morgenstern (1977) extended the concept of contractile skin as a tool to explain the unsaturated soil behaviour. The contractile skin acts like a thin membrane interwoven throughout the voids of the soil, acting as a partition between the air and water phases. It is the interaction of the contractile skin with the soil structure that causes an unsaturated soil to change its mechanical behaviour (i.e., volume and shear strength). In addition, the unsaturated soil properties also change in response to the position of the contractile skin (i.e., degree of saturation). In summary, it can be stated that an unsaturated soil has two phases that flow under the influence of a stress gradient (i.e., air and water) and two phases that come to equilibrium under the influence of a stress gradient (i.e., soil particles forming a structural arrangement and the contractile skin forming a partition between the fluid phases) (Fredlund and Rahardjo 1993).

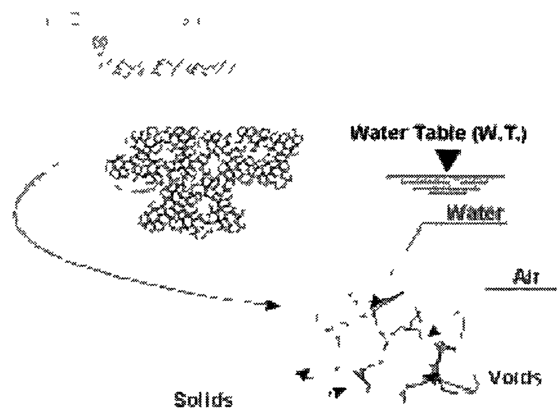


Figure 2.1 A phase diagram of soil indicating the weights and volumes of air, soil, water, and voids (from http://en.wikipedia.org/wiki/Soil_mechanics).

The contractile skin can be considered as part of the water phase with regard to changes in volume-mass soil properties; and it must be considered as an independent phase when describing the stress state and phenomenological behaviour of an unsaturated soil (Fredlund and Rahardjo 1993). Numerous research studies on the nature of the contractile skin point toward its independent role in the rational interpretation of the mechanics of unsaturated soils. Lyklema (2000) showed that the distribution of water molecules across the contractile skin takes the form of a hyperbolic tangent function as shown in Figure 2.2.

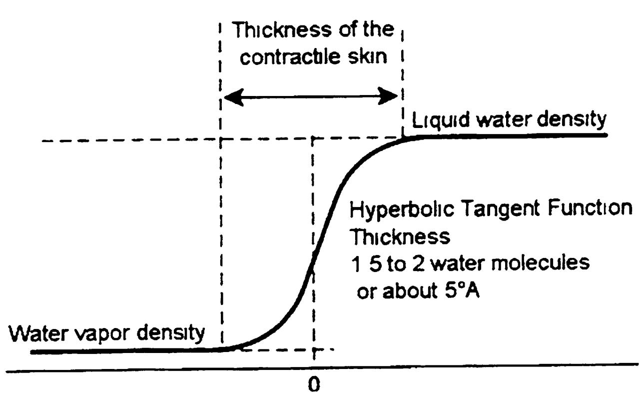


Figure 2.2 Density distribution across the contractile skin (from Lyklema 2000).

2.2.2 Stress state variables

Terzaghi (1936) explained the effective stress principle valid for saturated soils as "All the measurable effects of a change of stress, such as compression, distortion and a change in the shearing resistance are exclusively due to changes in effective stress. Every investigation of the stability of a saturated body of earth requires the knowledge of both the total and the neutral stresses"

Terzaghi's effective stress equation satisfies the conceptual framework of continuum mechanics and has proven to be useful in the interpretation of the engineering behaviour of different soils such as clays, silts, sands and gravels that are in a state of saturated condition. The effective stress state of a saturated soil can be expressed in a matrix form as given below:

$$|\sigma'| = \begin{vmatrix} (\sigma_x - u_w) & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & (\sigma_y - u_w) & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & (\sigma_z - u_w) \end{vmatrix} \tag{2.1}$$

The success in implementing the Terzaghi's effective stress principle for saturated soils in conventional engineering practice has prompted numerous investigators to extend the same to unsaturated soils Table 2.1 summarizes some of the key equations proposed in the literature for interpretation of unsaturated soil behaviour in terms of single stress state variable.

Table 2.1 Proposed effective stress equations for unsaturated soils (one stress state variable) (modified after Vanapalli 1994)

Equation	Author	Parameters	Eqn.
$\sigma' = \sigma + p''$	Donald, 1956	p'' : pressure deficiency of the pore water	(2.2)
$\sigma' = \sigma - (u_a + u_c)$	Hilf, 1956	u_c : capillary pressure	(2.3)
$\sigma' = \sigma - \beta' u_w$	Croney et al., 1958	β' : holding or bonding factor dependent on degree of saturation and is a measure of the numbers of bonds under tension effective in contributing to the shear strength σ : Total normal stress	(2.4)

$\sigma' = (\sigma - u_a) + \chi(u_a - u_w)$	Bishop, 1959	χ : parameter related to the degree of soil saturation	(2.5)
$\sigma' = \sigma - \varphi p''$	Aitchison, 1960	φ : parameter varying between zero to one depending on degree of saturation p'' : pore-water pressure deficiency	(2.6)
$\sigma' = \sigma - \beta p''$	Jennings, 1961	β : a statistical factor of the same type as the contact area p'' : negative pore-water pressure deficiency	(2.7)
$\sigma' = \sigma_i - (u_a - u_w) + \chi(\sigma - u_a)$	Newland, 1965	σ_i : intrinsic stress arising from interparticle forces χ : a parameter related to the degree of soil saturation	(2.8)
$\sigma' = \sigma - u_a + X_m(h_m + u_a)$ $+X_s(h_s + u_a)$	Richards, 1966	h_m : matric suction h_s : solute suction X_m : effective stress parameter for matric suction X_s :effective stress parameter for solute suction	(2.9)
$\sigma' = (\sigma - u_a) + a_w(u_a - u_w)$	Lambe & Whitman, 1979	a_w : ratio of area of water-mineral and water-water contact of total area of "wavy" plane	(2.10)
$\sigma' = \sigma + \chi_m p''_m + \chi_s p''_s$	Aitchison, 1973	p''_m : matric suction p''_s : solute suction	(2.11)
$\sigma' = (\sigma - u_a) + \chi(u_a - u_w) - R - \varsigma T_s$	Allam & Sridharan, 1987	χ :a parameter representing the proportion of the total void area occupied b the water on a reference plane R : osmotic suction ς : interface parameter on the reference plane	(2.12)
$\sigma' \left[\frac{(u_a-u_w)_k}{(u_a-u_w)_m}\right]^{-0.55}$ $= (\sigma - u_a) + (u_a - u_w)$	Khalili & Khabbaz, 1998	$(u_a - u_w)_f$: matric suction in the specimen at failure condition $(u_a - u_w)_m$: air entry value	(2.13)

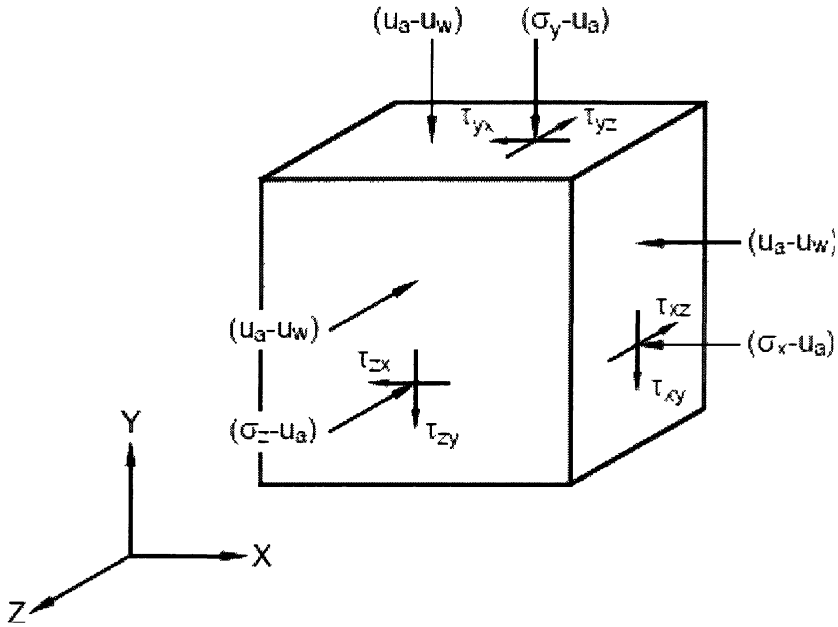


Figure 2. 3 Triaxial loading conditions on a cubical soil element of infinitesimal dimensions (Fredlund and Rahardjo, 1993).

The equation proposed by Bishop (1959) (i.e., Eqn. 2.5) is more commonly used due to its simplicity in practical applications. Several investigators extended this equation and used it in proposing constitutive relationships for interpreting unsaturated soils behavior. However, some investigators questioned the validity of the proposed single stress-state equation given by Bishop (1959) as it does not provide reliable results for the interpretation of the mechanical behaviour of unsaturated soils (shear strength and volume change properties) and suggested that the mechanical behaviour should be independently related to the stress state variables $(\sigma_n - u_a)$ and $(u_a - u_w)$ (Jennings 1961, Burland 1964, Blight 1967).

In 1977, Fredlund and Morgenstern proposed a framework for unsaturated soils showing that any two of the following three stress state variables can be used to define the stress state of an unsaturated soil: net normal stress, $(\sigma_n - u_a)$ and matric suction, $(u_a - u_w)$, $(\sigma_n - u_w)$ and $(u_a - u_w)$, $(\sigma_n - u_a)$ and $(\sigma_n - u_w)$. The stress state variables more often used in the interpretation of unsaturated soils behaviour are $(\sigma_n - u_a)$ and matric suction are $(u_a - u_w)$. Extending continuum mechanics principles, these stress state variables can be represented in matrix form as follows (Fredlund 1979b):

$$\begin{vmatrix} (\sigma_x - u_a) & \tau_{yx} & \tau_{zx} \\ \tau_{xy} & (\sigma_y - u_a) & \tau_{zy} \\ \tau_{xz} & \tau_{yz} & (\sigma_z - u_a) \end{vmatrix} \tag{2.2}$$

$$\begin{vmatrix} (u_a - u_w) & 0 & 0 \\ 0 & (u_a - u_w) & 0 \\ 0 & 0 & (u_a - u_w) \end{vmatrix} \tag{2.3}$$

Fredlund et al. (1978), suggested that the stress state variables; net normal stress, $(\sigma_n - u_a)$ and matric suction, $(u_a - u_w)$ combination was more appropriate because only one stress state variable is affected when the pore water pressure changes. The effects of externally applied loads and the effects of environmental changes can readily be separated in terms of stress changes using this combination. In addition, because the pore-air pressure is typically atmospheric, the stress state of net normal stress is simply the normal stress (Fredlund and Rahardjo 1993)

2.3 Expansive Soils

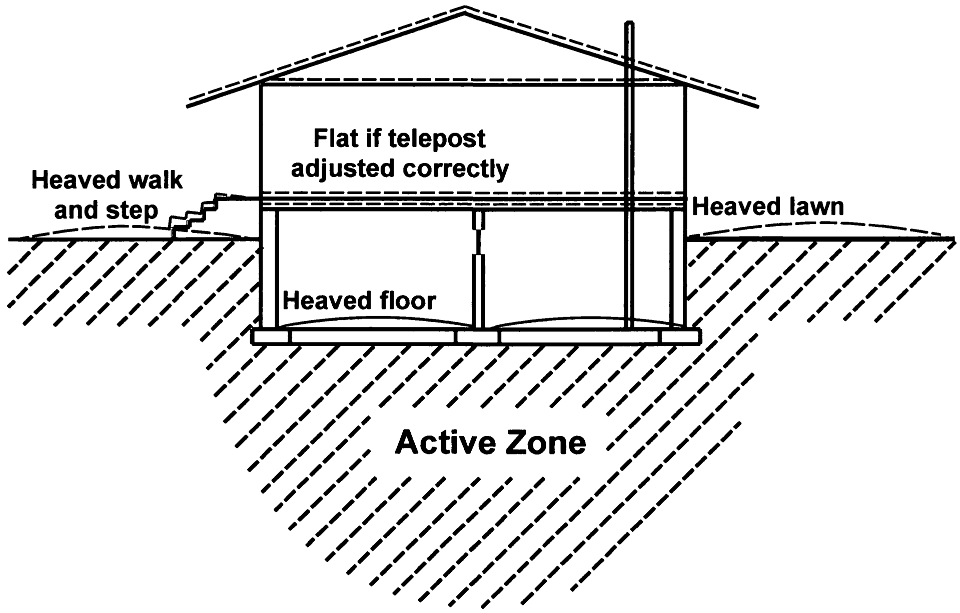
The interest in studying the expansive soils began in the late 1930's when swelling problem was noticed by the American Office of Land Reforms in 1938. It was soon realized that expansive soils are commonly found in many arid and semi-arid areas of the world such as Argentina, Australia, Burma, Canada, China, Cuba, Ethiopia, Ghana, India, Israel, Iran, Mexico, Morocco, Rhodesia, South Africa, Spain, Turkey, United States, and Venezuela (Chen 1975). In Canada, expansive soils are found in Western provinces which include Saskatchewan, Alberta and Manitoba. In many of these regions, evapo-transpiration exceeds the precipitation and the expansive soils are typically found in a state of unsaturated condition. These soils swell or shrink with changes in their natural water content.

2.3.1 Foundation damage

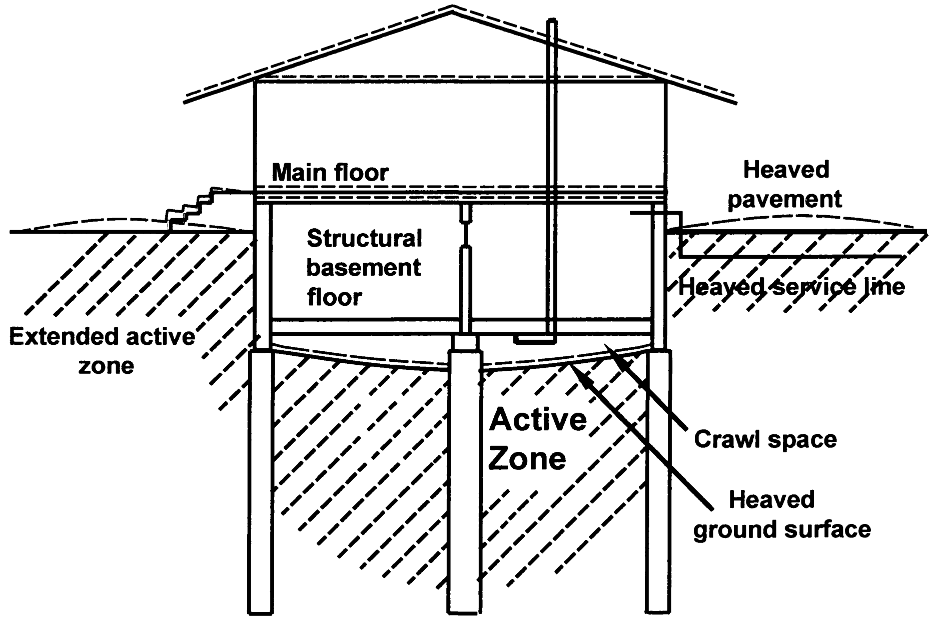
Expansive soils can cause more damage to structures such as light buildings, pavements and pipelines to list a few. Structures built on expansive soils tend to undergo moderate to severe cracking problems due to swelling and shrinkage behaviour (Nelson and Miller 1992). Because of the building loads on different portions of a structure's foundation significantly vary; the resultant uplift pressure will also be different. For example, the exterior corners of a uniformly-loaded rectangular slab foundation (Figure 2.4a) will only exert about one-fourth of the normal pressure on a swelling soil of that exerted at the central portion of the slab (Rogers. et al., 1993). As a result, the corners tend to be lifted up relative to the central portion. This phenomenon will also be influenced due to the differences in water content variation within the soil at the edge of the slab. Such differential movements of the foundation can cause significant distress to the framing of a structure.

Drilled pier foundations (Figure 2.4b) can also be adversely affected by expansive soil behaviour if the piers are not sufficiently deep (Rogers. et al., 1993). Frequently, the corner piers of a pier-supported structure are lifted up during swelling in the wet season, and then break their skin friction bond with the ground when the soil shrinks away from the pier in the following dry season. Loss of this "skin friction" decreases the pier's ability to support building loads. The pier functionality can likely fail if the soil strain is significantly large.

Shallow pipes, especially plastic pipes, buried in the zone of seasonal moisture fluctuation, are exposed to enormous stresses by shrinking soils. If water or sewage pipes break, then the resultant leaking moisture can aggravate swelling damage to nearby structures (CFEM 2007). Concrete drainage devices can also be adversely affected by expansive soils as well. Swelling clays can lift and crack concrete ditches, seriously impairing their ability to convey runoff. Subsequent contraction may leave a void under the concrete, leading to piping and erosion as runoff flows under the ditch.



(a)



(b)

Figure 2.4 (a) Ground movements associated with the construction of shallow footings on an expansive soil. (b) Ground movements associated with the construction of a house founded on piles in expansive soils (Hamilton 1977).

2.3.2 Swell potential

The swell potential mainly depends on the properties of its solid and water phases and the interaction between them. Many of the factors influencing the mechanism of swelling also affect, or are affected by the physical soil properties such as the plasticity or density (Nelson and Miller 1992). The factors influencing the swelling potential can be categorized in three different groups; namely, (i) the stress state; (ii) the soil characteristics that influence the basic nature of the internal force field; and (iii) the environmental factors that influence the changes that may occur in the internal force system.

The tables below summarize the various soil properties and environmental factors that influence the swell potential.

2.3.3 Identification

Many researchers have made significant contributions towards identifying the expansive soils and the various factors that influence the expansive soil behaviour and the total heave. These indicators are based on measurements of the Atterberg Limits (liquid limit, plastic limit, and plasticity index), particle size distribution, percent swell, swell pressure, and coefficient of rebound obtained in the swell-consolidation test. Table 2.4 provides a summary of the laboratory tests used in identification of expansive soils.

Table 2.2 Soil properties influencing swell potential (Nelson and Miller 1992)

Factor	Description	References
Clay mineralogy	Clay minerals which typically cause soil volume changes are montmorillonites, vermiculites, and some mixed layer minerals. Illites and Kaolinites are infrequently expansive, but can cause volume changes when particle sizes are extremely fine (less than a few microns).	Grim, 1968; Mitchell, 1973, 1976; Snethen et al., 1977
Soil water chemistry	Swelling is repressed by increased cation concentration and increased cation valence. For example, Mg^{2+} cations in the soil water would result in less swelling than Na^{+} cations.	Mitchell, 1976
Soil suction	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is related to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water	Snethen, 1980; Fredlund & Morgenstern, 1977; Johnson, 1973; Olsen & Langfelder, 1965; Aitchison et al., 1965
Plasticity	In general, soils that exhibit plastic behaviour over wide ranges of moisture content and that have high liquid limits have greater potential or swelling and shrinking. Plasticity is an indicator of swell potential.	--
Soil structure and fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at higher water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water potential.	Johnson & Snethen, 1978; Seed et al., 1962
Dry density	Higher densities usually indicate closer particle spacings, which may mean greater repulsive forces between particles and larger swelling potential.	Chen, 1975; Komornik & David, 1969; Uppal, 1965

Table 2.3 Environmental factors (Nelson and Miller 1992)

Factor	Description	References
Initial moisture condition	A desiccated expansive soil will have a higher affinity for water, or higher suction, than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to dying influences, and shrink more be considered in conjunction with the expected range of final suction conditions.	--
Moisture variations	Changes in moisture in the active zone near the upper part of the profile primarily define heave. It is in those layers that widest variation in moisture and volume change will occur.	Johnson, 1969
Climate	Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have pronounced, short wet period.	Holland & Lawrence, 1980
Groundwater	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture.	--
Drainage and manmade water sources	Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of moisture and fluctuating water tables contribute to moisture.	Krazynski, 1980; Donaldson, 1965
Vegetation	Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in area of varying vegetation.	Burkley, 1974
Permeability	Soils with higher permeability, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell.	Wise & Hudson, 1971; De Brujin, 1965
Temperature	Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings.	Johnson & Stroman, 1976; Hamilton, 1969
Stress history	An over consolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting-drying cycles, swell is unaffected.	Mitchell, 1976; Kassiff & Baker, 1971
In-situ condition	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in-situ measurements and observations.	--
Loading	Magnitude of surcharge load determines the amount of volume change that will occur for given moisture content and density. An externally applied load acts to balance inter particle repulsive forces and reduces swell.	Holtz, 1959
Soil profile	The thickness and location of potential expansive layers in the profile considerably influence potential movement. Greatest movement will occur in profiles that have expansive clays extending from the surface to depth below the active zone. Less movement will occur if expansive soil is overlain by non expansive material or overlies bedrock at a shallow depth.	Holland & Lawrence, 1980

Table 2.4 Laboratory tests used in identification of expansive soils (Nelson and Miller 1992)

Test	Reference	Properties Investigated	Parameters Determined
Atterberg limits	ASTM Standards (1991)	Plasticity, consistency	Plasticity index, I_p
Liquid limit	ASTM D-4308	Upper limit water content of plasticity	Liquidity index, L_I
Plastic limit	ASTM D-4318	Lower limit water content of plasticity	Shrinkage ratio, R
Shrinkage limit	ASTM D-427	Lower limit water content of soil shrinkage	Linear shrinkage, L_S
Clay content	ASTM D-422	Distribution of fine-grained particles sizes	Percent finer than 2 μm
Mineralogical tests	Whitting (1964)	Mineralogy of clay particles	
X-ray diffraction	ASTM STP 479 (1970)	Characteristic crystal dimensions	Basal spacings
Differential thermal analysis	Barshad (1965)	Characteristic reactions to heat treatments	Area and amplitude of reaction peaks on thermo grams
Electron microscopy	McCrone & Delly (1973)	Size and shape of clay particles	Visual record of particles
Cation-exchange capacity	Chapman (1965)	Charge deficiency and surface activity of clay particles	CEC (meq/100g)
Free swell test	Holtz & Gibbs (1956)	Swell upon wetting of unconsolidated unconfined sample of air dried soil	Free swell= $(V_{wet}-V_{dry})/V_{dry}*100\%$
Potential volume change meter	Lambe (1960b)	One-dimensional swell and pressure of compacted, remolded sample under semi-strain controlled conditions	SI (swell index)(lb/ft^2) PVC(potential volume change)
Expansion index text	Uniform Building Code	One-dimensional swell under 1psi (6.9 kPa), surcharge of sample compacted to 50% saturation initially	Expansion index (EI)
California bearing ratio test	Yoder & Witczak (1975) Kassiff et al. (1969)	One-dimensional swell under surcharge pressure of compacted, remolded samples on partial wetting	Percent swell CBR (%)
Coefficient of linear extensibility (COLE) test	Brasher et al. (1966)	Linear strain of a natural soil clod when dried from 5psi (33 kPa) to oven dry suction	COLE and LE (%)

Table 2.5 summarizes guidelines that can be used for roughly estimating the probable volume changes of expansive soils; based on soil classification properties and other conventional tests well known to the geotechnical engineers.

Table 2.5 Data for estimating of probable volume changes for expansive soils (Chen 1975)

Laboratory and field data			Probable expansion percent total Volume change	Swelling pressure (ksf)	Degree of expansion
Percentage passing No.200 sieve	Liquid limit (%)	Standard penetration resistance (blows/ft)			
>35	>60	>30	>10	>20	Very high
60-95	40-60	20-30	3-10	5-20	High
30-60	30-40	10-20	1-5	3-5	Medium
<30	<30	<30	<1	1	Low

2.3.4 Heave prediction modelling

The knowledge of several properties of expansive soils is necessary to reliably predict the heave behaviour. There are several techniques available in the literature for predicting the heave in expansive unsaturated soils which will be discussed in Chapter 3. Several constitutive models also have been proposed for the volume change behaviour of unsaturated soils that can be used to interpret or model the heave behaviour of expansive soils (Matyas and Radhakrishna 1968, Fredlund and Morgenstern 1977, Alonso et al. 1990, Sheng et al., 2007). Alonso (1998) has undertaken a comprehensive review of earlier studies in literature and summarized several key features of expansive soils that have to be considered when developing a constitutive model for behavioural prediction purposes.

(i) Behaviour under simple paths

For a given soil, dry density, and confining stress, the final swelling upon wetting depends on the initial water content (or the initial suction) of the soil. The higher is the initial water content of the specimen; the lower will be the swelling (Holtz and Gibbs, 1956).

For a given soil, dry density, and initial water content (or initial suction), the swelling strain is influenced by the stress level acting on the soil. The higher is the stress level; the lower will be the swelling strain upon wetting (Holtz and Gibbs, 1956).

For a given soil, confining stress, and initial water content (or initial suction), the swelling strain upon wetting depends on the initial dry density of the soil. The higher is the initial dry density of the soil; the lower will be the swelling strain upon wetting (Holtz and Gibbs, 1956).

Final swelling pressure depends on the initial dry density of the soil without pronounced effects of initial water content (Holtz and Gibbs, 1956; Sridharan et al., 1986; and Mesri et al., 1994). At high initial water content, swelling pressure tends to be equal to the initial suction of the soil which is in acceptance with the effective stress concept.

During progressive wetting process, swelling pressure may reach an absolute maximum value at an intermediate water content (or suction) and may decay to reach a residual value. This behaviour has been observed by many researchers such as Alonso et al. (2001), Romero et al. (2003), and Lloret et al. (2003).

(ii) Behaviour under complex paths involving changes in suction and mechanical loading

At low stress level, wetting-drying cycles induce an accumulation of expansion (or swelling) (Alonso et al. 2001). At high stress level, there is an accumulation of contraction (or shrinkage) of the soil after cyclic wetting and drying. The experimental evidence shown in Dif and Bluemel (1991) and Alonso et al. (2001) is a proof of this behaviour for expansive soils.

(iii) Long-term and osmotic effects

The long-term effects as addressed in Alonso (1998) are related to primary and secondary behaviour of expansive soils such as compression under saturated and unsaturated state, swelling pressure development. The experimental proof of this behaviour is available from Komornik and Zeitlen (1979). The secondary effects have also been addressed and modeled by Navarro and Alonso (2001) as a local dehydration process. The osmotic effects are mainly related to the effects of pore solute concentration on the mechanical behaviour of expansive soils. The consequence of this consideration is that whether osmotic suction has to be considered separately as stress state variable in the modeling of unsaturated behaviour of expansive soils. Barbour and Fredlund (1989) showed that osmotic suction can be considered as one of the stress state variables for unsaturated expansive soils.

Chen (1975) studied both the properties of expansive soils and the features heave prediction modelling and summarized the key factors that should be used in 1-D heave prediction techniques without considering mineralogical composition

(i) Swelling pressure

In a conventional laboratory consolidation test, after the sample has been saturated and swelled to its maximum extent, the specimen is typically loaded gradually until it returns to its initial volume and the pressure required to attain its original volume (i.e., constant volume) is typically referred to as the swelling pressure. Swelling pressure is an in-built or intrinsic property of expansive soil; reflects only the swelling characteristics of the soil and will not be changed by placement conditions or environmental conditions. In other words, the swelling pressure is a unique property of an expansive soil.

(ii) Initial moisture content

The two important properties affecting the swelling percent and swelling pressure of an expansive soil are the water content and dry density. The lower the initial water content the higher would be the swell. The water content in a soil affects both the stress state and the volume change behaviour. In other words, water content is one of the key parameters that influence the total heave. Different soil structures are possible when an expansive soil is compacted at various densities and the different water contents contribute to different volume change behavioural characteristics.

(iii) Initial dry density

Several research studies show that both of the swelling percent and swelling pressure increase with the increase of dry density at constant initial water content (Chen

1975). When dry density decreases, swelling pressure rapidly approaches zero; when dry density increases, swelling pressure rapidly increases.

(iv) Surcharge pressure

The detrimental volume increase can be controlled or significantly reduced in expansive soils if sufficient load or surcharge is applied. Thus, a certain surcharge load can be recommended to control the foundation movement.

(v) Soil profile

The thickness of expansive soil affects the magnitude of total heave. If the thickness of high swell potential soils is thin, the potential total heave will be small. At the same time, for a thick stratum of low swelling soil, the total heave can be considerable over a long period of time.

(vi) Time allowed for swell.

The time required for the soil to reach its maximum swell potential may vary considerably depending essentially on the initial density, permeability, and the stratum thickness.

There are more variables needed to be considered in expansive soils, such as water content variation, swell fatigue and depth of heave to better understand the various parameters that influence the volume change behaviour.

2.3.5 Water content variation

There will be no volume change in expansive soils if the natural water content is constant. In other words, the structures founded on clays will not be subject to movement if the water content of the clay remains unchanged. Changes in water content contribute to significant changes the pore-water pressures (i.e., suction). The water content changes can be attributed to climatic factors, change in the depth of water table, water uptake by vegetation, removal of vegetation, or the excessive watering of a lawn. There are examples of causes for foundation heave as a result of the changes in the water content of the soil (Table 2.6, Figure 2.5). These changes are from the environment or from man-made causes. Lightly built construction on either shallow foundation or pile foundation can be affected.

Table 2.6 Examples of causes for foundation heave resulting from soil moisture content changes (Headquarters, U.S. Department of the Army 1983)

Change in field environment from natural conditions	Significant variations in climates, such as long droughts and heavy rains, cause cyclic water content changes resulting in edge movement of structures. Changes in depth to the water table lead to changes in soil water content
Changes related to construction	Covered areas reduce natural evaporation of moisture from the ground, thereby increasing the soil water content. Covered areas reduce transpiration of moisture from vegetation, thereby increasing the soil water content. Construction on a site where large trees were removed may leads to an increase of moisture because of prior depletion of soil water by extensive root systems. Inadequate drainage of surface water from the structure leads to pending and localized increases in soil water content. Defective rain gutters and downspouts contribute to localized increases in soil water content. Seepage into foundation sub soils at soil/foundation interfaces and through excavations made for basements or shaft foundations leads to increased soil water content beneath the foundation. Drying of exposed foundation soils in excavations and reduction in soil surcharge weight increases the potential for heave. Aquifers tapped provide water to an expansive layer of soil.
Usage effects	Watering of lawns leads to increased soil water content. Planning and growth of heavy vegetation, such as trees, at distances from the structure less than 1-1.5 times the height of mature trees, aggravates cyclic edge heave. Drying of soil beneath heated areas of the foundation, such as furnace rooms, leads to soil shrinkage. Leaking underground water and sewer lines can cause foundation heave and differential movement.

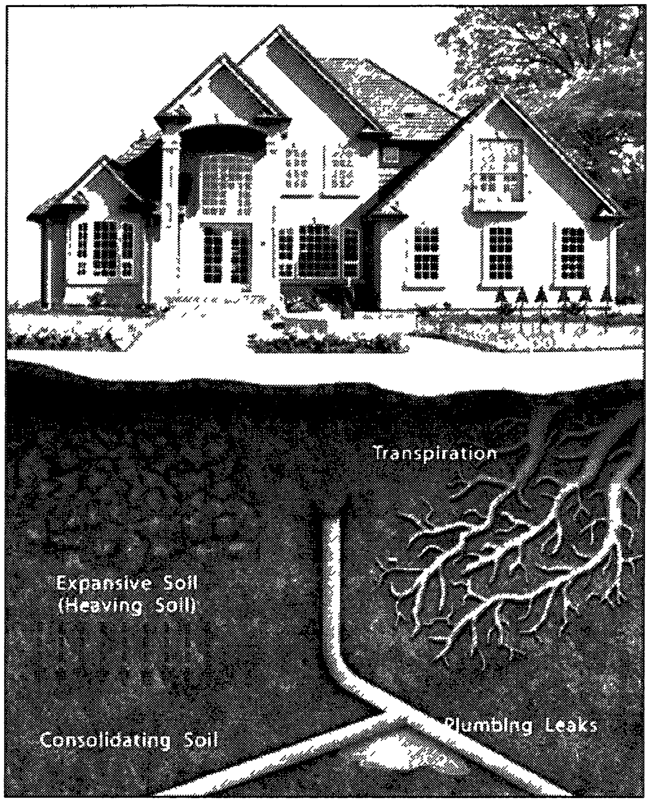


Figure 2.5 Changes in soil moisture content (http://en.wikipedia.org/wiki/Soil mechanics)

2.3.6 Fatigue

Fatigue is one of the reasons that pavements placed on expansive clays undergo seasonal movement due to wetting and drying and reach a point of stabilization after a number of years (Chen 1988). Figure 2.6 summarizes results from these experiments.

Fatigue of swelling was also observed in other research studies undertaken using controlled suction tests (Tripathy et al. 2002). Some researchers believe that if drying and wetting cycles are repeated, the swelling during the first cycle would be appreciably higher than that in subsequent cycles. Tripathy et al. (2002) studied the results from cyclic swelling and shrinkage paths of compacted expansive soil specimens and stated that, (i) the swelling and shrinkage path is reversible in terms of water content and void ratio once the specimen reaches an equilibrium condition (i.e., the vertical deformations during swelling and shrinkage are the same), and this generally occurred after about four swell-shrink cycles; (ii) the equilibrium swell-shrink path varies with the changes in surcharge pressure and swell-shrink pattern; (iii) the equilibrium swell-shrink path was unaffected by the initial placement conditions (i.e., dry density and water content).

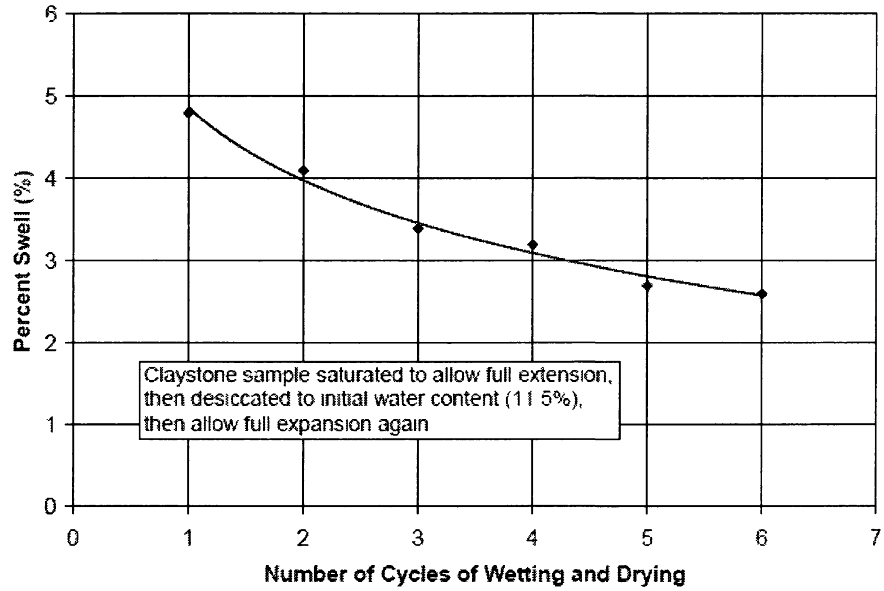


Figure 2.6 Effect of wetting and drying cycles on percent swell (modified after Chen 1965).

