Effect of Temperature on Hydro-mechanical Behavior of Compacted Expansive soil

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ABSTRACT

Temperature effects on the behavior of compacted soil is essential mainly in the landfill barrier materials subjected to elevated temperatures due to bio-chemical reactions of the waste contained. Previous research indicate that subjecting unsaturated fine-grained soils to temperatures below boiling point of water, properties such as volumetric shrinkage, swelling pressure and shear strength are influenced. These changes have been attributed to the thermal variations causing alterations in the physical and chemical behavior at the micro level. This research work is aimed to investigate the effect of temperature on hydro-mechanical behavior of expansive clays under temperatures of 25°C, 40°C and 60°C. The scope of this research is finding variation of hydro-mechanical behavior in physical properties and engineering properties of expansive clay under different temperatures. Experimental findings showed that thermally induced volume change under different temperatures and loads, thermal development of pre-consolidation pressure have direct influence on swelling pressure. It was concluded that elevated temperatures increase swell potential, shrinkage properties as well as compressibility and hydraulic conductivity. The unconfined compressive strength is also studied on the desiccation path of saturated soil at different temperatures and time intervals. The results revealed that both temperature and period of drying have direct influence on the compressive strength. The highest compressive strength is obtained at 4 hours drying at 60°C. The results of this research are in good agreement with those in the literature on thermo-hydro-mechanical studies on expansive clay.

Keywords: Expansive soils, hydro-mechanical properties, effect of temperature.

ÖZ

Katı atık depolama sistemlerinde şilte olarak kullanılan sıkıştırılmış kil, atıkların biyokimyasal reaksiyonlar geçirmesi esnasında yüksek ısılara maruz kalabilir. Yapılan araştırmalar ince taneli zeminlerin suyun kaynama derecesi altındaki ısılara maruz kalırlarsa hacimsel değişim, şişme basıncı ve kayma mukavemetlerinin değiştiğini gösterir. Bu değişimler zeminlerin mikro seviyedeki fiziksel ve kimyasal değişimden kaynaklanmaktadır.

Bu araştırmanın amacı şişen bir kilin hidromekanik davranışının 25°C, 40°C and 60°C ısılarında deneysel olarak irdelemeyi içerir. Araştırma kapsamında hidromekanik davranış yanında fiziksel ve mühendislik karakteristiğinin farklı ısılar altında incelenmesi de vardır. Deneysel çalışma sonucunda farklı ısı ve yükler altında oluşan aşırı konsolidasyon basıncının şişme basıncı ile direk ilişkili olduğu gözlemlenmiştir. Sonuç olarak, yüksek ısıların şişme potansiyelini, büzülmeyi, sıkışabilirliği ve hidrolik iletkenliği artırdığı gözlemlenmiştir. Sıkıştırılıp suya doygun hale getirilen nümunelerin farklı ısılarda kurutulurken, farklı zaman dilimlerinde serbest basınç dayanımları da ölçülmüştür. Bu sonuçlara gore ısının ve kuruma süresinin serbest basınç dayanımına etkisi vardır. En yüksek basınç dayanımı 4 süre ile 60°C’de kurutulduğunda elde edilmiştir. Bu sonuçlar literatürde şişen zeminler üzerinde yapılan ve ısının mekanik özelliğe etkisini içeren çalışmaların bulguları ile uyum içerisindedir.

Anahtar kelimeler: Şişen zeminler, hidro-mekanik davranış, ısının etkisi

To my Mother and Father: they are my love and heroes. Thank you for giving me the opportunity to study and for opening my eyes. I am so thankful to God for my heroes. Dear Mom and Dad, words cannot explain enough my emotions and gratitude for all you have done for me. Your love and your support has aided me not only to succeed in my studies, but also guided me in my whole life; and for that, I thank you.

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Chapter 1

INTRODUCTION

1.1 Background

It is essential to understand the effect of temperature on the hydro-mechanical behavior of expansive soils in applications of nuclear waste disposal facilities, buried electrical cables, utility pipe lines and ground thermal energy storage (Moritz 1995; Abdel-Hadi and Mitchell 1981; Davies and Banerjee 1980; Slegel and Davis 1977). These applications have made the study of thermal effects on the mechanical properties of soils and thus the thermo mechanical behavior of soils as one of the major issues of Geotechnical Engineering.

Previous researchers stated that subjecting the unsaturated fine-grained soils to high temperatures induces changes in volume and shear strength. All the earlier research works have attributed the thermo-mechanical behavior to the changes in the thermally induced physico-chemical forces within clay particles, before it was realized that heating influenced the mineralogy of the saturated fine-grained soils above 60°C. (Graham et al. 2001), Sridharan and Venkatappa Rao (1973), Morgenstern and Balasubramonian (1980), and Mitchell (1993) showed that alterations in pore-fluid valence, concentration, permittivity, and effect of temperature have significant influence on the physico-chemical forces among the clay particles, consequently controlling the mechanical behavior of fine grained soils. Thus, it is understood that variation in the pore-fluid temperature may alter the shear strength behavior and induce volumetric strains. Laloui (2001) has studied and prepared a review paper of research done on thermo-mechanical behavior of the saturated fine grained soils, where he stated that stress history factor which affects the thermally induced volumetric changes causing permanent contraction in normally consolidated expansive clays while causing reversible expansion in the highly over consolidated clays. Moreover, higher preconsolidation pressures are observed in normally consolidated expansive clays upon reloading when subjected to heating and cooling cycles and that different clay types show reduced elasticity at higher temperatures. It is also stated that hydraulic conductivity of the clays increases with temperature.

The purpose of this research is to present and compare the results of an experimental work on compacted natural expansive clay with consideration of their effect of temperature on hydro-mechanical behavior of expansive clay.

This study includes the work on compacted naturally expansive clay on difference temperature and, shrink-swell potential, compressibility and soil suction.

The materials used in this study is basically natural expansive clay taken from behind of stadium in eastern Mediterranean university of Famagusta, north Cyprus.

The laboratory experiments methods used in this research work are as listed below. Common methods are implemented using ASTM Standards of testing. On the other hand, some special tests are conducted without a specific standard.

This study consists of five chapters. Chapter 2 includes a literature survey on effect of temperature on hydro-mechanical behavior on expansive soils and information on certain topics of unsaturated soils is also given, which includes water characteristic curve, suction, historical about a problem of unsaturated soils and expansive soils.

Chapter 3 introduces the materials and methodology of this study providing complete information on applied testing methods and apparatus. Chapter 4 contains various data analysis, comparative curves, tables and results achieved from measurements of physical properties of specimen and assessment of their hydro--mechanical properties under different temperature, including swelling potential, shrinkage behavior, soil water characteristic curves and Chapter 5 consists of the overall conclusions of this study together with some recommendations for future work.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Studies on unsaturated soils are recently very popular in geotechnical engineering. This is mainly because of increasing depth of the vadoze zone mainly due to global warming, and the most important Civil Engineering activities being held in this zone. Therefore, unsaturated soils are often encountered in geotechnical or environmental engineering works (Rao and Anusha 2012).

Unsaturated clayey soils generally possess high plasticity and are very sensitive to changes in water content, undergoing excessive volume changes. These soils which increase in volume because of an increase in water contents are categorized as expansive soils. These highly plastic soils may cause cracks in road bases, railways, foundations, and structures. Thereafter expansive soils cause unsightly structural defects. Therefore, they are considered as poor material for foundations.

2.2 Unsaturated Soils

Soils located above the ground water level and compressed soils are regularly unsaturated, for the reason that of evaporation and transpiration by vegetation, they have negative pore-water pressures. Weather variations affect the top soil located nearby the surface by changes in moisture content, therefore resulting changes in shear strength and volume of the soil. Equations developed for saturated soils can be practical on soils with degrees of saturation of 85% or more. Conversely, it becomes necessary to exert unsaturated soil mechanics principles for fewer degrees of saturation (Rahardjo et al., 2002). In recent years many progresses have been made in prediction of engineering behavior of unsaturated soils. Unsaturated soils are supposed to contain capillary pores which are not full with water (Sillers et al., 2001). Unlike the saturated coefficient of hydraulic conductivity, k_s, the unsaturated coefficient of hydraulic conductivity k_w is not a constant but varies with suction (Rahardjo et al., 2002).

2.2.1 Definition of Suction in Unsaturated Soils

In general in all types of soils, samples are unsaturated instantly after the compaction. The orientation to absorb and sustain water in these samples is basically affiliate on the initial matric suction, the negative pore-water pressure in the soil due to capillary and surface sorption forces is distinctive as matric suction. This is the difference in the middle of pore-air and pore-water pressure and is a function of soil water content, soil structure and soil type.

Consequently, in areas above the ground water table matric suction at all times exists (Agus et al., 2001; Tripathy et al., 2003). Osmotic suction, liaise to the negative pressure of the soil which is relevant to the amount of dissolved salt in pore-water. In saturated soils, if it gets exposed to chemical contamination, the osmotic suction leftovers almost constant, while it can be varied significantly in unsaturated soils. As soil dries, the concentration of soluble ions increases and consequently osmotic component of suction would rise as well (Peroni and Tarantino, 2003). Total suction is expressed as the relative humidity of the pore-water steam in the soil. Total suction contain of osmotic suction and matric suction. It shall be noted that in the absence of salts and other pollution in the soil mass, can be neglected and total suction,, equals to matric suction, (Thakur et al., 2006).

Assessment of osmotic suction is a key value to match total and matric suction measurements. Once determining soil water characteristic curve application of just one method is not sufficient to cover all ranges of suction. In slight suction ranges, measurement methods are generally based on transmission of free water, for instance axis-translation technique, and just matric suction can be measured. In high ranges of suction total suction is measured and measurement techniques are based on vapor out-migration, for instance in psychrometer. It is incumbent to match matric and total suction measurements through the evaluation of the osmotic suction when the water retention curve over the entire span of the suction is being traced (Peroni and Tarantino, 2003; Boso et al., 2003).

2.2.2 Soil Water Characteristic Curve (SWCC)

The soil water specifications curve defines the relationship between soil suction and whichever the degree of saturation, S, or gravimetric water content, w, the volumetric water content, θ. The soil water specifications curve provides a noetic and interpretative instrumentation by which the behavior of unsaturated soils can be understood.

As the soil changes from a saturated state to drier conditions, the repartition of the soil, water, and air phases alteration as the stress state changes. The moisture area of interaction between soil particles reduction with a rise in the soil suction (Vanapalli and Fredlund, 2000; Fredlund and Rahardjo, 1993). Two way of wetting and drying are gain for SWCCs, which are analogous to compression and swelling curves of consolidation test, that is to say desorption (drying) way and adsorption (wetting) way. In drying process as the water content decreases, the soil suction increases and a reverse process happens in wetting process as the soil gets wet (Agus et al., 2001; Rahardjo et al., 2002).

