
A UNIVERSAL METHOD FOR ASSESSING INTRINSIC EXPANSIVENESS OF SOILS

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ABSTRACT

Many of the attempts made over the past six decades to find a universal system for assessing expansiveness of soils using soil index data have failed to follow the basic principles of soil mechanics. By overcoming most of these limitations Gourley and Schreiner (1993a) developed a new procedure that allows comparison of intrinsic expansiveness of soil samples prepared to have stable micro-fabric and consistent stress history. In this research, the same procedure is used on twenty-seven natural clayey soil samples of varying geological, geomorphological and geographical origin obtained from Kenya, Sudan, Eritrea and South Africa. Each of these samples was tested for Atterberg limits, volume change behaviours and soil suction. Statistical analysis was conducted on different soil parameters derived from these tests to obtain a significant relationship with their intrinsic expansiveness using measured swell. The analysis confirmed that most of the significant relationships obtained contain swell index, C^*_s , showing the identicalness of the soil properties responsible for volume change behaviour of both saturated and unsaturated clayey soils. Depending on the cost and the significance, the analysis recommended three major models that can be used as a screening system in the assessment of intrinsic expansiveness. For any soil it is possible to obtain preliminary information regarding its intrinsic expansiveness using the cheapest of the recommended models that needs liquid and plastic limit tests and hydrometer analysis, which are the routine tests of geotechnical site investigation. A more detailed assessment can be achieved by including only the shrinkage test. The most reliable assessment needs addition of consolidation test with the unloading stage. All of the models allow obtaining information regarding the intrinsic expansiveness of soils as early as site investigation stage for successful engineering design. Moreover, they are anticipated to promote worldwide exchange of information regarding these problematic soils.

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List of Symbols

$^{\circ}\text{C}$:	Degree centigrade
C_c^*	:	Intrinsic compressibility (Burland 1990)
C_s^*	:	Intrinsic swell index
C.F.	:	Clay fraction
DEL	:	Double electrical layer
ER	:	Eritrea
e_L	:	Void ratio at the liquid limit
e_p	:	Void ratio at the plastic limit
e_{SL}	:	Void ratio at the shrinkage limit
e_{15}	:	Void ratio after swelling under 15 kPa vertical stress
Δe	:	Change in void ratio during swelling under 15 kPa vertical stress
G_s	:	Specific gravity of a soil
H	:	Sample thickness
H_0	:	Sample thickness before swelling
$\Delta H/H_0$:	Vertical strain during swelling under 15 kPa vertical stress
ICL	:	Intrinsic compression line
ISL	:	Intrinsic swelling line
e_{100}^*	:	Void ratio corresponding to 100 effective stress in ICL
e_{1000}^*	:	Void ratio corresponding to 1000 effective stress in the ICL
e_{100}^s	:	Void ratio corresponding to 100 effective stress in the ISL
I_p	:	Plasticity index
I_s	:	Shrinkage index
KE	:	Kenya
kPa	:	Kilopascal or kN/m^2
MPa	:	Mega Pascal
OCR	:	Over consolidation ratio
SA	:	South Africa
S_r	:	Saturation ratio
SU	:	Sudan
u_a	:	Pore-air pressure
u_w	:	Pore-water pressure
u_{SL}	:	Suction at shrinkage limit
w_L	:	Liquid limit
w_p	:	Plastic limit
w_{SA}	:	Shrinkage limit according to ASTM procedure
w_{SB}	:	Shrinkage limit according to HMSO procedure
μ	:	Micrometers
ε_{ex}	:	Expansive strain on soaking
σ'_v	:	Axial or vertical effective stress
\AA	:	Armstrong
Ψ	:	Total suction of a soil
π	:	Osmotic suction
$(u_a - u_w)$:	Matric suction
%	:	Percentage
$^{\circ}$:	Degree of an angle

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Declaration

In conformity with the regulations of the University of KwaZulu-Natal, I hereby state that the work submitted in this paper is my own work, unless specifically indicated to the contrary in the text. Further more this dissertation has not been submitted to any other University for any purpose whatsoever.

"When utilizing past experience in the design of a new structure we proceed by analogy and no conclusion by analogy can be considered valid unless all the vital factors involved in the cases subject to comparison are practically identical. Experience does not tell us anything about the nature of these factors and many engineers who are proud of their experience do not even suspect the conditions required for the validity of their mental operations. Hence our practical experience can be very misleading unless it combines with it a fairly accurate conception of the mechanics of the phenomena under consideration." Karl Terzaghi

CHAPTER ONE

1. GENERAL INTRODUCTION

1.1 BACKGROUND

Expansive soils are natural clayey soils that due to their distinct intrinsic characteristics are subject to excessive volume change with changes in water content or suction. Suction results in the moving out of water and a decrease in volume of the soil. On the other hand, addition of water leads to decrease of the suction and an increase in volume of the soil. The cause of the volume changes is the interaction between the clay minerals of the soil and the soil water. Intrinsically expansive soils typically contain clay minerals that attract and absorb water. When water is added to these expansive clays, the water molecules enter between clay plates. As more water is absorbed, the plates are forced further apart, leading to an expansion of the soil's volume. Similarly, on drying the water molecules will leave the clay plates resulting in shrinkage of the soil, and most probably leaving behind cracks.

The ability of a clay mineral to absorb and adsorb water is an intrinsic property of the clay. It is not altered by moisture content or suction, which exists at a particular time. The intrinsic expansiveness of a clayey soil results from its mineral composition and grading and its interaction with water (Schreiner 1999). In summary, intrinsic expansiveness is the soil property that relates the change in water content, and thus volume change, to the suction change. In natural clayey soils the relationship is not linear due to the soil's stress history, fabric, etc. A clayey soil with a high intrinsic expansiveness will exhibit large water content or volume change as compared with a clayey soil of low intrinsic expansiveness when both are subjected to the same change of suction under identical initial stresses, suction, micro-fabric and stress history conditions.

Expansive soils are known for their severe damage to buildings, road pavements, pipelines, irrigation channels, and other structures founded on them. The overall destructive impact of such clayey soils exceeds most

natural hazards (Chen 1988). Even the most conservative estimate, although subjective, shows that expansive soils are major contributors to the burden that natural hazards place on the economy (Nelson and Miller 1992). The importance of accurate identification of expansive soil as early as possible, in the site investigation stage, to minimize not only failures but also costly remedial works is evident from this.

Geotechnical engineers started to recognize the problem of expansive soils starting in the late 1930's. Prior to this time, damages were assumed to be due to poor construction and settlement of foundation at one corner without recognition of the role of expansive soils (Chen 1988). Since that time, however, geotechnical engineers started to realize that the cause was other than settlement. With increasingly intensive use of concrete slab on ground construction, damage to structures caused by expansive soils has further increased. This increased the worldwide concern to know the behaviour of these soils for proper design. The concerns lead to international conferences about the soils since the beginning of 1960's. The conferences resulted in considerable progresses in the understanding of the nature of expansive soils. Several attempts have also been made to improve the performance of expansive soils in the field with greater emphasis on design criteria and construction precautions for structures founded on them. Unfortunately present day knowledge of expansive soils has not reached a stage at which rational solutions can be assigned to the problem. Still there is only limited understanding of expansive soils behavior and soil-structure interaction.

One of the bottlenecks for the limited knowledge available about the soils is lack of a proper identification, and thus classification, of expansive soils, a tool that classifies clay soils purely on the basis of intrinsic expansiveness regardless of the stress history, fabric, etc. This tool would be a useful means of determining the risk related to any clayey soil and selecting economical remedial works for structures already founded on them in order to avoid or minimize further distress.

In the last six decades in the attempt of formulating a satisfactory identification and classification system, many researchers, from all over the world, have recommended different methods. Among others, the

contributions of Altmeyer (1955); Holtz and Gibbis (1956); Seed *et al* (1962); van der Merwe (1964); Ranganatham and Satyanarayana (1965); Raman (1967) (quoted in Djedid *et al* 2001); Dakshanamurthy and Raman (1973); Holtz *et al* (1973); Weston (1977); Brackley (1979); BRE (1980); Williams and Donaldson (1980); Snethen (1984); Williams *et al* (1985); Pidgeon (1987); Chen (1988); and McKeen (1992, 2001) can be mentioned as examples. These researchers produced a number of empirical models correlating soil state and classification parameters with swell behaviour.

Unfortunately, the success of the models in classifying expansive soils and predicting swell has been limited to the specific sets of soil data from which they are derived (Oloo *et al* 1987 and Schreiner 1987a). A universally accepted method of classifying is still required (Oloo *et al* 1987, Schreiner 1987a and Schreiner 1988). As summarized by Schreiner (1987a) and Oloo *et al* (1987), the problem common to most of these attempts has been:

- Most of the authors have failed to follow the basic principles of soil mechanics. They have been using either compacted or undisturbed samples of natural soils, both of which include an unknown stress history and unknown micro-fabric in the samples. The specific stress, and suction or moisture content conditions of testing hampers comparison between soils. Thus, applying these methods under conditions different from those for which they have been originally developed will almost always lead to false predictions.
- Research by Schreiner and Burland (1990) shows that micro-fabric has also major effects in testing for swell in the laboratory and can cause collapse to occur during swell testing. Such an unknown factor has never been considered in the development of the models.
- Uniform soil state conditions have not always been used as the basis of comparing swell for different soil samples. This applies to the initial density and water content of soil samples at the start of testing.
- Most of these models have been derived on the basis of numbers of swell data too small to be conclusive.

- Adequate representation of clayey soils of variable geological, geographical and geomorphologic origin in the derivation of the models has been rare.
- Exhaustive analysis has never been done to identify classification parameters relevant to intrinsic expansiveness.

1.2 THE NEED FOR RESEARCH

It is clear from the preceding discussions that expansive soil engineering is still missing an important tool that allows comparison of expansiveness of soils regardless stress history and micro-fabric. It was with this concern that a research has been going on starting from the late 1980's by Schreiner and other co-researchers.

After identifying the limitations of all the previous attempts a forward step has been made. A new procedure of sample preparation was developed that subjects reconstituted samples of clayey soils to identical stress history and stable micro-fabric prior to testing for volume change so as to allow pure comparison of expansiveness between samples (Gourley and Schreiner 1993a). The same researchers employed a small group of clayey samples to investigate for a significant relationship between intrinsic expansiveness and index test data. Other than these, sufficient samples representative of different geological geographical and geomorphological origin have not yet been tested for the development of a reliable model that can, potentially, be used universally. This research is designed to answer these limitations.

1.3 PROBLEM STATEMENT

The problem to be addressed in this research can be stated as:

With the growing need of people to use expansive soils as founding ground a universal means of estimating intrinsic expansiveness of clays is required for better understanding and worldwide sharing of knowledge about the soils.

1.4 HYPOTHESIS

The hypothesis to be examined in this research is that intrinsic expansiveness of clayey soils can be estimated from the results of one or more index tests.

1.5 ORGANIZATION

The dissertation is organized into seven chapters. The first chapter introduced the over-all problem situations and define the problem statement, and hypothesis of the study. Chapter 2 of the dissertation will present the literature review. A detailed discussion will be given regarding intrinsic expansiveness, swelling, and heave. This will form the basis for the planning of the research. Chapter 3 will describe the planning of the research. It specifies the motivation, scope, purpose, and objective of the research. The methodology employed in testing and analysing data obtained from the tests will be described in detail in chapter 4. The results obtained will be presented in chapter 5 and discussed in chapter 6. Finally, chapter 7 will draw conclusions and recommendations from the research.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 INTRODUCTION

2.1.1 Background

The engineering significance of expansive soils is due to their distressing and thus destructive impact on the structures built on them. The distressing effect arises from the volume change behavior of the soils with change in their moisture content. The damage caused by the soils is worth billions of Rands annually. Although some progress has been achieved toward understanding the behavior of these soils, the present day knowledge has not reached a stage at which an absolute solution can be assigned to any problem related to the soils. One of the bottlenecks for this being lack of a proper identification technique of the soils, solely based on their intrinsic expansiveness. The purpose of this dissertation is to develop such a technique that can be used universally to identify and classify the soils. This chapter in particular is devoted to reviewing the literature regarding these problematic soils.

2.1.2 Chapter Outline

This review gives state-of-art on soil intrinsic expansiveness, shrink-swell behaviour of clayey soils, soil suction, and identification and classification of expansive soil. Section 2.1 gives an introduction. Section 2.2 includes definition of intrinsic expansiveness and mechanical analogy for intrinsic expansiveness. In addition, the section covers the mode of formation and factors affecting intrinsic expansiveness of soils. Section 2.3 starts by defining swelling and shrinkage followed by giving their mechanical analogy, factors that affect them, techniques in use to measure them, and models that describe them. Soil suction's definition, components and means of measuring are given in section 2.4. The most common expansive soil identification and classification methods available in the literature are critically assessed in section 2.5 and 2.6 respectively.

2.1.3 Terminology

The definitions of terms used in this dissertation follow the accepted usage in the literature. However, some terms that are believed to have a controversial meaning are defined in advance for better understanding of the dissertation. In this section only the definitions of some general terms are given. A number of other specific terms which are not well defined in the literature or are used inconsistently are elaborated on during the course of discussion in the dissertation.

In geotechnical engineering the word “clay” has three different meanings. Unless it is clearly stated which use is intended, sometimes it may lead to confusion. The term is commonly used to describe particle size of a soil. In this case it refers to all constituents of the soil with nominal diameter less than 2- μm as measured from their settling velocity in water (e.g. BSI 1990; ASTM 1990) and should be “clay-sized particles”.

In other cases the term is used to refer to minerals, which are naturally occurring hydrous aluminum silicates. When intended to represent these minerals “clay minerals” should be used. In a natural soil the majority of the particles smaller than 2- μm diameter are clay minerals. However, the term clay to refer to particle size is not appropriate to describe all clay minerals since few of them have nominal diameter greater than 2- μm (Mitchell 1976). Likewise, a soil may contain some particles finer than 2- μm which are not clay minerals.

The term clay is also used to describe a soil with a cohesive property. In some literature the meaning is refined by specifying the percentage of clay sized fraction, although there is disagreement between authors in specifying the marginal percentage (BRE 1990). To avoid confusion the word clay is not used to represent this meaning throughout the dissertation.

Another word quite often used in geotechnical engineering with some degree of ambiguity is *clayey*. Some literatures define the word to represent a soil containing a known amount of clay-sized particles. Throughout this

dissertation the author preferred to use the term to describe a soil containing more than zero percentage of clay-sized particles.

2.2 INTRINSIC EXPANSIVENESS

2.2.1 History

The concept of intrinsic expansiveness was first introduced by Brink (1955) to describe the mode of formation of intrinsically expansive soils in the context of South Africa. He clearly specified that one of the significant factors that determines the degree of heaving of a soil is the intrinsic expansiveness of the soil. Using the same idea Baikoff and Burke (1965) recommended a classification system for identifying whether a soil is potentially expansive or not. There were also efforts to qualitatively describe intrinsic expansiveness by Seed *et al* (1962), Dakshnamurthy and Raman (1973), Tadanier and Nguyen (1984) and others. Most of these attempts failed to properly conceptualise intrinsic expansiveness from soil mechanics point of view until the initiation made by Schreiner (1987a).

The conceptualization specified the need for a standard method for identifying soil's intrinsic expansiveness. The continued research on the problems finally resulted in a simple procedure for identifying intrinsic expansiveness of reconstituted soil samples. The procedure first came as unpublished report (Gourley and Schreiner 1993a), which was later published (Schreiner 1999). The test procedure strictly followed the principles of soil mechanics and can easily be used to compare soil of different stress histories, geological, geographical and geomorphological origins, etc.

The procedure was tested on eight natural samples collected from Kenya and Sudan. Using the result of these few samples, a new method of identifying intrinsic expansiveness of a soil and a model for determining swell index were developed. The model used the standard index tests.

The significant limitations of the research were the number of samples used and their variation. The reliability of the model developed from these few

samples is therefore questionable. It is one of the intentions of this research to include as many natural clayey soil samples as possible and as diversified as possible.

The overall testing procedure developed for identifying the intrinsic expansiveness is detailed in section 2.5.4.2. To understand intrinsically expansive soils the following subsections are devoted to defining some terms, describing the mechanical analogy, and identifying factors that affect them and their mode of formation.

2.2.2 Definition

Before Schreiner (1987a) there was no clear distinction between intrinsic expansiveness, swell or swell potential, and heave. The descriptive report clearly conceptualised the expansive property of a soil. Other than defining soil expansiveness, swell and heave the report used a mechanical analogy to describe the concept behind the terminology. This dissertation has adopted the same definitions and mechanical analogy.

The cause of volume change of clayey soils due to changes in water content or suction is the interaction between clay minerals and water. Clay minerals differ based on the affinity they have to water. The affinity of clay minerals of a soil to water is an intrinsic property that cannot be altered by moisture content or suction that exists at a particular time (Schreiner 1987a). The intrinsic expansiveness of a soil is thus defined as a property of a soil resulting from mineral composition and grading and its interaction with water (Gourley and Schreiner 1993a; Schreiner 1999).

The intrinsic expansiveness relates the change in water content, and thus the volume change, to the suction change. A soil with a high intrinsic expansiveness will show a larger volume change than a soil of low intrinsic expansiveness when both are subjected to the same initial conditions and some changes.

2.2.3 Mechanical Analogy

The mechanical analogy for intrinsic expansiveness is the stiffness or load displacement characteristics of a spring (Schreiner 1987a). Regardless of the applied load (suction) or the displacement (moisture content) the stiffness (intrinsic expansiveness) will remain unchanged. Graphical representation of the analogy is given in Figure 2.1.

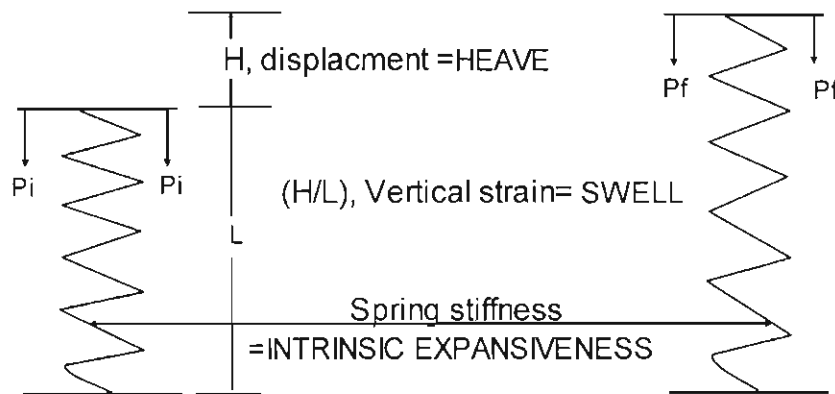


Figure 2.1 Mechanical analogy of intrinsic expansiveness, swell and heave (Schreiner 1987a).

2.2.4 Factors Affecting the Intrinsic Expansiveness of Soils

The intrinsic expansiveness of a soil is dependent upon various factors including mineralogical composition, particle-size distribution and soil-water interaction. These factors are explained in greater detail in the following subsections.

2.2.4.1 Mineralogical Composition of Soils

The mineralogy of a soil is the fundamental information for understanding the intrinsic expansiveness of a soil. Basically, it is mineralogy of a soil that controls the interaction of a soil as it controls the sizes, shapes, and surface characteristics of particles in a soil. Sand and silt are inert to water. An increase in moisture content of soils dominantly containing these particles only results filling of the voids without a significant increase in volume. The case is different in a soil predominated by clay minerals. The consequence of wetting is an increase in volume of the soil. The reason for such behavior is the characteristic properties of the clay minerals such as their structure,

surface charge, surface area, and electric double layer (Moore and Reynolds 1989).

Clay minerals can either be crystalline or amorphous. The crystalline clay minerals are hydrous aluminosilicates, with some replacement of aluminium by iron and magnesium with small amounts of alkali and alkali-earth metals. The unique structure of a crystalline clay mineral determines its shape, surface charge, and surface area. The surface area of the minerals is one of the major factors determining the interaction of such minerals with water.

Soils are also known to contain some amorphous clay minerals. The amorphous materials include aluminosilicates, iron and aluminum oxides, and silica that are too small or poorly crystalline to produce recognizable crystal phase peaks on the XRD pattern (Wan *et al* 2002). Research made on the volume change behavior of amorphous clay is very limited. Therefore most of the following discussions focused on the crystalline clay minerals, which are also the dominant clay minerals.

Most crystalline clay minerals are formed from alternating layers of basic structural units or sheets. Each layer consists of tetrahedral, composed of silicon oxide, and octahedral sheets, composed of aluminum hydroxide or magnesium hydroxide, sandwiched together, one or two tetrahedral sheet(s) with one octahedral sheet (Mitchell 1976). The classification of crystalline clay minerals is based on the precise structure of this layer—whether there are two sheets or three, and whether the octahedral sheet is composed of aluminum oxide or magnesium oxide. The structures of some common crystalline clay minerals are shown in Figure 2.2.

Kaolinite is the simplest clay mineral containing alternate layers of silicate tetrahedral and alumina octahedral sheets, giving a 1:1 basic unit. Inter-sheet bonding in the basic unit is of primary valence type, which is very strong, making the sheets inseparable (Sharma 1998). More than 100 sequences of the sheets chemically bond to form a unit Kaolinite crystal structure up to 1000 Å thick (Gourley and Schreiner 1993c). There is no substitution in the sheets and the basic units are held together by van der

Waals forces. This bond hold the 1:1 layers tightly together leaving little interlayer space for adsorption of cations or water (Dixon 1977). In summary, the low surface area for cation and water adsorption and absorption is the major reason for the low intrinsic expansiveness of kaolinitic soils.

Conversely, montmorillonite has the highest intrinsic expansiveness. A unit montmorillonite particle may be made up of only three basic units, each consisting of an alumina octahedral sheet between two silicate tetrahedral sheets, giving a thickness of 10 Å (Gouley and Schreiner 1993c). The inter-basic unit bond is strong like that of kaolinite. However, substitutions of ions in the tetrahedral and octahedral sheets of the clay minerals results in excessive unbalanced negative charge that significantly increase the cation exchange capacity and affinity for water. Consequently, a unit layer of montmorillonite 9 Å thick can adsorb 200 Å of absorbed water (Yong and Warkentin 1975).

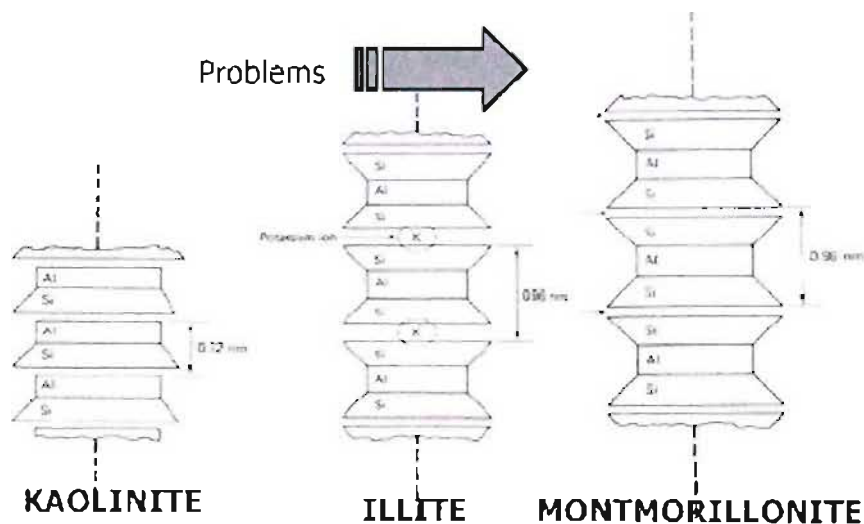


Figure 2.2 Structure of the major clay minerals (Yong and Warkentin 1975).

The forces within basic units of clay minerals are often stronger than those holding the basic units together, within a unit layer. The strong binding force within the basic units is the reason for the platy shape of clay minerals (Thomas 1998). The thickness of the major clay minerals is in the following order: Montmorillonite < Illite < Chlorite < Kaolinite (Yong and Warkentin 1975). The variation in surface area per unit mass is primarily determined

by the thickness of the platy clay minerals, the smaller the thickness the greater the surface area. Therefore, the inverse of the order holds for surface area.

Surface area of clay minerals significantly contributes to the geotechnical properties of clayey soils including strength, volume change, compressibility, plasticity, permeability, and water retention Sharma (1998). Intrinsic expansiveness increases with increase of clay minerals of high surface area. One may then recommend the use of surface area as a means of assessing intrinsic expansiveness. The methods that are now in use to determine surface area are time consuming, expensive and not reliable.

Substitution of one ion with another of nearly equal size but lower valance results in negatively charged clay mineral. The main substitution includes aluminum for silicon in the tetrahedral sheet and magnesium, iron, lithium or zinc for aluminum in the octahedral sheet. These substitutions are the reason for most of the charge of minerals such as montmorillonite, vermiclite, illite, and a minor part of minerals such as Kaolinite (Yong and Warkentin 1975). The negative charges of the clay surfaces are compensated by exchangeable cations to maintain electro-neutrality.

The second source of electric charge on clay particles is unsatisfied valance charges at the edges of the clay minerals, which are referred as broken-bond charges. The broken bonds are between oxygen and silicon and between oxygen and aluminum. The amount of this charge increases with a decrease in clay particle size, because the proportion of edge area to total area is increased. Small clay minerals like montmorillonite for instance get additional negative charges surfaces that increase the surface area, and thus absorption and adsorption of cations and water.

Generally volume change behavior of clayey soils is more due to the surface charge than to the edge charge. The clay minerals edge can be approximated as a convex cylindrical surface. Because of capillary forces water films are less stable on the convex surface of the edge than on the platy face surface (Deryagin and Churaev 1984). This is the reason why the

water films on clay edges are rather thinner than on face surfaces. Sokolov (1990), for instance, shows that the water film at the clay edges can be ten times thinner than at the face surfaces. Therefore, in the following discussions the term charge is only meant to describe those on the surface unless specified.

In order to preserve electrical neutrality, clay mineral surfaces strongly attract cations and repel anions. Under dry condition some cations are strongly held by the negatively charged clay surfaces. In fact more cations than the attracted cations are required to neutralise the electro-negativity of the clay surfaces. These additional cations with some associated ions exist at some distance from the surface as salt precipitates (Duran Gamarra 1986; Sharma 1998). With start of adding water, the clay surface and the cations attract water. Additionally the salt precipitates go into solution.

With addition of more water, the process of hydration results in the formation of two distinct structural water layers: adsorbed and absorbed water layers. The adsorbed water occurs immediately adjacent to the surface of the clay mineral and is relatively strongly held by the clay surface. Absorbed water is less well bound and exists after the adsorbed layer. Both adsorbed and absorbed water layers are collectively known as Double Electrical Layer (DEL). The water adjacent to absorbed water is free.

Within the adsorbed water two layers can be distinguished. The water layer closest to a clay surface is formed by molecules which hydrate the clay crystal surface and adsorbed cations. The water molecules are firmly attached to the clay particles, mainly by hydrogen and ion-dipole bonds. The outer layers of the adsorbed water are formed by water molecules, which are highly oriented toward the clay surface due to dipole-dipole interactions with the water molecules, attached immediately to the clay surface. The dipole-dipole attracting force gets weaker with distance from the clay surface as the orientating influence of the surface on the water molecules decreases. Each successive water layer is held less strongly. Generally, adsorbed layer is up to four water layers thick (Gourley and Schreiner 1993c). With elevation of temperature the thickness of the layer

diminishes and at 65 °C it is completely disturbed (Yong and Warkentin 1975).

In presence of an excess amount of water than required for the formation of the adsorbed layer, the thickness of water surrounding the clay surface increases and results in the formation of an absorbed layer. The concentration of the dissociated cations in this layer decreases with distance from the clay surface. The thickness of the diffuse layer is mainly controlled by the following factors: stress (pore water pressure), clay mineralogy; type of the exchangeable cations; concentration and chemical composition of the pore solution; dielectric constant of the solution, pH and temperature.

Successful application of the DEL theory in predicting the volume change behavior of saturated clayey soils are reported by several researchers, including Bolt (1956), Olson and Mesri (1970), Sridharan and Jayadeva (1982), Stepkewska (1990), Yong and Mohamed (1992), and Gleason *et al* (1997). Komine and Ogata (1994, 1996, 2003) and Sharma (1998) have recently reported application of the theory for predicting the volumetric change behavior of intrinsically expansive soils.

The Shortcome of the DEL theory mainly arises due to the simplified assumptions. These assumptions are that ions in the diffuse layer are point charges and do not interact with each other, that the distribution of the ions is uniform, and that the clay surfaces are in parallel arrangement. Some researchers proposed alternative theories for clay swelling such as wetting film theory (Deryagin and Churaev 1984), mixture theory (Hucckel 1992), and regular solution theory (Graber and Mingelgrin 1994). Nevertheless, the limitations of the DEL are mainly for very dense soils where development of the diffuse layer is hindered (Sharma 1998).

The DEL of neighbouring unit layers or clay particles may interact, resulting in a net repulsive force between them. If they approach extremely close to each other there is a possibility of development of attractive forces between the double layers (Sharma 1998).

2.2.4.2 Grain Size Distribution

Grain size distribution is one of the major factors affecting intrinsic expansiveness of a soil. Constituent particles of soils are grouped into: clay, silt, sand, etc. Each size fraction contributes to different geotechnical properties of the soil. From the volume change point of view, both sand and silt behave differently from clay minerals, the main reason being the interaction of the minerals with water. Quite often the former two particles hardly interact with water whereas the clay minerals are known for their strong interaction.

Thickness of a clay mineral can be indirectly assessed using its surface area. Montmorillonite has a surface area about 8 times that of illite and 40 times that of kaolinite (Gourley and Schreiner 1993c). It is fact that the smaller the clay particle size, the greater the surface area for absorbing and adsorbing water, thus is the greater the intrinsic expansiveness. Equally important to the clay mineral particle size, the clay-water interaction affects the expansiveness of a soil which will be considered in the next section.

2.2.4.3 Soil-water Interaction

Intrinsic expansiveness of a soil is also affected by soil-water interaction, particularly clay minerals-water interaction. The basic mechanism of clay mineral-water interaction has attracted the attention of several researchers who have proposed a number of theories to explain the phenomenon. The two benchmark theories frequently used in the literature to explain microscopic volume change behavior of clayey soils are particle energy and osmotic pressure. Gouy and Chapman were the first to develop the particle energy theory in 1910's, which was then refined by Stern to describe the ionic distribution in the immediate vicinity of the surface of clay particle. On the other hand, Bolt (1956) was the first to develop the osmotic pressure theory which was then extended by Sridharan and Jayadeva (1982). These two theories are detailed in the following paragraphs.

The concept of osmotic pressure can be used to describe the swelling process and to estimate the swelling of clayey soils. The existence of a

gradient in the concentration of dissolved ions between two regions of liquid and a separation by a semi-permeable membrane are two essential prerequisites for osmotic pressure difference to occur (Schreiner 1988). A semi-permeable membrane permits flow of the solvent but not the solute. The attractive force, between the negatively charged surface of a clay particle and the attached cations, prevents the diffusion of the cations away from the surface. This creates the effect of a semi-permeable membrane.

Bolt (1956) was the first to use the concept of osmotic pressure to calculate swelling of saturated clayey soils. He considered the clay particles as semi-permeable membrane. Schreiner (1987b) recommended, instead, considering the semi-permeable membrane at the interface between the absorbed water and the free water. Cations are not free to move out of the absorbed layer mainly due to the attractive force of the negatively clay surface and the partly due that of the associated anions. Therefore, it still would be reasonable to assume an osmotic pressure difference between the absorbed water with its exchangeable cations and the free water with its dissolved salts.

In the osmotic model, clay particles remain separated and volume change takes place without alteration of clay particles arrangement. For saturated soil as noted from Bolt (1956), the applied effective stress was the reason for the expulsion of water from the absorbed layer. The removal of water from a soil could either be due to evaporation to the air or drainage under suction gradient. These processes decrease the effective size of the particles and the absorbed water and thus decrease the overall soil volume.

Successful applications of the osmotic theory in predicting the volume change of saturated clayey soils are reported by several investigators, including Bolt (1956); Olson and Mesri (1970); Sridharan and Jayadeva (1982); Stepkewska (1990); Yong and Mohamed (1992); Mitchell (1993); Shang *et al* (1994); and Gleason *et al* (1997).

The particle energy theory is based on the balance of attractive and repulsive forces between adjacent particles and it assumes clay surfaces as

non-wettable. The clay surface charge is balanced by an equivalent number of oppositely charged cations in the adsorbed and absorbed layers.

One of the factors affecting thickness of these layers is the type of clay mineralogy. Several authors (including Gourley and Schreiner 1993c) argue that the volume change of a soil is mainly due to the change in thickness of the absorbed water layer. They further commented that there is a slight variation in the thickness of the adsorbed water layer with type of clay mineral, but as it is insignificant compared to that of absorbed water layer it might be reasonable to consider it as a constant. Because of the significance of the absorbed layer on the intrinsic expansiveness of soils it will be given more emphasis in the following paragraphs.

The development of an absorbed layer is affected by several factors among which the major ones are: clay mineralogy, type of the exchangeable cations, and concentration and chemical composition of the pore solution.

It would be easy to show variation in intrinsic expansiveness of clay minerals may arise due to difference in the thickness of absorbed water layer by taking montmorillonite and kaolinite minerals as examples. As reported by Gourley and Schreiner (1993c) the thickness of the absorbed layer of montmorillonite and kaolinite particles when their water demand is satisfied is 800 Å and 400 Å respectively. By taking the ratio between the thickness of the absorbed layer and clay particle they showed that the theoretical potential volume change for montmorillonite is 50 times that of kaolinite from completely dry to saturated condition (Sharma 1998). This confirms the dependence of intrinsic expansiveness of a soil on clay mineral particle size or surface area.

Exchangeable cations are cations that can be exchanged with other suitable cations provided that chemical and electrical charge stability can be achieved. The increase of a valence of exchangeable cations strengthens bonds between the cations and the clay surface and consequently reduces the diffuse layer thickness. Generally, hydration ability decreases with increase of cation radius. The lower hydration ability of a cation contributes to strengthening of its adsorption bonds with a clay surface. Practicality of

this argument can be shown by taking Ca-montmorillonite and Na-montmorillonite. The former montmorillonite is characterized by Ca dominated adsorbed layer whereas the latter montmorillonite is characterized by an adsorbed layer dominantly containing Na ions (Yong and Warkentin 1975). Such difference shows that intrinsic expansiveness of a soil may increase with decrease of the valence of the cations. Liquid limit, which is assumed to have a significant relationship with the maximum thickness of the adsorbed layer, was used by Lambe and Whitman (1979) to compare the volume change behavior of soils due to changes in the valence of the exchangeable cations.

Further more, if the exchangeable cations have the same valence, the increase of their radius should lead to decrease of the adsorbed layer thickness (except H^+). These theoretical propositions were confirmed by numerous experimental investigations. Ovcharenko *et al* (1974), quoted in Tchistiakov (2000), for instance showed that at equal concentrations of the pore solution the adsorbed layer thickness decreases due to the presence of the different ionic forms of vermiculite in the order of: $Li > Na > NH_4 > Ca > Cu > Mn > Co$.

Generally, if other physico-chemical conditions are constant, the increase of salt concentration causes reduction of the adsorbed layer (Gregory 1989) and void ratio (Bolt 1956; Sridharan and Jayadeva 1982). The reduction in the thickness of the adsorbed layer is due to osmotic pressure difference developed with increase of salt concentration in the free water that leads to flow of water out of the diffuse layer to the free water layer.

Decrease of the salt content of pore solution of a soil could happen due to natural processes such as rainfall. In such case due to the increase of the water content the thickness of the adsorbed layer may increase. Research by Schreiner and Burland (1991) on Black Cotton soil of Kenya showed an increase of the swell with leaching of the salts of the soil. Saturation of soils with ground water most probably brings insignificant change in the salt concentration. It is thus more reasonable to assume that greater intrinsic

expansiveness of a soil may be recorded with the use of salt free water than with water containing some amount of salt.

In contrary to this, some experiments such as by Zlotchevskaya and Korolev (1988) showed that replacement of distilled water by dilute solutions of NaCl in Na-forms of clays could cause increase enlargement of the absorbed layer. For the Na-form of the minerals a possible mechanism of this phenomenon may be the following. While only distilled water is present in a pore medium, some of the H^+ diffuses to the exchange complex of clay minerals. The permeating of NaCl solution causes exchange of some H^+ for Na^+ , which leads to increase of absorbed layer.

As a conclusion it can be said that swelling and shrinking of intrinsically expansive soil could result from change in the concentration of the salt content of pore solution, as well as change in the water content, suction and applied pressure.

2.2.5 Origin of the Intrinsically Expansive Soils

The origin of expansive soils is related to a complex combination of conditions and processes that result in the formation of clay minerals that are inherently expansive. Conditions and processes that determine clay mineralogy include composition of parent material (Brink *et al* 1982) and degree of physical and chemical weathering to which the materials are subjected (Geological Society Engineering Group 1990). Igneous, metamorphic and sedimentary rocks can all be parent materials for intrinsically expansive soils. Some of the commonly known rocks known to produce expansive soils include: gabbro (norite), basalt, diabase, andesite, schist, tillite, shale, mudstone, and to lesser extent dolerite and granite (Brink 1955).

The mineralogical composition of basic rocks such as norite, basalt, dolerite, diabase, and andesite is essentially ferro-magnesian minerals (plagioclase feldspars, amphibole, and pyroxines). Often black clayey soil with very high intrinsic expansiveness (as it predominantly contains montmorillonite clay

mineral) is derived from the decomposition of the ferro-magnesian minerals under poor drainage and an alkaline environment as Brink (1955) stated.

The ferro-magnesian minerals contain insignificant quantity of potassium. Immediate weathering products of these minerals are chlorite and vermiculite. The stabilizing cation of these minerals is Mg-ion which can easily be surrounded by a shell of water molecules. Since the stabilizing power of the Mg ion is very poor and as the vermiculite has a potential of absorbing water, leaching of the ion proceeds very rapidly finally resulting in the formation of montmorillonite (De Bruijn 1955).

It is also possible to obtain yellow montmorillonitic soil if the rock has relatively better internal drainage. In the presence of a very high internal drainage usually red clayey soils (mainly containing kaolinite and illite clay minerals) are more likely to develop. The formation of the red colouration comes with oxidation of iron liberated during the transformation of montmorillonite to kaolinite. Formation of such soils, which are intrinsically less expansive soils, is favoured not only by good drainage but also by an acidic environment.

Quartz grains often dominate acidic igneous rocks such as granite. Weathering of the unstable ferro-magnesian minerals results in a highly porous soil that allows free movement of water and air. Such structure allows formation of stable clay minerals such as kaolinite and illite (Brink 1955). However, there are some reports where montmorillonitic clay minerals were found to be the dominant minerals derived from such soils (Brink 1955; and Kellar 1964) as reported in Gourley and Schreiner 1993c).

There is also a possibility of obtaining kaolinite and montmorillonite rich soils from weathering of metamorphic rocks, such as Schist. The montmorillonite clay minerals are most probably derived from potassium-free plagioclase feldspars of the parent rock than potassium-rich muscovite, which is common in such rocks (De Bruijn 1955).

Weathering of sedimentary rocks such as shales, mudstone, and tillite may produce expansive soils either by chemical changes of the sediments or

from the pre-existing expansive minerals. The expansive soils are dominated by montmorillonite and interlayered clay minerals (such as illite-montmorillonite).

2.3 SWELL-SHRINK BEHAVIOUR OF CLAYEY SOILS

2.3.1 Background

Intrinsically expansive soils undergo substantial volume changes associated with swelling and shrinkage processes. Factors significantly affecting the swell-shrink behavior of natural and compacted clayey soils are soil properties, environmental conditions, and stress states (Sridharan and Venkatatappa Rao 1973). It is the intent of this section to give a general review on these factors and the techniques of measuring swell and shrinkage of a soil.

2.3.2 Swelling

2.3.2.1 Definition

Swell of intrinsically expansive soil may be described as the volumetric strain due to a decrease in suction (increase in moisture content) or applied stress. In laboratory testing of intrinsically expansive soils, swell is usually caused by a reduction of the matrix suction to zero under a constant vertical applied stress. The sample is usually confined laterally in an oedometer. Most testing is performed without the knowledge of the initial suction and the radial or horizontal applied stress. The final matrix suction is generally zero but both the initial and final solute suctions are seldom known.

For saturated soil only, the volume change of the soil will be equal to the volume of water taken by the sample. In expansive soil engineering the case of complete saturation is relatively of low importance. In the partially saturated state the relationship between the volume change of the soil will be affected by factors such as the micro-fabric and the structure of the soil and the hysteresis in the suction-moisture content relationship.

Swell is not a primary soil property. It is the strain, which results from an interaction between suction, stress and the intrinsic expansiveness of the soil. The intrinsic expansiveness is a primary soil property and cannot be changed without changing the soil. Swell is specific to stress, suction and void ratio conditions under which it is measured. Therefore, magnitude of swell can be determined by suction and total stress history of a soil and the intrinsic expansiveness.

2.3.2.2 Mechanical Analogy

Swelling process under decreasing suction may be visualized in terms of a linear spring system shown systematically in Figure 2.1. The displacement of the end of spring (heave) is the summation of the strains (swell) caused by the change in applied load (suction) acting against the spring stiffness (intrinsic expansiveness). Regardless of the load (suction), the spring stiffness will be changed. Practically the system will be non-linear, but similar analogy may be used.

2.3.2.3 Mechanism of Swelling

The basic concept of clayey soil swelling is the balancing of forces of interaction among the clay mineral surfaces, ions, and water. A number of theories have been proposed by researchers to describe the mechanism, such as osmotic and particle energy theories as discussed in section 2.2.4.3. However, agreement has not been reached about the most appropriate theory describing the mechanism at all the stage of swelling.

In the sense of osmotic theory, swelling is the consequence of the physico-chemical interaction between clay particles, DEL, and free water layer. Clay particles with excess charge on their surfaces cause ion concentration in the DEL to exceed that of free water. The difference in ion concentration results diffusion of water that forces the clay particles apart, causing swelling. A successful application of the osmotic theory in predicting the volume change of unsaturated clayey soils is reported by Komine and Ogata (1994, 1996).

The particle energy theory, assumes that swelling is the consequence of reduction in the potential energy of the interlayer water as a result of its

interaction with adjacent clay surfaces. Accordingly, thickening of water film surrounding a clay particle continues until the negative surface charge of the clay surface is balanced by positive charge of the DEL.

Various attempts were made to compare osmotic and particle energy theories. One good example is Stocker (1969) who reported the equality of the repulsive forces derived from the two theories. The work by Sridharan and Jayadeva (1982) also obtained a reasonable agreement between theoretical and experimental void ratio-effective stress curves based on the two theories. This may show that the two theories basically describe the same physico-chemical processes only in different terms.

Other researchers commented that the two theories hold at different conditions. In environments with high water contents and large interlayer distances, the osmotic theory provides the best explanation for soil swelling (Thomas and Moody 1962). However, at low to moderate interlayer distances, several clay particles with widely varying charge densities produced nearly identical swelling pressures supporting particle energy theory (Viani *et al* 1983).

2.3.2.4 Phases of Swelling

Graphical representation of swelling as a function of square root of time gives a curve that generally looks like Figure 2.3. From the curve three stages can be identified (Day 1999), namely: primary swelling, secondary swelling and no swelling. Primary swelling involves closing of macro pore spaces, and destruction and disorientation of large clay aggregates. There is a possibility of enlargement of the pore space volume as a result of this process.

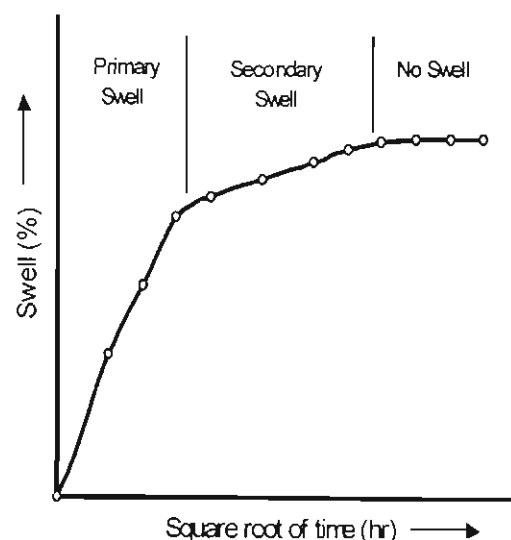


Figure 2.3 Phases of soil swelling (Day 1999).

The rate of primary swelling depends upon several factors. Afes *et al* (1998) investigated the difference in primary swelling as a consequence of variation in clay mineral type and dry density. The report stated that rate of primary swelling of a clayey soil declines with increase in proportion of its intrinsically expansive clay minerals. This phenomenon could be due to the low permeability of such minerals and the swelling process as it starts on external surfaces of the particles and slowly weaken the inter-particle bonds. There is a possibility of an increase in the rate of primary swelling with increase of initial suction of the clayey soil. The rapid swelling is assumed to be attributed to the ease of water molecules to fill in the macro-cracks produced due to desiccation. Afes *et al* (1998) also experimentally demonstrated the effect of dry density. The research finding showed an increase in rate of primary swelling with increase dry density regardless the clay mineralogy.

Most of the macro-pore spaces are filled during the primary swelling stage. Secondary swelling involves closure of micro cracks and further reduction of entrapped air. The rate of secondary swelling is faster than that of primary swelling. The research results of Afes *et al* (1998) confirmed this fact. With increase of highly expansive minerals, such as montmorillonite, in a soil, the rate of secondary swelling gets slower; whereas the amount of swell recorded gets smaller with increase of the percentage of non-expansive minerals such as kaolinite. The research by Afes *et al* (1998) also showed a decrease in the rate and an increase in the percentage of secondary swelling with decrease of dry density.

The secondary swelling continues until complete development of DEL. Clay minerals such as kaolinite show almost negligible percentage of swelling following the secondary swell compared to that of montmorillonite. Complete saturation of intrinsically expansive soil is a rare case due to possible entrapment of air. This is more significant in montmorillonitic soils than in kaolinitic soils.

2.3.2.5 Factors Affecting Swelling

Factors affecting swelling of a soil can be generally categorized into soil characteristics, environmental conditions, and stress state. General descriptions of these factors are given in the following paragraphs.

Most significant of soil characteristics affecting swelling is intrinsic expansiveness, which is determined by mineralogical and physico-chemical properties of the soil. Detail explanations regarding the different factors affecting intrinsic expansiveness are documented in section 2.2.4.

Other soil characteristics that were not discussed which are believed to have a significant effect on soil swelling include: percentage of clay minerals, orientation of the clay particles, initial density or void ratio, cementation and fabric.

It was made clear in the preceding discussion that swelling of a clayey soil is the function of its clay mineral components (White and Pichler 1959). Swelling of a given clayey soil is reduced in proportion to the amount of its silt, sand, and other non-clay minerals. Quantification of this may need further research.

Clay particles orientation or arrangement seems also to affect their accessibility to moisture and thus swelling characteristics. Still there is no agreement regarding a generalized effect of particle orientation. Yong and Warkentin (1975) for instance argue that swelling strain record of Na-montmorillonite that contains parallel particle orientation is the greatest. On contrary, swelling reported by Seed and Chan (1962) is the greatest for clayey soil with random or flocculated particle arrangement.

Variation in initial dry density or void ratio is known to significantly affect swelling of clayey soils. Other factors being constant, swelling strain upon wetting increases as soil density increases. The easiest way of showing this phenomenon is by observing swell of soils initially prepared in different initial dry density using different compaction techniques. Several researches have been performed to demonstrate that, including Holtz and Gibbs

(1956); Seed *et al* (1962); Booth (1975); Oloo *et al* (1987); Afes *et al* (1998); Subba Rao *et al* (2000); and Attom *et al* (2001).

The swell of compacted samples is significantly influenced by the initial soil structure. Work by Seed and Chan (1962) showed that method of compaction induce different structures at given moisture contents and densities. As stated by Oloo (1987) there is very little difference between the swelling characteristics of samples prepared by static and dynamic compaction except at high and low moisture contents. The slight difference can be explained in terms of arrangement of particles as a consequence of the compaction at different moisture contents. At moisture content greater than the optimum, static compaction produces a flocculated orientation whereas dynamic compaction produces a dispersed orientation (Yong and Warkentin 1975). Of these two, the flocculated arrangement shows a greater swelling due to the increase of inter-particle spacing. The swelling characteristics are thus less sensitive to the method of compaction. Results by Attom *et al* (2001) documented the highest swelling with the use of dynamic compacting technique followed by static, and kneading compaction respectively.

The presence of cementing materials such as iron hydroxides, carbonates, silica and various organic molecules between clay particles might limit volume increase of clayey soil on swelling. The reason for the restrain of swelling is not yet clear. Most probably it could either be by affecting the bond between the clay particles or by affecting the physico-chemical property of the diffuse double layer (Yong and Warkentin 1975).

Soil fabric, which is the arrangement of individual particles, has a significant influence on soil swelling behaviour. A soil with open fabric subjected to a high stress, for instance, can exhibit collapse in the final stage of a wetting path (Sharma 1998). The results reported by Escario and Saez (1973) are good example that supports this feature. Schreiner and Gourley (1993) stated that the risk of collapsing during wetting could be avoided with application of external stress. Taking this summary one step further, researches such as Delage and Lefebvre (1984); Atabek *et al* (1991); and

Wan *et al* (1995) showed that compacting dry of optimum of clayey soils results formation of bi-modal pore size distribution as the soil exhibits a fabric made up of aggregates of varying size. On the contrary, on wet side of optimum compaction tends to produce a more homogenous matrix-dominated fabric and a pore distribution with a single peak (Gens *et al* 1995).

Swell is a function of factors such as applied stress, and initial water content or suction. The amount of swell decreases with the intensity of the applied stress. Work of Kassif *et al* (1973); Escario and Saez (1973); and Brackley (1975) can be mentioned as evidence for this. Often, a linear relationship between swell and the logarithm of confining stress has been reported (Alonso 1998). Results reported by Richards *et al* (1984) and Justo *et al* (1984) suggested that the higher the initial suction, the larger the measured swell. As initial suction decreases swell decreases (Alonso 1998). Eventually, as soil conditions approach the saturated state, expansion may vanishes. Moreover, the relationship between swell and suction reduction is not linear, with the rate of swell increasing as suction is reduced (Sharma 1998).