2.3.7 Environmental factors

Environmental factors such as climate involving precipitation, evaporation and transpiration significantly influence the movement of water in expansive soils. The depth and degree of desiccation affects the amount of swell. Climate condition partially affects the desiccation. The swelling soil located below ground water will not pose a problem to the structure (Chen 1975, Nelson and Miller 1992). The thickness of the swelling soil stratum is, therefore, limited by the depth to ground water. However, ground water fluctuates, and receding ground water sometimes contributes to additional swelling. Damaging swelling soil problems are seldom encountered for a soil profile with ground water located short distances beneath the footing. Capillary movement and vapour transfer will be such magnitude that it takes only a short period of time before the thin layer of swelling soil below the structure becomes completely saturated (Chen 1975).

2.3.8 Active zone

The depth of heaving is defined as the depth of soil that is contributing to heave due to soil expansion at any instant of time. Nelson et al. (2003) defined this depth as "active zone" which generally refers to the zone of soil that either is contributing to or has the potential to produce heave. In order to predict heave at any particular time, it is necessary to define the zone of soil that has experienced an increase in water content and what the swell potential of that zone is. Even though several years may be required for heave to occur, it is important to take into account the ultimate amount of heave that may occur in the life time of the structure in foundation designs.

Engineers have attempted to determine the zone of soil that is being wetted using different definitions, each of which considers a particular emphasis. Nelson et al. (2001) put forth the following four definitions regarding depth of water migration for purposes of clarity and consistency.

1) Active zone is that zone of soil that is contributing to heave due to soil expansion at any particular time. The active zone will normally vary with time.

2) Zone of Seasonal Moisture Fluctuation is that zone of soil in which water contents change due to climatic changes at the ground surface.

3) Depth of Wetting is the depth to which water contents have increased due to the introduction of water from external sources, or due to capillarity after the elimination of evapo-transpiration. The external sources can include such things as irrigation, seepage from ponds or ditches, broken water lines, and others.

4) Depth of Potential Heave is the depth to which the overburden vertical stress equals the swelling pressure of the soil. This represents the maximum depth of Active Zone that could occur.

The depth of active zone (Figure 2.7) can be determined by field measurements of moisture content or soil heave with depth over a number of seasons. Generally, the depth of heave depends on many factors, including the type of soil, the soil profile, changes in atmospheric conditions, and depth to the water table (Kumar 2000). Although no specific criteria considering these variables were found, the critical depth to the water table is probably a function of subgrade strength, subgrade permeability, subgrade capillarity, and the ratio of the design vertical live load stress to the live load plus dead load vertical stress (Ridgway et al. 1996). These items are important because the strength of the subgrade must be assessed at the effective stress level (i.e., the total stress less the pore pressure), whereas the driving force to cause failure is at the total stress level.

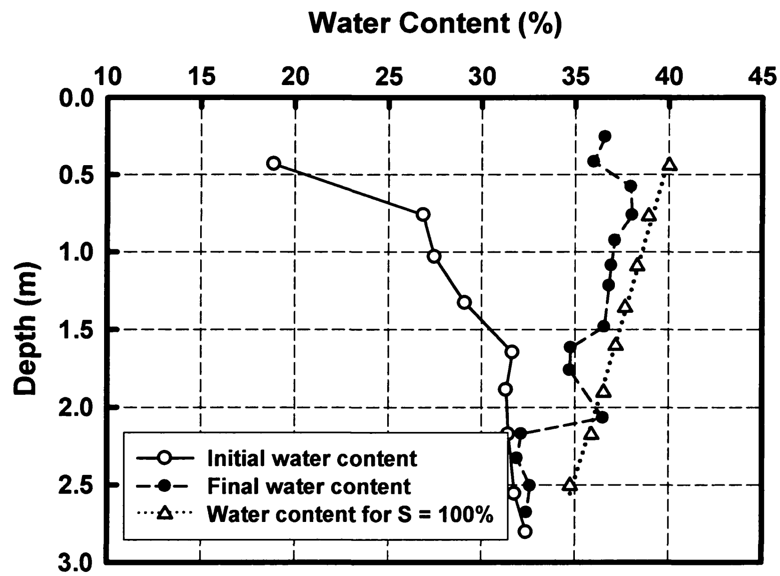


Figure 2.7 Variation of in situ water content with respect to the depth in active zone (modified after Yoshida et al. 1983).

2.4 Summary

There is a world-wide interest in understanding the expansive soils behaviour because of the various problems it poses particularly to the lightly loaded structures. Several researchers and practitioners have significantly contributed to our knowledge towards better understanding the expansive soils behaviour. This chapter mainly summarizes the general information about expansive soils extending unsaturated soil mechanics principles.

CHAPTER 3

HEAVE PREDICTION METHODS

3.1 Introduction

Conventional soil mechanics used for interpreting the engineering behavior of saturated soils cannot be applied for expansive soils as they are typically in a state of unsaturated condition. The engineering behavior of expansive soils is sensitive to both the changes in water content and as well as suction. Swell pressures associated with changes in water content and suction of unsaturated expansive soils contribute to undesirable heave movements contributing to significant damages to structures. The heave in the expansive soil typically continues until the degree of saturation is equal to 100%.

Practicing engineers rely on using a variety of design, construction, and stabilization techniques to reduce losses associated with expansive soils. However, the success of the techniques or procedures will mainly depend on key information such as the reliable determination or estimation of the swelling pressure and the soil heave. Reliable determination or estimation of the swelling pressure and the soil heave has proven to be difficult to date. Due to these reasons, the design of foundations in expansive soils has been one of the greatest challenges facing geotechnical engineers (Nelson et al. 2003).

Reliable design of foundations on expansive soils should include analyses of expected heave and consequences of foundation movement over the design life of the structure. The heave prediction techniques in unsaturated expansive soils have been studied both using the 3-D (i.e., Vu & Fredlund 2004; Masia et al. 2004; Wray et al. 2005) and the 1-D analysis (Johnson and Snethen 1978, Fredlund 1983, Hamberg and

Nelson, 1984 and Mitchell and Avalle 1984). The 1-D heave is more commonly used in the estimation, prediction and measurement in engineering practice applications. Most of the available methods in the literature for the prediction of heave generally make use of the linear relationship between void ratio (vertical strain) and the logarithms of net normal stress or soil suction (Fredlund 1983, Dhowian 1990 and Nelson et al. 2006).

Numerous techniques and analytical procedures have been proposed and millions of research dollars may have been invested in to collecting the valuable field data related to heave in expansive soils under different scenarios (for example, McKeen 1980; Snethen 1980; Fredlund 1983; Hamberg & Nelson 1984; Dhowian 1990; Mckeen 1992; Nelson & Miller 1992; Fityus & Smith 1998). This chapter provides a comprehensive summary of available heave prediction methods for expansive soils in the literature.

3.2 Constitutive Relationships for Volume Change Behavior

Several constitutive relationships are available in the literature using the mechanics of unsaturated soils to interpret the volume change behavior (Matyas and Radhakrishna 1968, Alonso et al. 1990, Fredlund and Rahardjo 1993). The basic constitutive relationships can be written in three different forms; which include the elasticity parameters, the compressibility parameters and the volume-mass form (Fredlund and Rahardjo 1993, Clifton et al. 1999). These relationships are summarized in this section.

3.2.1 Elasticity form

The elasticity form of the volume change constitutive relationships for an unsaturated soil can be formulated as an extension of the equations used for saturated soils. The soil is assumed to behave as an isotropic, linear elastic material, and the form for soil structure can be shown in below:

$$d\varepsilon_v = 3\left(\frac{1-2\mu}{E}\right)d\left(\sigma_{mean} - u_a\right) + \frac{3}{H}d\left(u_a - u_w\right) \tag{3.1}$$

where, $\sigma_{mean} = (\sigma_x + \sigma_y + \sigma_z) / 3$ is the average total normal stress

For a complete description of the volume change behavior in an unsaturated soil, a water phase constitutive relation is also required, which can be written as:

$$\frac{dV_w}{V_0} = \frac{3}{E_w}d\left(\sigma_{mean} - u_a\right) + \frac{d\left(u_a - u_w\right)}{H_w} \tag{3.2}$$

where, E_w is the water volumetric modulus associated with a change in $(\sigma - u_a)$; H_w is the water volumetric modulus associated with a change in $(u_a - u_w)$.

The above constitutive relationships are formulated for a general three-dimensional loading.

3.2.2 Compressibility form

The constitutive equations can also be written in a compressibility form which is more commonly used in conventional soil mechanics. The equation for the soil structure of an unsaturated soil to represent general three dimensional loading is given below:

$$d\varepsilon_v = m_1^s d\left(\sigma_{mean} - u_a\right) + m_2^s d\left(u_a - u_w\right) \tag{3.3}$$

where, $m_1^s = 3(1 - 2\mu) / E$ is the coefficient of compressibility with respect to net normal stress; $m_2^s = 3/H$ is the coefficient of compressibility with respect to matric suction.

The compressibility form for the water phase in terms of $(\sigma - u_a)$ and $(u_a - u_w)$ stress state variables is:

$$\frac{dV_w}{V_0} = m_1^w d\left(\sigma_{mean} - u_a\right) + m_2^w d\left(u_a - u_w\right) \tag{3.4}$$

where, $m_2^s = 3/E_w$ is the coefficient of water volume change with respect to net normal stress; $m_2^s = 3/H_w$ is the coefficient of water volume change with respect to matric suction.

3.2.3 Volume-mass form

The volume-mass form for the constitutive relationships for an unsaturated soil involves the use of void ratio, e, water content, w, and/or degree of saturation, S. Since the void ratio change can be independent of the water content change for an unsaturated soil, the volume change constitutive equation for three-dimensional loading is written in terms of void ratio as below:

$$de = a_t d\left(\sigma_{mean} - u_a\right) + a_m d\left(u_a - u_w\right) \tag{3.5}$$

where, $a_t = \partial e / \partial(\sigma - u_a)$ is the coefficient of compressibility with respect to a change in net normal stress, $d(\sigma - u_a)$; $a_m = \partial e / \partial(u_a - u_w)$ is the coefficient of compressibility with respect to a change in matric suction, $d(u_a - u_w)$.

For a complete volume-mass characterization, a second constitutive relationship is required. Such a relationship can be written as a water content relationship for three-dimensional loading as below:

$$dw = b_t d\left(\sigma_{mean} - u_a\right) + b_m d\left(u_a - u_w\right) \tag{3.6}$$

where, $b_t = \partial w / \partial(\sigma - u_a)$ is the coefficient of water content change with respect to a change in net normal stress, $d(\sigma - u_a)$; $b_m = \partial e / \partial(u_a - u_w)$ is the coefficient of water content with respect to a change in matric suction, $d(u_a - u_w)$.

Figure 3.1 shows these constitutive relationships which can be visualized in the form of volume-mass constitutive surfaces on three-dimensional plots, with a_t , a_m , b_t

and b_m as the slopes of the constitutive surfaces.

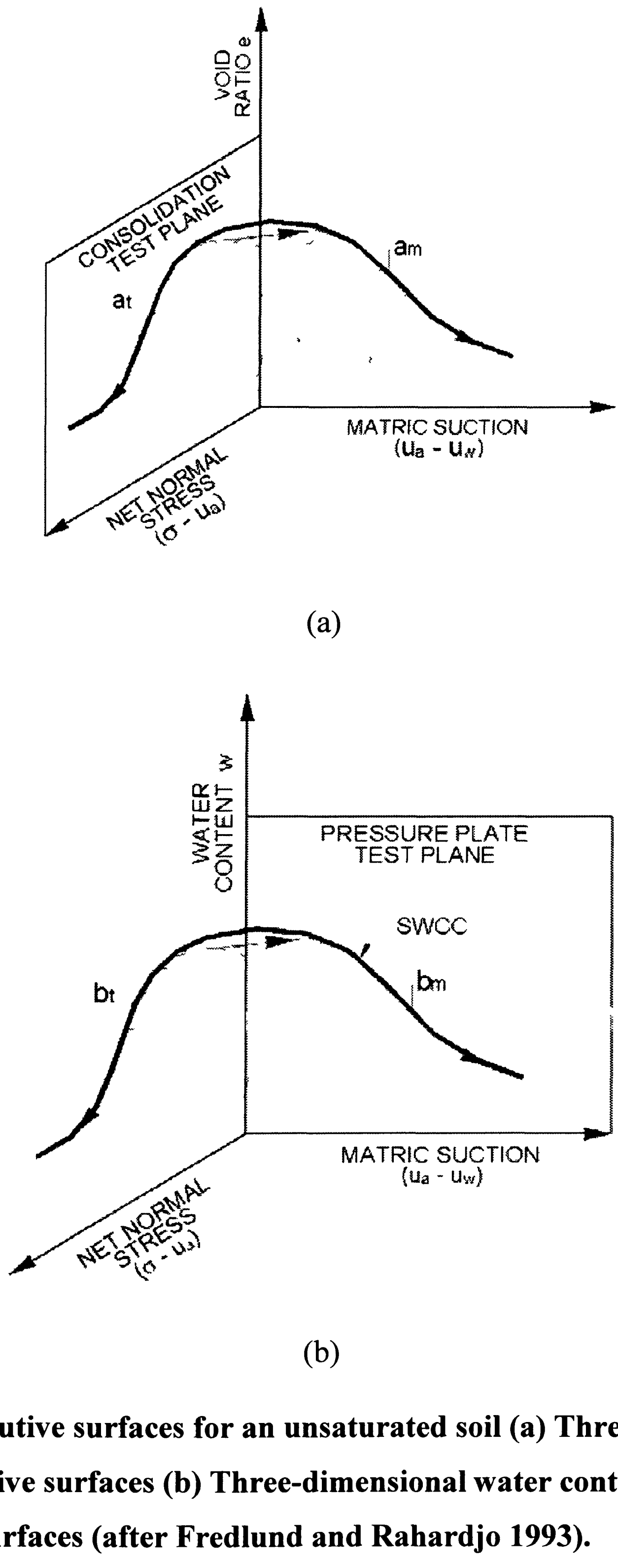


Figure 3.1 Constitutive surfaces for an unsaturated soil (a) Three-dimensional void ratio constitutive surfaces (b) Three-dimensional water content constitutive surfaces (after Fredlund and Rahardjo 1993).

The constitutive surface can also be shown by plotting the volume-mass parameters versus the logarithm of the stress state variables (Nelson and Miller 1992). The constitutive surfaces exhibit a symptom of "uniqueness" (Fredlund and Rahardjo 1993). The "uniqueness" of the constitutive surfaces demonstrates that there is only one relationship between the deformation and stress state variables (Chao 2007). Similar form of constitutive relationships can also be formulated for unloading conditions.

3.3 Heave Prediction Methodologies

Lightly loaded structures constructed on expansive soils are often subjected to severe distress subsequent to construction over a period of time as a result of changes in water content and associated pore-water pressures (i.e., suction) in the soil. The most commonly damaged structures are near ground surface structures such as roadways, airport runways, pipe lines, small buildings, irrigation canals and spillway structures. This section provides a brief background of the heave prediction methodologies commonly used in engineering practice.

Heave prediction methodologies were first developed in the late 1950s, and originated as an extension of methods used to estimate volume change due to settlement in saturated soils using results of one-dimensional oedometer (consolidation) tests (Chao 2007). Heave prediction methodologies have been refined continuously as knowledge and understanding of unsaturated soil behavior has increased. Taylor (1948) proposed a mathematical model describing settlement of a layer of saturated expansive soil. Jennings and Knight (1957) first proposed the extension of settlement theory to heave prediction using oedometer tests. Salas and Serratosa (1957) presented the oedometer heave prediction model in terms of the logarithmic pressure, and incorporated the "swelling pressure" of a soil into the equation. Their equation was of the same form as that presented by Taylor (1948). The "swelling pressure" of a soil was first defined by Palit (1953), as the pressure in an oedometer test required to prevent a soil sample from swelling after being saturated. Aitchison (1973) was one of the earliest investigators to propose a method for calculating moisture-related ground movements taking account of change in pore-water (i.e., suction model). Fredlund et al. (1980) was probably the first of the investigators to provide a theoretical framework to include soil suction in the prediction of heave in expansive soils. There are several techniques or procedures used in geotechnical engineering practice to estimate the swelling pressure, swell potential and the 1-D heave in expansive soils. These techniques can be divided into three main categories based on: (i) empirical methods; (ii) oedometer test methods; and (iii) soil suction methods.

3.3.1 Empirical methods

Empirical methods use soil classification parameters to predict swelling behavior of expansive soils. These methods are developed based on the limited data collected on local or soil from a region. Table 3.1 summarizes the list of several empirical methods from the literature.

3.3.2 Oedometer test methods

Prediction methods based on oedometers tests are more widely used when compared to the other two methods. The swelling pressure determined from oedometer test methods is one of the key parameters used in the determination of the 1-D heave. Table 3.2 and Table 3.3 summarize these methods.

Depending on the loading procedure, several methods are developed such as the free swell tests, the overburden swell tests; and the constant volume swell (*CVS*) tests using conventional oedometer test methods. The analysis of the 1-D oedometer tests should also take into account the loading and unloading sequence, surcharge pressure, sample disturbance, and apparatus compressibility. In many of the conventional consolidometer test procedures only the total stress is controlled.

Table 3.1 Summary of the empirical methods

Author	Equation/Description	Eqn.
Seed et al. (1962)	$SP = 0.00216 I_p^{2.44}$ *SP:* swelling potential, %; I_p : plasticity index.	(3.7)
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 is the thickness of nonexpansive layer;	(3.8)
Ranganathan & Satyanarayana (1965)	$SP = 0.000413 I_s^{2.67}$ I_s : shrinkage index, *(LL-SL)*.	(3.9)
Nayak & Christensen (1971)	$SP = 0.00229 I_p (1.45c) / w_i + 6.38$ $P_s(psi) = \left[\left(3.58 \cdot 10^{-2}\right) I_P^{1.12} c^2 / w_i^2\right] + 3.79$ w_i : initial water content; P_s is the swelling pressure; c: clay content.	(3.10) (3.11)
Vijayvergiva & Ghazzaly (1973)	$SP = 1/12\left(0.4LL - w_i + 5.55\right)$ $\log SP = 0.0526\gamma_d + 0.033LL - 6.8$ LL: liquid limit.	(3.12) (3.13)
Schneider & Poor (1974)	$\log SP = 0.9(I_p / w_i) - 1.19$	(3.14)
Chen (1975)	$SP = 0.2558e^{0.08381 I_p}$	(3.15)
Weston (1980)	$SP = 0.00411 LL_w^{4.17} \sigma_v^{-3.86} w_i^{-2.33}$ LL_w : weighted liquid limit	(3.16)
Picornell & Lytton (1984)	$\Delta H = \sum_{1}^{n} f_i \left(\Delta V / V\right)_i H$ H : the stratum thickness; $\Delta v/v_i$: volume change with respect to initial volume. f_i: factor to include the effects of the lateral confinement;	(3.17)
Dhowian (1990)	$\Delta H = (SP\%)\dfrac{H}{100}$	(3.18)

Table 3.2 Oedometer tests used for heave prediction (modified and expanded after the original contribution of Nelson and Miller 1992)

Tests	Location	Description	Reference
Double oedometer method	South Africa	Two tests performed on adjacent samples; a consolidation-swell test under a small surcharge pressure and a consolidation test, performed in the conventional manner but at natural moisture content. Analysis accounts for sample disturbance and allows simulation of various loading conditions and final pore-water pressures.	Jennings & Knight (1957)
Volumenometer method	South Africa	Uses specialized apparatus, air-dried samples were inundated slowly under overburden pressure.	DeBruijin (1961)
Sampson, Schuster & Budge method	Colorado, USA	Two tests performed on adjacent samples to simulate highway cut conditions; a consolidation-swell test under overburden surcharge, and constant volume-rebound upon load removal test.	Sampson et al. (1965)
Noble method	Canada	Consolidation-swell tests of remolded and undisturbed samples at various surcharge loads to develop empirical relationships for Canadian prairie clays.	Noble (1966)
Sullian and McClelland method	USA	Constant volume test, samples initially under overburden pressure on inundation.	Sullivan & McClelland (1969)
Komornik, Wiseman & Ben-Yacob method	Israel	Constant volume tests at various depths and swell-consolidation tests at various initial surcharge pressures representing overburden plus equilibrium pore water suction, used to develop swell versus depth curves.	Komornik et al. (1969)
Navy method	USA	Swell versus depth curves determined by consolidation-swell tests at various surcharge pressures representing overburden plus structural loads.	Navy (1971)
Wong & Yong Method	England	Swell versus depth is determined as in Komornik, Wiseman & Ben-Yacob method and Navy method; but surcharge loads of overburden plus hydrostatic pore water pressures are used.	Wong & Yong (1973)
USBR method	U.S.A	Double sample test, a consolidation-swell under light load, and a constant volume test.	Gibbs (1973)
Direct model method	Texas, USA	Consolidation-swell tests on samples inundated at overburden or end-of-construction surcharge loads.	Smith (1973)
Simple Oedometer	South Africa	Improved from double oedometer test. Single sample loaded to overburden, then unloaded to constant	Jennings et al. (1973)

		seating load, inundated and allowed to swell, followed by usual consolidation procedure.	
Mississippi State Highway Dept. method	Mississippi, USA	Consolidation-swell tests on remolded or undisturbed samples inundated at overburden surcharge loads.	Teng et al. (1972; 1973) Teng & Clisby (1975)
Controlled strain test	Colorado, USA	Constant volume swell pressure obtained on inundation followed by incremental, strain-controlled pressure reduction.	Porter & Nelson (1980)
University of Saskatchewan	Canada	Constant volume test. Procedure includes sample disturbance and apparatus deflection.	Fredlund et al. (1980)
Sridharan, Rao & Sivapullaiah method	India	Tests results from three methods, namely, a) conventional consolidation tests, b) equilibrium void ratios for different consolidation loads, and c) constant volume method are combined to study the swelling pressure of expansive soils. Results show that method a) gives a upper bound value; method b) gives the least value; and method c) gives the intermediate value.	Sridharan et al. 1986
Erol, Dhowian & Youssef method	Saudi Arabia	Assessment of the various oedometer test methods of ISO (improved swell oedometer test), CVS (constant volume swell test) and SO (swell overburden test) is used for heave prediction.	Erol et al. (1987)
Shanker, Ratnam & Rao method	India	Studying the multi-dimensional swell behaviour by testing cubic soil samples in oedometers. Swelling of samples is allowed to occur in 1-, 2- or 3- dimensions under a token surcharge.	Shanker et al. (1987)
Al-Shamrani & Al-Mhaidib method	Saudi Arabia	The stress path triaxial cell and oedometer are used to evaluate the vertical swell of expansive soils under multi-dimensional loading conditions. Several series of triaxial swell tests were conducted in which the influence of confinement on the predicted vertical swell was evaluated.	Al-Shamrani & Al-Mhaidib (1999)
Basma, Al-Homoud & Malkawi method	Jordan	Two commonly used method, zero swell test and the swell-consolidation test; and two relatively new techniques, "restrained swell test" and "double oedometer swell test" are using to study the swell pressure of the expansive soil. The restrained swell test is believed to give more reasonable results for swell pressure determination and thus is considered to more closely resemble field conditions.	Basma, et al. (2000)
Subba Rao & Tripathy	India	One-dimensional oedometer is used to study the swell-shrinkage behaviour of the compacted	Subba Rao & Tripathy

method	expansive soils. The compression-rebound tests were conducted on aged and un-aged compacted specimens by incrementally loading them up to a certain surcharge and then unloading. And the cyclic swell-shrinkage tests were carried out in fixed ring oedometers with the facility for shrinking the specimens at fixed temperature under constant surcharge pressure.	(2003)

The evolution of heave prediction methodologies using oedometer tests has been largely related to determination of the index parameters (i.e., swelling index, C_s; heave index C_ρ, C_H) and use them in the heave prediction equations. Burland (1962) first proposed using the slope of the rebound portion of the consolidation swell curve. Fredlund (1983) indicated that the slope of the unloading curve from consolidation-swell tests is approximately the same as the slope of the rebound curve determined from constant volume tests. Fredlund (1983) method and Nelson and Miller (1992) method used test results from both the consolidation-swell test and the constant volume test to determine the index parameters (i.e., C_s and C_ρ). Nelson and Miller (1992) method (Eq.(3.21)) uses the same equation as Fredlund (1983) method (Eq.(3.19)). Feng et al. (1998) presented a comprehensive comparison study of swell pressure as determined by different oedometer test methods. Nelson et al. (1998) and Bonner (1998) presented a method of estimating the index parameter using test results from only consolidation-swell tests. Nelson et al. (2006) refined the analysis and developed the methodology for determining the percent swell as a function of the inundation pressure.

3.3.2.1 Fredlund (1983) method:

Fredlund (1983) proposed an equation that can be used to calculate the 1-D heave in expansive soils using the constant volume swell (*CVS*) oedometer test results. The equation is shown below (Eq.(3.19) (see Table 3.3):

$$H = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} C_s \frac{H_i}{1+e_0} \log\left\{\frac{P_f}{P'_s}\right\}$$

where, H_i = thickness of the i_{th} layer, $P_f\ (= \sigma_y + \Delta\sigma_y - u_{wf})$ = final stress state, P'_s = corrected swelling pressure, C_s = swelling index, σ_y = total overburden pressure, $\Delta\sigma_y$ = change in total stress, u_{wf} = final pore-water pressure; and e_0 = initial void ratio.

The *CVS* test procedure involves inundating the sample in the oedometer while seated under a nominal load, which is typically 7 kPa (i.e., 1 psi). The load on the sample is increased to prevent any volume increase or swelling of the sample. The maximum applied stress required maintaining constant volume is the swell pressure. When the specimen no longer exhibits a tendency to swell, the applied load is further increased in a series of increments in a manner similar to that of a conventional consolidation test. Once the recompression branch of the consolidation curve has been established, the sample is rebounded by complete load removal in order to establish the swelling index, C_s.

The test has two main measurements; namely, corrected swelling pressure, P'_s and the swelling index, C_s. The undisturbed soil sample taken from the in-situ condition was subjected to the overburden stress (total stress) and matric suction. The total stress plane is combined of total stress and matric suction to provide an indication of the initial stress state in an expansive soil. The volume change with stress state can be computed by the change in stress state and the swell index, C_s.

The matric suction is brought to zero during inundation, but is not measured prior to this condition. However, the total stress on the specimen is increased to keep the specimen from increasing in volume. The swelling pressure represents the sum of the in-situ overburden stress and the matric suction of the soil translated into the total stress plane (Figure 3.2). Therefore, the swelling pressure is dependent on the in-situ suction.

The measured swelling pressure is underestimated unless the effect of "sampling disturbance" and "apparatus compressibility" are taken into account (Fredlund 1969). The interpretation of the *CVS* test must include a correction for the compressibility of the consolidation apparatus, the compressibility of filter paper, and the seating of the porous stones and the soil specimen. Sample disturbance will result in a measured swelling pressure that is lower than the in-situ value. Therefore, to determine the corrected swelling pressure, P'_s, the laboratory data should be corrected to account for the compressibility of the apparatus. The correction of sampling disturbance is also used in order to establish *P'ₛ* The "uncorrected" swelling pressure is typically low to use in the prediction of total heave. Predictions using "corrected" swelling pressures may often be twice the magnitude of those computed when no correction is applied (Fredlund 1983).

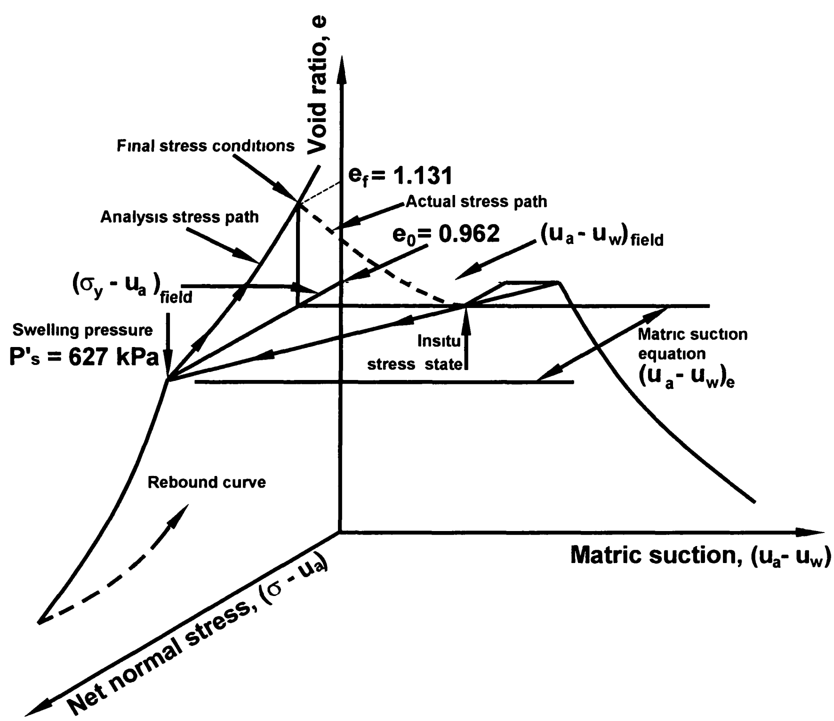


Figure 3.2 Stress paths representing swelling of Regina soil (modified from Fredlund, 1983).

The following procedure is suggested for obtaining P'_s from *CVS* test results. When interpreting the laboratory data, an adjustment should be made to the data in order to account for the compressibility of the oedometer apparatus. Desiccated, swelling soils have a low compressibility, and the compressibility of the apparatus can significantly affect the evaluation of in situ stresses and the slope of the rebound curve (Fredlund 1969). Because of the low compressibility of the soil, the compressibility of the apparatus should be measured using a steel plug substituted for the soil specimen. The measured deflections should be subtracted from the deflections measured when testing the soil. The adjusted void ratio versus pressure curve can be sketched by drawing a horizontal line from the initial void ratio, which curves downward and joins the recompression curve adjusted for the compressibility of the apparatus (Fredlund and Rahardjo 1993). Second, a correction can be applied for sampling disturbance after determining the equipment compressibility. Sampling disturbance increases the compressibility of the soil, and does not permit the laboratory specimen to return to its in situ state of stress as its in situ void ratio. More detailed testing procedures of this technique are available in ASTM D4546-2000.

Casagrande (1936) proposed an empirical construction on the laboratory curve to account for the effect of sampling disturbance when assessing the preconsolidation pressure of a soil. A modification of Casagrande's construction was extended for determining P'_s.

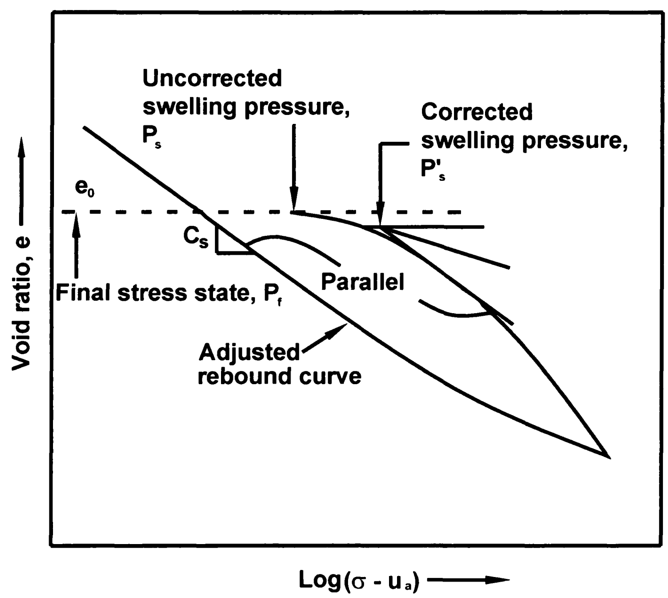


Figure 3.3 Construction procedure to determine the corrected swelling pressure incorporating the effect of sampling disturbance (modified from Fredlund 1987)

In this method, the point of maximum curvature where the void ratio versus pressure curve bends downward onto the recompression branch is determined (Figure 3.3). At the point of the maximum curvature, a horizontal line and a tangential line are drawn. The "corrected" swelling pressure is designated as the intersection of the bisector of the angle formed by these lines and a line parallel to the slope of the rebound curve which is placed in a position tangent to the loading curve (Fredlund and Rahardjo 1993).

Table 3.3 Summary of oedometer tests methods

Author	Equation/Description	Eqn.
Fredlund (1983)	$\Delta H = C_S \frac{H_i}{1+e_0} \log\left\{\frac{P_f}{P'_S}\right\}$ H_i: thickness of the i_{th} layer; $P_f\,(=\sigma_y + \Delta\sigma_y - u_{wf})$: final stress state; P'_s: corrected swelling pressure; C_s: swelling index; σ_y: total overburden pressure; $\Delta\sigma_y$: change in total stress; u_{wf}: final pore-water pressure; e_0: initial void ratio.	(3.19)
Dhowian (1990)	$\Delta H = H \frac{C_S}{1+e_0} \log \frac{P_S}{P_0}$ C_s :swell index; P_s : swelling pressure; P_0 : effective overburden pressure.	(3.20)
Nelson & Miller (1992)	$\Delta H = H \frac{C_\rho}{1+e_0} \log(\frac{\sigma'_f}{\sigma'_{cv}})$ C_ρ : heave index; σ'_{cv} : swelling pressure from constant volume swell test; σ'_f: vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.	(3.21)
Nelson et al., (2006)	$\Delta H = H\ C_H\ \log\left[\frac{\sigma'_{cv}}{(\sigma'_{vo})_z}\right]$	(3.22)
	$C_H = \frac{\%S_A}{\log\left[\frac{\sigma'_{cv}}{(\sigma'_i)_A}\right]}$ C_H : heave index; σ'_{cv} : swelling pressure from constant volume swell test; $\sigma'vo$: vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.	(3.23)

Fredlund (1983) method has been tested to predict 1-D heave on Regina soil, and there is a good agreement between the estimated heave and the in-situ measured heave. More details of this case study are available in Yoshida et al. (1983) and Fredlund and Rahardjo (1993). Discussions in later chapters of this thesis show that Fredlund (1983) method provides better estimates of the measured heave values in comparison to other heave prediction techniques. However, the determination of the corrected swelling pressure, P'_s from laboratory tests is elaborate and time consuming.

3.3.3 Soil suction methods

The swelling pressure and the 1-D heave in expansive soils can be more reliably measured or calculated using soil suction methods as they are based on the information of the stress state (i.e., suction). In these methods, the influence of suction is taken into account through the use of different parameters. Several heave prediction formulations based on soil suction methods proposed by various researchers are summarized in this section.