The past studies have showed that the study of soil suction is compulsory in understanding the unsaturated soil behavior. SWCC present relationship between soil suction, ψ, and the water content, w. Additional, the usefulness of the SWCC for determining unsaturated soil properties for instance hydraulic conductivity, shear strength, compressibility and swelling potential has also been demonstrated by several researchers (Thakur et al., 2006). In case of swell potential, amount of water absorbed by the soil can be associated to suction through SWCC. As suction is decreased from its initial value, water is absorbed and thus swelling pressure develops (Agus and Schanz, 2003).

The soil water characteristic curve has been found to be a useful tool in the estimation of engineering properties for unsaturated soils, such as hydraulic conductivity and the shear strength functions (Vanapalli and Fredlund, 2000). It also contains essential information concerning the amount of water existing in pores at any suction, the stress state in soil-water and pore size distribution (Sillers et al., 2001). The SWCC indicates the space available for the water to flow through the soil at different matric suctions due to the fact that water can only flow through the water filled pores. Therefore, unsaturated hydraulic conductivity can be estimated indirectly from the saturated hydraulic conductivity and SWCC.

The most widely used SWCC fitting functions in literature are the models designed by Fredlund and Xing (1994), van Genuchten (1980) and Brooks and Corey (1964) (Thakur et al., 2006). In SoilVision database assessment of saturated and unsaturated

soil properties is accessible based on the volume-mass properties and grain-size distribution.
Croney and Coleman (1961) inchoate that the total suction at zero water content for a diversity of soils was a few of below 1000000 kPa. Fredlund and Rahardjo (1993) similarly found from the gravimetric water content versus suction correlation for various sand and clay soils that at zero water content the suction methods a value of Nearly 980000 kPa. This value is moreover supported by thermodynamic consider (Richards, 1965).

The SWCC is influenced by type of soil, texture, and mineralogy. For clay soil kind, SWCC has effect on stability limit of the clay. Fleureau et al. (2002) established a good relationship between the liquid limit and shelvy of void ratio versus suction and liquid Limit and shelvy of water content versus suction for wetting path. Marinho (2005) apply liquid Limit of clay, composed with suction capacity and stress history to generate SWCC.

Sample of a specific soil, in spite of having the same contexture and mineralogy can have not the same SWCC due to different incipient water content, void ratio, stress history, and compaction energy. Samples compacted at not the same water contents upshot in different contexture of the soil (Lambe, 1960; Gens et al., 1995; Delage and Graham, 1996). Vanapalli et al., (1999) consider an important different in the SWCC of clay until compacted at different water Content, apply the similar compaction energy. They presented that the SWCC seems to be nearly the same over suction level from 20000-1000000 kPa for samples tested with different primary water content. Soil contexture appears to have no effect on SWCC in this level of suctions.

Fleureau et al. (2002) reported that drying path of sample from slurry demonstrate the highest capability to keep water or can be get used to main drying curve of a soil. They furthermore found that the wetting direction of sample compacted optimum water content was nearly the similar as wetting direction of sample from slurry.

Al-Mukhtar et al. (1999) considered the influence of axial compaction stress on drying curve of compacted smectite. It was found that the water content against suction curve of sample having less than axial stress (i.e., 1 MPa) was located over that of sample having upper than axial stress (i.e., 10 MPa) for relative humidity (RH) level from 100% to 98% (or equipollent to total suction from 0 kPa to 2700 kPa). For RH lower than 98% (or total suction more than 2700 kPa), the drying curves of examples were resembling. Al-Mukhtar et al. (1999) offered that for RH leveling from 0 to 98%, suction is controlled via micro-pores; their extent and repartition are not effect by the compaction procedure.

At RH>98%, suction is controlled via macro-pores. The suction determined at this RH level are more susceptible to the test boundary state and variation in specimen density. Delage et al. (1998) considered the swelling properties and water retention of FoCa7 clay under controlled suction and zero applied stress. They apperceive justly reversible responses to suction term, in this time water content and volume change. For the period of these changes, it was observed that the air volume fatigued permanent they stated that the reversibility is pertaining to the prevailing role of a saturated microstructural level, strongly influenced by the physico-chemical bonds existing among water and the active clay.

Agus (2005) decided that no general main drying (i.e., the water content, void ratio and degree of saturation versus suction curves) can be defined by the experimental outcomes since the main drying curve should be found by the sample firstly slurry situation. Agus (2005) also inchoate that the as collected suction (i.e., 22700 kPa) is a limiting suction under which major reduction in suction does not induce any significant rise in the sample level of saturation.

It was also inchoate that the wetting-drying curves of the as ready sample used was reversible since the drying and wetting paths do not ever palpate the boundaries (i.e., the drying curve of specimen by saturated situation and the wetting curve of sample from oven-dried situation).

2.2.3 SWCC Models

Different soil water characteristic fitting models were proposed by soil scientists in order to model the attained laboratory data in a more smooth and complete way, some of the most popular fittings that are cited in SoilVision software program are given in this section.

Brooks and Corey equation (1964) which was the first attempt to apply an equation to describe the soil water characteristic curve (Equation 2.1).

$$w_w = w_r + (w_s - w_r)\left[\frac{a_c}{\psi}\right]^{n_c} \tag{2.1}$$

Where:

- Ww is the gravimetric water content at any soil suction,
- Wr is the residual gravimetric water content,
- Ws is the saturated gravimetric water content,
- a_c is the bubbling pressure in (kPa),
- n_c is the pore size index and
- Ψ is the soil suction.

Gardner equation (1964) is a continuous equation for the first coefficient of permeability function (Equation 2.2).

$$w_w = w_{rg} + (w_s - w_{rg})\left[\frac{1}{\left[1 + a_g \psi^{n_g}\right]}\right]$$

(2.2)

Where:

- W_w is gravimetric water content at any soil suction
- W_{rg} is residual gravimetric water content
- W_s is saturated gravimetric water content
- a_g is a soil parameter which is primarily a function of the air entry value (in kPa)
- n_g is a soil parameter which is primarily a function of the rate of water extraction from the soil once the air entry value has been exceeded.

Van Genuchten equation (1980) presented an equation with flexibility of fitting a wide range of soils by using three parameters (Equation 2.3).

$$w_w = w_{rvg} + (w_s - w_{rvg})\left[\frac{1}{\left[1+(a_{vg}\psi)^{n_{vg}}\right]^{m_{vg}}}\right]$$

(2.3)

Where:

- W_w is the gravimetric water content at any soil suction,
- W_{rvg} w is residual gravimetric water content,
- Ws is saturated gravimetric water content,
- a_{vg} a is a soil parameter which is primarily a function of the air entry value in (kPa)
- n_{vg} n is a soil parameter which is primarily a function of the rate of water extraction
- from the soil once the air entry value has been exceeded
- m_{vg} is fitting parameter

Mualem (1976) proposed some simplifications to van Genuchten's equation (1980) to reduce the number of fitting parameters from three to two (Equation 2.4).

$$w_w = w_{rm} + (w_s - w_{rm})\left[\frac{1}{\left[1+(a_m\psi)^{n_m}\right]^{\left(1-\frac{1}{n_m}\right)}}\right]$$

(2.4)

Where:

- Ww is the gravimetric water content at any soil suction,
- Wrm is residual gravimetric water content,
- Ws is saturated gravimetric water content,
- am is a soil parameter which is primarily a function of the air entry value in (kPa),
- n_m is a soil parameter which is primarily a function of the rate of water extraction from the soil once the air entry value has been exceeded .

Burdine (1953) was the other scientist who proposed some simplifications to Van Genuchten's equation (1980) and reduced the number of fitting parameters from three to two (Equation 2.5).

$$w_w = w_{rb} + (w_s - w_{rb})\left[\frac{1}{\left[1+(a_b\psi)^{n_b}\right]^{\left(1-\frac{2}{n_b}\right)}}\right]$$

(2.5)

- Where:
- ${}_w w$ is the gravimetric water content at any soil suction,
- W_{rb} is residual gravimetric water content,
- W_s is saturated gravimetric water content,
- a_b is a soil parameter which is primarily a function of the air entry value in (kPa)

- n_b is a soil parameter which is primarily a function of the rate of water extraction

  From the soil once the air entry value has been exceeded.

  Fredlund and Xing (1994) presented an equation with three parameters which fit a wide range of soils and also is modified to be more accurate in high ranges of suction (Equation 2.6).

$$w_w = w_s \left[1 - \frac{\ln\left(1 + \frac{\psi}{h_r}\right)}{\ln\left(1 + \frac{10^6}{h_r}\right)} \right] \left[\frac{1}{\left[\ln\left[\exp(1) + \left(\frac{\psi}{a_f} \right)^{n_f} \right] \right]^{m_f}} \right]$$

(2.6)

Where:

- W_W is the gravimetric water content at any soil suction,
- W_S is saturated gravimetric water content,
- a_f is a soil parameter which is primarily a function of the air entry value in (kPa)
- n_f is a soil parameter which is primarily a function of the rate of water extraction

  From the soil once the air entry value has been exceeded
- m_r is a soil parameter which is primarily a function of the residual water content
- h_r is suction at which residual water content occurs (kPa)

2.3 Expansive Soils

Expansive soils are defined as plastic soils that display high volume change when their environmental conditions are altered from dry to wet. They have the ability to swell when they are in contact with moisture and to shrink when moisture is removed. These volume changes in soils cause many problems to structures in Civil Engineering construction projects. Construction on these soils imposes problems with excessive swelling and shrinkage due to varying moisture contents. Consequently, soils tend to lift the substructure built on them or cause shrinkage settlement of structures. Expansive soils have been responsible for many structural hazards that result in great financial losses in many parts of the world. In a study conducted for the National Science Foundation, it was informed that the expansive soils cause damage to structures, exclusively under light buildings and pavements, which is much more detrimental than the other natural catastrophes, including earthquakes and floods (Jones and Holtz, 1973). Several countries in the world, including the United States, Israel, India, South Africa, and Australia, have reported infrastructure damage problems caused by the volume change of expansive soils (Nelson and Miller, 1992).

Figure 2.1: Expansive soils
Source: http://www.geology.ar.gov/

Figure 2.2: Damage of expansive soils on structure
Source: http://www.basementsystems.ca& , /http://www.structuredfoundationrepairs.com/

The annual cost of damage done to Civil Engineering structures built in expansive soils is estimated to be billions of pounds worldwide. Expansive clays are extensively distributed worldwide, and are a source of great damage to foundations and buildings. These soils can cause heavy economic losses, as well as being a source of hazard to the population (Phanikumar and Rao 2008). Therefore, a thorough investigation of the properties of expansive soils helps understanding the causes of failure of structures.