2.3.2.6 Measuring Soil Swell

Swelling strain and pressure are the two major test results reported in literature to describe volume change behaviour of intrinsically expansive soils due to intake of water. The tests used to obtain these results can be divided into two major categories. The first sets are those tests which permit intake of water without control of the suction or water content. The other set are those tests which control the intake of water either by suction or by water content. The first sets of tests are generally quick, simple to perform and inexpensive. The limitation of these methods is that they only provide data at the end point of the wetting process (Schreiner 1988). The other set are slow, expensive and complex both in equipment and technique.

One-dimensional oedometer apparatus has become widely used for facilitating general understanding of swelling behaviour of intrinsically

expansive soils. The main oedometer test procedures for measuring soil swell and swelling pressure are given hereunder.

One of the tests is known as consolidation after swell. Jennings and Knight (1957) are the first to describe this type of test which has been further modified over years, the most recent being by Jennings *et al* (1973). The test involves placing a soil at field or compaction moisture content under a relatively small vertical total stress and then supplying the set-up with water. Starting with application of water the volume change of the sample is monitored. After reaching a complete wetting up the sample goes through consolidation by increasing vertical stress. The consolidation would be used to estimate the swelling of the soil under different possible vertical stresses. Swell pressure from this test can be determined as the vertical stress during consolidation at which the void ratio becomes equal to the initial void ratio (Chen 1973).

The second is known as swell under load test. This test was developed by Holtz and Gibbs (1956). The procedure is similar to the above-mentioned one, except that vertical total stress, which is equivalent to field stress, is applied before swelling and consolidation after swelling is omitted.

The other test is known as unloading after swell, which was recommended by Sullivan and McClelland (1969). The sample set-up is still the same as the already mentioned two procedures. In this test, the vertical strain that develops with wetting is prevented by increasing the vertical total stress. Following the completion of the wetting up the vertical stress is decreased. This rebound curve would be used to get estimate of the swell under any vertical stress. On the other hand, swell pressure from this test is the vertical stress at the end of the constant volume change (Holtz and Gibbs 1956).

Due to the expense of running the direct tests there is a growing interest in predicting swell of soils indirectly using simple tests such as clay content, soil suction, Atterberg limits, shrinkage limit, or combination. The model that serve such purpose can be developed by correlating swelling strain record of a soil with test result of the simple tests mentioned. Diversified

origin of soil samples is required to develop a correlation that represents better the swell behaviour of soils. It is one of the major objectives of this research to develop such a tool that can be used universally.

Most models previous developed used swell results of undisturbed soil samples or compacted samples prepared to closely represent the undisturbed conditions. Different researchers (including Oloo *et al* 1987; Schreiner 1987a; and Schreiner 1988) challenged the use of such models for different soils from which they are originally developed, the major reason being variation of swell as a result of factors such as soil fabric, stress history, dry density, etc. A promising work was made by Gourley and Schreiner (1993a) and Schreiner (1999) that resolved the site and soil specific use of previous models by using reconstituted soil samples with stable soil fabric and consistent stress history.

2.3.3 Shrinkage

2.3.3.1 Definition

Shrinkage of an intrinsically expansive soil may be described as the reduction in volume resulting from an increase in suction or decrease in water content. According to Sridharan and Prakash (1998) capillary pressures induced by evaporation of water from the soil are the main reasons for the volume reduction. As the evaporation continues, the radius of the meniscus developed in the soil water continues to decrease until the shear stresses induced by the capillary pressures are equalized by the shear strength at the particle level.

Sridharan and Prakash (1998, 2000a) experimentally proved that the shrinkage process is a packing phenomenon and is primarily controlled by the relative grain size distribution of the soil. During the process, larger void spaces between sand particles are filled with finer sand and silt particles, and smaller void spaces between silt particles are filled by finer clay particles. Furthermore, in case of soils with the same particle size distribution the shear resistance at the particle level will determine the

shrinkage. Comparatively soils with high shear resistance shrink less than soils with low shear resistance.

2.3.3.1 Phases of Shrinkage

Different works (such as Tempany 1917; Haines 1923; Stirk 1953; Warkentin and Bozozuk 1961; Yong and Warkentin 1975; Popescu 1980; Ho et al 1992; Biwei *et al* 1998; and Tripathy *et al* 2002) have reported assessment of soil volume change with gradual decrease of moisture content upon drying. These assessments produce shrinkage curves that describe change in void ratio (or volume) with change in moisture content.

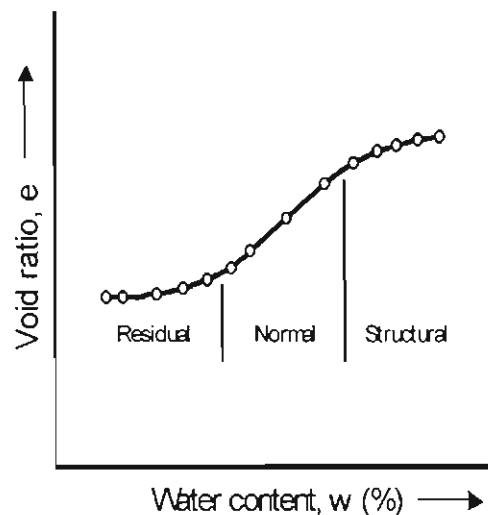


Figure 2.4 Phases of soil shrinkage (Tripathy *et al* 2002).

The shrinkage curve of initially saturated clayey soil specimen shows different phases of deformation (see Figure 2.4). Three phases of deformations are identified, which are: structural shrinkage, normal shrinkage, and residual shrinkage (Haines 1923; Yong and Warkentin 1975; Popescu 1980; and Tripathy *et al* 2002). Structural shrinkage is observed on clayey soils with well-developed crumb structure (Popescu 1980). The phase is characterized by a decrease in volume which is less than the volume of water lost. The water lost in this phase comes from a few large and stable pores where capillary forces hold it. During the normal shrinkage phase, volume change is in direct proportion to the amount of water removed (Popescu 1980); and all the pore spaces almost remain filled with water (Haines 1923; and Biwei *et al* 1998). With further decrease of water some soil particles start to come in contact and resist further decrease of volume. Therefore, during residual shrinkage decrease in sample volume is less than the volume of water lost. Some additional shrinkage occurs on further drying due to additional fabric arrangement, and in some cases to bending of particles as stated by Yong and Warkentin (1975).

2.3.3.2 Factors Affecting Shrinkage

It is believed that the same factors that affect swelling will also control the mechanics of shrinkage. Characteristics of the shrinkage curve vary with the nature of the soil. Total shrinkage increases with increase of initial water content. This is a function of the type and amount of clay minerals of the soil, the mode of geological deposition, the depositional environment which determines both particle arrangement and overburden pressure, and degree of weathering. Increase percentage of sand-silt particles in the clayey soil reduce total shrinkage because they dilute the clay and decrease the volume water held by the soil (Yong and Warkentin 1975). With decrease of particle size of clay minerals there is more surface area for holding water molecules, thus resulting an increase in the initial water content.

The shrinkage limit also depends upon the fabric and the type of clay minerals. A more random arrangement increases the shrinkage limit. On contrary, a more parallel arrangement decreases the shrinkage limit. Particle interaction leads to desaturation at higher water content for random edge-to-face arrangement of particles than for the more parallel arrangement. The additional volume of water is trapped between particles in the random arrangement and is not affected by the forces holding water at soil surfaces. The shrinkage limit for most clayey soils occur at soil suction values in excess of 1MPa (Yong and Warkentin 1975). A low shrinkage limit is usually related with large volume change, e.g. montmorillonite will have a value of 10-15% while kaolinite will be 20-25% (Yong and Warkentin 1975).

2.3.3.3 Determining shrinkage curve of clayey soils

Shrinkage curves of clayey soils can be experimentally determined by measuring water content and volume as a saturated soil loses water. Determination of the average water content of a soil sample during each stage of drying involves a two-step process. Recording the weight of the sample during the successive stages of shrinkage is the first step of the process. When loss of water is stopped, the soil sample is oven dried. Using the dry density and the water content obtained the water content at the

different stages of the drying can be calculated back. Unless a controlling mechanism is adopted the shrinkage process may lead to non-uniform water distribution as drying proceeds at the surface and water must move from inside the sample. The direct consequence of non-uniformity is formation of cracks, which must be avoided for accurate volume measurement of the sample. The uniformity can be ensured with the use of a small soil sample, slow drying or allowing the sample to equilibrate in a saturated atmosphere after a short drying period.

Accurate measurement of volume is more difficult. The existing methods for measuring total volume include: mercury method (BSI 1990; and ASTM 1998), wax method (ASTM 1992) and calliper method (Gourley and Schreiner 1993a; and Schreiner 1999). The first two methods are based on the principles of Archimedes' whereas the last one uses a direct measurement.

In the mercury method, the volume of a sample is calculated from the volume of displaced mercury with immersion of the sample in a mercury bath. The major problem associated with extensive use of mercury is the health hazard. Yong and Warkentin (1975) also mentioned that at low water contents dry samples might be broken under the pressure due to the weight of the mercury.

The wax method uses a water bath instead of mercury bath. To prevent entry of water the soil sample is coated by paraffin wax. The disadvantage of the usage of paraffin coating is that the sample can be used for only one measurement. Several samples must be used to define the shrinkage curve, which need consideration of variability of the samples. The method assumes that a wax coating layer is developed over the surface of a soil specimen. Practically, there is a possibility of obtaining some voids uncoated between the surface of the specimen and the wax layers and some voids can be filled with the wax in the process of coating.

Unlike mercury and wax method, the venier caliper method directly measures dimensions of a sample to calculate its volume. For accuracy of this method the soil sample needs to be regular shaped, hard enough for

easy measurement, and without cracks. Research done by Gourley and Schreiner (1993a) used reconstituted soil samples which go into successive compression in an oedometer to obtain regular and easy to handle samples for the calliper method. The same research also avoided the risk of formation of cracks by controlled removal of water by chemical means in a temperature and humidity controlled laboratory.

Different factors determine the formation of cracks. It could be environmental factors such as humidity and temperature, and soil factor such as particle size distribution, type and amount of clay minerals, arrangement of clay minerals, moisture content, etc (Yong and Warkentin 1975). Cracks form where the cohesion of the soil is lowest. Where drying is not uniform and under high temperature, cracks will form in the wetter soil. Intrinsically expansive soil such as montmorillonitic soils may show more cracking than non-expansive soils such as kaolinitic soils. In any type of soil the formation of a crack is more probable in the structural and normal shrinkage stage than in the residual shrinkage.

In presence of cracks, determination of the volume of the specimen by the mercury displacement technique has been proven to be more accurate (Tripathy *et al* 2002). The other condition where the mercury method has advantage over calliper method is when the sample is not strong enough to handle easily, as the required handling will deform the sample (Yong and Warkentin 1975).

2.4 SOIL SUCTION

2.4.1 Introduction

Suction is one of the two stress variables which control soil behaviour, the other being total applied stress. It is one of the most important parameters describing the moisture condition of unsaturated soils and it has a major influence on soil strength and volume change (Schreiner and Gourley 1993; and Fredlund 1998). It will be the purpose of this section to define the term soil suction, describe its components, and to explain in details the filter paper method of measuring its magnitude.

2.4.2 Definition

In wet climates where soils remain saturated, the effective stress controls the behaviour of the soils. For instance, if the effective stress changes then the soil undergo a volume change. In less wet climates the water deficit results in a negative pore pressure (or positive suction). If the negative pore pressure is sufficiently large, air enters the spaces between the solid particles in the soil and the effective stress is no longer adequate to describe the behaviour of the soil. In such case the soil behaviour has to be investigated in terms of applied stress and suction.

According to the First Expansive Soils Conference by the Review Panel (1965) soil suction is the negative gauge pressure relative to the external gas pressure on the soil water to which a pool of pure water must be subjected to in order to be in equilibrium through a semi-permeable membrane. In general, the drier the soil, the higher the soil suction (Lee and Wray 1995). The water content in a soil reduces significantly with increasing suction. As suction increases, flow occurs through increasingly smaller size pores. The connectivity of the voids or pores continues to reduce with increased values of suction. Large increases in suction eventually lead to a relatively small change in water content (or degree of saturation).

2.4.3 Components of Soil Suction

The total suction of a soil (Ψ) is comprised of matric suction ($u_a - u_w$) and osmotic suction (π):

$$\Psi = (u_a - u_w) + \pi$$

where u_a = pore-air pressure and

u_w = pore-water pressure

As stated earlier total suction corresponds to the free energy of the soil water, while the matric and osmotic soil suction are the components of the free energy. The matric component is associated with inter particle capillary

menisci and particle surface hydration mechanisms, and the osmotic component arise from the presence of dissolved solutes in the pore fluid.

2.4.3.1 *Matric suction*

Matric suction is the equivalent suction derived from the measurement of the partial pressure of the water vapor in equilibrium with the soil water, relative to the partial pressure of the water vapor in equilibrium with a solution identical in composition with the soil water. Houston *et al* (1994) simplified the definition of matric suction to the affinity a soil has for water in the absence of any salt content gradients. Matric suction is commonly associated with a capillary phenomenon arising from the surface tension of water. It is primarily the matric suction component that is of interest with regard to the engineering behavior of unsaturated soils. Laboratory data have indicated that a change in total suction is essentially equivalent to a change in the matric suction for many unsaturated soil suctions (Fredlund 1998).

2.4.3.2 *Osmotic suction*

Osmotic suction is the equivalent suction derived from the measurement of the partial pressure of the water vapor in equilibrium with a solution identical in composition with the soil water, relative to the partial pressure of water vapor in equilibrium with free pure water (Aitchison 1964). Osmotic suction is caused by the presence of soluble salts in the soil water and it is independent of water content and surcharge pressure.

2.4.4 Measurement of Soil Suction

There are many soil suction measurement techniques and instruments in the fields of soil science and engineering. To mention some: calibrated filter paper, pressure plate, polyethylene glycol osmotic system, suction plate, pressure membrane, vacuum desiccator, osmotic tensiometer, centrifuge, and oedometer. The list includes direct and indirect techniques and laboratory and in-situ testing instruments for determining soil suction. Most of these techniques and instruments have limitations with regard to range of measurement, equilibration times, and cost. Of these methods the filter

paper method is selected on the basis of cost, simplicity, and application across a wide range of suction. A brief description of the method is given in the following paragraphs.

The idea of using calibrated absorbent material to measure soil suction can be traced back to 1910's. In 1916 Schull was the first to use especially selected seeds as absorbent material. Instead of seeds Hansen (1926) adopted blotting paper strips. Gardner (1937) was then the one to replace the blotting paper strips by calibrated filter papers assuming that it is more probable to get uniform product of filter paper (Leong *et al* 2002).

There are different filter paper types that can be used for measuring soil suction. Reports by different researchers (including Sibley and Williams 1990; and Leong *et al* 2002) stated that Whatman No.42 filter paper is the most suitable one because of its robustness in use, sensitivity to moisture change, thickness, small pore size distribution and uniformity, and stability under heating to 110°C. The problem of hysteresis of the filter paper is avoided by using the material on the wetting cycle only (Chandler and Gutierrez 1986). Drying the papers at 105-110°C may alter the absorption properties of the filter paper irreversibly and the papers must therefore be used once only and then discarded.

When air dry, the filter paper exhibits a high suction relative to the soil water causing soil pore fluid to pass to the filter paper. The flow continues until the suction in the water, in the filter paper and in the soil are in equilibrium. At equilibrium the filter paper and the soil are applying the same stress to the pore fluid i.e. the suction in the filter paper is the same as the suction in the soil. If the amount of water transferred to the filter paper is small and the sample of soil is large (relative to that amount of water) then the suction established in the filter paper will be nearly the same as the initial suction in the soil (Al-Khafaf and Hanks 1974).

The filter paper can be used to measure either total or matric suction. The filter paper method is based on the assumption that a filter paper will come to equilibrium with respect to moisture flow with a soil having a specific suction. When the filter paper is placed in direct contact with the soil, water

will flow from the soil into the filter paper until equilibrium is reached. When the filter paper is not in contact with the soil, only water vapor flow will occur. In the contact method, the filter paper measures matric suction, and in the non-contact method, the filter paper measures total suction. The filter paper method measures suction indirectly, and the measurement accuracy is dependent on the moisture-suction relationship of the filter paper.

Since the accuracy of the filter paper method is dependent on its moisture-suction relationship, the calibration procedure for the filter paper is very important. A number of calibration curves for Whatman No.42 filter papers have been published in the literature (e.g. Fawcett and Collis-George 1967; McQueen and Miller 1968; Al-Khafaf and Hanks 1974; Hamblin 1981; and Chandler and Gutierrez 1986; Chandler *et al* 1992; Schreiner and Gourley 1993; Houston *et al* 1994; and Leong *et al* 2002). The source of the variation could be due to small differences in the testing procedure or incorporation of different factors that affect soil suction. In this research the procedure followed was according to Schreiner and Gourley (1993) and the calculation of the total suction is made using the generalized formula given in Schreiner and Gourley (1993):

$$h = 10^{5.9 - \frac{T^{0.4}}{15} - 0.091W_p}$$

where: h = suction (kPa)

w_p = filter paper water content (%)

T = temperature °C

2.5 ASSESSMENT OF THE INTRINSIC EXPANSIVENESS OF SOILS

2.5.1 Background

As stated earlier the swelling and shrinking behaviors of intrinsically expansive soils cause severe damage to buildings, road pavements, pipelines, irrigation channels, and other structures founded on them. The damage they cause makes intrinsically expansive soils the major contributors to the burden that natural hazards place on the economy (Nelson and Miller 1992). Evident from this is the importance of accurate

identification of these soils as early as possible, in the site investigation stage, for appropriate sampling, testing, and design so as to minimize not only failures but also costly remedial works. With the idea of obtaining a universal system for assessing the expansiveness of soils many efforts have been made for more than six decades. Different authors suggested a holistic approach of assessment by including geological, field and laboratory assessments. This section is intended to discuss these assessment systems.

2.5.2 Geological Assessment of Expansiveness

Preliminary indication regarding the likely occurrence of intrinsically expansive soil in an area can be assessed from the study of its geology and geomorphology. On the other hand, the type and quantity of the clay minerals present in a soil can be determined using mineralogical assessment techniques.

Formation of expansive clay minerals depends upon presence of a suitable parent material. The parent materials that can likely be the source of intrinsically expansive soils have been discussed in section 2.2.5. Geological maps can be used as first indicators of the possible distribution of the expansive soils based on the presence of the suitable parent rock. In an initial study, all areas underlain by basic igneous or argillaceous sedimentary rocks should be considered as a potential source of the soils. Engineering and agricultural soil maps and local knowledge also serves as a means of identifying potential sources of expansive soils.

The formation of an intrinsically expansive soil not only depends on suitable parent material but also on the physical and chemical conditions under which the weathering takes place (Gourley and Schreiner 1993b). Residual intrinsically expansive soils can be formed from weathering of basic and pyroclastic rocks under poor leaching and poor drainage landform. The effect of poor leaching and drainage landform is insignificant on the formation of expansive soils as a result of weathering of argillaceous sedimentary rocks and transported soils, which previously contain expansive clay minerals. Another very reliable indicator of the presence of expansive soils is gilgai, a small mound that occurs in a regular pattern

commonly in plain areas spacing between 6 and 20 meters from each other (Gourley and Schreiner 1993b). It occurs as a result of the entry of water through major shrinkage cracks and chemical changes within the soil thus causing localized heave.

Clay minerals are largely responsible for the intrinsic expansiveness of soils. Some authors believe that quantifying the type and quantity of the clayey minerals could be used as a means of assessing expansiveness of a soil. A variety of techniques are recommended to serve this purpose. Though a great deal of research has been done in the various techniques, the test results require highly specialized experts to interpret and specialized apparatus required are costly and not economically available in most soil testing laboratories (Chen 1988). A brief description of the various techniques is given in the following paragraph.

X-ray is the most widely used technique that permits estimation of the proportion of clay minerals. The technique measures the spacing between two layers of atoms in crystal structure since each clay mineral has a unique chemical composition that is reflected in its crystal structure. Other popular mineralogical methods include dye absorption, differential thermal analysis and electron microscope. In the dye absorption technique, dyes and other reagents which have a characteristic colour when absorbed by clay minerals have been used in the identification (Chen 1988). The differential thermal analysis is based on the fact that clay minerals show characteristic exothermic or endothermic reactions during heating at a constant rate. The technique is not always correct and presence of mixed layer minerals make data interpretation more difficult (Gourley and Schreiner 1993b). Electron microscope offers a means of directly observing the clay minerals. The main purpose of such examination is to determine mineralogical composition, texture, and internal structure. In general, the use of the mineralogical studies alone in the assessment of expansiveness is not only limited by its cost but also by its inadequacy to estimate the volumetric change behavior of soils.

2.5.3 Assessment of Expansiveness from the Soil Profile

Soil profiling by a competent materials and geotechnical engineer might bring valuable information that can be used toward identifying intrinsically expansive soils. There are several characteristic features which may result from the intrinsic expansiveness and which can be readily identified in the field according to the Jennings *et al* (1973) guide for soil profiling. Some of these features are briefed in the following paragraphs.

One of the primary indicators of intrinsic expansiveness is the type of soil. It is possible to qualitatively classify soil type in the field. This can be achieved among other techniques by polishing a partially dry soil piece with a smooth object (Gourley and Schreiner 1993b). Presence of shiny surface on the surface of the soil indicates the presence of clay minerals in the soil. It is more likely that soils with shiny surfaces may have higher expansiveness than soils with dull surfaces.

The soil consistency is frequently used as a better identifying parameter than the soil type. When dry and slightly moist, intrinsically expansive soils are characterized by stiff and very stiff consistency (Jennings *et al* 1973) and fairly high bulk density (Gourley and Schreiner 1993c). On wetting, the soil expands resulting the lowering of the bulk density and strength and the consistency changes to soft to firm. Due to their low hydraulic conductivity, intrinsically expansive soils tend to wet-up more slowly than non-expansive soils.

Preliminary identification of an intrinsically expansive soil in the field might also be possible using different structural features ranging from large vertical and inclined cracks to micro-shattering. These features can only be used as indicative since their formation is highly dependent on the environment. In a region with high seasonal moisture variation, intrinsically expansive soils can be identified by open or closed fissures, slickensides, and shattering or micro-shattering.

With experience at regional or local level, a set of soil colours might be used as preliminary indicator of intrinsically expansive soils. For instance it is a

common in some region to consider black soils as potentially expansive soils. Colour alone should not be used as full indicator; it has to be combined with other parameters mentioned.

All clayey soils, when moist to dry, will have a high suction. In soils of low intrinsic expansiveness suction decreases rapidly with moisture content increase whereas in soils of high intrinsic expansiveness it requires a relatively large increase in moisture content to reduce the suction. This can be crudely tested by placing a small sample of soil on the tongue (Gourley and Schreiner 1993c). They also noted that a highly expansive soil tend to stick to the tongue for longer than a less expansive soil if both are initially at the same suction.

2.5.4 Laboratory Assessment of Intrinsic Expansiveness

Simple soil laboratory tests, such as index tests, may offer reliable means of assessing expansiveness. Nevertheless, due to lack of basic understanding of the swelling process of expansive soils and lack of a standardized definition of intrinsic expansiveness, swell and heave, the list of assessment techniques, procedures, swell models and expansive soil classifications recommended in the last two decades are a lot. The limitation of all the previous attempts have been identified and documented by different researchers among which Oloo *et al* (1987); Schreiner (1987a); Schreiner (1988); and Nelson and Miller (1992) could be mentioned as good examples. There is no intent to reproduce all the reviews in this dissertation.

It is worthwhile to make a clear distinction between procedures for assessing expansiveness and estimating swell and heave. As stated earlier, intrinsic expansiveness is a soil property whereas swell is a strain measured under a particular set of test conditions and heave is displacement that results from a particular set of conditions and changes of volume. The point of concern in this research is assessment of the intrinsic expansiveness of soils. However, there is no direct means of measuring the intrinsic expansiveness. There have been attempts to develop an indirect means of

assessment by developing models, which correlate tests data of one or more soil properties with swell data.

Generally, models that can be used for assessing expansiveness need to meet certain criteria. Firstly, the testing procedure for determining the soil properties and swell must follow the principles of soil mechanics. To obtain information regarding the soil properties as quickly as possible the test needs to be fast, easy, and cheap to run. Index type tests have been the choice of a number of researchers for this purpose. Routine usage of models also requires simplicity and straightforwardness. Lastly, the model should be reasonably accurate to assess intrinsically expansive soils of broad geological, geomorphological and geographical origin. To achieve this, adequate representation of the possible sources of variables during modeling is important.

Index type tests such as Atterberg limits are not absolute measures of any soil property. Because they are standard procedures used under standard conditions, they are indirect measures of the combined effects of several soil properties (Gourley and Schreiner 1993b). Additionally, because of their simplicity and cheapness to run the tests they have been used frequently in the assessment of expansiveness. A brief summary of the usage is given in the following two subsections.

2.5.4.1 Previous swell models and expansive soil classifications

The literature contains a considerable number of swell models and classification systems for assessing expansiveness of soils. As pointed by Oloo *et al* (1987) and Gourley and Schreiner (1993a) there is little agreement between the authors regarding the necessary input data for determining the models. Some consider that expansiveness is linked to a single parameter. Altmeyer (1955); Ranganatham and Satyanarayana (1965); Snethen (1980); and Chen (1988) have proposed assessment schemes which respectively give the expansiveness as a function of the shrinkage limit, the shrinkage index and the plasticity index (Table 2.1). For soils with clay content between 8 and 65 %, Seed *et al* (1962) also proposed that expansiveness is related to the plasticity index (Table 2.1).

Other authors link the expansiveness to two parameters. This group contains Raman (1967) (quoted in Djedid *et al* 2001); BRE (1980); Pidgeon (1987); and Chen (1988), which are given in Table 2.2. The first of these is based on shrinkage and plasticity index. The second used plasticity index with percentage of soil particles smaller than 2- μm in diameter. Following this is the one based on plasticity index and initial moisture content. The last one is based on the liquid limit and the percentage of soil particles with a diameter less than 74- μm . There are some expansiveness assessment charts that fall into this group. Vijayvergiya & Ghazzaly (1973) (quoted in Djedid *et al* 2001) employed Casagrande classification chart and defined the "A" line by $I_p=0.73 (W_L-20)$ that act as a boundary for assessing expansiveness (Figure 2.5), with expansive soils above the line and non-expansive soil below it. Dakshnamurthy & Raman (1973) also used the Casagrande plasticity chart but further categorized the expansiveness based on the liquid limit. According to them soils with liquid limit 0-20 % are non-expansive, 20-35 % are weakly expansive, 35-50 % are averagely expansive, 50-70 % are highly expansive, 70-90 % are very highly expansive, and >90 % are critically expansive (Figure 2.6).

Table 2.1 Previous works in swell models and classifications based on one parameter.

Author(s)	Model	Classification				
		W_s (%)				
Altmeyer (1955)*	-	W_s (%)	<10	10-12	>12	
		Expansiveness:	High	Marginal	Low	
Seed <i>et al</i> (1962)	$\epsilon_s=2.16 \times 10^{-3} (I_p)^{2.44}$	ϵ_s (%)	>25	5-25	1.5-5	0-1.5
		I_p (%)	>35	20-35	10-20	0-10
		Expansiveness:	Very high	High	Medium	Low
Ranganatham & Satyanarayana (1965)*	$\epsilon_s=4.13 \times 10^{-4} (I_s)^{2.67}$	I_s (%)	> 60	30 - 60	20 - 30	0 - 20
		Expansiveness:	Very high	High	Medium	Low
Snethen (1984)	-	I_p (%)	> 60	30 - 60	20 - 30	0 - 20
		Expansiveness:	Very high	High	Medium	Low
Chen (1988)	$\epsilon_s=0.258e^{0.08381I_p}$	I_p (%)	>35	20-35	10-35	0-15
		Expansiveness:	Very high	High	Medium	Low

* quoted in Djedid *et al* (2001).

Lastly, other authors think that at least three parameters are necessary in order to evaluate the expansiveness of soils. The Holtz and Gibbs (1956) classification and the Holtz, Dakshnamurthy and Raman (1973) classification, given in Table 2.2 belong to this category. The first gives the

expansiveness as a function of the percentage of the clayey fraction, the plasticity index and the shrinkage limit whereas the second linked to the plasticity index, the liquid limit and the shrinkage limit.

The chart developed by van der Merwe (1964) that correlated expansiveness with plasticity index, percentage of particles smaller than 2 mm in diameter and activity (see Figure 2.7) are part of this group After increasing the sample set, this chart was modified by Williams and Donaldson (1980) (Figure 2.8). There is also another chart recommended by BRE (1980) as shown in Figure 2.9.

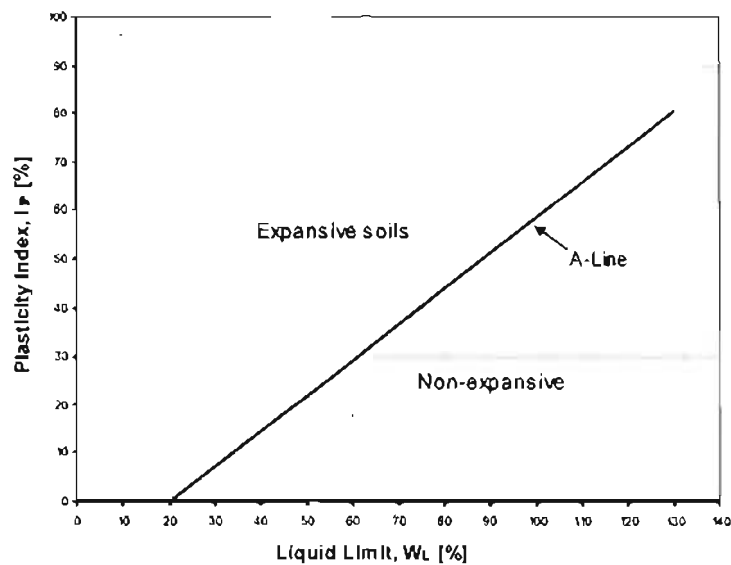


Figure 2.5 Classification chart after Vijayvergiya & Ghazzaly (1973).

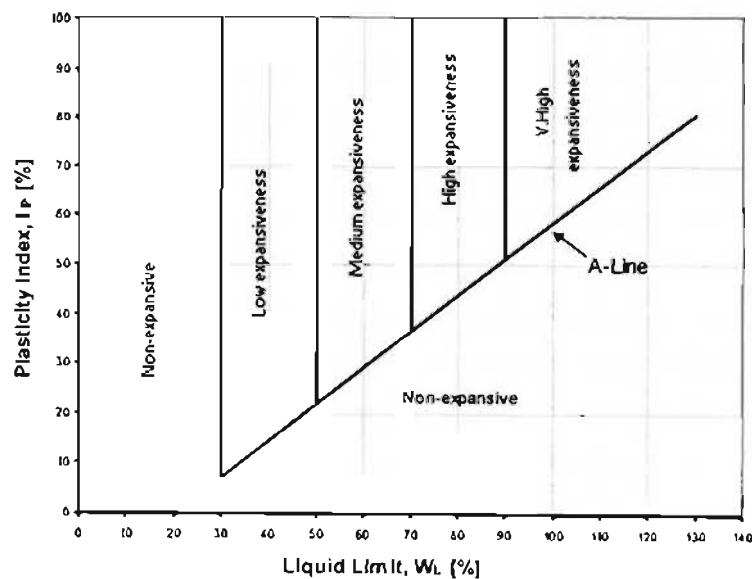


Figure 2.6 Classification chart after Dakshanamurthy & Raman (1973).

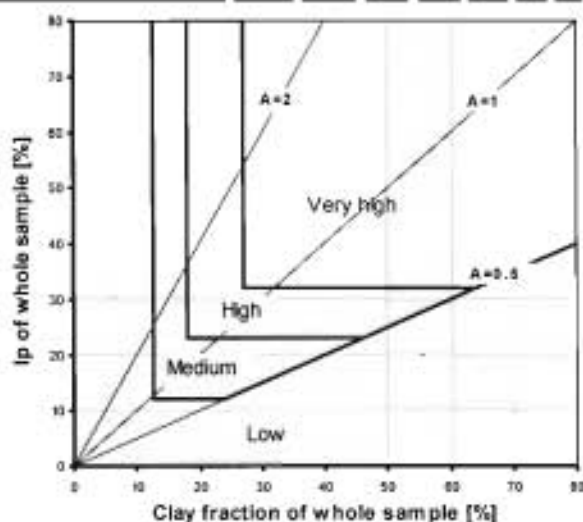


Figure 2.7 Classification chart after van der Merwe (1964).

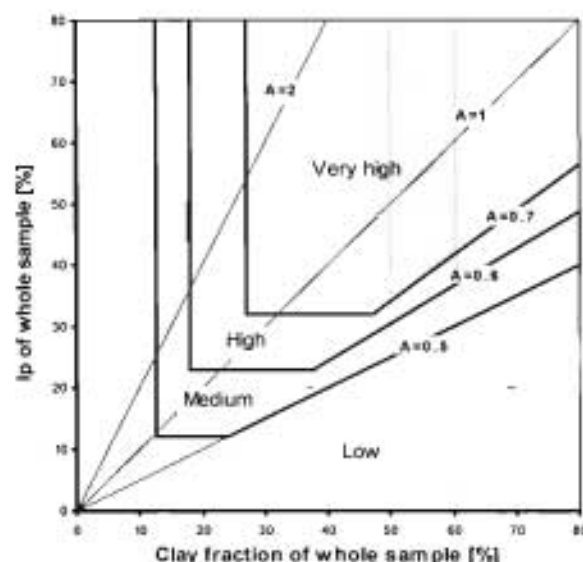


Figure 2.8 Classification chart after Williams and Donaldson (1980).

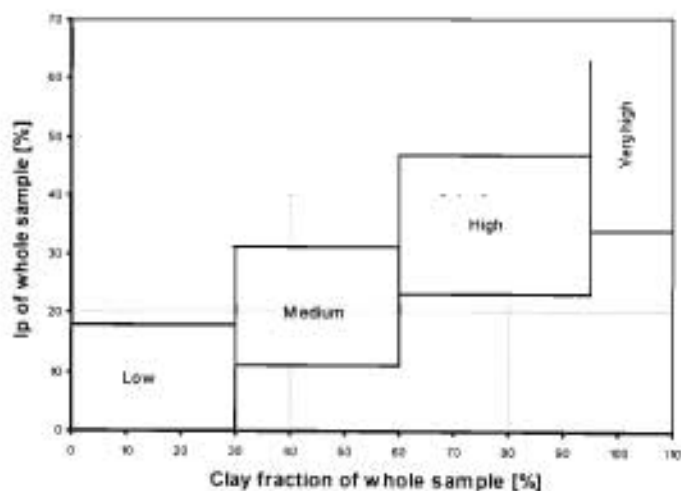


Figure 2.9 Classification chart after BRE (1980).

Table 2.2 Previous works in swell models and classifications based on two and three parameters.

Author(s)	Model	Classification				
Raman (1967)*	-	I_p (%):	>32	23-32	12-23	<12
		I_s (%):	>40	30-40	15-30	<15
		Expansiveness:	Very high	High	Medium	Low
Vijayvergiya & Ghazzaly (1973)*	$\log \epsilon_s = 0.033W_L - 0.083w + 0.458$					
BRE (1980)**	-	I_p (%):	>35	22-35	18-22	<18
		< $2\mu\text{m}$ (%):	>95	60-95	30-60	<30
		Expansiveness:	Very high	High	Medium	Low
Pidgeon (1987)	$\epsilon_s = 0.732I_p - 0.92w + 3.68$					
Chen (1988)	-	W_L (%):	>60	40-60	30-40	<30
		< $74\mu\text{m}$ (%):	>95	60-95	30-60	<30
		Expansiveness:	Very high	High	Medium	Low
Holtz and Gibbs (1956)	-	I_p (%):	>35	25-41	15-28	<18
		I_s (%):	<11	7-12	10-16	>15
		< $2\mu\text{m}$ (%):	>28	20-31	13-23	<15
		Expansiveness:	Very high	High	Medium	Low
Holtz, Dakshanamurthy & Raman (1973)*	-	W_d (%):	<7	7-12	10-15	>15
		W_L (%):	>70	50-70	35-50	20-35
		I_p (%):	>35	25-35	15-25	<18
		Expansiveness:	Very high	High	Medium	Low

*quoted in Djedid *et al* (2001).

** quoted in Gourley and Schreiner (1993b)

The models and classifications discussed so far are only mentioned to give examples otherwise a lot more like these have been recommended. It is evident from the summary that there is no agreement on what parameters to use as necessary input data. This is the simplest limitation of the attempts. The serious drawbacks of the assessment schemes developed are discussed in the following paragraphs.

Firstly, there was lack of a standard definition of relative or intrinsic expansiveness, swell, swell pressure and heave (Nelson and Miller 1992). The unclearness of the terminology resulted in the developed models describing different properties.

Secondly, most of the authors have failed to follow basic principles of soil mechanics in deriving the expansiveness models and classification schemes (Schreiner 1987a; and Oloo *et al* 1987). Uniform soil state conditions have not always been used as the basis of comparing swell for different soil

samples. Either compacted or undisturbed samples of natural soils, both of which include an unknown stress history and unknown micro-fabric in the samples were commonly used. The use of the models to soils of different stress history and suction leads to wrong prediction.

Universal acceptance of assessment systems requires representation of all types of clayey soils of variable geological, geomorphological, and geographical origin. Most of the previous attempts failed to meet this criterion. Often the previous derivations were not comprehensive enough to include many sources of variability in the soil samples. Generally, the numbers of the samples used in the derivations of the models were too small to draw acceptable conclusions (Oloo *et al* 1987).

Last, but not least, exhaustive analysis has never been done to identify possible parameters that can define the models better.

In the last 15 years, there has been a great deal of efforts by Schreiner and Gourley to develop a procedure that considers most of the limitations of the previous attempts. The procedure developed by these two researchers is discussed in the following section.

2.5.4.2 Schreiner and Gourley's Procedure

The previous discussions clearly showed that the earlier efforts for assessing expansiveness are not satisfactory. A different procedure was then recommended by Gourley and Schreiner (1993a) that addressed most of the previous limitations. The basic limitation was not following basic principles of soil mechanics. Comparison of swell data between samples in their procedure was possible since the soils were tested under stable micro-fabric and consistent and known stress and suction history. Details of the procedure are documented in Gourley and Schreiner (1993a). Only a brief description of the procedure is given here.

The first stage of the procedure involves destroying the previous stress history of samples. This is achieved by using slurries prepared at a moisture content slightly greater than the liquid limit of the samples. It follows from this, consolidation of the reconstituted soil sample in a conventional

oedometer to obtain not only sample discs of regular shape for ease of measurement of its dimensions but more importantly to get soil samples of stable micro-fabric. The sample discs then go through unconfined shrinkage using a controlled moisture removal technique with dimensions and weight measured every 24 hours until the samples pass their shrinkage limits. Following each set of measurements, the samples also go through suction tests using the non-contact filter paper method. At last, the dried samples get prepared to fit into oedometer rings where they are soaked to zero suction to measure their swell. The generalized testing stages when plotted in void ratio versus stress and suction is shown in Figure 2.10.

The interpretation of results obtained from such testing involved two important steps. The first one was obtaining a significant parameter for measuring expansiveness. Following this was finding a significant correlation of the chosen measure of expansiveness with the index tests data.

Using eight natural clayey soil samples, collected from Kenya and Sudan, the following three important findings were reported in Gourley and Schreiner (1993a) or Schreiner (1999):

1. Intrinsic expansiveness can be estimated significantly using expansive strain which is defined as follows:

$$\epsilon_{ex} = \frac{e_{15} - e_{SL}}{1 + e_{SL}} \times 100$$

where ϵ_{ex} = expansive strain

e_{15} = void ratio after swelling under 15kPa vertical stress from below the shrinkage limit,

e_{SL} = void ratio at shrinkage limit

2. Strong correlation was obtained between the expansive strain and the plastic limit and the shrinkage limit, which was given as:

$$\epsilon_{ex} = 32.5 + 2.4W_p - 3.9W_s \quad (R^2=94)$$

where W_p = plastic limit

W_s = shrinkage limit

3. Though it would be expensive, stronger correlation was also obtained between expansive strain and swell index (C_s^*):

$$\epsilon_{ex} = 644C_s^* - 18.4 \quad (R^2=97)$$

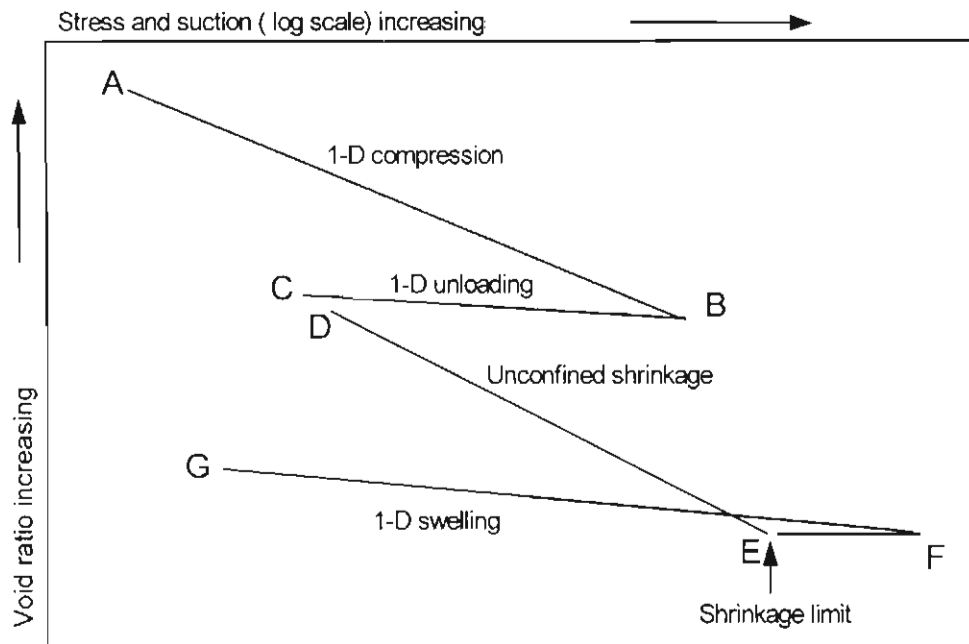


Figure 2.10 Test procedure for assessing intrinsic expansiveness of soils using reconstituted samples (after Gourley and Schreiner 1993a).

To the knowledge of the author, so far there is no other research reported that confirm or otherwise the initiations made by Schreiner and Gourley (1993a). As stated earlier, unlike the other previous attempts the researchers developed a testing procedure that satisfies the principles of soil mechanics. It might be then worthwhile to further expand their ideas and develop a universally useful laboratory expansiveness assessing scheme.

2.5.5 Summary

Three procedures have been discussed which are commonly used in the assessment of expansiveness: geological, field and laboratory assessments. Not all of these procedures would be required for every project but definitely one alone would not be enough. As the three of them can support each other, a greater confidence in the assessment can be obtained with the combined use.

Geological and geomorphological studies may provide crude understanding of the expansiveness of soils. A generalization should not be taken based on these assessments. They should only be used as a preliminary information

that need to be supported with laboratory assessments. Mineralogical studies identify and quantify clay minerals of soils but fail to address the volumetric change behavior of soils. Moreover, most of the techniques used are highly advanced requiring specialized personnel and equipment, which makes them costly for routine assessment of expansiveness (Chen 1988). Nevertheless, results of such tests have importance to strengthen the field and laboratory assessments.

Field assessment techniques in general provide a qualitative indication of intrinsic expansiveness. The reliability of the techniques mainly depends on the experience of the geologist, engineering geologist, or materials and geotechnical engineer involved in the assessment and the assessment can only be used as rough indicators.

By far the most reliable means of assessing expansiveness could be laboratory assessment. Unfortunately, most previous attempts have limitations which are bottlenecking their confident use. The only promising attempt so far is the one made by Schreiner and Gourley that does need further research, which is the intention of this research.

CHAPTER THREE

3. PLANNING OF THE RESEARCH

3.1 INTRODUCTION

In the previous chapter it was made clear that further research is required to assess expansiveness of soils. This chapter first gives a brief summary of the motivation for the research. It then describes the scope of the research intended and finally it gives the main objectives that are expected to achieve by the research.

3.2 MOTIVATION OF THE RESEARCH

Despite the continuous effort to find a universal means of assessing expansiveness, still expansive soil engineering is missing that important tool. The tool would have been used to minimize the damaging effect of the soils in the earlier site investigation stage of any project.

The discussion in Chapter two made it clear that achieving a successful assessment of expansiveness is only possible by developing a laboratory assessment technique that reinforces the geological and field assessments. Lack of such important tool in the field of materials and geotechnical engineering attracted a number of researchers who tried to develop swell models and classify expansive soils. Critical research analysis on these attempts by Oloo *et al* (1987) challenged their validity. Following this, a continuous research made by Schreiner and Gourley yielded a new laboratory procedure for assessing intrinsic expansiveness. No research is yet reported that assesses the validity of this initiation. Moreover, there is a growing interest of materials and geotechnical engineers for an internationally useful laboratory expansiveness assessment scheme. The universal importance of such tool motivated the author to further research on the initiation by Gourley and Schreiner (1993a).

3.3 SCOPE AND PURPOSE OF THE RESEARCH

The research by Gourley and Schreiner (1993a) or Schreiner (1999) reported a new approach toward assessment of expansiveness. The procedure recommended by the researchers allows comparison of expansiveness of soils regardless of stress history and micro-fabric, which were the bottlenecks of previous researches. Destroying stress history was made possible by using reconstituted soil samples, a common technique in saturated soil mechanics. The possibility of collapse of samples on wetting, due to unstable micro-fabric, was minimized by consolidating the samples in odometer. The researchers tested the practicality of the procedure on eight clayey samples from Sudan and Kenya. Two correlations were developed from the initiation. The first one is between expansive strain and the plastic and shrinkage limits for assessing expansiveness from simple standard index tests. The other one is between expansive strain and the swell index that can be used to assess expansiveness from laboratory oedometer tests on saturated reconstituted samples.

This research is mainly proposed to verify the validity of the procedure outlined with intent of developing a universally applicable scheme for assessing expansiveness. To serve these purposes the author intended to include as many as possible clayey samples of diversified clay particles percentage and geological, geomorphological, and geographical origins.

3.4 OBJECTIVES OF THE RESEARCH

Under the primary goal of obtaining a universal system for assessing intrinsic expansiveness of soils this research targets obtaining:

1. soil parameters that best represent intrinsic expansiveness; and
2. a significant correlation between soil classification parameters and intrinsic expansiveness in terms of index test data.

The methodology employed to achieve these objectives are discussed in the following chapter.

CHAPTER FOUR

4. RESEARCH METHODOLOGY

4.1 INTRODUCTION

The research methodology of Gourley and Schreiner (1993a) was used in this study besides minor refinement with observation from a wide range of clayey soils used in the study. The details of these procedures are discussed from section 4.2 to 4.5. At the end of the chapter the statistical methods employed to analyze the data obtained from the tests are discussed.

4.2 SAMPLING

4.2.1 Schreiner and Gourley's Research

The Schreiner and Gourley research was made on six samples collected from Kenya and two samples from Sudan. The research included clayey soils that are known to be highly expansive and some clayey soils that plot to the right of the 'A line' of Casagrande's Plasticity Chart. The significant limitation of the research was lack of significant number of samples representing clayey soils of different geological, geographical and geomorphological origin, and diversified liquid limits, clay contents and clay mineralogy. This limitation is one of the reasons for the initiation of this study.

4.2.2 Samples Used for this Research

There is no doubt that to develop a universally acceptable identification and classification tool an exhaustive analysis needs to be done. This study is no different. An attempt is made to include as many samples as possible and to make them as diversified as possible.

All soils containing clay minerals have intrinsic expansiveness, as the characteristic is related to the ability of a clay mineral within a soil to adsorb and absorb water (Schreiner 1987b; and Oloo *et al* 1987). Therefore, it was preferred to use any clayey soils for this study.

After it was confirmed to include any clayey soils a decision was made to use either natural or artificial clayey soils. Bearing in mind the intended representation of the reality preference was given to natural clayey soils. There was also a consideration of using artificial clays and blends of artificial and natural clayey soils to represent a specific characteristic, such as liquid limit, mineralogy etc, but due to the time limitations, the study was restricted only to naturally occurring clayey soils.

Following this it was necessary to specify the sources of the natural clayey samples. The research of Schreiner and Gourley reported Kenya and Sudan as the source of the samples used. It was decided to further broaden the source of data by obtaining samples from South Africa and Eritrea. This would definitely give a wider range of geographical and climatic diversification of the soil sample sets. Initially there was an intention to include samples from Italy and Australia but both did not pass beyond the planning stage.

Having identified the countries from which to obtain the samples, representing the most commonly known clayey soils in both countries followed. With intent to get a comprehensive understanding of the intrinsic expansiveness an attempt was made to include all the possible factors that can lead to variation in the properties of natural clayey soils. Some of the major factors considered include geological origin, topography, clay mineralogy, mode of soil formation, and amount of clay-sized particles.

Intrinsic expansiveness may be affected by the clay minerals present. Smectites are highly intrinsically expansive, illites and kaolinite less so. Iron and aluminum oxides, allophanes, and halloysites have little intrinsic expansiveness. The most abundant form of smectite groups is montmorillonite. Montmorillonite clay minerals are derived from basic and intermediate igneous and metamorphic rocks containing calcic feldspars and ferromagnesian minerals (Gourley and Schreiner 1993c). Kellar (1964) (quoted in Gourley and Schreiner 1993c) also stated that acidic igneous rocks such as granite might also lead to formation of montmorillonite. Even where rock origin is the same, it is possible to get both reddened kaolinite

groups and dark smectite groups depending on weathering conditions and environment. The smectite clay minerals are often formed under poorly drained flatter ground whereas the kaolinite clay minerals are formed under well-drained sloping ground. To include such clayey soils in the sample set of this study, it was decided to add clayey soils derived from basalt and granite under well drained and poorly drained ground conditions.