3.3.3.1 Hamberg & Nelson (1984) methods

Hamberg and Nelson (1984) presented an approach (Eq.(3.41), see Table 3.3) that can be used to predict heave of expansive soils using the relationship between water content and volume change in the range of shrinkage limit to liquid limit.

$$H = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} C_w \frac{H_i}{1+e_0} \Delta w$$

where, Δw is the change in water content;

The parameter, C_w in Eq.(3.42) represents the variation of volume of soil specimens with respect to water content (see Figure 3.4).

$$C_w = \frac{\Delta e}{\Delta w}$$

where, Δe is the change in void ratio; and Δw is the change in water content.

The parameter, C_w can be obtained using the Clod test which is the modified form of COLE (coefficient of linear extensibility; Brasher et al. 1966) test. The COLE test is originally developed to determine the heave beneath airfield pavements (McKeen 1981, McKeen and Hamberg 1981).

The test procedure involves coating soil samples with a liquid resin (i.e., DOW Saran F310) that allows for volume measurements at different moisture conditions (Nelson and Miller 1992). Once the resin dries on the soil sample, it acts as a flexible membrane, containing the soil material with its natural soil fabric intact. The resin is essentially waterproof when exposed to liquid water for a short time, but it permits gradual water vapor flow to and from the sample. The volume of a soil sample of any shape may be determined by weighing the soil clod while it is submerged underwater on a balance. The reading of the balance, adjusted for the weight of the pan and water, is a direct measurement of buoyant force on the sample. Sample volume can then be determined by Archimedes' principle.

In the original COLE procedure, each resin-coated sample was brought to 33 kPa suction in a pressure plate device. The sample volumes were determined at the initial adjusted moisture condition using the weighing procedure as described above. The samples were then oven dried for 48 hours, followed by another volume determination (Nelson and Miller 1992).

A COLE value was defined as the normal strain that occurs from the moist to the dry condition, defined with reference to the dry dimension, as Eq.(3.24). (Grossman et al. 1968)

$$COLE = \frac{L_M - L_D}{L_D} = \frac{L_M}{L} - 1 = \left[\frac{\Upsilon_{dD}}{\Upsilon_{dM}}\right]^{0\ 33} - 1 \tag{3.24}$$

where, L_M is the length of moist sample at 33 kPa suction; L_D is the length of oven dried sample; γ_{dM} is the dry density of moist sample at 33 kPa suction; and γ_{dD} is the dry density of oven dried sample.

The basic difference between the CLOD test and the COLE test procedures is that in the CLOD test, volume changes are monitored along a gradually varying moisture change path. This results in a smooth shrinkage or swelling curve for each sample (Nelson and Miller 1992). The basic CLOD test procedure to develop a shrinkage curve is as follows: (i) coat a sample with resin according to the COLE procedure specifications and measure its volume; (ii) allow sample to dry slowly in air; (iii) when the sample reaches a constant weight under laboratory humidity, volume and weight measurements are taken; (iv) samples are oven dried for 48 hr and taken a final volume and weight measurement.

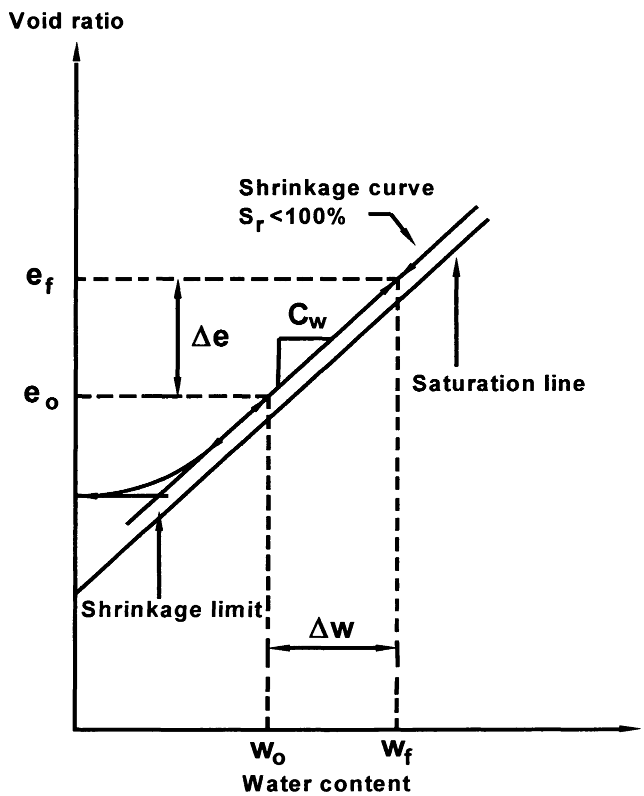


Figure 3.4 Water content versus void ratio relationship and the determination of suction modulus ratio, C_w (modified after Hamberg 1985).

The tests data provide void ratio and water content at various points. Void ratio and water content are both directly related to soil suction, thus the relationship between void ratio and water content expresses the effect of suction on void ratio. This relationship is valid only at water contents greater than the shrinkage limit; below the shrinkage limit, changes in water content are not accompanied by changes in volume, by definition (Hamberg 1985). For silty clay soils, the C_w versus water content relationship shows linear behavior for the water content greater than shrinkage limit (Hamberg 1985; Figure 3.4).

The limitations of Eq.(3.41) are: (i) the equation does not take into account the effect of applied load; and (ii) there are difficulties in estimating the C_w values for the water contents close to shrinkage limit due to the nonlinearity in that range. Table 3.4 lists the other most commonly used soil suction methods.

Table 3.4 Summary of soil suction methods.

Author	Equation/Description	Eqn.
Aitchison (1973)	$\Delta H = \frac{1}{100} \int_0^{H_s} I_{pt}\ \Delta u\ \Delta h$ ΔH :surface movement; I_{pt} :instability index of the soil; Δu: change in suction, in pF units, at depth z below the ground surface; Δh: thickness of the soil layer under consideration; H_s: depth of the design suction change	(3.25)
Lytton (1977)	$\Delta H = -\gamma_h\ H\ \log_{10}\left(h_f / h_i\right) - \gamma_\sigma\ H\ \log_{10}\left(\sigma_f - \sigma_i\right)$ h_f, h_i :final and initial water potentials; σ_f: applied octahedral normal stress; σ_i:is the octahedral normal stress above which overburden pressure restricts volumetric expansion; γ_h, γ_σ :two constants characteristic of the soil.	(3.26)
Johnson & Snethen (1978)	$\Delta H = H \frac{C_\tau}{1+e_0} \log \frac{h_0}{h_f + \alpha\sigma_f}$	(3.27)
	$C_\tau = \alpha\ G_s\ /\ (100\ B)$	(3.28)
	$\log h_0 = A - B\ w_0$ H : the stratum thickness; C_τ: suction index; α : compressibility index; e_0: initial void ratio; h_f: final matric suction , kPa; σ_f: final applied pressure, (overburden + external load), kPa;	(3.29)

	h_0 : matric suction without surcharge pressure, kPa
Snethen (1980)	$\Delta H = H \frac{C_\tau}{1+e_0}\left(A - B\, w_0\right) - \log(\tau_{mf} + \alpha\, \sigma_f)$ (3.30) $C_\tau = \alpha\, G_s / 100\, B$ (3.31) $\log \tau_m = A - B\, w$ (3.32) C_τ: suction index; τ_{mf} : final matric suction; σ_f : final applied pressure (overburden + external load); α, : compressibility factor *A, B*: constants (y-intercept and slope of soil suction versus water content curve, respectively).
McKeen (1980, 1992)	$\Delta H = -\gamma_h\, H\, \log \frac{h_f}{h_i}$ (3.33) $\gamma_h = -\frac{\Delta V / V_i}{\log_{10} \frac{h_f}{h_i}}$ (3.34) $\Delta H = H\, C_h\, \Delta\tau f\, s$ (3.35) $C_h = (-0.02673)(dh / dw) - 0.38704$ (3.36) $f = (1 + 2K_0)/3$ (3.37) $s = 1 - 0.01(\%SP)$ (3.38) γ_h :suction compression index; h_f, h_i : final and initial weighted suction, respectively. $\Delta v/v_i$: volume change with respect to initial volume. C_h: suction compression index, is the slope of volume change-soil suction curve; $\Delta\tau$ is the suction change in *pF*; *f* is the lateral restraint factor; K_0 is the coefficient of earth pressure at rest, equal to 1; *s* is the coefficient for load effect on heave; *SP* is the percent of swell pressure applied.

	$\Delta H = I_{pt} \, \Delta u \, H$	(3.39)
Mitchell & Avalle (1984)	$I_{pt} = \frac{\Delta L / L}{\Delta w} \cdot \frac{\Delta w}{\Delta u}$	(3.40)
	I_{pt} : instability index;	
	Δu :soil suction change.	
	$\Delta H = H \frac{C_w}{1+e_o} \Delta w$	(3.41)
	$C_w = \frac{\Delta e}{\Delta w}$	(3.42)
Hamberg & Nelson (1984)	$\Delta H = H \frac{C_h}{1+e_o} \Delta \log(h)_i$	(3.43)
	$C_h = C_w \, D_h$	(3.44)
	C_w :suction modulus ratio;	
	Δw: change in water content;	
	C_h: suction index with respect to void ratio;	
	D_h : suction index with respect to moisture content.	
	$\Delta H = H \frac{C_\psi}{1+e_0} \log \frac{\psi_i}{\psi_f}$	(3.45)
	$C_\psi = \frac{\alpha G_s}{100B}$	(3.46)
	$\Delta H = H \frac{\alpha G_s}{1+e_0} (w_f - w_i)$	(3.47)
Dhowian (1990)	$\Delta H = H \, C_w \, (w_f - w_i)$	(3.48)
	$C_w = \alpha \, G_s / (1+e_0)$	(3.49)
	C_ψ: suction index;	
	ψ_i : initial suction;	
	ψ_f : final suction;	
	α, : volume compressibility factor	
	B : slope of suction versus water content relationship;	
	G_s: specific gravity of solid particles;	
	C_w : moisture index	
Fityus & Smith (1998)	$\Delta H = H \, I_v \, a \, (w_{oi} - w_{of})$	(3.50)

$$I_v = 0.019 - 0.034 \log(\sigma_v) \quad (3.51)$$

I_v : volume index;

α: empirical factor accounting for confining stress differences in lab and field;

w_{0i} , w_{0f} : average initial water content and the average final water content, respectively;

σ_v : vertical stress at the midpoint of layer.

3.4 Details of Heave Prediction Techniques

Early swelling behavior studies were based on limited studies that focused on developing empirical relationships useful for explaining the local expansive soil behavior. Presently, oedometer tests methods (Table 3.3) and soil suction methods (Table 3.4) are more commonly used in engineering practice. Fredlund (1983) method and Hamberg and Nelson (1984) methods were discussed in greater detail in the earlier section. This section briefly provides the details of other heave prediction methodologies chronologically.

3.4.1 Aitchison (1973) method

This is probably the one of the earliest methods published in the literature that uses soil suction as a tool in the estimation of soil suction heave (Jaksa et al. 2009). This method was originally proposed for estimating heave in residential foundations built on expansive soils. The surface movement, ΔH can be estimated using the equation below (Eq.(3.25), see Table 3.4):

$$\Delta H = \frac{1}{100}\int_0^{H_s} I_{pt}\ \Delta u\ \Delta h$$

where, I_{pt}, the instability index of the soil, is defined as the percent vertical strain per unit change in suction; Δu is the change in suction, in pF units, at depth z below the ground surface; Δh is the thickness of the soil layer under consideration; H_s is the depth of the design suction change.

The ratio of vertical strain to suction change was experimentally observed and defined as instability index, I_{pt} by Aitchison and Woodburn (1969), Aitchison (1970), and Lytton and Woodburn (1973). This index is equivalent to the suction index, C_τ used by Snethen (1980) and Johnson (1979), or suction compression index, γ_h used by McKeen and Hamberg (1981). The Australian Standard for the design and construction of residential slabs and footing, AS 2870-1996 (Standards Australia 1996), specifies three methods for the estimation of $I_{pt:}$

1) Laboratory tests. Three such tests are suggested: the shrink-swell test, AS 1289.7.1.1 – 1992; the loaded shrinkage test, AS 1289.7.1.2 – 1992; and the core shrinkage test, AS 1289.7.1.3 – 1992 (Standards Australia 1992);

2) Correlations between the shrinkage index I_{pt} and other clay index tests;

3) Visual-tactile identification of the soil by an engineer or engineering geologist having appropriate expertise and local experience.

The method of proposing and estimating the index, I_{pt} has been widely adopted throughout the geotechnical engineering community (Mitchell 1989; Fityus and Smith 1998; Jaksa et al. 2009). The method 3) was named as visual-manual method (Mitchell, 1989). This technique involves a visual inspection of the soil and manually moulding and kneading the soil in order to estimate its plasticity index (Jaksa et al. 2009). Mitchell (1979) presented an approximate relationship between I_P and I_{pt} which is often used in the visual-tactile method to estimated I_{pt}. This relationship for estimating I_{pt} will be shown in section 3.4.6, Mitchell and Avalle (1984) method.

However, by its nature, the method is highly classifier-dependent (Jaksa et al. 2009). In addition, the laboratory and in-situ tests performed to obtain the required parameters (i.e., $I_{pt,}$ Δu) for the approach are both time consuming, expensive and difficult.

3.4.2 Lytton (1977) method

The method was proposed to calculate the heave and rate of heave of foundations resting on expansive soil deposits. The approach is based on few simple laboratory tests to determine the swell properties of the soil mass and some field observations of the shrinkage crack network. The ground surface heave experienced by an elemental volume of soil due to a change in water potential can be calculated using Eq.(3.26) (see Table 3.4):

$$\Delta H = -\gamma_h H \log_{10}\left(h_f / h_i\right) - \gamma_\sigma H \log_{10}\left(\sigma_f - \sigma_i\right)$$

where, h_f, h_i are the final water potentials; σ_f is the applied octahedral normal stress; σ_i is the octahedral normal stress above which overburden pressure restricts volumetric expansion; and $\gamma_{h,}$ γ_σ are two constants characteristic of the soil.

The volumetric strain can be represented as Eq.(3.52):

$$\frac{\Delta V}{V} = -\gamma_h \log_{10}\left(h_f / h_i\right) - \gamma_\sigma \log_{10}\left(\sigma_f - \sigma_i\right) \tag{3.52}$$

The volumetric strains have the same design as the first term (i.e., $-\gamma_h \log_{10}(h_f/h_i)$). The contribution of the second term (i.e., $-\gamma_\sigma \log_{10}(\sigma_f/\sigma_i)$), which was the contrary sign, is only considered with increasing depth until the strains become zero.

3.4.3 Johnson and Snethen (1978) method

Soil heave is induced by suction change within the active zone. When an expansive soil is wetted, its soil suction decreases and the soil volume increases. This concept was used as a tool in expressing the unit heave may be expressed as below (Eq.(3.27), see Table 3.4):

$$\Delta H = H \frac{C_\tau}{1+e_0} \log \frac{h_0}{h_f + \alpha \sigma_f}$$

$$C_\tau = \alpha \cdot G_s / (100 \cdot B)$$

$$\log h_0 = A - B \cdot w_0$$

where, H is the stratum thickness; C_τ is the suction index (Eq.(3.28)); α is the compressibility index; e_0 is the initial void ratio; h_f is the final matric suction , kPa; σ_f is the final applied pressure (overburden + external load), kPa; and h_0 is the matric suction without surcharge pressure (Eq.(3.29)), kPa.

The initial matric suction can be measured by thermocouple psychrometer or filter paper method. Soil suction measurements include preparing several undisturbed soil cubes and modifying the moisture contents of the samples to fit the range of field conditions of moisture variation (Snethen and Huang 1992). The resulting soil suction versus moisture content relationship is used to establish the required parameters. The parameters A and B, and compressibility factor are determined from the plotted results of the soil suction test procedure. A is the soil suction value (logarithmic scale) at zero moisture content; B is the slope of soil suction versus moisture content curve. The compressibility index, α is the slope of specific volume, $(1+e)/G_s$, versus moisture content curve. The suction index, C_τ, reflects the rate of change of void ratio with respect to soil suction. The initial soil suction, h_0, is measured during suction testing and the final suction profile is assumed as one of four suggested by Snethen (1980): (i) zero throughout the depth of active zone; (ii) linearly increasing with depth through the active zone; (iii) saturated water content profile; (iv) constant at some equilibrium value.

3.4.4 Snethen (1980) method

This method uses soil suction data (Figure 3.5) to characterize the expansive soil:

$$\log \tau_m = A - B\, w \qquad (3.53)$$

where, τ_m is the matric suction without surcharge pressure (i.e., atmospheric pressure)

(Eq.(3.53)). The slope of the line, B is determined by calculating the inverse of the change in water content over one cycle of the soil suction scale. The intercept, A is calculated by applying Eq.(3.32) at soil suction equal to 95.8 kN/m^2.

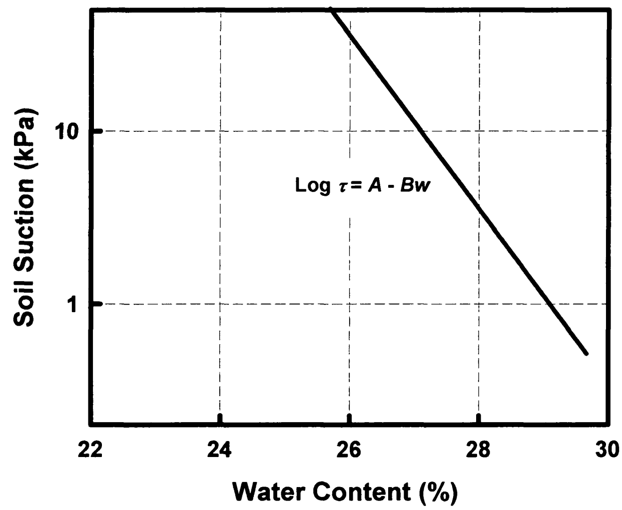


Figure 3.5 Soil suction versus water content relationships for Blue Hill Shale (Modified from Snethen 1980)

The volume change and swelling pressure of an expansive clay stratum is estimated using the equation below (Eq.(3.30), see Table 3.4) and Eq.(3.54):

$$\Delta H = H\frac{C_\tau}{1+e_0}\left[\left(A - B\,w_0\right) - \log(\tau_{mf} + \alpha\ \sigma_f)\right]$$

$$\log P_s = A - (100Be_0 / G_s) \tag{3.54}$$

$$C_\tau = \alpha\ G_s / 100\ B$$

where, C_t is the suction index (Eq.(3.31)); w_0 is the initial moisture content, percent; τ_{mf} is the final matric suction; α is the compressibility factor; σ_f is the final applied pressure (overburden + external load); A, B is constants (y-intercept and slope of soil suction versus water content curve, respectively).

The suction index, C_t reflects the rate of change of void ratio with respect to soil suction. The laboratory data necessary to apply the equation include G_s, e_0, A, B, w_0, and α, all of which (expect G_s) can be determined in the soil suction test procedure (Snethen 1980). The remaining two variables, τ_{mf} and σ_f are functions of assumed depth of active zone and the assumed final soil suction profile. The compressibility factor, α for highly compressible clays (i.e., CH) is commonly set equal to one, because the voids of these soils are filled with water within a wide range of moisture contents (Snethen 1980). In the absence of measured data, the compressibility factor may be roughly estimated from the plasticity index, PI as: (i) $PI < 5$, $\alpha = 0$; (ii) $PI > 40$, $\alpha = 1$; (iii) $5 < PI < 40$, $\alpha = 0.0275PI$-0.125.

The equation provides predictions of in-situ volume change of a soil stratum with respect to field conditions of soil composition, structure, initial and equilibrium moisture profiles, and confining pressures.

3.4.5 McKeen (1980, 1992) methods

McKeen (1980, 1992) used the suction compression index γ_h introduced by Lytton (1977) as a tool to propose a heave prediction technique as shown below (Eq.(3.34), see Table 3.4):

$$\gamma_h = -\frac{\Delta V / V_i}{\log_{10} \dfrac{h_f}{h_i}}$$

where, γ_h is suction compression index; h_f, h_i is final and initial weighted suction, respectively. $\Delta v/v_i$ is the volume change with respect to initial volume.

To study the volume change on expansive soil, a measure of the physical phenomena was needed to facilitate rating, classifying, and discussing various soils, γ_h was selected for this purpose. Selection of this index was based on: a suction-based approach to the problem was needed, and a volume-response coefficient was required for volume change studies. The suction compression index γ_h can be determined using different testing methods. The required data are a measured volume change and a suction change over which the volume change occurred (McKeen 1980). These data must cover the range of moisture suction expected in the field environment. The measurements may take place in a 1-D oedometer or a 3-D configuration (i.e., unrestrained soil clods) (McKeen 1980). One test routinely used for this purpose is the COLE (coefficient of linear extensibility) used by the U.S. Department of Agriculture Soil Conservation Service. All procedures for determining γ_h require suction measurements. For convenience, the empirical relationships between this parameter and soil activity and cation exchange activity, or percentage of clay are established by statistical studies (McKeen and Nielsen 1978):

For clay content is between 40% and 70% and high activity:

$$\gamma_h = 0.00179C - 0.041 \tag{3.55}$$

For clay content is between 25% and 70% and low activity:

$$\gamma_h = 0.00057C - 0.00057 \tag{3.56}$$

where, C = percent < 2 μm (ASTM D422)

Thus, McKeen (1980) method can be expressed as below (Eq.(3.33), see Table 3.4):

$$\Delta H = -\gamma_h \, H \, \log \frac{h_f}{h_i}$$

McKeen (1992) method can be shown as below (Eq.(3.35)).

$$\Delta H = H \, C_h \, \Delta\tau \, f \, s$$

$$C_h = (-0.02673)(dh / dw) - 0.38704$$

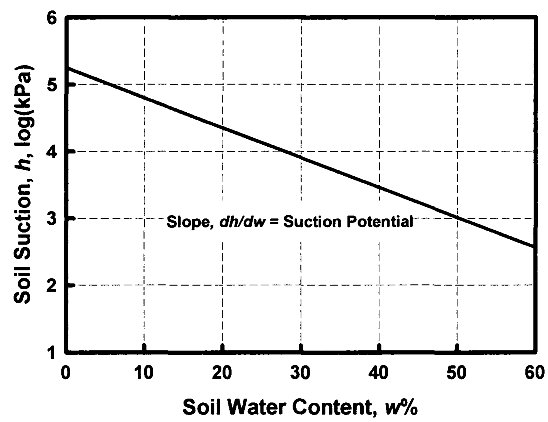
where, C_h, the suction compression index (Eq.(3.36)), is the slope of volume change-soil suction curve; $\Delta\tau$ is the suction change in pF; f is the lateral restraint factor (Eq. (3.37)):

$$f = (1 + 2K_0)/3$$

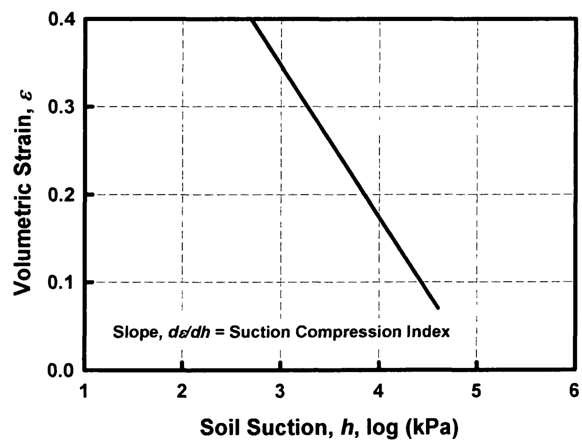
K_0 is the coefficient of earth pressure at rest, equal to 1; s is the coefficient for load effect on heave (Eq.(3.38));

$$s = 1 - 0.01(\%SP)$$

where, SP is the percent of swell pressure applied.



(a)



(b)

Figure 3.6 Determination of the suction compression index (modified from McKeen 1992).

McKeen (1981, 1985 and 1990) proposed a classification scheme by correlating the suction potential index, *dh/dw* and the suction compression index, C_h (Figure 3.6) using a database on Texas soils. In this correlation, a linear relationship is suggested such that 85% of the C_h values are larger than would be predicted by the relationship (Olsen et al. 2000). The required information includes suction potential, *dh/dw*, for which relatively simple suction and water content measurements are needed. This method provides a useful index for estimating the more fundamental suction compression index, which also requires the CLOD test (Olsen et al. 2000). McKeen (1992) further develops quantitative criteria for using both suction potential and suction compression index values to differentiate five categories of swell potential ranging from very high to non-expansive, as shown in Table 3.5.

Table 3.5 McKeen's swell potential categories (Olsen et al. 2000)

Category	Swell Potential	Suction Potential *dh/dw*	Suction Compression Index, C_h
I	Very High (McKeen calls this category "Special Case")	> -6	< -0.227
II	High	-6 to -10	-0.227 to -0.120
III	Moderate	-10 to -13	-0.120 to -0.040
IV	Low	-13 to -20	-0.030 to non-expansive
V	Non-expansive	< -20	Non-expansive

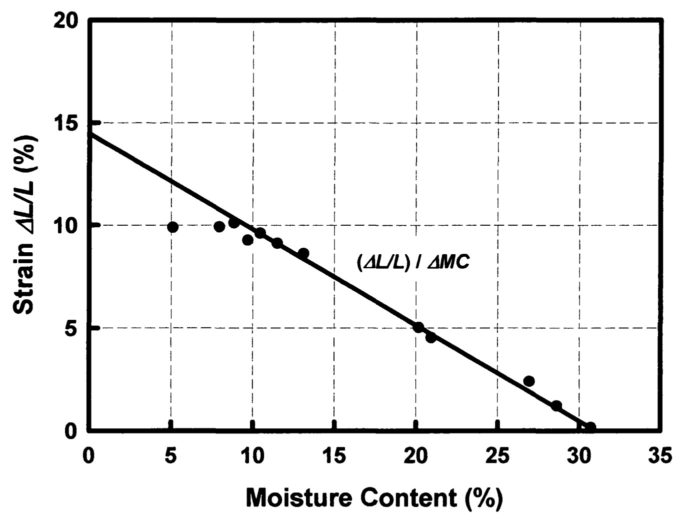
McKeen's suction potential, *dh/dw* and suction compression indices, C_h are obtained on undisturbed clods of soil and they are governed not only by the type and amount of clay in a soil, but also by the structure and pore-fluid chemistry of the soil, that result from its geologic origin and history (Olsen et al. 2000).

3.4.6 Mitchell and Avalle (1984) method

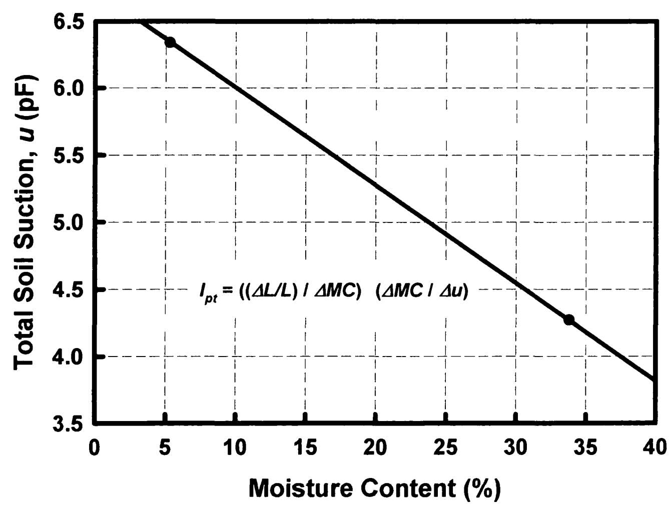
A simple and relatively quick method is presented to enable the prediction of expansive soil movements from soil suction changes. This method is assumed that vertical strain of the expansive soil is linearly proportional to soil suction. Therefore, the ground heave can be expressed as Eq.(3.39) (see Table 3.4):

$$\Delta H = H\, I_{pt}\, \Delta u$$

where, I_{pt} is the instability index; Δu is the soil suction change.



(a)



(b)

Figure 3.7 Determination of I_{pt} (modified from Mitchell and Avalle 1984).

The instability index I_{pt} is obtained from the core shrinkage test. It involves the measurement of the linear strain versus moisture content relationship, $\varepsilon_v/\Delta w$, and the moisture characteristic c ($c = \Delta w/\Delta u$), of unconfined undisturbed core samples (Figure 3.7; Eq.(3.40)). Generally, the more clayey the soil, the higher is its moisture characteristic (Morris and Gray 1976, Mitchell 1979). Several soil specimens are required for obtaining information about different initial water contents. The specimens are air dried for a period of two days, during which the length and mass of the shrinkage core are measured frequently. It is then oven dried to obtain the moisture content. The moisture characteristic, c, can be determined directly through soil suction test using thermocouple psychrometer or the filter paper methods on companion specimens. The instability index, I_{pt} is calculated as the slope of linear dimension change versus moisture content, $\Delta\varepsilon_v/\Delta w$, times the moisture characteristic, c (Eq.(3.40)):

$$I_{pt} = \frac{\varepsilon_{vert}}{\Delta u} = \frac{\Delta L / L}{\Delta w} \cdot \frac{\Delta w}{\Delta u}$$

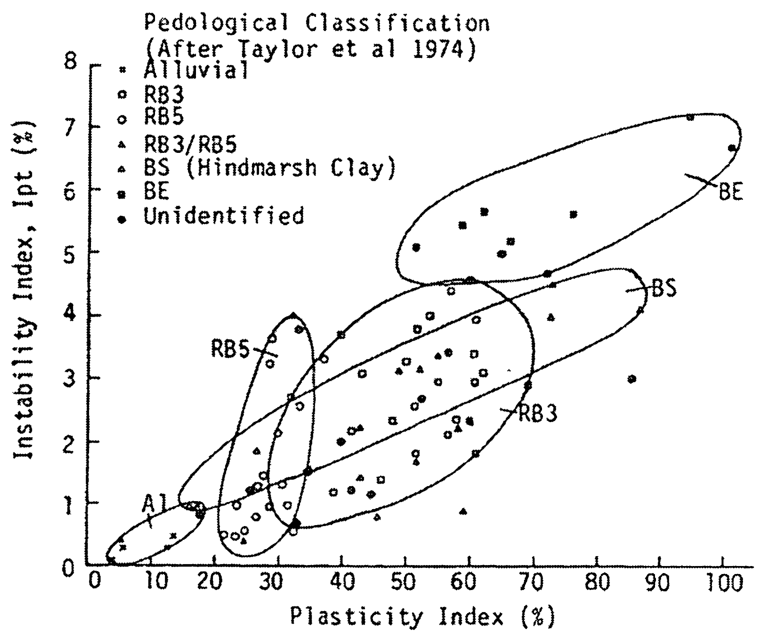


Figure 3.8 Relationship between instability index and plasticity index (Mitchell and Avalle 1984).

Mitchell and Avalle (1984) shows the core shrinkage tests results of 80 samples from 18 sites in South Australia and three samples from one site in Victoria, Australia in Figure 3.8. It seen that considerable scatter exists in the relationship between I_{pt} and I_p so that a simple relationship does not appear to exist; however, in generally, the higher I_p, the higher the I_{pt}. If the plasticity of a soil is assessed in a qualitative way (i.e. visual-tactile method), then from experience with a particular geological location, a reasonable accurate value of I_{pt} may be adopted without testing (Mitchell and Avalle 1984).

3.4.7 Hamberg and Nelson (1984) method:

Heave is related to soil suction change and soil suction is dependent on the moisture content of the soil; therefore, heave may be predicted by measuring changes in moisture content. Hamberg and Nelson (1984) extended this philosophy and proposed a method to determine the heave of expansive soils using suction modulus ratio, C_w, (Eq.(3.42)) .The amount of heave can be calculated using (Eq.(3.41), see Table 3.4):

$$\Delta H = H \frac{C_w}{1+e_0} \Delta w$$

If the final soil suction profile is available instead of moisture content change profile (ΔH can be calculated using the equation below Eq.(3.43), see Table 3.4):

$$\Delta H = H \frac{C_h}{1+e_0} \Delta \log (h)_t$$

$$C_w = \Delta e / \Delta w$$

$$C_h = C_w \cdot D_h$$

where, C_h is the suction index with respect to void ratio (Eq.(3.44)); D_h is the suction index with respect to moisture content.

Actual ground heave may be adjusted depending on the confinement situation:

$$\Delta H_{act} = f\ \Delta H \tag{3.57}$$

where, f is the correction factor, 0.33-1.0.

3.4.8 Dhowian (1990) method

Dhowian (1990) estimated heave of expansive shale formation based on soil suction change. The model described the field volume change where the anticipated heave is compared with direct measurement obtained from the field station. For the purpose of obtaining in-situ heave measurements, an instrumented field station was established in the central region of Saudi Arabia (Figure 3.9). Expansive shale predominates near the ground surface in this area. Undisturbed shale samples were obtained from the field and used for the improved oedometer tests (*ISO*) and the constant volume tests (*CVS*), and the swell overburden tests (*OSO*). The swell parameters obtained from results of oedometers tests are used to predict the anticipated field heaves using the following relationship (Eq.(3.20), see Table 3.3):

$$\Delta H = H\frac{C_s}{1+e_0}\log\frac{P_s}{P_0}$$

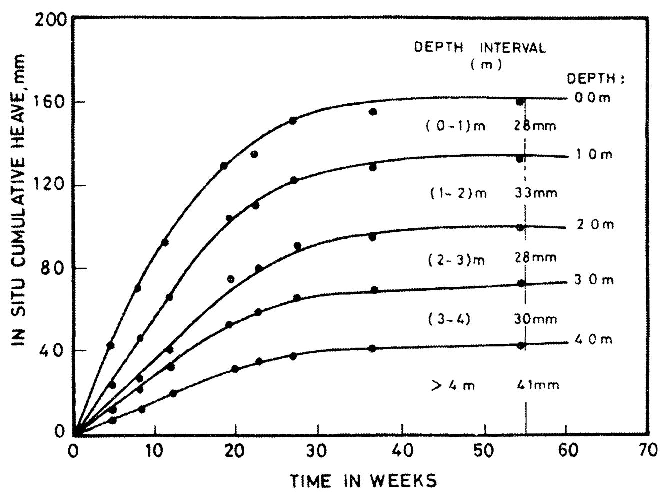


Figure 3.9 Development of heave with time for different depth intervals (Dhowian 1990).

The method uses the same equation (Eq.(3.19)) as Fredlund (1983). Where, C_s is the swell index; P_s is the swelling pressure; P_0 is the effective overburden pressure.

Amongst the oedometer tests, *ISO* gave the highest swelling pressure value, *OSO* gave the least value, and the *CVS* is in between. The C_s value from *OSO* was significantly higher than *ISO* and *CVS*. The estimated heaves calculated from Eq. (3.20) using tests results from three types of oedometer tests are compared with the measured field heave values. The agreement between the predictions based on *CVS* tests and the field heave is quite well (Al-Shamrani and Dhowian 2002) (Figure 3.10).