Expansive soils mostly are clay, desiccation and swelling in clay soils associate to many parameters for instance swelling potential (Osipove et.al, 1987) and (Popesco, 1980) and (Day , 1994) presented the swelling potential increment with the number of wetting and drying clays when the specimen are allowed to fully shrink to water content equal to or less than the shrinkage limit. One of the significant parameters of soil drying is temperature. With alteration of temperature other soil physical properties are also altered. For instance average evaporation rate, final void ratio, surface crack ratio increment with decreased with decreased temperature. The water content initially decreased linearly and finally reached stabilization. Alteration of temperature can reason the cracking phenomena, if the increases temperature, rate of evaporation rises with temperature, consequently soil water content reductions with this process and after that develop crack in the soil surface. Soil physical characteristics may be considered whichever as micro scale or macro scale factors. Micro scale parameters consist of two parts: mineralogical and chemical properties, and macro scale parameters include engineering properties of the soil. Significant characteristic of clay minerals in the small size of their crystals. As aforesaid temperature has an effect on clay.

Swelling soils can also be useful in applications such as the construction of low-hydraulic-conductivity liners and covers used as buffer material for nuclear waste isolation, municipal waste containment, cut-off walls for dams, etc.. The desiccation cracks which develop upon desiccation of the buffers have a significant effect on their hydraulic properties. Albrecht and Benson (2002) establish that the hydraulic conductivity of clay liner material may increase from 1×10^{-9} m/s for wet and intact soil, to 1×10^{-10} m/s for the material after cracking. The experiments of Boynton and Daniel (1985) showed that the hydraulic conductivity of a soil sample with cracks was much higher than that of the non-cracked sample. The cracks induced by shrinkage also cause zones of weakness in a soil mass which cause decreasing strength and increasing / compressibility. Structures built on clayey soils would be affected by hydro-mechanical changes caused by cracking. Consequently desiccation cracks of clayey soils attract much attention of researchers and scientists. A number of studies have been done on this topic such as Lau (1987), Morris et al., (1992), Abu-Hejleh and Znidarcic, (1995),Miller et al., (1998), Yesiller et al., (2000), Nahlawi and Kodikara (2006). Natural soils are highly complex, being influenced by a large number of factors such as. mineral composition, clay content, relative humidity, layer thickness and size, boundary and compaction condition, which influence the formation of desiccation cracks (Albrecht and Benson, 2001; Nahlawi and Kodikara, 2006; Rodríguez et al., 2007; Tang et al., 2007; Tang et al., 2008). However, the mechanism of desiccation cracking is still not well understood. It is generally observed that the water content variation is the most important parameter that controls the initiation and propagation of desiccation cracks. But the o rate of evaporation is directly related to temperature and relative humidity. Temperature has a direct impact on water content.

2.4 Shrinkage of Expansive Soils

At the micro-scale it may be considered that the basic process behind the shrinkage of a soil is decrease in liquid pressure and hence increase in suction caused by the evaporation at the interphase menisci, acting as an attractive force between the components of the soil matrix. Initially in the saturation phase the menisci are located at the external boundary of the soil body, hence resulting in application of suction at the boundary and an increase of effective stress throughout the soil matrix causing shrinkage of the sample.

When an initially saturated soil is under drying conditions, water-air menisci start to occur at the soil surface. As a result, capillary suction in the upper layer is developed. This capillary suction generates first an arrangement of soil particles and thus a significant volume change. Note that during this process, the soil is remaining saturated. When a certain degree of consolidation is reached, the arrangement of soil particles stops and with further water evaporation, air starts to enter the deeper layer of the soil .Desiccation macro-cracks are likely to be limited by the drying shrinkage (Corte and Higashi, 1960). Train or boundary conditions of friction or any other: In general, these restrictions of various causes (Huckel, 1992) are presented. The stress concentration in the soil air, and inherent in heterogeneous, such as the ground soil structure factors. In this context, because (i) arising from deterrence, and therefore gradient of soil moisture, which do not respect the strain compatibility conditions (see e.g. (Kowalski, 2003).This work makes (i) and (ii) has been paid. Regardless of the internal structure of the earth, allowing the ground to move the item and not the dry conditions of adjacent elements not merely. In early studies, the generation and stress the following changes in the geological properties of land, located in the dry state. Since fluid loss, soil, its capacity generated by the mechanism described above eliminate lose traction forces. In addition to facilitating the accumulation of stress, which ultimately failed, stretching and tearing (Laughter, 1961).

2.5 Effect of Temperature on Soil Response

Morris et al. (1992) showed that cracking in soils during desiccation is controlled by soil suction which is similar to other soil physical properties, such as compression index, Poisson's ratio, shear strength, tensile strength, and specific surface energy. Existence of small pores in clays cause higher suctions hence they are more sensitive to the increase of desiccation cracks. Cracks occur due to tensile stresses induced by soil suction exceeding the cementation strength of particles. Kayyal et al. (1995) stated that temperature and relative humidity controls rate of moisture evaporation, hence the rate of increase in the suction potential. Higher rate of tensile stress increase occurs on the surface layer, when soil samples are desiccating at higher temperatures. The desiccation cracking behavior is therefore strongly temperature dependent. Many other soil properties related to shrinkage and desiccation cracking behavior are also directly influenced by temperature. There are many other factors related to desiccation cracking behavior of clayey soils. Although there has been a lot of work done on this subject in the last several decades, the relationship of crack parameters with temperature is still not understood well at present. The micromechanical interactions between soil and water phases should be studied thoroughly and coupled thermo-hydro-mechanical behavior should be taken into consideration.

Tang and Cui (2005) studied the behavior of clay at temperatures ranging from 20°C to 60°C. They have observed that a rise in temperature reduced the soil water holding capacity. However, because of the difficulty of crack beginning and propagation, there are so many reasons linked to desiccation cracking performance of clayey soils. While so many researches have been done during the past decades, altering relationship of crack limits with temperature is difficult to understand.

2.5.1 Temperature dependence of water evaporation rate

The rate and duration of water evaporation depends on various factors such as temperature, relative humidity, wind velocity, solar radiation, soil suction, salt concentration, soil pore size and layer thickness, etc. (Kayyal, 1995; Cui et al., 2005 ; Prat et al., 2006; Rodríguez et al., 2007). At higher temperatures, the water molecule motion velocity and kinetic energy are higher, and the viscosity, the interfacial tension of water and the water retention capacity of soil are lower (Tang and Cui, 2005). Therefore, soil water molecules escape more easily to the atmosphere at higher temperatures, resulting in higher water loss rate.

Nevertheless, the temperature itself cannot be regarded as the unique variable controlling water evaporation rate in this investigation. Indeed, according to Kelvin’s Law, a higher temperature corresponds to a lower relative humidity. In other words, at a higher temperature, the relative humidity gradient between soil surface and the upper air layer (namely the vapor pressure gradient across soil-air interface) is higher. This can equally explain the higher evaporation rate observed at higher temperatures.

2.5.2 Temperature effects on physical properties of water

Water is one of the components related to soil behavior that are influenced by temperature. Physical properties of water such as density, vapor pressure, viscosity, dielectric constant and surface tension are changed by changing temperature. Weight of water at high temperature is lower than the weight of water at low temperature at the similar volume. Furthermore it is observed that water vapor pressure rises by increasing temperature. The vapor pressure is related directly to relative humidity which explains the amount of water vapor that exists in a gaseous mixture of air and water. At constant relative humidity, rise in temperature results in increasing amount of water vapor in air. Water viscosity decreases by increasing temperature and viscosity is commonly understood as thickness or as resistance of a liquid to flow. Consequently, an increase in temperature results in reduction of resistance of a liquid to flow. When high temperature water and low temperature water are located in identical capillary tubes and allowed to flow below the effect of gravity, low temperature water takes longer time to flow through the tube than high temperature water. Rise in water viscosity due to rise in temperature is the cause why permeability of compacted soil rises with the increase in temperature.

2.5.3 Temperature effects on pore-water chemistry

Pore-water in expansive clay soils include dissolved salt due to very high concentration of cations on the surface of the clay to equilibrate its negative charge when the clay is in dry state. The soluble salt in the soil pore-water produces osmotic suction. The osmotic suction is achieved by the total suction, salt solution and is influence by the concentration of the solution. From relative humidity records, the total suction of salt solution can be determined using Kelvin equation (Thomson, 1871) that is the thermodynamic connection between total suction and relative humidity of the vapor in the locality of soil (Sposito, 1981).

2.5.4 Temperature effects on clay mineralogy

The increment in temperature may cause the smectite mineral to become unbalanced and transform to more constant silicate phases known as illitation. This similarly results in reducing the ability of the clay to retain water. The kinetics of the smectite to illite response are strongly related to temperature, time, and K^+ pore-water doping (Wersin et al., 2006).

De la Fuente et al. (2000) have stated that the transformation to illite happens when clays undergo heating for 6 months to 1 year at temperatures of 120-160 °C, which is much higher than the maximum temperature for clays (80 °C) not to transform into illite. Drief et al. (2002) investigate that the transformation of smectite to illite also happens when smectite experiences temperatures up to 50°C for 30 days in K-enriched sea water solution.

2.6 Temperature Effects on Hydro-Mechanical Behavior of Clays

2.6.1 Water retention behavior

Tang and Cui (2005) studied temperature effects on water retention behavior of compacted sodium bentonite using vapor equilibrium technique in desiccator. They have concluded that water retention curve of compacted bentonite at 20°C was above the water retention curve of compacted bentonite at 80°C. Therefore, it can be concluded that increase in temperature results in decreasing ability of bentonite to retain water. They have also found that the suction change rate by increasing temperature is -1.1×10^{-3} (Log MPa/°C). Using vapor equilibrium technique, Romero et al. (2000) performed a study on the effects of temperature on the water retention behavior of Boom clay. The results of this study indicated that for a given water content total suction at 20°C was higher than that at 80°C. The change in total suction obtained from the experiment was higher than the change in total suction due to the change in the surface tension of water. Romero et al. (2000) stated that the differences are not only due to the change in the surface tension but also due to the change in the clay fabric and the pore water chemistry of the clay. The change in the clay fabric and pore-water chemistry due to temperature is expected to be irreversible (Romero et al., 2000).

2.6.2 Temperature effects on swelling strain

Temperature effects on the swelling strain or swelling under load have been investigated by some researchers (Komine and Ogata, 1998; Romero et al. (2005); Villar and Lloret, 2004). For the same vertical load (500 kPa), Romero et al. (2005) reported that increase in temperature from 30-80 °C resulted in reduction in swelling strain of compacted clay.