Natural clayey soils can be residual, transported, or pedogenic. The type of residual clayey soil formed depends on the age of the land surface, climate during soil development, composition of parent rock, and topography of formation (Gourley and Schreiner 1993c). Transported clayey soils can be categorized into two groups. The first group includes those clayey soils formed from transportation and deposition of weathered soil. The second group encompassed those soils that have undergone additional weathering after transportation. Where possible these sources of variations were considered during the choice of the sample. Pedogenic clayey soils, such as lateritic clay, were also included in the sample set.

The Schreiner and Gourley research initiated the uncertainty of the empirical classification of expansive soils using the Casagrande's Plasticity Chart. The uncertainty came with the observation of some soils of high intrinsic expansiveness to plot to the right of the 'A line', which is supposedly restricted to non-expansive soils. It was then decided to consider such clayey soils in the study.

Although the magnitude of swelling is much less than that of smectites, there is a report for intrinsic expansiveness of kaolinite clay (Seed *et al* 1962; and Brackley 1975); and illite clay (Seed *et al* 1962). These soils are often known as non-expansive. To give a better depth to the study some of these 'non-expansive' soils were included.

Previous classification of expansive soils used parameters such as plasticity index, liquid limit and clay content. To investigate if there is any significant relationship between these parameters with intrinsic expansiveness an effort was made to encompass clayey soils with a broad range of these

parameters. The samples used had a plasticity index varying from 5 to 59 %, a liquid limit varying from 26 to 88 %, and clay content varying from 7 to 77 %.

On the basis of the above factors, twenty-two natural clayey soil samples with diverse geological, climatic, and geographical origin, covering wide ranges of liquid limits and clay content, and representing different soil types were included in the study. A detailed description of these soils are given in the next two subsections.

4.2.2.1 Samples from South Africa

A total of seventeen samples were collected from South Africa. Of these samples, twelve were collected from the vicinity of Durban and Pietermaritzburg. Most of these samples were obtained from exposed profiles, within depths of 0.5-1.5 m from the ground surface. These soil samples provided a reasonable coverage of the possible clayey soils in the area. The collected clayey soils had geological origin of Pietermaritzburg shale, Dwyka Tillite, Dolerite (or Basalt), Granite, Schist, and Laterite. The determination of the parent rock for these soils was made by Dr R.R. Maud, from Drennan, Maud and Partners, during sample collection.

In-situ weathering product of Pietermaritzburg shale - originally estuarine clayey material (Ware and Jermy 1998) - is known for producing highly expansive soil in the area. Five samples, one from Pietermaritzburg and four from Durban, were obtained to represent the possible slight differences in the soil type derived from the shale. The sample from Pietermaritzburg was a colluvial deposit, which was collected from Howick area. It has a blocky structure and dark gray colour. Topography, soil colour and soil structure differences were considered while choosing the Pietermaritzburg shale derived clayey soils from the Durban area. One of the samples from Durban was collected from the bottom of the hill in Sea Cow Lake. Other two were obtained from hilltops of Sea Cow Lake and Mount Edgecombe areas. The last sample was from Kwa Mashu, which was collected some years ago by Drennan, Maud and Partners.

The other rock type in the region known to produce potentially expansive soil is Dwyka Tillite. A soil sample was obtained from Ridgeview to represent the soil derived from this rock. The soil has a dark yellow colour and a slickensided structure.

Weathering of Dolerite or Basalt can lead to formation of various types of soil. The major types of the soils derived from this type of rock are black clay, which is mostly expansive and reddish clay, which is less expansive. Both, black and reddish clayey soils were collected from the Mount Edgecombe area in Durban.

To represent clayey soil of in-situ weathering product of Granite, another common rock in the region (which exist as an intrusive (Maud 1989)) a sample was taken from the bottom of Botha's Hill in Durban. The sample has a dark brown colour.

A reddish yellow clayey soil sample, derived from Schist, was also obtained from Botha's Hill, Durban. The soil has a shattered structure.

From Pedogenic soils a dusky red clay sample derived from in-situ weathering of Laterite was obtained from Hillcrest in Durban.

Soil sample from Berea Red sand, which is in-situ weathered aeolian deposit, was also included in the sample set.

The remaining five samples were from Westmead, Stanger, Kilnerpoort, Rondebosch, and Kilbarchan. These soil samples were collected many years ago by the CSIR (Council for Scientific and Industrial Research) and had been brought to the Soil Mechanics Laboratory of University of KwaZulu-Natal for the study some years previously. These samples were obtained from areas of severe swelling and shrinkage problems.

4.2.2.2 Samples from Eritrea

Five samples were brought from Eritrea. These samples covered the range of clayey soils traditionally known for high expansiveness, non-expansiveness, and moderate expansiveness. From the highly expansive soils, the

commonly known 'Black Cotton' soil derived from basalt was included. Within this group of clayey soils an effort was made to represent soils developed at different topography. Thus, samples were collected from Adi-Guaedad, and Mai-Harish, the former representing highland and the latter lowland. To represent non-expansive clayey soils, one sample was obtained from soil stockpile, commonly used for making bricks and traditional kitchen utensils, in Gura area. The other was a reddish clayey sample derived from Laterite from Asmara. Additionally one colluvial clayey sample, which is derived from the weathering of the surrounding Granodiorite rocks, was collected from Korbarya.

4.2.3 Amount of Sample Collected

A total of 2 kg was collected for each soil sample. In cases where the soil seemed more silty, sandy, and gravelly a slightly larger sample size was collected to avoid shortage.

4.3 CHOICE OF TEST PROCEDURE

For the full description of the relevant engineering properties and the determination of the intrinsic expansiveness of the soil a series of tests were recommended for the study. The tests include:- physical description of the samples, grain-size distribution, index tests, specific gravity, and volume change tests. The motivations behind the choice of these particular tests and the specific procedures followed to meet the study objectives are discussed in greater details in the following subsections.

4.3.1 Physical Description of Samples

Intrinsic expansiveness of a soil cannot be directly observed in a soil but characteristic features which may result from intrinsic expansiveness can be identified. These features can be described with good degree of accuracy using the guide given by Jennings *et al* (1973). Like any soil description for engineering purposes moisture condition, colour, consistency, structure, soil type, and origin of the soil should be included. Additional information such as organic content should also be included in the description.

The moisture content of the soil can be used to estimate where the sample is relative to the shrinkage limit. According to Schreiner and Gourley (1995) a soil described as slightly moist is near to the shrinkage limit.

Colour is some times used for identifying, regionally or locally, soils with high intrinsic expansiveness. For instance the so-called *black cotton* soil which is well known for having high intrinsic expansiveness is common in India, Australia and some African countries including Egypt, Sudan, Eritrea, Ethiopia and Gana. This does not necessarily mean that all black soils have high intrinsic expansiveness or all intrinsically expansive soils are black. Some good examples are given by authors that support this argument. Significant intrinsic expansiveness was recorded in brown clays (De Bruijn 1975); yellow montmorillonitic clay (Brink 1955); pinkish to white montmorillonitic soil (van der Merwe 1955); (Andesitic derived light gray to pink clays (Gourley and Schreiner 1993b); and red clay of Kenya (Dagg and Russam 1966; and Gourley and Schreiner 1993b). Therefore soils of red, yellow, orange, black, brown, olive, and gray colour were included in the study.

All clayey soils have the characteristics of increase in consistency with drying. The typical consistencies of soils with significant intrinsic expansiveness ranges between stiff to slightly stiff when slightly moist to dry and soft to firm when wet (Gourley and Schreiner 1993b). Stickiness and the time required for wetting up increases with increase of intrinsic expansiveness.

Structure of a soil may give some hint about the expansive nature of clayey soils. Structural features of soils of significant intrinsic expansiveness include open or closed fissures, slickensides and shattering or micro-shattering. These features are more commonly observed in areas with a wide range of seasonal wetting and drying (such as Africa) than in areas where the environment remains relatively constant. Under these conditions the absence of these structures does not necessarily mean the soil has no intrinsic expansiveness. In this study an attempt was made to represent

soils of the same origin but of different structures to observe if there exists difference in intrinsic expansiveness.

Soil type in general and clay content in particular has been used as an assessment of a soil's intrinsic expansiveness. Some authors disagree with this approach. Clayey soils usually contain more than one clay mineral with different intrinsic expansivenesses. It is evident that the increase in intrinsic expansiveness will not be the same if the clay contents of two soils are increased for instance by smectite and kaolinite groups respectively. One of the objectives of this study is to prove or disprove the above mentioned argument by checking if a significant relationship between the clay content and intrinsic expansiveness does exist.

The origin of the soil may also gives a suggestion whether or not there is a potential for the formation of expansive soil. For example soils of basic igneous rock origin (like basalt, norite, dolerite, andesitic lava) have the potential of deriving clay soils which are either black (that develop under poor drainage condition) or red soil (that develop under drained ground condition). Of these two, the former soils are known for high expansiveness. Similarly, soils with argillaceous rock origin (such as shales, mudstones, tillite) have a similar potential of deriving clayey soils which are highly expansive.

4.3.2 Grain-Size Distribution

One of the major factors affecting the intrinsic expansiveness is the grading of the soil (Schreiner 1999). As pointed out in section 2.5.4.1 much of the previous researches recommend an empirical relationship between the clay content and soil expansiveness. This was among the missing information from the data set given by Schreiner and Gourley (1993a). This parameter has been included in this study to find out if there is any relationship with intrinsic expansiveness.

Laboratory analysis of particle size distributions are usually based on combined sieving and sedimentation methods. Sieving is the process of separating particles coarser than 75- μm through a series of sieves. There

are two methods of sieving: dry and wet. Dry sieving is usually the best choice for purely cohesionless soil. For cohesive soil the recommended method is wet sieving. Therefore wet sieving was employed in this study.

Sedimentation is a process of obtaining the distribution of particles smaller than 75- μm based on Stokes' equation. The analysis is usually made by either pipette or hydrometer method. In the pipette method, sample withdrawal with a pipette is made at certain times and depths to determine the amount of particles less than a certain size that are still in suspension. In the hydrometer method, a hydrometer is used to measure suspension density at various times, thus reflecting the amount of particles which remain in suspension after a certain settling time. Due to its simplicity and rapidness the hydrometer method was employed in this study.

4.3.3 Specific Gravity

The specific gravity of particles is required for determining the soil porosity or void ratio, which can be related to fabric structure (Geological Society Engineering Group 1990). In the previous set of data, provided by Gourley and Schreiner (1993a), the specific gravity was calculated from the measured volume, measured moisture content and mass of dry soil of odometer test samples. Laboratory specific gravity determination technique was employed in this study.

Generally there are three methods of specific gravity determination: gas jar, small pycnometer, and large pycnometer. The first method is suitable for all soils including those containing large particles. Small pycnometer is used for soils containing clay, silt and sand sized particles. The third method is suitable for soils containing up to medium sized gravel. Because of the suitability for the type of soils collected, the small pycnometer was used for this study.

In the specific gravity test it is easier to begin the test with an oven-dried sample but this is not recommended for some clayey (Geological Society Engineering Group 1990) and organic soils. Drying of some clayey soils can result in the loss of intra-particle water which can lead to an

underestimation of the soil's specific gravity. In organic soils the effect of drying is related to the difficulty of rewetting the soils. Therefore, in such cases it is advisable to test the samples at their natural water content. The oven-dried weight of the sample can either be obtained by drying the sample at the end of the test or by determining the moisture content on separate sample. Because of the possibility of such susceptible clayey soils and soils with some organic matter in the sample set, the second option was followed in this study.

4.3.4 Index Tests

Soil index tests as quantified by Atterberg limits (such as liquid, plastic, and shrinkage limits), are the most widely used indicator of expansive potential (Nelson and Miller 1992). More specifically, most of the previous attempt toward development of a universal classification scheme for expansive soils used these parameters. Since one of the major objectives of this study is to find the most strongly linked of these parameters to the soil's expansiveness, all liquid, plastic and shrinkage limit tests were included in the series of laboratory tests.

Presently, two methods are in wide use for the determination of liquid limit of soils: Casagrande and cone penetration methods. One of the main limitations of the Casagrande method is that it is difficult to adapt this method to soils with low plasticity because the soil mass slides rather than flows towards the groove (Sridharan and Prakash 2000b). Also, there is a problem in cutting the groove in such soils. In order to overcome the various limitations of the Casagrande method, the cone method has been introduced. However, researches such as that of Sridharan and Prakash (2000b) showed that the Casagrande method gives higher liquid limits than the cone penetration for liquid limits higher than about 60 %, and lower values when the liquid limit is lower than about 60 %. To be consistent and because of its ease and probability of giving reproducible results the cone penetration method was adopted in this study. If the models obtained from this research include liquid limit as a variable it will then be important to

convert Casagrande results into that of cone penetration using appropriate references for accurate usage of the models.

Plastic limit, on the other hand, is usually determined by roll or thread method. The same method was employed for this study.

Shrinkage limit determination involves measuring the change in volume with reduction of the moisture content of the soil with drying. Two techniques exist for measuring the change of volume of soil samples: mercury bath and wax coating.

Most standard procedures for determining the shrinkage limit use mercury bath. As mercury is a hazardous substance the standards recommend its usage in a fume cupboard or in presence of an air extractor fan. Thus, the hazardous nature of mercury limits its use in ordinary laboratory.

The mercury bath method may underestimate or overestimate the volume of a soil. One of the main reasons for this is that the mercury tends to penetrate into the specimen through the voids and is often too difficult to make sure that all the mercury has escaped from the voids with drying. The other uncertainty of the mercury method is related to the density of the liquid. Mercury is one of the liquids with very high density. So, the volume of a specimen immersed in the liquid might be reduced due to the influence of the load applied from the liquid into the specimen.

Related to the above-mentioned problems and uncertainty of the mercury method some standards recommend using wax coating system instead. This method also has its own limitation; it might underestimate or overestimate the volume. This is due to the fact that the wax might penetrate into the specimen through voids or some voids might be left uncoated between the surface of the specimen and the wax layers.

After reviewing the limitation of the widely used procedures Gourley and Schreiner (1993a) recommended a vernier calliper with precision of 0.005 mm to measure the thickness and diameter of regular disc shaped soil specimens. These dimensions are then used to calculate the volume of the

specimen. This approach, besides improving the accuracy of the volume measurement, is easy and can be used in an ordinary laboratory by any qualified laboratory technician without any health risk. The procedure of sample preparation for shrinkage limit determination (see section 4.4.2.2) also gives an extra benefit, as it is prepared in such a way as to give enough strength for the specimen. The strength acquired allows for better handling and measurements. Due to these benefits, a vernier calliper was used in the volume measurement in this study.

4.3.5 Soil Suction

Soil suction (negative water pressure) is one of the stress variables of unsaturated soils in general and expansive soils in particular that controls volume change of the soils (Schreiner and Gourley 1993). There are both direct and indirect methods of soil suction measurements. Of the available methods, the filter paper method, which provides an indirect means for measuring soil suctions in the laboratory, is commonly chosen because of its ability to measure a wide range of suction, inexpensively and relatively simply.

The filter paper method assumes that when initially dry filter paper and a soil specimen are allowed to stay together in an air-tight container for approximately seven days (Swarbrick 1995), the specimen and the filter paper are assumed to attain the same suction. With this understanding, both total and matric suction can be measured using the filter paper. The total suction can be measured using the non-contact method. The total suction assumes that a soil and a filter paper reach suction equilibrium through vapor flow without allowing contact. On the other hand the matric suction is measured if the filter paper is allowed to absorb water through fluid flow (contact method). In this case the filter is kept in contact with the sample.

4.3.6 Swelling Test

Different procedures are in use for determining the swelling of clayey soils (see section 2.3.2.6). The most common swell measurement tests involve

the use of the one-dimensional consolidation apparatus, the oedometer. An ordinary one-dimensional oedometer test was employed in this study because of its simplicity.

4.4 SAMPLE PREPARATION TECHNIQUES

The thoughts behind the choices of test procedure for the study were discussed in the above subsection. The following is a discussion of the sample preparation techniques employed, starting from the time of acceptance of the sample in the laboratory till the samples were ready for the different chosen laboratory tests. For simplification, the sample preparation is discussed as general and specific. The general sample preparation is dealing with techniques involved to prepare the sample for respective laboratory tests. In the specific sample preparation, further processes involved to prepare the samples for each laboratory test are explained.

4.4.1 General Sample Preparation

Most of the soils collected were in a dry condition by the time they reached the laboratory, except a few. In either of the cases great care was taken in order to preserve the received moisture content throughout the laboratory-testing period, unless the test required otherwise. This was to avoid alteration of the physico-chemical behavior of the soil from the initial condition due to a decline of the soil moisture. For this reason it was more preferable to use wet sample preparation technique, out of which representative samples would be obtained for the series of laboratory tests.

The wet sample preparation started by wetting up the whole 2 kg size of each sample with water in a pan till a slurry that could easily flow was obtained. To avoid any variation of the soil water chemistry, distilled water was used during the whole sample preparation process. The slurry was left for 3 to 7 days, depending on the soil type, to give time for the water to completely separate the individual particles. Less time was given for the clayey soils with significant amount of sand and silt compared to pure clay. Most of the clayey soils known to be highly expansive required much more

time due to their low permeability characteristic. To facilitate the process of separation the samples were mixed for few minutes every day and water was topped up where required. When a reasonable separation of the individual particles was thought to have achieved, the sample was further mixed for fifteen minutes using a mechanical mixer. The slurry was then allowed to settle and slightly harden by leaving open the sample pan in a temperature-controlled laboratory.

The soil particles in the slightly hardened sample are expected to be layered, with the coarser particles at the bottom and lighter particles at the top. Additionally due to the circulation of the mixer at the center, coarser particles will concentrate at the center. Considering these variations the subdivisions to get a representative sample for the volume change tests, index tests, grain size analysis, and specific gravity was planned.

Each sample was split using a spatula in such a way as to produce eight slices, all the slices meeting at the center of the sample. In order to obtain representative samples alternate slices were obtained for each test. A total of four sub-samples for the four major tests, that are assumed to represent the original sample, were collected from each sample.

4.4.2 Specific Sample Preparation

Wet samples were obtained for each of the four major tests that the study concentrated on. Further sample preparation is often required to make the samples ready for the methods of testing. All the required sample preparation procedures for each of the chosen tests are discussed in the following subsections.

4.4.2.1 Particle-size analysis

As most of the samples contain sand, silt and clay sized particles the particle analysis should involve wet sieving and hydrometer analysis. Wet sieving allows determination of the particle size distribution down to the fine sand size and the hydrometer analysis allows the determination of the rest of the particle sizes.

In most of the soil samples collected, beside some few coarser particles, the dominant part of the soil was less than sand particle size. The sample size required for both types of tests was not more than 100 g. So some of the samples had to be reduced to this size ahead of the test.

Particle size analysis requires a dry soil sample. To obtain the dry sample the soil was oven dried to a constant weight prior to testing. Considering the possible effects of drying at the ordinary temperature (105-110 °C) it was preferred to dry the samples at a temperature of 50 °C. Where the dried sample was in excess of 100 g, it went through additional preparation to obtain the required sample size. A small size riffing box was used for the process of splitting the soil.

The sedimentation test requires sample not more than 50 g. In some cases when the silt and clay particles, passing 75- μm , were in excess extra sample preparation was done. That is, the sample that passed 75- μm was oven dried at 50 °C and it was subdivided into the required weight.

4.4.2.2 Specific gravity tests

Part of the sample separated for specific gravity was not mixed properly in the first hand and it had some wetness as well. So the sample was spread for air-drying in the laboratory where the temperature is less than 30 °C. The sample was continuously mixed for the purpose of facilitating the drying on one hand and for obtaining a homogenous soil on the other hand. After two to three days of spreading, the soil was collected and divided using a riffing box to obtain a representative sample of 50 g.

The small pycnometer test is usually performed on samples whose particles are less than 2 mm. As some of the soils of the study had particles that could retain on 2 mm sieve, to be on the safe side all the samples were passed through sieving on the 2 mm sieve. Those particles that retained were crushed till they passed the sieve. In this way samples less than 2 mm in size were prepared for the specific gravity test. The sample prepared in such way was further divided into four, each 10 g, using riffing box, of which the two went through the specific gravity test.

4.4.2.3 Liquid and plastic limit tests

For both liquid limit and plastic limit tests, the wet sample was mixed with distilled water and sieved using a 425- μm sieve. The fraction of soils passing the sieve was collected in a pan. The pan was placed in an oven until most of the water floating on the soil was evaporated. This drying process was done under continuous supervision to avoid complete drying of the water from the surface. After most of the surface water was evaporated the sample was mixed and allowed to dry further by continuous mixing till the desired dryness for the liquid limit is achieved. The sample for plastic limit was obtained from the one prepared for liquid limit when the soil was dry enough.

In some of the highly plastic clayey samples evaporation of the water in the oven was impractical, as the particles took too long to settle and separate from the water. In such cases the sample was air dried in the laboratory. Use of air fan and continuous mixing also facilitated the process of drying. The continual mixing besides facilitating the drying helped in obtaining a homogenous sample.

As it was discussed in the beginning of this subsection the sample prepared for liquid and plastic limit tests were only those passing 425- μm sieve. The liquid and plastic limits scarcely represented the whole sample since it was common for significant fractions to be retained on 425- μm sieve. An empirical correction has been recommended by a number of researchers to represent the whole sample. Of which the one recommended by Gourley and Schreiner (1993a) and Schreiner (1999) is as follow:

$$W_L(\text{whole soil}) = W_L \times (\% \text{ passing } 425\text{-}\mu\text{m})/100$$

$$W_p(\text{whole soil}) = W_p \times (\% \text{ passing } 425\text{-}\mu\text{m})/100$$

Where: W_L = Liquid Limit

W_p = Plastic Limit

% passing 425- μm as obtained from particle size analysis.

In addition, the plasticity index of the whole soil was calculated using the general formula:

$$I_p = W_L (\text{whole soil}) - W_p (\text{whole soil})$$

The same approach was followed in this study in order to represent to the index test results of the whole sample.

4.4.2.4 Volume change tests

The intrinsic properties of fine-grained soils may be a valuable tool for understanding the behavior of natural soils (Burland 1990). One of these properties is the *intrinsic expansiveness*, which was introduced by Schreiner (1987a) to describe the one-dimensional swelling behavior of a reconstituted soil sample. The properties of reconstituted clayey soils are termed *intrinsic* since they are inherent to the soil and independent of the natural state (Lutenegger and Cerato 2003). This means that the intrinsic properties should only be related to the soil composition and constituents and are not influenced by soil structure.

It has been mentioned by different authors including Schreiner (1987a) and Oloo *et al* (1987) that one of the drawbacks of most of the previous attempts for obtaining a universal expansive soil classification system was related to the use of soils of different stress history in the comparisons. This was the reason for site-specific validation of the previous attempts.

The other observation that attracted some researchers, such as Schreiner and Burland (1991), was the potential of collapse during oedometer swelling tests. Saturated soils have a zero swell potential. Swell estimates are made on dry samples to simulate the worst scenario. The previous swell prediction attempts were based on either compacted or undisturbed natural samples, which may have had unstable micro-fabric that could potentially add errors to the prediction. The other observation made was the variability of the swell depending on the initial moisture content or liquid limit.

A solution that resolves the above-mentioned two major problems was introduced by Gourley and Schreiner (1993a) from saturated soil

mechanics. Saturated soil modeling has been possible with the introduction of reconstituted sample preparation. The sample preparation involves two major steps. The first step is to destroy the memories of the structure, which is made by forming slurry at a moisture content slightly greater than the liquid limit of the soil. The final stage of the preparation involves consolidation to the desired initial test condition. This sample preparation technique avoids the effect of unknown stress history and fabric and will ease the comparison between samples of the same soil prepared under different stress condition or different soil types under the same stress conditions. The same procedures are used in this study. The details of the different stages of the procedure are given in the following paragraphs.

Until liquid limit test results were obtained, the samples split for the volume change tests were stored in a controlled laboratory where the temperature was kept below 20 °C. From the experience of Burland (1990) reconstituted sample can be prepared from natural soil deposits if the soil is mixed with water to give an initial moisture content ranging from approximately 1.25 to 1.5 times its Liquid Limit. The reconstituted sample preparation followed for this study was the recommendation of Burland (1990).

Once the liquid limit results were obtained, the samples were sieved to obtain part of the soil sample smaller than 2 mm in diameter. The moisture content of this fraction of the samples was determined using the oven drying method. From the results of moisture content, liquid limit and weight of sample, the amount of water that should be added was decided. In most cases an attempt was made to make the moisture content about $1.25 \times W_L$. During preparation, the sample remains submerged in water inside the oedometer ring to avoid entrapment of air. In such instances soils of high permeability may gain more water. To avoid gaining of more water than the required amount, for those soil samples which were known to have high percentage of sand and silt particles, the moisture increment was targeted for $1.2 \times W_L$.

Finally, to obtain a homogenous sample in terms of particle size and moisture content distribution each of the samples had to undergo a

thorough mixing using a spatula in a pan. Frequent mixing was done on each sample throughout a day to avoid soil lumps and to facilitate the homogeneity of the sample. Depending on the soil type the process of mixing was continued to a maximum of five days. The highest time was given for those samples which are highly plastic clayey soils. During the process of mixing to avoid loss of moisture from the soil the pan was placed in a sealed plastic bag. The process of mixing and storing of the sample in-between was made in a temperature-controlled laboratory. With the use of controlled laboratory the risk of condensation was also minimized. The reconstituted sample prepared in such way was then ready for de-airing.

In all the preparation of the reconstituted samples the water used was de-aired distilled water. The reason for the choice of the distilled water was to avoid any chemical reaction with the soil water and thus alteration of the soil sample. The distilled water was further de-aired in order to obtain a sample that was completely free of entrapped air. The de-aired distilled water was stored in the temperature-controlled laboratory to keep the temperature of the water and the soil constant. One clear benefit of this is to avoid the risk of condensation.

The slurry will definitely have entrapped air in between the particles. The volume of air contained in each sample would differ. Unless the air is removed and replaced by water (i.e., the soil is saturated) comparison of the intrinsic expansiveness of the soils would be difficult. It was therefore decided to use a vacuum desiccator to obtain air free slurry. Each sample was de-aired for 1 hour under 90 kPa vacuum.

During the de-airing process with the removal of the air the soil would become porous. The pore space should, thus, be filled with water to acquire saturated slurry. At the end of de-airing water was allowed to enter into the pan containing the slurry. To facilitate the replacement of the void with de-aired distilled water the water was added so slowly and in-between the vacuum desiccator was shaken. At the end, the de-aired distilled water was allowed to fill-in to about 10 mm above the surface of the slurry. To additionally facilitate the replacement of the air void by the de-aired distilled

water the pressure of the vacuum was released slowly. During this process some of the water was forced into the voids. After this the slurry was ready for the compression test.

The stage that immediately followed the de-airing was one-dimensional compression by increasing the effective stress of the reconstituted sample in an oedometer. To see if there exist any relationship between saturated swell and unsaturated swell the sample was unloaded. This loading and unloading cycle produces sample discs of consistent stress history, strong to handle, and free of unstable micro-fabric. Details of the overall procedure followed in the preparation of the discs in the oedometer are described in the following paragraphs.

When the samples were in the de-airing stage the oedometer apparatus was setup. For this stage rigid oedometer rings of 80.4 mm internal diameter and 20 mm height were used. The rings were lightly greased before assembly. The lightly greased ring was placed on top of a porous stone (of bigger diameter than the ring) inside the oedometer cell. To avoid vertical and horizontal movements the ring was clamped within the oedometer cell. After placing a filter paper on top of the lower porous stone the assembly was filled with de-aired distilled water to a level of 10 mm above the filter paper.

Once the de-airing process was finished, filling of the oedometer ring with the slurry was commenced. The transfer of the slurry was done by hand. Use of a hand was preferred, as it was believed to minimize entrapment of air because of its speed and the ability to carry a large amount of slurry at a time. The transfer was made quickly and the slurry remained submerged inside the oedometer ring. The other benefit derived from the use of a hand was in its ability to feel whether the ring was properly filled or not. In cases where there were empty spaces inside the ring the slurry was rearranged to completely fill in the spaces, with the least pressure of the hand as possible. The oedometer ring was filled into a level 5 mm from the top edge. Following leveling of the surface of the slurry, a filter paper, a porous stone and loading cap were consecutively placed on top of the sample.

The assembled cell was finally placed in the loading frame and set up for the oedometer compression test. A dial gauge was used for the recording of the volume changes during the loading and unloading processes. The initial reading of the dial gauge was recorded and the sample was left overnight. After 24 hours the dial gauge reading was recorded and then the vertical stress on the sample was increased to 2 kPa. Following that the stress of the sample was increased to 4, 8, 16, 32, 64, 128, 250, 500, and 1000 kPa and then decreased to 500, 200 and 100 kPa with 24 hours interval in-between each stage. In all these cases the reading was taken after the 24 hours of the load application. Following the last change of stress, the sample was left for 72 hours to allow for creep. The dial gauge reading for the last load was taken at the end of the third day. At the end of the third day, the load applied to produce 100 kPa of stress was removed.

Immediately after the removal of the load, the water in the cell was removed by sucking it up with a small plastic pipe to avoid possibility of swelling. Dismantling of the assembly and obtaining the soil sample in the ring then followed. The circular disc shaped sample was then extracted from the ring using hydraulic jack sample extractor. During the extraction extreme care was taken to avoid any distortion of the sample. Then the sample was blotted with tissue paper for excess surface water and immediately put into a sealed plastic bag for the unconfined shrinkage test that followed.

4.5 LABORATORY EXPERIMENTAL METHODS

In the previous section the procedures involved in the preparation of samples for the respective tests were described. Now the procedures followed in the testing of the samples will be discussed.

4.5.1 Grain Size Distribution

The grain-size analysis involved sieve analysis and sedimentation analysis. The wet sieve analysis was done as per clause 9.2 of BS: 1377-part 2 (BSI 1990). The hydrometer analysis was done according to clause 9.5 of BS: 1377-part 2, 1990 (BSI 1990), except the use of minus 75- μm soil

fractions, as recommended by ASTM designation D 422-63 (ASTM 1990), instead of minus 63- μm specified by the standard.

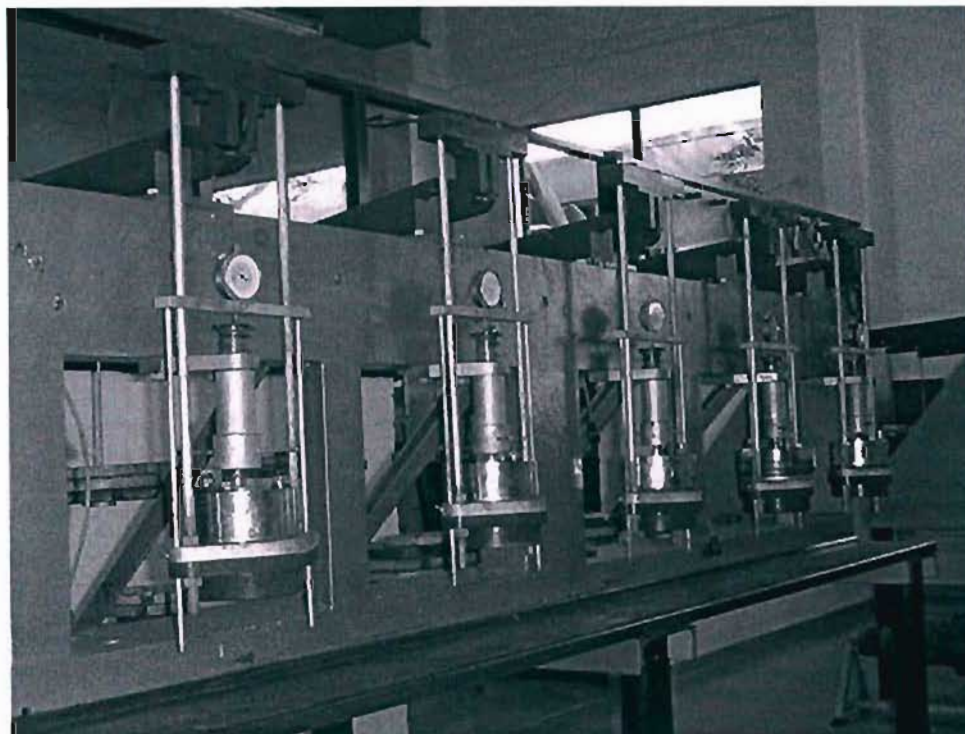


Figure 4.1 Oedometer set-up.

As specified previously to avoid the effect of high temperature on the properties of soil particles, sample preparation for the grain size analysis involved drying to a constant weight in an oven under a temperature of 50 °C. About 150 g was obtained from the dried sample. Distilled water was added till slurry was formed. A dispersing agent was also added to the slurry. After 24 hours of mechanical shaking of the slurry, a 75- μm sieve was used to separate the sample into two. The first part, which consisted of the soil fraction retained on the sieve, was used for sieve analysis. The second part consisting of the soil fraction passing the sieve was used for hydrometer analysis. A jet of distilled water from a wash bottle was used to facilitate the separation. Both samples were oven dried separately at about 50 °C to a constant weight. The weight of the dried samples was recorded to 0.01 g.

The proportion retained on 75- μm sieve went through dry sieve analysis using a nest of sieves of: 9.5 mm, 4.5 mm, 2 mm, 600- μm , 425- μm , 300-

μm , and 150- μm and 75- μm sizes. The weight retained in each of these sieves was recorded to 0.01 g for determining the distribution.

About 50 g of the oven dried sample passing 75- μm was obtained for hydrometer analysis by riffing. The standard hydrometer analysis as per BSI (1990) was then performed on this sample.

4.5.2 Specific Gravity Tests

The specific gravity tests were carried out following the small pycnometer method as specified in clause 8.3 of BS: 1377-part two (BSI 1990). The test was carried out in duplicate for each soil used in the study using a pycnometer of capacity 50 ml.

Air-dried samples weighing 10 g were used for each test. Each sample was then transferred into a cleaned and dried small pycnometer. The weight of the content, the pycnometer and stopper were recorded to 0.001 g. De-aired distilled water was added to a level few millimeters above the surface of the soil inside the pycnometer. The pycnometer and its contents were then placed in a vacuum desiccator where the air bubbles in the sample were completely removed by applying a vacuum of 90 kPa. When there was no air bubble movement the vacuum was released and the soil in the pycnometer was stirred. Following that the pycnometer was again placed in the vacuum desiccator and the same vacuum was applied. The same process was repeated whenever it was felt that the soil still contained air. Having ensured that the soil contains no air, the pycnometer was taken out of the desiccator and was filled with de-aired distilled water to the graduation showing 50 ml volume. Then, the weight of the pycnometer and its constituents (soil and de-aired distilled water) were recorded to 0.001 g. After weighing the pycnometer was emptied and washed. Again the pycnometer was filled with de-aired distilled water to the graduation showing volume of 50 ml and the weight was recorded to the nearest 0.001 g. Finally, the empty pycnometer was dried in an oven for few a minutes and its weight was recorded.

4.5.3 Liquid Limit Tests

The liquid limit of each soil sample was determined in duplicate by the cone penetrometer method as specified by BS: 1377-Part 2, 1990, clause 4.3, cone weight of 80 g and apex angle of 30° (BSI 1990).

The liquid limit of a soil using this method is the moisture content at 20 mm penetration which is achieved by allowing free falling of the cone for about 5 seconds. To get a clear picture of the characteristics of the soil it was decided to check the moisture content when the penetration was ± 5 mm from the liquid limit penetration. Assuming that three points would properly represent moisture content versus penetration relationship, the following penetration ranges were used as a guide: 15-18 mm, 18-21 mm, and 21-25 mm where a sample for moisture content was taken to represent each of the range. Three penetration check-ups were taken for the assurance of homogeneity of the paste before sampling was accomplished.

4.5.4 Plastic Limit Tests

Using soil paste left after the liquid limit test, the plastic limit was determined in duplicate for each soil sample as outlined in clause 5.3 of BS: 1377-Part 2, 1990 (BSI 1990).



Figure 4.2 Plastic and liquid limit tests.

4.5.5 Unconfined Shrinkage Limit Tests

The shrinkage limit tests are carried out based on the Schreiner and Gourley's procedure. The procedure differs from shrinkage limit test of BS: 1377-Part 2, clause 6.3, (BSI 1990) in the method of sample preparation and measurement of volume. As pointed out previously the samples are prepared by a process known as reconstituting from which saturated samples but regular shaped and hard enough to easily work on are obtained. Controlled moisture removal is done on the sample until a constant volume is attained. Indirect volume measurement is taken during drying at intervals. The details of the procedure are outlined in the following four subsections.

4.5.5.1 Volume and mass measurement

Section 4.4.2.4 has clearly stated the speed of the process of removal of the oedometer ring from the oedometer and the extraction of the disc shaped sample from the oedometer ring. Immediately after the extraction the sample was taken into a temperature-controlled laboratory where the rest of the unconfined shrinkage limit test was carried out.

At this stage usually the samples tend to be sticky. For safe sample handling and measurements, and to avoid loss of detached particles, a very thin but strong circular plate of known weight and thickness was used to rest the sample on. The plate was used throughout the shrinkage limit test. The true weight and thickness of the specimen was obtained by excluding the weight and thickness of the plate, wherever it was included in the measurements.

Prior to undertaking any measurement the thickness of each disc was checked throughout its circumference while resting on a plate. It is true that the disc might be slightly wedge shaped as there is no way of keeping the oedometer cap horizontal during the process of loading and unloading. From the measurements it was then possible to locate points on the circumference showing the thinnest and the thickest points of the disc. A circular filter paper of the of the same diameter as that of the samples was

prepared before the start of the tests in such way that two diameters perpendicular to each other were drawn on it. This filter paper was used as a guide to accurately make a scratch in the circumference of the disc firstly in the thinnest and thickest points and secondly at the end points of the second line perpendicular to the first line. The scratches were then used to lightly draw the two diameters on the disc using a pointed instrument. Numbers were given to the lines in order to get measurements of the same line at the different stages of shrinkage.

Beginning from the time the sample was removed from the oedometer, the confinement was released and the sample was free to change volume (three dimensionally) and mass with loss of moisture to the atmosphere. To avoid such unnecessary changes all the above-mentioned processes were done as quickly as possible.

Immediately after drawing of the two diameters on the disc the weight of the disc was recorded to a precision of 0.0001 g. Following that the thickness of the disc was measured to a precision of 0.0001 mm and the diameter with a precision of 0.01 mm. These measurements were all done as quickly as possible. When the measurement was finished the sample was placed in a sealed plastic container where the controlled removal of moisture was accomplished.

4.5.5.2 Moisture removal

Immediately after the finishing of the initial measurements the disc, resting on the plate, was placed inside a sealed plastic bag to avoid uncontrolled water loss that would consequently lead to decrease of mass and volume. The drying process could have been facilitated by air-drying the sample. However, this was not preferred before the sample passed the shrinkage limit for various reasons including the formation of cracks on the sample as the foremost. If cracks are formed volume estimation using vernier calliper would be impossible. Besides that the other important parameter that this study is expected to produce was swell index of the samples. One-dimensional swell test should be accomplished on samples without any cracks; otherwise the swelling will first close the void developed by the

cracks before it starts showing vertical increment of thickness. This underestimates the swell index of the sample. The preferred approach chosen was then to slowly remove the moisture content using a desiccating agent while the sample is inside the sealed plastic bag. Once it passed the shrinkage limit the sample was allowed to air dry in a temperature-controlled laboratory till a constant weight is achieved.

Silica gel was used as the desiccating agent in this study, which is blue in colour when dry and pink when wet. The moisture removal efficiency of dry silica gel in time is affected by different factors out of which type of soil and moisture content of the soil can be mentioned as good examples. Clayey soils with significant amount of sand particles will have large sized voids that allow quicker release of moisture than highly plastic soil with very small voids.



Figure 4.3 Equilibration of filter paper for suction measurement.

The moisture removal can be achieved by placing a known quantity of dry silica gel for a known time. To avoid the possibility of distortion and cracking of the sample the removal of moisture was planned to be not greater than 3.5 g per 2 days. This was possible by using silica gel of about 10-15 % of the weight of the specimen. After two days of allowing the silica to absorb the moisture liberated from the sample, the sample and the silica gel would be removed from the bag. After recording the weight, the thickness, and the diameters the sample was replaced inside the sealed bag with new silica gel.

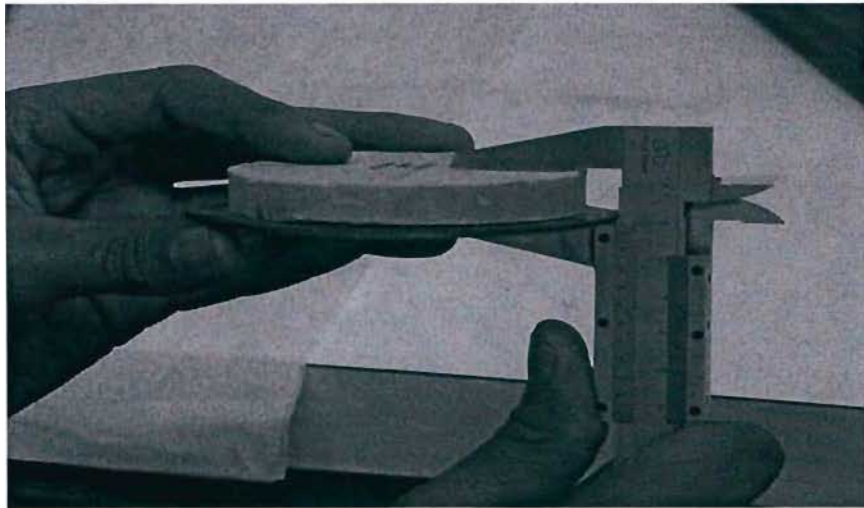


Figure 4.4 Measuring thickness of shrinking soil disc using vernier calliper.

Such sequence of drying and measuring continued until the shrinkage limit was passed. To check if the shrinkage limit is reached mass versus volume graph was plotted. When the sample is wet a unit change in weight yields a significant change in volume. At this stage the relationship yields a very steep sloped line. When the sample is about the shrinkage limit the change in volume completely ceases. The relationship yields a horizontal line at this stage.

After the sample passed the shrinkage limit the time required to remove the required amount of moisture tend to increase. To facilitate this process the study found that air-drying of the sample that passed the shrinkage limit in a temperature-controlled laboratory is satisfactory. Samples were exposed and the change in weight was regularly monitored. Recording of the dimensions was done roughly every 2 g decline of moisture content till a constant weight was reached. Once the sample reached a constant weight preparation for the swelling test commenced.

Before discussing the swelling test there is one important test that was run on the sample during the process of unconfined shrinkage test that needs a brief description. The test is to measure total soil suction. The procedures followed for this test are described in the following subsection.

4.5.6 Total suction tests

4.5.6.1 Procedure followed

The total suction measurement was done using the non-contact filter-paper method according to Chandler and Gutierrez (1986), which was improved first by Schreiner (1988) and later Schreiner and Gourley (1993).

4.5.6.2 Starting the test

During the drying procedure one of the observations was the presence of condensation. The phenomenon was happening in some of the soils in the initial stages of drying. In the highly sandy soils it was extended several days after the start of drying. This was a problem for the suction measurement, as the test does not work in the presence of condensation conditions.

The non-contact filter means of suction measurement considers reaching suction equilibrium of soil sample and filter paper, kept in a confined environment together, through vapor flow. Condensation disrupts the process of reaching equilibrium. There was no better solution for this problem besides further drying before starting of the suction test.

4.5.6.3 Reaching suction equilibrium

The suction test was started after condensation of the samples was stopped. Prior to each testing, a dry waterproof plastic bag was prepared by placing some silica gel inside for overnight. Two Whatman No.42 filter papers (which are 90mm in diameter) were oven-dried for more than two hours. Three perforated plates, cut to size of the filter paper, were also prepared for every sample to aid as spacers for avoiding contact between the sample and the filter paper but allow diffusion of vapor. Following the recording of dimensions and weight of the specimen, one filter paper was taken out from the oven and sandwiched between two perforated plastic spacers and immediately placed between the bottom of the specimen and the plate. On top of the sample the third plastic spacer was placed which was followed by the second filter paper. To avoid contact between the top

filter paper and the plastic bag, a 25 mm long PVC pipe of 90 mm diameter was used. After this assembly was done the plastic bag remained sealed for seven days. By the end of the seventh day the suction of the soil and the filter paper is assumed to have reached equilibrium. Therefore, after the seventh day the amount of weight gained by the filter papers, the change in mass and the dimensions of the sample were recorded.

4.5.6.4 Weighing of filter paper

The weights of filter papers were recorded by using pre-weighed airtight weighing bags in a balance of precision 0.0001 g. The true weight of the filter papers could be obtained by excluding the weight of the weighing bag. After recording the weight of the weighing plastic bags the filter paper was quickly transferred to the weighing bag by lightly holding using tips of two fingers. The weight of the weighing bag with its filter paper was recorded. The filter paper was removed from the weighing bag and dried to constant weight for about two hours at a temperature of 105 °C. The weighing process employed for measuring the dry weight of the filter paper was the same as before. This test requires speed in order to avoid loss or gain of moisture during the transfer of the filter paper to the weighing bag.

4.5.6.5 Number of suction measurements

Suction measurement was repeated after removal of moisture using silica gel. At least three suction measurements were taken before the sample reached the shrinkage limit. One additional suction test was performed after the sample had passed the shrinkage limit.

4.5.7 Swelling Tests

Following the unconfined shrinkage limit and suction tests, each of the samples went through swelling tests using the conventional oedometer to obtaining the swelling index of the samples. Due to the removal of the moisture content the diameter of the disc often decreases. The disc had to fit into an oedometer ring for one-dimensional swelling test to be accomplished. For the purpose of consistency all samples were reduced to a size of 70 mm for the oedometer test. By the time they reached the

shrinkage limit the samples were hard and brittle. A modified trimming procedure was followed to get a disc of consistent thickness throughout its circumference that exactly fit the 70 mm oedometer ring. The swelling test was accomplished on samples prepared in such a way.

4.5.7.1 Sample trimming

Sample trimming started immediately after the last drying process for shrinkage limit and suction tests. The trimming process used simple instruments as shown in Figure 4.5.

The sample was sandwiched between two plastic rubbers. The plastic rubbers were used to avoid the risk of concentration of loads in contact points and absorb movements. Oedometer top caps were used outside the rubbers. A pre-dried and weighed oedometer ring was tied up to hang inside the clamp with its sharp edge to sample assembly. The sample assembly was then tightened in the G-clamp with great care to make the sample, the rubbers and the oedometer top caps co-axial (Figure 4.6).

The step followed after the above-mentioned preparation was trimming of the part of the sample that extended outside the top caps. The rough trimming was accomplished by a metal cutting hacksaw of very fine teeth (of 32 teeth per inch). A metal file of fine tread was used to finally smooth the surface and obtain an exact fit in the oedometer ring. The checking up of the fit of the sample to the oedometer ring was done regularly at this stage. The ring was allowed to approach the sample by passing over the top cap. To exactly locate the places where the



Figure 4.5 Trimming instruments.



Figure 4.6 Sample trimming set-up.

diameter was greater than the ring, the ring was rotated with as a light pressure as possible. The places that showed a sign of touching were filed further till the entire sample entered the ring. When the sample reached the other end of the ring the assembly was dismantled.

So far the sample was made to fit in the 70 mm ring. Most probably the sample may have a wedge shape. For accurate measurement of the swelling the disc should be further trimmed to achieve a constant thickness. This was achieved by rubbing the surfaces of the disc against fine sand paper.

The swelling should be accomplished inside the odometer ring. Therefore, the swelling behavior of the sample should be considered in order to decide to what thickness to leave the disc. For the rough swell estimation Gourley and Schreiner's (1993a) correlation was consulted. If the addition of the swelling expected with the thickness of the disc exceeds the thickness of the odometer thickness further reduction was done using sand paper as specified earlier. Using this way discs that exactly fit-in the 70 mm oedometer ring and swell completely inside the ring were obtained.

4.5.7.2 Oedometer setting up

After the sample preparation using the trimming techniques the discs are ready for one-dimensional swelling test in an oedometer. For the test, a dry oedometer cell and oven-dried porous stones of known weight and thickness were prepared. The sample assembly was as follows. In side the oedometer cell first a porous stone was placed, then the oedometer ring with its sharp edge upward and the sample disc exactly touching the porous stone. On top of the sample another porous stone and top cap were placed. The assembled oedometer cell was finally placed in the loading frame under stress of 15 kpa. This applied stress represents the pressure of an average clayey soil at a depth of about 1m. Dial gauge of precision of 0.001 mm was used in the oedometer set up to monitor the vertical movement.

The initial reading of the dial gauge was recorded immediately following completion of the oedometer setting up. To avoid change of moisture content of the sample in the initial 24 hours of loading, the oedometer cell

assembly was sealed with cellophane. At the end of 24 hours the reading was taken and the cellophane was removed. After this reading the wetting up of the sample was started.

4.5.7.3 Sample wetting up

During the process of drying the pore spaces occupied by water will gradually be filled with air. The process of saturation can be facilitated if the air is allowed to diffuse out. So, to gain from this idea, during the first two days wetting up was maintained to a level little below the sharp edge of the oedometer ring. Right after that the soil was completely submerged. The wetting up was accomplished using de-aired distilled water. Dial gauge readings were taken every 24 hours until vertical movement had nearly ceased.

4.5.7.4 Sample drying

The wetting up process is assumed to produce samples that are completely saturated. Moisture content determinations of samples under such condition were used to interpolate the moisture content of the samples during the drying stages of the unconfined shrinkage limit test. The moisture content obtained would be useful to determine the void ratio.

At the end of the swelling test the oedometer assembly was dismantled and the oedometer ring with the disc and the lower porous stone passed through a series of steps to determine the moisture content.

Immediately after the removal of the sample from the oedometer the water was siphoned out and some of the water in the surface of the ring were cleaned up with tissue paper. After this the weight of the oedometer ring with its disc and saturated lower disc was recorded. To determine the saturated weight of the porous stone, the stone was saturated and weighed following oven drying. The thickness of the saturated disc was recorded. Following these measurements the porous stone and the oedometer ring with the disc was left overnight in an oven after which the dry weight was recorded.

4.6 ANALYSIS OF DATA

Regression analysis is a useful statistical technique for both identifying related variables and for modelling and predicting the relationship between the same variables. The technique compares individual variables with one another and calculates estimates of the strength, or magnitude, of the statistical relationship.