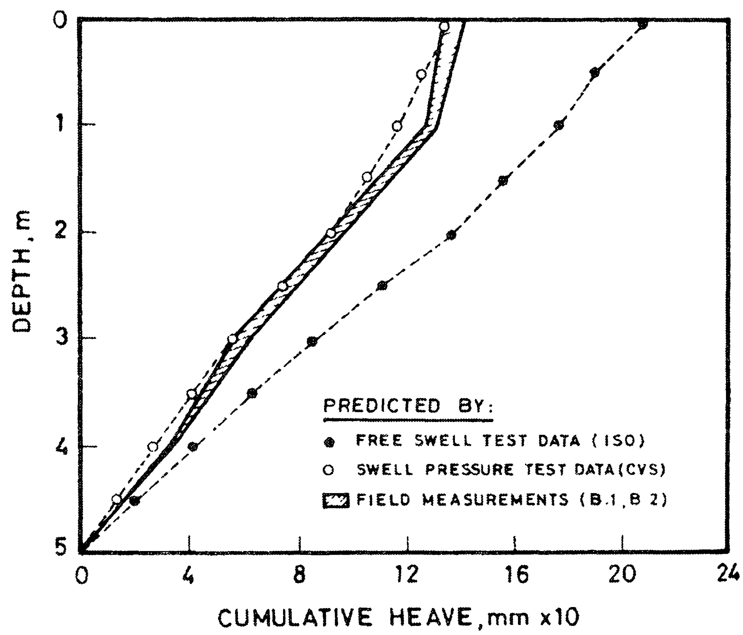


Figure 3.10 Measured and predicted heave based on oedometer technique (Dhowian 1990).

Apparently, the linearity (Figure 3.11) was between swell and log-suction for most homogeneous undisturbed shale samples tested under controlled conditions. Such a trend is analogous to the *e*-log *P* curve in the consolidation; hence, the suction method can be present in the equation as Eq.(3.45) (see Table 3.4):

$$\Delta H = H\frac{C_{\psi}}{1+e_0}\log(\psi_i/\psi_f)$$

where, C_ψ is the suction index; ψ_i is the initial suction; ψ_f and is the final suction.

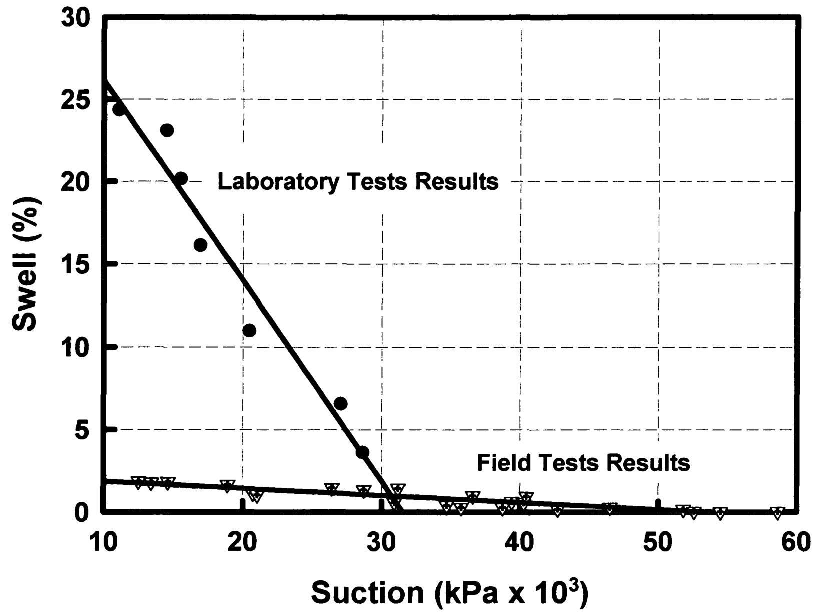


Figure 3.11 Suction during the course of swelling was measured in the field as well as in the laboratory (modified from Dhowian 1990).

The suction index C_ψ reflects the ratio of change of the void ratio and the soil suction and can be calculated by Eq.(3.46):

$$C_\psi = \frac{\alpha G_s}{100B}$$

where, α, the volume compressibility factor, is the slope of specific volume versus water content plots; B is the slope of suction versus water content change; and G_s is the specific gravity of solid particles.

The equation tends to overestimate field heave considerably. The discrepancy between the predicted and measured heave may be attributed to the experimentally determined parameters α and B. For this purpose, the average change in soil suction and water content coupled with the average ultimate swell measured at the end of the observation period are back-calculated to obtain the parameters α and B. The values are presented in Table 3.6 with the laboratory data. The discrepancy between the field and laboratory value of α is attributed to the measurement of volumetric swell rather than the vertical swell because of the lateral restraint in the oedometer chamber. Therefore, one third of the experimentally determined value is considered as the field value of α and substituted in the suction method.

Table 3.6 Swell and suction parameters obtained from laboratory tests (Dhowian 1990).

Parameter	Clay Shale		Silty Shale
	Lab Data	Field Data	
α (m^3/κN)	0.090	0.029	0.085
B	0.047	0.051	0.070
C_ψ	0.517	0.152	0.327

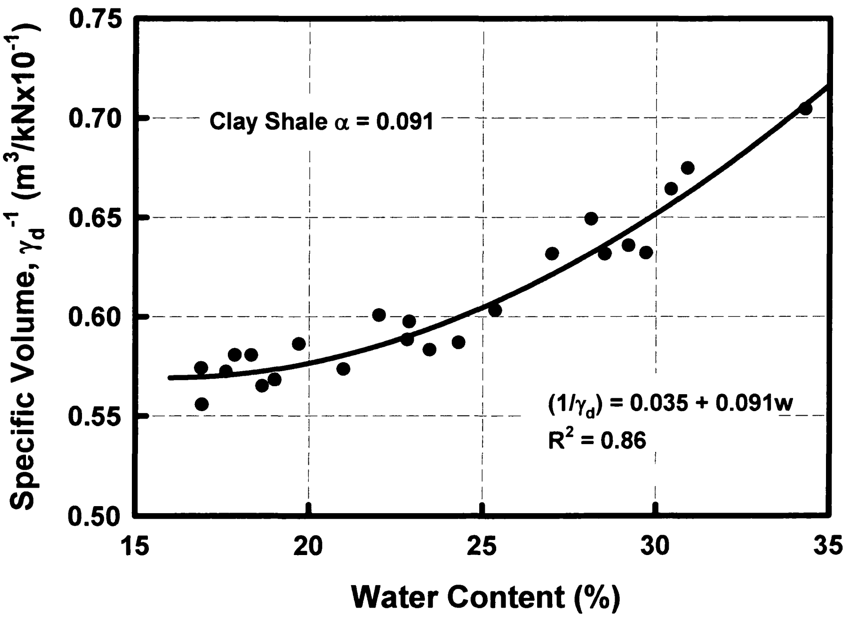


Figure 3.12 Specific volume versus water content plot for clayey shale (modified from Dhowian 1990).

The slope of the linear part (Figure 3.12) is defined as the compressibility factor α, hence

$$\Delta e = \alpha \ G_s \ (w_f - w_i) \tag{3.58}$$

where w_i, w_f are the initial and final moisture contents, respectively. The log-suction versus swell relationship has been found out to be as

$$\Delta e = C_\psi \log(\psi_i / \psi_f) \tag{3.59}$$

Thus, the relationship for C_ψ is equal to

$$C_\psi = \alpha\ G_s\ (w_f - w_i) / \log(\psi_i / \psi_f) \tag{3.60}$$

Using the suction method, the heave prediction can be showed as below (Eq.(3.47)):

$$\Delta H = H \frac{\alpha G_s}{1+e_0}(w_f - w_i)$$

By introducing a term C_w, which is defined as the moisture index, the equation can be shown as below (Eq.(3.49)):

$$C_w = \alpha\ G_s / (1 + e_0)$$

$$\Delta H = H\ C_w\ (w_f - w_i)$$

The equations above (Eq.(3.48)) indicate that heave is linearly proportional to the change in moisture content. It must be noted that in calculating C_w, the experimentally determined compressibility factor α is used instead of the field value.

3.4.9 Nelson and Miller (1992) method:

The method presented in Nelson and Miller (1992) used test results from both the consolidation-swell test and the constant volume test to determine the index parameter. The method uses the same equation (Eq.(3.19)) as Fredlund (1983).

"Free-field" heave is the amount of heave that the ground surface will experience due to wetting of the sub-soils with no surface load applied. Because the surface load applied by slab-on-grade floors is relatively smaller in comparison to the swell pressure generated by an expansive soil, the heave of slabs is essentially the same as the free-field heave (Chao 2007). The general equation for predicting heave using the oedometer methods can be presented as below (Eq.(3.21), see Table 3.3).

$$\Delta H = H \frac{C_\rho}{1+e_0} \log(\frac{\sigma'_f}{\sigma'_{cv}})$$

where, ΔH is the free field heave; C_ρ is the heave index (i.e., this index is equal to the corrected swelling index, C_s in Fredlund (1983) method); σ'_{cv} is the swelling pressure from constant volume swell test; σ'_f is the vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.

3.4.10 Fityus and Smith (1998) method:

The surface movement prediction presented in this method integrates volume changes which occur in sub-layers of soil over the depth of the active zone to give an estimate of the total movement at the ground surface. This method requires the information of change in gravimetric water content as a function of depth through the soil profile, as well as an appropriate index to relate moisture change directly to volume change. As the amount of volume change ΔH is affected by the confining stress of the overburden, a volume change index is used which takes this into account (Eq.(3.50), see Table 3.4).

$$\Delta H = H\ I_v\ a\ (w_{oi} - w_{of})$$

$$I_v = 0.019 - 0.034 \log(\sigma_v)$$

where, I_v is the volume index (Eq.(3.51), Table 3.4); α is an empirical factor accounting for confining stress differences in lab and field; w_{0i} , w_{0f} is the average initial water content and the average final water content, respectively; σ_v is the vertical stress at the midpoint of layer.

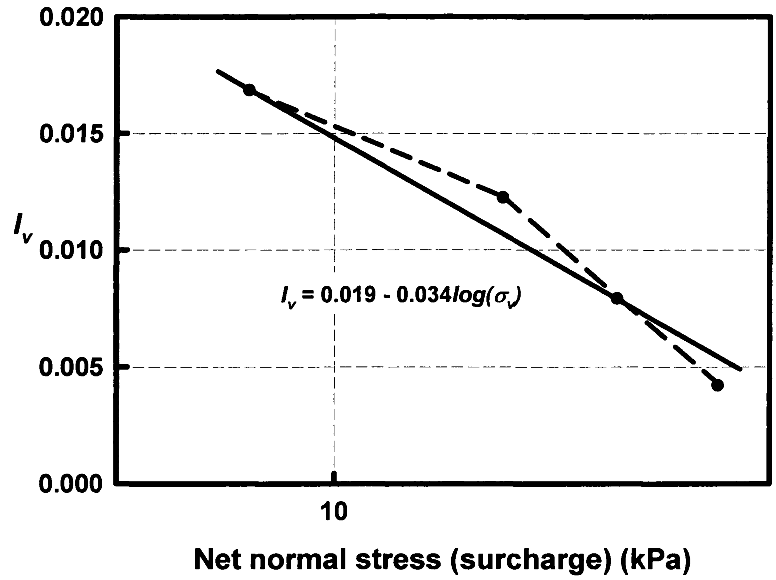


Figure 3.13 Estimation of the volume change index, I_v for Maryland clay (modified from Fityus and Smith 1998)

The load dependent volume index, I_v was determined from a series of simple, one dimensional swell tests, in which a clay sample is allowed to swell under a vertical confining stress that was equal to the calculated vertical stress experienced under field conditions. The tests were conducted on Maryland clay. The approach described here is based on a number of assumptions (Fityus and Smith 1998): (i) it is assumed that a given change in water content will always correspond with the same change in strain (ii) it is assumed that the volume index, I_v determined from swelling clay tests can be employed to predict both swelling and shrinking behavior (iii) it is assumed that the difference between one and three dimensional volume changes can be reasonably accommodated using a factor, α, of 0.33.

3.4. 11 Nelson et al. (2006) method:

Nelson et al. (2006) describes a methodology to determine heave using oedometer test data. An important constitutive parameter used in the method is the heave index, $C_{H,}$ which is the ratio of the percent swell observed in the oedometer test to the vertical stress applied to the sample when it was inundated (i.e., the inundation pressure).

When oedometer test data is plotted in a two dimensional format, the entire stress path is projected onto the plane defined by the (σ - u_a) and ε_v axes. The consolidation-swell test data follows the projected path ABCF, whereas the constant volume test data follows the projected stress path ABD. Example test data is shown as below. The slope of the loading portion of the curve shown in the figure is the compression index C_c, and of the rebound portion of the curve is the rebound index C_s. The volumetric strain experienced during inundation is the percent swell.

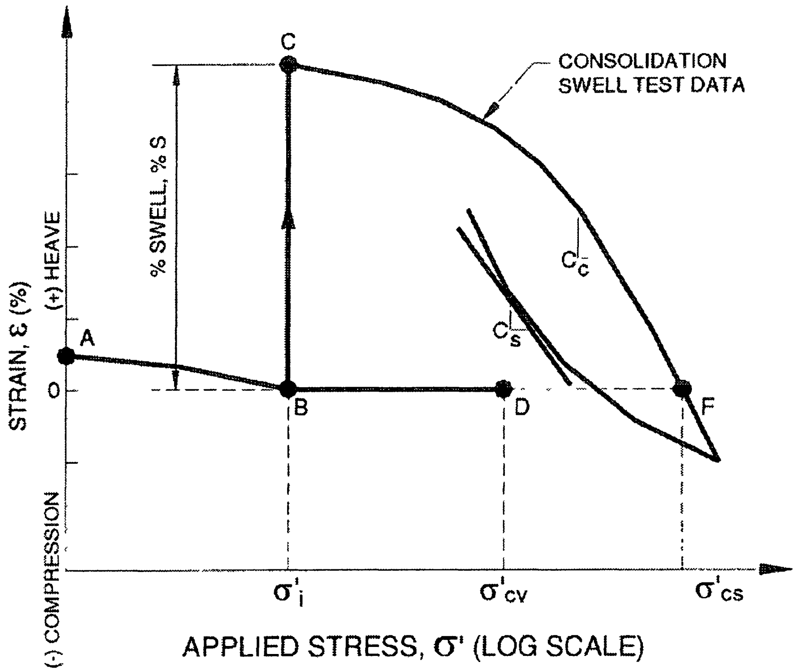


Figure 3.14 Terminology and notation for oedometer tests (Nelson et al. 2006).

Figure 3.15 shows the vertical overburden stress states at three different depths in a soil profile with similar soil throughout. At all points all samples are in a condition of zero lateral strain with a vertical overburden stress equal to σ'_{vo}. If a consolidation-swell test is conducted on a sample identical to that at depth, Z_A, at an inundation stress, $(\sigma'_i)_A = (\sigma'_{vo})_A$, the sample will swell by an amount $\%S_A$ as shown in Figure 3.16. Similarly for a sample at depth Z_B, the sample would be subjected to an inundation stress, $(\sigma'_i)_B = (\sigma'_{vo})_B$, and the sample would swell by an amount $\%S_B$.

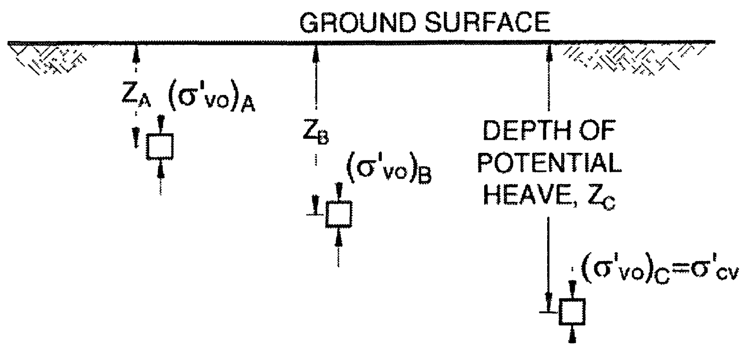


Figure 3.15 Vertical overburden stress states at three different depths (Nelson et al. 2006).

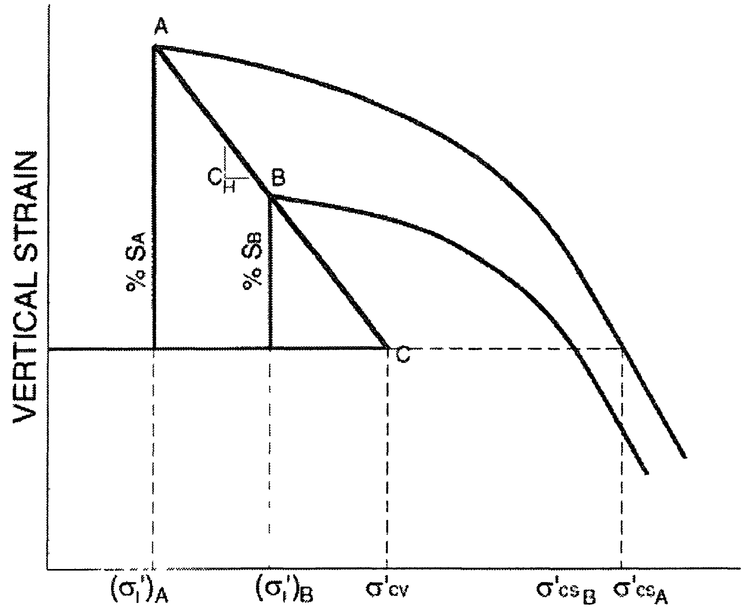


Figure 3.16 Hypothetical oedometer test results for stress states shown in Figure 3.11 (Nelson et al. 2006).

The general equation for predicting heave or settlement in a soil stratum of thickness *H* can be shown as Eq (3.61):

$$\Delta H = H\frac{\Delta e}{1+e_0} \tag{3.61}$$

For uniform vertical strain throughout the stratum the strain is equal to:

$$\varepsilon_v = \frac{\Delta e}{1+e_0} \tag{3.62}$$

Substituting Eq.(3.62) into Eq.(3.61):

$$\Delta H = H\ \varepsilon_v = H\ \%S \tag{3.63}$$

At any depth in the soil, the percent swell that will occur will fall along the line ABC. For all practical purposes that line can be defined by a straight line connecting point A (the point defined by the percent swell in a consolidation-swell test) and point C (the point corresponding to the constant volume swell pressure, σ'_{cv}). The slope of that line is denoted by the heave index, C_H which is shown as a relationship below (see Eq.(3.23)):

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

If values of C_H and σ'_{cv} are known, the vertical strain, or percent swell, that will occur during inundation at any depth z in a soil profile can be determined from Eq.(3.23). Eq.(3.23) can be re-written as Eq.(3.64), when the soil at depth z is inundated, the stress on the soil is the overburden stress, $(\sigma'_{vo})_z$. This value is therefore used for the inundation stress, σ'_i .in Eq.(3.64).

$$(\varepsilon_v)_z = \%S = C_H \log\left[\frac{\sigma'_{cv}}{(\sigma'_{vo})_z}\right] \tag{3.64}$$

Therefore, heave prediction can be shown as below (Eq.(3.22), see Table 3.4):

$$\Delta H = H\ C_H\ \log\left[\frac{\sigma'_{cv}}{(\sigma'_{vo})_z}\right]$$

where, σ'_{cv} is the swelling pressure from constant volume swell test; σ'_{vo} is the vertical stress at the midpoint of the soil layer for the conditions under which heave is being computed.

Nelson et al. (1998, 2006) indicated that an accurate method to determine C_H would require several consolidation-swell tests at different inundation pressures and a constant volume test. This is neither practical nor economical. Therefore, the relationship between σ'_{cv} and σ'_f was proposed so that the value of the heave index, C_H can be determined from a single consolidation-swell test (Nelson et al. 2006). A relationship between σ'_{cv} and σ'_{cs} exists that is of the form:

$$\sigma'_{cv} = \sigma'_i + \lambda\left(\sigma'_{cs} - \sigma'_i\right) \tag{3.65}$$

This method for determination of the heave index is considered to be practical and rational but the actual value of λ to be used for different soils should be investigated on a case by case basis. Generally, a value of λ in the range of 0.5 to 0.7 may be reasonable.

3.5 Summary

Early research studies during the period from 1950's thru 1900's were based on limited studies that focused on developing empirical relationships but were found to be not universally valid but only useful for explaining the local expansive soil behavior. Aitchison (1973) was the earliest investigator who introduced the soil suction models introducing the concept of instability index (i.e., I_{pt}) for calculating moisture-related ground movements. This instability index, I_{pt} has been widely used by geotechnical researchers (Mitchell 1989, Fityus and Smith 1998, Jaksa et al. 2009).

Several other techniques have been proposed using oedometer test methods or soil suction methods for estimating the surface heave in expansive soil with lightly loaded structures (see Table 3.4). Some techniques are based on stress state variables approach (i.e., Snethen (1980) method, Fredlund (1983) method) and others do not use the stress state variables approach (i.e., Hamberg and Nelson (1984) method, Nelson and Miller (1992) method). However, in all these methods various parameters are required to estimate the 1-D heave from time consuming laboratory and/or in-situ tests. Most of the tests are expensive and difficult to be performed by conventional geotechnical engineers. There is a need to propose a 1-D heave prediction method that is much simpler, inexpensive to use that can be used universally on expansive soils from all regions of the region. To address this objective, a simple technique is proposed to estimate the 1-D heave in expansive soils which requires only the information of plasticity index, I_p and variation in water content with respect to depth in the active zone in the next chapter.

CHAPTER 4

A SIMPLE TECHNIQUE FOR ESTIMATING THE *1-D* HEAVE IN EXPANSIVE SOILS

4.1 Introduction

Several techniques available in the literature for estimating the 1-D heave in expansive soils with lightly loaded structures were discussed in Chapter 3. However, there are limitations in using these techniques because of one or more of the following reasons:

i) these techniques are not universally valid as they are proposed using only limited soils data collected locally;

ii) they do not use the stress state variables approach that provides a rational basis for interpretation;

iii) the various parameters required in these approaches can only be obtained from time consuming laboratory or in-situ tests that are expensive and difficult to be performed.

In this Chapter, a simple technique is proposed to estimate the 1-D heave in expansive soils which requires only the information of plasticity index, I_p and variation in water content with respect to depth in the active zone. The proposed simple technique is based on the approaches suggested by Fredlund (1983) and Hamberg and Nelson (1984). The proposed technique was tested on a case study results summarized by Yoshida et al. (1983) to check the validity. There was good agreement between the measured and the predicted 1- D heave for the case study using the proposed technique.

4.2 Background

There are several procedures used in geotechnical engineering practice to estimate the swelling pressure and the 1-D heave in expansive soils. These techniques can be divided into three main categories based on: (i) empirical methods; (ii) oedometer test methods; and (iii) soil suction methods.

Empirical methods use soil classification parameters such as Atterberg limits (Snethen et al. 1977), plasticity index and percent clay (Nayak and Christensen 1971), and percentage swell (Yoder and Witczak 1975). These methods are developed based on limited data collected locally.

The oedometer test methods for estimating 1-D heave in expansive soils use swelling pressure as a tool (Fredlund and Rahardjo 1993). Depending on the loading procedures, several methods are developed such as the free swell tests, the overburden swell tests; and the constant volume swell (*CVS*) tests using conventional oedometer test methods.

The soil suction methods are based on the information of the stress state (i.e. suction) and can be considered to be more reliable for measuring the swelling pressure and calculating the 1-D heave in expansive soils. In these methods, the influence of suction is taken into account through the use of different parameters such as suction modulus ratio, C_w (Hamberg 1985), suction index, C_ψ (Johnson and Snethen 1978), instability index, I_{pt} (Mitchell adn Avalle 1984), swelling index, C_s and corrected pressure, P'_s (Fredlund 1983, Fredlund and Rahardjo, 1993), or suction compression index, C_h (McKeen 1992).

Fredlund (1983) proposed an equation that can be used to calculate 1-D heave in expansive soils using the constant volume swell (*CVS*) oedometer test results. The two key parameters required in this technique are swelling index, C_s and initial stress state, P_0. A realistic estimation of initial stress state, P_0 can be obtained based only on the measurement of several indices taking account of the changes in void ratio due to both the net normal stress, $(\sigma - u_a)$ and matric suction, $(u_a - u_w)$. For the purpose of simplification, the initial stress state, P_0 may be assumed to be equal to the corrected swelling pressure, P'_s. However, the determination of the corrected swelling pressure, P'_s from laboratory test is still elaborate and time consuming.

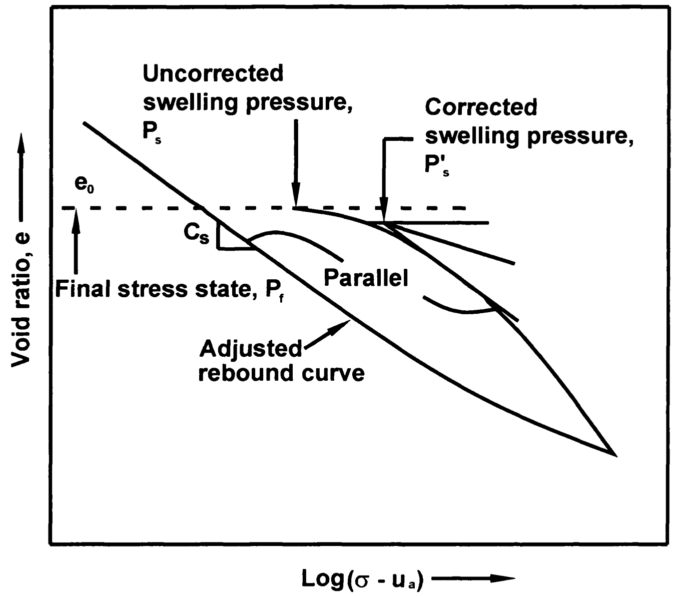
Hamberg and Nelson (1984) presented a different approach that can be used to predict 1-D heave in expansive soils using the relationship between water content and volume change in the range of shrinkage limit to liquid limit. This method is relatively simpler compared to Fredlund (1983) method; however, the influence of the in-situ stress state is not taken into account in this approach.

A simple technique is proposed in this Chapter to estimate the 1-D heave in expansive soils for lightly loaded structures. The proposed technique is developed by combining both the Fredlund (1983) and Hamberg and Nelson (1984) approaches to derive a new relationship. Some of the limitations associated with both the approaches are alleviated in the proposed simple technique.

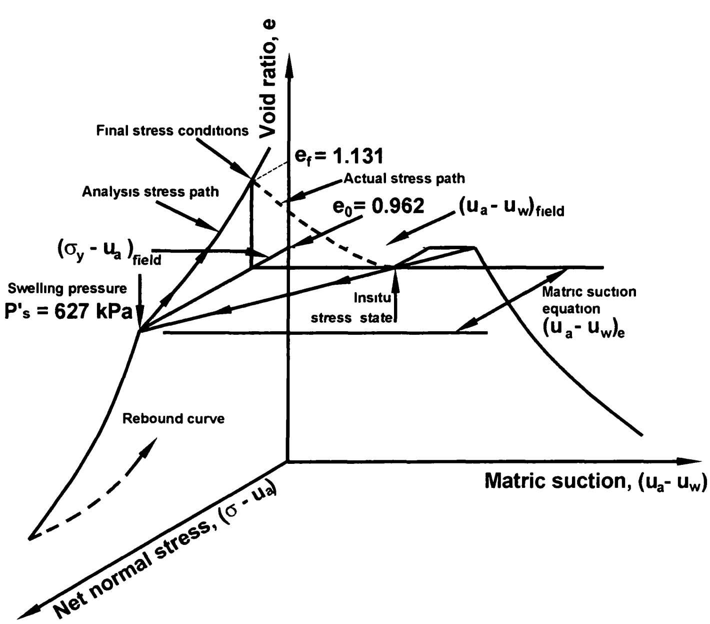
4.3 1-D Heave Determination Methods

4.3.1 Fredlund (1983) method

Fredlund (1983) suggested that matric suction in an expansive soil specimen can be eliminated by immersing it in water. As a result, total stress changes in the specimen mainly reflect as the swelling behavior of the expansive soil. The *CVS* oedometer method can be used as a tool to simulate this behavior in a laboratory environment (ASTM D4546-2000). In this test, the specimen in the oedometer is inundated after the application of a token overburden pressure. The overburden pressure is increased restricting the swelling as the specimen tends to swell. The pressure at which no more swelling occurs is called the "uncorrected" swelling pressure. The specimen is then loaded and unloaded following the conventional consolidation testing procedures to estimate the 'corrected swelling pressure' as shown in Figure 4.1(a) and (b).



(a)



(b)

Figure 4.1 (a) Construction procedure to determine the corrected swelling pressure incorporating the effect of sampling disturbance (modified from Fredlund 1987) (b) Stress paths representing swelling of Regina soil (modified from Fredlund 1983).

The procedure suggested by Fredlund (1987) to estimate the corrected swelling pressure involves the determination of compressibility of the apparatus from the actual deformation measurements and also takes account of sampling disturbance extending the modified graphical method (Casagrande 1936). This procedure is based on the assumption that the in-situ soil suction is transferred onto the effective stress plane.

The 1-D heave in expansive soils can be more reliably determined using Eq.(4.1) which incorporates the corrected swelling pressure, P'_s.

$$H = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} C_s \frac{H_i}{1+e_0} \log\left\{\frac{P_f}{P'_s}\right\} \tag{4.1}$$

where, H_i = thickness of the i_{th} layer, P_f ($= \sigma_y + \Delta\sigma_y - u_{wf}$) = final stress state, P'_s = corrected swelling pressure, C_s = swelling index, σ_y = total overburden pressure, $\Delta\sigma_y$ = change in total stress, u_{wf} = final pore-water pressure; and e_0 = initial void ratio.

More details with respect to the determination of the corrected swelling pressure P'_s and the 1-D heave calculations were discussed in Chapter 3 and are also available in Yoshida et al. (1983) and Fredlund and Rahardjo (1993).

4.3.2 Hamberg and Nelson (1984) method

Hamberg and Nelson (1984) proposed a method to determine the 1-D heave in expansive soils using suction modulus ratio, C_w as given below.

$$H = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} C_w \frac{H_i}{1+e_0} \Delta w \tag{4.2}$$

where H_i = thickness of the i_{th} layer, Δw = change in water content; and e_0 = initial void ratio.

The suction modulus ratio, C_w in Eq.(4.2) represents the variation of void ratio (i.e. volume in 1-D heave) of soil specimens with respect to water content (Eq.(4.3); Figure 4.2).

$$C_w = \frac{(e_f - e_0)}{\Delta w} \tag{4.3}$$

where e_0 and e_f = initial and final void ratio of soils, respectively; and Δw = change in water content.

The parameter, C_w can be obtained using the Clod test which is the modified form of COLE (coefficient of linear extensibility; Brasher et al. 1966) test. The COLE test is originally developed to determine the heave beneath airfield pavements (McKeen 1981, McKeen and Hamberg 1981). For silty clay soils, the void ratio versus water content relationship shows linear behavior for the water contents greater than the shrinkage limit (Hamberg 1985; Figure 4.2).

The limitations of Eq.(4.2) are:

i) the equation does not take into account the effect of applied load, and

ii) there are difficulties in estimating the C_w values for the water contents close to shrinkage limit due to the nonlinearity in that range.

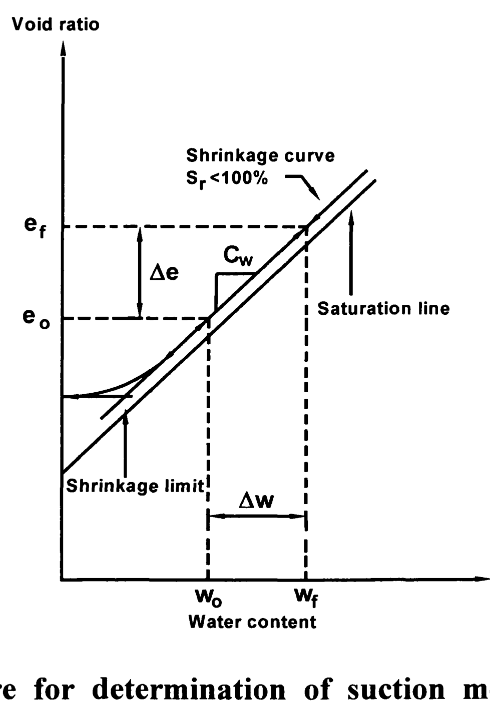


Figure 4.2 Procedure for determination of suction modulus ratio, C_w from water content versus void ratio relationship (modified after Hamberg 1985).

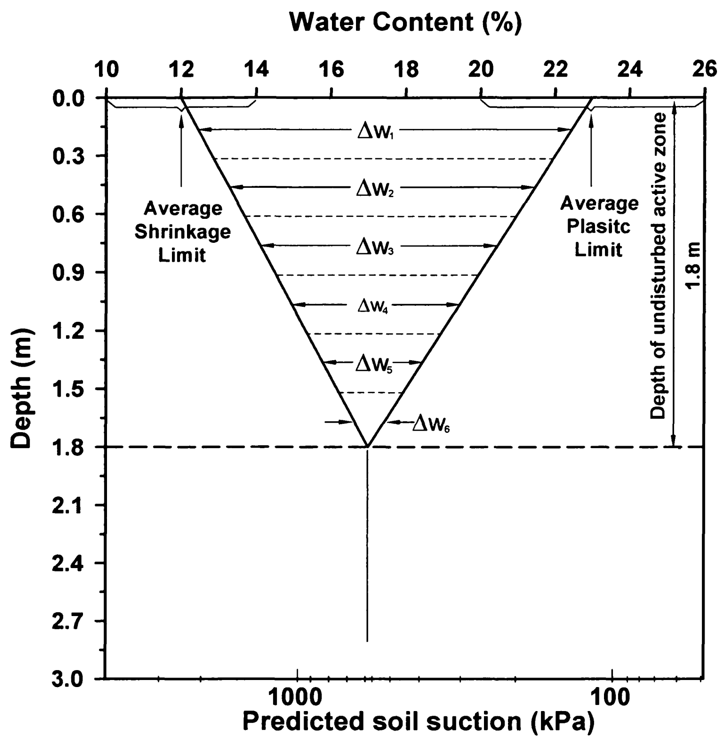


Figure 4.3 Idealized moisture boundary profile (Hamberg 1985).

However, experience from several case studies results from the literature show that the typical variation of water content in field conditions is in the range which is above the shrinkage limit to plastic limit or slightly higher. Figure 4.3 shows the idealized moisture boundary profile for the Pierre shale, Fort Collins, Colorado (Hamberg 1985). As it can be seen, the possible variation of water content in field conditions can be in the range of shrinkage limit to plastic limit.