Under different vertical loads (500, 1500, and 3000 kPa), Lloret and Villar (2004) observed that the decrease in swelling strain due to rise in temperature reduced by raise in vertical load. This decrease is due to reduction in hydration in amongst the primary layers which is the dominant mechanism in the swelling (Pusch et al., 1990).

2.6.3 Temperature effects on swelling pressure of compacted clay

Previous surveys on the effect of temperature on swelling pressure revealed that temperatures induced rise in the swelling pressure of clay. Pusch et al. (1990) presented that the rise in temperature of a bentonite reduced the hydration force due to a decrease in the hydrates in the smectite surface within bentonite and increased the osmotic pressure in the molecular body. Therefore, the decrease or increase in the swelling pressure of compacted bentonite depends on the prevailing factors occuring in the type of bentonite used. This is analogous to the heat-induced increase in osmotic pressure in ionic or molecular system which leads to increase in the swelling pressure. Interestingly, Cho et al. (2000) reported that increase in temperature results in increasing swelling pressure of Ca-bentonite used in their study. Cho et al. (2000) also stated that besides the balance between the two components (i.e., hydration force and osmotic pressure), increase in temperature also results in increasing pore-water pressure due to the differences of thermal expansion of the pore water and the skeleton. However, there is no information about pore water chemistry and osmotic suction of the bentonite used in the study performed by Cho et al. (2000).

2.6.4 Temperature effect on desiccation cracking behavior

Desiccation cracking is mainly governed by soil suction and tensile strength; when the tensile stress induced by suction increase exceeds the tensile strength, cracks occur. Unfortunately, unlike the temperature effects on compressive shear strength (see for instance Mitchell, 1964; Cui et al., 2000; Villar and Lloret, 2004), to the author's knowledge, the temperature effects on the tensile strength have not been studied. Previous results showed that soil shear strength in general decreases with temperature (Tang and Shi 2010) (Michell, 1964; Cui et al., 2000); this leads to expect a decrease of tensile strength with temperature increase. As far as the suction change is concerned, as indicated by Kayyal et al. (1995), the rate of suction increase is directly related to the evaporation rate. Therefore, at higher temperatures, higher suction increase rate can be expected (Chao-Sheng Tang1).

Chapter 3

MATERIALS AND METHODS

3.1 Materials Used

The soils used in this study are expansive clay taken from behind of stadium of Eastern Mediterranean University, Famagusta, North Cyprus. A wide testing program was performed in examining the physical properties and thermo-hydro-mechanical behavior of the materials. Laboratory tests were conducted mostly conforming to standard procedures of American Society for Testing and Materials (ASTM).

3.2 Test Methods

3.2.1 Specific Gravity (Gs) (ASTM D854-92)

Specific gravity (G_s) is defining the classic technique of measuring the density of aggregates and soils which is the proportion of density of material to density of water. For determination of G_s of natural expansive clay 50 g of oven dried specimen was taken and placed in 50 ml standard bottles and inundated with distilled water for 24 hours. Regarding low coefficient of permeability of clay, the saturation of specimen could only be achieved by accurate elimination of the air bubbles by a vacuum pump. The specific gravity of the expansive soil used in this study is determined to be 2.62.

3.2.2 Hydrometer Analysis (ASTM D422-54T)

The portion of the soil with particle size less than 0.075 mm, which could not be separated by means of sieves are analyzed by their rate and time of sedimentation under gravity using Stoke's Law. This test was carried out using 50 g of dried specimen, 100 g of sodium hexametaphosphate solution. The soil sieved through sieve number 200 (0.075 mm) was poured into a cylinder and filled with distilled water to 1000 ml volume. A hydrometer was inserted into the cylinder gradually and readings were recorded at certain time intervals which continued for at least 3 days.

3.2.3 Atterberg Limits (ASTM D4318-98)

The liquid limit and plastic limit of soils are frequently referred to as the Atterberg limits, which are used to classify fine grained soils or fine grained segment of soils. Albert Atterberg (1846 – 1916) presented basically six limits of stability in fine-grained expansive soils which are (a) upper limit of viscous flow, (b) liquid limit, (c) sticky limit, (d) continuity limit, (e) plastic limit and (f) shrinkage limit. Nowadays in engineering studies, the only Atterberg limits used are the liquid limit (LL), plastic limit (PL) and shrinkage limit (SL). Universally the water contents at which the soil passes form one state to another are called consistency limits and are expressed as water content in percentage (w%).

The liquid limit is theoretically defined as the water content at which the behavior of a clayey soil changes from plastic to liquid state. The factual liquid limit experiment of Atterberg's contained mixing a pat of expansive clay in a round-bottomed brass bowl of 10–12 cm diameter. A groove was cut through the pat of expansive clay by a grooving tool, and the bowl was mechanically lifted and dropped until 13 mm of closure of the groove occurs in its middle. This procedure is repeated at increasing water contents and the water content corresponding to 25 blows is taken as the liquid limit.

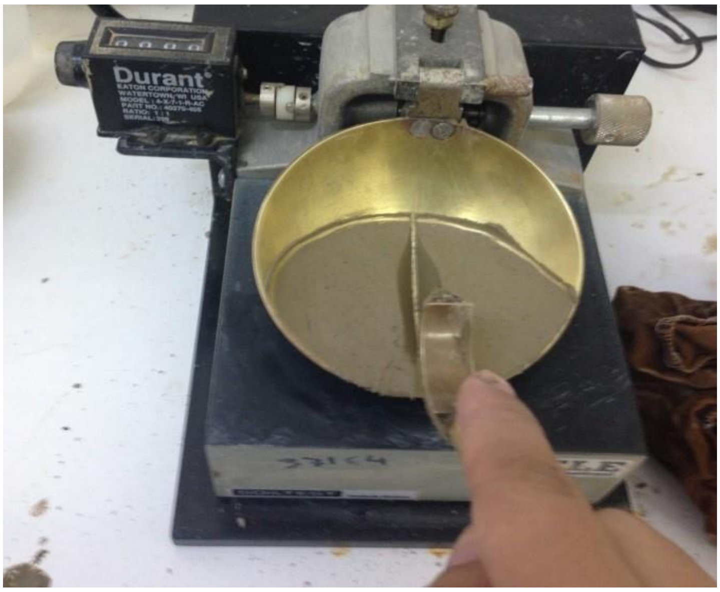

Figure 3.1: Liquid limit

The plastic limit experiment was carried out on the similar samples used for liquid limit test. Around 2 g of soil was rolled to 3 mm diameter and the water content is taken by oven drying at 105° C.

3.2.4 Laboratory Compaction Characteristics of Soil

Standard compaction experiment was used to estimate the relationship between water content and dry unit weight of specimen through the compaction curves. It was performed by automatic dynamic compactor using a hammer of 2.5 kg falling from 305 mm height and generating a compressive energy of 600 kN-m/m3. The water content after each compaction was found by taking samples from the compacted specimens.

The compaction is applied after 24 hours of mellowing time of soil kept in tightly sealed nylon bags. The moulding water content and dry density relationship is plotted as the compaction curve. The maximum dry density and the corresponding water content (optimum) is obtained as the peak value of compaction curve.

Figure 3.2: (a) Wet soil prepared and mellowed in nylon bag, (b) Dynamic compactor

3.2.5 One –dimensional swell test

One-dimensional swell tests were held to determine the swell potential of each mix. The specimens were compacted at their optimum water contents and maximum dry densities and left below 7 (kPa) surcharge pressure in metal rings of 50 mm diameter and 20 mm height. Nearly 5 mm of space is given from the top of each ring to allow to swell for one- dimensional swell.

From each admixture a number of identical specimens were prepared and placed in consolidation cells enclosed with porous stone and filter paper at the bottom and top to permit the passage of water through specimen while restricting dispersion of specimens. The specimens were then left to swell under inundation with distilled water. The gauge readings were recorded at determined time intervals for the first 120 minutes followed by taking readings twice a day until the swelling ceased. A series of tests were conducted at room temperature and at elevated temperatures of 40°C and 60°C to study the effect of temperature on one-dimensional swell behavior.

Figure 3.3: Consolidometer used in one-dimensional swell test.

3.2.6 Volumetric Shrinkage Test

When swelling test finished, the specimens were taken out from water and located in room temperature (20-25º C) for observing the desiccation shrinkage at room temperature. Specific care was taken to hold the drying conditions like to wet conditions trusting that soil conditions would not diverge for the duration of these two phenomena.

Therefore, the curb rings and during the drying method in order to emulate the natural situations and also to control the level of moisture loss. The height of selected samples, mass and average diameter were measured every day until the remaining water content was attain; that is until no further evaporation occurred at 25°C several shrinkage relationships were established based on the shrinkage mensuration, the most important of which is the shrinkage curve of void ratio vs. water content during drying. At the end of swelling time (7 days), the specimen were taken out from apparatus and left in oven for desiccating part at 40ºC and another sample left in oven at 60 ºC. Specific care was taken not to lose even small parts of soil fragments during the desiccating process so as not to effect the mass of the specimen. The average of height and diameters, also the weight of mass for each specimen (sample drying under room temperature, 40 c and 60 c) were measured daily till the remaining water content was attain, that is, no further soil water evaporated. The results are shown in chapter 4. Many relationships were established for shrinkage measurements on the shrinkage based.

Figure 3.4: Volumetric shrinkage

Figure 3.5: Consolidation under different temperatures

3.2.7 Suction Measurements

Soil suction evaluation methods are classified in three approaches, primary which measure the total suction, other approaches which measure total and matric suction and last methods which only measure the matric suction. In this study primary technique (chilled-mirror psychrometer and VET) was applied to assessment total and matric suction of specimen in order to design the SWCC of compacted natural expansive clay.

3.2.8 Vapor Equilibrium Technique Using Salt Solution

The vapor equilibrium technique or (VET) was used to measure drying and wetting curve of compacted sample for suction higher than 2000 kPa. If the temperature variation can be sustain as high as 0.5 °C, this method can be used to control suction for level higher than 1000 kPa. In this study, one types of desiccator (small desiccator) was used. The small desiccator (special glass desiccator with fastened cap) was used for experiment just one sample. The use of small desiccator can eschew the influence of the variances in initial suction of the samples since just one sample was located in the desiccator. For using the small Desiccator, the salt solution used was half of its volume.

This effects in proportion of air (5.8 cm^3) per cm^2 of solution surface is smaller than the maximum value that suggested by ASTM E104-85 (i.e., 25 cm3 /cm2) (ASTM, 1997). Several moll and saturated salt solutions have been used to influence whole suction for specimen by varying relative humidity of vapor space in desiccator.

This study has used VET method to get wet and dry curves in unconfined condition at (25, 40, and 60 °C).