Three measures of intrinsic expansiveness are recommended by Gourley and Schreiner (1993a) or by Schreiner (1999). These are change in void ratio, vertical strain on soaking, and the expansive strain on soaking. These measures will be tested their relationship against sixteen indirect indicators: W_L , W_p , I_p , Clay Fraction (C.F.), Activity, W_s , e_{PI} , e_{WL} , e_{WP} , e_{SL} , e_{15} , C_s^* , C_c^* , I_s , $W_p - W_s$, and u_{SL} . In this study multivariate regression analysis will be used to isolate the most significant indirect indicator from all of the expansiveness measures.

CHAPTER FIVE

5. RESULTS PRESENTATION

5.1 BACKGROUND

As it has been mentioned in the previous chapter that a total of twenty-two types of clayey soil samples are used in this research. For the purpose of assessing the reliability of the testing procedure two of the samples were tested in duplicate. The average of the duplicate test results is reported for these samples unless otherwise intended to show the variations. Additionally, the eight samples from the Gourley and Schreiner's (1993a) research is used in the analysis toward obtaining a universal means of assessing expansiveness.

To separate the samples codes were given based on their source of country. The codes start by ER, KE, SA, and SU to represent Eritrea, Kenya, South Africa and Sudan respectively. The visual descriptions of the samples, which were made in the field from the soil profile or on soil samples brought to the laboratory, with their place of origins, is tabulated in Table 5.1.

The series of tests carried out on the samples are: grain size distribution, specific gravity, liquid limit, plastic limit, shrinkage limit, soil suction, and swell. The results obtained from these tests and their analysis is presented in the following sections.

5.2 TEST RESULTS

5.2.1 Grain-Size Distribution

The processed data of the results obtained from the sieve and hydrometer analysis made on the samples collected from Eritrea and South Africa to determine their grain size distributions are summarized in Table 5.2.

Of the twenty-two samples collected for this research, one, which was the replica of sample SA-1, was broken by accident during the shrinkage test. The whole data collected for this replica before it was broken is therefore void.

Table 5.1 Set of soil samples used from own and Schreiner and Gourley's research.

Source of sample	Code	Visual description
University of KwaZulu-Natal, Durban, South Africa	SA-1	Dark reddish orange, clay fine-medium SAND, low organic matter, in-situ weathering of Aeolian deposit.
Ridgeview, Durban, South Africa	SA-2	Dark yellow, blocky with shiny surfaces, slightly silty CLAY with occasional fine to coarse sand, low organic matter, DWAYKA TILLITE.
Hill Crest, Durban, South Africa	SA-3	Dusky red, fine to medium sandy CLAY, medium organic content, LATERITE.
Botha's Hill, Durban, South Africa	SA-4	Dark reddish yellow, micro-shattered, slightly fine to medium sandy CLAY, low organic matter, amphibolite hornblende SCHIST.
Mount Edgecombe, Durban, South Africa	SA-5	Black, micro-shattered, CLAY, high organic matter, Pietermaritzburg SHALE.
Mount Edgecombe, Durban, South Africa	SA-6	Dark black, shattered, slightly fine sandy CLAY, medium organic matter, BASALT.
Sea Cow Lake, Durban, South Africa	SA-7	Olive, gravelly CLAY, moderate organic matter, Colluvium.
Sea Cow Lake, Durban, South Africa	SA-8	Black, CLAY, medium organic matter, Pietermaritzburg SHALE.
Howick, Pietermaritzburg, South Africa	SA-9	Dark grey, blocky, fine to medium sandy CLAY, low organic matter, Colluvium.
Westmead, South Africa [†]	SA-10	Light reddish brown to cream, silty CLAY, low organic content.
Stanger, South Africa [†]	SA-11	Dark olive, slightly sandy CLAY, low organic matter.
Kwa Mashu, Durban, South Africa	SA-12	Dark yellow, blocky, slightly sandy CLAY, low organic matter, Pietermaritzburg SHALE.
Kilnerpoort, South Africa [†]	SA-13	Dark brown, silty CLAY, low organic matter.
Rondebosch, South Africa [†]	SA-14	Yellow to orange, Silty CLAY, moderate organic matter.
Kilbarchan, South Africa [†]	SA-15	Dark yellow, Sandy CLAY, low organic matter.
Botha's Hill, Durban, South Africa	SA-16	Dark brown, slightly clayey SAND, low organic matter, GRANITE.
Mount Edgecombe, Durban, South Africa	SA-17	Dark reddish orange, Clay SAND, low organic matter, DOLERITE intrusive.
Adi-Guædad, Hamasien, Eritrea	ER-1	Dark grey, CLAY, micro-shattering structure, low organic matter, residual BASALT derived Black Cotton soil.
Mai-Harish, Seraye, Eritrea	ER-2	Light gray, sandy silty CLAY, low organic matter, BASALT.
Korbarya, Akeleguzay, Eritrea	ER-3	Light brown, low organic matter, GRANODIORITE.
Guræ, Akeleguzay, Eritrea	ER-4	Light brown, highly sandy CLAY, low organic matter.
Asmera, Hamasien, Eritrea	ER-5	Light red, slightly sandy CLAY, low organic matter, LATERITE.
Nairobi-Mombassa road, Athi plains, Kenya	KE-1*	Very dark grey to black, slightly silty CLAY with occasional fine to coarse gravel, high organic content.
Aock-Muga School, South Nyanza, Kenya	KE-2*	Grey to black CLAY with occasional sand and gravel, high organic content.
Road C63, Nairobi, Kenya	KE-3*	Yellow to red, mottled yellow and white, stained dark red on joints, silty CLAY, low organic content, VOLCANIC ASH.
Road C68, Nairobi, Kenya	KE-4*	Dark red, CLAY, low organic content.
Road A104, Gilgil, Kenya	KE-5*	Yellow brown, clayey SILT, moderate organic content.
Road A104, Kakura-Nairobi road, Kenya	KE-6*	Yellow to brown, stained black on joints, silty CLAY, low organic content.
Gezira University, Wad Medani, Sudan	SU-1*	Dark brownish grey, gravelly silty CLAY with calcareous granules, moderate organic content.
El Faw Irrigation Scheme, Sudan	SU-2*	Grey brown, gravelly silty CLAY, moderate organic content.

* data set from Schreiner and Gourley's research.

† samples obtained from CSIR.

As planned, diversified samples were obtained for this research in terms of grain size distribution. The sample set included samples with sand percentage ranging between 7 and 81%; silt percentage ranging between 2 and 41%; and clay fraction (C.F.) ranging between 17 and 78%.

Table 5.2 includes the percentage of soil particles passing 425- μ . The standard liquid limit and plastic tests are performed on soil samples passing 425- μ . This percentage is used in the interpolation of the test results to represent to that of the whole sample using the formula given in section 4.2.2.3.

Table 5. 2 Particle size distribution, Atterberg limits and particle density of the sample set.

Sample	Sand [%]	Silt [%]	C.F. [%]	Pass 425 μ [%]	W _L	W _p	Specific Gravity (kg/m ³)
					[% whole sample]		
SA-1	56	2	42	98	39	16	2.728
SA-2	27	15	59	97	47	14	2.698
SA-3	21	25	54	93	45	26	2.780
SA-4	17	7	77	95	61	27	2.792
SA-5	12	21	67	98	44	25	2.604
SA-6	15	22	63	99	53	27	2.717
SA-7	7	22	71	99	62	20	2.769
SA-8	13	34	53	98	42	22	2.683
SA-9	20	15	65	96	61	33	2.604
SA-10	26	27	47	99	65	26	2.868
SA-11	28	13	59	92	54	21	2.788
SA-12	53	11	36	87	30	16	2.828
SA-13	55	8	37	93	50	13	2.670
SA-14	10	41	49	94	64	38	2.887
SA-15	50	24	26	94	22	17	2.798
SA-16	68	6	26	80	25	12	2.629
SA-17	81	2	17	98	25	9	2.749
ER-1	11	15	74	93	82	27	2.754
ER-2	31	25	44	95	45	13	2.756
ER-3	34	19	47	98	49	21	2.776
ER-4	50	11	39	84	43	16	2.719
ER-5	16	13	73	94	54	27	2.784
KE-1	-	-	-	94	118	48	2.60
KE-2	-	-	-	92	64	25	2.61
KE-3	-	-	-	100	59	27	2.78
KE-4	-	-	-	100	69	36	2.79
KE-5	-	-	-	83	68	37	2.68
KE-6	-	-	-	83	84	42	2.77
SU-1	-	-	-	93	39	22	2.44
SU-2	-	-	-	98	67	39	2.66

5.2.2 Specific Gravity

The processed specific gravity test results for the soil samples from this research and from Schreiner and Gourley's research are also presented in Table 5.2. The specific gravity (G_s) values fall within the range of 2.44 to 2.89.

5.2.2 Liquid and Plastic Limits

The processed data from the liquid and plastic limit tests, which is interpolated to represent to that of the whole sample, is reported in Table 5.2. The interpolated results given are for the whole soil set. The liquid limit of the whole sample set ranges between 22 and 118 % and the plastic limit between 9 and 48 % and the plasticity index between 5 and 70 %.

The sample set fairly included most natural clayey soils in terms of liquid and plastic limits. It includes both expansive and non-expansive soils according to the classification chart of Vijayvergiya & Ghazzaly (1973), which is based on the Casagrande plasticity chart (see Figure 5.1). Moreover, samples considered to have very high, high, medium, and low intrinsic expansiveness according to Van Der Merwe (1964), Williams and Donaldson (1980), and BRE (1980) charts are included in the set (see Figures 5.2 - 5.4).

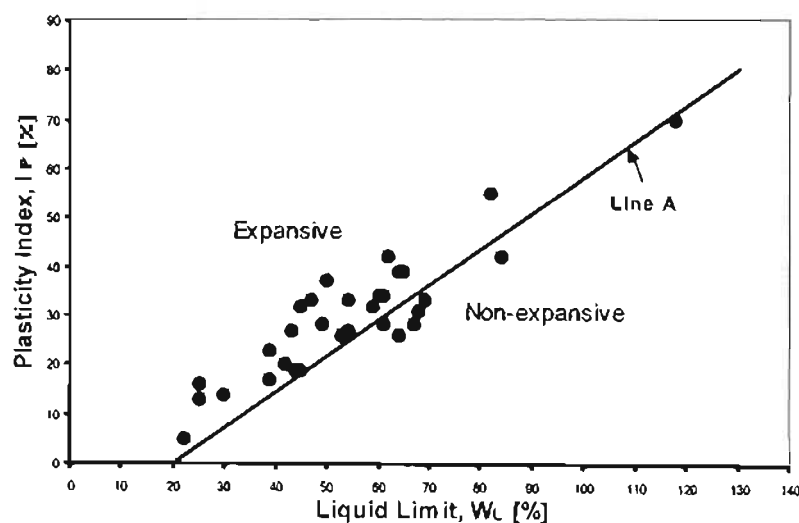


Figure 5.1 Distribution of all the samples in the Vijayvergiya & Ghazzaly (1973) chart.

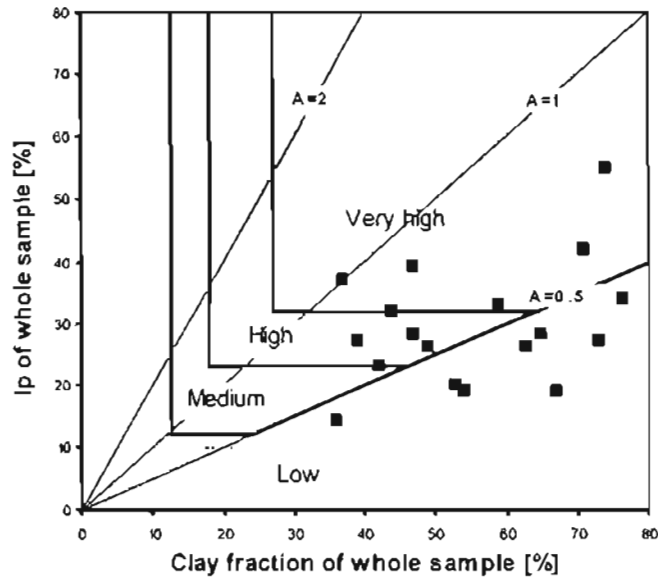


Figure 5.2 Distribution of this research samples in the Van Der Merwe (1964) chart of intrinsic expansiveness.

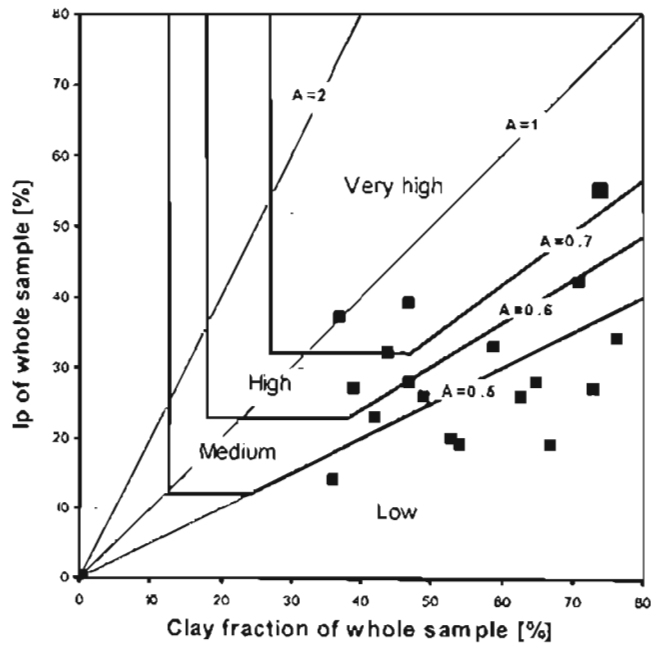


Figure 5.3 Distribution of this research samples in Williams and Donaldson (1980) expansiveness classification chart.

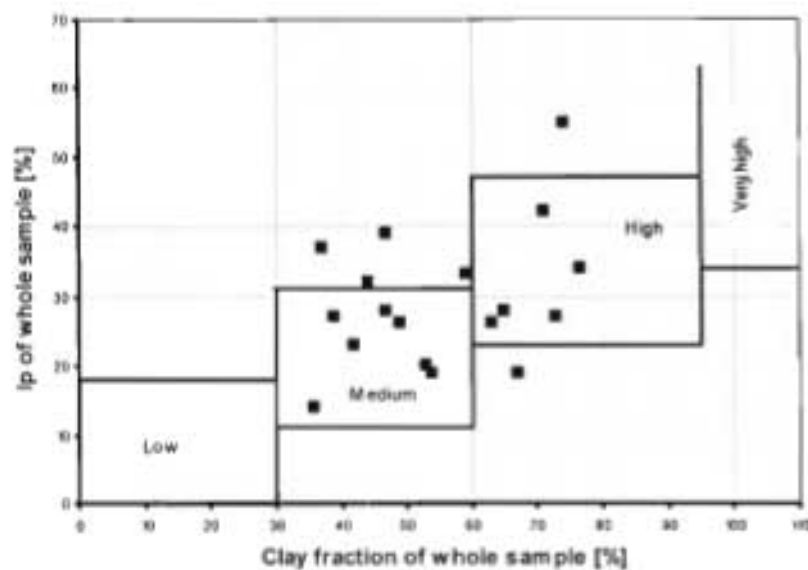


Figure 5.4 Distribution of this research samples in BRE (1980) classification chart.

5.2.3 One-dimensional Compression and Unloading

The consolidation readings and processed data to obtain the void ratio at each vertical stress are presented in the appendix (see Table A.1). The process of calculating the void ratios involved a number of steps. Firstly, the dial gauge readings were factored to obtain change in height. The change of height with the height of the solids (calculated using dry weight the samples, obtained from end of the shrinkage test), specific gravity of the soils and area of the oedometer ring (which is 80.04 mm in diameter) were used to determine void ratio changes. The first thickness measurement of the samples, immediately after removal from the oedometer for unconfined shrinkage test, and their corresponding void ratios, as obtained by back calculation after the shrinkage test was elapsed, were used to represent the thickness and void ratio of the sample at stress of 100 kPa. These readings as a start and the change of height and change of void ratios were used to calculate backwards the height and void ratio of the samples at the different stages of the stresses. Graphical representation of the effective stress versus void ratio of the samples used is given in Figure 5.5. Since the stress history of the samples was destroyed before loading, most of the loading and unloading curves are expected to be linear, though slight curves are observed in some of the samples.

From the loading and unloading curves of the compression plots it is possible to study both the intrinsic compressibility and expansiveness of the soils. A number of research reports including that of Burland (1990) and Lutenegger and Cerato (2003) specified a significant relationship between intrinsic compressibility of a reconstituted soil and the slope of the loading curve. On the other hand, the report given by Gourley and Schreiner (1993a) affirm a significant relationship between intrinsic expansiveness and slope of the unloading curve.

The intent of this research is to report the intrinsic expansiveness, but the data generated could also be potentially useful for assessing intrinsic compressibility of the soils. For clarification, the loading curve is named as intrinsic compression line (ICL), after Burland (1990), and the unloading curve as intrinsic swelling line (ISL).

A number of variables can be extracted from the compression curves that require evaluating the significance of their relationship with the measured swell. One variable is the slope of the ICL between effective stresses of 100 and 1000 kPa, which was defined by Burland (1990) as intrinsic compression index, C_c^* , that can be obtained from:

$$C_c^* = e_{100}^* - e_{1000}^*$$

where e_{100}^* and e_{1000}^* are void ratios respectively corresponding to 100 and 1000 kPa effective stresses. In fact, Cerato and Lutenegger (2004) challenged the uniqueness of the intrinsic compression index, claiming its variability depended on sample preparation. Using few soil samples, they discovered a significant change in the position of the ICL with variation in the initial moisture content of a sample.

For the samples prepared at $1.25 \times w_L$, Gourley and Schreiner (1993a) reported a significant relationship of C_c^* with measured swell. The same research, however, obtained a more significant relationship with the intrinsic swell index, C_s^* , which is defined as:

$$C_s^* = e_{100}^s - e_{1000}^*$$

where e_{100}^s represents the void ratio at 100 kPa effective stress in the ISL.

Since both variables represent the volume change behaviours of a soil, their magnitude is recorded (see Table 5.3) from the compression curves to check if they have a relationship with expansiveness.

5.2.4 Unconfined Shrinkage

In the controlled moisture removal stage of the shrinkage test, weights and dimensions of the samples were recorded. These data were processed to calculate the moisture content and void ratio of the samples at each interval of the measurement (see Table A.2 in the appendix). At the end of each shrinkage test the oven-dried weight of the sample was taken, from which the moisture content is obtained. This final moisture content was used to back calculate the moisture contents of the samples at the different stages of the shrinkage. From the calculated moisture contents and the diameter and height measurements of the samples the volume and bulk density of the soils are then calculated. Furthermore, the bulk densities obtained are used with the specific gravity and moisture contents to calculate the void ratios.

For every stage of the shrinkage process of a sample, where measurement was made, the processing of the data yielded its moisture content and corresponding void ratio. Graphical representation of the relationship between these two variables is presented for every sample in Figure 5.5.

The shrinkage limit of a soil is obtained using graphical projection of its shrinkage curve. There are a number of standards and procedures used in the interpolation of the shrinkage limit from the shrinkage curve. Most of the standards have similarity but there is a significant difference between the ASTM and HMSO (1952) (quoted by Gourley and Schreiner 1993a). The former standard starts the test at a moisture content above the liquid limit that approximately confirms the saturation of the sample at the start. Thus, the 100% saturation line is drawn on the graph using the relationship $e = wG_s$. The shrinkage limit is then obtained from the intersection of the saturation line with the horizontal line through the e_{SL} . This intersection point is represented by B (Figure 5.5) and the moisture content at the intersection is considered to be the shrinkage limit and is named as w_{SA} .

The HMSO method of testing and interpretation of the shrinkage limit is different from that of the ASTM. The test is performed at moisture content much lower than the liquid limit for the purpose of getting harder samples which are easy to handle. Due to this fact, the test does not ensure saturation at the start. The shrinkage limit from this procedure is obtained by projecting the linear part of the curve, from the start of the test, until it intercepts the horizontal line representing the e_{SL} . For the sake of distinguishing from the previous point, the intersection is noted as A (Figure 5.5) and the moisture content of this intersection as w_{SB} .

In such a way interpolated e_{SL} , w_{SA} , w_{SB} values are presented in the Table 5.3. The table also contains the air void of the soils at their liquid limit, e_{WL} and plastic limit, e_{WP} . Determining the air void at liquid limit involved the use of the relationship $e_{WL}=w_L G_s$, as samples are most probably at saturated state at and above the liquid limit. However, the air void at plasticity limit involved interpolation from the shrinkage curve.

Out of the twenty-one samples used in the test, sample SA-16 and SA-17 showed a different pattern. These samples did not show a pattern of a straight line at the start of the test (see Figure 5.6). Thus, it was not possible to determine the shrinkage limit according to the ASTM and HMSO methods. Therefore, these two samples are neglected and are not included in the further analysis.

5.2.5 Soil Suction

The process of shrinkage was interrupted to obtain suction of the soil. During the interruption mainly two sets of data were collected. The first being the weight and dimension of the sample for determining the moisture content and the void ratio of the soil. The second was the increase in moisture content of the filter paper from oven-dried state due the equilibration process with the soil sample for obtaining the suction of the soil. The moisture content and void ratio calculations were done the same way as for the unconfined shrinkage test. The suction test was done in a temperature controlled laboratory, where on average the laboratory was kept at about 21 °C. The test involved collecting data from filter paper

placed below and above the sample in order to represent the possible moisture content variation. It is assumed that equilibration of matric suction would be achieved between the soil sample and the filter papers after keeping them in a sealed environment without contact for seven days. In most cases it was not possible to run the suction test during the saturated stage of the samples due to condensation problems that can easily lead to erroneous suction results. The top and bottom filter paper moisture content readings were averaged to calculate the total suction using the formula recommended by Schreiner and Gourley 1993 (see section 2.4.4). The recorded and processed data during the suction test, both the suction and the sample measurements, are presented in Table A.3 in the appendix.

The suction data obtained for each sample is plotted in void ratio versus suction graph to get the suction at the shrinkage limit (see the example in Figure 5.6). It is clear from the graphs that the suction curve tends to asymptote out toward the shrinkage limit. Interpolation is used to obtain the suction at the shrinkage limit, u_{SL} . The Schreiner and Gourley's approach used a straight line that fits the data best. The suction that corresponds the e_{SL} in the equation that represents the straight line is u_{SL} . Considering the nature of suction curves for most clayey soils where they asymptote out around the oven dried state, it may seem better to take the best fit straight line for the last readings. Accordingly obtained interpolations of u_{SL} for each soil sample are shown in Table 5.3.

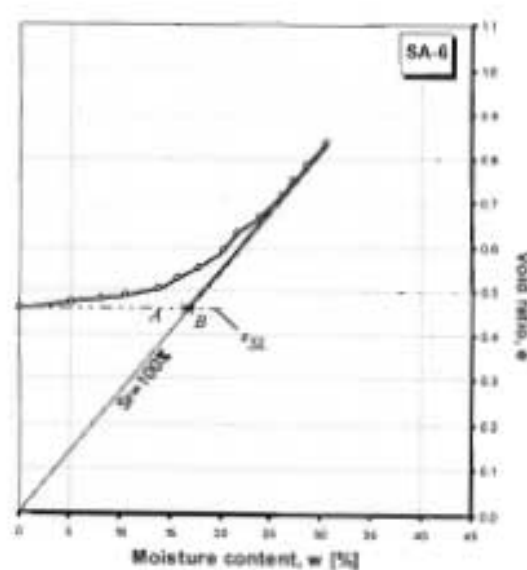
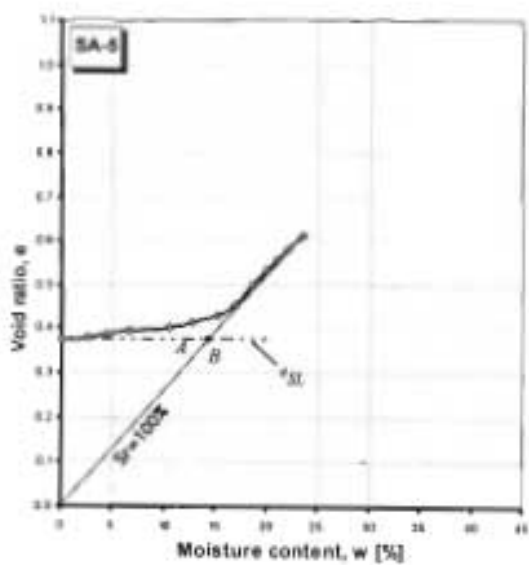
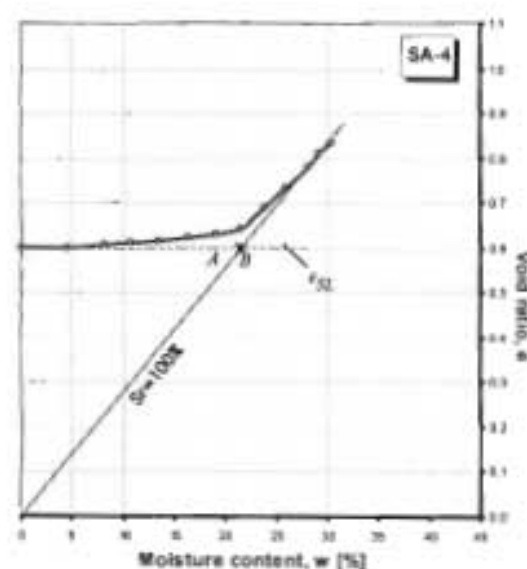
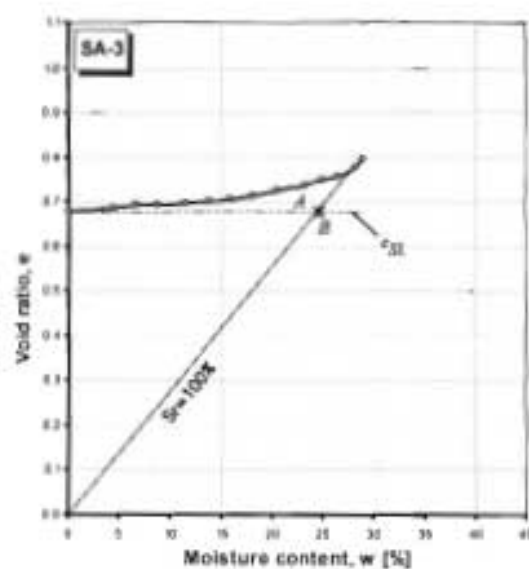
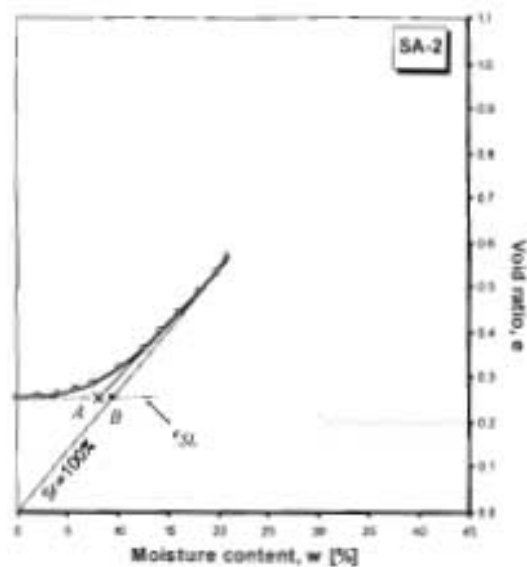
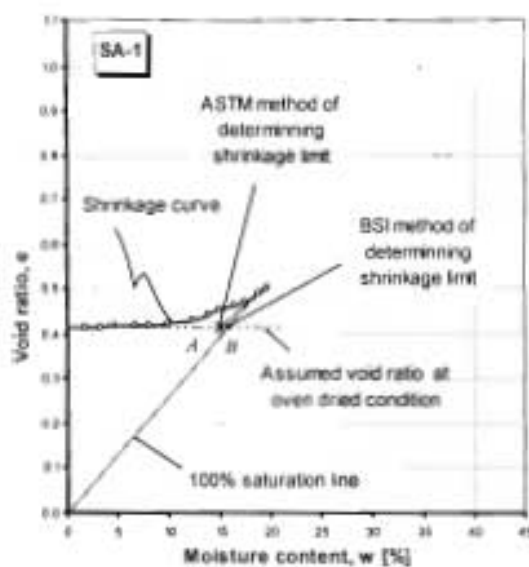
5.2.6 Swell Test

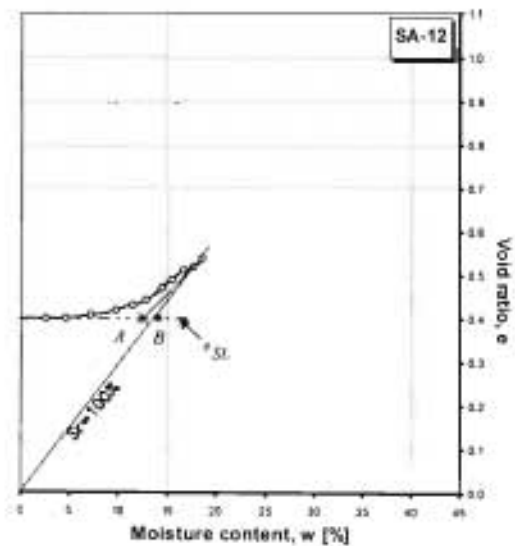
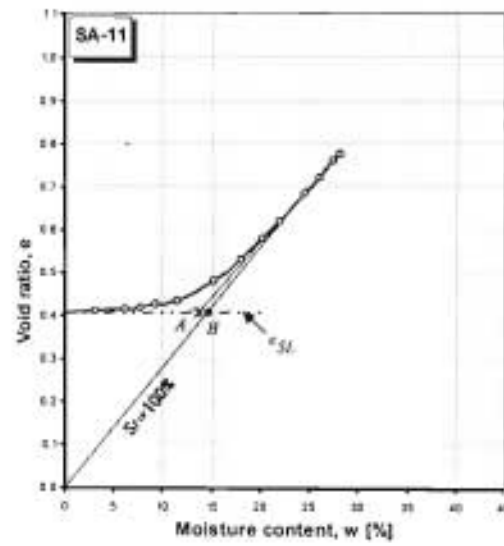
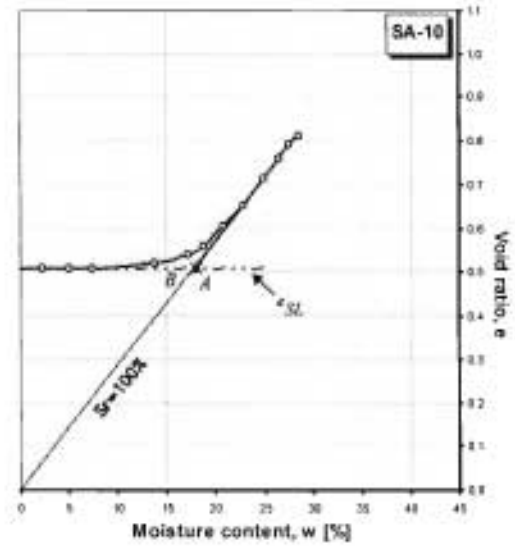
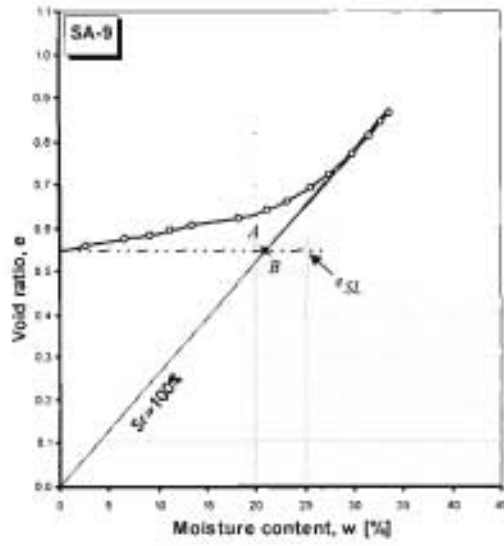
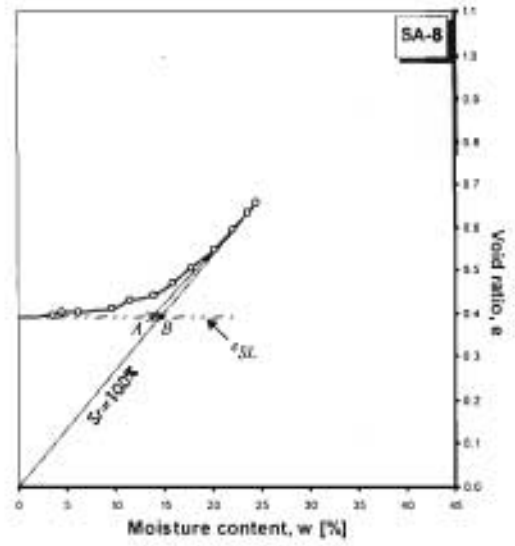
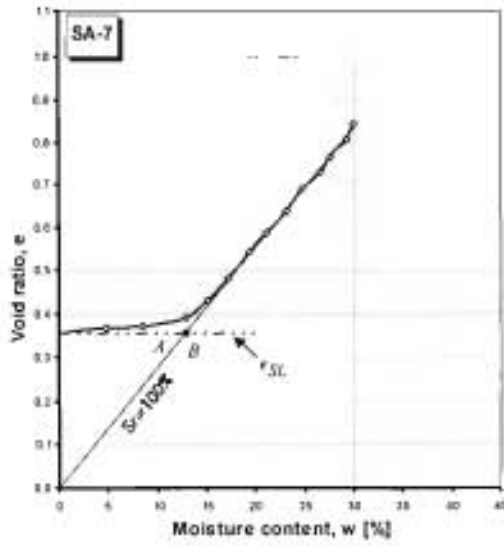
The swell test, which was done in the oedometer under the vertical stress of 15 kpa, involved recording the change in thickness of the sample discs using dial gauge. The horizontal movement was assumed to be zero as the samples were made to exactly fit the 70 mm diameter oedometer rings. The recordings continued until the change in thickness became insignificant. The thickness recording using the dial gauge is used in the calculation of the vertical strain. The vertical strain is calculated from the thickness change expressed as a percentage of the original thickness of sample. Graphical representation of the samples' percentage of vertical strain versus the square root of time is given in Figure 5.7. The figure shows swell strains

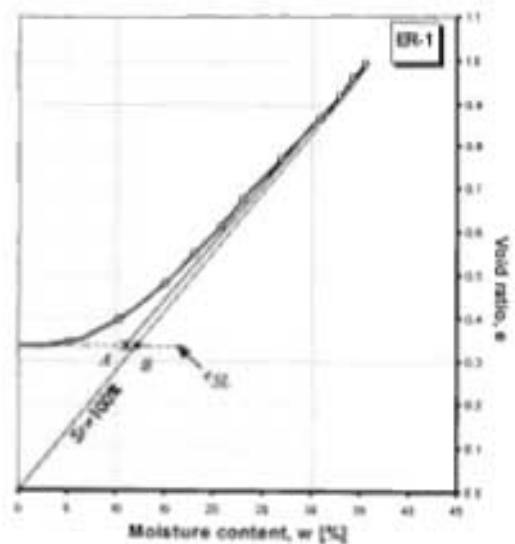
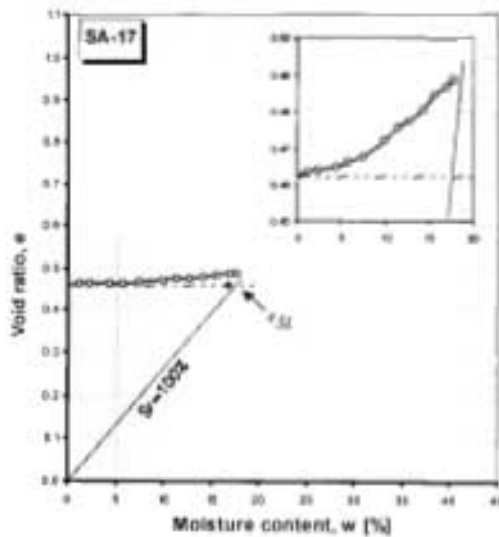
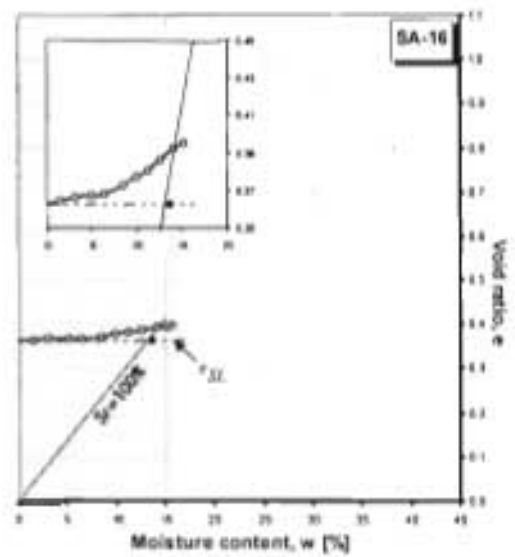
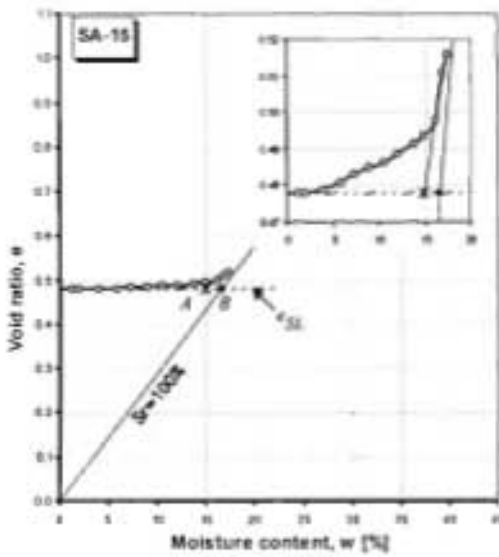
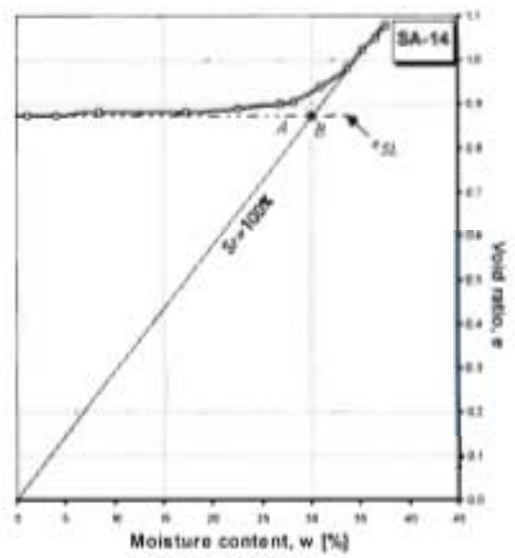
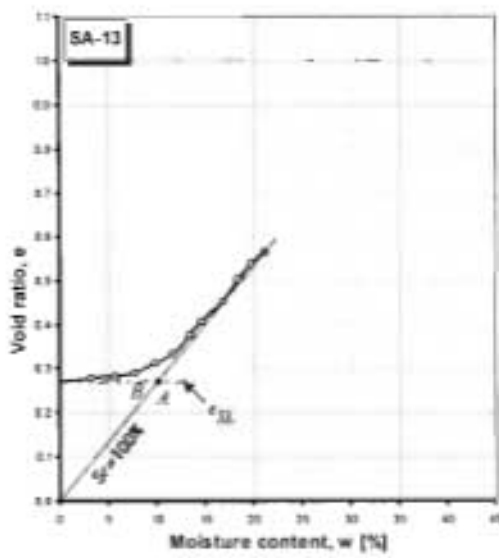
obtained from different sources. The first was the percentage of one-dimensional volume increase due to release of the stress of the sample discs from the stress of 100 kPa to 15 kPa. Following this was the maximum swell strain the soil can attain with change of its moisture state from complete desiccation to full saturation.

An odd reading was collected for sample SA-15; the plot of the processed data obtained showed that the soil was shrinking instead of swelling. This particular sample had coarse sandy particles, which made the fit-in of the sample to the oedometer ring difficult. When finally the sample disc was fitted some space between the disc and the ring along the circumference is observed. It is evident from these facts that the swell was not one-dimensional. Since this particular sample is inconsistent with the rest of the samples where only one-dimensional swelling is assumed, it is not used with the rest of the samples in the analysis.

The sample discs at the end of the swell test were taken out for measuring their weight and thickness. The sample discs were also dried to get their oven-dried weight. From these measurements, moisture content and air void of the samples were calculated. The specific gravity of the soil samples was used to calculate the volume of the sample and the bulk density. The air void of the samples was calculated using the moisture content, bulk density, specific gravity of the samples and specific gravity of water. These records and calculation results are tabulated in Table 5.3.







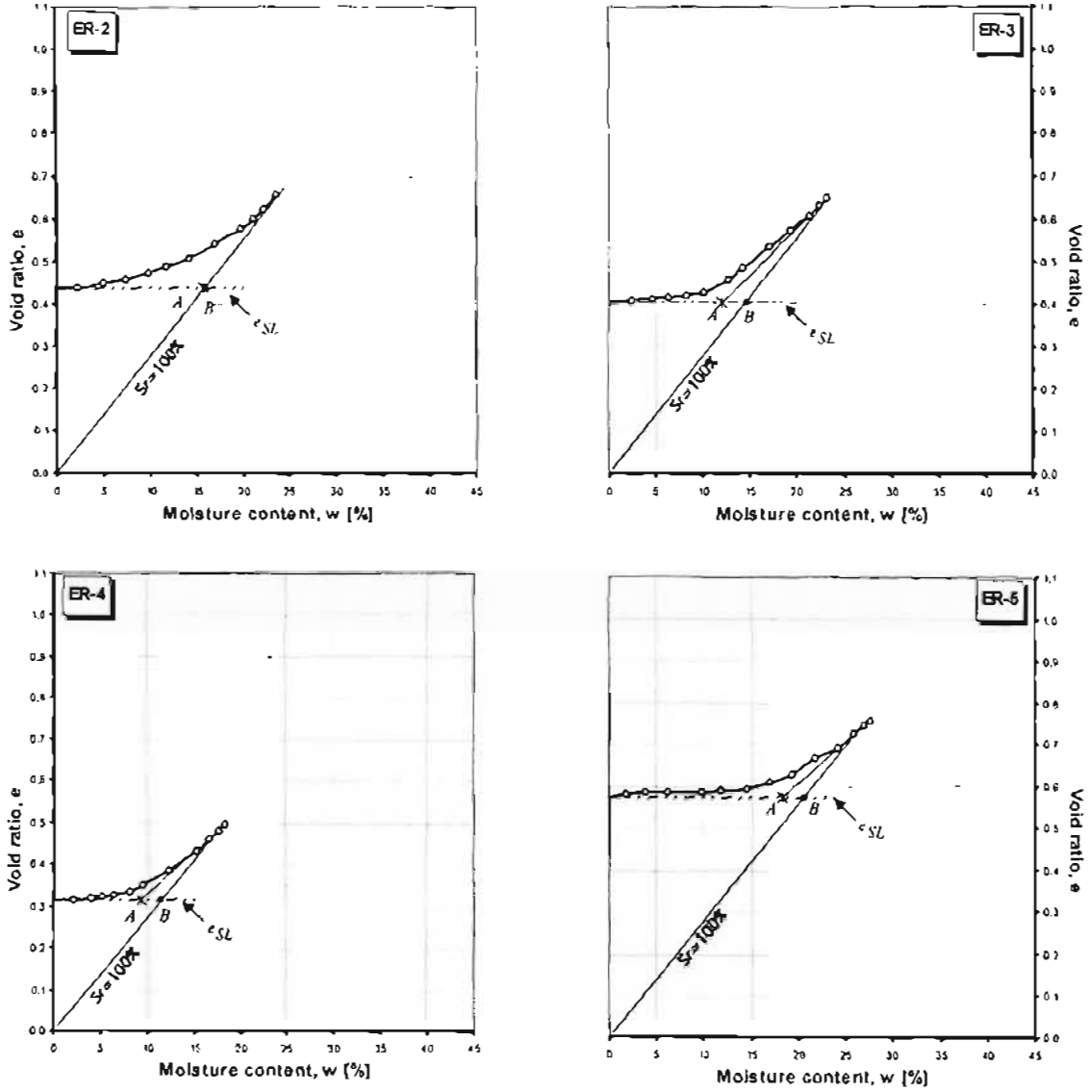


Figure 5.5 Shrinkage curves of the samples of this research.