4.3.3 The proposed technique

The equation proposed by Fredlund (1983) (i.e. Eq.(4.1) to determine the 1-D heave in expansive soils can be rewritten as below.

$$\Delta H = C_s \frac{H_i}{1+e_0} \log\left\{\frac{P_f}{P_s'}\right\} = C_s \frac{H_i}{1+e_0} \log P_f - C_s \frac{H_i}{1+e_0} \log P_s' \qquad (4.4)$$

The positive (i.e. first part) and negative (i.e. second part) of the equation (i.e. Eq.(4.4)) represent compression and heave due to overburden and swelling pressure, respectively. In other words, the amount of heave calculated using the second part of Eq.(4.4) and Eq.(4.2) can be written as Eq.(4.5).

$$C_s \frac{H_t}{1+e_0} \log(P_s') \propto C_w \frac{H_t}{1+e_0} \Delta w \tag{4.5}$$

The above equation can be further simplified as

$$\log P_s' \propto \frac{C_w}{C_s} \Delta w \tag{4.6}$$

A correction parameter, K can be introduced to rewrite the relationship shown in Eq.(4.7) as below:

$$\log(K \cdot P_s') = \frac{C_w}{C_s} \Delta w \tag{4.7}$$

An expression in terms of swelling pressure can be derived from Eq.(4.7).

$$P_s' = \frac{10^{\left(\frac{C_w}{C_s}\Delta w\right)}}{K} \tag{4.8}$$

Substituting Eq.(4.8) into Eq.(4.4) yields

$$\Delta H = C_s \frac{H_t}{1+e_0} \log P_f - C_s \frac{H_t}{(1+e_0)} \log \frac{10^{(\frac{C_w}{C_s}\Delta w)}}{K} \tag{4.9}$$

In a simplified form, Eq.(4.9) can be written as below:

$$\Delta H = C_s \frac{H_t}{1+e_0} \log \frac{KP_f}{10^{(\frac{C_w}{C_s}\Delta w)}} \tag{4.10}$$

The derived Eq.(4.10) can be used for estimating the 1-D heave in natural expansive soils. The required information includes the swelling index, C_s, the suction modulus ratio, C_w and the correction parameter, K. In the following section, empirical relationships that can be used to obtain the three parameters (i.e., C_w, C_s and K) are provided.

4.4 Simple Relationships for Estimating the Parameters C_w, C_s and K

Relationships between C_w versus I_P and C_s versus I_P are developed using the published data on natural expansive soils from various regions of the world.

The C_w versus I_P relationship in Figure 4.4 shows that C_w value can be approximated as a constant value of 0.024 for the I_P values greater than 30 % (Eq.(4.11)). Expansive soils typically have I_P values greater than 30%.

$$C_w = 0.024 \text{ (for } I_P \geq 30) \tag{4.11}$$

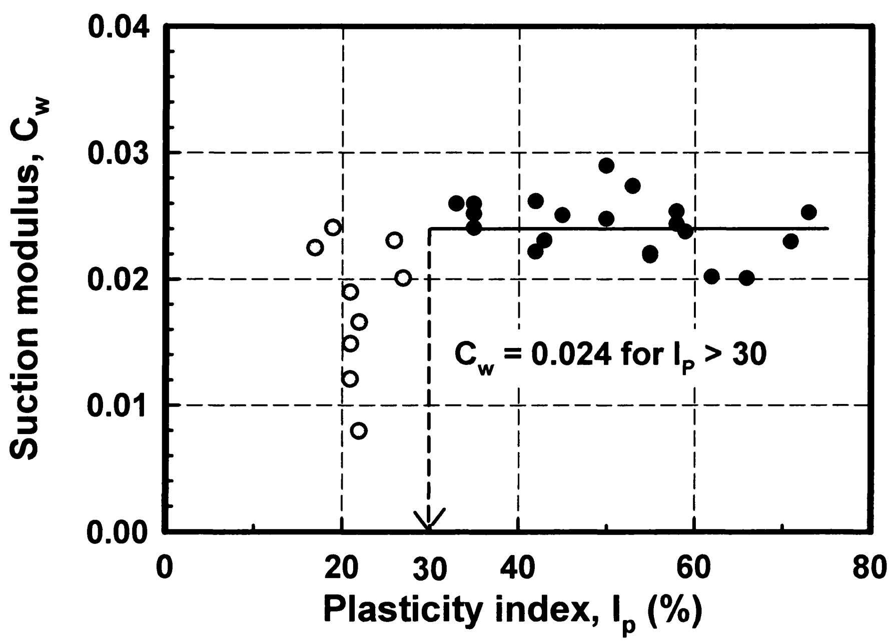


Figure 4.4 Values of plasticity index, I_P and suction modulus ratio, C_w from several laboratory tests results (Data for generating this relationship is collected from several publications. These publications are summarized in the reference section).

The relationship between the swelling index, C_s and the plasticity index, I_P using the published results from the literature is shown in Figure 4.5. The swelling index, C_s values were determined as per ASTM D4546-2000 using undisturbed samples collected from the zone of depth of zero to 2.5m of the sites, which is typically the active zone in several regions of the world. In addition, the equipment compressibility and sampling disturbance correction was applied following the procedure suggested by Fredlund (1987) for determining the C_s values. The relationship shows that the swelling index, C_s value increases exponentially with increasing I_P (Eq.(4.12)).

$$C_s = 0.0193e^{0.0343(I_P)} \tag{4.12}$$

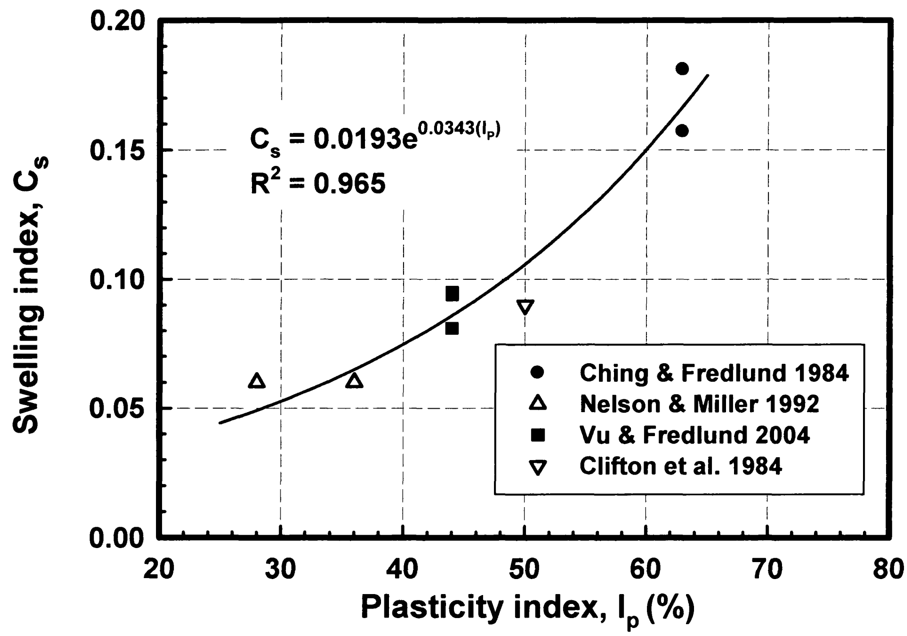


Figure 4.5 Relationship between the plasticity index, I_P and corrected swelling index, C_s from a several laboratory tests results.

The relationship between the correction parameter, K and the water content change, Δw using the data from five case study results is shown in Figure 4.6. These case studies are from different regions of the world that include Canada, Sudan and U.S.A. (i.e., Fredlund 1969, Hamberg and Nelson 1984, Osman and Sharief 1987, Snethen and Huang 1992). Four of the above five case studies do not have the information of the swelling index, C_s (excluding Fredlund 1969). Therefore, the swelling index, C_s values were estimated using Eq.(4.12) based on their plasticity index, I_P values.

The best-fitting equations shown in Figure 4.6 indicate that the correction parameter, K versus Δw relationships can be expressed as Eq.(4.13).

$$K = \omega e^{\theta(\Delta w)} \tag{4.13}$$

In addition, the best-fitting equation for all the five case studies (see Figure 4.6) show that the factor, θ is independent of the type of soil. In other words, θ value may be assumed to be constant with a value of 0.64 for various natural expansive soils with plasticity index, I_P values ranging 25 to 45%. Therefore, Eq.(4.13) can be rewritten as below.

$$K_I = \omega e^{0.64(\Delta w)} \tag{4.14}$$

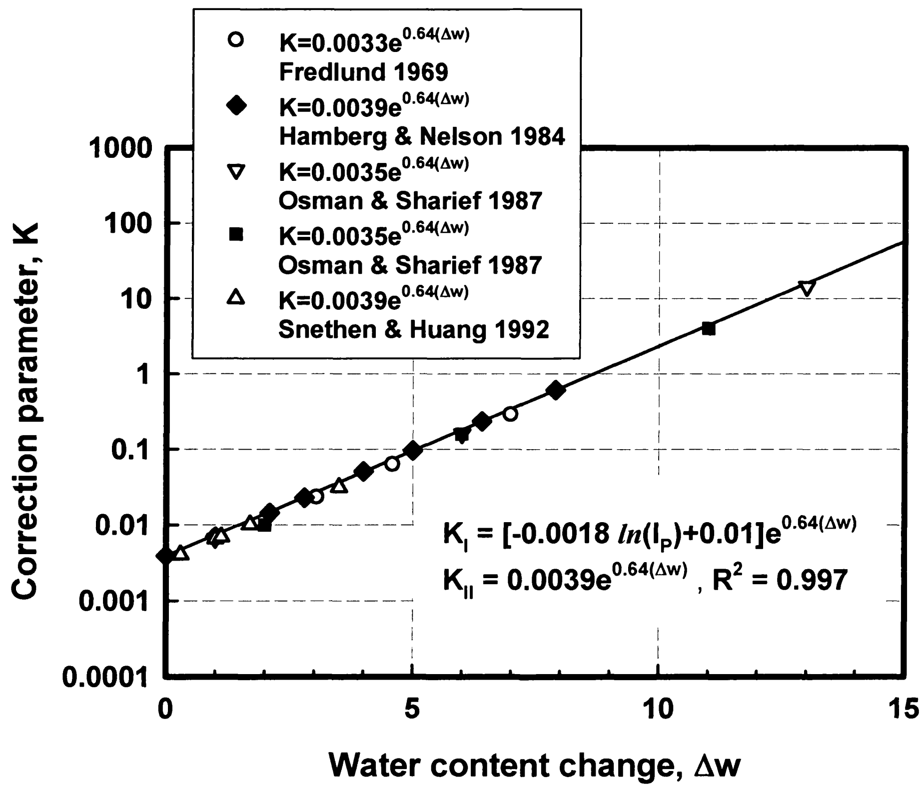


Figure 4.6 Relationship between correction parameter, *K* and water content change, *Δw*.

Figure 4.7 shows the variation of the factor, ω with the plasticity index, I_P of the soils. The factor, ω increases nonlinearly with increasing I_P; the factor, ω can be expressed as a function of I_P as shown in Eq.(4.15).

$$\omega = -0.0018 ln(I_P) + 0.01 \tag{4.15}$$

Equation (4.14) can be rewritten as Eq.(4.16) (hereafter referred to as K_I).

$$K_I = [-0.0018\,ln(I_P) + 0.01]e^{0.64(\Delta w)} \tag{4.16}$$

The representative best-fitting equation for all the five case studies data is shown in Eq.(4.17) (hereafter referred to as K_{II}).

$$K_{II} = 0.0039e^{0.64(\Delta w)} \tag{4.17}$$

In the present study, both K_I and K_{II} were used to estimate the 1-D heave of a case study.

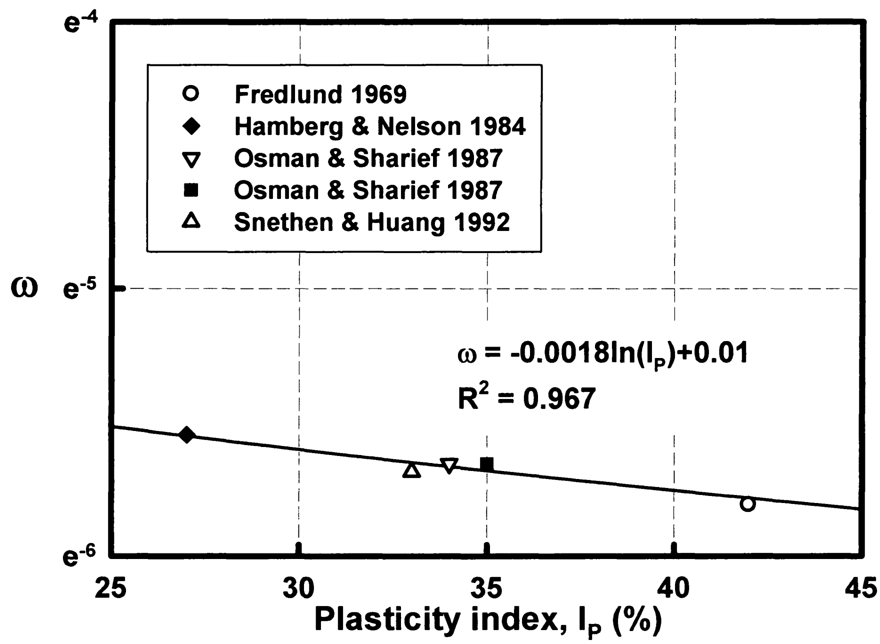


Figure 4.7 Relationship between ω and plasticity index, I_P.

4.5 Analysis of the Proposed Technique Using a Case Study Results

The validity of the proposed technique is tested using a case study results published by Yoshida et al. (1983). The average liquid limit, ω_L and plastic limit, ω_P of the expansive soil (i.e., Regina clay) are 77% and 33%, respectively with an average natural water content of 29%. Comparisons are provided between the measured heave and the estimated heave using the proposed method (using K_I and K_{II}) and the methods proposed by Fredlund (1983) and Hamberg and Nelson (1984).

4.5.1 Description of the site

The site used in the present study is located in Saskatchewan, Canada (Yoshida et al. 1983), which has lightly loaded industrial building constructed on it. The undisturbed specimens used for the laboratory tests for measuring the corrected C_s values were obtained from the active zone (i.e. 0 to 2.3 m).

The plasticity index, I_P of the specimen from the clay in the active zone of 0 to 2.3m depth is 43%. Therefore, the value of ω is estimated as 0.0032 using Eq.(4.15). The average value of the initial void ratio, e_0 for each layer is 0.962 (Yoshida et al. 1983). The suction modulus ratio, C_w and the swelling index, C_s can be estimated to be 0.024 and 0.084 using Eqs (4.11) (Figure 4.4) and (4.12) (Figure 4.5), respectively.

The distribution of initial and final water content with depth along with the saturation water content (i.e. 100% degree of saturation) and the estimated initial soil water characteristic curve of Regina soil for the site are shown in Figure 4.8 and Figure 4.9, respectively. The information of water content change, Δw is derived from Figure 4.8 to calculate the correction parameter, K (i.e. K_I (Eq.(4.16)) and K_{II} (Eq.(4.17)).

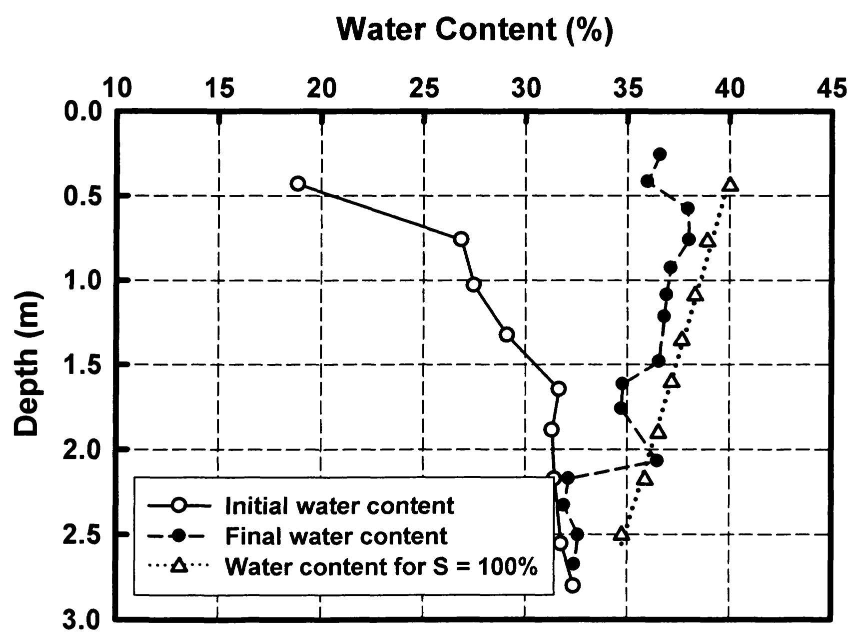


Figure 4.8 Variation of in situ water content with respect to the depth. (Yoshida et al. 1983)

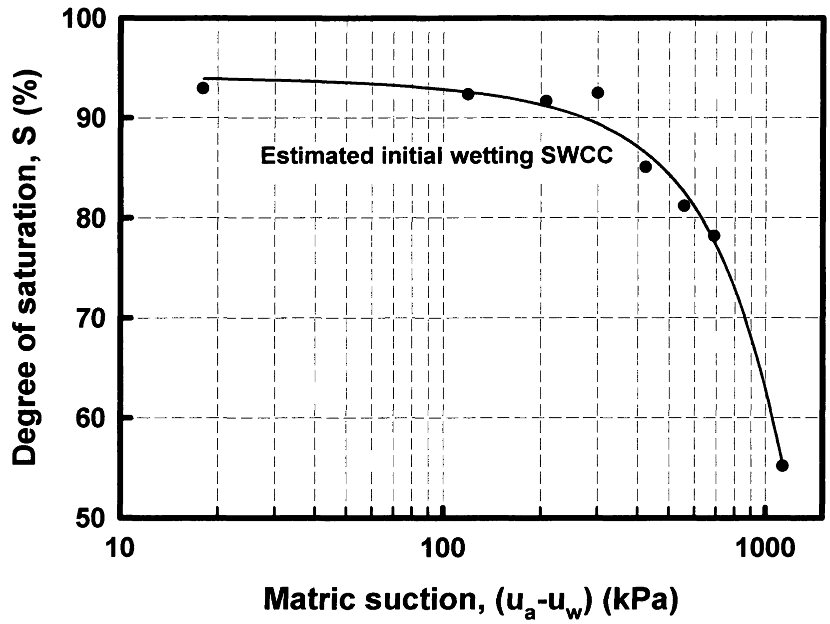


Figure 4.9 Estimated initial wetting soil water characteristic curve of Regina soil. (Yoshida et al. 1983)

4.5.2 Comparison between the measured and the estimated heaves

The distribution of K (K_I) values with depth has been showed in Figure 4.10. Table 4. 1 and Figure 4.11 show the comparison between the measured and the estimated 1-D heaves using three different methods (i.e. Fredlund 1983, Hamberg and Nelson 1984 and the proposed method using K_I and K_{II}). The ratios of the estimated total heaves to the measured total heaves were estimated as 1.11, 2.13, 1.04 and 0.95 using Fredlund (1983), Hamberg and Nelson (1984) and the proposed method (using K_I and K_{II}), respectively.

The results of the analysis show that Hamberg and Nelson (1984) method overestimates the 1-D heave of the case study. This may be attributed to ignoring the effect of overburden pressure as explained in Section 4.3. On the other hand, the calculated heaves using Fredlund (1983) and the proposed technique in the present study show good agreement with the measured total heaves.

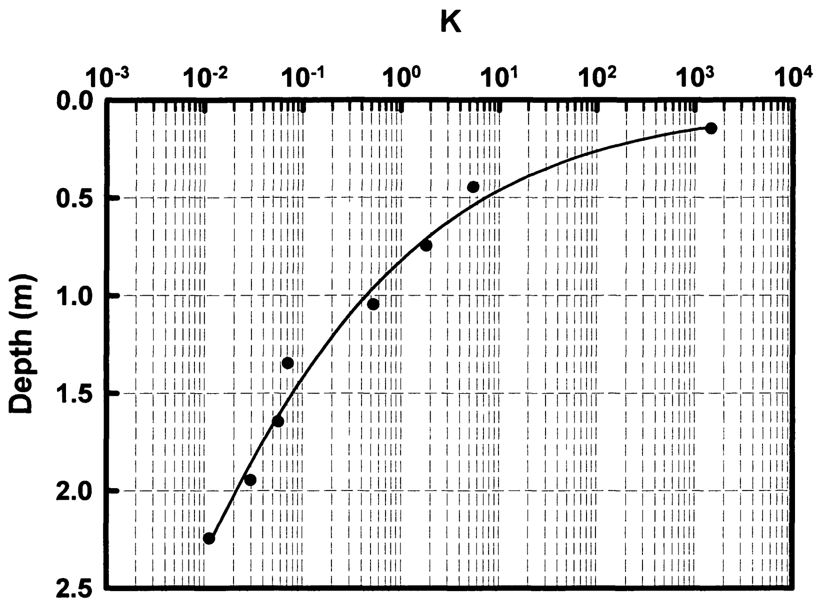


Figure 4.10 Distribution of K values with depth.

Table 4. 1 Summary of the case study data and the comparison between the measured and the estimated heave using different methods.

Depth (m)	Fredlund (1983) (mm)	Hamberg & Nelson (1984) (mm)	Proposed (K_I) mm	Proposed (K_{II}) mm
0.15	26	39	11	11
0.45	22	46	19	18
0.75	19	40	16	15
1.05	16	33	15	14
1.35	14	21	14	12
1.65	11	20	12	11
1.95	8	16	12	10
2.25	2	11	11	10
Total	118	226	110	101
EM/MH*	1.11	2.13	1.04	0.95

*Estimated heave (EH)/ Measured heave (MH); Measured heave (MH) = 106 mm

The estimated heave is close to the measured heave using the parameter, K_I in comparison to K_{II}. In other words, the total heave can be more reliably estimated when the plasticity index, I_P values of the expansive soil at the site. However, heave comparisons were reasonable using the parameter, K_{II} (i.e. $\omega = 0.0039$).

The technique proposed in this Chapter is simple and needs only the information of plasticity index, I_P and the variation of water content with respect to depth in the active zone, whereas the parameters required for Fredlund (1983) method are elaborate and hence time consuming and expensive.

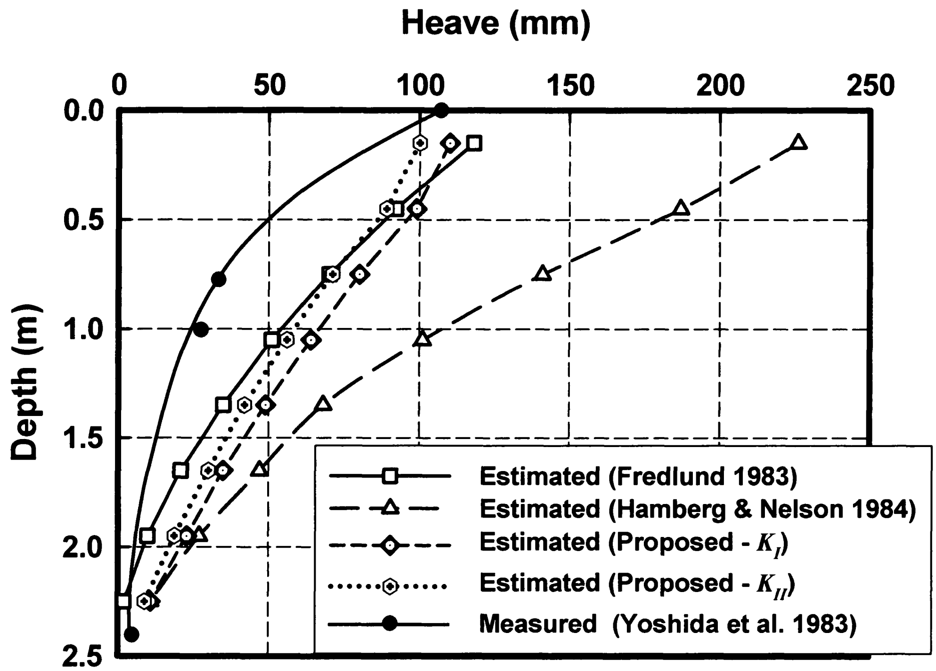


Figure 4.11 Measured and estimated heave using different methods.

4.6 Summary and Conclusion

Several methods available in the literature for estimating the 1-D heave were discussed in Chapter 3. Of the various methods available, the 1-D heave in natural expansive soils with lightly loaded structures is commonly determined using the methods proposed by Fredlund (1983) or Hamberg and Nelson (1984). However, determination of the parameters required for using Fredlund (1983) method is expensive and time-consuming. The Hamberg and Nelson (1984) method is relatively easier but it does not take into account of the effect of overburden pressure and hence may over estimate the heave. A simple technique is proposed in this Chapter to estimate the 1-D heave in natural expansive soils by deriving a new relationship from Fredlund (1983) and Hamberg and Nelson (1984) methods alleviating some of the limitations of both these methods. The proposed technique was tested on case study results of Regina clay (Yoshida et al. 1983). The results of the study presented in this paper show that the proposed method can be used reliably in estimating the 1-D heave. The proposed technique is simple and needs only the information of plasticity index, I_p and the variation of natural water content with respect to depth in the active zone of natural expansive soils.

CHAPTER 5

CASE STUDIES ANALYSIS

5.1 Introduction

A simple technique was proposed in Chapter 4 to estimate the 1-D heave in natural expansive soils by deriving a new relationship from Fredlund (1983) and Hamberg and Nelson (1984) methods. The proposed equation for estimating the 1-D heave requires three parameters; namely, the corrected swelling index, C_s the suction modulus ratio, C_w and a parameter, K which is a function of water content and plasticity index, I_P. The parameters C_s and C_w are well known to the conventional geotechnical engineers, which can be determined from laboratory tests. However, these tests are time consuming and need extensive laboratory testing methods and hence expensive. Therefore, empirical equations were proposed between the corrected swelling index, C_s versus the plasticity index, I_P and suction modulus, C_w versus plasticity index I_P using the published data on natural expansive soils from various regions of the world. The parameter, K is related to the water content change, Δw with respect to depth in the active zone. A relationship has been proposed between the correction parameter, K and the water content change, Δw using five case study results published in the literature (i.e., Fredlund 1969, Hamberg and Nelson 1984, Osman and Sharief 1987, and Snethen and Huang 1992). In addition, the correction parameter K, was expressed in two different forms; namely, K_I method (which is a function of plasticity index, I_p) and K_{II} method (which is an empirical relationship derived from the results of the five case studies data).

In Chapter 4, the proposed technique was tested on a case study of Regina clay (i.e., Yoshida et al. 1983). Reasonably good comparisons were obtained between the measured and the estimated 1-D heaves using three different methods (i.e., Fredlund 1983, and using two different methods of the proposed technique (i.e., K_I method and K_{II}

method). The ratios of the estimated total heaves to the measured total heaves were equal to 1.11, 1.04 and 0.95 using Fredlund (1983) and the proposed technique (using K_I and K_{II} methods), respectively.

In this chapter, 13 case studies results from different regions of the world which include Australia, Canada, Chile, Saudi Arabia, Sudan, United States, and Yugoslavia published in the literature are summarized and the proposed technique was used for estimating the 1-D heave of expansive soils. Also, comparisons are provided between the measured and estimated 1-D heave using the proposed method. There is a good comparison between the measured and estimated 1-D heave for 8 of the 13 case studies. The differences in the measured and estimated heave values for these case studies were less than 30%, which is reasonable. The estimate heave for all these case studies was greater than the measured heave. Therefore, the proposed technique is conservative from an engineering practice point of view. Reasons associated with the differences between the measured and estimated heave for the remainder of the five case studies are discussed.

5.2 Review of the Proposed Technique for Estimating 1-D Heave

The Fredlund (1983) method uses the changes in void ratios corresponding to the initial and final stress states and the corrected swelling index C_s in the calculation of the total 1-D heave in expansive soils. The following equations can be used for the calculation of total heave:

$$e = \frac{G_S \gamma_w}{\gamma_d} - 1 \tag{5.1}$$

$$\Delta e = e_f - e_0 = -C_s \log \frac{P_f}{P_i} \tag{5.2}$$

$$\Delta H = H\frac{\Delta e}{1+e_0} \tag{5.3}$$

The change in void ratio (Eq.(5.2)) can be rewritten by incorporating the soil properties and the stress states to derive the Fredlund (1983) method as given:

$$\Delta H = C_s\frac{H}{1+e_0}\log\left\{\frac{P_f}{P'_s}\right\} \tag{5.4}$$

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

Hamberg and Nelson (1984) proposed a method to determine the 1-D heave in expansive soils using suction modulus ratio, C_w as given below:

$$\Delta H = C_w\frac{H}{1+e_0}\Delta w \tag{5.5}$$

$$C_w = \frac{\left(e_f - e_0\right)}{\Delta w} = \Delta e / \Delta w \tag{5.6}$$

A simple technique was proposed by deriving a relationship from Fredlund (1983) and Hamberg and Nelson (1984) methods as given below:

$$\Delta H = C_s\frac{H_i}{1+e_0}\log\frac{KP_f}{10^{(\frac{C_w}{C_s}\Delta w)}} \tag{5.7}$$

The equation requires values of the final stress state P_f, water content change Δw, suction modulus ratio C_w and the corrected swelling index, C_s. Chapter 4 provides the details of the derivation for Eq. (5.6).

In expansive soils, the initial water content and the water content change control the amount of swell in expansive soils (Chen, 1975). The information about the change in water content Δw, can be obtained from the field investigation studies. Some of the case studies results published in the literature were analyzed using heave prediction techniques that do not require the field water content information (i.e., Snethen 1980, Fredlund 1983). Due to this reason, the change in water content distribution with respect to depth was not available. However, the water content change was calculated using Eq.(5.8) (Fredlund and Rahardjo 1993).

$$\Delta w = S_f \ \Delta e / G_s + e_0 \ \Delta S / G_s \tag{5.8}$$

And, the final water content can be obtained from Eq.(5.9)

$$w_f = w_0 + \Delta w \tag{5.9}$$

where, G_s = specific gravity, S_f = final assumed degree of saturation, S_i = initial degree of saturation, ΔS = change in degree of saturation, Δw = water content change. The estimated 1-D heave will be maximum possible value if the final assumed degree of saturation S_f = 100%.

The suction modulus ratio, C_w can be measured from laboratory Clod tests; for silty clay soils, the void ratio versus water content relationship shows linear behavior for the water contents greater than the shrinkage limit (Hamberg 1985). Other clayey and expansive soils also show similar behavior (Tripathy et al. 2002). However, a value of 0.024 can be used for soils with plasticity index, I_P values greater than 30 (Eq.(5.10)). More discussions about the C_w values were discussed in Chapter 4. Typically expansive soils have I_p values greater than 30%.

$$C_w = 0.024 \quad I_p \geq 30\% \tag{5.10}$$

The corrected swelling index, C_s value can be measured using the procedures detailed by Fredlund (1983) from 1-D Constant Volume Swell (*CVS*) oedometer test. The determination of the swelling index, C_s value is time consuming and requires elaborate testing equipment and also needs different corrections that include swelling pressure, compressibility of the apparatus, and sampling disturbance. A simple empirical relationship shown in Eq. (5.11) was proposed for estimating the corrected swelling index, C_s from the plasticity index, I_P value (Eq.(5.11)) alleviating the time consuming and extensive testing techniques.

$$C_s = 0.0193e^{0.0343(I_P)} \tag{5.11}$$

Two empirical relationships have been proposed for estimation of correction parameter, K using five case study results (see Chapter 4 for more details). The first one is the K_I method (Eq.(5.12)) and the second one is the K_{II} method (Eq.(5.13)); The parameter K_I varies with the changes of plasticity index, I_p and water content change, Δw; however K_{II} is a relationship which is a function of Δw.

$$K_I = \left[-0.0018\,ln(I_P) + 0.01\right]e^{0.64(\Delta w)} \tag{5.12}$$

$$K_{II} = 0.0039e^{0.64(\Delta w)} \tag{5.13}$$

The proposed technique needs only the information of dry unit weight, γ_d, plasticity index, I_p and the variation of natural water content with respect to the depth in the active zone of natural expansive soils. These soil properties and can be easily obtained from conventional geotechnical tests.

5.3 Summary of Five Case Studies Measured and Estimated 1-D Heave Results

Relationships for estimation of the corrected swelling index, C_s and the parameters

K_I and K_{II} for using the proposed technique were developed from the soil properties of the five case studies. These relationships were discussed in greater detail in Chapter 4 and are briefly summarized in this Chapter. These five case studies results are summarized and comparisons are provided between the measured heave and estimated 1-D heave using the proposed technique in this Chapter.

5.3.1 Case Study A (Fredlund 1969)

Hamilton (1965, 1968) studied the swelling behavior of expansive soil beneath an industrial building in Regina, Saskatchewan. The swelling in the building was due to the flooding associated with a break in the water line which occurred during the summer of 1962. Several investigations were undertaken to study the swelling of the building slab floor. These investigations included conducting 1-D consolidation and swelling tests on samples collected from three different depths below the concrete slab floor of the industrial building. In addition, ground movement gauges were also installed at three depths and precise elevation readings were measured over a period of time. The investigations studies were summarized in Fredlund (1969). The measured 1-D heave of the industrial building was equal to 84 mm.

The heave calculated using Eq. (5.3) was equal to 151mm (Fredlund 1969). The values required for performing the calculation include C_s and P'_s which were measured from *CVS* tests. The ratio between the calculated heave and the measured heave was equal to 1.79.

The initial water content at the site for three different depths was available. However, the final water content was not available. The water content change is calculated by using Eq.(5.2) and Eq.(5.8). The final degree of saturation, S_f was assumed to be equal to 1. The initial and final water content distribution with respect to depth is shown in Figure 5.1.

The 1-D heave for this case study was estimated using the proposed technique, following the two methods (i.e., K_I and K_{II}). The average plasticity index, I_P of the

Regina soil in the active zone depth was equal to 42%. The suction modulus ratio, C_w and the corrected swelling index, C_s were estimated to be equal to 0.024 and 0.094 by using Eq.(5.10) and Eq.(5.11), respectively. The estimated heave values using the K_I and K_{II} methods were respectively 110mm and 103mm respectively. The ratios of estimated heave to the measured heaves using these two methods were equal to 1.32 and 1.23 respectively.