Figure 3.6: Vapor equilibrium technique

The test has performed by using small desiccator since it has got only one specimen, equipped and light with fastened cap. The molal suction solution (NaCl) was calculated based on Lang (1967) equations and its data reported by Pelper and Pitzer. The test started at 60 °C by specimen preparation. Statically compact have been used for specimens to reach the predicted dry density. Specimens have been placed in small desiccator. The desiccator was placed (with the solution and specimen) in climate chamber which can control the temperature to 0.1 °C. The chamber can control the 200 kPa suction with the 0.1 °C temperature. 500 kPa is the lowest suction that use by VET in this study. The temperature of chamber increased incrementally by 1 °C per hour due to preventing temperature variances among air in glass, specimen and salt solution. By using a caliper the specimens dimensions have been measured as well as their mass. By the time, the specimens mass became constant, assumed equilibrium had been reached.

3.2.9 One-Dimensional Consolidation (ASTM D 2435-96)

One-dimensional consolidation method prepare an evaluation of degree and level of settlement, while the specimen is confined laterally, permit vertical drainage below surcharge which is a gradually applied controlled load. Apply loading stage of consolidation test was started when that the soil has attain its maximum swell. The sequence of loading was based on $\Delta P/P = 1$, and each load increment was applied after 24 hours. This investigate procedure was frequentative two periods in this research program. In the first set of testing the readings were taken by a digital data logger of Wykeham Ferrance, , and in set of testing, deformation transducers and independent Data attainment Unit were used to record and investigate the data by Data System 7 (DS7) software of ELE international Inc.

Figure 3.7: Consolidation test

Figure 3.8: VET under different temperature

3.2.10 Measurement of Hydraulic Conductivity

The hydraulic conductivity of a specific soil is the determination of its potential to permission the flow of a fluid through it. This fluid can be either gas or liquid. Usually the geotechnical researchers are interested just by liquid permeability and the main liquid is normally water.

Designation of Hydraulic Conductivity with Consolidation experiment since content of settlement in a sample subjected to extra charge load has a direct relationship by the ratio of drainage of water through that specimen, thus the saturated hydraulic conductivity (k_s) could be determination indirectly from the consolidation test results.

The unsaturated hydraulic conductivity was as well as assessed applying the gain ks data in Fredlund and Xing (1994) and van Genuchten (1980) Eq. matching to their relative SWCC fitting pattern in Soil Vision softwear.

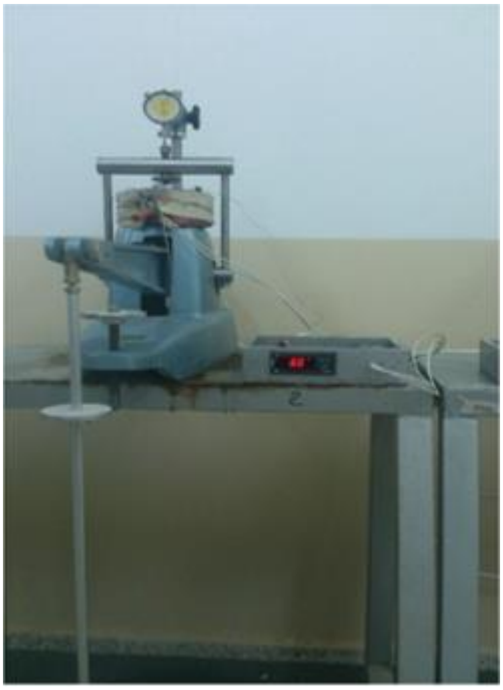

Figure 3.9: One-dimensional consolidation test under different temperatures

3.2.11 Unconfined compression test

The objective of this test is to characterize the unconfined compressive strength of a cohesive soil specimen. We will determine this by the unconfined compression test.

The unconfined compression test is with far the most famous technique of soil shear testing for the reason that it is one of the fastest and inexpensive techniques of evaluating shear strength. This technique is used mainly for saturated, expansive soils improved from thin-walled sampling cylinder. The unconfined compression test is improper for dry expansive clay under different temperature for the reason that the materials would decrease separately without some land of lateral confinement.

Figure 3.10: Preparation part in sand path for unconfined

Figure 3.11: Unconfined compressive

To perform an unconfined compression test, the sample is extruded from the sampling tube. first, I should saturated samples in sand bath, preparing sand bath consist of washing sand and put the sample in sand bath after covering all samples with specially paper for prevention swelling samples, mixed sands and specimens. A cylinder-shaped specimen of clay soils is make such that the ends are reasonably flat and the length-to-diameter level is on the order of two.

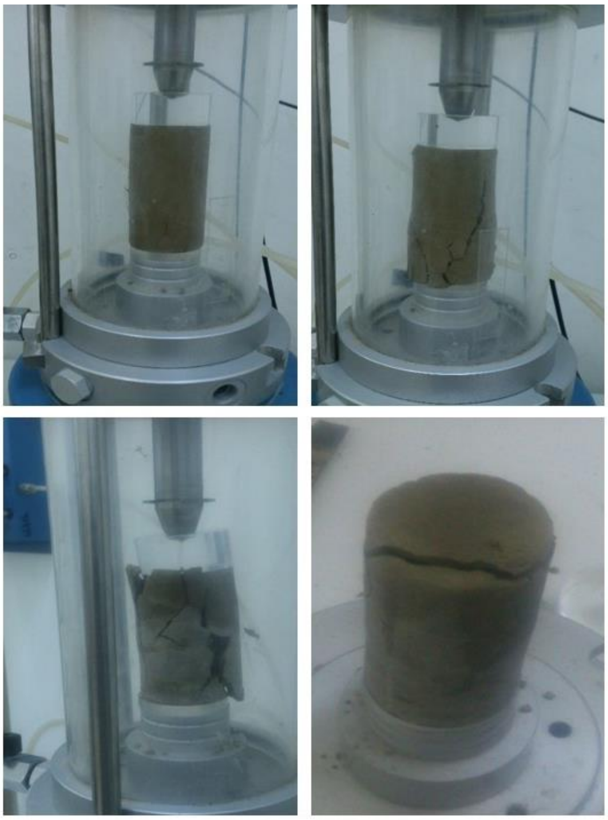

Figure 3.12: Clops sample under load in unconfined compressive

The soil specimen is located in a loading framework on a metal plate; with turning a crankshaft, the operator increases the level of the bottom plate. The bottom of the soil specimen is restrained by the bottom of plate, which is dependent to a calibrated proving ring. As the top of plate is increases, a vertical load is applied to the specimen. The operator turns the crank at a determined ratio so that there is constant strain level. The apply load is gradually increased to shear the specimen, and readings are recorded from time to time or periodically of the force applied load to the specimen and the resulting about deformation and collapse deformation.

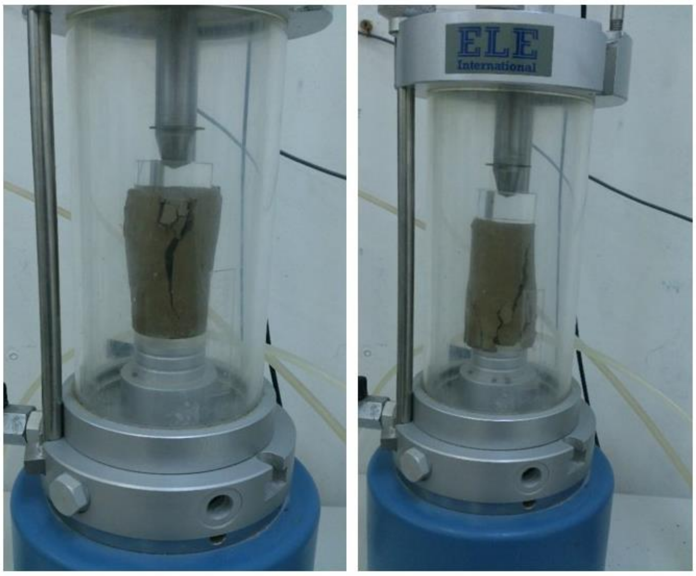

Figure 3.13: Unconfined compressive

3.2.12 Soil Vision Program

Different soil water specifications fitting models were suggested by soil researchers in command to model the attained research laboratory data in a more flat and complete way, some of the most favorite fittings that are cited in SoilVision software platform are given in this section. Mualem and Vankenachten which was the imprimis effort to apply an equation to describe the soil water characteristic curve. Software (Soil Vision 3.34) is an information-based system that different fitting functions can develop the SWCC and calculating suction data.

Figure 3.14: Soil vision program
Source: www.sngsoft.com

Chapter 4

RESULTS AND DISCUSSIONS

The first part of the experimental program contained of physical properties of the selected expansive clay soil. The second part contains volume change research, consisting of swelling, shrinking part and compressibility tests on the natural expansive clay soil in normal condition and under different temperature.

4.1 Specific Gravity

Specific gravity of a particular kind of soil has an important role in estimation of its void ratio. Value of G_s is similarly linked with shrinkage properties and hydrometer analysis closely. In this study it was calculated carefully according to ASTM standard 854-to be is 2.62.

4.2 Physical properties

This table displayed the physical properties of expansive clay soil.

Table 4.1: Physical properties of the expansive soil

Physical properties	
Liquid limit (%)	61
Plastic limit (%)	30
Plasticity index (%)	31
Linear shrinkage (%)	19.2
Specific Gravity, G_S	2.62
Silt	20%
Sand	10%
USCS classification	CH

4.3 One-mensional Swell Test Results

One-dimensional swell experiment results are given as plots of percent swell ($\Delta H/H_0$ x100) versus logarithm of time. These curves are plotted with respect to logarithm of time in order to be able to see the completion of primary swell part. Primary swell percentage is the major component of the total swell percentage. Total swell is consisted of initial, primary and secondary parts. The secondary swell happens gradually and usually completes in a very long time period. Therefore secondary swell is given in terms of slope of the secondary swell part of the swell curve. Figures 4.1-4.3 display swell curves obtained under 25° C, 40° C and 60° C respectively, and Figure 4.4 shows the comparison of all. Table 4.2 displays the swell parameters obtained from swelling test under different temperatures. It can be observed that is (%) increases with increasing temperature, also t_{is} (min), s_p (%) and t_{ps} (min) increase with increasing temperature. Therefore, temperature influences the swell potential, primary swell percentage being almost the same at both 25°C and 40°C, while it increases by 30% under 60°C. Table 4.2 shows primary swell (S_w%), time for primary swell (t_s) and for each sample.