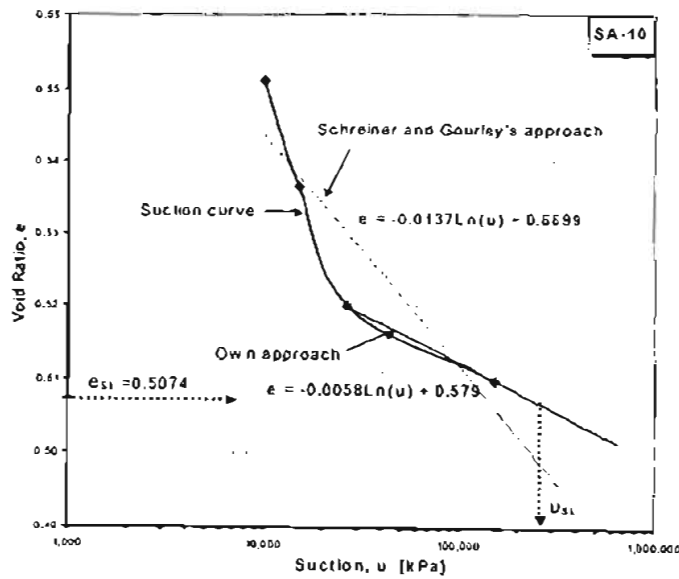
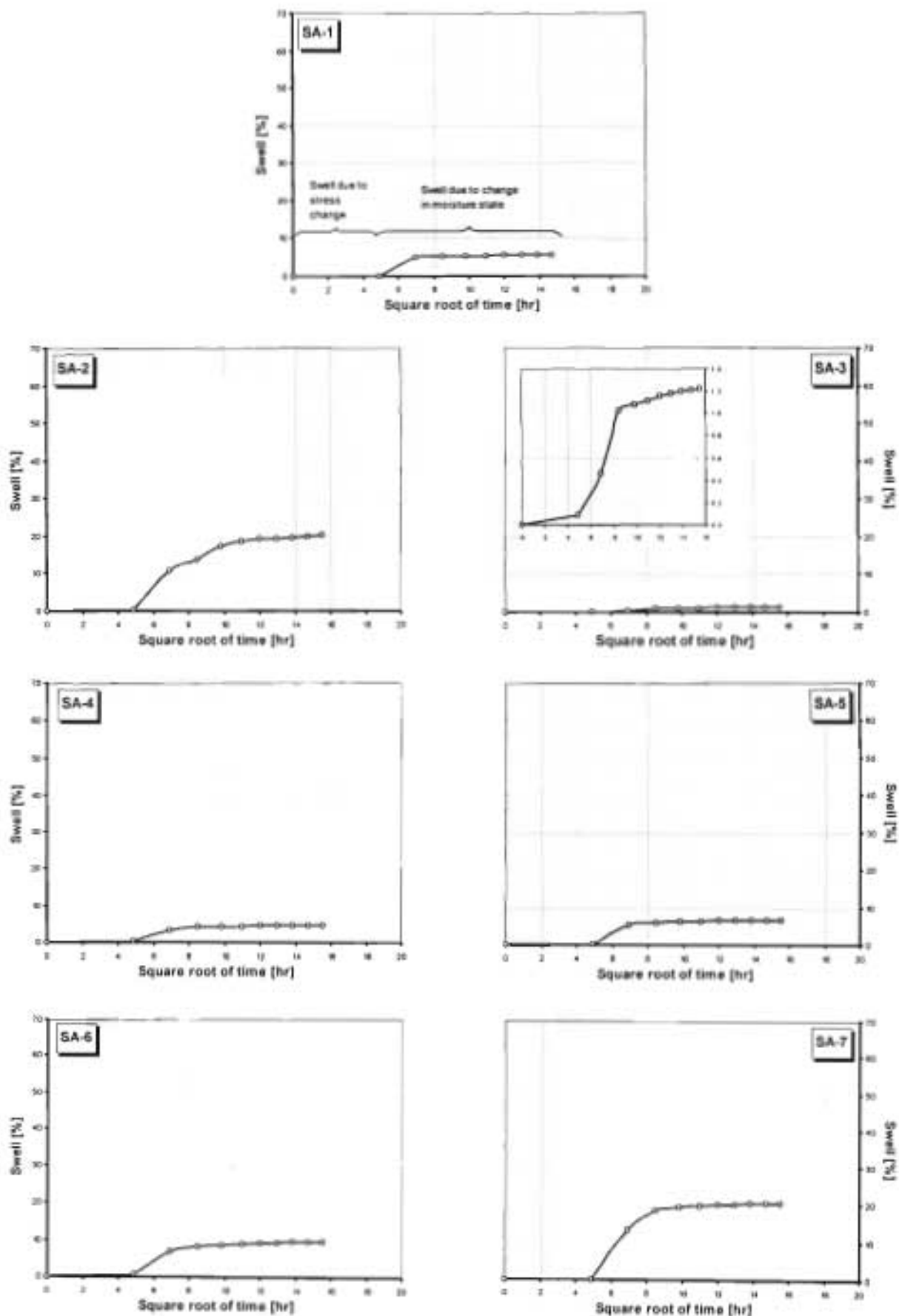
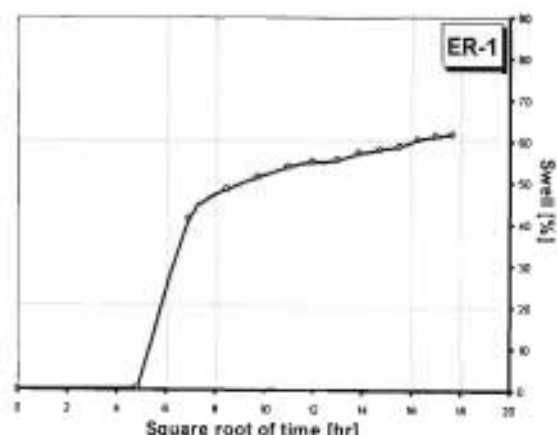
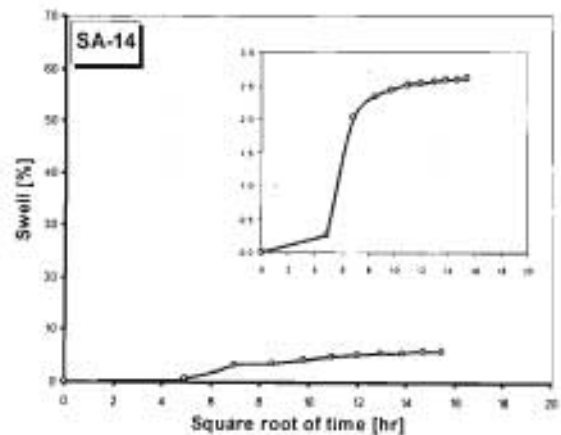
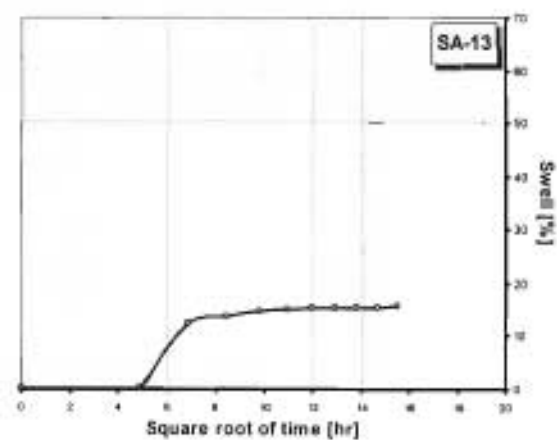
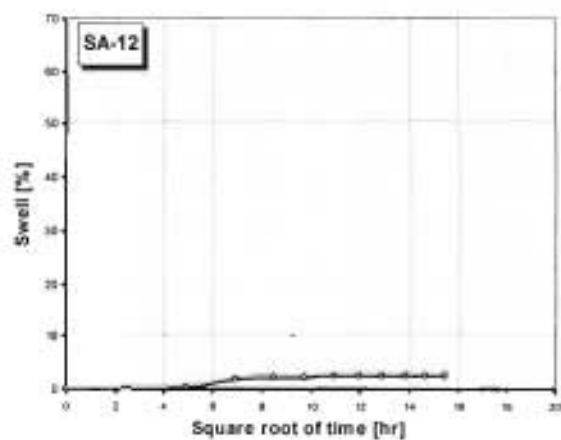
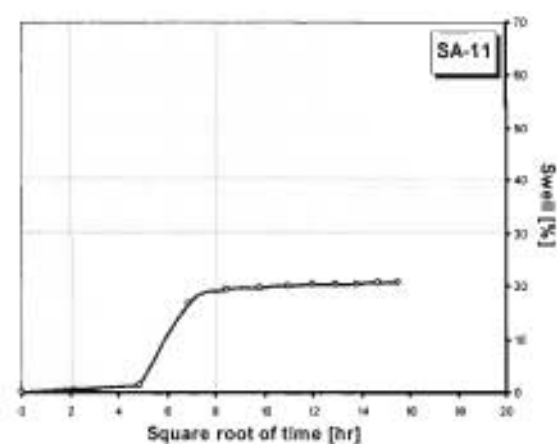
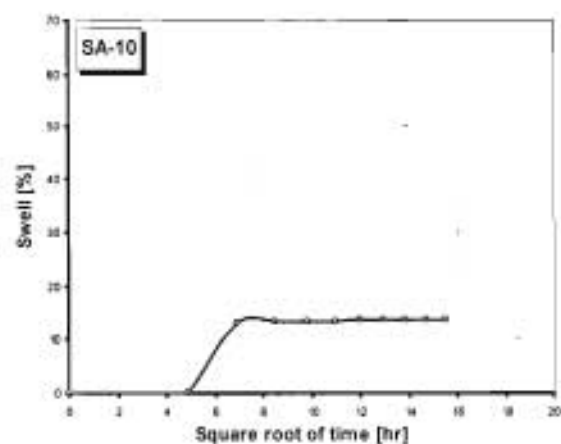
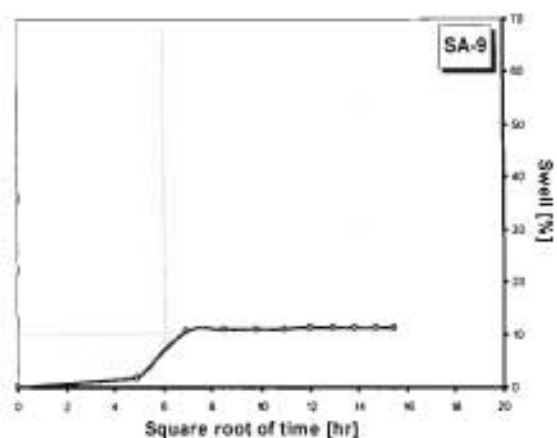
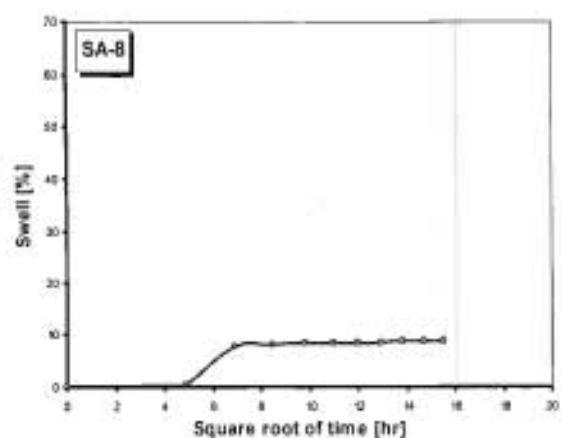


Figure 5.6 An example for the interpolation of the suction at shrinkage.





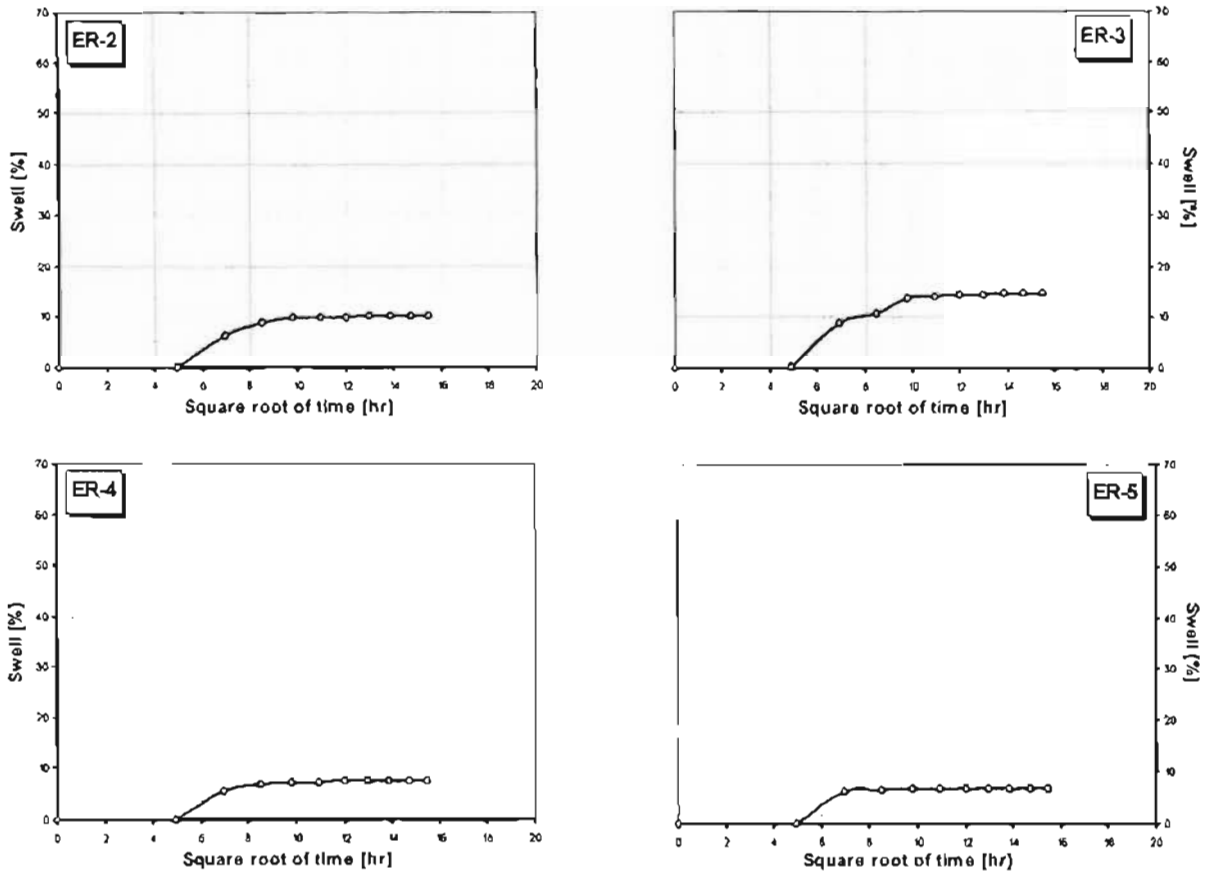


Figure 5.7 Variation of vertical strain of the samples with square root of time.

5.2.7 Summarized Results

The volume changes of the soils in the saturated state are presented in plots of void ratio against logarithm of effective stress. Plots of void ratio against logarithm of suction are instead used to represent the volume changes during the unsaturated states. The summary of all the volume change tests performed on the samples of the current research and that of the Schreiner and Gourley are presented in Figure 5.8. The figures also included a curve representing the relationship $e=wG_s$ for each suction reading of moisture content. These lines represent the void space occupied by water during each step of suction measurement.

Most variables previously used in the assessment of expansiveness and those which are suspected to have an influence on expansiveness of soils are included for analysis. From index tests the following variables are included: C.F., w_L , e_L , w_p , e_p , I_p , Activity, w_{SA} , w_{SB} , I_{SA} , I_{SB} , (w_p-w_{SA}) , $(w_p-$

w_{SB}), and e_{SL} . On the other hand from oedometer tests the variables C_c^* and C_s^* are employed. These variables are shown in Table 5.3. Three measures of expansiveness recommended in Schreiner and Gourley researches are also given in the table. The first is the change in void ratio, Δe , on soaking which is defined as:

$$\Delta e = (e_{1S} - e_{SL})$$

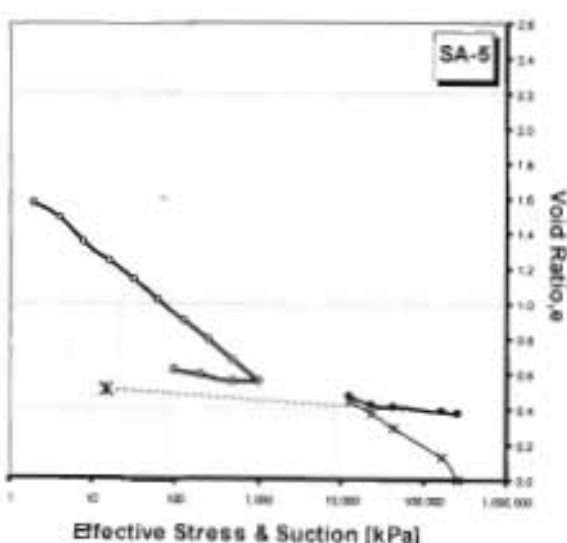
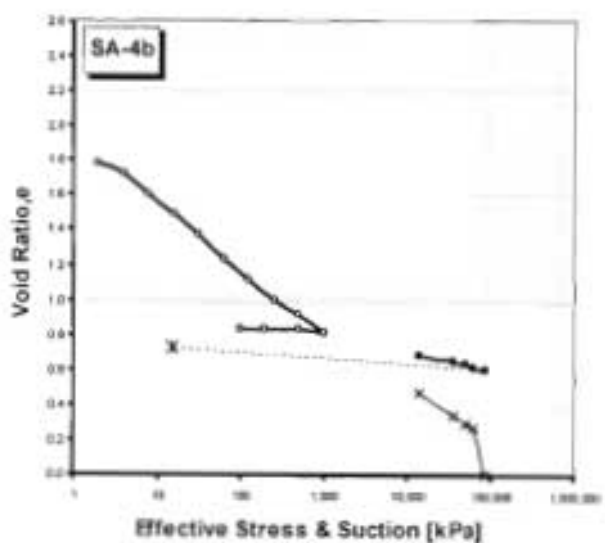
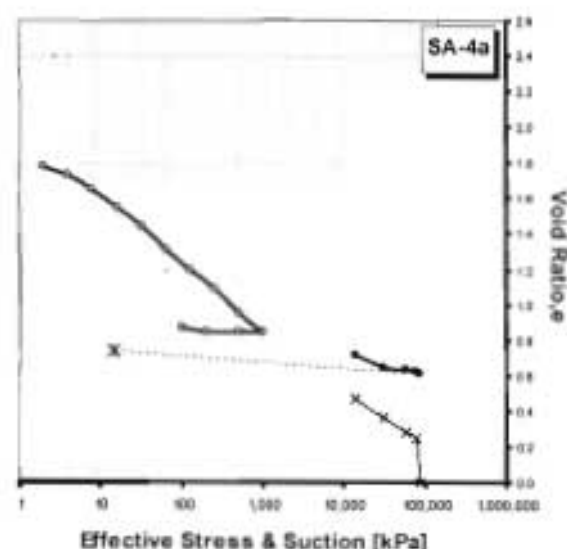
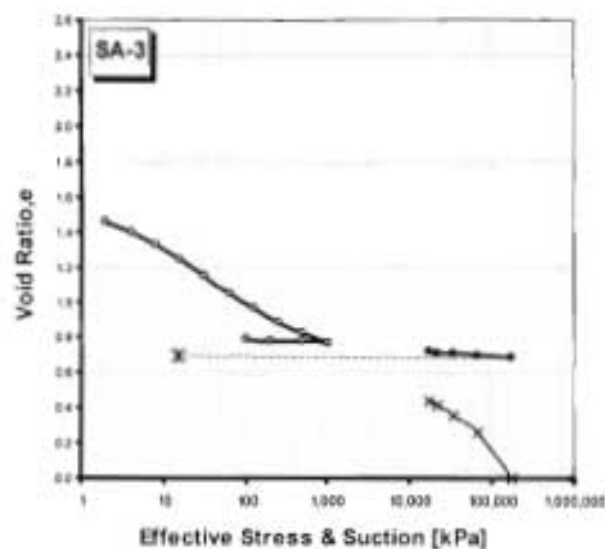
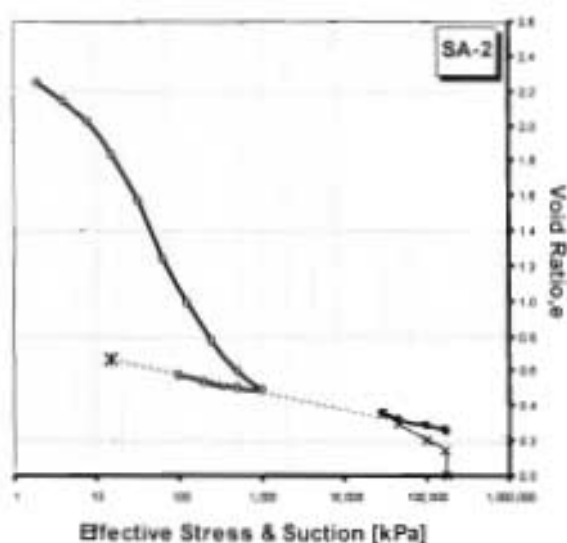
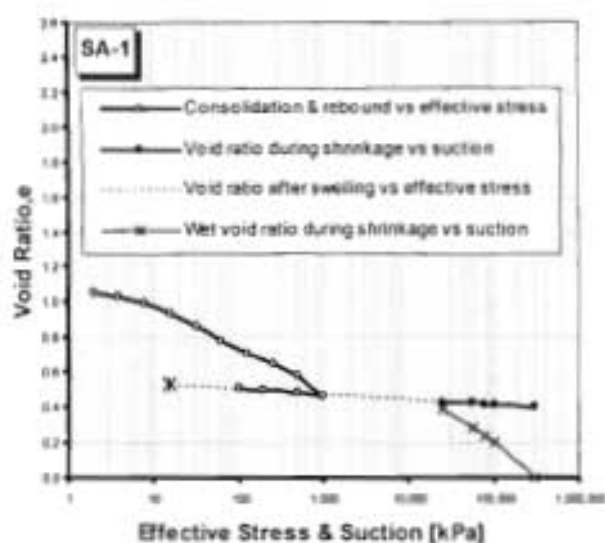
The second one is expansive strain (ϵ_{ex}) on soaking which is defined as:

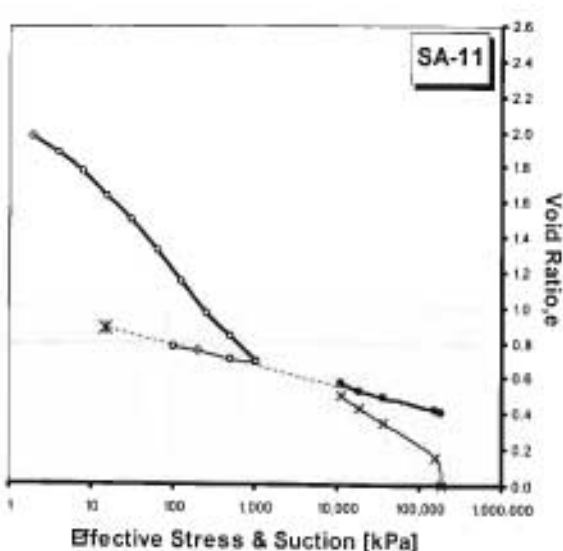
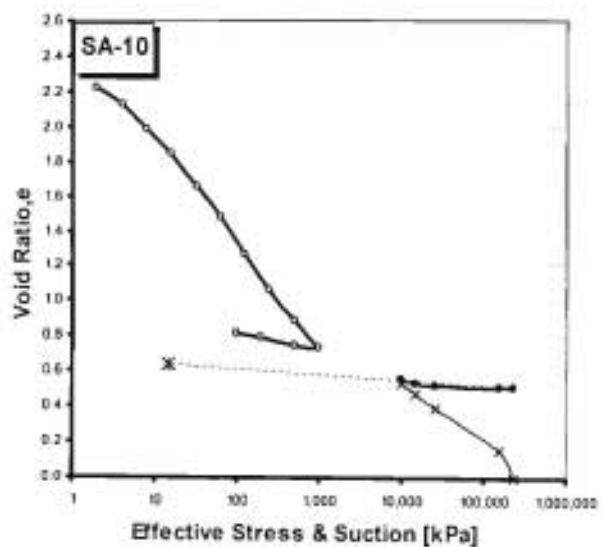
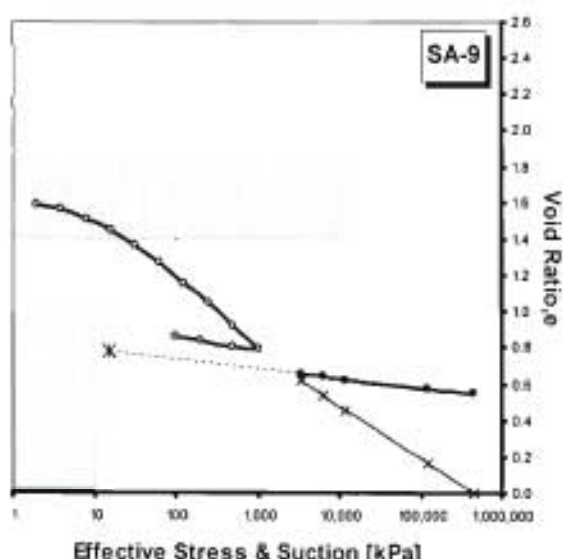
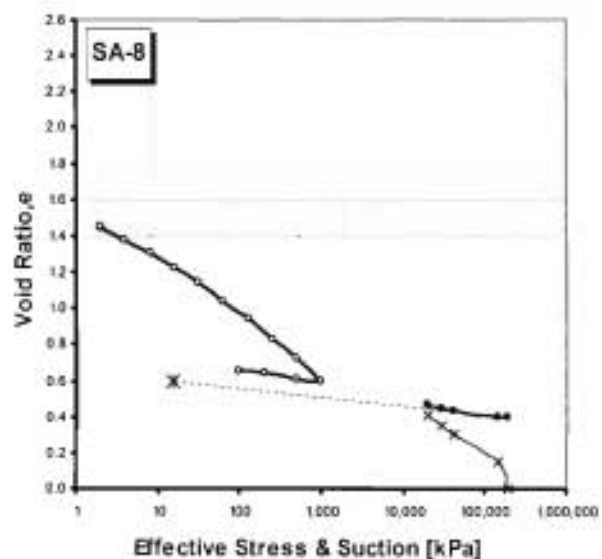
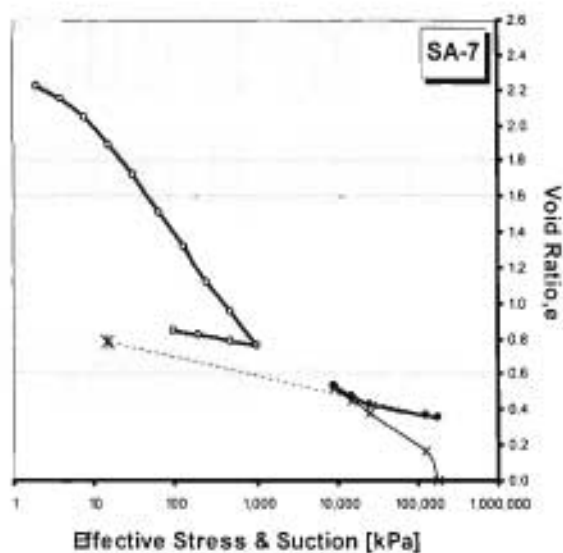
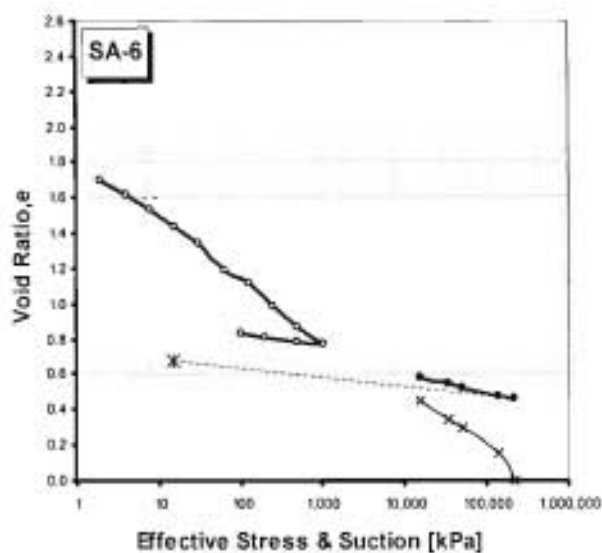
$$\epsilon_{ex} (\%) = \frac{(e_{1S} - e_{SL})}{(e + e_{SL})} \times 100$$

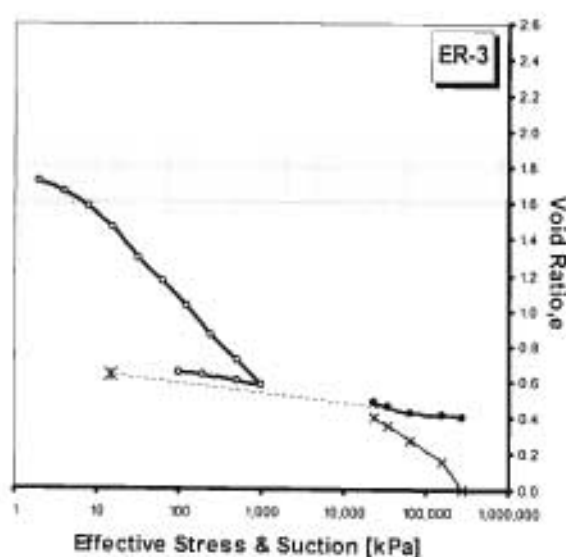
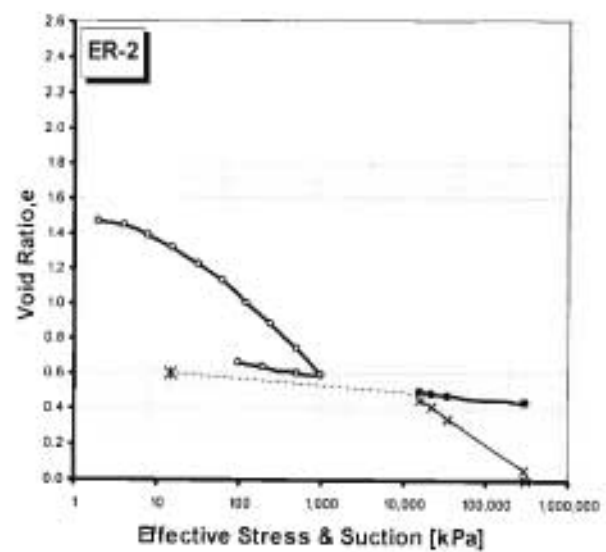
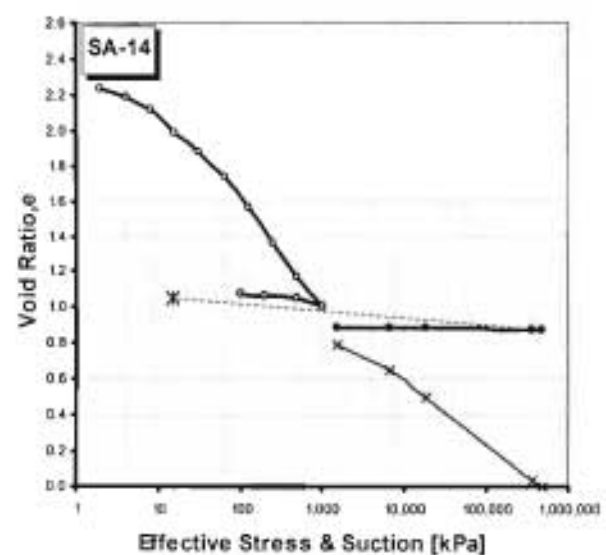
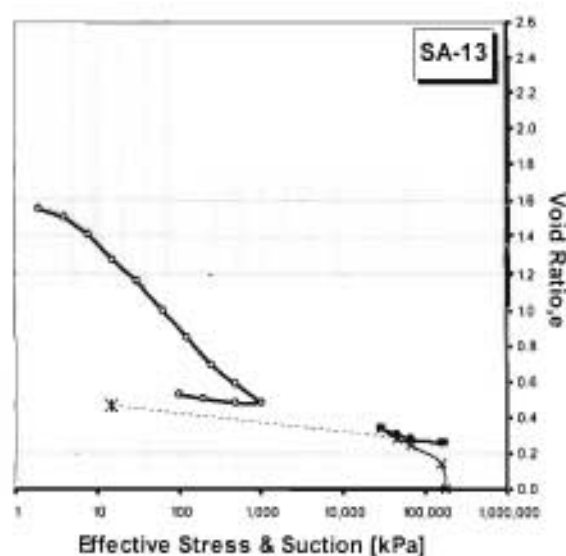
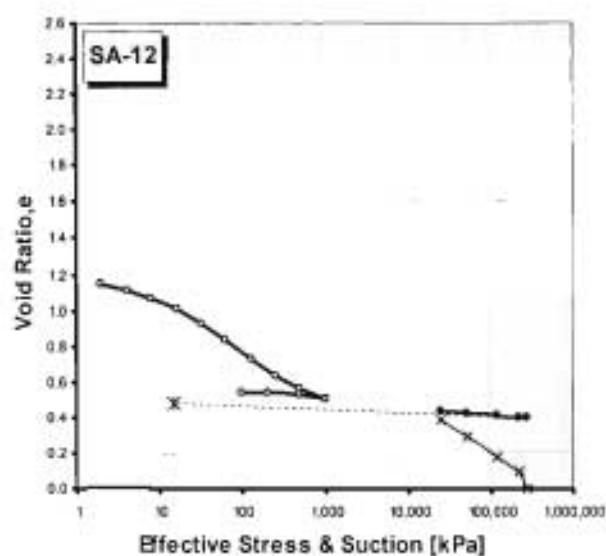
The last one is vertical strain, $\Delta H/H_0$, on soaking under 15 kPa vertical stress from the swell tests.

5.2.8 Reliability of the Tests

As mentioned previously, of the two samples planned to run in duplicate for the purpose of assessing the reliability of the tests only one (SA-4) made it throughout the series of tests. The variation between the two sub-samples in the different tests is given in Figures 5.9-5.12. The slight difference in the saturated loading and unloading, shrinkage limit and swell of the sub-samples can be explained by their variation in the particle size distribution. The particle size distribution showed that SA-4a is less clayey than SA-4b. From theoretical background, relatively, the former sub-sample is expected to be more expansive than the latter, as it has high percentage of clay-sized particles. The figures are supporting this fact. The former showed more consolidation, lower shrinkage limit and higher swell. The figures also prove the repeatability and reliability of the testing procedures, as the slight variation observed between the sub-samples has been explained by another factor.







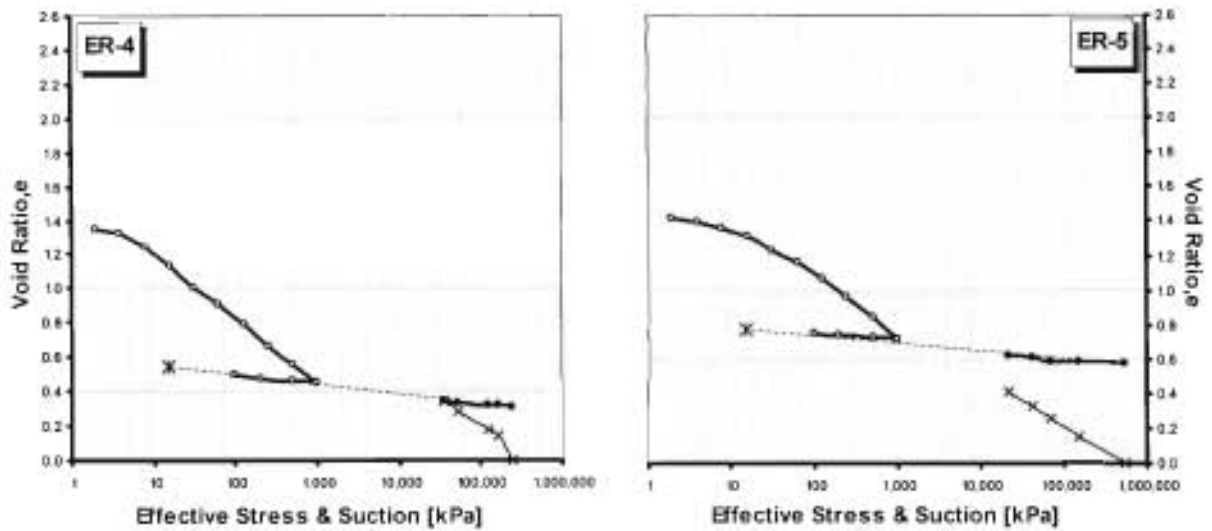


Figure 5.8 Volume change behavior of the samples used in this research.

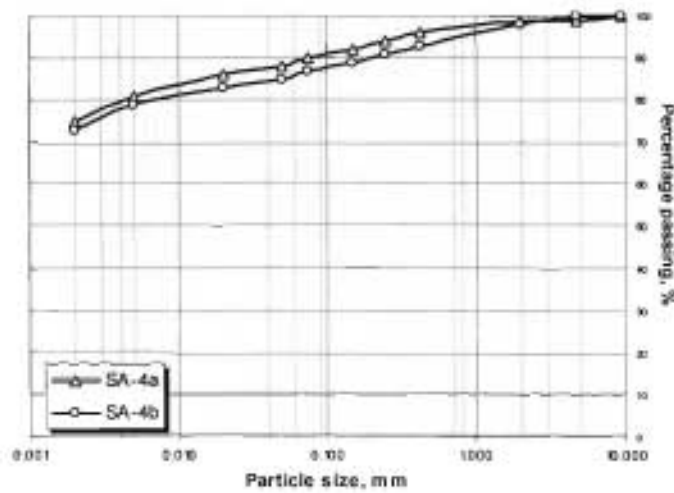


Figure 5.9 Comparison of particle size distribution test results of the two sub-samples of SA-4.

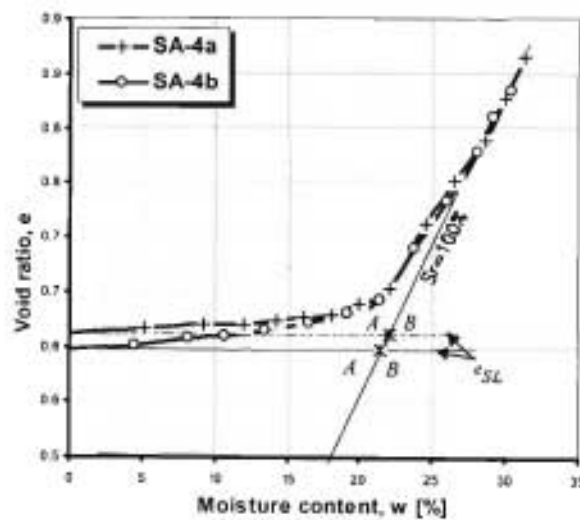


Figure 5.10 Comparison of the shrinkage limit test results of SA-4a & 4b.

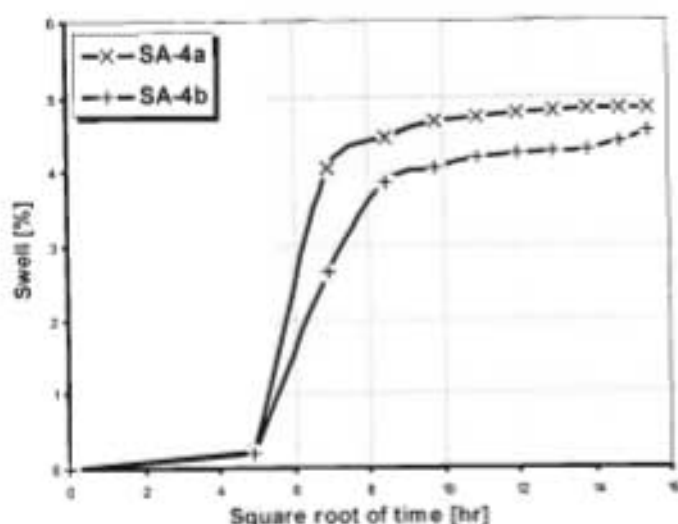


Figure 5.11 Comparison of the swelling test results of SA-4a & 4b.

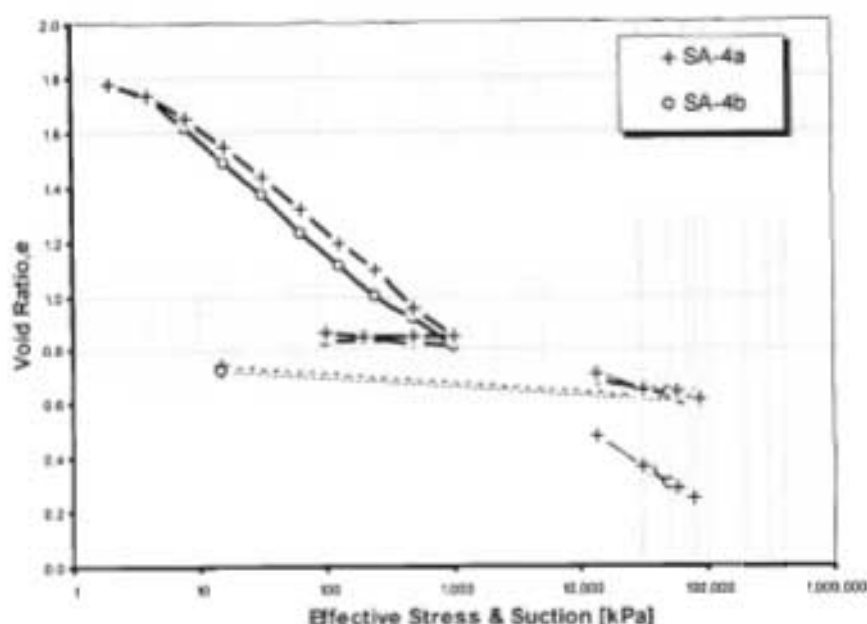


Figure 5.12 Comparison of the results of the series of volume change tests of the two sub-samples.

5.3 STATISTICAL ANALYSIS OF RESULTS

The major intention of this research is to develop a model that establishes a statistically significant relationship between measure of expansiveness and index tests. A multivariate regression analysis was performed for each of the three measures of expansiveness on the summarized test results tabulated in Table 5.3. The relationships of the measures of expansiveness

Table 5.3 Summarized test results of the samples which went through the series of volume change tests.

Tests:	Hydrometer	Liquid and plastic limits			Shrinkage			Combination of index tests						Consolidation			Swell	Suction	Expansiveness measures			
Sample	C.F. (%)	WL (%)	W _p (%)	I _p (%)	W _{SA} (%)	W _{SB} (%)	e _{SL}	I _{SA} (%)	I _{SB} (%)	W _p -W _{SA} (%)	W _p -W _{SB} (%)	Activity	e _{wL}	e _{PL}	C _s	C _c	e ^s ₁₀₀	e _{1s}	u _{SL} (KPa)	Δe	e _{ex} (%)	ΔH/H ₀ (%)
SA -1	42	39	16	23	16	15	0.4150	23	24	0	1	0.72	1.0192	0.4568	0.0375	0.2184	0.5027	0.5351	106,113	0.1201	8	6
SA -2	59	47	14	33	10	8	0.2541	38	39	5	6	0.67	1.2432	0.3976	0.0769	0.6477	0.5685	0.6590	175,155	0.4049	32	20
SA -3	54	45	26	19	25	25	0.6783	20	20	1	1	0.43	1.2410	0.7555	0.0268	0.2584	0.7931	0.6931	172,394	0.0148	1	1
SA -4	77	61	27	34	22	22	0.6046	39	39	5	5	0.51	1.6843	0.7429	0.0217	0.3517	0.8496	0.7323	98,035	0.1276	8	5
SA -5	67	44	25	19	14	14	0.3736	30	30	11	11	0.33	1.1466	0.6380	0.0595	0.3766	0.6118	0.5060	260,947	0.1324	10	7
SA -6	63	53	27	26	17	14	0.4574	36	39	10	13	0.49	1.4541	0.7447	0.0649	0.3429	0.8323	0.6811	221,657	0.2237	15	9
SA -7	71	62	20	42	13	13	0.3560	49	49	7	7	0.69	1.7276	0.5849	0.0791	0.5394	0.8430	0.7782	178,909	0.4222	31	20
SA -8	53	42	22	20	15	12	0.3917	27	30	7	10	0.47	1.1294	0.5884	0.0674	0.3189	0.6579	0.5976	193,615	0.2059	15	9
SA -9	65	61	33	28	21	21	0.5463	40	40	12	12	0.51	1.5974	0.8465	0.0737	0.2940	0.8640	0.7853	462,814	0.2390	15	11
SA -10	47	65	26	39	18	18	0.5074	47	47	8	8	1.05	1.8753	0.7489	0.0769	0.5502	0.8118	0.6391	231,520	0.1318	9	14
SA -11	59	54	21	33	15	13	0.4088	39	41	6	8	0.67	1.5144	0.6066	0.0897	0.4767	0.7762	0.8784	182,653	0.4696	33	21
SA -12	36	30	16	14	14	13	0.3988	16	17	2	3	0.54	0.8617	0.4432	0.0363	0.2368	0.5394	0.4646	278,958	0.0658	5	3
SA -13	37	50	13	37	10	10	0.2707	40	40	3	3	1.37	1.3409	0.3683	0.0457	0.3984	0.5655	0.5101	243,249	0.2394	19	16
SA -14	49	64	38	26	30	30	0.8732	34	34	8	8	0.67	1.8473	1.0866	0.0663	0.4526	1.0763	1.0495	480,166	0.1763	9	6
ER -1	74	82	27	55	12	11	0.3331	70	71	15	16	0.86	2.2506	0.7719	0.1480	0.6242	0.9921	1.4540	377,770	1.1208	84	62
ER -2	44	45	13	32	16	16	0.4379	29	29	-3	-3	0.94	1.2591	0.5055	0.0671	0.3276	0.6568	0.6054	307,552	0.1674	12	10
ER -3	47	49	21	28	15	12	0.4037	34	37	6	9	0.76	1.3671	0.5969	0.0725	0.4223	0.6498	0.6383	270,860	0.2346	17	15
ER -4	39	43	16	27	12	9	0.3130	31	34	4	7	0.93	1.1652	0.4424	0.0465	0.3356	0.4933	0.5455	239,427	0.2325	18	8
ER -5	73	54	27	27	21	18	0.5732	33	36	6	9	0.43	1.4895	0.7578	0.0419	0.2614	0.7549	0.7680	539,112	0.1948	12	7
KE -1*	-	118	48	70	13	14	0.3900	105	104	35	34	-	3.080	1.250	0.1700	0.8262	2.2500	1.750	45,517	1.3200	95	85
KE -2*	-	64	25	39	11	14	0.3800	53	50	14	11	-	1.670	0.650	0.0740	0.4076	1.2200	0.840	30,000	0.4500	33	33
KE -3*	-	68	37	31	23	30	0.8200	45	38	14	7	-	1.820	0.820	0.0540	0.4076	1.3900	0.970	15,556	0.1400	8	9
KE -4*	-	84	42	42	25	25	0.7100	59	59	17	17	-	2.330	0.710	0.0640	0.5928	1.5900	1.140	19,444	0.3500	20	19
KE -5*	-	39	22	17	23	23	0.5400	16	16	-1	-1	-	0.950	0.540	0.0250	0.1445	0.7100	0.570	-	0.0150	1	1.5
KE -6*	-	67	39	28	20	25	0.6500	47	42	19	14	-	1.780	1.040	0.0820	0.3705	1.4900	1.220	15,143	0.5300	32	32
SU -1*	-	59	27	32	13	13	0.3700	46	46	14	14	-	1.640	0.750	0.1000	0.3335	1.1500	0.980	46,154	0.6000	44	41
SU -2*	-	69	36	33	10	15	0.3300	59	54	26	21	-	1.930	1.000	0.1450	0.5928	1.4700	1.220	39,500	0.7900	59	55

* samples from Gourley and Schreiner (1993a)

Table 5.4 Results of the multivariate regression analysis of the individual and combined expansiveness indicator variables.

Parameters	Expansiveness measures					
	Δe		e_{ex}		$\Delta H/H_0$	
	R ²	t	R ²	t	R ²	t
C.F. %	17.3	1.886	15.0	1.732	11.6	1.496
W _L	61.0	6.255	55.1	5.535	62.1	6.399
W _P	20.7	2.557	15.3	2.124	23.1	2.738
I _P	67.1	7.138	65.8	6.928	65.6	6.899
W _{SA}	20.1	-2.504	25.6	-2.930	21.2	-2.590
W _{SB}	9.2	-1.595	13.4	-1.965	7.5	-1.427
e _{SL}	11.7	-1.816	16.5	-2.220	11.3	-1.788
I _{SA}	78.9	9.680	75.3	8.735	80.7	10.221
I _{SB}	79.8	9.925	76.7	9.060	79.1	9.739
W _P -W _{SA}	62.7	6.489	57.6	5.822	62.7	6.489
W _P -W _{SB}	66.8	7.091	62.2	6.416	67.9	7.266
Activity	5.0	0.948	6.2	1.057	11.2	1.463
e _L	58.3	5.917	52.5	5.252	59.1	6.005
e _P	26.8	3.022	21.5	2.613	29.5	3.231
C [*] _S	87.1	12.967	86.7	12.782	87.0	12.931
C [*] _C	56.8	5.731	56.1	5.654	52.7	5.474
e [*] ₁₀₀	45.8	4.593	39.2	4.013	52.5	5.255
e ₁₅	74.1	8.451	67.4	7.185	72.0	8.014
u _{SL}	6.5	-1.292	5.9	-1.224	11.5	-1.765
I _P , Activity	67.3	5.522, -1.724	65.4	5.233, -1.465	70.5	5.665, -1.057
I _P , C.F. %	63.1	4.457, 0.901	61.9	4.438, 0.695	68.5	5.380, 0.275
I _P , W _L	68.7	2.432, 1.119	66.1	2.799, 0.513	68.1	2.128, 1.383
I _P , W _{SA}	72.0	6.668, -2.046	73.8	6.650, -2.717	72.1	6.451, -2.158
I _P , W _{SB}	69.6	6.908, -1.418	70.8	6.863, -2.027	67.3	6.621, -1.124
W _P , W _L	68.7	-2.432, 6.068	66.1	-2.799, 6.001	68.1	-2.128, 5.822
W _P , W _{SA}	73.4	6.946, -6.902	72.8	6.463, -7.13	79.6	8.295, -8.158
W _P , W _{SB}	72.2	7.367, -6.66	71.0	6.907, -6.791	71.9	7.421, -6.464
W _L , W _{SA}	84.1	9.838, -5.908	83.9	9.319, -6.551	86.4	10.730, -6.550
W _L , W _{SB}	82.5	10.019, -5.425	82.0	9.556, -5.985	81.0	9.630, -4.885
C [*] _L , C [*] _C	87.2	7.558, 0.544	86.9	7.490, 4.700	87.0	7.752, 0.215
C [*] _L , I _{SA}	90.9	5.615, 3.182	89.3	5.609, 2.413	91.6	5.578, 3.622
I _P , I _{SA}	79.1	-0.401, 3.709	75.3	-0.056, 3.051	81.5	-1.029, 4.548
I _P , I _{SB}	80.5	-0.970, 4.069	77.0	-0.621, 3.431	80.3	-1.174, 4.23
I _{SA} , C.F. %	69.3	5.203, -0.135	67.3	5.061, -0.311	75.4	6.444, -0.994
I _{SB} , C.F. %	72.2	5.628, -0.223	70.2	5.449, -0.402	77.0	6.738, -1.070
W _L , W _P , C.F. %	63.4	4.155, -2.883, 0.908	63.0	4.108, -3.170, 0.929	68.6	5.045, -3.328, 0.332
W _L , W _P , Activity	68.8	5.429, -3.45, -1.895	68.4	5.396, -3.678, -1.902	71.4	5.48, -3.384, -1.261
I _P , I _{SA} , C.F. %	69.3	-0.049, 1.737, -0.140	67.3	0.070, 1.579, -0.247	75.4	0.047, 2.050, -0.860
I _P , I _{SB} , C.F. %	72.3	-0.159, 2.23, -0.258	70.2	-0.037, 2.049, -0.378	77.0	0.155, 2.350, -0.919
I _P , W _{SA} , W _L	84.6	-0.843, -4.868, 4.340	84.1	-0.558, -5.099, 3.857	88.0	-1.726, -6.158, 5.665
I _P , W _{SB} , W _L	83.6	3.993, -4.558, 4.416	82.7	3.933, -4.685, 3.974	82.4	3.707, -4.33, 4.452
I _{SA} , W _{SA} , W _L	84.6	4.091, -2.304, 0.856	84.1	4.049, -2.495, 0.567	87.9	3.99, -3.319, 1.746
I _{SA} , W _{SB} , W _L	85.1	4.456, -2.477, 1.143	84.4	4.446, -2.593, 0.835	85.5	4.626, -2.278, 1.123
I _{SB} , W _{SA} , W _L	85.3	4.317, -2.724, 1.447	84.8	4.263, -2.928, 1.115	87.5	3.804, -3.835, 2.545
I _{SB} , W _{SB} , W _L	83.6	3.993, -2.037, 1.227	82.7	3.933, -2.153, 0.959	82.4	3.707, -1.965, 1.373
C [*] _S , W _{SA} , W _L	91.7	4.566, -2.624, 3.502	91.5	4.522, -3.253, 3.122	92.8	4.493, -3.253, 4.166

with individual and combined variables (see Table 5.4), as explained by simple linear regressions, were checked for their percentage of variability (R^2) and level of significance (t value).

The models developed from such analysis have a possible form of (varying depending on the number of indicator variables used):

1. I_{exp} (described by $\Delta H/H_o$, Δe , or ε_{ex}) = $aX + b$

where X is an indicator variable, b is a constant and a is a coefficient.

2. I_{exp} (described by $\Delta H/H_o$, Δe , or ε_{ex}) = $aX + bY + c$

where X and Y are indicator variables, a and b are coefficients and c is a constant.

3. I_{exp} (described by $\Delta H/H_o$, Δe , or ε_{ex}) = $aX + bY + cZ + d$

where X, Y, and Z are indicator variables, a, b and c are coefficients and d is a constant.

Some of the most important models obtained are described in the next chapter.

CHAPTER SIX

6. DISCUSSION OF THE RESEARCH FINDINGS

6.1 INTRODUCTION

In chapter five only research results were presented. This chapter discusses the research findings in greater detail in four sections. The first section is devoted to explain the basic soil mechanics behind the test results obtained. In the second section the models developed to describe the intrinsic expansiveness of soil in terms of index test results are presented and another separate section is given for explaining the appropriate usage of the models. The last section then gives a classification system for assessing intrinsic expansiveness.

6.2 VOLUME CHANGE RESULTS

As presented in the previous chapters the series of volume change tests performed on the reconstituted samples are one-dimensional compression, one-dimensional unloading, unconfined shrinkage, and one-dimensional swelling.

The shape of compression curves when plotted in a semi-logarithmic scale is affected by a number of factors. Results of the current and Schreiner and Gourley's researches are illustrating that the shape of the curves can be specifically affected by stress history and compressibility of the soil. Under absence of stress history the curves are expected to describe the decrease of void ratio with increase in effective stress in a straight line. The research results agree with this fact for most of the samples. Nonetheless, results of some of the highly clayey samples showed slight curve at the beginning of the line. The most probable explanation for the curves is the high compressibility nature of these soils where slight application of stress during the process of preparation of the samples result a significant reduction in void ratio. No similar effect is observed on the compression curves of clayey soils with more sandy and silty particles.