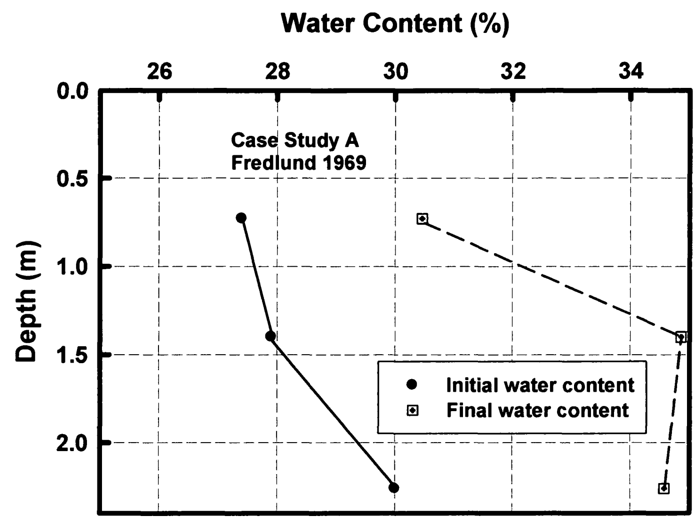


Figure 5.1 Water content variation with respect to depth for the Case study A (modified after Fredlund 1969)

Table 5. 1 Summary of the measured and estimated values for Case study A

Depth (m)	Δw %	e_0	Measured P'_s(kPa)	Measured C_s	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.73	3.10	0.859	442	0.094	0.0032	0.0046			
1.40	6.99	0.983	335	0.085	0.0029	0.0039	110	103	84
2.26	4.58	0.975	81	0.096	0.0038	0.0053			
Ratio							1.27	1.17	-
Fredlund (1969) *CVS* test method:									151 mm

For this case study, 1-D heave were also estimated from the K_I and K_{II} methods using the measured C_s (i.e., 0.094). The estimated heave value with the proposed technique using the K_I and K_{II} methods were 107mm and 98mm respectively. The ratios of the estimated heave to measured heave values using these two methods (i.e., K_I and K_{II}

methods) were equal to 1.27 and 1.17.

The reasons associated with the estimated heave values slightly higher than the measured values may be attributed in part to the assumption that the soil is saturated when the heave measurements were made. However, such an assumption may not be valid as the entire active zone depth of the expansive soil may not be in a state of saturated condition. In addition, the differences may also be attributed to the assumed value of C_s.

5.3.2 Case Study B (Hamberg and Nelson 1984)

The field test site located in Fort Collins, Colorado was used by the U.S. Colorado Army Waterways Experiment Station to study 1-D heave behavior of expansive soil. This region has a cool and semi-arid climate. The changes in water content were measured and 1-D heave data were monitored on lightly loaded plastic barriers on grade at the site. Water content measurements were obtained using nuclear moisture gauges. Elevation and water content readings were taken for a period of twenty months (Miller et al. 1995). The initial and final water content variations with respect to depth are shown in Figure 5.2.

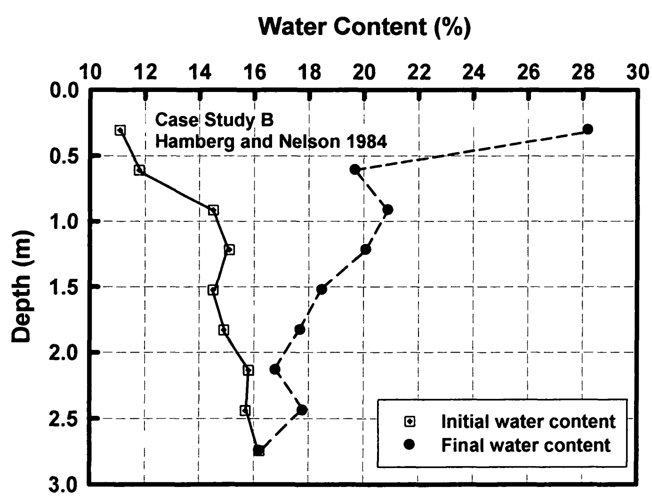


Figure 5.2 Water content variation with respect to depth for the Case study B (Hamberg and Nelson 1984)

The average plasticity index, I_P of the test site with Pierre Shale in the active zone is 28%. The measured suction modulus ratio, C_w from Clod test and the estimated suction modulus ratio, C_w using Eq.(5.10) were 0.016 and 0.024, respectively. The estimated C_s value is equal to 0.05 (Eq.(5.11)). The average dry unit weight is 19.62kN/m^3. Table 5.2 summarizes the measured and the estimated heave using the two methods of the proposed technique (i.e., K_I and K_{II} methods) and the Hamberg and Nelson (1984) method. The ratio of estimated heave to measured heave values using the K_I and K_{II} methods and the Hamberg and Nelson (1984) method were 1.29, 1.2 and 1.6 respectively.

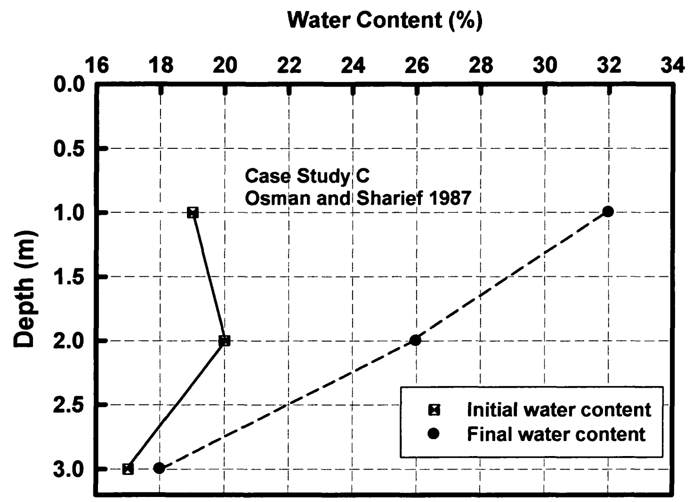
Table 5.2 Summary and comparison between the estimated heave by using different methods (Case study B)

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.305	17.1	1.19	2.4086	2.3471	93	92	72
0.610	7.9	0.98	0.0982	0.0957			
0.915	6.4	0.98	0.0518	0.0505			
1.220	5.0	0.86	0.0273	0.0266			
1.525	4.0	0.78	0.0144	0.0140			
1.830	2.8	0.78	0.0144	0.0140			
2.135	1	0.79	0.0076	0.0074			
2.440	2.1	0.83	0.0144	0.0140			
2.745	0	0.82	0.0040	0.0039			
Ratio					1.29	1.28	-
Hamberg and Nelson (1984) soil suction method:							116mm

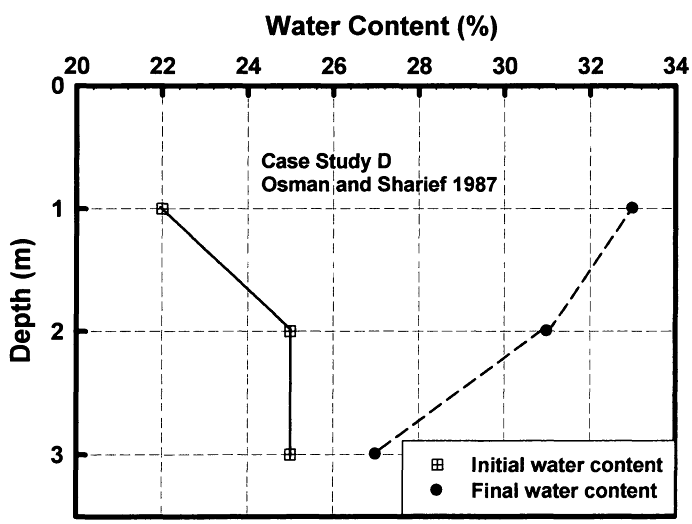
For this case study, 1-D heave were also estimated from the K_I and K_{II} methods using the measured C_w (i.e., 0.016). The estimated heave value with the proposed technique using the K_I and K_{II} methods were 87mm and 88mm respectively. The ratios of the estimated heave to measured heave values using these two methods (i.e., K_I and K_{II} methods) were equal to 1.21 and 1.23. This case study proves that using the measured C_w provides better results than using the estimated C_w.

5.3.3 Case Study C & D (Osman and Sharief 1987)

Osman and Sharief (1987) measured heave values at two different locations on expansive soil deposits in Sudan. The properties of the subsoil conditions were measured by collecting soil samples from boreholes at the two test sites and testing them in the laboratory. Field data collected include soil surface movements and water content distributions with depths after long term flooding of both the sites.



(a)



(b)

Figure 5.3 Water content variation with respect to depth for the Case study a) C, b) D (modified from Osman & Sharief 1987)

The measured heave values at the sites were 142mm and 150mm, respectively. The initial and final water content distributions along with depth are shown in Figure 5.3(a) and Figure 5.3(b), respectively.

The average plasticity index, I_P of the soil samples collected in the active zone for the two different sites studied were 35% and 34%, respectively. The suction modulus ratio, C_w is 0.024 for the two sites using Eq.(5.10). The estimated corrected swelling index, C_s for the two sites (i.e., C and D) were respectively 0.064 and 0.062 using Eq.(5.11), respectively. The total stresses have been provided for both the sites by Osman and Sharief (1987).

Osman and Sharief (1987) used oedometer tests (i.e., swell overburden load test, *CVS* test and free swell test) results to estimate the 1-D heave. The estimated maximum heave by Osman and Sharief (1987) for both the sites are 295mm. Using the K_I and K_{II} methods of the proposed technique, the ratios (i.e., estimated heave to measured heave) for the case study C are 1.08, 1.11; and for the case study D are 1.03, 1.06, respectively.

Table 5.3 Summary and comparison between the estimated heave and measured heave (Case study C)

Depth (m)	Δw %	P_f(kPa)	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
1	13	18	11.2688	16.0101	154	157	142
2	6	38	0.1277	0.1814			
3	1	56	0.0052	0.0074			
Ratio					1.08	1.11	-
Osman and Sharief (1987) oedometer method:							295mm

Table 5.4 Summary and comparison between the estimated heave and measured heave (Case study D)

Depth (m)	Δw %	P_f(kPa)	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
1	11	17.5	3.2258	4.4514	155	159	150
2	6	38	0.1315	0.1814			
3	2	56	0.0107	0.0140			
Ratio					1.03	1.06	-
Osman and Sharief (1987) oedometer method:							295mm

5.3.4 Case Study E (Snethen and Huang 1992)

The case study results summarized in this section were based on the studies from Snethen and Huang (1992) on a test site near Wynnewood, Oklahoma City. The climate in this region is classified as moist sub-humid to dry sub-humid. Undisturbed soil samples were collected from five continuously sampled borings within a 1 m radius to a depth of approximately 4 m. The soil samples were collected to obtain reliable initial natural water conditions. The soil at the site is a tan and reddish brown, moderate to high plasticity clay. The ground water table was observed to be at a depth of 3m from site investigation studies. The measured heave was equal to 180mm.

Snethen and Huang (1992) used soil suction method to estimate 1-D heave. More details about this method have been discussed in Chapter 3 (Table 3.4). The initial soil suction at the test site was estimated using filter paper technique (ASTM D5298, 2003). Snethen (1980) stated that, Russam (1961, 1965), Richards (1976), and Johnson (1976, 1977, 1978) have all prepared the guidelines for estimating final suction profiles. The recommended guidelines can be summarized as four assumptions: (*i*) suction increases linearly with depth in the active zone, (*ii*) final soil suction was assumed based on saturated water content profile; (*iii*) final soil suction is constant at some equilibrium value; and (*iv*) soil suction equal to zero throughout the depth of active zone. Assumption (*i*) requires a hydrostatic relationship with zero soil suction at ground surface and increasing with depth to the actual profile value at the depth of active zone. Assumption (*iii*) requires that the final soil suction file be a constant value through the depth of active zone. Assumption (*iv*) requires that in the final soil suction profile, the soil suction is zero over the entire depth of active zone. These three assumptions (i.e., *(i), (ii)* and *(iv)* are not realistic with respect field behavior observations and are not useful in the prediction or estimation of the 1-D swell. Assumption (*ii*) requires the information of saturated water content to estimate the final suction value; this is probably the most realistic and practical approach for estimating potential heave since it involves measured physical properties of the soils rather than assumed relationships (Snethen 1980).

Therefore, the assumption (*ii*) was used and the estimated heave based on this assumption was equal to 157mm using Snethen and Johnson (1978) method. The ratio between the estimated heave and measured heave was equal to 0.87.

The information related to the initial and final water content were not available in Snethen and Huang (1992); however, the soil suction change, Δu for assumption (*ii*), and the ratio (water content change to soil suction change, $\Delta w/\Delta u$) for each soil layer were available. These details are summarized in Table 5.5. The water content change, Δw were back calculated from this information and summarized (Table 5.5, Column 4).

Table 5.5 Soil properties from Wynnewood site (Snethen and Huang 1992)

Depth (m)	Soil suction change Δu (pF)	(kPa)	Water content change(%) / Soil suction change (*pF*)	Water content change Δw (%)
0.5	4.29	1899	2.8	12
1	4.24	1716	4.44	19.65
1.5	4.63	4159	3.66	16.94
2	4.78	5941	4.78	22.86
2.5	4.30	1956	3.47	14.92

Table 5.6 Summary and comparison between the estimated heave by using different methods (Case study E)

Depth (m)	e_0	P_f(kPa)	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.5	0.600	4.5	8.0227	8.4420	290	291	180
1	0.546	12.9	1073.07	1129.15			
1.5	0.479	21.1	189.4026	199.3020			
2	0.438	29.	8372.23	8809.81			
2.3	0.485	38.0	51.9912	54.7086			
Ratio					1.61	1.62	-
Snethen and Johnson (1978) method:							157mm

Snethen and Huang (1992) provided the details of final stresses at the test site. The maximum plasticity index, I_P determined from the soil samples in the active zone is 33%. The estimated suction modulus ratio, C_w using Eq.(5.10) is 0.024. The estimated corrected swelling index, C_s is equal to 0.06 by using Eq.(5.11). The estimated 1-D heave using the K_I and K_{II} methods of the proposed technique are summarized in Table 5.6. The ratio of estimated heave to measured heave values using K_I and K_{II} methods were 1.61 and 1.62 respectively. The reason for high ratios may be attributed to the technique used in estimating the water content change, Δw, which was derived from the suction data obtained from filter paper method.

In expansive soils, even small changes water content from their natural conditions contribute to detrimental swelling (Chen, 1975). In the proposed technique, the water content change values were back calculated from the assumed soil suction changes and are approximate values (see Table 5.5). The discussions demonstrated that the proposed method is highly sensitive to water content measurements and care should be observed towards obtaining these values for reliable estimation of the 1-D heave.

5.3.5 Summary of five case studies results

The five case studies summarized are from three different countries (i.e., Canada, Sudan and U.S.A.). There are several other methods available to calculate the 1-D heave in expansive soils (Snethen and Johnson 1978, Fredlund 1983, Hamberg and Nelson 1984). The two key properties that required for providing comparisons between the measured and estimated 1-D heave using the different methods are the C_w and C_s.

The Fredlund (1983) method requires corrected swelling index, C_s and corrected swelling pressure, P'_s to calculate 1-D heave. Both these properties can be measured from *CVS* tests. Although C_s can be estimated by using Eq.(5.11), Fredlund (1983) method can not be used for providing comparisons between the estimated and measured heave values without the information of measured P'_s. Measured data with respect to corrected swelling index value, C_s from the *CVS* test was available only for one case study (Fredlund 1969). Hamberg and Nelson (1984) were the only investigators who measured the suction modulus ratio, C_w measured from Clod test. The values of C_w and C_s were estimated using Eq.(5.10) and Eq.(5.11) when the information was not available.

Comparisons between the measured and the estimated 1-D heaves using the different methods (i.e., Hamberg and Nelson 1984 and the proposed technique (using K_I

and K_{II} methods) for all the five case studies are summarized in Table 5.7).

Table 5.7 Summary of five case studies.

Reference	Fredlund (1969)	Hamberg & Nelson (1984)	Osman & Sharief (1987)	Osman & Sharief (1987)	Snethen & Huang (1992)
Site Location	Regina Canada	Colorado U.S.A.	Sudan	Sudan	Wynnewood U.S.A.
Case study	A	B	C	D	E
Measured Heave (mm)	84	72	142	150	180
Hamberg & Nelson (1984) Method (mm)	134	116	238	250	613
Estimated Heave by Author (mm)	151	116	295	295	2157
Proposed Technique I (using K_I) (mm)	110	93	154	155	290
Proposed Technique II (using K_{II}) (mm)	103	92	157	159	291
Estimated C_s	0.082	0.05	0.064	0.062	0.06
Estimated C_w	0.024	0.024	0.024	0.024	0.024
Ratio I	1.32	1.29	1.08	1.03	1.61
Ratio II	1.23	1.28	1.11	1.06	1.62

*Ratio *I* / *II* = Estimated heave using proposed technique (*I* or *II*) / Measured heave

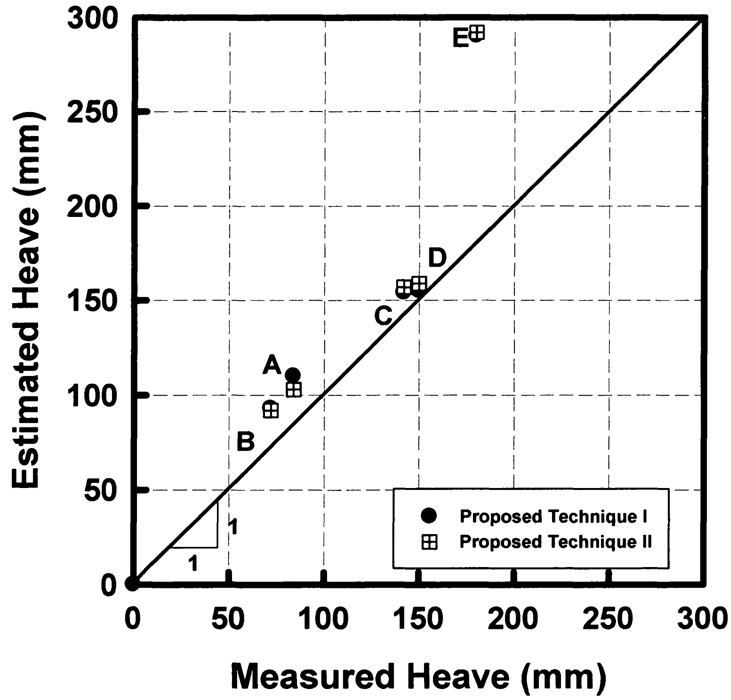


Figure 5.4 Comparison between the measured and estimate 1-D heave of the five case studies using the proposed technique

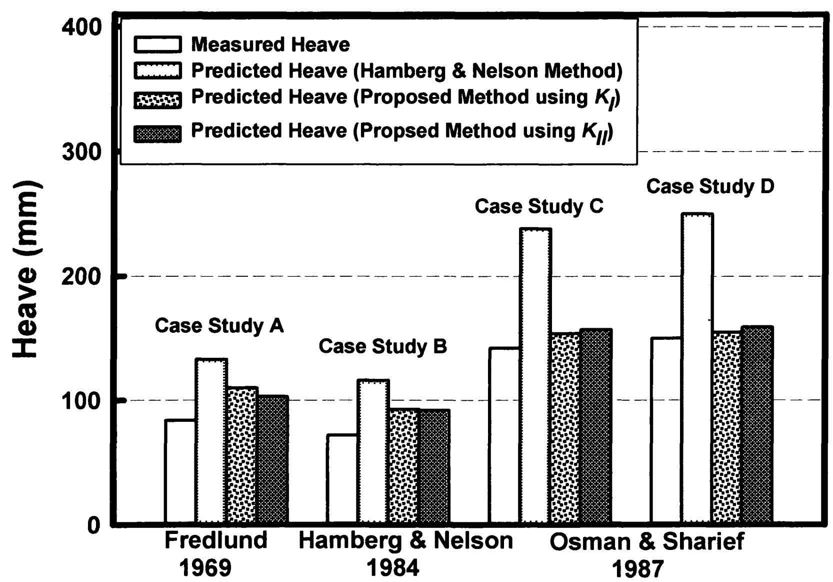


Figure 5.5 Comparison of measured and estimated 1-D heave of four case studies using different methods

Figure 5.4 provides the summary and comparison between the five case studies using the proposed technique (using K_I and K_{II} methods). Significant differences between the measured and estimated values of 1-D heave for the case study E (i.e., Snethen and Huang 1992) may be attributed to the indirect method of obtaining water content change Δw, which was derived from the suction data obtained from filter paper method.

Figure 5.5 provides summary of measured and estimated heave using the proposed technique (using K_I and K_{II} methods) and Hamberg and Nelson (1984) methods for four case study results (i.e., A, B, C and D). The proposed technique provides reasonable results for all the four case studies. The case studies (i.e., B, C and D) for which the water content variation with respect to depth data were available directly from field investigation studies provided excellent comparisons between the measured and estimated heave values. Hamberg and Nelson (1984) method overestimates the 1-D heave for all the five case studies. This may be attributed to ignoring the effect of overburden pressure as explained in Chapter 4 (Section 4.3).

The estimated heave to the measured values using the proposed technique (i.e. using K_I and K_{II} methods) is greater than unity for all the case studies, which is conservative from engineering practice. The differences between the measured and estimated heave is less than 30% which are reasonable as the approach is not overly conservative. The results of one case study E (i.e., Snethen and Huang 1992) are however higher (Table 5.7). The reasons associated with such a behavior were already discussed earlier in a different section.

5.4 Summary of Other Case Studies Results and Analysis

The results of the five case studies (Table 5.7) suggest that the proposed method is both simple and reliable to use. The estimated heave values however are highly sensitive to water content change readings. In this section, data obtained from eight more case studies from the literature are summarized and comparison between the in-situ measured surface heave and the estimated 1- D heave using the proposed technique are provided. The soil properties of these case studies were not used in deriving the empirical relationships. In other words, these are independent case studies. The objective of using these additional case studies is to check the validity and to understand the limitations of the proposed technique.

5.4.1 Case Study F (Snethen 1980)

The study area of this case study was located in Hayes, Kansas. Kansas's continental climate is highly changeable. The overall annual precipitation for the state falls between April and September. A group of eight field sites were monitored for a period of two years to collect data of 1-D heave. The initial field water content distribution was also measured along the depth of active zone which was approximately 2.5 m. Snethen (1980) summarized the key factors that influenced the field heave behavior as climate, drainage, and vegetation.

The average liquid limit, ω_L and plastic limit, ω_P of the expansive soil are 80% and 25%, respectively. The average dry unit weight and plasticity index, I_P of the expansive clay in the active zone were 16.82 kN/m^3 and 55% respectively. The overburden pressure at midpoint of each of the four soil layers in the active zone at different depth levels of 0.73m, 1.10m, 1.68m and 2.44m were provided in Snethen (1980) as 4.5kPa, 20.38kPa, 30.03kPa and 46.12kPa, respectively The suction modulus ratio, C_w and the corrected swelling index, C_s details were not available. These values are estimated to be 0.024 and 0.127 by using Eq.(5.10) and Eq.(5.11), respectively.

The measured maximum heave for this site was 165mm from field monitoring results. Snethen (1980) used soil suction technique (see Chapter 3, Eq.(3.30) and Eq.(3.32)) to calculate the 1-D heave for this site as 206mm (see Table 5.9). The initial water content variation with respect to depth was in situ measured at four different depths in the active zone from 0 to 2.5 m (i.e., 0.73m, 1.10m, 1.68m and 2.44m). The final soil suction was assumed by Snethen (1980) and shown in Table 5.8. As discussed earlier in case study E, the final soil suction distribution was assumed based on an assumption; namely, the saturated water content profile.

Nine soil samples from the test site were used for obtaining soil suction measurement using thermocouple psychrometers. Eq.(5.14) was estimated by the soil suction tests results.

$$\log \tau_m = A - B\, w \tag{5.14}$$

where, τ_m (tsf) is the matric suction without surcharge pressure (i.e., atmospheric pressure); A, B are constants; w is the measured water content,%.

The final soil suction assumption requires that the saturated water content be used in Eq.(5.13) to estimated the final soil suction values (Snethen 1980). Therefore, for using the proposed technique, the final water content was back-calculated from Eq.(5.14) as the values of the parameters A, B and the estimated final soil suction are available in Snethen (1980) (see Table 5.8).

Table 5.8 Final water content calculation using soil suction data

Assumed final soil suction (tsf)	Assumed final soil suction (kPa)	A	B	Calculated final water content (%)
3.1	297	5.107	0.1786	25.8
1.9	182	5.107	0.1786	27.3
2.3	220	5.107	0.1786	26.6
8.6	824	6.195	0.2439	21.6

The distribution of initial and estimated final water content with depth for the site (Figure 5.6) are used to obtain the water content change along with depth in the site. Comparisons are provided between the measured heave and proposed technique (using K_I and K_{II} methods) (Table 5.9).

The proposed technique *I* & *II* (Eq.(5.12) and Eq.(5.13)) have been used to predict the maximum 1-D heave. The ratios (i.e., estimated heave using proposed method *I* and

II) / measured heave) are 1.24 and 1.07, respectively. As demonstrated earlier, the proposed method is highly sensitive to water content measurements. The differences between measured heave and estimated heave may be attributed to the estimated values of final water content.

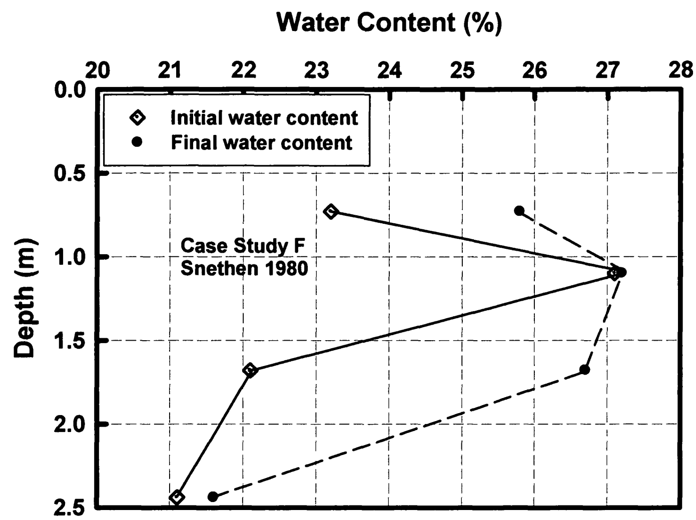


Figure 5.6 Distribution of in-situ moisture content with depth.

Table 5.9 Summary and comparison between the measured and estimated heave using different methods.

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.73	0.27	0.629	0.0032	0.0046	205	177	165
1.10	0	0.745	0.0029	0.0039			
1.68	0.52	0.613	0.0044	0.0054			
2.44	0.47	0.589	0.0038	0.0053			
Ratio					1.24	1.07	-
Snethen (1980) soil suction method:							206 mm

5.4.2 Case Study G (Yoshida et al. 1983)

The proposed technique was tested on case study results of Regina clay (Yoshida et al. 1983). The details of this case study were summarized in Chapter 4. The distribution of initial and final water content with depth is shown in Figure 5.7. Table 5.10 shows the comparison between the measured and the estimated 1-D heaves using different methods.

The results of the study show that the proposed method can be used reliably in estimating the 1-D heave. The ratios of the estimated total heaves to the measured total heaves are 1.04 and 0.95 using the proposed technique (using K_I and K_{II}), respectively.

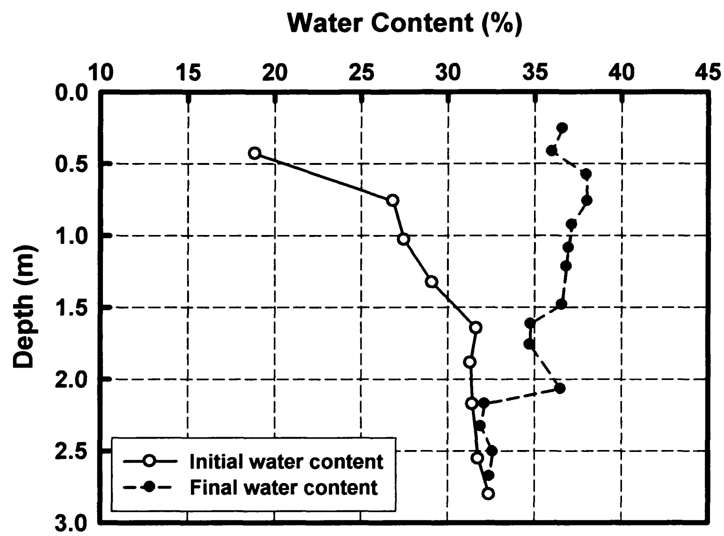


Figure 5.7 Variation of in situ water content with respect to the depth. (Yoshida et al. 1983)

Table 5.10 Summary of the case study data and the comparison between the measured and the estimated heave

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.15	21.3	0.962	2694.321	3253.366	110	101	106
0.45	12.6	0.962	9.9881	12.0606			
0.75	10.8	0.962	3.3149	4.0027			
1.05	8.9	0.962	0.9636	1.1636			
1.35	5.8	0.962	0.1305	0.1573			
1.65	5.4	0.962	0.1306	0.1251			
1.95	4.4	0.962	0.0544	0.0657			
2.25	2.9	0.962	0.0208	0.0251			
Ratio					1.04	0.95	-
Fredlund (1983) method:							118 mm

5.4.3 Case Study H (Dhowian et al. 1987)

The tests site is located in Al-Ghatt (Middle region) of Saudi Arabia. The general conditions are dry and hot; most of rain falling is between January and May. The study area has clay shale or silty shale deposits that are expansive in nature. An instrumented field station was established to investigate 1-D heave behavior of these deposits. A saturation system was installed at the site to provide access to water. In-situ water content measurements were made through a nuclear device. The expansive soil was wetted through the sand drains for a period of 54 weeks. Measurements of water content, soil suction, and heave were taken throughout this period. The maximum heave measured directly from this instrumented field station was 200 mm. The depth of the active zone in this deposit was estimated as 6m. The properties of the typical expansive soils in Al-Ghatt are listed in Table 5.11.

Table 5.11 Soil properties of soils in Al-Ghatt site

Physical properties	Clay Shale	Silty Shale
Dry unit weight, γ_d	18.5	18.5
Liquid limit, ω_L	65	46
Plastic limit, ω_P	30	21
Plasticity index, I_P	35	25
Specific gravity, G_s	2.78	2.7

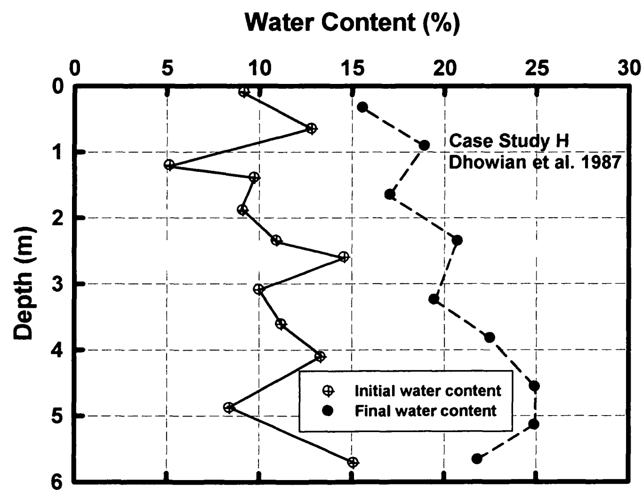


Figure 5.8 Distribution of in-situ moisture content with depth.

The distribution of initial and final water content with depth for the site measured by Dhowian et al. (1987) is shown in Figure 5.8. According to the information of water content change, Δw, which is derived from Figure 5.8, correction parameter K can be calculated. The active zone of this site has been divided into 6 layers with the information of water content distribution. The site contains both the clay shale (CH, I_p = 35%) and the silty shale (CL, I_p=25%). The soil properties of clay shale, which has higher tendency swell, were used for estimating 1-D heave of the site.

The suction modulus ratio, C_w and the average corrected swelling index, C_s in the active zone were estimated to be 0.024 (Eq.5.9) and 0.064 (Eq.5.10), respectively. Comparisons between the measured heave and the estimated heave for each soil layers are provided using the proposed methods (by using K_I and K_{II}) in Table 5.12.

Table 5.12 Comparison between the measured and the estimated heave of the Case study H using the proposed technique .

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
1	10.1	0.4317	0.0648	0.0505	280	275	200
2	7	0.4317	0.0466	0.0505			
3	9	0.4317	0.0641	0.0695			
4	9.2	0.4742	0.0246	0.0260			
5	16.3	0.4742	0.0068	0.0074			
6	6.8	0.4742	0.0068	0.0074			
Ratio					1.40	1.38	-
Dhowian (1990) *CVS* oedometer test method:							200 mm

5.4.4 Case Study I (Maksimovic and Tonkovic 1987)

The studied site is located in Middle East area close to the junction point of Syria, Iraq and Turkey. Dry season from June to September with maximum daily temperature in July and August; it is one of the main reasons for expansive clay activity. The high swelling potential of soils in this region had been reported in many previous literature studies (i.e. Donaldson 1969).

The maximum vertical movement of the free ground surface due to seasonal changes in water content was measured to be 150mm. The results of the data summarized however are based on soil investigation studies conducted only over a time period of four months. Maksimovic and Tonkovic (1987) reported that the swelling problem requires continuous soil investigations and monitoring of data at least for a period of one year;

however, it preferable to extend these studies for several years. Therefore, the reported heave (i.e., 150 mm) may not be the maximum heave value for this site. Two bore holes were drilled for monitoring the soil water content and active zone variation. The water content in the site is mainly associated with infiltration activity related to rainfall. The maximum depth of the active zone is about 9 to 12 meters. The final stress was considered to equal to overburden pressure.

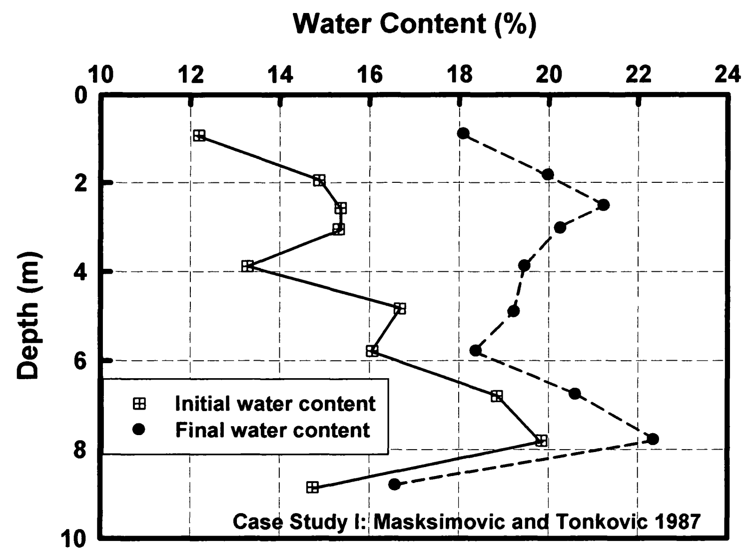


Figure 5.9 Distribution of in-situ moisture content with depth.

Table 5.13 Summary and comparison between the measured and the predicted heave of case study.

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
1	6.91	0.524	0.3230	0.3249	166	164	150
2	6.29	0.603	0.2172	0.2185			
3	5.25	0.601	0.1116	0.1123			
4	5	0.566	0.0951	0.0957			
5	3.67	0.578	0.0406	0.0408			
6	2.63	0.524	0.0209	0.0210			
7	2.57	0.609	0.0201	0.0202			
8	2	0.656	0.0139	0.0140			
9	1.72	0.477	0.0117	0.0117			
Ratio					1.11	1.09	-
Maksimovic and Tonkovic (1987) swelling potential method:							150 mm

The liquid limit, ω_L is in the range of 50% to 70% within the top 5 meters and increases to about 100% to 120% for depth values greater than 5 meters. The maximum plasticity index, I_P of the specimen from the clay in the active zone is 40%. The average dry unit weight is 17.52 kN/m^3. The suction modulus ratio, C_w and the corrected swelling index, C_s in the active zone were estimated to be 0.024 (Eq.5.9) and 0.08 (Eq.5.10), respectively. The distribution of initial and final water content with depth for the site is shown in Figure 5.9, and Δw is derived from this figure.