Table 4.2: One-dimensional swell parameters under different temperatures (25° C, 40° C and 60° C)

Temperature	s_i (%)	t_s(min)	s_p(%)	t_{ps}(min)
25°C	0.9	10	5.2	128
40°C	1.2	14	5.5	140
60°C	2.5	16	7.2	170

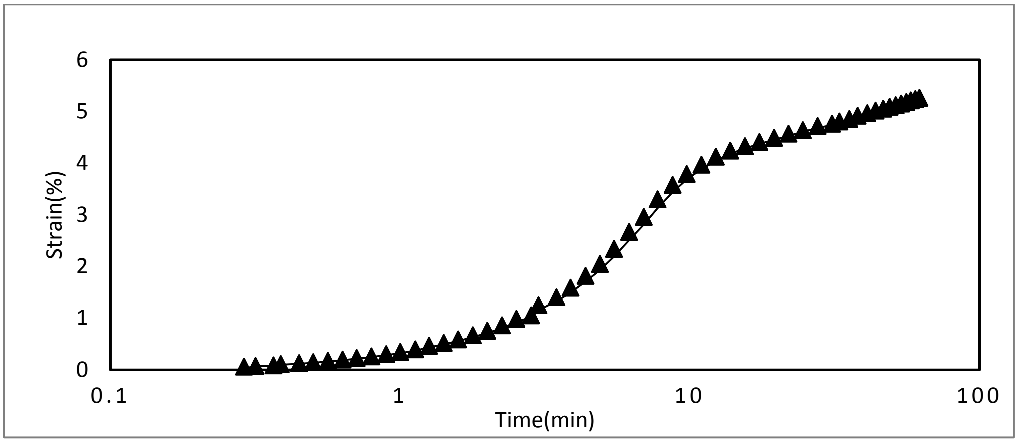

Figure 4.1: One-dimensional swell curve at 25°C

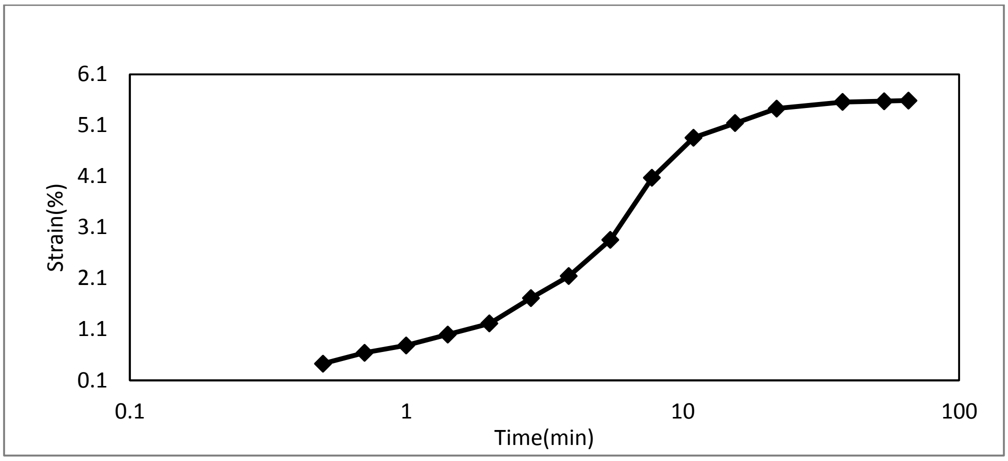

Figure 4.2: One-dimensional swell curve at 40°C

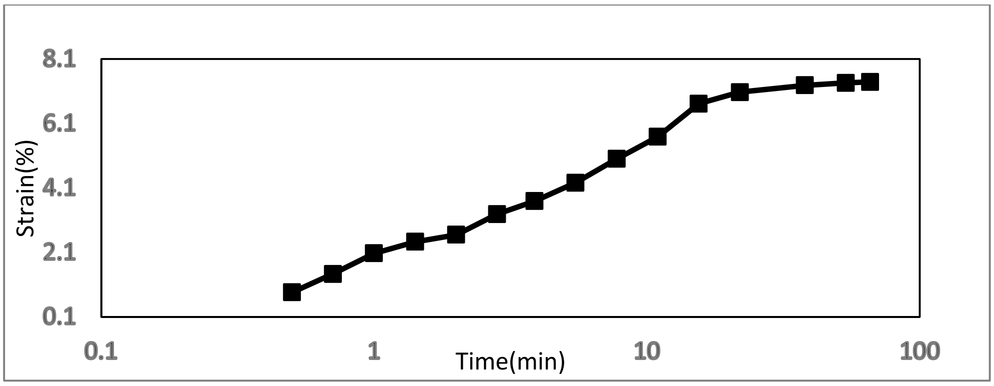

Figure 4.3: One- dimensional swell curve at 60°C

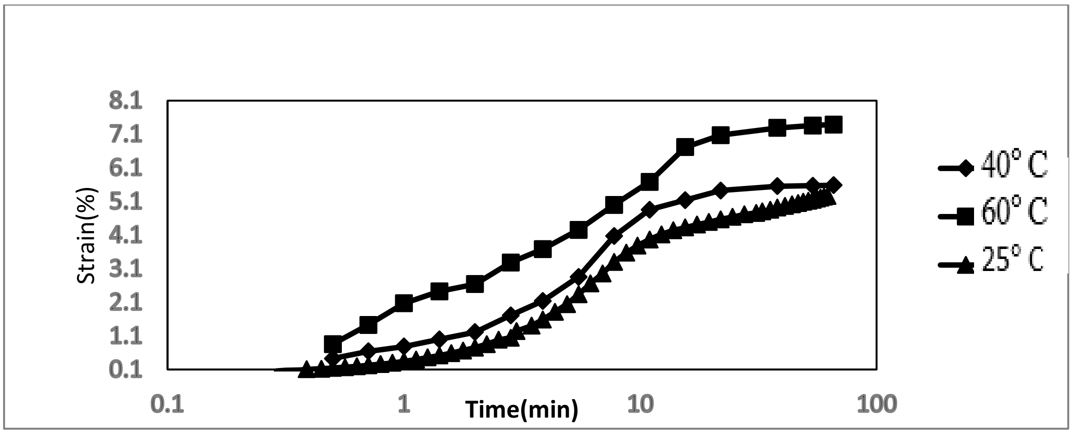

Figure 4.4: One-dimensional swell curves under different temperatures

4.4 Volumetric Shrinkage Test Results

After completion of one-dimensional swell tests, the specimens were dried at 25°C, 40°C and 60°C and the diameters and heights were measured at certain time intervals to study the variation of volumetric, axial and diametral strains with time along the shrinkage path. The desiccating samples are observed to have different textures due to the effect of different temperatures. Figures 4.4-4.6 present the volumetric, axial and diametral strains versus time plots respectively at different temperatures. It can be observed that all the shrinkage strains increased and ceased much faster with increased temperatures. Therefore saturated soil shrinks faster and with higher magitudes when subjected to higher temperatures. This behavior is in good agreement with the swelling behavior of specimens, higher the swelling capacity, higher is the shrinkage at higher temperatures. The clay water absorption increased with temperature hence there is more water absorbed, hence more water space to be lost upon drying, therefore more shrinkage.

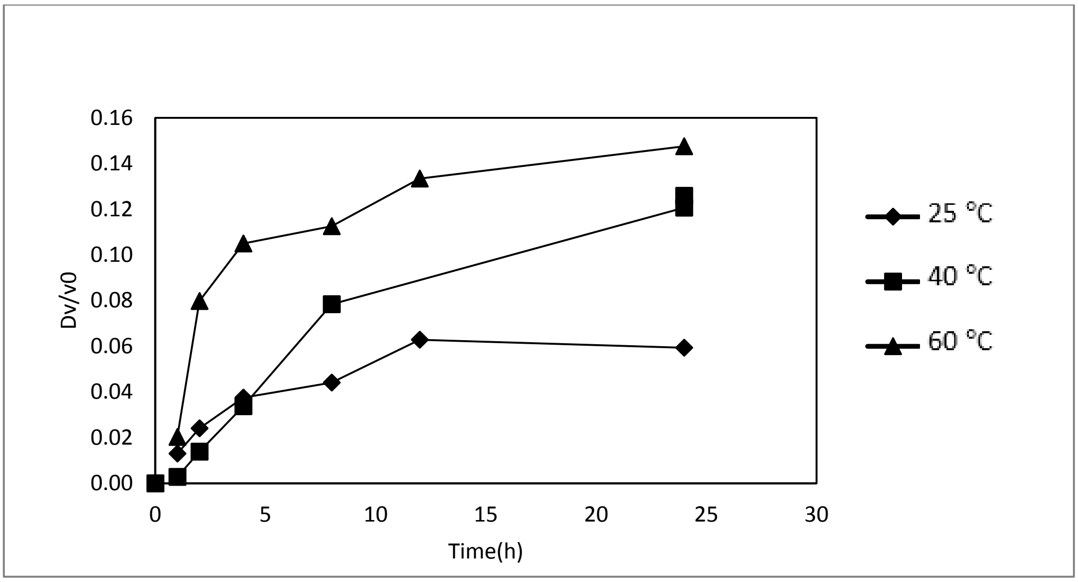

Figure 4.5: Volumetric shrinkage-time relationship under different temperatures

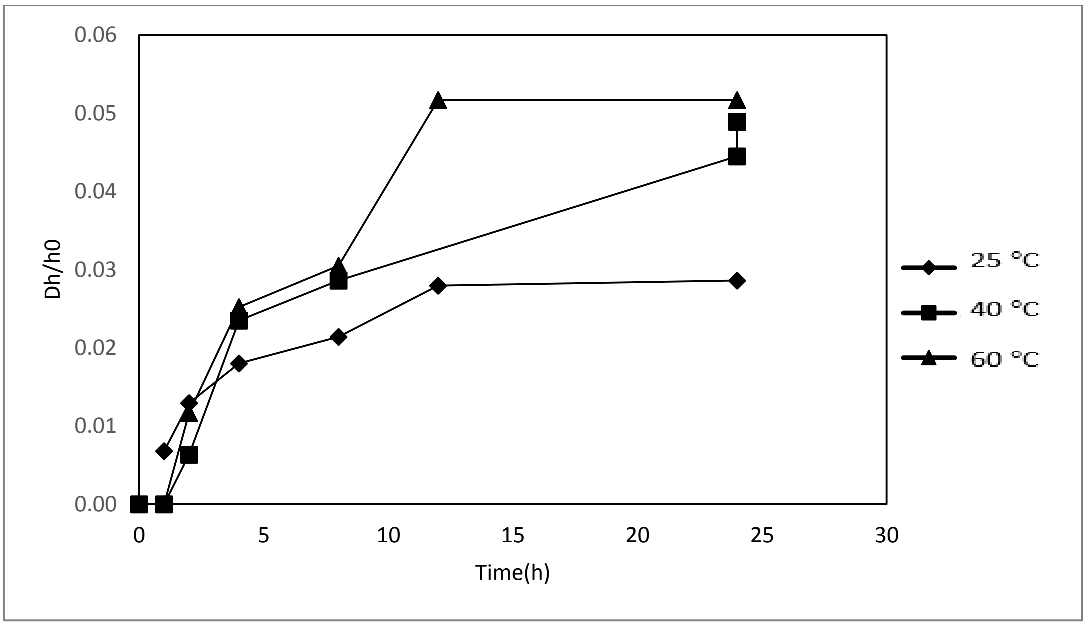

Figure 4.6: Axial shrinkage-time relationship during drying under different temperatures