By definition the compressibility of a soil is represented by the slope of the compression line (Lambe and Whitman 1979). The test results confirm that soils which are more clayey have steeper compression line than those which are less clayey.

The unloading portions of the consolidation curve are also found to be nearly straight lines for most of the samples tested. In contrast to the compression lines, the unloading lines show increase of the void ratio with decline of effective stress.

Similar to the unloading lines are saturated swelling curves. These curves are assumed to be straight lines and are found to be roughly parallel to their respective unloading lines. Atkinson and Bransby (1978) and Lambe and Whitman (1979) also noted the validity of such argument.

It has been stated in the literature (Sridharan and Rao 1971; Yong and Warkentin 1975; Sridharan and Prakash 1998; Sridharan and Nagaraj 2000) that capillary forces initiate the shrinkage process. The capillary stresses depend on pore size, the smaller the pore-size the higher the capillary stresses. Smaller pore sizes are characteristic of soils dominated by clay minerals such as montmorillonite. With water loss, radius of the meniscus of the soil water decreases and menisci retreat into the soil mass until the shear stress induced by the capillary stresses are equalized by the shear strength mobilized due to interparticle friction and cohesion at the particle level.

The capillary stress remains strong in highly expansive soils giving them curves showing their saturation for extended void ratios. Clayey soils with small percentage of clay particles were found to stay saturated for lesser moisture contents, because of their small capillary stress. In some of such soils it was observed that when sandy and silty particles are in a very large proportion it was difficult to record any saturation point. The very open texture of the soil is one of the factors that facilitated the loss of water during measurement of the weight of the discs. One may learn from this the need for reconsideration on the sample preparation method for these particular types of soils. Because the preparation of the samples into OCR of

10 might have brought the granular particles so close to each other leaving behind almost no space for volume increase with adsorption and absorption of water molecules by the clay particles. It might be a useful further research to see the overall volume change behaviour of such clayey soils by preparing their samples at OCR less than 10.

Because of its reliability the non-contact filter paper method was employed for the assessment of the variation of the matric suction of the soil during the drying process of the shrinkage test. Apparently due to condensation it was not possible to obtain the overall suction curve. From most of the diagrams showing the logarithm of suction versus air void relationships, for the conditions where accurate measurement of the matric suctions were possible, generally it is possible to divide the curve into two. The first phase which starts with the beginning of the suction test are steep sloped, roughly, straight lines. Around the end of the test are the second phase which are basically straight lines with very gentle slopes. These observations agree with the theoretical phases of the tests given by Gourley and Schreiner (1993a) and Schreiner (1999) and test results of Fleureau *et al* (2002). Both authors mentioned that the transitional matric suction between the two phases termed as the suction of the shrinkage limit, u_{SL} . Modaressi *et al* (1996) and Kohgo (2002) stressed that this pressure plays an important part in modelling the behaviour of the soils as it corresponds to the change of behaviour of the soil from plastic to elastic.

The state of moisture content of the soils was changing during the series volume change tests. Each of the samples remained saturated during the one-dimensional compression test. The degree of saturation of the samples started declining during the shrinkage test and finally reached desiccation. The swell test brought the desiccated samples to full saturation.

In the state of saturation the effective stress governs the volume change behavior (Alonso 1998). The volume change under this state is always equal to the water volume change (Ho *et al* 1992). Once the soils started to be unsaturated, effective stress is no longer adequate to govern volume change, as the soils will additionally contain air between the solid particles.

In such cases both applied stress and suction will result the volume changes (Schreiner and Gourley 1993).

Nonetheless, Gourley and Schreiner (1993a) pointed out the possibility of assessing the intrinsic expansiveness of unsaturated clayey soils using their volume change findings under saturated state. In fact, using their few samples, they demonstrated that C^*_s and C^*_c , respectively, are the most significant variables describing the volume change behaviour of the unsaturated clayey soils. Therefore, they remarked that the volume change in both saturated and unsaturated states is influenced by the same soil properties. The recent experimental results also support these remarks (see the following section).

6.3 MODELS FOR ASSESSING SOIL'S EXPANSIVENESS

6.3.1 Previous Assessment Systems

Considering the importance of exhaustive analysis in order to get a reliable system for assessing soil expansiveness priority was given to check if the results of the soils studied in this project show any significant trend in the previous assessment systems. Because of their popularity the analysis was performed on the following classification systems in terms of index tests: modified plasticity chart after Vijayvergiya & Ghazzaly (1973), modified plasticity chart after Dakshanamurthy & Raman (1973), classification chart after van der Merwe (1964), classification chart after Williams & Donaldson (1980), and shrinkage potential table by the BRE (1980).

Figures 6.1 to 6.4 are clearly showing the ineffectiveness of the previous classification systems to predict properly soil's intrinsic expansiveness. The most serious discrepancy lies on the categories of expansiveness in which the different classification systems do not agree one to another. For each type of clayey soil all the methods show a considerable variation of expansiveness. For instance a soil that is considered to have a high expansiveness in one of the classification system might have a low expansiveness in another classification system. Either it could be explained by the uniqueness of the sample preparation used in this study or by mineralogy of the clay particles. In conclusion, of the previous methods,

none is found to reliably classify the clayey soils tested. It is evident from this the need for a new model independently from the twenty seven samples studied under the same sample preparation and series of tests.

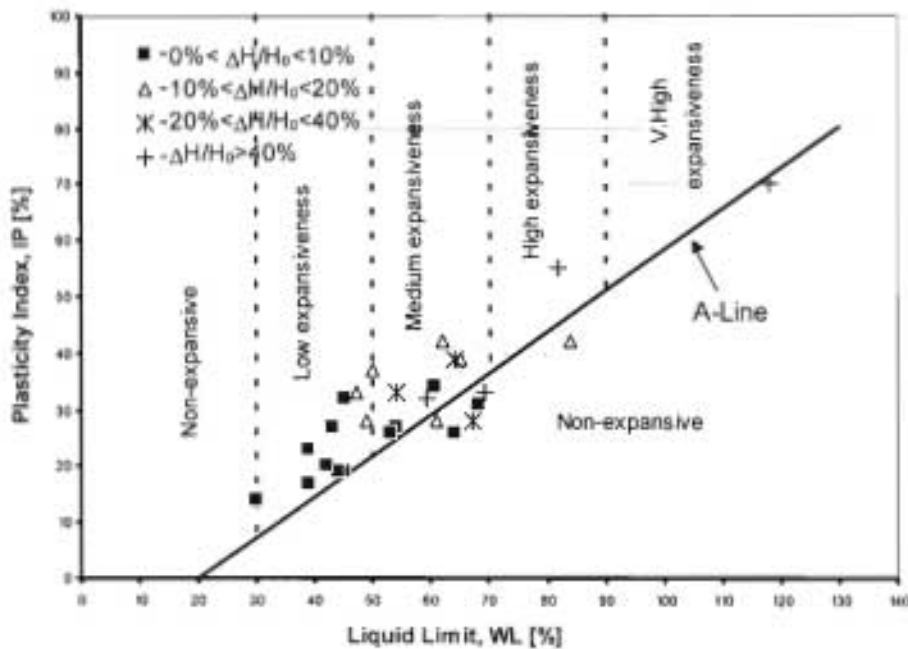


Figure 6.1 Comparisons of Vijayvergiya & Ghazzaly (1973) and Dakshanamurthy & Raman (1973) classification charts with the measured intrinsic expansiveness.

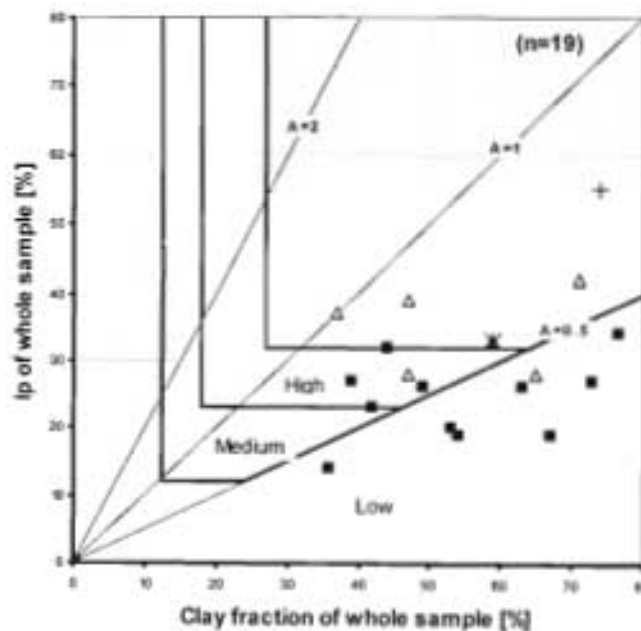


Figure 6.2 Comparisons of van der Merwe (1964) classification chart with the measured intrinsic expansiveness.

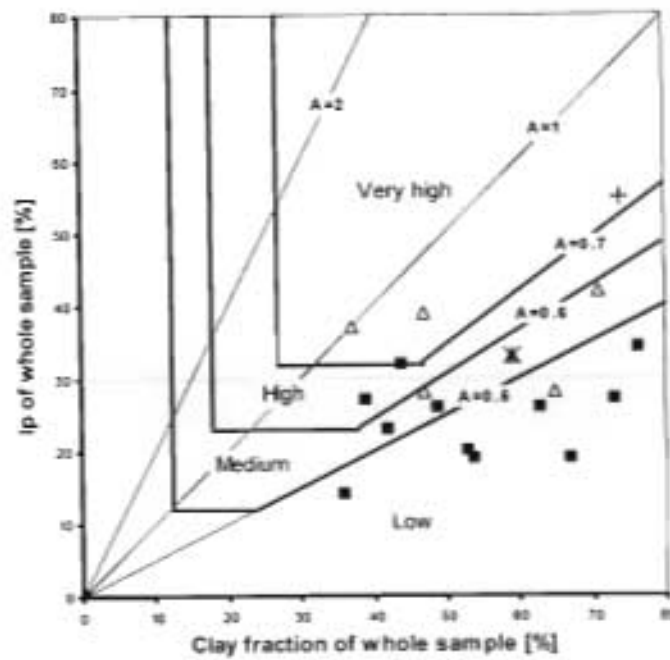


Figure 6.3 Comparisons of Williams & Donaldson (1980) classification chart with the measured intrinsic expansiveness.

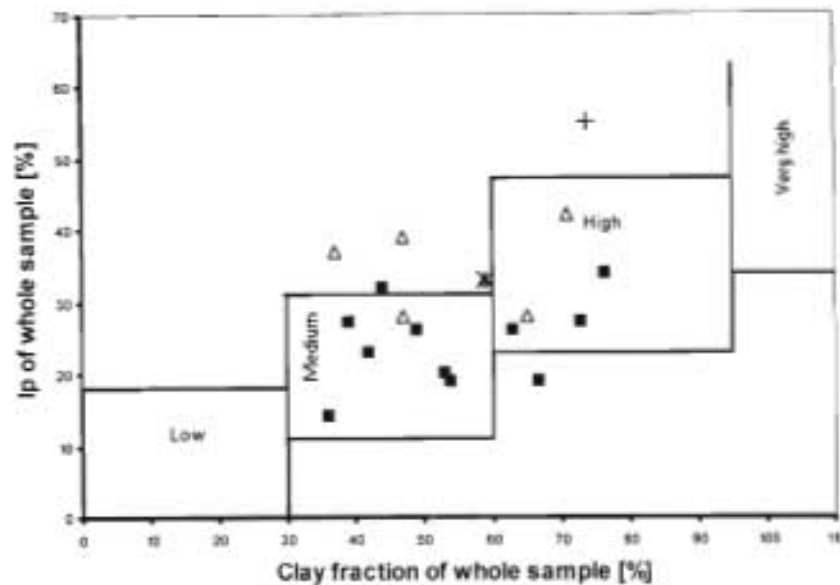


Figure 6.4 Comparisons of Williams & Donaldson (1980) classification chart with the measured intrinsic expansiveness.

6.3.2 New Models

Individual and combination of the indicator variables were checked if they have a significant relationship with the different measures of expansiveness as presented in Table 5.4. Besides its significance, the choice of a particular model for assessing expansiveness also needs consideration of its cost. The cost of a model is mainly influenced by the cost of the tests required to

derive the indicator variables included in the model. Most geotechnical site investigations involve simple and cheap tests including liquid limit, plastic limit and hydrometer, which could be performed in any ordinary geotechnical laboratory. The use of indicator variables obtained from these tests is thus the cheapest. The next expensive test used in the study is shrinkage limit, which is not part of the routine geotechnical site investigation because of its cost. But the most expensive and time consuming of all the tests used, which drives useful expansiveness indicator variables is the oedometer testing. Not most ordinary geotechnical laboratories contain this apparatus.

The multivariate analysis given in Table 5.4 presented a number of possible models to describe intrinsic expansiveness using the three measures. For the purpose of facilitating the choice of a model the following sections are describing the possible models according to the cost of the testing they require.

6.3.2.1 Models requiring oedometer testing

Research findings of Schreiner and Gourley using eight samples showed that the best models contain intrinsic swell index and intrinsic compression index, both to be obtained from the oedometer testing. When the eight samples are analyzed together with the nineteen samples from this study still the best models contain intrinsic swell index but not intrinsic compression index.

As it was the case in the Schreiner and Gourley's research, the analysis of the twenty seven samples is showing that of all the individual soil parameters statistically analyzed C_s^* is the most significant indicator to describe the intrinsic expansiveness:

$$\Delta e = 8.103C_s^* - 0.236 \quad [R^2=87.1, n=27] \quad (\text{Model 1a})$$

$$\epsilon_{ex} = 598.365C_s^* - 18.493 \quad [R^2=86.7, n=27] \quad (\text{Model 1b})$$

$$\Delta H/H_o = 521.52C_s^* - 17.476 \quad [R^2=87.0, n=27] \quad (\text{Model 1c})$$

For this particular indicator variable change of void ratio is the most significant measures of intrinsic expansiveness (with $R^2=87.1$) unlike the

finding of Schreiner and Gourley’s research, which was ϵ_{ex} . Graphical representation of the relationship between the predicted intrinsic expansiveness that uses C_s^* , which is determined under saturated condition, and the measured change in void ratio on soaking is given in Figure 6.5. The figure is showing that all the samples are randomly distributed at about the line of equality in the range ± 0.27 of Δe for the Model 1a.

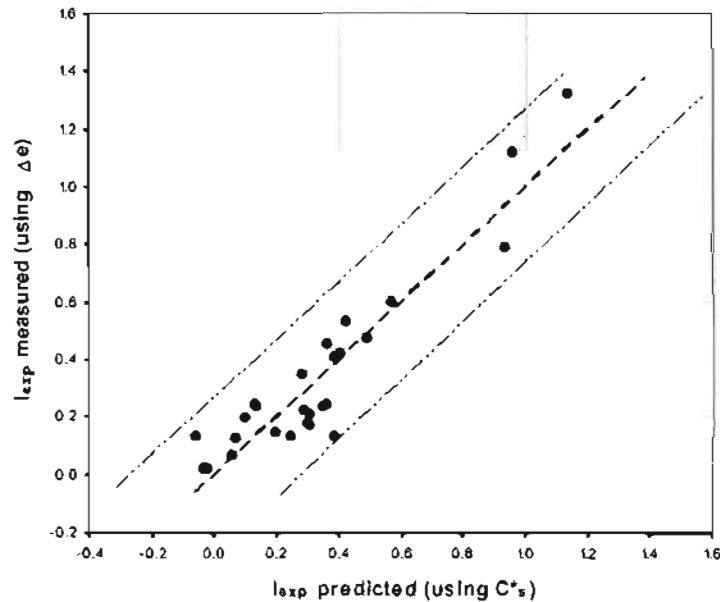


Figure 6.5 Comparison of intrinsic expansiveness predicted using Model 1a and measured using change of void ratio.

Schreiner and Gourley’s research has not reported any model described using more than two indicator variables. The analysis performed in this study is however showing that the most significant relationship obtained of the entire individual and combined indicators contains C_s^* , W_L , and W_{SA} . The models containing these individual variables under the different measures of expansiveness are:

$$\Delta e = 4.833C_s^* + 6.816 \times 10^{-2} W_L - 1.28 \times 10^{-2} W_{SA} - 0.182 \quad [R^2=91.7 \%, n=27] \quad (\text{Model 2a})$$

$$\epsilon_{ex} = 358.347C_s^* + 0.455W_L - 1.19W_{SA} - 7.732 \quad [R^2=91.5 \%, n=27] \quad (\text{Model 2b})$$

$$\Delta H/H_o = 285.48C_s^* + 0.49W_L - 0.95W_{SA} - 12.80 \quad [R^2=92.8 \%, n=27] \quad (\text{Model 2c})$$

The three measures of intrinsic expansiveness showed a comparable significance, swell strain being the best with $R^2=92.8 \%$. The samples are

distributed about the line of equality in the range ± 11 of $\Delta H/H_0$ % for Model 2c (see Figure 6.6).

The presence of the intrinsic swell index in the most significant models might show the identicalness of the soil properties responsible for volume change behavior of both saturated and unsaturated clayey soils. This proves the possibility of studying expansiveness of unsaturated soils from their volume change behavior at their saturated state.

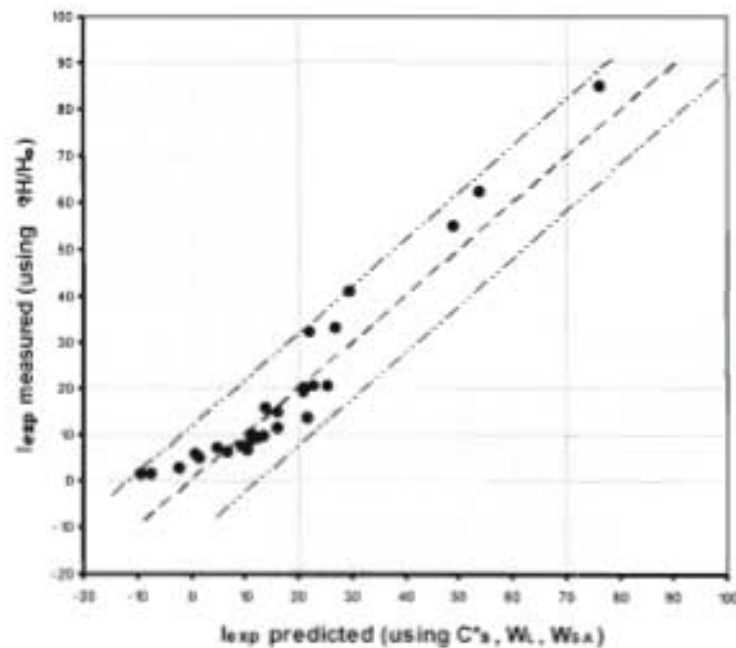


Figure 6.6 Comparison of intrinsic expansiveness predicted using Model 2c and measured using expansive strain.

The practical use of the variable as indicator of expansiveness, however, is limited because of its cost. Thus, it might be important to look for cheaper models.

6.3.2.2 Models requiring shrinkage testing

Shrinkage test is relatively cheaper than oedometer test. Shrinkage limit alone is a very poor indicator of intrinsic expansiveness:

$$\Delta e = 0.779 - 2.63 \times 10^{-2} W_{SA} \quad [R^2=20.1 \%, n=27] \quad (\text{Model 3a})$$

$$\epsilon_{ex} = 60.787 - 2.194 W_{SA} \quad [R^2=25.6 \%, n=27] \quad (\text{Model 3b})$$

$$\Delta H/H_0 = 48.677 - 1.737 W_{SA} \quad [R^2=21.2 \%, n=27] \quad (\text{Model 3c})$$

$$\Delta e = 0.597 - 1.54 \times 10^{-2} W_{SB} \quad [R^2=9.2 \%, n=27] \quad (\text{Model 3d})$$

$$\epsilon_{ex} = 46.963 - 1.375W_{SB} \quad [R^2=13.4 \%, n=27] \quad (\text{Model 3e})$$

$$\Delta H/H_o = 34.533 - 0.898W_{SB} \quad [R^2=7.5 \%, n=27] \quad (\text{Model 3f})$$

The significance of the indicator gets better with the use of shrinkage index (which is the difference between liquid limit and shrinkage limit):

$$\Delta e = 1.53 \times 10^{-2} I_{SA} - 0.288 \quad [R^2=78.9 \%, n=27] \quad (\text{Model 4a})$$

$$\epsilon_{ex} = 1.105 I_{SA} - 21.352 \quad [R^2=75.3 \%, n=27] \quad (\text{Model 4b})$$

$$\Delta H/H_o = 0.996 I_{SA} - 21.287 \quad [R^2=80.7 \%, n=27] \quad (\text{Model 4c})$$

$$\Delta e = 1.597 \times 10^{-2} I_{SB} - 0.316 \quad [R^2=79.8 \%, n=27] \quad (\text{Model 4d})$$

$$\epsilon_{ex} = 1.159 I_{SB} - 23.531 \quad [R^2=76.7 \%, n=27] \quad (\text{Model 4e})$$

$$\Delta e = 1.424 \times 10^{-2} W_L - 2.82 \times 10^{-2} W_{SA} - 9.22 \times 10^{-3} \quad [R^2=84.1 \%, n=27] \quad (\text{Model 5a})$$

$$\epsilon_{ex} = 1.005 W_L - 2.332 W_{SA} + 5.105 \quad [R^2=83.9 \%, n=27] \quad (\text{Model 5b})$$

$$\Delta H/H_o = 0.925 W_L - 1.864 W_{SA} - 2.575 \quad [R^2=86.4 \%, n=27] \quad (\text{Model 5c})$$

$$\Delta e = 1.550 \times 10^{-2} W_L - 2.40 \times 10^{-2} W_{SB} - 0.154 \quad [R^2=82.5 \%, n=27] \quad (\text{Model 5d})$$

$$\epsilon_{ex} = 1.11 W_L - 1.988 W_{SB} - 6.805 \quad [R^2=82.0 \%, n=27] \quad (\text{Model 5e})$$

$$\Delta H/H_o = W_L - 1.45 W_{SB} - 13.887 \quad [R^2=81.0 \%, n=27] \quad (\text{Model 5f})$$

$$\Delta H/H_o = 1.025 I_{SB} - 22.468 \quad [R^2=79.1 \%, n=27] \quad (\text{Model 4f})$$

Further analysis, using the liquid limit and shrinkage limit as separate variables in a single regression produces even better correlations:

Far better models than Model 5 are obtained when an additional factor of plasticity index is used. In all the above-mentioned models the shrinkage limit determined according to ASTM standard produced relatively better correlation compared that determined according to BSI. It is also the case when the I_p indicator is included:

$$\Delta e = 1.739 \times 10^{-2} W_L - 3.2 \times 10^{-2} W_{SA} - 5.33 \times 10^{-3} I_p + 4.109 \times 10^{-2} \quad [R^2=84.6 \%, n=27] \quad (\text{Model 6a})$$

$$\epsilon_{ex} = 1.162 W_L - 2.521 W_{SA} - 0.265 I_p + 7.605 \quad [R^2=84.1 \%, n=27] \quad (\text{Model 6b})$$

$$\Delta H/H_o = 1.292 W_L - 2.305 W_{SA} - 0.62 I_p + 3.278 \quad [R^2=88.0 \%, n=27] \quad (\text{Model 6c})$$

By far the second most significant model (following Model 2c) obtained in the entire multivariate regression analysis is Model 6c. Plot of the predicted intrinsic expansiveness using this model versus measured intrinsic

expansiveness using expansive strain is showing that the studied samples are distributed only in the range ± 14.5 of $\Delta H/H_0$ % for the line of equality (Figure 6.7).

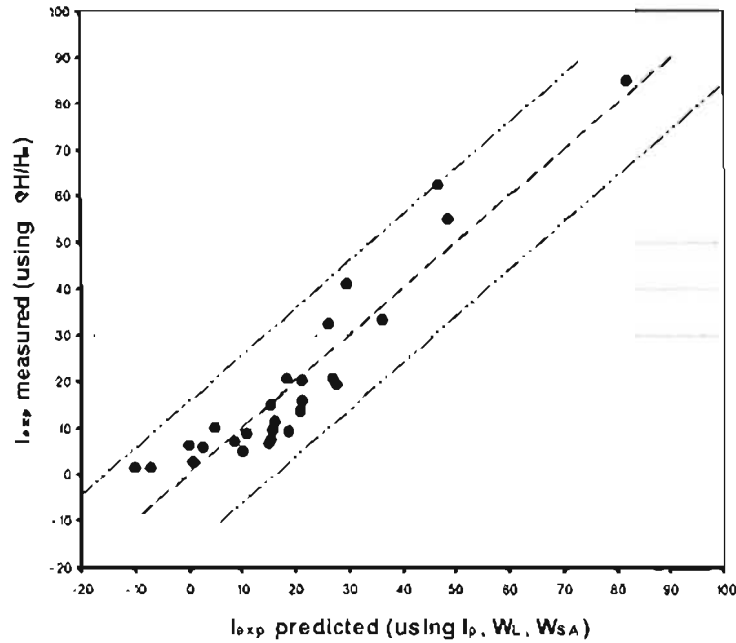


Figure 6.7 Comparison of intrinsic expansiveness predicted using Model 6c and measured using expansive strain.

6.3.2.3 Models requiring plastic and liquid limits and clay fraction

Separate uses of indicator variables such as W_p , W_L and C.F. produced very poor correlations (see Table 5.4). Better correlations were obtained with the combined use of the variables:

$$\Delta e = 2.196 \times 10^{-2} I_p - 0.357 \quad [R^2=67.1 \%, n=27] \quad (\text{Model 7a})$$

$$\Delta e = 2.04 \times 10^{-2} W_L - 1.56 \times 10^{-2} W_p - 0.432 \quad [R^2=68.7\%, n=27] \quad (\text{Model 7b})$$

$$\Delta H/H_0 = 1.273 I_p - 8.848 \text{Activity} - 18.421 \quad [R^2=70.5 \%, n=19] \quad (\text{Model 7c})$$

$$\Delta H/H_0 = 1.332 W_L - 1.555 W_p - 12.734 \text{Activity} - 12.493 \quad [R^2=71.4 \%, n=19] \quad (\text{Model 7d})$$

Comparison of the predicted and measured (using Model 7d) intrinsic expansiveness is showing that the studied samples are distributed in the range ± 20 of $\Delta H/H_0$ % for the line of equality (Figure 6.8). As shown in the figure a reasonable number of the samples lie on the line of equality, only with few samples widely distributed from the line. Additionally, the model is

reasonably significant and cheap. It is thus possible to use the model as a preliminary means of assessing intrinsic expansiveness.

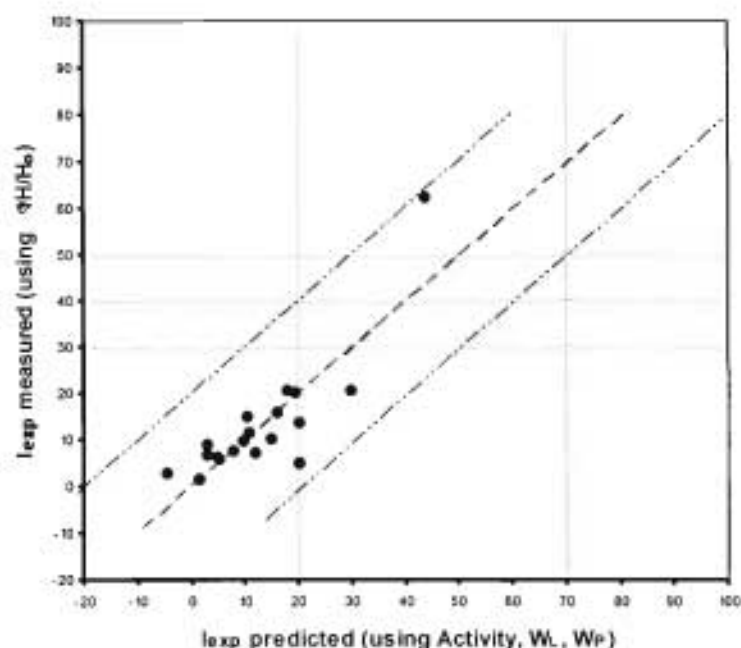


Figure 6.8 Comparison of intrinsic expansiveness predicted using Model 7d and measured using expansive strain.

6.3.2.4 Comparison between the recommended models

The preceding discussion suggested three major models varying in cost and significance that could be used in the analysis of intrinsic expansiveness of soils. From the measured intrinsic expansiveness these models showed a general trend of underestimation of both soils with low and high intrinsic expansiveness. On the contrary, the models gave overestimation for soils with medium intrinsic expansiveness. Actually there is a slight difference between the three models regarding the over and under estimation. In Model 2c, out of the 27 samples used 55.6 % were underestimated and the rest 44.4 % overestimated, showing the skeweness of the samples from the model. In Model 6c the inverse is true; 55.6 % of the samples were overestimating and 44.4 % underestimating. The samples are far better normally distributed from Model 7d than from Model 2c and Model 6c although the samples are most scattered from the model (see Figure 6.9-6.11). Generally, the statistical analysis is showing that intrinsic expansiveness can be reasonably assessed using Models 7d, 6c and/or 2c.

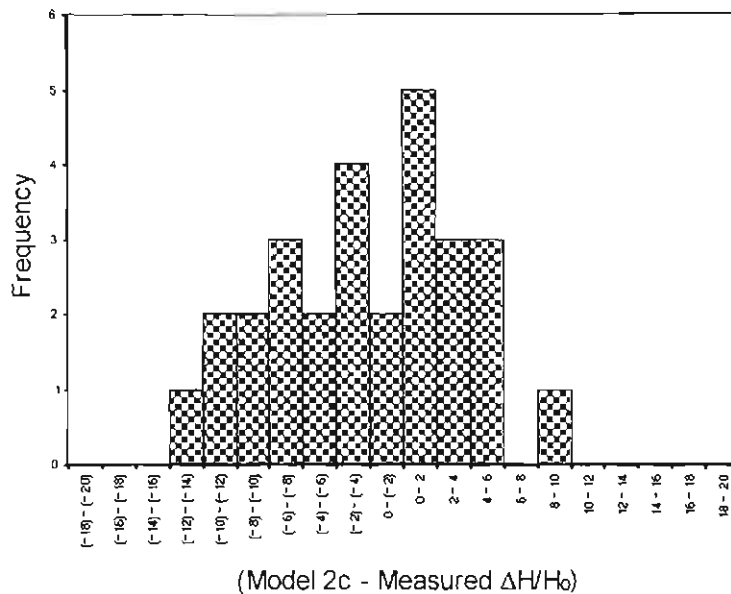


Figure 6.9 The distribution of the samples used with the difference between Model 2c and the measured intrinsic expansiveness.

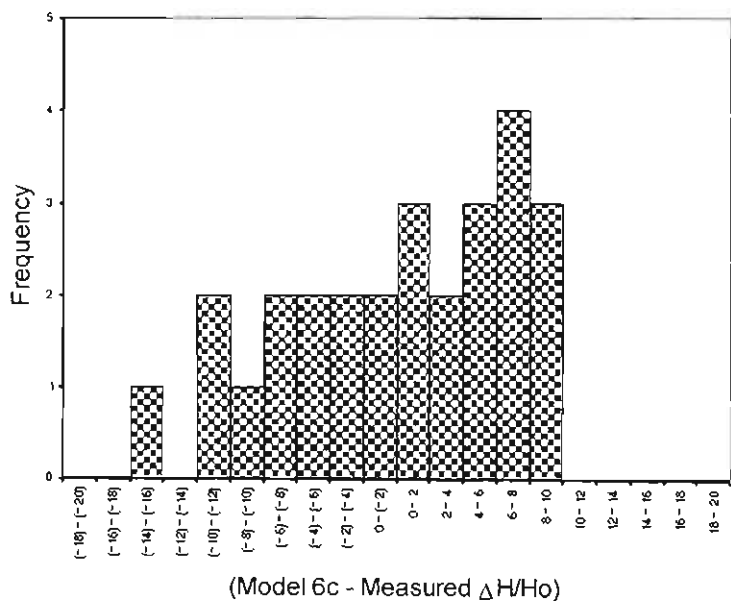


Figure 6.10 The distribution of the samples used with the difference between Model 6c and the measured intrinsic expansiveness.

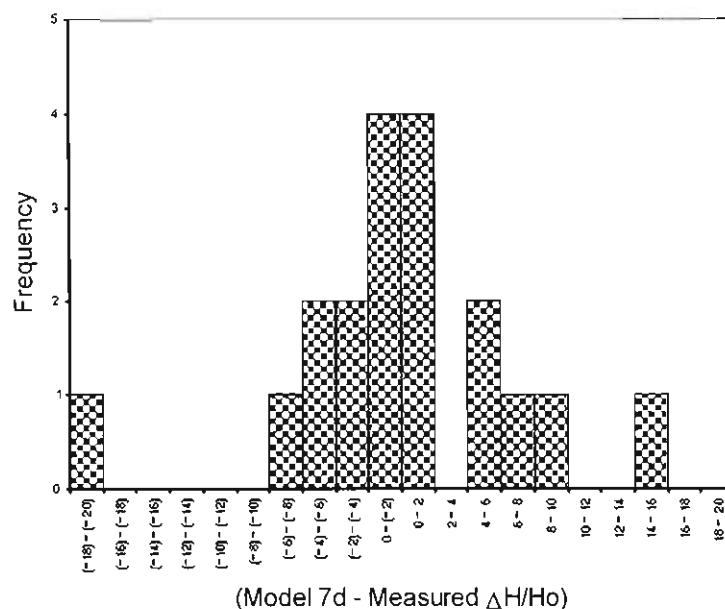


Figure 6.11 The distribution of the samples used with the difference between Model 7d and the measured intrinsic expansiveness.

6.3.2.5 Summary

It has been shown clearly that with increase of the significance of the model to describe expansiveness the cost of the variables included in the model increases. For instance the use of the most significant model, which is Model 2c, could be limited in routine geotechnical site investigation due to the presence of the expensive variable C_s^* . On the other extreme the cheapest but reasonably significant model, which is Model 7d, could be used as a preliminary source of information in the intrinsic expansiveness of soils as all the variables included in the model could easily be obtained from the routine geotechnical site investigation testing. Model 6c is an average model regarding the significance and cost. The model only requires one less test from Model 2c, which is the intrinsic swell index, and one additional test from Model 7d, which is the shrinkage limit. Among others, a reasonable choice of a model requires consideration of both significance and cost of the model. In some cases there might be other dominant facts that dictate the choice of the model. It is thus reasonable to leave the decision for the geotechnical engineer involved in the investigation after considering the specific conditions of the project.

6.4 APPLICATION OF THE MODELS

The preceding discussions suggested three major models that can be used in the assessment of intrinsic expansiveness as early as the geotechnical site investigation stage of a project. The universal application of the models could be partly justified by the absence of highly clustered distribution of the samples originated from the same country and by the random distribution of the samples from the line of equality of the models (Figure 6.12 - 6.14).

Although the final decision on the choice of a model is to be left for the geotechnical engineer, it is still one of the primary objectives of this research to lay a general guideline on how to use the three models. It is the suggestion of the author to use the three recommended models as a screening system toward achieving a reliable assessment of expansiveness. Due to its ease of obtaining Model 7d could be used as initial source of expansiveness information for any project. Depending on the result obtained from the analysis and the sensitivity of the project a decision could be made regarding the need for further analysis. When the decision favors the need for further analysis an additional test of shrinkage limit could be performed to use Model 6c. If still the analysis using this model is not precise enough, an oedometer consolidation test could be performed with the unloading stage to use the most reliable model, which is Model 2c.

Inappropriate usage of developed models is a common phenomenon in geotechnical engineering. It is then important to describe the most appropriate application of the models. In the first place the models are strictly intended for estimating intrinsic expansiveness and comparing soils. That is, to indicate whether a particular soil is potentially expansive or not, from which the engineers would know the need for additional tests or otherwise. Using appropriate input data, which should be obtained from tests performed in the same way as that of the modeling process, the model will derive a value in percentage. This value will then be employed as a guide to the intrinsic expansiveness of the soil. The calculated expansiveness would ranges between 0 and 100 % for the range of soils used although there is no theoretical upper limit, representing very low and

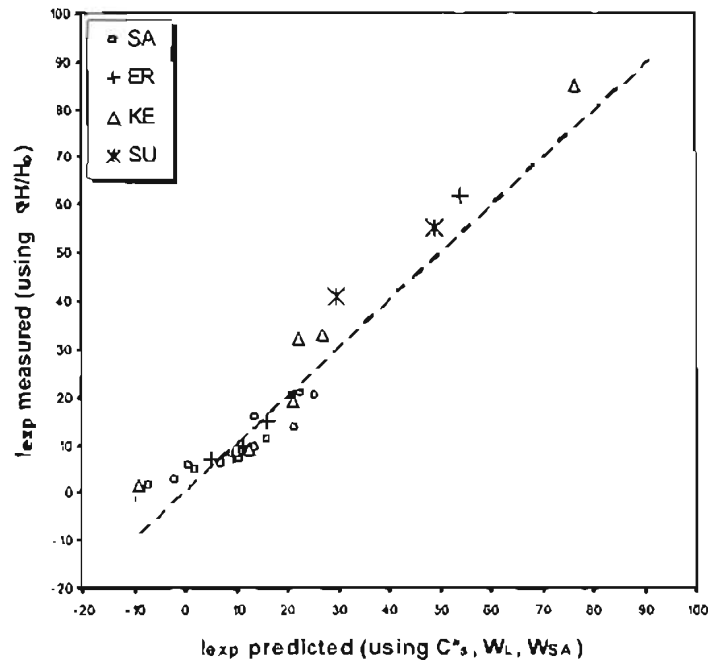


Figure 6.12 Distribution of the samples from the line of equality of the measured and predicted (using Model 2c) intrinsic expansiveness according their origin of country.

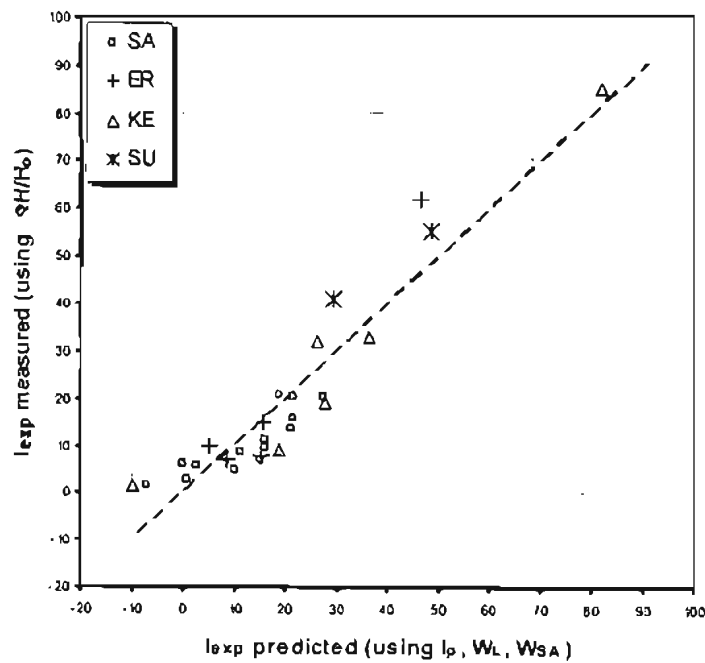


Figure 6.13 Distribution of the samples from the line of equality of the measured and predicted (using Model 6c) intrinsic expansiveness according their origin of country.

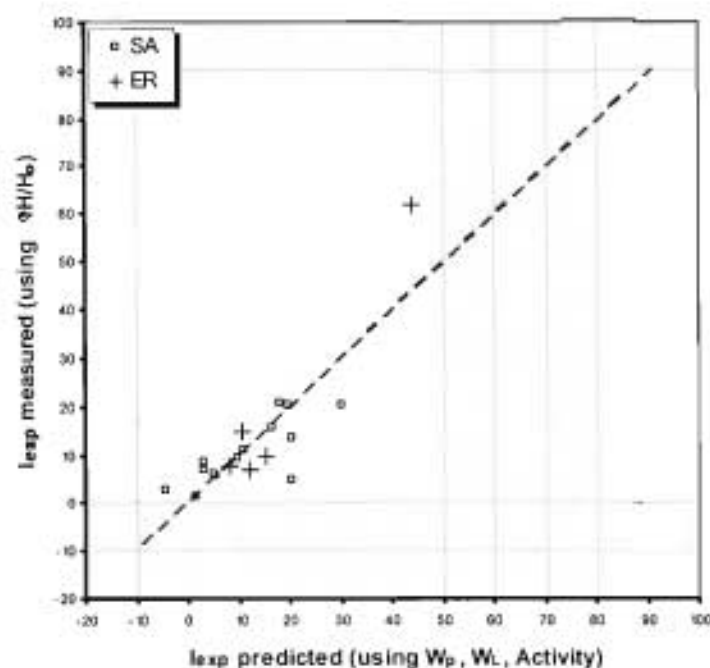


Figure 6.14 Distribution of the samples from the line of equality of the measured and predicted (using Model 7d) intrinsic expansiveness according to their origin of country.

high intrinsic expansiveness respectively. Engineering judgment would be required for the appropriate interpretation of values obtained between the extremes. For consistent use of the models worldwide the engineering judgment has to be replaced, which requires further research from what has been achieved so far. To close this gap for the moment, however, it might be worth recommending a rough guide, which is discussed in the following section.

6.5 CLASSIFICATION OF SOIL'S EXPANSIVENESS

Assessing expansiveness of soils using the models developed may not fully meet the urgent need of materials and geotechnical engineers, and engineering geologists. These engineers anticipate a step forward toward developing a soil expansiveness classification system to evaluate each soil's propensity to be expansive. One of the criteria toward achieving a reliable classification system is finding out a correlation between the field volume change behaviors of the soils tested in the study with the models developed from the laboratory testing results. A good example of such type of attempt is the one made by Jennings and Kerrich (1962). The researchers have

arrived at figures for unit expansiveness (inches expansiveness per inch depth of expansive soil) from double oedometer tests on unloaded soils and their volume change behavior in the field. Unit expansiveness, which is dependent on the thickness of non-expansive overburden, was taken as being the maximum at the upper boundary of the expansive soil layer and decreasing with depth along a parabolic curve. Van der Merwe (1964) has moved the attempt of Jennings and Kerrich (1962) a step forward. He additionally presented a factor to represent the relative decrease of expansiveness with depth and recommended categories of potential expansiveness.

At this stage of this research field volume change observation of the soils has not been performed. But one of the continuations of this research might be achieving that goal. However, due to its urgency by taking the recommendation of van der Merwe (1964) as a base and by including the possible alteration of the volume change behavior of the soils with the use of the reconstituted samples, the following categories are recommended to serve as a rough guide for identification of expansive soils:

- $\Delta H/H_0 > 20\%$ very high intrinsic expansiveness;
- $\Delta H/H_0 15-20\%$ high intrinsic expansiveness;
- $\Delta H/H_0 10-15\%$ medium intrinsic expansiveness;
- $\Delta H/H_0 5-10\%$ low intrinsic expansiveness; and
- $\Delta H/H_0 < 5\%$ non-expansive.

CHAPTER SEVEN

7. CONCLUSION AND RECOMMENDATIONS

For decades soil expansiveness was assessed using methods that fail to satisfy the basic principles of soil mechanics. It was only possible to overcome the previous limitations with the development of a new procedure by Gourley and Schreiner (1993a). These researchers recommended a new procedure that allows comparing intrinsic expansiveness of soils on reconstituted samples prepared into consistent stress history and stable micro-fabric.

Twenty-seven natural samples (nineteen from own research and 8 from Gourley and Schreiner's (1993a) research), which have been prepared in accordance with the Schreiner and Gourley's procedure, went through a series of atterberg limit and volume change tests. The analysis of the results from these tests showed that soil intrinsic expansiveness could be assessed best using a model described by shrinkage limit, plastic limit, liquid limit, and swell index (Model 2c). Unfortunately, the routine use of this model is limited, as it requires shrinkage test and a conventional consolidation test in an oedometer with the one-dimensional unloading stage which is time consuming and expensive to run. The second most reliable model recommended, which is Model 6c, requires only a shrinkage test in addition to the routine liquid and plastic limits tests. The shrinkage test is a relatively cheap test that does not need specialized apparatus. The cheapest of all the models recommended is Model 7c that can easily be obtained from routine tests including liquid limit, plastic limit and hydrometer analysis. Depending on the cost and sensitivity of the project these three models can be used in-series as a screening system to assess expansiveness.

Since deviation of testing techniques from which the models are originally developed might bring a significant difference in the predicted intrinsic expansiveness it is vital to specify the testing procedures. The liquid limit should be based on the BS 1377, part 2, 1990. This standard recommends the use of cone penetration technique. There are a number of researches

done so far that question the equivalence of the results obtained from this method and the Casagrande method. It is the recommendation of the author that when the latter method is used it might be preferable to convert the results using appropriate references before usage in the models recommended. It will be part of the continuation of this study to see the difference between the two methods. The standard also request the use of part of the whole sample, only particles passing 0.425 mm sieve. Therefore, the results obtained from the tests must be adjusted to represent to that of the whole sample using the formula recommended by Gourley and Schreiner (1993a). The shrinkage limit should be performed using reconstituted samples as it was specified in the Schreiner and Gouley's research. The test is easy but it is time consuming. Alternatively, Gourley and Schreiner (1993a) recommended ASTM D 427-83, 1992, or equivalent standards. Nevertheless it is important to mention the need for future research to validate the equality of the results obtained from the tests.

All the models mentioned in this study should not be used in the prediction of actual swell or heave of natural soils as there are a number of other factors than intrinsic expansiveness which needs consideration such as: void ratio, density, vertical stress, change in suction, stress history, soil micro-fabric, etc. They are strictly to be used only for estimating the relative or intrinsic expansiveness of soils and for comparing expansiveness among soils.

Clearly, this study cannot be considered exhaustive, being significant only for the data analyzed. However, it is considered that the methodology followed the basic principles of geotechnical engineering and the models are open for further refinement with addition of more samples, especially samples with very high intrinsic expansiveness. In line with this the need for field volume change observation of the soils is also urgent. The present day foundation design techniques for example some stiffened raft design procedures require the use of some indicator variables such as the I_p . It is thus important to replace these indicator variables with more significant ones which have been identified in this research.

APPENDIX

In this chapter the original data as obtained from the different tests, or that has gone through analysis or calculation are given for further reference. For systematic presentation of these data sets, the chapter presented four tables. Table A.1 gives the laboratory result of oedometer consolidation test of the soil samples from own research. Unconfined shrinkage test result is given in Table A.2. Table A.3, on the other hand, contains the raw and analyzed data of the suction tests. The last table, Table A.4, gives swelling results.