Comparisons between the measured heave and the estimated heave using the proposed technique (using K_I and K_{II} methods) are provided in Table 5.13. The estimated heave obtained by using the proposed technique is 166 mm and 164 mm respectively using the K_I and K_{II} methods. All the data required for using both these methods were available for this case study; therefore, the estimated heave values were in close agreement with the in-situ measured heave.

5.4.5 Case Study J (Retamal et al. 1987)

The La-Dehesa site studied by Retamal et al. 1987 is located on the basin of Santiago, Chile. The climate is warm tempered, and the rainfall and snow fall normally are between May and August. The test site was built excavating the surface soil in a depth of 0.15m and then building the control points formed by a concrete block poured in place and a steel point; after installing the control point, the surface was covered with a polyethylene membrane (Retamal et al. 1987). The final stress for the test sites was considered to be equal to the overburden pressure only (i.e., no surcharge loading). The water content variation monitored from May, 1986 to April, 1987 (1 year) could be attributed only to rainfall (climatic effect). The maximum vertical movements observed for the site was 70mm; but the residual expansion was not observed (Retamal et al. 1987). Therefore, the reported heave was not the maximum heave value for this site.

The depth of the active zone is about 2 m. The liquid limit, ω_L and average plastic limit, ω_P of the expansive soil (i.e. dark brown clay) are 46-67% and 20%, respectively.

The average plasticity index, I_P of the clay in the active zone is 45%. The average dry unit weight is 16.25kN/m^3. The suction modulus ratio, C_w and the corrected swelling index, C_s were estimated to be 0.024 (Eq.5.9) and 0.097 (Eq.5.10), respectively. The distribution of initial and final water content with depth for the site is shown in Figure 5.10. Comparisons are provided between the measured heave and the estimated heave using the proposed technique (using K_I and K_{II} methods) in Table 5.14.

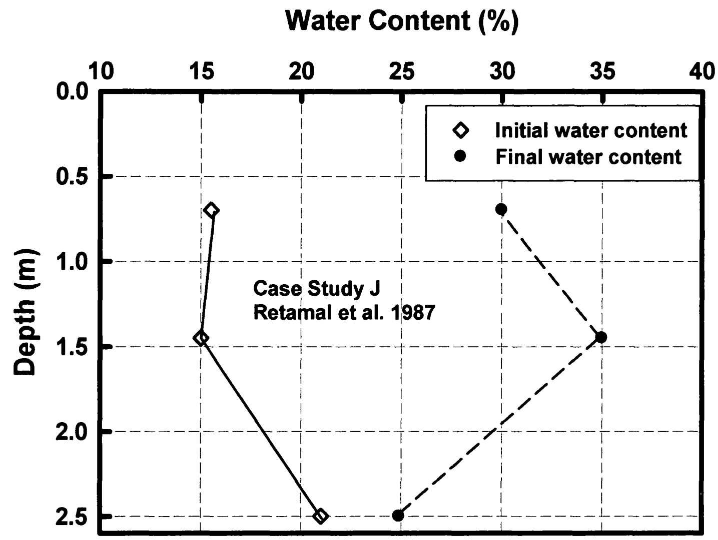


Figure 5.10 Distribution of in-situ moisture content with depth (La-Dehesa site).

The 1-D heave prediction is dependent on the measured water content distribution and time allowed for swell (see Chapter 2: section 2.3.4 for more details). The water content distribution for this site was available at only three different levels (i.e., 0.70m, 1.45m and 2.50m) over a depth of 2.5m. The variation of water content over the other depths had to be approximated. In addition, Retamal et al. (1987) stated that there was residual expansion but not observed in the test site; in other words, the observation time was inadequate and the heave provided (i.e., 70mm) is not the maximum heave in the studied site. This may be the reasons that the proposed technique shows overestimated results, and the heave estimation by using proposed technique is 1.30 – 1.43 times more than the maximum observed heave.

Table 5.14 Summary and comparison between the measured and the predicted heave of case study (La-Dehesa site).

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm)
0.70	14.5	0.56	32.912	41.814	100	91	70
1.45	20	0.64	1523.495	1412.648			
2.50	4	0.66	0.038	0.047			
Ratio					1.43	1.30	-
Retamal et al. (1987) estimated heave:							Unavailable

5.4.6 Case Study K (Nelson and Miller 1992)

The site is located in Fort Collins, Colorado. Colorado has a cool and semi-arid climate. The depth of the active zone was estimated to be equal to 1.8 m. The water content data was collected over several seasons (i.e., 596 days) after a floor slab had been placed on the ground surface. The nuclear moisture probes were used for measuring in-situ water content distribution over a depth of 1.8 m. The measured heave was equal to 82mm. The final stress was estimated to be equal to 6.37 kPa, 12.74 kPa, 19.11 kPa, 25.48 kPa, 31.85 kPa and 38.22 kPa for each soil layer at different depth levels of 0.3m, 0.6m, 0.9m 1.2m, 1.5m and 1.8m respectively. The predicted heave, which is equal to 110mm, has been provided in Nelson and Miller (1992).

The average liquid limit, ω_L and plastic limit, ω_P of the expansive soil (i.e., Pierre shale) is 50% and 22%, respectively. The plasticity index, I_P of the specimen from the clay in the active zone is 28%. The average dry unit weight is 19.62 kN/m^3. The measured suction modulus ratio, C_w is 0.019 (from Nelson and Miller 1992). The suction modulus ratio, C_w and the corrected swelling index, C_s in the active zone can be estimated to be 0.024 and 0.05 as well by using Eq.(5.10) and Eq.(5.11), respectively. The distribution of initial and final water content with depth for the site is shown in Figure 5.11. Comparisons are provided between the measured heave and the estimated heave using the proposed technique (using K_I and K_{II}) in Table 5.15. Though the proposed technique (using K_I and K_{II}) have overestimated heave in this site, this is a valuable case study to test the limitation of the proposed technique.

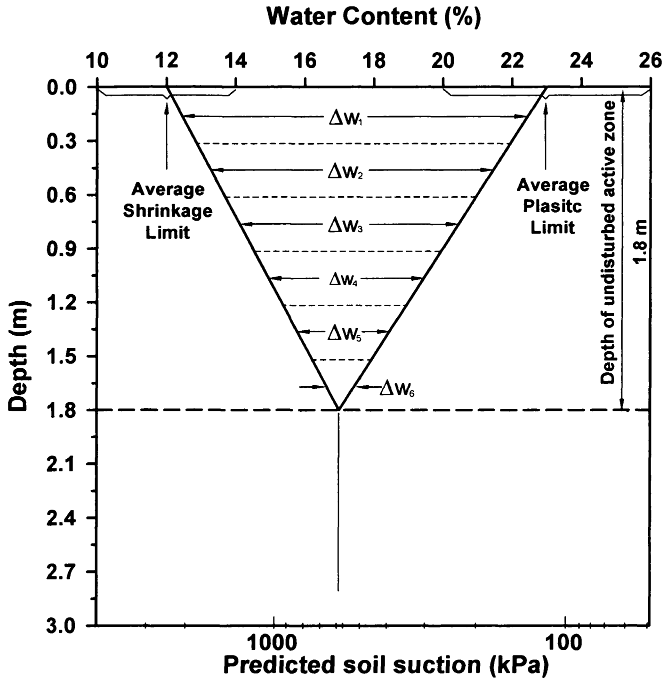


Figure 5.11 Idealized moisture boundary profile for the Pierre shale, Fort Collins (modified from Hamberg 1985).

Table 5.15 Summary of comparison between the measured and the predicted heave of case study.

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm
0.3	10.1	0.9	2.5678	2.5023	116	117	82
0.6	8.2	0.8	0.7611	0.7417			
0.9	6.4	0.7	0.2405	0.2344			
1.2	4.6	0.6	0.0760	0.0741			
1.5	2.7	0.55	0.0225	0.0220			
1.8	0.9	0.5	0.0071	0.0069			
Ratio					1.42	1.43	-
Nelson and Miller (1992) soil suction method:							110mm

It should be noticed that, in the proposed approach, the suction modulus ratio, C_w is estimated to 0.024 while the plasticity index, I_p of the soil is higher than 30%. In this case study, the plasticity index I_p of the soil in the site is equal to 28%. Hence, it is not appropriate to use the estimated C_w (i.e., 0.024). By using this measured C_w value (i.e., 0.019), the estimated heave using the proposed technique is 88 mm and 89 mm respectively by K_I and K_{II} methods, respectively. The differences may be attributed to using the estimated value of $C_{w.}$ This case proves that, using the measured value of $C_{w.}$ provides more reasonable results than using the estimated value.

5.4.7 Case Studies L & M (Fityus et al. 2004)

The test site, which is called Maryland site, is established in Newcastle, Australia to measure the long-term (i.e., 7 years) behavior of 1-D heave of the regional expansive soils in both open and covered areas. The region has a temperate, near coastal climate with an annual rainfall typically between 1000 and 1200 mm per year. The open area test site facilities the measurement of free field heave while the covered area (with a surcharge of approximate 3kPa) provides valuable data to understand the differences 1-D heave characteristics in comparison to open area.

The test field site was extensively instrumented and data was collected over a long period of time. The instrumentation includes 154 surface movement indicators, 28 subsurface movement indicators, 9 neutron probe for in-situ measurement of soil water content and 6 in-situ filter paper devices for measurement of soil suction. Probably, this is the most well instrumented test site reported in the literature for measurement of 1-D heave. In spite of all the care, it was reported that the authors had difficulties in collecting reliable data sometimes due to instrumentations problems. The predicted ground movement by using Fityus and Smith (1998) method (see Chapter 3, section 3.4.10) for the open area is 41mm, which is an underestimation of the measured value (i.e., 75mm) (Fityus et al. 2004).

The depth of the active zones for the open area and cover area are approximately 1.5 m and 0.5 m, respectively. The liquid limit, ω_L and plastic limit, ω_P of the expansive soil (i.e. Maryland clay) are 70% and 25%, respectively. The plasticity index, I_p of the specimen from the clay in the active zone is 45%. The average dry unit weight is 15.52 kN/m3. The suction modulus ratio, C_w and the corrected swelling index, C_s can be estimated to be 0.024 and 0.09 by using Eq.(5.10) and Eq.(5.11), respectively. The distribution of initial and final water content with depth for open and covered areas of the test sites are shown in Figure 5.12 and Figure 5.13, respectively. Comparisons between the measured heave and the estimated heave using the proposed technique (using K_I and K_{II} methods) are summarized in tables 5.15 and 5.16, respectively.

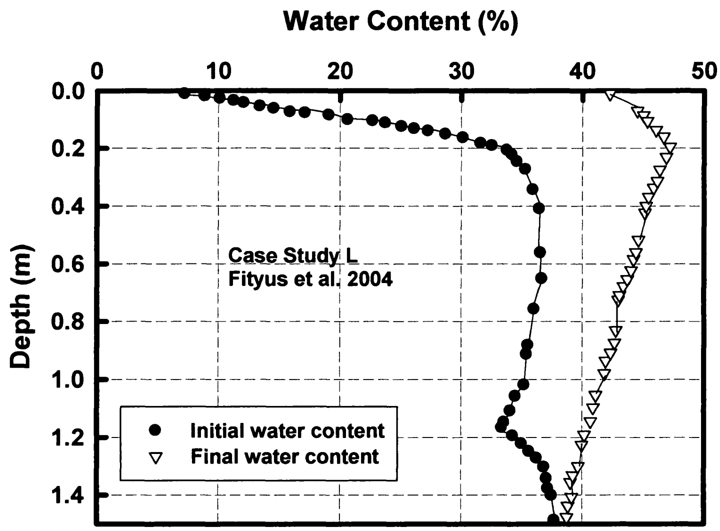


Figure 5.12 Distribution of in-situ moisture content with depth (open area).

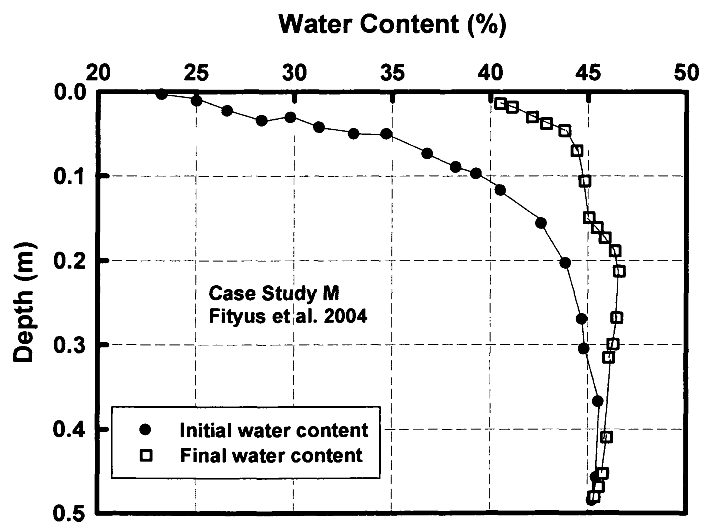


Figure 5.13 Distribution of in-situ moisture content with depth (cover area).

There is an excellent comparison between the measured 1-D heave and the estimated values for the covered area (see Table 5.16); however, there are significant differences for open area test site. Fityus et al. 2004 summarized that the neutron probes used in open area has proven to be an effective means for long-term monitoring of in-situ water content changes. However, the extraction of absolute water content data from neutron probe counts has proven to be problematic. Further, the shrinkage cracks extended from the ground surface down to the subsurface in the open area. While the first reason of problems associated with collection of water content using neutron probe has been of some concern; however, measurements of water contents in the zone of shrinkage cracks were a challenge. These two factors may have contributed to some errors in the data collection of water content distribution with respect to depth in open area. Due to this reason, there are differences between the estimates of the 1-D heave using the proposed technique in comparison to the measured values for the open area test site.

Table 5.16 Summary and comparison between the measured and the predicted heave (open area).

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm
0.25	11.89	0.69	6.3510	7.8681	101	109	75
0.50	8.1	0.69	0.5616	0.6957			
0.75	7	0.69	0.2778	0.3441			
1.00	6.59	0.69	0.2137	0.2647			
1.25	4.16	0.69	0.0451	0.0559			
1.50	1.01	0.69	0.0061	0.0074			
Ratio					1.35	1.45	-
Fityus and Smith (1998) method:							41mm

Table 5.17 Summary and comparison between the measured and the predicted heave (covered area).

Depth (m)	Δw %	e_0	K_I	K_{II}	Proposed Technique I (mm)	Proposed Technique II (mm)	Measured Heave (mm
0.1	5.2	0.794	0.0878	0.1087	40	38	35
0.2	2.7	0.794	0.0177	0.0220			
0.3	1.5	0.794	0.0082	0.0102			
0.4	0.6	0.794	0.0046	0.0057			
0.5	0.1	0.794	0.0034	0.0042			
Ratio					1.14	1.09	-
Fityus and Smith (1998) method:							41mm

5.5 Summary and Comparison

Table 5.17 summarizes the key details with respect to the measured 1-D heave and estimated heave values using the proposed technique *(i.e.,* K_I and K_{II} methods) for the 13 different case studies analyzed in the Chapter. The estimated heave to the measured values using the proposed technique (i.e. using K_I and K_{II} methods) is greater than unity for all the case studies.

Table 5.18 Summary of case studies

Case Study	**A***	**B**	**C**	**D**	**E****	**F****
Reference	Fredlund (1969)	Hamberg & Nelson (1984)	Osman & Sharief (1987)	Osman & Sharief (1987)	Snethen & Huang (1992)	Snethen (1980)
Site Location	Regina Canada	Colorado U.S.A.	Sudan	Sudan	Wynnewood U.S.A.	Kansas U.S.A.
Plasticity Index (%)	42	28	35	34	33	55
Measured Heave (mm)	84	72	142	150	180	165
Proposed Technique I (mm)	110	93	154	155	290	205
Proposed Technique II (mm)	103	92	157	159	291	177
Ratio I (%)	1.32	1.29	1.08	1.03	1.61	1.24
Ratio II (%)	1.23	1.28	1.11	1.06	1.62	1.07
Measured C_s	0.094[@]	-	-	-	--	-
Measured C_w	-	0.016[@@]	-	-	-	-

Case Study	**G**	**H***	**I**	**J**	**K**	**L**	M
Reference	Yoshida et al. (1983)	Dhowian et al. (1987)	Maksimovic & Tonkovic (1987)	Retamal et al. (1987)	Nelson & Miller (1992)	Fityus et al. (2004)	Fityus et al. (2004)
Site Location	Regina Canada	Saudi Arabia	Yugoslavia	Santiago Chile	Colorado U.S.A.	Newcastle Australia	Newcastle Australia
Plasticity Index (%)	43	25-35	40	45	28	45	45
Measured Heave (mm)	106	200	150	70	82	75	35
Proposed Method I (mm)	110	280	166	100	116	101	40
Proposed Method II (mm)	101	275	164	91	117	109	38
Ratio I (%)	1.04	1.40	1.11	1.43	1.42	1.35	1.14
Ratio II (%)	0.95	1.38	1.09	1.31	1.43	1.45	1.09
Measured C_s	0.09	-	-	-	-	-	-
Measured C_w	-	-	-	-	0.019@@@	-	-

*Final moisture content is assumed to be saturated.

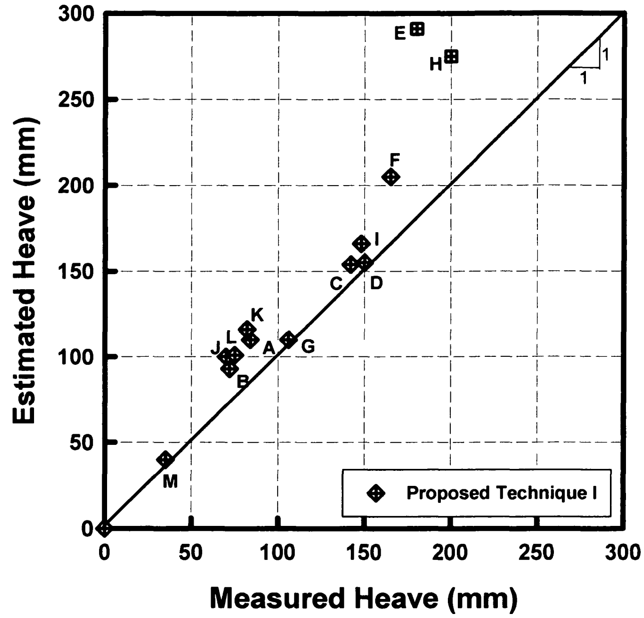
** Final water content is calculated from soil suction distribution.

*** Plasticity index is assumed to be the maximum value in the tested field.

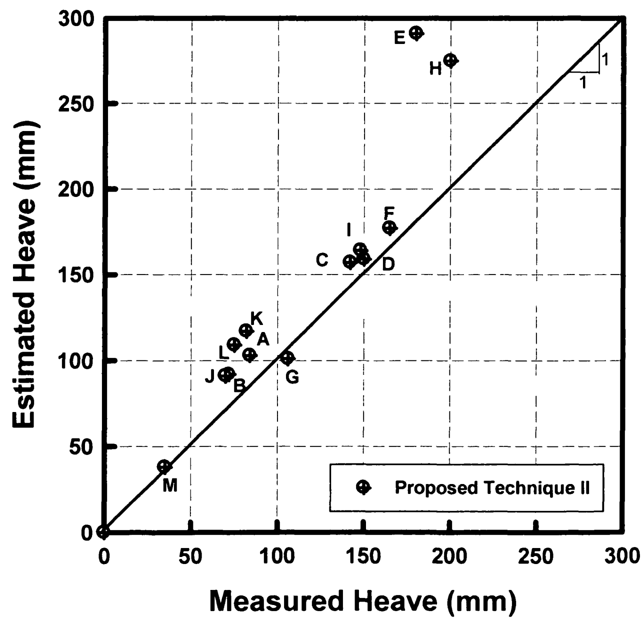
@ Using measured C_s, the estimated heave by using proposed technique *I* and *II* are 107mm, 98mm (i.e., ratio: 1.27, 1.17), respectively.

@@ Using measured C_w, the estimated heave by using proposed technique *I* and *II* are 87mm, 88mm (i.e., ratio: 1.21, 1.23), respectively.

@@@ Using measured C_w, the estimated heave by using proposed technique *I* and *II* are 88mm, 89mm (i.e., ratio: 1.07, 1.08), respectively.



(a)



(b)

Figure 5.14 Comparison of 13 case studies, (a) proposed technique *I*; (b) proposed technique *II*

Figure 5.14 shows the comparison between measured and estimated 1-D heave for the 13 case studies. The measured and estimated values for both case study *E* (Snethen and Huang 1987) and case study *H* (Dhowian et al. 1987) have greater difference in comparison with other 11 case studies. Significant differences between the measured and estimated values of 1-D heave for them may be attributed to using assumed field condition for heave estimation.

For case study A (Regina, Fredlund 1969), the water content change was obtained by assuming the soil was saturated when the heave measurements were made. This will result in maximum heave values in the tested field. The assumption is not valid since the entire active zone depth may not be in a saturated condition. In addition, the differences is also attributed to the estimated value of C_s. Using measured C_s (i.e., 0.094), which was provided in the literature, the results obtained by using proposed technique *I* and *II* are 107mm, 98mm (i.e., ratio: 1.27, 1.17) respectively.

For case study B (Colorado, Hamberg and Nelson 1984, ratio: 1.29, 1.28) and case study K (Colorado, Nelson and Miller 1992, ratio: 1.42, 1.43), the measured suction modulus ratio, C_w of the expansive soil (I_p = 28%) were provided in literature, which are 0.016, 0.019, respectively. In case study B, using measured C_w, the estimated heaves for the proposed technique *I* and *II* are 87mm, 88mm (i.e., ratio: 1.21, 1.23) respectively. In case study K, using measured C_w, the estimated heaves for proposed technique *I* and *II* are 88mm, 89mm (i.e., ratio: 1.07, 1.08), respectively. In the proposed technique, the suction modulus ratio, C_w is estimated to 0.024 while the plasticity index, I_p of the soil is higher than 30%. The overestimation for both of the case studies may be attributed to using the estimated value of C_w. The case studies are proves that there is need to improve the technique for dealing the soils which has plasticity index I_p lower than 30%. However, the two case studies still got reasonable results without the time-consuming laboratory tests (i.e. Clod test).

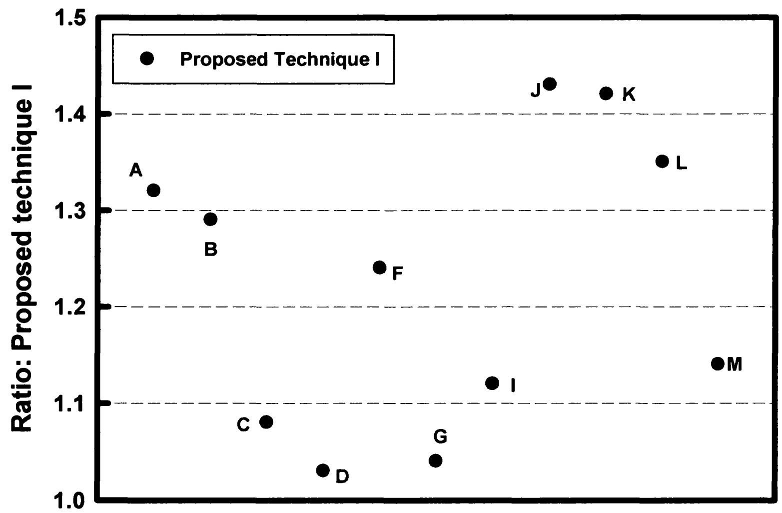
Both of the parameters C_s, C_w are estimated by using the relationships between laboratory measured C_s, C_w and plasticity index, I_p. Case studies A (Fredlund 1969), B

(Hamberg and Nelson 1984) and K (Nelson and Miller 1992) could have better estimation results when using the measured C_s or C_w to instead of estimated values.

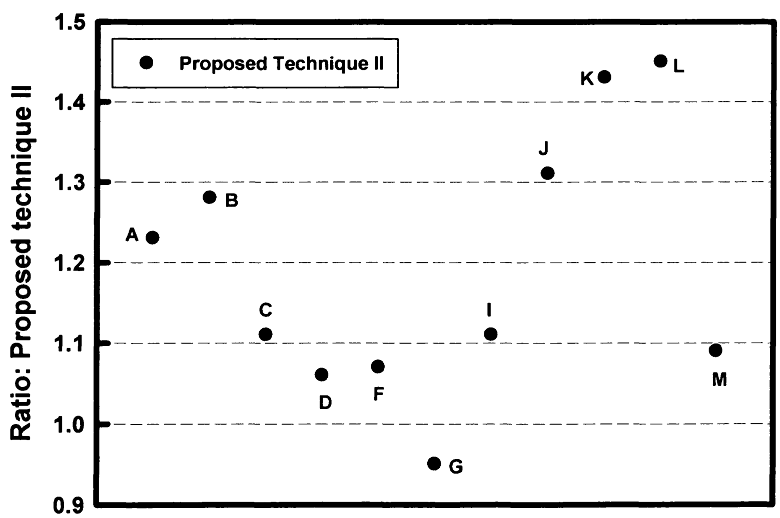
For the case study E (Wynnewood, Snethen and Huang (1992)), the final water content used in proposed technique (K_I and K_{II} methods) is derived from the values of soil suction obtained from filter paper method. The ratios for proposed technique *I* and *II* are 1.61, 1.62, respectively. For case study *H* (Saudi Arabia, Dhowian et al. 1987), the expansive soils in the studied site include both clay shale (I_p = 35%) and silty shale (I_p = 25%); but the soil properties of clay shale, which has higher tendency to swell, were used for predicting the maximum heave. The two case studies show much greater ratios than others. This is considered less favorable since it can result in overtreatment of the expansive soils (Snethen 1980). As discussed earlier, the proposed techniques (*I* and *II*) are highly sensitive to the water content change Δw, and the plasticity index, $I_{p.}$ Therefore, the two case studies have overestimation results.

In case study J (Santiago, Retamal et al. 1987), the water content distribution was just provided in three different soil layers that the variation of water content for other depths need to be approximated; and the reported heave was not the maximum heave value for this site. These may be the reasons that cause overestimation results. The case study L (Newcastle, Fityus et al. 2004) was reported that the authors had difficulties in collecting reliable water content distribution due to instrumentations problems. Using the inaccurate water content change should be the main reasons that cause overestimation. The case study L may also show that, with the consideration of all the essential data required for heave prediction technique, there are still factors (i.e., the sensitivity and accuracy of the equipments, the ground surface features) will certainly affect the heave prediction results. Considering these factors may help the geotechnical engineers build more stable structure on expansive soils.

The ratios obtained from 11 case studies (i.e., excluding case study E and case study H) have been shown in Figure 5.15. The differences between the measured and estimated heave for 8 of these case studies is less than 30%, which are reasonable as the approach is not overly conservative. The results of case studies J, K and L are 30% overestimated to the measured results, which are considered less favorable than other 8 case studies.



(a)



(b)

Figure 5.15 Summary and Comparison of ratios obtained from 13 case studies, (a) proposed technique *I*; (b) proposed technique *II*

5.6 Conclusion

Several researchers and practitioners have significantly contributed 1-D or 3-D heave prediction techniques to understand the expansive soils behavior. However, most of these techniques require time-consuming laboratory and/or in-situ tests, which are expensive and difficult to be performed by geotechnical engineers. Hence, the key objective of the research presented in this thesis is to propose a 1-D heave prediction technique, which is simple, inexpensive to use and universally valid. This technique requires only the information of plasticity index I_p, the dry unit weight, γ_d and the variation in natural water content, Δw with respect to depth in the active zone of natural expansive soils. The proposed technique was tested using data from 13 case studies published in literature.

The comparison results show that, the proposed technique can be used to obtain reasonable result of 1-D heave in natural expansive soils. Using the reliable data from field investigation, including Atterberg Limits, in-situ moisture variations and soil profile in the active zone, are necessary requirements to successfully estimate 1-D heave while using the proposed technique. The plasticity index, I_p and water content change, Δw can highly affect the technique. In spite of some limitations, the proposed technique is simple and can be used by the geotechnical engineers in the estimation of 1-D heave in expansive soils routinely in practice.

CHAPTER 6

CONCLUSIONS AND RECOMENDATIONS

Expansive soils can be defined as soils which exhibit significant volume changes with varying water contents. They are conventionally considered as problematic soils that contribute to financial losses. Practicing engineer is routinely interested in the estimation of 1-D heave and its impact on infrastructure. The currently used techniques for predicting the 1-D heave can be divided into three categories: (i) empirical methods; (ii) soil suction methods and (iii) oedometer test methods. The limitations of these techniques are: i) they are not universally valid; ii) they do not use the stress state variables approach; iii) the various soil parameters required in these techniques can only be obtained from time consuming laboratory or in-situ tests that are expensive and difficult to be performed.

The key objective of the research presented in this thesis is to provide a technique to estimate the 1-D heave of expansive soils. This technique is proposed by deriving a new relationship from Fredlund (1983) and Hamberg and Nelson (1984) methods alleviating some of the limitations of both these methods.

The proposed technique requires only the information of plasticity index, I_p, the dry unit weight, γ_d and water content variation with respect to the depth in the active zone of natural expansive soils. It was tested on 13 case studies from seven different countries. The good agreement between the measured and estimated heave shows that the proposed technique can be used reliably in estimating the 1-D heave. The results also show that (*i*) the estimated heaves using the proposed technique is higher than the measured heaves, which can be considered as a conservative approach; (*ii*) the plasticity index, I_p and water content change, Δw can highly affect the estimation using this technique; (*iii*) the technique can obtain reliable estimation results with accurate data provided from field investigations.

Other topics recommended to be further investigated including the following:

1. Four empirical relationships have been proposed for estimation of the parameters C_w, C_s and K using laboratory tests results and case study results from published literatures. It is recommended that these relationships need to be further improved using more data from the literature to improve the reliability of the proposed technique.
2. Several studies show that water will continue to migrate within the soils and bedrock through their entire design life period. However, significant influence of water migration is typically observed in the top 6 m below the ground surface. Investigations to predict 1-D heave as a function of time by modifying this technique would be more valuable.

The proposed technique is simple and should encourage geotechnical engineers to implement the technique for unsaturated soils into practice.

REFERENCE

Aitchison, G. D. 1960. Relationships of moisture stress and effective stress functions in unsaturated Soils. *In Proceedings of the Conference on Pore Pressure and Suction in Soils.* Butterworths, London, UK. pp.47–52.

Aitchison, G. D. 1965. Moisture equilibrium and moisture changes in soils beneath covered areas. Statement of the Review Panel, *Engineering Concepts of Moisture Equilibrium and Moisture Changes in Soil,* London, pp. 7-22.

Aitchison, G. D. 1965. Soil properties, shear strength, and consolidation. *In Proceedings of 6th International Conference Soil Mechanics and Foundation Engineering*, Montreal, Canada, **3**: 318 – 321.

Aitchison., D. 1970. A statement of the problems the engineer faces with expansive soils. *Proceedings 2nd International Research and Engineering Conference on Expansive Clay Soils*, Texas A & M University, College Station, pp. 33-51.

Aitchison, G. D. 1973. The quantitative description of the stress-deformation behavior of expansive soils – preface to set of Papers. *In Proceedings of the 3rd International Conference on Expansive Soils*, **2**: 79 – 82.

Aitchison, G. D. and Woodburn. J. A. 1969. Soil suction in foundation design. *In Proceedings of the 7th International Conference of Soil Mechanics and Foundation Engineering,* Mexico, **2**: 1-9.

Allam, M. M. and Sridharan, A. 1987. Effect of repeated wetting and drying on shear strength. *Journal of the Geotechnical Engineering*, ASCE **107**: 121-438

Allman, M. A., Delaney, M. D. and Smith, D. W. 1998. A field study of seasonal ground movements in expansive soils. *Proc., 2nd Int. Conf. on Unsaturated Soils*. International Academic Publishers, Beijing, 309–314.

Alonso E. E. 1998. Modeling expansive soil behavior. *In Proceedings of the Second International Conference on Unsaturated Soils*, Beijing, China. **1**: 37-30.

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

Alonso, E. E., Lloret, A., Gens, A. and Salvado, M. 2001. Overcosolidation effects on secondary compression rates. *In the Proceeding of the 15th International Conference on Soil Mechanical of Geotechnical Engineering*, Istanbul.

Al-Shamrani, M. A. and Al-Mhaidib, A. I. 1999. Prediction of potential vertical swell of expansive soils using a triaxial stress path cell. *Quarterly Journal of Engineering Geology and Hydrogeology*, **32**(1): 45-54.

Al-Shamraani, M. A. and Dhowian, A. W. 2002 Experimental study of lateral restraint effects on the potential heave of expansive soils. *Engineering Geology,* **69**: 63-81.

ASTM 1970. Special procedures for testing soil and rock for engineering purposes. *ASTM, Special Technical Publication 479*, 5th ed.

ASTM Standards 1991. Natural building stones: soil and rock. *Annual Book of ASTM Standards*, Vol. 4, Philadelphia.

ASTM 2000. Standard test methods for one-dimensional swell or settlement potential of cohesive soils. *Annual Book of ASTM Standards*, D4546-2000, **04**(8): 853-859.

ASTM D5298 2003. Standard Test Method for Measurement of Soil Potential (Suction) Using Filter Paper. *Annual Book of ASTM Standards,* 04.09*, Soil and Rock, American Society for Testing and Materials*, West Conshohocken, PA.

Barbour, S. L. and Fredlund, D. G. 1989. Mechanisms of osmotic flow and volume change in clay soils, *Canadian Geotechnical Journal*, **26**: 551–562.

Basma, A. A., Al-Homoud, A. S. and Malkawi, A. I. 2000. Swelling-shrinkage behaviour of natural expansive clays. *Appl. Clay Sci.*, **11**: 211-227.

Barshad, I. 1965. Thermal analysis techniques for mineral identification and mineralogical composition. *Methods of Soil Analysis*, American Society of Agronomy Monograph, No. 9, Ch. 50.

Beal, N. S. 1984. Direct determination of linear dimension versus moisture content relationship in expansive clays. *In Proceeding of the 5th International Conference of Expansive Soils*. Adelaide, Australia. pp. 62-66.

Bishop, A. W. 1959. The principle of effective stress. *Lecture delivered in Oslo, Norway*, in 1955; published in Teknisk Ukeblad, **106**(39): 859-863

Blight, G. E. 1967. Effective stress evaluation for unsaturated soils. *Soil Mechanics and Foundations*, **93**(2): 125-148.