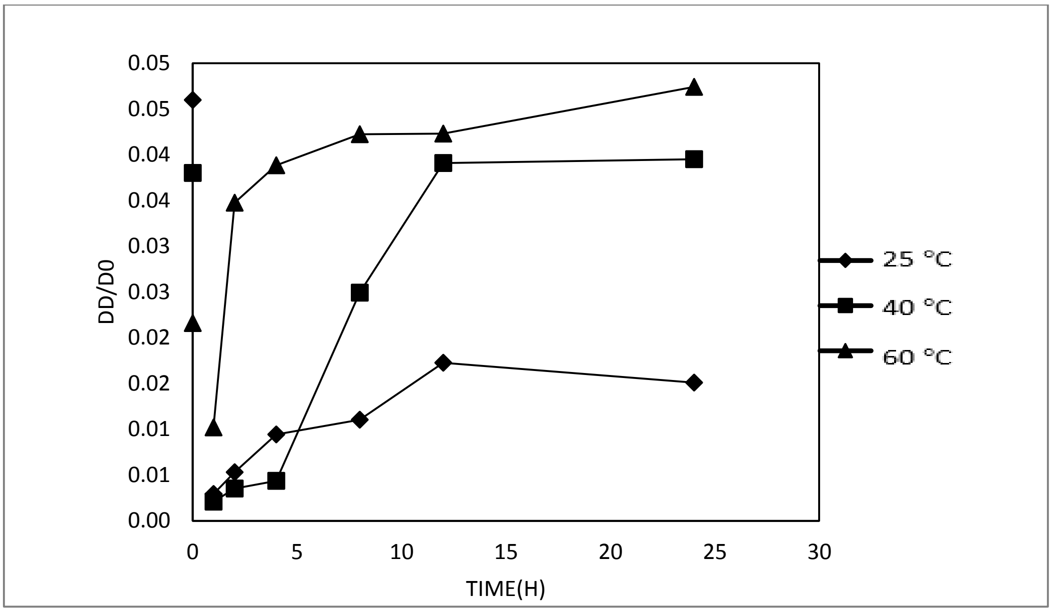

Figure 4.7: Diametral shrinkage-time relationship during drying under different temperatures

Figure 4.6 shows the relative change in vertical normal strain ΔH/H0 as a function of time and figure 4.7 shows the relative change in horizontal strain ΔD/D0 with respect to time and finally figure 4.5 display the relative change in volumetric strain ΔV/V0 as a function of time .Figure 4.8 presents the shrinkage curves and hyperbolic fitting curve fitted by Soil Vision software. Table 4.2 displays the shrinkage curve fitting parameters.

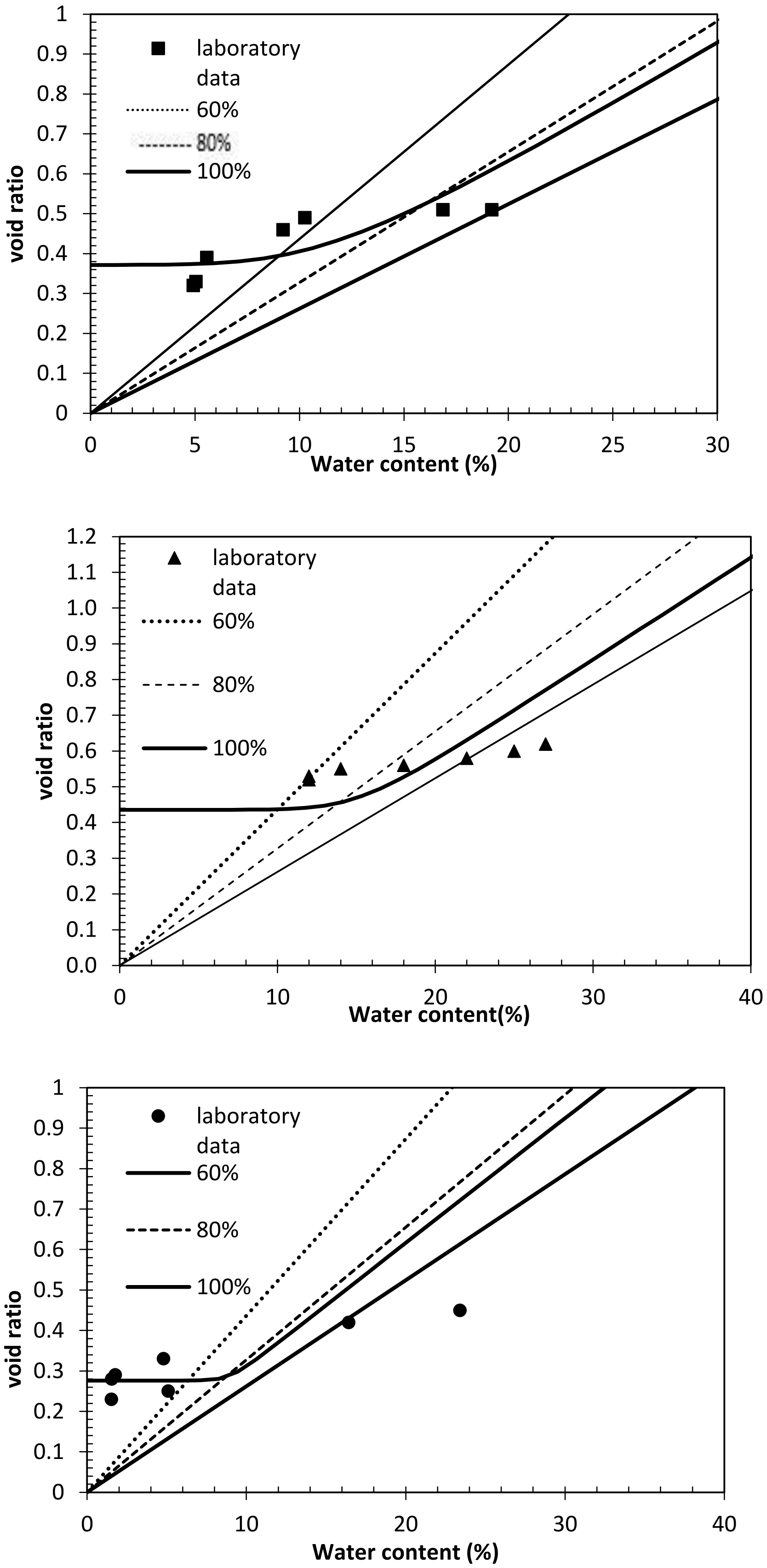

Figure 4.8: Shrinkage curves of specimens drying at (a) 25°C, (b) 40°C and (c) 60°C

Studying the parameters in Table 4.3, it can be concluded that the shrinkage limit is reached at a lower water content that is the desiccation shrinkage ceased at much lower water content at higher temperatures since the rate of drying is faster. The reason for the final void ratio to be smaller at higher temperatures is due to the higher volumetric shrinkage strains at higher temperatures. Therefore, all the swell-shrinkage findings are compatible with each other.

Table 4.3: Shrinkage parameters of hyperbolic model from Soil Vision software.

Temperature	a_{sh}	b_{sh}	c_{sh}	Shrinkage limit
25°C	0.44	0.197943	8.47	0.20
40°C	0.37	0.1206	4.28	0.12
60°C	0.28	0.0896	15.21	0.09

The parameter ash represents the minimum void ratio the dried specimens attained, and the b_{sh} values are the minimum water content values at which volume change commenced and is equal to the slope of the degree of saturation line on the shrinkage plot. The c_{sh} is curvature of the shrinkage curve and is referred to as shrinkage limit. Studying the parameters in Table 4.3 it can be observed that there is a significant decrease in volume change.

4.5 One-dimensional Swell-Consolidation Test Results

Results of this test are given as void ratio versus logarithm of effective consolidation pressure. The parameters obtained from these curves, are the compression index (C_c), swell index (C_s), swelling pressure (p_s') and pre-consolidation pressure (σ_p').