Table A.1 Results of consolidation of the soil samples from South Africa and Eritrea

Stress (KPa)	Reading		Corrected Reading	ΔH (cm)	Ht (cm)	Δe	e	Reading		Corrected Reading	ΔH (cm)	Ht (cm)	Δe	e	
	Inner	Outer						Inner	Outer						
SA-1								SA-2							
0	14	10.4	0.000		18.976			5	15.7	0.000		21.421			
2	14	3.6	0.068	-0.068	18.908	-0.0092	1.0546	4	18.2	0.175	-0.175	21.248	-0.0318	2.2398	
4	13	3.0	0.274	-0.206	18.702	-0.0260	1.0266	2	1.2	0.745	-0.570	20.676	-0.1036	2.1362	
8	11	14.0	0.564	-0.290	18.412	-0.0394	0.9871	23	15.1	1.406	-0.661	20.015	-0.1201	2.0160	
16	9	13.8	0.966	-0.402	18.010	-0.0547	0.9325	18	11.3	2.444	-1.038	18.977	-0.1886	1.8274	
32	7	4.4	1.460	-0.494	17.516	-0.0672	0.8653	11	6.6	3.891	-1.447	17.530	-0.2630	1.5644	
64	4	6.0	2.044	-0.584	18.932	-0.0794	0.7858	2	4.3	5.714	-1.823	16.707	-0.3313	1.2331	
128	1	13.4	2.570	-0.526	16.406	-0.0715	0.7143	19	9.3	7.064	-1.350	14.357	-0.2453	0.9878	
250	24	4.5	3.059	-0.489	15.917	-0.0665	0.6478	13	7.3	8.284	-1.220	13.137	-0.2217	0.7681	
500	21	10.1	3.603	-0.544	15.373	-0.0740	0.5738	8	15.0	9.207	-0.923	12.214	-0.1677	0.5983	
1000	18	10.2	4.402	-0.799	14.574	-0.1087	0.4652	4	16.3	9.794	-0.587	11.627	-0.1067	0.4916	
500	18	7.9	4.225	0.177	14.751	0.0241	0.4892	5	2.5	9.732	0.062	11.689	0.0113	0.5029	
200	18	14.9	4.155	0.070	14.821	0.0095	0.4988	7	2.1	9.536	0.196	11.885	0.0356	0.5385	
100	18	17.8	4.126	0.029	14.850	0.0039	0.5027	7	18.6	9.371	0.165	12.050	0.0300	0.5685	
SA-3								SA-4a							
0	16	3.0	0.000		20.241			7	9.6	0.000		21.239			
2	6	6.8	1.962	-1.962	18.279	-0.3225	1.4636	25	9.2	1.404	-1.404	19.835	-0.2340	1.7749	
4	4	9.3	2.337	-0.375	17.904	-0.0616	1.4020	24	2.8	1.668	-0.264	19.571	-0.0440	1.7309	
8	2	3.8	2.792	-0.455	17.449	-0.0748	1.3272	21	14.4	2.152	-0.484	19.087	-0.0807	1.6503	
16	24	13.2	3.298	-0.506	18.943	-0.0832	1.2440	18	12.2	2.774	-0.622	18.465	-0.1037	1.5466	
32	21	17.2	3.658	-0.580	16.383	-0.0921	1.1520	15	7.4	3.422	-0.648	17.817	-0.1080	1.4386	
64	18	14.0	4.490	-0.632	15.751	-0.1039	1.0481	11	12.4	4.172	-0.750	17.067	-0.1250	1.3136	
128	16	4.6	4.984	-0.494	15.257	-0.0812	0.9669	7	17.7	4.919	-0.747	16.320	-0.1245	1.1891	
250	13	11.7	5.513	-0.529	14.728	-0.0870	0.8799	4	18.2	5.514	-0.595	15.725	-0.0992	1.0900	
500	11	17.8	5.852	-0.339	14.389	-0.0557	0.8242	0	13.3	6.363	-0.849	14.876	-0.1415	0.9485	
1000	10	2.6	6.204	-0.352	14.037	-0.0579	0.7663	22	10.6	6.990	-0.627	14.249	-0.1045	0.8440	
500	10	8.8	6.142	0.062	14.099	0.0102	0.7765	22	10.6	6.990	0.000	14.249	0.0000	0.8440	
200	10	10.5	6.125	0.017	14.118	0.0028	0.7793	22	12.8	6.968	0.022	14.271	0.0037	0.8477	
100	10	18.9	6.041	0.084	14.200	0.0138	0.7931	23	3.7	6.859	0.109	14.380	0.0182	0.8658	

SA-4b								SA-5							
0	18	3.4	0.000		20.110			8	19.7	0.000		20.006			
2	10	7.6	1.558	-1.558	18.552	-0.2787	1.7790	4	8.6	0.931	-0.931	19.075	-0.1642	1.5689	
4	8	15.3	1.881	-0.323	18.229	-0.0578	1.7212	1	19.7	1.400	-0.469	18.806	-0.0827	1.4861	
8	5	13.0	2.504	-0.623	17.606	-0.1114	1.6098	22	15.0	2.247	-0.847	17.759	-0.1494	1.3367	
16	2	3.5	3.199	-0.895	16.911	-0.1243	1.4855	17	18.6	2.811	-0.564	17.195	-0.0995	1.2372	
32	23	17.2	3.862	-0.663	16.248	-0.1186	1.3659	16	18.5	3.412	-0.601	16.594	-0.1060	1.1312	
64	19	17.4	4.660	-0.798	15.450	-0.1427	1.2242	13	8.4	4.113	-0.701	15.893	-0.1237	1.0075	
128	16	11.8	5.318	-0.656	14.794	-0.1173	1.1068	10	5.6	4.741	-0.628	15.265	-0.1108	0.8967	
250	13	12.0	5.914	-0.598	14.196	-0.1070	0.9999	7	1.7	5.380	-0.639	14.826	-0.1127	0.7840	
500	11	2.9	6.405	-0.491	13.705	-0.0878	0.9120	3	16.2	6.035	-0.655	13.971	-0.1156	0.6685	
1000	8	6.8	6.966	-0.561	13.144	-0.1003	0.8117	25	10.4	6.693	-0.658	13.313	-0.1161	0.5524	
500	8	15.8	6.876	0.090	13.234	0.0161	0.8278	25	13.7	6.660	0.033	13.346	0.0058	0.5582	
200	8	16.9	6.865	0.011	13.245	0.0020	0.8298	1	9.9	6.498	0.162	13.508	0.0286	0.5868	
100	8	18.9	6.845	0.020	13.265	0.0036	0.8333	2	4.1	6.356	0.142	13.650	0.0251	0.6118	
SA-6								SA-7							
0	13	15.3	0.000		22.434			16	6.9	0.000		22.060			
2	7	17.0	1.183	-1.183	21.251	-0.1873	1.6930	14	7.8	0.391	-0.391	21.669	-0.0684	2.2198	
4	5	10.0	1.653	-0.470	20.781	-0.0744	1.8185	12	10.4	0.785	-0.374	21.295	-0.0655	2.1541	
8	2	12.6	2.227	-0.574	20.207	-0.0909	1.5277	9	6.0	1.409	-0.644	20.651	-0.1127	2.0414	
16	24	13.7	2.816	-0.589	19.618	-0.0933	1.4344	4	19.1	2.278	-0.869	19.782	-0.1521	1.8893	
32	21	16.6	3.387	-0.571	19.047	-0.0904	1.3440	0	4.7	3.222	-0.944	18.838	-0.1652	1.7240	
64	17	0.0	4.353	-0.966	18.081	-0.1529	1.1911	19	0.5	4.464	-1.242	17.596	-0.2174	1.6068	
128	17	13.4	4.819	-0.466	17.615	-0.0738	1.1173	13	12.8	5.541	-1.077	16.519	-0.1885	1.3181	
250	10	8.8	5.685	-0.866	16.749	-0.1371	0.9802	7	15.1	6.718	-1.177	15.342	-0.2060	1.1121	
500	6	17.8	6.375	-0.890	16.059	-0.1092	0.8709	3	0.9	7.660	-0.942	14.400	-0.1649	0.9472	
1000	3	12.4	7.029	-0.654	15.405	-0.1035	0.7674	22	16.2	8.707	-1.047	13.353	-0.1833	0.7639	
500	3	19.8	6.955	0.074	15.479	0.0117	0.7791	23	9.0	8.579	0.128	13.481	0.0224	0.7863	
200	4	16.2	6.791	0.164	15.643	0.0260	0.8050	0	9.1	8.378	0.201	13.682	0.0352	0.8215	
100	5	13.4	6.619	0.172	15.816	0.0272	0.8323	1	1.4	8.255	0.123	13.805	0.0215	0.8430	
SA-8								SA-9							
0	17	4.4	0.000		19.310			15	2.7	0.000		19.100			
2	10	3.8	1.406	-1.406	17.904	-0.2436	1.4513	13	3.2	0.395	-0.395	18.705	-0.0734	1.5838	
4	8	1.4	1.830	-0.424	17.480	-0.0735	1.3779	12	8.7	0.540	-0.145	18.560	-0.0269	1.5569	
8	6	2.0	2.224	-0.394	17.086	-0.0683	1.3096	11	1.3	0.814	-0.274	18.286	-0.0509	1.5080	
16	3	9.4	2.750	-0.526	16.580	-0.0911	1.2184	9	8.5	1.142	-0.328	17.958	-0.0609	1.4450	
32	1	5.2	3.192	-0.442	16.118	-0.0768	1.1419	7	7.8	1.549	-0.407	17.551	-0.0756	1.3694	
64	23	7.7	3.787	-0.575	15.543	-0.0996	1.0422	4	14.8	2.079	-0.530	17.021	-0.0985	1.2710	
128	20	8.1	4.383	-0.598	14.947	-0.1033	0.9389	1	14.0	2.687	-0.608	16.413	-0.1129	1.1580	
250	17	4.8	4.998	-0.635	14.312	-0.1100	0.8289	23	13.6	3.291	-0.604	15.809	-0.1122	1.0458	
500	14	1.6	5.628	-0.630	13.682	-0.1092	0.7198	20	7.8	3.949	-0.658	15.151	-0.1222	0.9236	
1000	10	7.0	6.374	-0.748	12.938	-0.1293	0.5905	16	16.0	4.667	-0.718	14.433	-0.1334	0.7902	
500	10	19.0	6.254	0.120	13.056	0.0208	0.6113	17	5.4	4.573	0.094	14.527	0.0175	0.8077	
200	11	13.8	6.106	0.148	13.204	0.0256	0.6369	18	1.9	4.408	0.165	14.692	0.0307	0.8383	
100	12	5.9	5.985	0.121	13.325	0.0210	0.6579	18	15.7	4.270	0.138	14.830	0.0256	0.8640	

SA-10								SA-11							
0	15	3.2	0.000		21.002			12	2.0	0.000		20.807			
2	13	8.2	0.350	-0.350	20.652	-0.0613	2.2199	4	4.9	1.575	-1.575	19.232	-0.2855	1.9731	
4	10	13.4	0.898	-0.548	20.104	-0.0960	2.1239	1	11.8	2.106	-0.531	18.701	-0.0963	1.8768	
8	6	15.6	1.676	-0.778	19.326	-0.1362	1.9877	24	9.0	2.712	-0.606	18.095	-0.1099	1.7689	
16	2	10.6	2.526	-0.850	18.478	-0.1488	1.8389	20	10.7	3.495	-0.783	17.312	-0.1419	1.8250	
32	23	3.8	3.596	-1.070	17.406	-0.1873	1.6515	14	19.8	4.204	-0.709	16.603	-0.1285	1.4965	
64	17	3.7	4.595	-0.999	16.407	-0.1749	1.4766	8	19.4	5.208	-1.004	15.599	-0.1820	1.3144	
128	11	2.8	5.804	-1.209	15.198	-0.2117	1.2649	5	1.2	6.190	-0.982	14.617	-0.1780	1.1364	
250	5	6.6	6.966	-1.162	14.036	-0.2035	1.0615	0	6.5	7.137	-0.947	13.670	-0.1717	0.9648	
500	0	5.0	7.982	-1.016	13.020	-0.1779	0.8836	21	9.7	7.905	-0.788	12.902	-0.1392	0.8255	
1000	20	0.1	8.831	-0.849	12.171	-0.1487	0.7349	17	13.0	8.672	-0.767	12.135	-0.1390	0.6865	
500	21	7.2	8.760	0.071	12.242	0.0124	0.7474	18	3.5	8.567	0.105	12.240	0.0190	0.7055	
200	23	10.6	8.526	0.234	12.476	0.0410	0.7883	19	3.7	8.365	0.202	12.442	0.0366	0.7422	
100	24	4.0	8.392	0.134	12.610	0.0235	0.8118	20	2.5	8.177	0.188	12.630	0.0341	0.7762	
SA-12								SA-13							
0	14	1.8	0.000		18.470			15	18.0	0.000		20.606			
2	10	18.8	0.630	-0.630	17.840	-0.0893	1.1421	11	6.2	0.954	-0.954	19.652	-0.1558	1.5560	
4	9	19.4	0.824	-0.194	17.646	-0.0275	1.1146	10	0.5	1.211	-0.257	19.395	-0.0419	1.5141	
8	8	7.8	1.140	-0.316	17.330	-0.0448	1.0698	7	2.5	1.791	-0.580	18.815	-0.0946	1.4194	
16	6	1.3	1.605	-0.465	16.865	-0.0659	1.0039	2	16.7	2.649	-0.858	17.957	-0.1400	1.2795	
32	3	4.8	2.170	-0.565	16.300	-0.0801	0.9237	22	1.4	3.404	-0.755	17.202	-0.1232	1.1563	
64	24	17.4	2.844	-0.674	15.826	-0.0956	0.8281	17	5.9	4.359	-0.955	16.247	-0.1558	1.0005	
128	21	6.0	3.558	-0.714	14.912	-0.1013	0.7269	12	8.4	5.354	-0.965	15.252	-0.1623	0.8382	
250	17	19.7	4.221	-0.663	14.249	-0.0940	0.6328	7	15.5	6.263	-0.909	14.343	-0.1483	0.6899	
500	15	6.6	4.752	-0.531	13.718	-0.0753	0.5575	4	9.1	6.927	-0.664	13.679	-0.1083	0.5816	
1000	13	8.2	5.136	-0.384	13.334	-0.0545	0.5031	1	7.2	7.546	-0.619	13.060	-0.1010	0.4806	
500	13	4.6	4.972	0.164	13.498	0.0233	0.5263	1	10.5	7.513	0.033	13.093	0.0054	0.4860	
200	12	16.8	4.850	0.122	13.620	0.0173	0.5436	2	1.5	7.403	0.110	13.203	0.0179	0.5039	
100	12	13.8	4.880	-0.030	13.590	-0.0043	0.5394	2	15.2	7.266	0.137	13.340	0.0223	0.5263	
SA-14								ER-1							
0	6	0.4	0.000		17.675			14	0.6	0.000		17.973			
2	4	2.7	0.377	-0.377	17.298	-0.0796	2.2317	11	9.7	0.509	-0.509	17.464	-0.1215	2.5288	
4	3	1.8	0.586	-0.209	17.089	-0.0441	2.1875	9	10.8	0.898	-0.389	17.075	-0.0928	2.4360	
8	1	5.8	0.946	-0.360	16.729	-0.0760	2.1115	8	10.8	1.498	-0.600	16.475	-0.1432	2.2928	
16	23	3.8	1.566	-0.620	16.109	-0.1309	1.9807	2	9.8	2.308	-0.810	15.665	-0.1933	2.0994	
32	20	18.4	2.020	-0.454	15.655	-0.0958	1.8848	23	9.8	3.108	-0.800	14.865	-0.1909	1.9085	
64	17	6.4	2.740	-0.720	14.935	-0.1520	1.7328	19	8.0	3.926	-0.818	14.047	-0.1952	1.7133	
128	13	3.7	3.567	-0.827	14.108	-0.1748	1.5582	14	6.6	4.940	-1.014	13.033	-0.2420	1.4713	
250	8	11.3	4.491	-0.924	13.184	-0.1951	1.3632	9	12.0	5.886	-0.946	12.067	-0.2258	1.2455	
500	4	0.0	5.404	-0.913	12.271	-0.1927	1.1704	5	8.3	6.723	-0.837	11.250	-0.1998	1.0458	
1000	24	4.0	6.164	-0.760	11.511	-0.1804	1.0100	1	3.8	7.568	-0.845	10.405	-0.2017	0.8441	
500	0	1.6	5.968	0.176	11.687	0.0372	1.0472	2	1.9	7.387	0.181	10.586	0.0432	0.8873	
200	1	9.3	5.911	0.077	11.764	0.0163	1.0634	3	11.0	7.096	0.291	10.877	0.0695	0.9567	
100	1	15.4	5.850	0.061	11.825	0.0129	1.0763	4	5.8	6.948	0.148	11.025	0.0353	0.9921	

ER-2								ER-3							
0	18	9.2	0.000		21.092			12	15.4	0.000		21.722			
2	11	17.6	1.316	-1.318	19.776	-0.1971	1.4741	6	12.3	1.231	-1.231	20.491	-0.1923	1.7171	
4	10	17.0	1.522	-0.206	19.570	-0.0308	1.4432	4	14.1	1.813	-0.382	20.109	-0.0597	1.6574	
8	8	19.0	1.902	-0.380	19.190	-0.0569	1.3863	2	4.4	2.110	-0.497	19.612	-0.0777	1.5798	
16	6	16.4	2.328	-0.426	18.764	-0.0838	1.3225	23	10.8	2.848	-0.738	18.874	-0.1153	1.4845	
32	3	10.3	2.989	-0.861	18.103	-0.0990	1.2235	17	19.6	3.958	-1.110	17.764	-0.1734	1.2910	
64	0	5.2	3.640	-0.651	17.452	-0.0975	1.1280	13	13.1	4.823	-0.866	16.899	-0.1352	1.1559	
128	21	1.8	4.474	-0.834	16.618	-0.1249	1.0010	9	1.8	5.736	-0.913	15.988	-0.1427	1.0132	
250	17	1.2	5.280	-0.806	16.812	-0.1207	0.8803	4	0.2	6.752	-1.016	14.970	-0.1588	0.8545	
500	12	5.8	6.234	-0.954	14.858	-0.1429	0.7374	24	7.8	7.876	-0.924	14.048	-0.1444	0.7101	
1000	7	7.2	7.220	-0.986	13.872	-0.1477	0.5897	20	2.8	8.526	-0.850	13.198	-0.1328	0.5773	
500	7	17.2	7.120	0.100	13.972	0.0150	0.6047	20	14.4	8.410	0.118	13.312	0.0181	0.5854	
200	8	16.2	6.930	0.190	14.162	0.0285	0.6332	21	15.0	8.204	0.206	13.518	0.0322	0.6276	
100	9	12.0	6.772	0.158	14.320	0.0237	0.6568	22	9.2	8.062	0.142	13.660	0.0222	0.6498	
ER-4								ER-5							
0	13	14.6	0.000		20.099			18	3.8	0.000		18.663			
2	11	14.3	0.403	-0.403	19.696	-0.0590	1.3526	14	5.8	0.780	-0.780	17.883	-0.1298	1.4184	
4	10	6.2	0.600	-0.197	19.499	-0.0288	1.3238	13	10.5	0.933	-0.153	17.730	-0.0255	1.3930	
8	8	0.0	1.140	-0.540	18.959	-0.0790	1.2447	12	11.8	1.120	-0.187	17.543	-0.0311	1.3618	
16	4	5.6	1.890	-0.750	18.209	-0.1098	1.1350	11	1.0	1.428	-0.308	17.235	-0.0512	1.3106	
32	24	17.0	2.778	-0.886	17.323	-0.1297	1.0053	8	10.1	1.937	-0.509	16.726	-0.0847	1.2259	
64	21	9.5	3.451	-0.875	16.648	-0.0988	0.9065	6	8.2	2.356	-0.419	16.307	-0.0697	1.1562	
128	17	9.1	4.255	-0.804	15.844	-0.1177	0.7888	3	11.4	2.800	-0.544	15.783	-0.0905	1.0657	
250	13	3.0	5.116	-0.861	14.983	-0.1260	0.8628	0	10.5	3.533	-0.633	15.130	-0.1053	0.9803	
500	9	8.4	5.862	-0.746	14.237	-0.1092	0.5536	22	0.7	4.231	-0.698	14.432	-0.1161	0.8442	
1000	5	15.4	8.592	-0.730	13.507	-0.1069	0.4467	18	1.8	5.020	-0.789	13.643	-0.1313	0.7129	
500	6	2.2	6.524	0.068	13.575	0.0100	0.4567	18	9.0	4.948	0.072	13.715	0.0120	0.7249	
200	6	16.3	6.383	0.141	13.716	0.0206	0.4773	18	18.0	4.858	0.090	13.805	0.0150	0.7399	
100	7	7.2	6.274	0.109	13.825	0.0160	0.4933	19	7.0	4.768	0.090	13.895	0.0150	0.7549	

Table A.2 Results of shrinkage limit test of the soil samples from South Africa and Eritrea

SA-1											
Mass (g)	w (%)	Height (mm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e	
		Ht-1	Ht-2	Ave	φ ₁	φ ₂	Ave.				
159.1687	19.6271	14.80	14.90	14.85	80.04	80.04	80.04	74718.9	2130	0.5027	
158.1706	18.8770	14.74	14.77	14.75	79.94	79.90	79.92	73993.4	2138	0.4881	
156.2816	17.4573	14.84	14.71	14.67	79.72	79.66	79.69	73169.1	2136	0.4715	
154.8787	16.4029	14.59	14.88	14.63	79.50	79.50	79.50	72622.0	2133	0.4605	
153.0015	14.9920	14.58	14.68	14.63	79.30	79.30	79.30	72232.4	2118	0.4527	
151.4403	13.8187	14.58	14.67	14.62	79.10	79.00	79.05	71753.2	2111	0.4430	
149.5014	12.3614	14.56	14.62	14.59	78.82	78.86	78.84	71213.8	2099	0.4322	
148.1732	9.8601	14.55	14.59	14.57	78.64	78.62	78.63	70739.6	2066	0.4227	
143.8641	7.9743	14.53	14.59	14.58	78.60	78.54	78.57	70569.3	2036	0.4192	
141.8938	6.6436	14.53	14.59	14.56	78.60	78.52	78.56	70551.3	2011	0.4189	
139.1861	4.5937	14.53	14.58	14.55	78.60	78.46	78.53	70473.2	1975	0.4173	
137.2876	3.1819	14.53	14.55	14.54	78.60	78.46	78.53	70400.6	1950	0.4158	
135.4196	1.7779	14.53	14.54	14.53	78.60	78.46	78.53	70376.3	1924	0.4154	

SA-2										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ ₁	φ ₂	Ave.			
121.9164	20.9031	12.04	12.06	12.050	79.46	79.48	79.47	59770.0	2040	0.5685
120.8414	19.8370	11.86	11.72	11.79	79.42	79.42	79.42	58406.8	2069	0.5327
119.2243	18.2334	11.52	11.53	11.53	79.18	79.14	79.16	56720.8	2102	0.4885
117.1005	16.1272	11.37	11.41	11.39	78.34	78.34	78.34	54901.1	2133	0.4407
115.2561	14.2982	11.29	11.18	11.24	77.94	77.70	77.82	53437.4	2157	0.4023
113.6601	12.7154	11.21	11.15	11.18	77.14	76.80	76.97	52020.5	2185	0.3652
111.1947	10.2705	11.08	11.11	11.10	76.12	76.00	76.06	50411.5	2208	0.3229
108.4640	7.5625	11.04	11.11	11.08	75.06	75.00	75.03	48967.0	2215	0.2850
106.7961	5.9085	10.94	11.11	11.02	75.00	74.86	74.83	48475.4	2203	0.2721
105.0251	4.1522	10.94	11.09	11.02	74.74	74.40	74.57	48106.4	2183	0.2624
102.9579	2.1022	10.94	11.00	10.97	74.74	74.40	74.57	47909.8	2149	0.2573
SA-3										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ ₁	φ ₂	Ave.			
139.4328	28.9126	14.27	14.13	14.200	79.82	79.88	79.85	71109.6	1961	0.7931
138.6178	28.1591	14.10	14.07	14.085	79.72	79.78	79.74	70339.5	1971	0.7737
138.8189	26.4959	14.02	14.03	14.025	79.48	79.58	79.53	69671.4	1964	0.7588
135.2559	25.0509	14.01	14.01	14.010	79.34	79.44	79.39	69352.1	1950	0.7488
133.3039	23.2462	14.01	14.01	14.010	79.12	79.14	79.13	68898.6	1935	0.7373
130.4046	20.5656	14.01	14.00	14.005	78.84	78.80	78.82	68335.4	1908	0.7231
127.8640	18.2167	13.98	13.99	13.985	78.56	78.68	78.62	67892.0	1883	0.7120
125.2828	15.8302	13.96	13.95	13.955	78.44	78.68	78.56	67643.0	1852	0.7057
123.2333	13.9354	13.96	13.88	13.920	78.44	78.68	78.56	67473.3	1826	0.7014
120.6055	11.5058	13.94	13.88	13.910	78.44	78.58	78.51	67339.0	1791	0.6980
117.6373	8.7616	13.92	13.86	13.890	78.44	78.50	78.47	67173.7	1751	0.6939
115.3160	6.6154	13.92	13.86	13.890	78.34	78.48	78.41	67071.0	1719	0.6913
112.9018	4.3834	13.91	13.82	13.865	78.32	78.38	78.35	66847.9	1689	0.6856
SA-4a										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ ₁	φ ₂	Ave.			
139.3955	31.3621	14.39	14.38	14.38	80.02	80.01	80.02	72308.9	1928	0.8858
137.8792	29.9331	14.27	14.19	14.23	79.62	79.62	79.62	70825.0	1947	0.8275
136.4984	28.6319	14.23	14.17	14.20	78.84	78.84	78.84	69297.7	1970	0.7881
134.2845	26.5456	14.18	14.11	14.14	78.26	78.10	78.18	67878.2	1978	0.7515
132.1705	24.5534	14.13	14.07	14.10	77.48	77.36	77.42	66353.1	1992	0.7122
129.5717	22.1044	13.94	13.99	13.96	76.48	76.40	76.44	64084.4	2023	0.6531
127.2738	19.9389	13.88	14.07	13.97	78.14	76.00	76.07	63491.1	2005	0.6383
125.3361	18.1129	13.85	14.01	13.93	76.04	75.88	75.96	63103.7	1988	0.6283
123.2095	16.1089	13.85	14.00	13.92	76.00	75.86	75.93	63031.2	1955	0.6264
121.3091	14.3180	13.85	13.98	13.91	75.94	75.82	75.88	62903.0	1929	0.6231
118.8970	12.0449	13.84	13.96	13.90	75.92	75.80	75.86	62802.1	1893	0.6205
115.8990	9.2197	13.84	13.96	13.90	75.92	75.80	75.86	62802.1	1845	0.6205
111.5410	5.1128	13.84	13.94	13.89	75.84	75.76	75.80	62657.7	1780	0.6168

SA-4b										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume	γ	e
		Ht-1	Ht-2	Ave	ϕ_1	ϕ_2	Ave.			
128.8008	30.3273	13.33	13.20	13.27	79.68	79.68	79.68	66144.8	1947	0.8333
127.6355	29.1482	13.27	13.14	13.21	79.34	79.34	79.34	65284.9	1955	0.8095
126.8683	28.1695	13.25	13.13	13.19	78.71	78.71	78.71	64179.2	1974	0.7789
124.5179	25.9937	13.25	13.10	13.18	77.46	78.00	77.73	62519.8	1992	0.7329
122.3875	23.8380	13.16	13.01	13.09	76.98	77.04	77.01	60947.8	2008	0.6893
120.0174	21.4398	13.09	12.94	13.02	76.06	76.24	76.15	59275.3	2025	0.6429
117.7359	19.1313	13.05	12.92	12.99	75.92	76.00	75.96	58843.9	2001	0.6310
115.0796	16.4435	12.99	12.89	12.94	75.86	75.90	75.88	58516.6	1967	0.6219
112.1064	13.4351	12.93	12.89	12.91	75.80	75.82	75.81	58273.2	1924	0.6152
109.4187	10.7155	12.90	12.87	12.89	75.74	75.80	75.77	58099.0	1883	0.6103
106.9514	8.2190	12.88	12.87	12.88	75.70	75.80	75.75	58023.3	1843	0.6082
103.3078	4.5322	12.87	12.83	12.85	75.62	75.68	75.65	57757.8	1789	0.6009
SA-5										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	ϕ_1	ϕ_2	Ave.			
133.0183	23.6181	13.78	13.53	13.65	79.60	79.56	79.58	67893.8	1959	0.6118
131.3745	22.0885	13.58	13.47	13.52	79.24	79.24	79.24	66673.8	1970	0.5829
130.2322	21.0289	13.52	13.46	13.49	78.76	78.76	78.76	65698.0	1982	0.5597
129.2547	20.1185	13.44	13.39	13.41	78.04	78.60	78.32	64604.7	2001	0.5337
127.7637	18.7329	13.34	13.34	13.34	77.50	77.76	77.63	63116.4	2024	0.4984
125.9670	17.0632	13.22	13.24	13.23	76.68	76.90	76.79	61248.4	2057	0.4541
123.9705	15.2078	13.16	13.15	13.15	76.16	76.44	76.30	60126.3	2062	0.4274
121.2313	12.6822	13.16	13.06	13.11	75.90	76.20	76.05	59528.6	2037	0.4132
118.9040	10.4994	13.16	13.00	13.08	75.82	75.80	75.81	59018.0	2015	0.4011
114.6752	6.5895	13.12	12.96	13.04	75.62	75.74	75.68	58635.8	1956	0.3920
112.3207	4.3814	13.09	12.96	13.02	75.50	75.68	75.59	58429.1	1922	0.3871
110.4326	2.6268	13.00	12.94	12.97	75.40	75.60	75.50	58043.9	1903	0.3780
SA-6										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	ϕ_1	ϕ_2	Ave.			
149.6380	30.4274	15.43	16.21	15.82	79.68	79.68	79.68	78860.2	1898	0.8323
147.5369	28.5960	15.19	15.93	15.56	79.28	79.28	79.28	76786.9	1921	0.7841
145.9958	27.2527	15.08	15.83	15.45	78.70	78.70	78.70	75158.7	1943	0.7462
144.7347	26.1535	14.96	15.71	15.33	78.34	78.34	78.34	73892.3	1959	0.7168
142.0478	23.8116	14.92	15.59	15.25	77.38	77.38	77.38	71716.2	1981	0.6663
139.5682	21.6503	14.84	15.48	15.16	76.74	76.80	76.77	70150.2	1990	0.6299
137.9035	20.1993	14.66	15.39	15.02	76.20	76.18	76.19	68478.7	2014	0.5911
135.2628	17.8977	14.54	15.33	14.93	75.46	75.44	75.45	66752.8	2026	0.5510
132.7718	15.7265	14.43	15.26	14.84	75.00	75.20	75.10	65736.1	2020	0.5273
130.8834	13.9062	14.39	15.18	14.78	74.74	74.66	74.70	64774.7	2018	0.5050
128.8826	10.5933	14.38	15.04	14.71	74.48	74.46	74.47	64049.8	1981	0.4882
124.1279	8.1923	14.38	15.01	14.69	74.42	74.32	74.37	63812.7	1945	0.4827
120.7310	5.2315	14.33	15.01	14.67	74.24	74.12	74.18	63379.0	1905	0.4728

SA-7										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
132.5706	30.0010	13.78	13.84	13.81	79.88	79.88	79.88	69183.5	1916	0.8430
131.8041	29.2494	13.53	13.58	13.55	79.78	79.80	79.79	67752.6	1945	0.8049
130.2169	27.6929	13.30	13.28	13.29	79.58	79.66	79.62	66144.8	1989	0.7621
129.0802	26.5783	13.18	13.17	13.16	79.28	79.20	79.24	64898.5	1989	0.7289
127.1884	24.7231	12.98	13.02	13.00	78.80	78.80	78.80	63387.3	2007	0.6888
125.6791	23.2431	12.95	12.92	12.93	77.86	77.80	77.83	61515.2	2043	0.6387
123.5178	21.1237	12.88	12.77	12.82	77.00	76.80	76.90	59543.0	2074	0.5862
121.8032	19.4423	12.70	12.65	12.67	76.40	76.28	76.34	57992.4	2100	0.5449
119.4701	17.1544	12.51	12.41	12.46	75.40	75.34	75.37	55588.8	2150	0.4803
117.3618	15.0870	12.33	12.27	12.30	74.64	74.60	74.62	53768.6	2183	0.4324
115.1354	12.9037	12.18	12.13	12.15	74.00	73.94	73.97	52212.8	2205	0.3909
110.5851	8.4416	12.06	12.11	12.08	73.66	73.64	73.65	51463.9	2149	0.3710
108.7825	4.7127	12.03	12.10	12.08	73.66	73.58	73.62	51336.8	2080	0.3676
SA-8										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
132.3417	24.4359	13.16	13.49	13.33	80.01	80.02	80.02	67003.9	1975	0.6579
131.3922	23.5431	13.09	13.49	13.29	79.58	79.58	79.58	66103.2	1988	0.6356
129.7986	22.0447	13.04	13.18	13.11	79.10	79.10	79.10	64423.7	2015	0.5941
127.7392	20.1084	13.03	12.92	12.98	78.58	78.22	78.40	62638.8	2039	0.5498
125.2431	17.7614	12.98	12.86	12.93	77.46	77.24	77.35	60758.8	2061	0.5034
123.3053	15.9393	12.90	12.75	12.83	77.00	76.58	76.79	59395.9	2076	0.4697
121.1547	13.9172	12.88	12.67	12.78	76.40	76.10	76.25	58335.2	2077	0.4434
118.6916	11.6012	12.87	12.66	12.77	75.80	76.00	75.90	57755.6	2055	0.4291
116.6542	9.6855	12.73	12.61	12.67	75.86	75.62	75.74	57084.4	2044	0.4125
112.8975	6.1533	12.72	12.59	12.66	75.62	75.44	75.53	56701.0	1991	0.4030
111.2114	4.5679	12.72	12.59	12.66	75.62	75.38	75.49	56641.0	1963	0.4015
110.1205	3.5422	12.67	12.59	12.63	75.60	75.30	75.45	56469.2	1950	0.3972
SA-9										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
136.6918	33.8643	14.98	14.70	14.83	80.00	80.02	80.01	74562.3	1832	0.8640
135.5998	32.6935	14.89	14.56	14.73	80.00	79.86	79.93	73886.5	1835	0.8471
134.3843	31.4845	14.66	14.50	14.58	79.82	79.42	79.62	72592.5	1851	0.8147
132.6190	29.7766	14.55	14.41	14.48	79.10	78.78	78.94	70868.4	1871	0.7716
130.3065	27.5137	14.53	14.34	14.44	78.20	77.86	78.03	69028.7	1888	0.7256
128.3756	25.6242	14.53	14.23	14.38	77.46	77.32	77.39	67642.3	1898	0.6910
125.8765	23.1786	14.48	14.21	14.35	77.00	76.68	76.84	66522.0	1892	0.6630
123.7328	21.0809	14.37	14.14	14.26	76.76	76.34	76.55	65806.6	1886	0.6401
120.9044	18.3131	14.31	14.11	14.21	76.48	76.06	76.27	64921.9	1882	0.6230
118.0018	13.5156	14.30	14.11	14.21	76.14	75.66	75.90	64270.9	1805	0.6067
113.7423	11.3045	14.26	14.04	14.15	76.00	75.80	75.80	63853.5	1781	0.5963
111.5198	9.1296	14.22	13.98	14.10	75.82	75.46	75.64	63359.5	1760	0.5839
109.0379	6.7009	14.21	13.94	14.08	75.66	75.30	75.48	62979.9	1731	0.5744
105.0388	2.7875	14.12	13.89	14.01	75.54	75.18	75.36	62467.6	1681	0.5816

SA-10										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
126.5536	28.5651	12.52	12.70	12.61	80.00	80.00	80.00	63384.8	1997	0.8118
125.5256	27.5208	12.41	12.57	12.49	79.96	79.98	79.97	62734.5	2001	0.7932
124.4244	26.4021	12.25	12.34	12.30	79.88	79.88	79.88	61616.1	2019	0.7613
122.8834	24.8368	12.06	12.17	12.12	79.46	79.36	79.41	60001.7	2048	0.7151
120.9256	22.8477	11.88	11.96	11.92	78.56	78.60	78.58	57808.3	2092	0.6524
118.9837	20.8546	11.72	11.81	11.77	77.94	78.00	77.97	56174.2	2118	0.6057
116.8485	18.7058	11.62	11.69	11.66	77.08	77.28	77.18	54527.0	2143	0.5586
115.3225	17.1555	11.60	11.69	11.65	76.68	76.80	76.74	53860.8	2141	0.5396
111.9452	13.7245	11.58	11.63	11.61	76.36	76.44	76.40	53201.2	2104	0.5207
105.7884	7.4699	11.49	11.59	11.54	76.28	76.44	76.36	52847.9	2002	0.5106
103.2574	4.8986	11.49	11.57	11.53	76.28	76.44	76.36	52802.1	1956	0.5093
100.5522	2.1504	11.49	11.55	11.52	76.28	76.44	76.36	52756.3	1906	0.5080
SA-11										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
125.2569	28.0731	12.63	12.63	12.63	80.01	80.02	80.02	63509.1	1972	0.7762
124.5947	27.3960	12.54	12.57	12.56	79.94	79.92	79.93	62997.9	1978	0.7619
123.2020	25.9720	12.48	12.49	12.49	79.26	79.26	79.26	61600.8	2000	0.7229
121.6810	24.4168	12.44	12.37	12.41	78.64	78.64	78.64	60252.3	2020	0.6851
119.1882	21.8680	12.33	12.24	12.29	77.50	77.46	77.48	57922.1	2058	0.6200
117.4734	20.1146	12.31	12.22	12.27	76.68	76.58	76.63	56565.9	2077	0.5820
115.3129	17.9055	12.07	12.14	12.11	76.00	75.82	75.91	54783.9	2105	0.5322
112.6196	15.1517	11.93	11.96	11.95	75.24	75.04	75.14	52968.6	2126	0.4814
108.9983	11.4490	11.86	11.88	11.87	74.30	74.12	74.21	51341.1	2123	0.4359
106.9112	9.3149	11.86	11.82	11.84	74.18	74.06	74.12	51087.2	2093	0.4288
105.2487	7.6150	11.77	11.80	11.79	74.10	74.00	74.05	50753.9	2074	0.4195
103.8744	6.2098	11.76	11.79	11.78	74.10	73.90	74.00	50642.4	2051	0.4164
100.7585	3.0239	11.75	11.79	11.77	74.02	73.82	73.92	50511.5	1995	0.4127
SA-12										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
148.0064	18.4781	13.58	13.52	13.550	80.04	80.03	80.04	68189.3	2142	0.5348
145.0499	17.7020	13.52	13.42	13.470	79.90	79.92	79.91	67555.3	2147	0.5210
143.7816	16.6728	13.43	13.34	13.385	79.84	79.84	79.84	67011.5	2146	0.5088
142.2732	15.4488	13.41	13.28	13.345	79.48	79.34	79.40	66078.9	2153	0.4877
140.9455	14.3714	13.39	13.21	13.300	78.84	79.14	78.99	65175.7	2163	0.4674
139.0482	12.8318	13.37	13.21	13.290	78.30	78.48	78.39	64141.1	2168	0.4442
137.3809	11.4789	13.36	13.21	13.285	77.98	78.14	78.06	63578.2	2161	0.4315
135.2356	9.7381	13.34	13.21	13.275	77.70	77.82	77.76	63043.0	2145	0.4194
132.2293	7.2986	13.33	13.21	13.270	77.38	77.48	77.43	62485.5	2116	0.4069
128.9367	4.6268	13.33	13.21	13.270	77.20	77.38	77.29	62259.8	2071	0.4018
126.3156	2.4099	13.33	13.21	13.270	77.20	77.32	77.26	62211.4	2030	0.4007

SA-13										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
137.6069	21.2386	13.29	13.39	13.340	80.02	80.00	80.01	67070.9	2052	0.5655
135.9484	19.7774	13.11	13.29	13.200	79.70	79.82	79.76	65952.9	2061	0.5394
134.5275	18.5255	13.02	13.14	13.080	79.22	79.24	79.23	64487.7	2086	0.5052
132.6288	18.8526	12.90	13.00	12.950	78.44	78.16	78.30	62358.7	2127	0.4555
130.0944	14.6197	12.80	12.95	12.875	77.24	77.12	77.18	60234.7	2160	0.4060
128.7436	13.4296	12.71	12.89	12.800	76.62	76.58	76.60	58987.1	2183	0.3769
126.8441	11.7560	12.57	12.70	12.635	75.90	75.94	75.92	57197.6	2218	0.3351
124.7429	9.9048	12.57	12.69	12.630	75.22	75.20	75.21	56110.5	2223	0.3097
122.2224	7.8841	12.56	12.65	12.605	74.76	74.66	74.71	55257.4	2212	0.2898
120.0463	5.7668	12.55	12.58	12.565	74.58	74.60	74.59	54905.2	2186	0.2816
117.1269	3.1947	12.53	12.55	12.540	74.50	74.54	74.52	54693.2	2142	0.2756
SA-14										
Mass (g)	w (%)	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
111.6852	37.6966	11.90	11.75	11.83	80.01	80.02	80.02	59461.2	1878	1.0763
110.8395	36.6540	11.68	11.67	11.68	80.00	80.00	80.00	58685.0	1889	1.0492
109.7268	35.2821	11.64	11.61	11.63	79.82	79.56	79.69	57981.6	1892	1.0246
108.4543	33.7133	11.49	11.47	11.48	79.32	79.30	79.31	56713.7	1912	0.9803
106.2528	30.9990	11.30	11.38	11.34	79.00	78.64	78.92	55472.4	1915	0.9370
104.0957	28.3396	11.26	11.30	11.28	78.48	78.44	78.46	54537.5	1909	0.9044
102.9736	26.9561	11.24	11.30	11.27	78.44	78.34	78.39	54392.0	1893	0.8993
99.4369	22.5957	11.21	11.26	11.24	78.40	78.24	78.32	54126.3	1837	0.8900
95.1879	17.3571	11.19	11.19	11.19	78.40	78.24	78.32	53909.5	1786	0.8824
88.0631	8.5730	11.16	11.19	11.18	78.40	78.22	78.31	53823.5	1636	0.8794
84.3885	4.0426	11.14	11.17	11.15	78.32	78.20	78.26	53658.6	1573	0.8737
82.1432	1.2743	11.14	11.17	11.16	78.32	78.20	78.26	53658.6	1531	0.8737
ER-1										
Mass	w %	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
101.9950	35.8351	10.94	11.11	11.025	80.01	80.02	80.02	55438.5	1840	0.9921
101.0675	34.4017	10.79	11.01	10.900	79.82	79.78	79.80	54515.8	1854	0.9589
100.0538	33.0536	10.67	10.78	10.725	79.68	79.58	79.63	53412.2	1873	0.9192
98.4833	30.9651	10.59	10.74	10.665	78.86	78.70	78.78	51985.6	1894	0.8680
95.5065	27.0065	10.50	10.58	10.540	77.46	76.90	77.18	49310.6	1937	0.7719
92.4758	22.9762	10.25	10.55	10.400	75.52	75.60	75.56	46634.5	1983	0.6757
90.7835	20.6992	10.17	10.32	10.245	74.70	74.70	74.70	44899.7	2021	0.6134
88.7699	18.0481	10.02	10.14	10.080	73.82	73.84	73.83	43153.5	2057	0.5608
86.5271	15.0655	9.96	9.90	9.930	72.76	72.62	72.69	41208.7	2100	0.4807
82.9133	10.2598	9.68	9.77	9.725	71.36	71.24	71.30	38829.2	2135	0.3952
79.0101	5.0693	9.60	9.53	9.565	70.56	70.56	70.46	37295.8	2118	0.3401

ER-2										
Mass	w %	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
145.0095	23.4623	14.51	14.13	14.320	80.00	80.00	80.00	71980.2	2015	0.6568
143.5833	22.2309	14.02	14.01	14.015	80.00	79.98	79.99	70429.5	2038	0.8212
142.2313	21.0969	13.97	13.97	13.970	79.52	79.64	79.58	69485.5	2047	0.5994
140.5739	19.6857	13.91	13.94	13.925	79.16	79.16	79.16	68532.5	2051	0.5775
137.4729	17.0455	13.91	13.84	13.875	78.44	78.40	78.42	67015.7	2051	0.5426
134.1006	14.1743	13.87	13.77	13.820	77.80	77.64	77.72	65563.7	2045	0.5092
131.2637	11.7590	13.76	13.70	13.730	77.44	77.36	77.40	64601.5	2032	0.4870
128.9771	9.8121	13.74	13.66	13.700	77.24	77.08	77.16	64061.2	2013	0.4746
126.1431	7.3992	13.85	13.60	13.625	77.00	76.90	76.95	63364.2	1991	0.4585
123.2770	4.9590	13.65	13.55	13.600	76.82	76.68	76.75	62919.5	1959	0.4483
120.0768	2.2344	13.57	13.54	13.555	76.74	76.56	76.65	62548.0	1920	0.4397
ER-3										
Mass	w %	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
139.7771	23.2619	13.71	13.61	13.660	80.03	80.02	80.02	68700.4	2035	0.6498
138.7898	22.3912	13.54	13.50	13.520	80.02	80.00	80.01	67074.2	2042	0.6323
137.5568	21.3039	13.43	13.44	13.435	79.74	79.60	79.67	66975.7	2054	0.6084
135.2612	19.2795	13.39	13.36	13.375	79.08	78.90	78.99	65543.2	2064	0.5740
132.7872	17.0802	13.32	13.27	13.295	78.36	78.14	78.25	63936.2	2077	0.5354
129.6180	14.3031	13.18	13.15	13.165	77.38	77.28	77.33	61831.1	2096	0.4848
127.9018	12.7897	13.11	13.08	13.095	76.82	76.82	76.82	60693.8	2107	0.4575
124.8157	10.0682	13.01	13.01	13.010	76.28	76.24	76.26	59423.8	2100	0.4270
122.8196	8.3080	13.01	13.01	13.010	76.08	76.08	76.08	59143.7	2077	0.4203
120.6959	6.4352	13.01	12.99	13.000	76.00	75.90	75.95	58896.4	2049	0.4143
118.6795	4.6570	13.00	12.99	12.995	76.00	75.96	75.93	58842.8	2017	0.4131
116.1480	2.4246	12.99	12.97	12.980	75.92	75.80	75.86	58666.5	1980	0.4088
113.3985	0.0000	12.98	12.96	12.970	75.80	75.70	75.75	58451.4	1940	0.4037
ER-4										
Mass	w %	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	φ1	φ2	Ave.			
147.0801	18.4205	13.80	13.85	13.825	80.02	80.02	80.02	69526.8	2115	0.4933
146.0712	17.6081	13.71	13.73	13.720	79.98	79.92	79.95	68878.1	2121	0.4793
144.9335	16.6921	13.66	13.89	13.675	79.50	79.66	79.58	68018.2	2131	0.4609
143.0583	15.1823	13.62	13.65	13.635	78.78	79.00	78.89	66648.3	2146	0.4314
139.5805	12.3661	13.56	13.61	13.585	77.66	77.88	77.77	64531.8	2163	0.3860
136.0022	9.5012	13.47	13.51	13.490	76.86	77.10	76.98	62785.3	2166	0.3485
134.3004	8.1310	13.41	13.47	13.440	76.60	76.78	76.69	62082.1	2163	0.3334
132.2104	6.4482	13.41	13.46	13.435	76.38	76.52	76.45	61671.2	2144	0.3245
130.5653	5.1237	13.40	13.46	13.430	76.26	76.52	76.39	61551.5	2121	0.3220
129.1324	3.9700	13.37	13.46	13.415	76.20	76.52	76.36	61434.5	2102	0.3195
126.7642	2.0633	13.36	13.45	13.405	76.12	76.45	76.29	61288.2	2069	0.3159

ER-5										
Mass	w %	Height (cm)			Diameter (mm)			Volume (mm ³)	γ (kg/m ³)	e
		Ht-1	Ht-2	Ave	ϕ 1	ϕ 2	Ave.			
138.7281	27.5989	13.89	13.90	13.895	80.00	80.02	80.01	69861.3	1986	0.7549
137.9067	28.8434	13.82	13.81	13.815	79.92	79.98	79.95	69355.0	1988	0.7421
136.8383	25.8607	13.82	13.77	13.795	79.52	79.68	79.59	68832.3	1994	0.7240
134.9499	24.1238	13.77	13.73	13.750	78.88	78.92	78.89	67210.4	2008	0.6883
132.2663	21.6555	13.75	13.72	13.735	78.32	78.42	78.37	66254.9	1996	0.6643
129.6881	19.2841	13.68	13.58	13.630	77.74	77.84	77.79	64778.9	2002	0.6272
127.1109	16.9137	13.64	13.52	13.580	77.38	77.46	77.41	63912.2	1989	0.6054
124.4420	14.4589	13.51	13.43	13.470	77.28	77.46	77.37	63329.0	1965	0.5908
121.4661	11.7217	13.46	13.43	13.445	77.18	77.44	77.31	63113.5	1925	0.5854
119.3405	9.7887	13.45	13.43	13.440	77.18	77.40	77.29	63057.4	1893	0.5839
115.4486	8.1851	13.45	13.42	13.435	77.18	77.40	77.29	63033.9	1832	0.5834
112.8712	3.8183	13.45	13.42	13.435	77.18	77.38	77.28	63017.6	1791	0.5829
110.5222	1.6558	13.45	13.41	13.430	77.08	77.38	77.23	62912.6	1757	0.5803

Table A.3 Results of suction tests of the soil samples from South Africa and Eritrea.