Bocking, K. A. and Fredlund, D. G. 1980. Limitations of the axis translation technique. *In Proceedings of the 4th International Conference on Expansive Soils*, Denver. pp. 1-18.

Bonner, J. P. 1998. Comparison of predicted heave using oedometer test data to actual heave. *Master's Thesis,* Colorado State University, Fort Collins, Colorado.

Brasher, B. R., Franzmeier, D. P., Valassis, V. and Davidson, S. E. 1966. Use of saran resin to coat natural soil clods for bulk density and water retention measurements. *Soil Science*, pp. 101-108

Brackley, L. J. 1975. A model of unsaturated clay structure and its application to swell behavior. *In the Proceeding of the 6th African Conference Soil Mechanics and Foundation Engineering*, Durban. Balkema, Rotterdam, pp. 65-70.

Buckley, E. L. 1974. Loss and damage on residential slab-on-ground foundations. *Construction Research Center, Dept. Civil Eng., Univ. Texas Arlington, Rep. TR-2-74.*

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*. Tran. South African Institute of Civil Engineering.

Burland, J. B. 1964. Effective stress in partly saturated soils. discussion of "Some Aspects of Effective Stress in Saturted and Partly Saturated Soils. " by Blight, G. E. and Bishop, A. W., *Geotechnique*. **14**: 65-68.

Burland, J. B. 1984. Building on expansive soils. *In the Procedings 1st national conference on science and technology of buildings with special reference to buildings in hot climates.* Khartoum.

Burland, J. B. 1961. Discussion on collapsible soils, *Proceedings 5th International Conference on Soil Mechanics and Foundation Engineering,* Paris **3:** 219–220.

Cameron, D. A. 1989. Tests for reactivity and prediction of ground movement. *I. E. Aust. Civ. Eng. Trans.*, **31**(3): 121–132.

Cameron, D. A. and Walsh, P. F. 1984. Damage to buildings on clay soils. *Australian Council of National Trusts Technical Bull.*

CFEM. 2007. *Canadian Foundation Engineering Manual.* 4th ed. Canadian Geotechnical Society, published by BiTech Press, Richmond, B.C.

Casagrande, A. 1936. The determination of the preconsolidation load and its practical significance. *In Proceeding of 1st International Conference Soil Mechanics Foundation Engineering*. **3**: 60-64.

Chao, K. C., Overton, D. D. and Nelson, J. D. 2006. Design and installation of deep benchmarks in expansive soil. *Journal of Surveying Engineering*, **132**(3): 124–131.

Chao, K. C., Overton, D. D. and Nelson, J. D. 2006. The effects of site conditions on the predicted time rate of heave. *In Proceedings of the 4th International Conference on Unsaturated Soils*. Carefree, Arizona. April, pp. 2086 – 2097.

Chao, K. C. 2007. Design principles for foundations on expansive soils. *Dissertation submitted in partial requirement for the Ph.D. Degree,* Colorado State University, Fort Collins, Colorado.

Chen, F. H. 1965. The use of piers to prevent the uplifting of lightly loaded structures founded on expansive soil. *Concluding Proc. Eng. Effects of Moisture Changes in Soils, Int. Res. Eng. Conf. Expansive Clay Soils*, Supplementing the Symposium in Print, Texas A & M Press, pp. 152-171.

Chen, F. H. 1973. The basic physical property of expansive soils. *Proceeding of the 3rd of International Conference of Expansive Soil*. Haifa, Israel. **1**: 17-25.

Chen, F. H. 1975. *Foundations on ExpansiveSoils*. Elsevier Scientific Pub. Co., Amsterdam, New York, NY.

Chen, F. H. 1988. *Foundations on expansive soils*. American Elsevier Science Publ., New York.

Chen, F. H. and Ma, G. S. 1987. Swelling and shrinkage behaviour of expansive clays, *In the Proceedings 6th International Conference on expansive soils*, New Delhi, India, **1**. 127–129.

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

Clifton, A. W., Wilson, G. W. and Barbour, S. L. 1999. The emergence of unsaturated soil mechanics: Fredlund Volume. *NRC Research Press,* National Research Council Canada, Ottawa, Ont.

Clifton, A. W., Yoshida, R. T. and Fredlund, D. G. 1984. Performance of dark hall, Regina, Canada, constructed on a highly swelling clay. *In Proceeding 5th International Conference on Expansive Soils,* Adelaide, Australia, pp. 197-201.

Croney, D. and Coleman, J. D. 1954. Soil structure in relation to soil suction (pF). *Journal of Soil Science,* **5**(1): 163-177.

Croney, D., Coleman, J. D. and Black, W. P. W. 1958. Movement and distribution of water in soil in relation to highway design and performance. *National Research Council, Highway Research Board, Special Rep.* Washington, D. C **40**:226 – 252.

Das, B. M. 2008. *Advanced Soil Mechanics - 3rd edition,* Taylor and Francis.

De Bruijin, C. M. A. 1961. Swelling characteristics of a transported soil profile at Leeuhof Vereeniging (Transvaal). *Proc. 5th Int. Conf. Soil Mech. Found. Eng.* **1**: 43-49.

De Bruijin, C. M. A. 1965. Some observations on soil moisture conditions beneath and adjacent to tarred roads and other surface treatments in South Africa. *Moisture Equilibrium and Moisture Changes Beneath Covered Areas.* A Symposium in Print, Butterworths, Australia: pp. 135-142.

Delage, P., Le, T. T., Tang, A. M., Cui, Y. J. and Li, X. L. 2005. Suction effects in deep Boom Clay block samples. *Géotechnique,* **57**(2): 239 – 244.

Dif, A. E. and Bluemel, W. F. 1991. Expansive soils under cyclic drying and wetting. *Geotechnical Testing Journal*, **14**(1): 96-102.

Donaldson, G. W. 1965. A study of level observations on buildings as indications of moisture movements in underlying soils. *Moisture Equilibrium and Moisture Changes Beneath Covered Areas.* A Symposium in Print, Butterworths, Australia, pp. 156-164.

Donaldson, G. W. 1969. The occurence of problems of heave and the factors affecting its nature. *Proceedings 2nd International Conference on Expansive Clay.* Texas A&M, College Station, Texas

Donald, I. B. 1956. The mechanical properties of saturated and partly saturated soils with special reference to negative pore water pressure. *In the Proceeding of the 2nd Australia - New Zealand Conference Soil Mechanical Foundation Engineering*. Christchurch, New Zealand. pp. 200-205.

Dhowian, A. W., Erol, A. O. and Sultan, S. 1987. Settlement predictions in complex Sabkha profiles. *Bulletin of the International Association of Engineering Geology,* **36**: 11-21.

Dhowian, A. W. 1990. Field performance of expansive shale formation, *Journal of King Abdulaziz University (Engineering Sciences)*, **2**: 165–82.

Dhowian, A. W., Erol, A. O. and Youssef, A. 1990. *Evaluation of Expansive Soils and Foundation Methodology in the Kingdom of Saudi Arabia.*

Dregne, H. E. 1976. *Soils of Arid Regions*. Elsevier Sci. Pub. Co., New York.

Durkee, D. B. 2000. Active zone and edge moisture variation distance in expansive soils. *Dissertation submitted in partial requirement for the Ph.D. Degree, Colorado State University*, Fort Collins, Colorado.

Erol, A. O., Dhowian A. and Youssef, A. 1987. Assessment of oedometer methods for heave prediction. *In proceedings of 6th International Conference on Expansive Soils*, Technical Session III, pp. 99-105

Feng, M., Gan, K. M. and Fredlund, D. G. 1998. A laboratory study of swelling pressure using various test methods. *In Proceedings of International Conference on Unsaturated Soils*, Beijing, China, International Acedemic Publishers, **6**, 350-355.

Fityus, S. G. 1998. The influence of climate as expressed by the Thornthwaite index on the design depth of moisture change of clay soils in the Hunter Valley. *Conference on Geotechnical Engineering and Engineering Geology in the Hunter Valley*, Conference Publications, Springwood, Australia: 251–265.

Fityus, S. G., Delaney, M. D. and Smith, D. W. 2004. The reliability of ground movement predictions based on AS2870. *Proceedings of the 6th Triennial Conference on Mine Subsidence*, Maitland, pp. 125-135.

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

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

Fleureau, J. M., Verbrugge, J. C., Huergo, P. J. and Gomes Correia, A. 2002. Aspects of the behaviour of compacted clayey soils on drying and wetting paths. *Canadian Geotechnical Journal,* **39**: 1341-1357.

Fredlund, D. G. 1969. Consolidometer Test Procedural Factors Affecting Swell Properties. *In Proceedings, 2nd International Conference on Expansive Clay Soils*, Texas, 435 – 456.

Fredlund, D. G. and Morgenstern, N. R. 1976. Constitutive relations for volume change in unsaturated soils. *Canadian Geotechnical Journal*, **13**: 261 – 276.

Fredlund, D. G. and Morgenstern, N. R. 1977. Stress state variables for unsaturated soils. *Journal of the Geotechnical Engineering Division, ASCE,* **103**(5):

447-466.

Fredlund, D. G., Morgenstern, N. R. and Widger, R. A. 1978. Shear strength of unsaturated soils. *Canadian Geotechnical Journal*, **15**(3): 313-321.

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

Fredlund, D. G. 1979b. Second Canadian geotechnical colloquium: Appropriate concepts and technology for unsaturated soils. *Canadian Geotechnical Journal*, **16**: 121 – 139.

Fredlund, D. G and Hasan, J. U. 1979. One dimensional consolidation theory: unsaturated soils. *Canadian Geotechnical Journal*, **16**(3): 521-530.

Fredlund, D. G., Hasan, J. U., and Filson, H. 1980. The prediction of total heave. *Proceedings 4th International Conference on Expansive Soils*. Denver, Colorado, pp. 1 – 11.

Fredlund, D. G. 1983. Prediction of ground movements in swelling clays. *31st Annual Soil Mechanics and Foundation Engineering Conference*. University of Minnesota, Minneapolis.

Fredlund, D. G. 1987. The prediction and performance of structures on expansive soils. *In Proceedings of International Symposium on Prediction and Performance in Geotechnical Engineering*, pp. 51-60.

Fredlund, D. G. and Rahardjo, H. 1993. *Soil Mechanics for Unsaturated Soil.* John Wiley & Son, Inc., New York, NY.

Gibbs, H. J. 1973. Use of a consolidometer for measuring expansion potential of soils. *Proc. Workshop Expansive Clays and Shales in Highway Design and Construction.* Univ. Wyoming, Laramie, May, 1: 206-213.

Grossman, R. B., Brashner, B. R., Franzmeier, D. P. and Walker, J. L. 1968. Linear extensibility as calculated from natural-clod bulk density measurement. *Soil science of Science American.* **32**(4): 570-573.

Grim, R. E. 1953. *Clay mineralogy*, McGraw-Hill, New York.

Grim, R. E. 1968. *Clay mineralogy*. McGraw-Hill Book Company, New York.

Haines, W. B. 1923. The volume changes associated with variations of water content in soil. *Journal of Agr. Science.* **13**: 296-310.

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

Hamberg, D. J. 1985. A simplified method for predicting heave in expansive soils. *M. S. thesis,* Colorado State University, Fort Collins, CO.

Hamilton, J. J. 1965. Shallow foundations on swelling clays in western Canada. *Proceedings, The International Research and Engineering Conference on Expansive Clay Soils*, Texas A & M University, College Station, TX, pp. 183-207.

Hamilton, J. J. 1968. Effects of natural and man-made environments on the performance of shallow foandations. *Proceedings, 21st Annual Canadian Soil Mechanics Conference*, Winnipeg, Man.

Hamilton, J. J. 1969. Effects of environment on the performance of shallow foundations. *Canadian Geotechnical Journal*, **6**: 65-80.

Hamilton, J. J. 1977. Foundations on swelling or shrinking subsoils. *Canadian Building Digest*, CBD-184.

Hilf, J. W. 1956. An investigation of pore-water pressure in compacted cohesive soils. *Ph.D. Thesis*, Technical Memorandum No. 654, U.S. Department of the Interior, Bureau of Reclamation, Design and Construction Division, Denver, CO.

Ho, D. Y. F., Fredlund, D. G. and Rahardjo, H. 1992. Volume change indices during loading and unloading of an unsaturated soil. *Canadian Geotechnical Journal*, **29**: 195-207.

Holland, J. E. and Lawrence, C. E. 1980. Seasonal heave of Australia clay soils. *In the Proceeding of the 4^th^ International Confernece on Expansive Soils*, ASCE, New York, 302–321.

Holtz, W. G. 1959. Expansive clays – properties and problems. *Quart. Colorado School Mines*, **54**(4): 89-677.

Holtz W. G. and Gibbs H. J. 1956. Engineering properties of expansive clays. *Transactions of the ASCE*. **121**: 641–677.

Jaksa, M. B., Cavagnaro, R. L. and Cameron, D. A. 1997. Uncertainties Associated with the visual –tactile method for quantifying the reactivity of expansive soils. *Australian Geomechanics*. **31**:84-91.

Jennings, J. E. 1961. A revised effective stress law for use in the prediction of the behavior of unsaturated soils. *In Proceedings of the Conference on Pore Pressure and Suction in Soils,* Butterworths, London, pp. 26-30.

Jennings, J. E. B. 1961. A Revised Effective Stress Law for Use in the Prediction of the Behavior of Unsaturated Soils. *Pore Pressure and Suction in Soils,* Butterworths, London, pp. 26 – 30.

Jennings, J. E. B. and Knight, K. 1957. The prediction of total heave from the double oedometer Test. *Proceedings of Symposium on Expansive Clays*, S. African Inst. Civil Eng., Johannesburg. **7**(9): 13 – 19.

Jennings, J. E. B., Firtu, R. A., Ralph, T. K. and Nagar, N. 1973. An improved method for predicting heave using the oedometer test. *Proc. 3rd Int. Conf. Expansive Soils*, Haifa, Israel, **2**: 149-154.

Johnson, L. D. 1969. Review of literature on expansive soils. *U. S. Army Eng. Waterways Exp. Sta.,* Vicksburg, MS, Misc. Paper S-73-17.

Johnson, B. J. 1973. Establishment of centipedegrass and St. Augustine grass with the aid of chemicals. *Agron. J.* **65**:959–962.

Johnson, L. D. 1977. Evaluation of laboratory suction tests for prediction of heave in foundation soils. *Army Eng. Waterways Exp. Station, Vicksburg, MS*, Rep. WES-TR-S-77-7, August.

Johnson, L. D. 1978. Predicting potential heave and heave with time in swelling soils. *U. S. Army Eng. Waterways Exp. Station, Vicksburg, MS, Tech.* Rep. S-78-7.

Johnson, L. D. 1979. Overview for design of foundations on expansive soils. *U. S. Army Eng. Waterways Exp. Sta., Vickburg, MS, Misc.* Paper GL-79-21.

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

Johnson, L. D. and Stroman, W. R. 1976. Analysis of behaviour of expansive soil foundations. *Army Eng. Waterways Exp. Sta.*, Vicksburg, MS, Rept. No. WES-TR-S-76-8, June.

Jones, D. E. and Holtz, W.G. 1973. Expansive soils – hidden disaster. *Civil Engineering: American Society of Civil Engineers*, **43**(8): 49-51.

Jury, W. A., Gardner, W. R. and Gardner, W. H. 1991. *Soil Physics*, 5th Edition. John Wiley & Sons, Inc., New York, NY.

Kassiff, G., Livneh, M. and Wiseman, G. 1969. Pavements on expansive clays.

Jerusalem Academic Press, Jerusalam, Israel.

Kassif, G. and Baker, R. 1971. Aging effects on swell potential of compacted clay. *J. Soil Mechanics and Foundation Div*., ASCE, SM 3: 529-540.

Khalili, N. and Khabbaz, M. H. 1998. A unique relationship for the determination of the shear of unsaturated soils. *Geotechnique*, **48**(5): 681-687.

Komornik, G. Livneh, M and Wiseman, G. 1969. Pavements on expansive clays. *Jerusalem Academic press,* Jerusalem, Isreal.

Komornik, A. and Zeitlen, J. G. 1970. Laboratory determination of lateral and vertical stresses in compacted swelling clay, *J. Material,s* **5:**108–128.

Komornik, A. and Zeitlen, J. G. 1979. Effect of swelling clay on piles. *Proceedings of the Eight International Conference on Soil Mechanics and Foundation Engineering*, Moscow, USSR, pp. 123-128.

Komornik, A. and David, D. 1969. Prediction of swelling pressure of clays. *Journal of the Soil Mechanics and Foundations Division, ASTM,* **95**: 209-225.

Krazynski, L. M. 1980. Expansive soils in highway construction-some problems and solutions. *The 4th Int. Road Fed. African Highways Conf.*, Nairobi, Kenya, January.

Kumar, C. P. 2000. Variation of soil moisture characteristics in a part of Hindan river catchment. *Technical Report*. National Institute of Hydrology.

Lambe, T. W. 1960b. The character and identification of expansive soils, soil PVC meter. *Federal Housing Administration*, Technical Studies Program, FHA 710.

Lambe, T. W. and Whitman R.V. 1979. *Soil Mechanics.* New York: Wiley.

Leroueil, S. and Vaughan, P. 1990. The general and congruent effects of structure in natural soils and weak rocks. *Géotechnique,* **40**: 467–488.

Li, X. D., Coles, B. J., Ramsey, M. H. and Thornton, I. 1995. Sequential extraction of soils for multielement analysis by ICP–AES. *Chem. Geol.* **124**: 109–123.

Li, J., Smith, D. W., Fityus, S. G. and Sheng, D. C. 2003. The numerical analysis of neutron moisture probe measurements. *Int. J. Geomech.*, **3**, 11–20.

Likos, W. J., Lu, N. and Sharkey, K. J. 2005. Laboratory characterization of steeply dipping expansive bedrock in the Rocky Mountain Front Range. *Journal of Geotechnical and Geoenvironmental Engineering*. **131**(9): 1162-1171.

Likos, W. J. and N. Lu. 2003. Automated humidity system for measuring total suction characteristics of clay. *Geotechnical Testing Journal*, **26**, 179–190.

Lloret, A., Villar, M. V., Sanchez, M., Gens, A., Pintado, X. and Alonso, E. E. 2003. Mechanical Behavior of heavily compacted bentonite under high suction changes. *Géotechnique*, **53**(1), 27-40.

Lyklema, J. 2000. *Fundamentals of Interface and Colloid Science*, **3**: Liquid-Fluid Interfaces, Academic Press.

Lytton, R. L. 1977. Foundations in expansive soils. *Numerical Methods in Geotechnical Engineering*. C.S. Desai and J.T. Christian, (Eds.), McGraw Hill Bokk Company.

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

Lytton, R. L. and Woodburn, J. A. 1973. Design and performance of mat foundation on expansive clay. *Proc. 3rd Int. Conf. Expansive Soils*. Haifa, Isreal 1: 301-308.

Masia, M. J., Totoev, Y. Z. and Kleeman, P. W. 2004. Modeling expansive soil movements beneath structures, *Journal of Geotechnical and Geoenvironmental Engineering,* **130**(6): 572–579.

Matyas, E. L. and Radhakrishna, H. S. 1968. Volume change characteristics of partially saturated soils. *Géotechnique,* **18**(4)**:** 432-448.

Maksimovic, M. and Tonkovic, *N.* 1987. Evaluation of the swelling potential and derivation of design parameters for the large site. *In the proceding of the 6th International Conference on Expansive Soils.* India.

McCrone, W. C. and Delly, J. G. 1973. *The particle Atlas*, 2nd ed., Principles and Techniques. Ann Arbor Science, Ann Arbor, MI.

Mckeen R. G. 1980. Field study of airport pavements on expansive clay. *In the proceding of the 4th International Conference on Expansive Soils*, Denver, Colorado, pp. 242–61.

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

McKeen, R. G. 1985. *Validation of Procedures for Pavement Design on Expansive Soils.* U.S. Dept. of Transportation, Federal Aviation Administration, Final Report.

McKeen, R. G. 1990. Climate-controlled soil design parameters for mat foundations. *J. Geotech. Eng.*, **116**(7): 1073–1094.

McKeen, R. G. 1992. A model for predicting expansive soil behavior. *Proceedings of 7th International Conference on Expansive Soils*, Dallas, Texas. **1**: 1 – 6.

McKeen, R. G. and Hamberg, D. J. 1981. Characterization of expansive soils. *Trans. Res. Rec. 790, Trans. Res. Board*, pp. 73 – 78.

McKeen, R. G. and Nielsen, J. P. 1978. Characterization of expansive soils for airport pavement design. *U. S. Department of Transportation, Federal Aviation Administration*, FFA Report No. FAA-RD-78-59.

Mesri, G., Lo, D. O. K. and Feng, T. W. 1994. Settlement of embankments on soft clays. *Proceedings of Settlement 94, College Station,Texas. American Society of Civil Engineers, GeotechnicalSpecial Publication* **40**(1): 8-56.

Mitchell, J. K. 1976. *Fundamentals of soil behavior*, John Wiley & Sons.

Mitchell, J. K. and Raad, L. 1973. Control of volume changes in expansive earth materials. *In the Proceeding of the Workshop of Expansive Clays and Shales in Highway Design and Construction.* Federal Highway Administration, Denver. pp. 200-219.

Mitchell, P. W. 1979. The structural analysis of footings on expansive soil. *Research Rpt. No. L. K. W. G.* Smithh & Assoc., Newton.

Mitchell, P. W. 1984. A simple method of design of shallow footings on expansive soil. *In the Proceeding of the 5th International Conference on Expansive Soils.* Australia.

Mitchell, P. W. 1989. Site investigation processes. *Course on Footings for Small Scale and Domestic Structures*, Linn Education and Training Services, pp. 89-241.

Mitchell, P. W. and Avalle, D. L. 1984. A Technique to predict expansive soils movement. *In Proceeding 5th International Conference on Expansive Soils*, Adelaide, Australia, pp. 124-130.

Miller, D. J., Durkee, E. B., Chao, K. C. and Nelson, J. D. 1995. Simplified heave prediction for expansive soils. *Proc. 1st intern. Conf. on Unsat. Soils,* pp. 891–897. Balkema, Rotterdam.

Morris, P. O. and Gray, W. T. 1976. Moisture conditions under roads in the Astrilian environment. *Rep. 68, Aust. Road Research Board.*

Navarro, V. and Alonso, E. E. 2001. Secondary compression of clays as a local de-hydration process. *Géotechnique*. **51**(10): 859-869.

Navy, Dept. of, Naval Facilities Engineering Command. 1971. *Design Manual-Soil Mechanics, Foundations and Earth Structures*. U. S. Naval Publications and Forms Center, NAVFAC DM-7.

Nayak, N. V. and Christensen, R. W. 1971. Swell characteristics of compacted expansive soils. Clay and Clay Minerals. **19**(4): 251–261.

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

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

Nelson, J. D., Overton, D. D. and Durkee, D. B. 2001. Depth of wetting and the active zone. *Expansive Clay Soils and Vegetative Influence on Shallow Foundations, ASCE*, Houston, Texas. pp. 95 – 109.

Nelson, J. D. and Chao, K. C. 2003. Design of foundations for light structures on expansive soils. *California Geotechnical Engineers Association Annual Conference*, Carmel, California.

Nelson, J. D., Reichler, D. K. and Cumbers, J. M. 2006. Parameters for heave prediction by oedometer tests. *In the proceedings of the 4th International Conference on Unsaturated Soils*. Carefree, Arizona, pp. 951 – 961.

Nelson, J. D., Chao, K. C. and Overton, D. D. 2007a. Definition of expansion potential for expansive soils. *Proceedings of the 3rd Asian Conference on Unsaturated Soils*. Nanjing, China.

Nelson, J. D., Chao, K. C. and Overton, D. D. 2007b. Design of pier foundation on expansive soils. *Proceedings of the 3rd Asian Conference on Unsaturated Soils.* Nanjing, China. April.

Newland, P. L. 1965. The behavior of soils in terms of two kinds of effective stress. *Proceedings of the Conference on Expansive Soils*, pp. 78-92.

Nobel, C. A. 1966. Swelling measurements and prediction of heave for a lacustrine clay. *Can. Geotechn. J.* **3**(1): 32-41.

Olsen, R. E. and Langfelder, L. I. 1965. Pore water pressures in unsaturated soils, *Journal of Soil Mechanics and Foundation Division, ASCE.* **91**: 127–150.

Olsen, H. W., Krosley, L., Nelson, K., Chabrillat, S., Goetz, A., and Noe, D. C. 2000. Mineralogy-swelling potential relationships for expansive shales. *Advances In Unsaturated Geotechnics*, ASCE, Denver, pp. 361–378.

Osman, M. A. and Sharief, A. M. E. 1987. Field and laboratory observations of expansive soil heave. *In Proceedings of 6th International Conference on Expansive Soils*, pp. 105 – 110. New Delhi, India.

Overton, D. D., Chao, K. C. and Nelson, J. D. 2006. Time rate of heave prediction for expansive soils. *GeoCongress 2006, ASCE,* Atlanta. pp. 1–6.

Palit, R. M. 1953. Determination of swelling pressure of black cotton soil. In *Proceeding* of 3rd *International Conference on Soil Mechanics and Foundation Engineering*. Switzerland.

Perko, H. A., Thompson, R. W. and Nelson, J. D. 2000. Suction compression index based on CLOD test results. *Proc. Sessions of Geo-Denver 2000, Geotechnical Spec. Publ. ASCE,* **99**: 393–408.

Porter, A. A. and Nelson, J. D. 1980. Strain controlled testing of soils. *Proc. 4th Int. Conf. Expansive Soils*, ASCE and Int. Soc. Soil Mech. Found. Eng., Denver, pp. 34-44.

Poulos, H. G. and Davis, E. H. 1980. *Pile Foundation Analysis and Design.* John Wiley & Sons, Inc., New York, NY.

Ranganathan, B. V. and Satyanarayana, B. 1965. A rational method of predicting swelling potential for compacted expansive clays: In *Proceedings of the 6th International Conference on Soil Mechanics and Foundation Engineering*, International Society for Soil Mechanics and Geotechnical Engineering, London, **1**: 92–96.

Retamal, E., Ortigosa, P. and Acevedo, P. 1987. Swelling prediction compared with in-situneasurement in Santiago, Chile. *In the proceding of the 6th International Conference on Expansive Soils.* India.

Reed, R. F. 1985. Foundation performance in an expansive clay. *Theory and Practice in Foundation Engineering, 38th Canadian Geotechnical Conference*, Edmonton, Alberta.

Richards, B. G. 1966. Measurement of the free energy of soil moisture by the Psychrometric technique using thermistors. *Moisture equilibria and moisture changes in soils beneath covered areas*, Sydney, Australia, pp. 39 – 46.

Richards, B. G. 1976. The significance of moisture flow and equilibria in unsaturated soils in relation to the design of engineering structures built on shallow foundations in Australia. *Symp. Permeability Capillarity, ASTM,* Atlantic City, NJ.

Ridgway, K. D., Sweet, A. R. and Cameron, A. R. 1996. Climatically induced floristic changes across the Eocene-Oligocene transition in the northern high latitudes, Yukon Territory, Canada. *Geological Society of America Bulletin*. 107: 676-696.

Rogers, J. D., Crane, K. M. and Snyder, D. L., 1993. Damage to Foundations from Expansive Soils. *Claims People*, **3**(4): 1-4.

Romero, E., Hoffmann, C. and Alonso, E. E. 2003. Permeability and pore size distribution changes of a bentonite enhanced sand. *13th European Conference on Soil Mechanics and Geotechnical Engineering. GEOPROC_2003*, Prague.

Russam, K. 1961. Estimation of subgrade moisture distribution. *Transportation and Communication Monthly Review*. **176**: 151-159.

Russam, K. 1965. The prediction of subgrade moisture conditions for design purpose. Moisture Equilibria and Moisture Change on Soils Beneath Covered Areas. Butterworth, Austrilia. pp. 233-236.

Salas, J. A. J. and Serratosa, J. M. 1957. Foundations on Swelling Clays. *In Proceeding 4th International Conference on Soil Mechanics and Foundation Engineering.* London, England, **1**: 424-428.

Sampson, E., Schuster, R. L. and Budge, W. D. 1965. A method of determining swell potential of an expansive clay. *Concluding Proc. Engineering Effects of Moisture Changes in Soils. Int. Res. Eng. Conf. Expansive Clay Soils*. Supplementing the Symposia in Print. Texas A & M Univ. Press, College Station, TX, pp. 255-275.

Schneider G. L. and Poor, A. R. 1974. The prediction of soil heave and swell pressures developed by an expansive clay, *Research Report*, No: TR-9-74, Construction Research Center, Univ. of Texas.

Seed, H. B., Woodward, R. J., Jr and Lundgren, R. 1962. Prediction of swelling potential for compacted clays, *Journal of Soil Mechanics and Foundation Engineering Division ASCE*, **88**: 53–87.

Shanker, N. B., Ratnam M. V. and Rao A. S. 1987. Multi-dimensional swell behaviour of expansive clays. *Proc. 6th Int. Conf. Expansive Soils*, New Dehli, India.

Sheng, D., Fredlund, D. G. and Gens, A. 2007. A new modelling approach for unsaturated soils using independent stress variables, *Canadian Geotechnical Journal*.

Sheng, D., Wriggers, P. and Sloan, S. W. 2007. Application of frictional contact in geotechnical engineering, *International Journal for Geomechanics*, ASCE, **7**(3): 176-185.

Smith, A. W. 1973. Method for determining the potential vertical rise, PVR. *Proc. Workshop Expansive Clays and Shales in Highway Design and Construction.* Univ. of Wyoming, Denver, CO, pp. 245-249.

Snethen, D. R. 1979. An evaluation of methodology for prediction and minimization of detrimental volume change of expansive soils in highway subgrades. *Vol. I, FHWA-79-49*, Federal Highway Administration, Washington, D. C.

Snethen, D. R. 1979. An evaluation of methodology for prediction and minimization of detrimental volume change of expansive soils in highway subgrades. *Vol. II, FHWA-79-50*, Federal Highway Administration, Washington, D. C.

Snethen, D. R. 1980. Characterization of expansive soils using soil suction data. *In Proceeding 4th International Conference on Expansive Soils*, Denver, Colorado, pp. 54–75.

Snethen, D. R. and Huang, G. 1992. Evaluation of Soil Suction-Heave Prediction Methods. *In Proceeding of 7th International Conference on Expansive Soils*, Texas, pp. 12-17.

Snethen, D. R. and Johnson, L. D. 1977. Characterization of expansive soil subgrades using soil suction data. *Presented at Moisture Influence on Pavement Materials – Characterization and performance Conference Section at 56th Annual Transportation Research Board Meeting*, Washington, D. C.

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

Snethen, D. R., Johnson, L. D. and Patrick, D. M., 1977. *An evaluation of expedient methodology for identification of potentially expansive soils.* Soil and Pavements Laboratory, U.S. Army Eng. Waterway Exp. Sta., Vicksburg, MS, Rep. No. FHWA-RE-77-94, NTIS PB-289-164.

Spangler, M. G. and Handy, R. L. 1982. *Soil Engineering.* 4th edition. Harper & Row, New York, NY, pp. 729–741.

Sridharan, A., Sreepada Rao, A. and Sivapullaiah, P. V. 1986. Swelling pressure of clays. *Geotechnical Testing Journal*, *GTJODJ*, **9**(1): 24-33.

Standards Australia 1996. *Residential slab and footings*, AS 2870. Sydney, Australia.

Subba Rao, K. S. and Tripathy, S. 2003. Effect of aging on swelling and swell-shrink behaviour 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. *Proc. 2nd Int. Res. Eng. Conf. Expansive Soils*. Texas A & M Univ. Press, College Station, TX, pp. 404-420.

Taylor, D. W. 1948. *Fundamentals of Soil Mechanics.* John Wiley & Sons, Inc., New York.

Teng, T. C. P. and Clisby, M. B. 1975. Experimental work for active clays in Mississippi. *Transport. Eng. J.* ASCE 101 (TEI): 77-95.

Teng, T. C. P., Mattox, R. M. and Clisby, M. B. 1972. A study of active clays as related to highway design. *Research and Development Division*, Mississippi State Highway Dept., Engineering and Industrial Research Station, Mississippi State University, MSHD-RD-72-045, pp: 134.

Teng, T. C. P., Mattox, R. M. and Clisby, M. B. 1973. Mississippi's experimental work on active clays. *Proc. Workshop on Expansive Clays and Shales in Highway design and Construction*, Univ. of Wyoming, Laramie, May, **2**: 1-17.

Terzaghi, K. 1936. The shear resistance of saturated soils. *In the Proceeding of the 1st International Conference of Soil Mechanical Foundation Engineering*. Cambridge, MA. **1**: 54-56.

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

Tripathy, S., Subba Rao, K. S. and Fredlund, D. G. 2002. Water content - void ratio swell - shrink paths of compacted expansive soils. *Canadian Geotechnical Journal,* **39**: 938-959.

Uppal, H. L. 1965. Field study on the movement of moisture in black cotton soils under road pavements, *Symp. Moisture Equilibria and Moisture Changes in Soils Beneath Covered Areas*, Butterworths, Astralia, pp. 686-693.

U.S. Department of the Army. 1983. *Engineering and Design – Settlement Analysis.* Washington, DC.

Vanapalli, S. K., Pufahl, D. E. & Fredlund, D. G. 1999. Relationship between soil-water characteristic curves and the as-compacted water content versus soil for a clay till. In *Proceedings of the XI Pan-American Conference on Soil Mechanics and Foundation Engineering*, Brazil, **2**, 991-998.

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.

Vijayavergiya, V. N. and Ghazzaly, O. I. 1973. Prediction of swelling potential for natural clays. *In Proceedings of Third International Conference on Expansive Soils*. Haifa, Israel, **1**, 227-236.

Vu, H. Q. and Fredlund, D. G. 2004. The prediction of one-, two-, and three-dimensional heave in expansive soils. *Canadian Geotechnical Journal* **41**(4): 713–737.

Walsh, P. F. and Cameron, D. A. 1997. The design of residential slabs and footings. *Standards Australia,* SAA HB28-1997.

Weston, D. J. 1980. Expansive roadbed, treatment for Southern Africa. *In the proceeding of the 4th International Conference on Expansive Soils*, **1**: 339-360.

Wise, J. R. and Hudson, W. R. 1971. An esamination of expansive clay problems in Texas. *Center for Highway Research*, University of Texas, austin, Res. Rep. 118-5.

Whittig, L. D. 1964. X-ray diffraction techniques for mineral identification and mineralogical composition. *Methods of Soil Analysis*, American Society of Agronomy Monograph, No. 9, Part I, Ch. 49.

Wong, H. Y. and Yong, R. M. 1973. A study of swelling and swelling force during unsaturated flow in expansive soils. *Proc. 3rd Int. Conf. Expansive Soils*, Haifa, Israel, **1**: 143-151.

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 Engineerin,g* **131**(3):311–324.

Yoder, E. J. D and Witczak, M. W. 1975. *Principles of Pavement Design.* 2nd Edition. New York: John Wiley and Sons Incorporation.

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.