Sample Code	Test No.	Filter paper									Disc Sample										
		Top			Bottom			Suction, kPa			mass g	w %	Height, mm			Diameter, mm			Volume mm ³	γ kg/m ³	e
		Wet mass, g	Dry mass, g	W, %	Wet mass, g	Dry mass, g	W, %	Top	Bottom	Ave			1	2	Ave	1	2	Ave			
SA-1	1	0.6789	0.5939	14.31	0.6789	0.5956	13.99	23565	25233	24399	149.2653	12.18	14.56	14.62	14.59	78.78	78.84	78.81	71159.5848	2098	0.4311
	2	0.6591	0.5986	10.11	0.6510	0.5900	10.34	56879	54179	55529	146.0133	9.74	14.55	14.59	14.57	78.64	78.60	78.62	70721.5931	2055	0.4223
	3	0.6619	0.6094	8.62	0.6638	0.6109	8.66	77752	77034	77393	143.5135	7.86	14.53	14.59	14.56	78.60	78.54	78.57	70569.2655	2034	0.4192
	4	0.6426	0.5993	7.23	0.6204	0.5780	7.34	104039	101657	102848	137.1981	3.11	14.53	14.55	14.54	78.60	78.46	78.53	70400.5598	1949	0.4158
SA-2	1	0.6859	0.5926	15.74	0.6854	0.5948	15.23	17457	19434	18445	116.3537	15.39	11.30	11.28	11.29	78.18	78.06	78.12	54113.8324	2150	0.4201
	2	0.6599	0.5926	11.36	0.6604	0.5948	11.03	43774	46887	45331	110.9926	10.07	11.06	11.11	11.09	76.10	75.90	76.00	50286.6568	2207	0.3198
	3	0.6564	0.6117	7.31	0.6277	0.5846	7.37	102258	100874	101566	108.3247	7.42	11.00	11.11	11.06	75.06	74.98	75.02	48865.5451	2217	0.2824
	4	0.6294	0.5987	5.13	0.6157	0.5857	5.12	161456	161649	161552	102.8146	1.96	10.94	11.00	10.97	74.74	74.40	74.57	47909.9482	2146	0.2573
SA-3	1	0.6821	0.5875	16.10	0.6804	0.5890	15.52	16195	18305	17250	130.0889	20.27	14.01	14.00	14.01	78.78	78.78	78.78	68266.0772	1906	0.7214
	2	0.7104	0.6240	13.85	0.6547	0.5657	15.73	25983	17499	21741	127.6136	17.99	13.98	13.98	13.98	78.52	78.68	78.60	67833.1750	1881	0.7105
	3	0.6599	0.5837	13.05	0.6922	0.6180	12.01	30670	38203	34436	125.0596	15.62	13.96	13.94	13.95	78.44	78.68	78.56	67618.7345	1849	0.7051
	4	0.6605	0.6036	9.43	0.6465	0.5922	9.17	65591	69228	67410	116.0284	7.27	13.92	13.86	13.89	78.40	78.50	78.45	67139.4877	1728	0.6930
SA-4a	1	0.7113	0.6087	16.86	0.6985	0.5971	16.98	13830	13468	13649	132.0973	24.48	14.10	14.06	14.08	77.48	77.36	77.42	66282.4984	1993	0.7103
	2	0.6698	0.5919	13.16	0.6991	0.6195	12.85	29994	32020	31007	129.3362	21.88	13.92	14.00	13.96	76.44	76.32	76.38	63963.8517	2022	0.6506
	3	0.6852	0.6223	10.11	0.7046	0.6405	10.01	56870	58073	57471	127.0918	19.77	13.88	14.06	13.97	76.12	75.98	76.05	63457.7580	2003	0.6374
	4	0.6378	0.5858	8.88	0.6312	0.5818	8.49	73603	79801	76702	118.7246	11.88	13.84	13.96	13.90	75.92	75.80	75.86	62802.0918	1890	0.6205
SA-4b	1	0.7135	0.6113	16.72	0.6824	0.5847	16.71	14233	14260	14247	122.3259	23.78	13.14	13.00	13.07	76.90	77.00	76.95	60783.0820	2012	0.6847
	2	0.6593	0.5876	12.20	0.6788	0.6045	12.29	36668	35990	36329	119.9068	21.33	13.08	12.94	13.01	76.00	76.18	76.09	59159.2073	2027	0.6397
	3	0.6820	0.6163	10.66	0.6580	0.5932	10.69	50651	51439	51045	117.5637	18.96	13.04	12.92	12.98	75.90	75.98	75.94	58790.3112	2000	0.6295
	4	0.6463	0.5903	9.49	0.6454	0.5877	8.82	64773	60430	62601	109.2036	10.50	12.90	12.87	12.89	75.74	75.80	75.77	58099.0296	1880	0.6103
SA-5	1	0.7048	0.6014	17.19	0.6908	0.5885	17.38	12885	12383	12634	126.3216	17.39	13.24	13.28	13.26	77.02	77.15	77.09	61883.2564	2041	0.4691
	2	0.6759	0.5919	14.19	0.6884	0.6027	14.39	24168	23207	23688	123.7389	14.99	13.16	13.15	13.15	76.12	76.40	76.26	60063.3064	2060	0.4259
	3	0.6894	0.6181	11.54	0.6857	0.6169	11.15	42166	45688	43927	120.9873	12.44	13.16	13.08	13.11	75.90	76.20	76.05	59528.5554	2032	0.4132
	4	0.6224	0.5935	4.87	0.6175	0.5881	5.00	170437	165867	168152	111.0985	3.25	13.06	12.94	13.00	75.43	75.64	75.54	58254.5297	1907	0.3630

SA-6	1	0.7064	0.6079	18.20	0.6880	0.5900	16.61	15855	14560	15208	137.5456	19.89	14.66	15.38	15.02	76.12	76.00	76.06	68233.8952	2016	0.5854
	2	0.6573	0.5843	12.49	0.6812	0.6056	12.48	34495	34569	34532	135.0424	17.71	14.52	15.30	14.91	75.42	75.42	75.42	69610.1780	2027	0.5477
	3	0.6628	0.5971	11.00	0.6664	0.6014	10.81	47140	49107	48124	132.5584	15.54	14.42	15.22	14.82	74.94	75.14	75.04	65542.6106	2022	0.5228
	4	0.6210	0.5868	5.83	0.6117	0.5786	5.72	139416	142593	141005	121.9553	6.30	14.34	15.01	14.67	74.30	74.20	74.25	63531.1791	1920	0.4761
SA-7	1	0.7181	0.6053	18.64	0.7001	0.5889	18.68	8525	9044	8284	121.5936	18.24	12.66	12.62	12.64	76.30	76.18	76.23	57886.4400	2108	0.5368
	2	0.6935	0.5949	16.57	0.6799	0.5860	16.02	14670	16463	15566	119.2705	16.96	12.50	12.38	12.44	75.32	75.20	75.26	55339.9407	2155	0.4742
	3	0.6801	0.5875	13.82	0.6695	0.5877	13.92	26102	25591	25846	117.1684	14.90	12.32	12.26	12.29	74.46	74.52	74.49	53559.6478	2188	0.4288
	4	0.6321	0.5959	6.07	0.6360	0.5984	6.28	132395	126733	129564	107.2124	5.13	12.04	12.10	12.07	73.66	73.80	73.63	51382.8844	2087	0.3688
SA-8	1	0.6869	0.5977	14.92	0.6813	0.5918	15.12	20730	19882	20306	123.0552	15.70	12.89	12.74	12.82	76.92	76.58	76.75	59287.7898	2076	0.4670
	2	0.6674	0.5901	13.10	0.6670	0.5868	13.09	30383	30449	30416	120.9481	13.72	12.88	12.67	12.78	76.34	76.02	76.16	58228.1144	2077	0.4408
	3	0.6607	0.5928	11.45	0.6718	0.6019	11.58	42890	41774	42332	118.5494	11.47	12.86	12.66	12.76	75.84	76.00	75.92	57763.4274	2052	0.4293
	4	0.6304	0.5968	5.63	0.6321	0.5981	5.88	145328	143874	144501	110.7532	4.14	12.68	12.59	12.64	75.60	75.30	75.45	56491.5862	1961	0.3978
SA-9	1	0.7518	0.6070	23.82	0.7271	0.5886	23.53	3213	3415	3314	125.2215	22.54	14.44	14.18	14.31	76.94	76.58	76.76	68221.5661	1891	0.6555
	2	0.7248	0.6033	20.14	0.7232	0.5979	20.96	6950	5856	6403	123.5274	20.86	14.36	14.14	14.25	76.74	76.30	76.52	65532.1880	1885	0.6382
	3	0.7109	0.6032	17.85	0.7018	0.5987	17.22	11218	12812	12015	120.7800	18.17	14.31	14.11	14.21	76.42	76.06	76.24	64870.8718	1862	0.6217
	4	0.6206	0.5853	6.03	0.6302	0.5908	6.67	132614	116899	125258	108.9642	6.63	14.20	13.94	14.07	75.66	75.30	75.48	62957.5417	1731	0.5739
SA-10	1	0.7061	0.5962	18.43	0.6926	0.5854	18.31	9937	10182	10064	116.3006	18.15	11.60	11.69	11.65	76.92	77.14	77.03	54268.6690	2143	0.5912
	2	0.6665	0.5720	16.52	0.6681	0.5744	16.31	14834	15495	15165	115.0144	16.84	11.60	11.68	11.64	76.58	76.78	76.68	53753.5400	2140	0.5366
	3	0.6695	0.5885	13.76	0.6854	0.6024	13.78	26435	26355	26395	111.7012	13.48	11.58	11.62	11.60	76.36	76.44	76.40	53178.3169	2101	0.5201
	4	0.6154	0.5841	5.36	0.6248	0.5932	5.33	153631	154854	154342	100.5913	2.19	11.50	11.55	11.53	76.34	76.44	76.39	52820.6615	1904	0.5099
SA-11	1	0.6971	0.5903	18.09	0.7047	0.5970	18.04	10672	10790	10731	117.2589	19.90	12.28	12.20	12.24	76.58	76.40	76.49	56244.5726	2085	0.5722
	2	0.6802	0.5895	15.39	0.6971	0.6031	15.59	18818	18044	18431	115.1522	17.74	12.06	12.14	12.10	75.88	75.72	75.80	54602.6428	2109	0.5271
	3	0.6651	0.5938	12.61	0.6728	0.5973	12.57	38195	33955	36075	112.9051	15.44	11.90	11.98	11.97	75.18	75.12	75.15	53093.5790	2127	0.4849
	4	0.6180	0.5864	5.39	0.6275	0.5956	5.36	152862	153919	153390	103.8110	6.15	11.78	11.79	11.78	74.10	73.90	74.00	50642.3950	2050	0.4164
SA-12	1	0.6650	0.5838	13.91	0.6848	0.6001	14.11	25643	24563	25103	137.1415	11.28	13.36	13.21	13.29	77.98	78.10	78.04	63545.6698	2158	0.4296
	2	0.6278	0.5699	10.16	0.6583	0.5928	10.75	96254	49717	52985	135.0405	9.58	13.34	13.21	13.28	77.68	77.78	77.73	62994.3714	2144	0.4183
	3	0.6230	0.5837	6.73	0.6254	0.5879	6.38	115342	124230	119786	132.0925	7.19	13.33	13.21	13.27	77.36	77.46	77.41	62453.2050	2115	0.4061
	4	0.6129	0.5917	3.58	0.6124	0.5910	3.62	223171	221397	222284	126.2540	2.45	13.33	13.21	13.27	77.20	77.32	77.26	62211.4340	2029	0.4007

SA-13	1	0.6601	0.5855	12.74	0.6756	0.5999	12.62	32751	33603	33177	128.5436	12.21	12.68	12.84	12.76	76.58	76.40	76.49	58634.0479	2192	0.3405
	2	0.6559	0.5929	10.63	0.6528	0.5898	10.68	51020	50426	50723	126.6905	10.70	12.57	12.70	12.64	75.84	75.88	75.86	57107.1918	2216	0.3069
	3	0.6638	0.6062	9.50	0.6397	0.5862	9.13	64568	69849	67209	124.6108	9.02	12.57	12.68	12.63	75.18	75.16	75.17	56028.6771	2224	0.2838
	4	0.6230	0.5914	5.34	0.6227	0.5916	5.26	154328	157145	155737	122.3856	7.21	12.56	12.67	12.62	74.96	74.86	74.91	55597.6878	2201	0.2756
SA-14	1	0.7552	0.5937	27.20	0.7490	0.5877	27.45	1582	1503	1543	100.6478	24.09	11.22	11.27	11.25	78.40	78.24	78.32	54174.4734	1858	0.8917
	2	0.7097	0.5805	22.26	0.7259	0.5923	22.56	4460	4189	4324	99.1041	22.19	11.21	11.24	11.23	78.40	78.24	78.32	54078.1204	1833	0.8883
	3	0.6940	0.5919	17.25	0.6697	0.5710	17.29	12734	12639	12687	94.8621	16.96	11.19	11.19	11.19	78.40	78.24	78.32	53908.5027	1760	0.8824
	4	0.6025	0.5947	1.31	0.5956	0.5877	1.34	359193	356745	357969	81.1978	0.11	11.14	11.17	11.16	78.32	78.20	78.26	53658.5760	1513	0.8737
ER-1	1	0.7024	0.5920	18.65	0.6907	0.5831	18.45	9498	9896	9697	92.2489	22.67	10.22	10.50	10.36	75.50	75.52	75.51	46393.6538	1988	0.6670
	2	0.6728	0.5839	15.23	0.6783	0.5855	15.85	19462	17075	18268	90.5702	20.44	10.16	10.30	10.23	74.70	74.70	74.70	44833.9206	2020	0.6110
	3	0.6675	0.5883	13.46	0.6635	0.5863	13.17	28157	29954	29056	88.6125	17.84	10.02	10.14	10.08	73.82	73.84	73.83	43153.5124	2053	0.5506
	4	0.6009	0.5932	1.30	0.6040	0.5967	1.22	380213	365892	363053	75.6399	0.59	9.59	9.48	9.54	70.56	70.56	70.56	37284.4485	2029	0.3397
ER-2	1	0.7002	0.6025	16.22	0.6812	0.5860	16.25	15814	15715	15765	132.8725	13.13	13.82	13.74	13.78	77.68	77.48	77.58	65138.6344	2040	0.4994
	2	0.6592	0.5738	14.88	0.6798	0.5929	14.66	20908	21924	21416	131.0858	11.61	13.76	13.69	13.73	77.40	77.36	77.38	64544.5660	2031	0.4857
	3	0.6659	0.5900	12.86	0.6755	0.6021	12.19	31917	38756	34337	128.8178	9.68	13.74	13.66	13.70	77.22	77.06	77.14	64027.9678	2012	0.4738
	4	0.6030	0.5903	2.15	0.6099	0.5957	2.38	301231	286920	294075	119.0499	1.36	13.56	13.54	13.55	76.72	76.56	76.64	62508.6571	1905	0.4388
ER-3	1	0.6803	0.5944	14.45	0.6544	0.5715	14.51	22887	22629	22758	129.4072	14.12	13.18	13.15	13.17	77.26	77.24	77.25	61703.2047	2097	0.4817
	2	0.6559	0.5830	12.50	0.6663	0.5928	12.39	34418	35244	34831	127.7201	12.63	13.11	13.08	13.10	76.74	76.78	76.76	60598.9803	2108	0.4552
	3	0.6627	0.6047	9.59	0.6478	0.5911	9.59	63365	63355	63360	124.6852	9.95	13.01	13.01	13.01	76.28	76.24	76.26	59423.8492	2098	0.4270
	4	0.6275	0.5958	5.32	0.6282	0.5958	5.44	155063	151293	153178	119.6154	5.48	13.00	12.99	13.00	76.00	75.88	75.94	58858.2507	2032	0.4134
ER-4	1	0.6603	0.5872	12.45	0.6522	0.5805	12.35	34820	35539	35179	135.8120	9.35	13.47	13.50	13.49	76.80	77.08	76.94	62696.7741	2166	0.3466
	2	0.6272	0.5668	10.66	0.6443	0.5836	10.40	50694	53480	52087	134.1673	8.02	13.41	13.47	13.44	76.58	76.74	76.66	62033.5707	2163	0.3323
	3	0.6376	0.5991	6.43	0.6217	0.5831	6.62	122995	118108	120552	132.1333	6.39	13.41	13.46	13.44	76.38	76.52	76.45	61671.2188	2143	0.3245
	4	0.6228	0.5931	5.01	0.6284	0.5981	5.07	185574	163558	164566	129.0730	3.92	13.37	13.48	13.42	76.20	76.52	76.36	61434.5099	2101	0.3195
ER-5	1	0.6848	0.5971	14.69	0.6677	0.5823	14.67	21782	21882	21832	129.4563	19.07	13.68	13.58	13.63	77.70	77.80	77.75	64712.2548	2000	0.6255
	2	0.6438	0.5757	11.83	0.6893	0.6180	11.54	39649	42150	40900	126.9726	16.79	13.62	13.52	13.57	77.32	77.46	77.39	63832.1426	1989	0.6034
	3	0.6551	0.5993	9.31	0.6598	0.6040	9.24	67204	68232	67718	124.3258	14.35	13.50	13.43	13.47	77.26	77.44	77.35	63272.7740	1965	0.5894
	4	0.6368	0.6047	5.31	0.6265	0.5923	5.77	155459	141006	148233	115.0159	5.79	13.45	13.42	13.44	77.18	77.39	77.29	63025.7431	1825	0.5832

Table A.4 Results of swelling tests of the soil samples from South Africa and Eritrea.

Sample	Time, hr	0	24	48	72	96	120	144	168	192	216	240		0	24	48	72	96	120	144	168	192	216	240	264	288	312
SA-1	Reading, mm	5.409	5.423	5.993	6.025	6.049	6.059	6.069	6.075	6.082	6.090			1.791	1.821	2.984	3.334	3.715	3.864	3.936	3.958	3.982	4.006	4.046			
	H, mm	12.480	12.494	13.064	13.096	13.120	13.130	13.140	13.146	13.153	13.161			11.150	11.180	12.343	12.693	13.074	13.223	13.295	13.317	13.341	13.365	13.405			
	Δh	0.000	0.014	0.570	0.032	0.024	0.010	0.010	0.006	0.007	0.008			0.000	0.030	1.163	0.350	0.381	0.149	0.072	0.022	0.024	0.024	0.040			
	Swell, %	0.000	0.115	4.815	5.078	5.276	5.359	5.441	5.491	5.548	5.614			0.000	0.269	10.690	13.826	17.240	18.575	19.220	19.418	19.633	19.848	20.206			
SA-3	Reading, mm	2.438	2.449	2.494	2.563	2.570	2.574	2.579	2.582	2.584	2.586	2.587		0.644	0.667	0.964	1.108	1.131	1.147	1.152	1.157	1.160	1.173	1.189			
	H, mm	12.190	12.201	12.246	12.315	12.322	12.326	12.331	12.334	12.336	12.338	12.339		11.910	11.933	12.230	12.374	12.397	12.413	12.418	12.423	12.426	12.439	12.455			
	Δh	0.000	0.000	0.045	0.069	0.007	0.004	0.005	0.003	0.002	0.002	0.001		0.000	0.023	0.297	0.144	0.023	0.016	0.005	0.005	0.003	0.013	0.016			
	Swell, %	0.000	0.090	0.459	1.025	1.083	1.116	1.157	1.181	1.198	1.214	1.222		0.000	0.191	2.656	3.851	4.041	4.174	4.216	4.257	4.282	4.390	4.523			
SA-4a	Reading, mm	1.268	1.272	1.735	1.782	1.806	1.811	1.820	1.823	1.824	1.824	1.825		3.473	3.484	4.104	4.185	4.217	4.236	4.251	4.258	4.264	4.266	4.272			
	H, mm	11.400	11.404	11.867	11.914	11.938	11.943	11.952	11.955	11.956	11.956	11.957		12.000	12.011	12.631	12.712	12.744	12.763	12.778	12.785	12.791	12.793	12.799			
	Δh	0.000	0.004	0.463	0.047	0.024	0.005	0.009	0.003	0.001	0.000	0.001		0.000	0.011	0.620	0.081	0.032	0.019	0.015	0.007	0.006	0.002	0.006			
	Swell, %	0.000	0.035	4.047	4.454	4.662	4.705	4.783	4.809	4.818	4.818	4.827		0.000	0.092	5.276	5.953	6.221	6.380	6.505	6.564	6.614	6.630	6.681			
SA-6	Reading, mm	1.645	1.706	2.437	2.581	2.633	2.666	2.691	2.699	2.710	2.715	2.724		1.540	1.548	3.143	3.812	3.912	3.961	3.992	4.007	4.022	4.029	4.040			
	H, mm	11.370	11.431	12.162	12.306	12.358	12.391	12.416	12.424	12.435	12.440	12.449		12.130	12.138	13.733	14.402	14.502	14.551	14.582	14.597	14.612	14.619	14.630			
	Δh	0.000	0.061	0.731	0.144	0.052	0.033	0.025	0.008	0.011	0.005	0.009		0.000	0.008	1.595	0.669	0.100	0.049	0.031	0.015	0.015	0.007	0.011			
	Swell, %	0.000	0.536	6.966	8.232	8.690	8.980	9.200	9.270	9.367	9.411	9.490		0.000	0.065	13.118	18.592	19.411	19.812	20.065	20.188	20.311	20.368	20.458			
SA-8	Reading, mm	2.134	2.168	2.978	3.048	3.054	3.070	3.082	3.084	3.088	3.090	3.095		2.276	2.427	3.323	3.359	3.360	3.380	3.384	3.389	3.396	3.400	3.406			
	H, mm	11.260	11.294	12.104	12.174	12.180	12.196	12.208	12.210	12.214	12.216	12.221		10.036	10.187	11.083	11.119	11.120	11.140	11.144	11.149	11.156	11.160	11.166			
	Δh	0.000	0.034	0.810	0.070	0.006	0.016	0.012	0.002	0.004	0.002	0.005		0.000	0.151	0.896	0.036	0.001	0.020	0.004	0.005	0.007	0.004	0.006			
	Swell, %	0.000	0.302	7.496	8.117	8.171	8.313	8.419	8.437	8.472	8.490	8.535		0.000	1.505	10.432	10.791	10.801	11.000	11.040	11.090	11.160	11.200	11.259			

SA-10	Reading, mm	0.153	0.169	1.461	1.487	1.494	1.504	1.509	1.511	1.514	1.515	1.519	SA-11	2.698	2.832	4.322	4.588	4.623	4.686	4.678	4.686	4.694	4.701	4.713						
	H, mm	10.022	10.038	11.330	11.356	11.363	11.373	11.378	11.380	11.383	11.384	11.388		9.750	9.884	11.374	11.640	11.675	11.718	11.730	11.738	11.746	11.753	11.765						
	Δh	0.000	0.016	1.292	0.026	0.007	0.010	0.005	0.002	0.003	0.001	0.004		0.000	0.134	1.490	0.266	0.035	0.043	0.012	0.008	0.008	0.007	0.012						
	Swell, %	0.000	0.160	13.051	13.311	13.381	13.480	13.530	13.550	13.580	13.590	13.630		0.000	1.374	16.656	19.385	19.744	20.185	20.308	20.390	20.472	20.544	20.667						
SA-12	Reading, mm	4.48	4.508	4.703	4.736	4.747	4.758	4.761	4.763	4.764	4.765	4.768	SA-13	4.102	4.111	5.385	5.540	5.616	5.660	5.681	5.692	5.700	5.709	5.723						
	H, mm	11.015	11.043	11.238	11.271	11.282	11.293	11.296	11.298	11.299	11.300	11.303		10.370	10.379	11.653	11.808	11.884	11.928	11.949	11.960	11.968	11.977	11.991						
	Δh	0.000	0.028	0.195	0.033	0.011	0.011	0.003	0.002	0.001	0.001	0.003		0.000	0.009	1.274	0.155	0.076	0.044	0.021	0.011	0.008	0.009	0.014						
	Swell, %	0.000	0.254	2.025	2.324	2.424	2.524	2.551	2.569	2.578	2.587	2.615		0.000	0.087	12.372	13.867	14.600	15.024	15.227	15.333	15.410	15.497	15.632						
SA-14	Reading, mm	1.144	1.223	1.473	1.521	1.587	1.640	1.668	1.691	1.701	1.719	1.736	ER-1	0.098	0.124	3.452	4.046	4.271	4.481	4.591	4.608	4.755	4.821	4.903	5.025	5.106	5.117			
	H, mm	10.034	10.113	10.363	10.411	10.477	10.530	10.558	10.581	10.591	10.609	10.626		8.160	8.186	11.514	12.108	12.333	12.543	12.653	12.670	12.817	12.883	12.965	13.087	13.168	13.179			
	Δh	0.000	0.079	0.250	0.048	0.066	0.053	0.028	0.023	0.010	0.018	0.017		0.000	0.026	3.328	0.594	0.225	0.210	0.110	0.017	0.147	0.066	0.082	0.122	0.081	0.011			
	Swell, %	0.000	0.787	3.279	3.757	4.415	4.943	5.222	5.451	5.551	5.731	5.900		0.000	0.319	41.103	48.382	51.140	53.713	55.061	55.270	57.071	57.880	58.885	60.380	61.373	61.507			
ER-2	Reading, mm	2.858	2.870	3.518	3.804	3.874	3.884	3.902	3.910	3.914	3.919	3.925	ER-3	4.108	4.139	5.001	5.202	5.496	5.529	5.546	5.566	5.573	5.578	5.582						
	H, mm	10.520	10.532	11.180	11.466	11.536	11.546	11.564	11.572	11.576	11.581	11.587		10.100	10.131	10.993	11.194	11.488	11.521	11.538	11.558	11.565	11.570	11.574						
	Δh	0.000	0.012	0.648	0.286	0.070	0.010	0.018	0.008	0.004	0.005	0.006		0.000	0.031	0.862	0.201	0.294	0.033	0.017	0.020	0.007	0.005	0.004						
	Swell, %	0.000	0.114	6.274	8.992	9.658	9.753	9.924	10.000	10.038	10.086	10.143		0.000	0.307	8.842	10.832	13.743	14.069	14.238	14.436	14.505	14.554	14.594						
ER-4	Reading, mm	3.199	3.211	3.880	4.034	4.064	4.078	4.090	4.097	4.101	4.105	4.109	ER-5	0.254	0.260	0.614	0.628	0.633	0.637	0.641	0.642	0.643	0.644	0.644						
	H, mm	12.100	12.112	12.781	12.935	12.965	12.979	12.991	12.998	13.002	13.006	13.010		6.170	6.176	6.530	6.544	6.549	6.553	6.557	6.558	6.559	6.560	6.560						
	Δh	0.000	0.012	0.689	0.154	0.030	0.014	0.012	0.007	0.004	0.004	0.004		0.000	0.006	0.354	0.014	0.005	0.004	0.004	0.001	0.001	0.001	0.000						
	Swell, %	0.000	0.099	5.628	6.901	7.149	7.264	7.364	7.421	7.455	7.488	7.521		0.000	0.105	6.316	6.561	6.649	6.719	6.789	6.807	6.825	6.842	6.842						

REFERENCES

- Afes, M., Didier, G. and Suratman, I., (1998). Swelling of mono-mineral clays and of their mixtures. Proceedings of the Second International Conference on Unsaturated Soils, Beijing, Volume 1, p.1-6.
- Aitchison, G.D., (1964). Engineering concept of moisture equilibrium and moisture changes in soils. Statement of review panel, in Moisture equilibrium and moisture changes in soils beneath covered areas, Butterworths, p. 7-21.
- Al-Homoud, A.S., Basma, A.A., Hussein Malkawi, A.I., and Al-Bashabshah, M. A., (1995). Cyclic swelling behaviour of clays. Journal of Geotechnical Engineering, ASCE, **121**(7): 562-565.
- Al-Khafaf, S. and Hanks, R.J., (1974). Evaluation of the filter paper method for estimating soil water potential. Soil Science, **117**(4): 194-199.
- Alonso, E. E., (1998). Modelling expansive soil behaviour. Proceedings of the Second International Conference on Unsaturated Soils, Beijing, Volume 2, pp. 37-70.
- ASTM (American Society for Testing and Materials), (1990). Standard Method for Particle Size Analysis of Soils (D 422-90). Annual Book of ASTM Standards, Vol. 04.08. Soil and rock (II). American Society for Testing and Materials, Philadelphia.
- ASTM (American Society for Testing and Materials), (1992). Test method for shrinkage factors of soils by wax method (D4943-89). Annual Book of ASTM Standards, Vol. 04.08. Soil and rock (II). American Society for Testing and Materials, Philadelphia.
- ASTM (American Society for Testing and Materials), (1998). Test method for shrinkage factors of soils by mercury method (D427-98). Annual Book of ASTM Standards, Vol. 04.08. Soil and rock (II). American Society for Testing and Materials, Philadelphia.
- Atabek, R.B., Felix, B., Robinet, J.C. and Lahlou, R. (1991). Rheological behaviour of saturated expansive clay materials. Workshop on stress partitioning in engineered clay barriers. Duke University, Durham, N.C.
- Atkinson, J.H. and Bransby, P.L., (1978). The Mechanics of Soils: an introduction to the critical state mechanics. McGraw-Hill Book Company, UK.
- Baikoff, E.M.A. and Burke, T.J., (1965). Practical determinations of type of foundation to be used in areas where heaving soils occur. Transactions, South African Institution of Civil Engineers, **5**(September): 189-195.

- Barshad, I., (1955). Adsorptive and swelling properties of clay-water system. In *Clays and clay technology: National Conference on Clays and Clay Technology. Proceedings California Division of Mines Bulletin.*
- Biwei, G., Yanhua, L. and Liangtong, Z., (1998). The relationship between residual water content and shrinkage limit for expansive soils. *Proceedings of the Second International Conference on Unsaturated Soils, Beijing, Volume 2, p. 202-207.*
- Bolt, G.H., (1956). Physico-chemical analysis of the compressibility of pure clays. *Geotechnique*, **6**: 86-93.
- Booth, A.R., (1975). The factors influencing collapse settlement in compacted soils. *Proceedings 6th Regional Conference on Soil Mechanics and Foundation Engineering, Volume 1, p. 57-63.*
- Brackley, I.J.A., (1975). A model of unsaturated clay structure and its application to swell behaviour. *Proceedings 6th Regional Conference on Soil Mechanics and Foundation Engineering, Volume 1, p. 71-79.*
- Brackley, I.J.A., (1979). The Prediction of Heave under Foundations. *Ground Profile. Newsletter of the Division of Geotechnical Engineering SAICE, No 19, p. 20-23.*
- Brink, A.B.A., (1955). The genesis and distribution of expansive soil types in South Africa. *Transactions South African Institution of Civil Engineers*, **5** (August): 267-272.
- Brink, A.B.A., Patridge, I.C. and Williams, A.A.B., (1982). *Soil survey for engineering.* Clarendon Press, Oxford.
- BSI, (1990). *Methods of test for soils for civil engineering purposes; BS 1377, part 2, classification tests.* British Standard Institution, London.
- BRE (Building Research Establishment), (1993). *Low-rise buildings on shrinkable clay soils. BRE-Digest 240, Part one, London.*
- Burland, J. B., (1990). On the compressibility and shear strength of natural clays. *Geotechnique*, **40**(3): 329-378.
- Cerato, A. B. and Lutenecker, A. J., (2004). Determining the Intrinsic Compressibility of Fine-Grained Soils. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE, (accepted).*
- Chandler, R.J. and Gutierrez, C.I., (1986). The filter-paper method of suction measurement. *Geotechnique*, **36**: 256-268.
- Chandler, R.J., Crilly, M.S., and Montgomery-Smith, G., (1992). A low cost method for assessing clay desiccation of low-rise buildings. *Proc. of the Inst. of Civil Engineers*, **92**(2): 82-89.

- Chen, F.H., (1973). The basic physical property of expansive soils. Proceedings of the 3rd International Conference on Expansive Soils, Haifa, pp.201-208.
- Chen, F.H., (1988). Foundations on Expansive Soils. American Elsevier Science Publ., New York.
- Crescimanno, G. and Provenzano, G., (1999). Soil shrinkage characteristic curve in clay soils: measurement and prediction. Soil Science Society of America Journal, **63**: 25-32.
- Dagg, M. and Russam, K., (1966). The relation between soil shrinkage and the development of surface cracks in an experimental road in Kenya. RRL Report No. 12, Road Research Laboratory, Ministry of Transport, England.
- Dakshnamurthy, V. and Raman, N., (1973). A simple method of identifying an expansive soils. Soils and Foundations, **13**(1): 97-104.
- Day, R.W., (1994). Swell-shrink behaviour of expansive compacted clay. Journal of Geotechnical Engineering, ASCE, **120**(3): 618-623.
- Day, R.W., (1999). Geotechnical and foundation engineering design and construction. McGraw-Hill Co., New York.
- De Bruijn, C.M.A., (1955). Discussion: The genesis and distribution of expansive soil types in South Africa. Transactions, South African Institution of Civil Engineers, **5**:435-436.
- De Bruijn, C.M.A., (1975). Moisture distribution and soil movements at Barkly East, Blumefontane and Richmond. Proceedings of the 6th African Regional Conference on Soil Mechanics and Foundation Engineering, Durban, South Africa, p. 81-88.
- Delage, P. and Lefebvre. G., (1984). Study of the structure of sensitive Champlain clay and of its evolution during consolidation. Canadian Geotechnical Journal, **21**: 21-35.
- Deryagin, B.V. and Churaev, N.V., (1984). Wetting films. Moskva: Nauka, pp.157.
- Dixon, J.B., (1977). Kaolinite and serpentine group minerals. In Dixon, J.B., Weed, S.B., Kittrick, J.A., Milford, M.H., and White, J.L. (eds.), Minerals In Soil Environments. p. 357-403.
- Djedid, A., Bekkouche, A. and Aïssa Mamoune, S.M., (2001). Identification and prediction of the swelling behaviour of some soils from the Tlemcen region of Algeria. Bulletin des laboratoires des ponts et chaussées, **233**:69-77.

- Duran Gamarra, A.J.D., (1986). Study of the effect of contact of the filter paper technique in the measurement of soil suction. MSc dissertation, University of London, UK.
- Escario, V. and Saez, J., (1973). Measurement of the properties of swelling and collapsing soils under controlled suction. Proceedings of the 3rd International Conference on Expansive Soils, Haifa, pp.195-200.
- Fawcett, R.G. and Collis-George, N., (1967). A filter paper method for determining the moisture characteristics of soil. Australian Journal of Experimental Agriculture and Animal Husbandry, **7**:162-167.
- Fleureau, J.M., Kheirbek-Saoud, S., Soemitro, R. and Taibi, S., (1993). Behaviour of clayey soils on drying-wetting paths. Canadian Geotechnical Journal, **30**: 287-296.
- Fleureau, J.M., Verbrugge, J.C., Huergo, P.J., Correla, A.G. and Kheirbek-Saoud, S., (2002). Aspects of the behaviour of compacted clayey soils on drying and wetting paths. Canadian Geotechnical Journal, **39**:1341-1357.
- Fredlund, D.F., (1998). Bringing unsaturated soil mechanics into engineering practice. Proceedings of the Second International Conference on Unsaturated Soils, Beijing, Vol. 2, p. 1-36.
- Gardner, R., (1937). A method of measuring the capillary tension of soil moisture over a wide moisture range. Soil Science, **43**:277-283.
- Gens, A., Vaunat, J. and Ledesma, A., (1995). Analysis of hydration of an engineered barrier in radioactive waste repository scheme using an elastoplastic model. Proceedings of the First International Conference on Unsaturated Soils, Paris, Volume 2, p.1065-1073.
- Geological Society Engineering Group, (1990). Tropical residual soils: Geological Society Engineering Group Working Party Report. The Quarterly Journal of Engineering Geology, Volume 23, pp.101.
- Gleason, M.H., Daniel, D.E. and Eykholt, G.R., (1997). Calcium and sodium bentonite for hydraulic containment application. Journal Geotechnical and Geoenvironmental engineering, ASCE, **123**(5): 438-445.
- Gourley, C.S. and Schreiner, H.D., (1993a). (Unpublished Report). A new approach to the determination of the expansiveness of Soils. Project Report PR/OSC/011/93, Overseas Development Administration, London.

- Gourley, C.S. and Schreiner, H.D., (1993b). (Unpublished Report). Assessment of the expansiveness of soils. Project Report PR/OSC/012/93, Overseas Development Administration, London.
- Gourley, C.S. and Schreiner, H.D., (1993c). (Unpublished Report). The extent and engineering significance of expansive soils. Project Report PR/OSC/021/93, Overseas Development Administration, London.
- Gourley, C.S. and Schreiner, H.D., (1993d). (Unpublished Report). Filter paper calibration for the TRL suction probe. Project Report PR/OSC/002/93, Overseas Development Administration, London.
- Gourley, C.S. and Schreiner, H.D., (1994). (Unpublished Report). Geotechnical properties of expansive soils. Project Report PR/OSC/015/93, Overseas Development Administration, London.
- Graber, E.R. and Mingelgrin, U., (1994). Clay Swelling and regular solution theory. *Environmental Science and Technology*, **28**(13): 2360-2365.
- Gregory, J., (1989). Fundamentals of flocculation. *Critical Reviews in Environmental Control*, **19**(3): 183-230.
- Haines, W.B., (1923). The volume change associated with variations of water content in soil. *Journal of Agricultural Science*, **13**: 296-310.
- Hamblin, A.P., (1981). Filter paper method for routine measurement of field water potential. *Journal of Hydrology*, **53**: 355-360.
- Hamilton, J.J., (1963). Volume changes in undisturbed clay profiles in western Canada: *Canadian Geotechnical Journal*, **1**(1): 27-42.
- Hanafy, E.A.D.E., (1991). Swelling/shrinkage characteristic curve of desiccated expansive clays. *Geotechnical Testing Journal*, ASCE, **14**(2): 206-211.
- HMSO, (1973). Soil mechanics for road engineers. Department of Environment. Transport and Road Research Laboratory.
- Ho, D.Y.F., Fredlund, D.G. and Rahardjo, H., (1992). Volume change indices during loading and unloading of unsaturated soil. *Canadian Geotechnical Journal*, **29**:195-207.
- Holtz, W.G. and Gibbs, H.J., (1956). Engineering Properties of Expansive Clays. *Transactions, ASCE*, **121**: 641-677.
- Houston, S.L., Houston, W.N., and Wagner, A.N., (1994). Laboratory filter paper suction measurements. *Geotechnical Testing Journal*, ASCE, **17**(2): 185-194.

- Hucckel, T.A., (1992). Water-mineral interaction in hygromechanics of clays exposed to environmental loads: a mixture theory approach. *Canadian Geotechnical Journal*, **29**: 1071-1086.
- Jennings, J.E. and Brink, A.B.A., (1978). Application of geotechnique to the solution of engineering problems-essential preliminary steps to relate the structure which provides its support. *Proceedings of Institution of Civil Engineers, Part 1*, **64**: 571-589.
- Jennings, J.E., Brink, A.B.A. and Williams, A.A.B., (1973). Revised guide to soil profiling for civil engineering purposes in Southern Africa. *Transactions, South African Institution of Civil Engineers*, **15**:3-12.
- Jennings, J.E. and Kerrich, J.E., (1962). Heaving of buildings and associated economic consequences particularly on O.F.S. gold fields. *Transactions, South African Institution of Civil Engineers*, **4**:244-246.
- Jennings, J.E. and Knight, K., (1957). The Prediction of Total Heave from the Double Oedometer Test. *The South African Institution of Civil Engineering*. p. 285-291.
- Justo, J.L., Delgado, A. and Ruiz, J., (1984). The influence of stress path in the collapse-swelling of soils at the laboratory. *Proceedings of the 5th International Conference on Expansive Soils, Adelaide*, p. 179-186.
- Kassif, G., Baker, R. and Ovadia, Y., (1973). Swell-pressure relationships at constant suction changes. *Proceedings of the 3rd International Conference on Expansive Soils, Haifa*, p. 201-208.
- Kellar, W.D., (1964). Processes of origin and alteration of clay minerals. *Seminar on Soil Clay Mineralogy*. University of North Carolina press, Chapel Hill, p.3-76.
- Kohgo, Y., (2002). Elastoplastic models for unsaturated soils with two suction effects and unsaturated behaviour. *Proceedings of the Third International Conference on Unsaturated Soils, Recife, Brazil, Vol. 3*.
- Komine, H. and Ogata, N., (1994). Experimental study on swelling characteristics of compacted bentonite. *Canadian Geotechnical Journal*, **31**: 478-490.
- Komine, H. and Ogata, N., (1996). Prediction for swelling characteristics of compacted bentonite. *Canadian Geotechnical Journal*, **33**:11-12.
- Komine, H. and Ogata, N., (2003). New equations for swelling characteristics of bentonite-based buffer materials. *Canadian Geotechnical Journal*, **40**(2): 460-475.
- Lambe, T.W. and Whitman, R.V., (1979). *Soil Mechanics*, John Wiley and Sons, New York.

- Lawton, E.C., Frigaszy, R.J. and Hardcastle, J.H., (1989). Collapse of compacted clayey sand. *Journal of Geotechnical engineering, ASCE*, **115**(9): 1252-1267.
- Lazanyi, I., Horvath, G. and Farkas, J., (1998). Volume change induced cracking of flood protection dikes built of clay. *Proceedings of the Second International Conference on Unsaturated Soils, Beijing, Vol. 2*, p. 213-218.
- Lee, H.C. and Wray, W.K., (1995). Techniques to evaluate soil suction-a vital unsaturated soil variable. *Proceedings of the First International Conference on Unsaturated Soils, France, Vol. 2*, p. 615-623.
- Leong, E.C., He, L., and Rahardjo, H., (2002). Factors affecting the filter paper method for total and matric suction measurements. *Journal of Geotechnical engineering, ASTM*, **25**(3): 1-12.
- Lutenegger, A.J. and Cerato, A.B., (2003). Intrinsic Compressibility of Fine-Grained Soils. *Canadian Geotechnical Journal* (Submitted for print).
- Maud, R.R., (1989). *Engineering Geology of Durban. Contributions to engineering Geology. Institute of engineering geology in collaboration with the Geological Survey of South Africa. Volume 1. Government Printer, Pretoria, South Africa.*
- McKeen, R. G., (1992). A model for predicting expansive soil behaviour. *7th International Conference on Expansive Soils, Dallas, Vol. 1*, pp. 1-6.
- McKeen, R.G., (2001). Investigating field behaviour of expansive clay soils. In *Expansive Clay Soils and Vegetative Influence on Shallow Foundations, Geotechnical Special Publication No. 115* (ed. C. Vipulanandan, M. B. Addison and M. Hasen), Reston: American Society of Civil Engineers, p. 82-94.
- McQueen, I.S. and Miller, R.F., (1968). Calibration of a wide range of gravimetric method for measuring moisture stress. *Soil Science*, **106**(3): 225-231.
- Mitchell, J.K., (1976). *Fundamentals of soil Behaviour. University of California, Berkeley, USA.*
- Mitchell, J.K., (1993). *Fundamentals of soil Behaviour. 2nd edn. Wiley, New York.*
- Modaressi, A., Abou Bekr, N. Fry, J.J., (1996). Unified approach to modelling partially saturated soils. *Proceedings of the First International Conference on Unsaturated Soils, France, Vol. 3*, p. 111-116.

- Moore, D.M. and Reynolds, R. C. Jr., (1989). X-Ray Diffraction and the identification and analysis of Clay Minerals, Oxford University, USA.
- Nelson, J.D. and Miller, D.J. (1992). Expansive Soil: Problems and Practice in Foundation and Pavement Engineering. John Wiley and Sons, USA.
- Oloo, S.Y., (1987). Identification and classification of expansive soils, Masters thesis, Imperial College, University of London.
- Oloo, S.Y., Schreiner, H.D. and Burland, J. B., (1987). Identification and Classification of Expansive soils. Proceedings of the 6th International Conference in Expansive soils, New Delhi, India, Vol. 1, p. 23-29.
- Olson, R.E. and Mesri, G., (1970). Mechanisms controlling compressibility of clays. Journal of the Soil Mechanics and Foundations Division, ASCE, **96**(SM6): 1863-1878.
- Osipov, V.I. and Babak, V.G., (1986). Nature and mechanism of clay swelling. 5th international IAEG Congress, Buenos Aires.
- Pidgeon, J.T., (1987). The Prediction of Differential Heave for the Design of Foundations on Expansive Soil Areas. Proceedings of the 9th African Regional Conference on Soil Mechanics and Foundation Engineering, Lagos, Nigeria.
- Popescu, M., (1980). Behaviour of expansive soils with crumb structure. Proceedings of the 4th Internal Conference on Expansive Soils, Vol.1, New York, p. 158-171.
- Ranganatham, S.V. and Satyanarayana, B., (1965). A rational method of predicting swell potential for compacted expansive clays. Proc. 6 ICSMFE, Vol.1, pp. 32.
- Richards, B.G., Peter, P. and Martin, R., (1984). The determination of volume change properties in expansive soil. Proceedings of the 4th Internal Conference on Expansive Soils, Adelaide, pp. 179-186.
- Schreiner, H.D., (1987a). The use of Predictive Methods in Expansive Soils Engineering. Proceedings of the 9th African Regional Conference on Soil Mechanics and Foundation Engineering, Lagos, Nigeria, Vol.1, p.135-141.
- Schreiner, H.D., (1987b). The measurement of solute suction in soils of high plasticity. Proceedings of the 9th African Regional Conference on Soil Mechanics and Foundation Engineering, Lagos, Nigeria, Vol.1, p.163-171.
- Schreiner, H.D., (1988). Volume change of compacted highly plastic African Clays. Ph.D thesis, Imperial College, University of London, pp. 313.

- Schreiner, H.D., (1999). A new approach to the determination of the expansiveness of Soils. Proceedings of the 12th African Regional Conference on Soil Mechanics and Foundation Engineering, Durban, South Africa.
- Schreiner, H.D. and Burland, J.B., (1991). A comparison of the three swell test procedures. Proceedings of the 10th African Regional Conference on Soil Mechanics and Foundation Engineering, Maseru, Lesotho.
- Schreiner, H.D. and Gourley, C.S., (1993). (Unpublished Report). Suction and its Measurement. Project Report PR/OSC/013/93, Overseas Development Administration, London.
- Schreiner, H.D. and Gourley, C.S., (1995). Determination of moisture content in soil profiling. Quarterly journal of engineering geology, **28**:195-198.
- Seed, H.B. and Chan, C.K., (1962). Studies of swell and swelling pressure characteristics of compacted clays. Highway Resource Board Bull. **313**:12-39.
- Seed, H.B., Woodward, R.J. and Lundgren, R., (1962). Prediction of Swelling Potential for Compacted Clays, Journal of Soil Mechanics and Foundation, Division of ASCE, **88** (SM3): 53-87.
- Shang, J.Q., Lo, K.Y. and Quigley, R.M. (1994). Quantitative determination of potential distribution in Stern-Gouy double-layer model. Canadian Geotechnical Journal, **3**:624-636.
- Sharma, R.S., (1998). Mechanical behaviour of unsaturated highly expansive clays. DPhil thesis, Keble College, University of Oxford, pp.263.
- Sibley, J.W. and Williams, D.J., (1990). A new filter material for measuring soil suction. Geotechnical Testing Journal, ASTM, **13**(4): 381-384.
- Snethen, D.R., (1984). Evaluation of Expedient Methods for Classification of Potentially Expansive Soils. Proceedings 5th International Conference in Expansive soils, Adelaide.
- Sokolov, W.N., (1990). Engineering-geological classification of clay microstructures. Proc. 6th, International IAEG Congress, **1**: 753-760.
- Songyu, L., Heyuan, L., Peng, J. and Yanjun, D., (1998). Approach to cyclic swelling behaviour of compacted clays. Proceedings of the 2nd International Conference on Unsaturated Soils, Beijing, China, **2**: 219-225.
- Sridharan, A. and Jayadeva, M. S., (1982). Double Layer Theory and Compressibility of Clays, Geotechnique, **32**(2): 133-144.

- Sridharan, A. and Nagaraj, H.B., (2000). Compressibility behaviour of remoulded, fine-grained soils and correlation with index properties. *Canadian Geotechnical Journal*, **37**:712-722.
- Sridharan, A. and Prakash, K. (1998). Mechanism Controlling the Shrinkage Limit of Soils. *Geotechnical Testing Journal, ASTM*, **21**(3): 240-250.
- Sridharan, A. and Prakash, K. (2000a). Shrinkage limit of soil mixtures. *Geotechnical Testing Journal, ASTM*, **23**(1): 3-8.
- Sridharan, A. and Prakash, K. (2000b). Percussion and cone methods of determining the liquid limit of soils: controlling mechanisms. *Geotechnical Testing Journal, ASTM*, **23**(2): 236-244.
- Sridharan, A. and Venkatatappa Rao, G., (1973). Mechanisms Controlling Volume Change of Saturated Clays and the Role of the Effective Stress Concept. *Geotechnique*, **23**(3): 359-382.
- Stepkowska, E.T., (1990). Aspects of the clay/electrolyte/water system with special reference to the geotechnical properties of clays. *Engineering Geology*, **28**: 249-267.
- Stirk, G.B., (1953). Some aspects of soil shrinkage and effects of crack upon water entry into the soil. *Soil Science*, **48**:56-66.
- Stocker, P.T., (1969). The structure and swelling of divalent montmorillonites-a review. *Journal of Agricultural Road Research*, **3**(9): 49-71.
- Subba Rao, K.S. and Satyadas, G.C., (1987). Swelling potentials with cycles of swelling and partial shrinkage. *Proceedings of the 6th International Conference on Expansive Soils, New Delhi, India, Vol. 1, p. 137-147.*
- Subba Rao, K.S., Rao, S.M. and Gangadhara, S., (2000). Swelling behaviour of desiccated clay. *Geotechnical Testing Journal, ASTM*, **23**(2): 193-198.
- Sullivan, R.A., and McClelland, B. (1969). Predicting heave of buildings on unsaturated clays. *Proceedings of 2nd International Resource Engineering Conference: Expansive Clay Soils. Texas A and M Univ. Press, College Station, TX, p.404-420.*
- Swarbrick, G.E., (1995). Measurement of suction using filter paper method. *Proceedings of the First International Conference on Unsaturated Soils, France, Vol. 2, p. 653-658.*
- Tadanier, R. and Nguyen, V.U., (1984). Index properties of expansive soils in New South Wales. *Proc. 5th Int. Conference on Expansive Soils, Adelaide, pp.321-326.*

- Tchistiakov, A.A., (2000). Physico-chemical aspects of clay migration and injectivity decrease of geothermal clastic reservoirs. Proceedings World Geothermal Congress 2000, Japan.
- Tempany, H.A., (1917). The shrinkage of soils. *Journal of Agricultural Science*, **8**:312-333.
- Thomas, G.W. and Moody, J.E., (1962). Chemical relationships affecting the water-holding capacities of clays. *Soil Science Society of America Proceedings*, **26**:153-155.
- Thomas, G.W., (1998). Quantifying Properties and Variability of Expansive Soils in Selected Map Units. Doctor of Philosophy thesis, Virginia Polytechnic Institute, State University.
- Tripathy, S., Subba Rao, K.S. and Fruduland, D.G., (2002). Water Content-void ratio swell-shrink paths of compacted expansive soils. *Canadian Geotechnical Journal*, **39**:938-959.
- van der Merwe, D.H., (1955). Discussion: The genesis and distribution of expansive soil types in South Africa. *Transactions, South Africa Institute of Civil Engineers*, **5** (December): 434-435.
- van der Merwe, D.H., (1964). Predictions of Heave from the Plasticity Index and Percent Clay Friction of Soils. *The Civil Engineers in South Africa*.
- Viani, B.E., Low, P.F. and Roth, C.B., (1983). Direct measurement of relation between interlayer force and interlayer distance in the swelling of montmorillonite. *Journal of Colloidal Interface Science*, **96**:229-244.
- Wan, A.W.I., Gray, M.N. Graham, J., (1995). On the relations of suction, moisture content, and soil structure in compacted clays. *Proc. 1st Int. Conf. Unsaturated Soils, Paris, Vol. 1*, 215-222.
- Wan, Y., Kwong, J., Brandes, H.G. and Jones, R.C., (2002). Influence of amorphous clay-size materials on soil plasticity and shrink-swell behavior. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **128**(12): 1026-1031.
- Ware, C.I. and Jermy, C.A., (1998). Geotechnical properties of "hippo mud" in the Durban area, South Africa. *Proceedings of 8th International Congress of the International Association of Engineering Geology, Vancouver*. Moore, D.P and Hungr, O. (Eds). A.A. Balkema, Rotterdam. 2699-2704.
- Warkentin, B.P. and Bozozuk, M., (1961). Shrinkage and swelling properties of two Canadian clays. In *Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering*. Dunod, Paris, p. 851-855.

- Weston, D.J., (1977). Expansive Soils and Road Construction in South Africa. National Institute for transport and Road Research, South Africa.
- White, W.A. and Pichler, E., (1959). Water absorption characteristics of clay minerals. Illinois State Geological Survey Circular 266.
- Williams, A.A.B. and Donaldson, G.W., (1980). Building on expansive soils in South Africa. Proceedings of 4th International Conference on Expansive Soils, Denver, Colorado, Vol. 2: 834-844.
- Williams, A.A.B., Pidgeon, J. T. and Day, P.W., (1985). Problem Soils in South Africa-State of Art Expansive soils. The Civil Engineers in South Africa.
- Williams, A.A.B., Pidgeon, J. T., and Day, P.W., (1987). Expansive soils in South Africa-State of Art Expansive soils. Proceedings of the 9th African Regional Conference on Soil Mechanics and Foundation Engineering, Lagos, Nigeria, Vol.1, P. 937-57.
- Yong, R.N. and Mohammed A.M.O., (1992). A study of particle interaction energies in wetting of unsaturated expansive clays. Canadian Geotechnical Journal, **29**:1060-1070.
- Yong, R.N. and Warkentin, B.P., (1975). Developments in Geotechnical Engineering 5: Soil Properties and Behaviour. Elsevier Scientific Publisher, Amsterdam.
- Zlotchevskaia, R. I., and Korolev V.A., (1988). Electrical Surface Effects in Clay Rocks. Moscow, MGU, pp. 177.