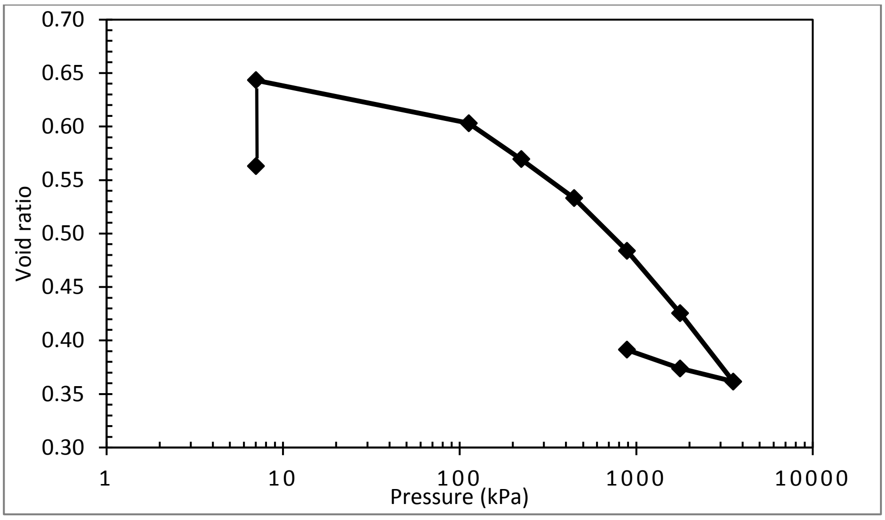

Figure 4.9: Void ratio versus effective consolidation pressure at 25° C

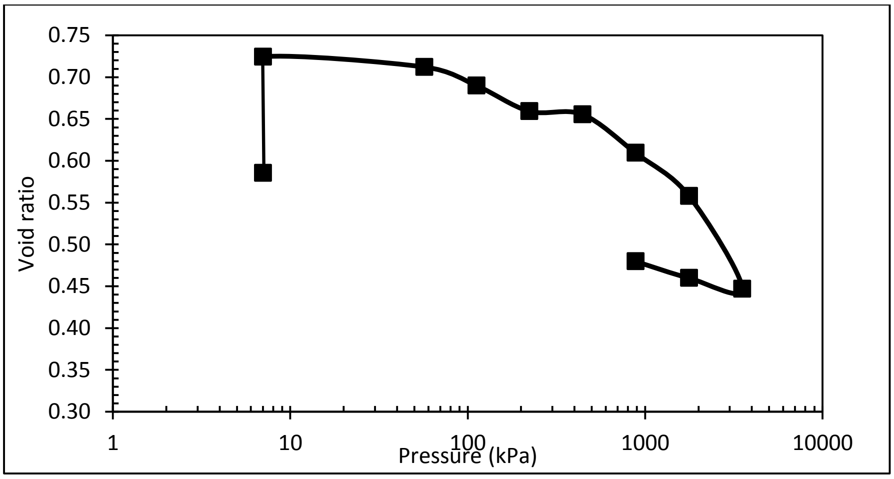

Figure 4.10: Void ratio versus effective consolidation pressure at 40° C

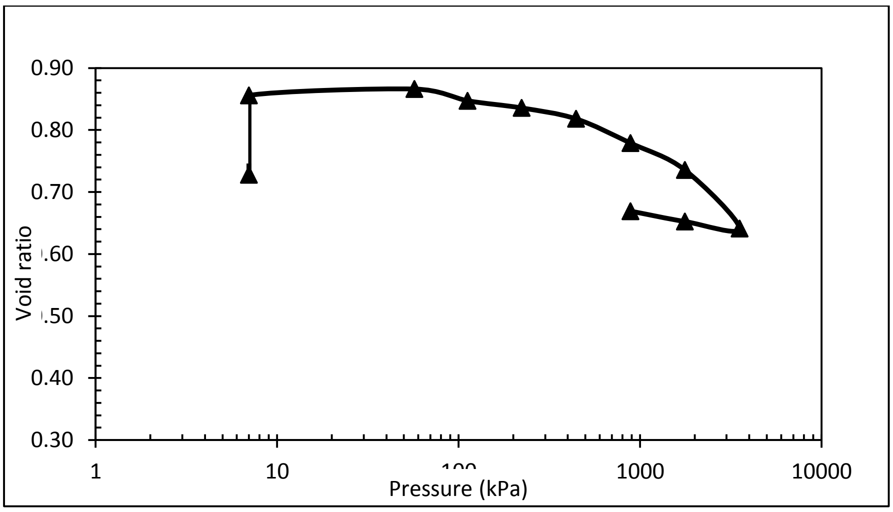

Figure 4.11: Void ratio versus effective consolidation pressure at 60° C

Figures 4.9-4.11 display the consolidation curves at 25° C, 40° C and 60° C respectively indicating increasing swelling and compressibility with increasing temperature.

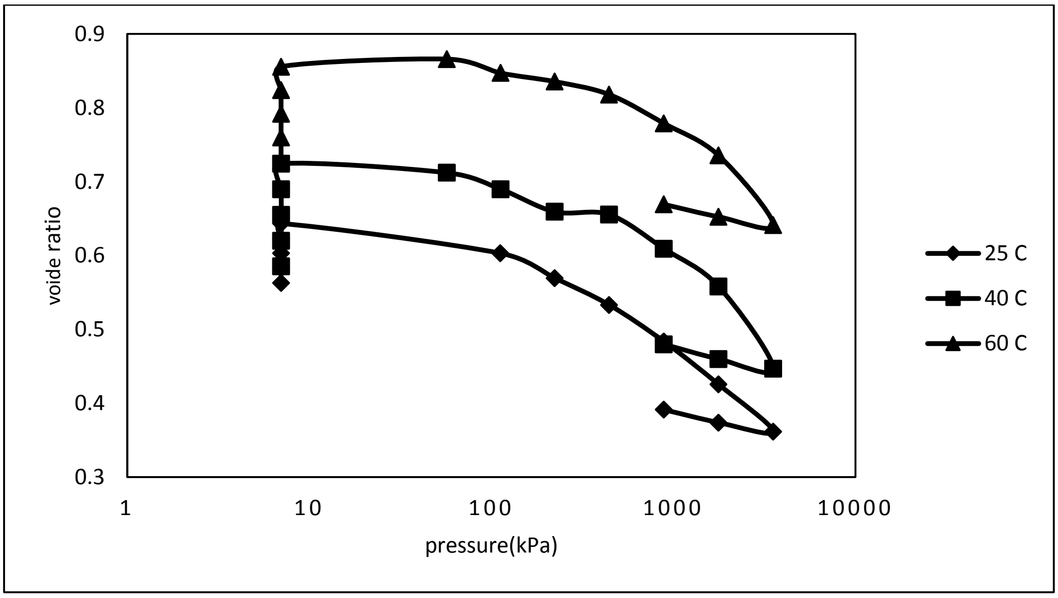

Figure 4.12: Void ratio versus effective consolidation pressure under different temperatures

Figure 4.12 depicts all the consolidation curves together from which it can be observed that the initial void ratio at the beginning of consolidation process is higher at elevated temperatures. Samples were let to swell freely before loading and at higher temperatures more water is attained in the voids hence higher void ratios were developed. Table 4.5 gives the consolidation test parameters, which clearly indicates increase in compression as well as rebound indices. The pre-consolidation pressure represents a measure of cementation for remolded samples, which is also increasing with temperature. During the drying phase desiccation bonds develop cementing particles together and consequently increasing the density .Higher the temperatures stronger are the desiccation bonds, therefore higher preconcolidation and swell pressures are attained.

Table 4.4: Consolidation test parameters under different temperatures (25° C, 40° C and 60° C)

Temperature	C_c	C_r	p'_s	$\sigma p'$
25° C	0.1838	0.0388	700	120
40° C	0.3804	0.0678	1600	140
60° C	0.3176	0.1577	2100	180

4.6 Unconfined Compression test results

The objective of this test is to characterize the unconfined compressive strength of a cohesive soil specimen. The unconfined compressive strength is determined along the drying path at different time intervals. The unconfined compression test specimens were dried at different temperatures and the unconfined compression tests were applied after 30 min, 1 h, 2 h and 4h drying periods. Figures 4.13- 4.15 present the stress versus strain curves obtained during testing of drying specimens. Studying these results, it can be observed that duration of drying increases the unconfined compressive strength as was expected. Since the soil gets drier and stiffer. It is also observed that drying temperature also increased the compressive strength. This result is compatible with the previous observations. Figure 4.16 and 4.17 summarizes these observations.

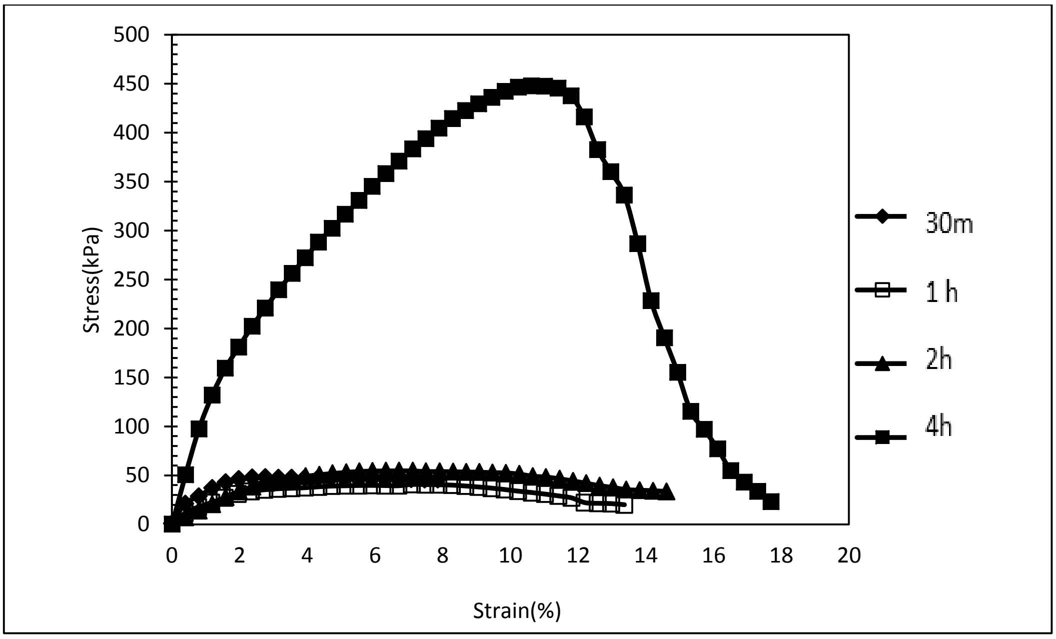

Figure 4.13: Stress-strain relationship of specimens drying at 25° C in different time intervals

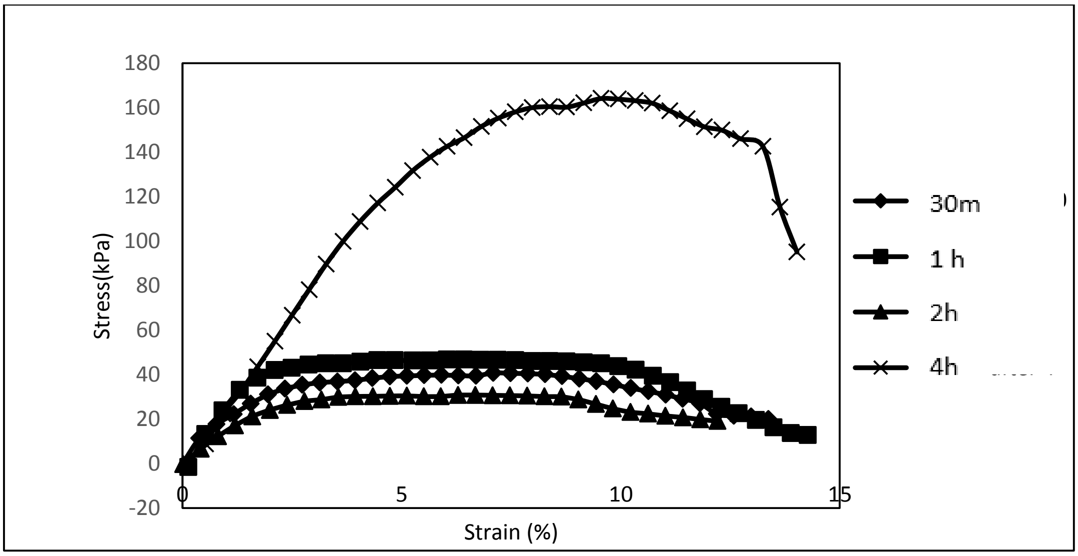

Figure 4.14: Stress-strain relationship of specimens drying at 40° C in different time intervals

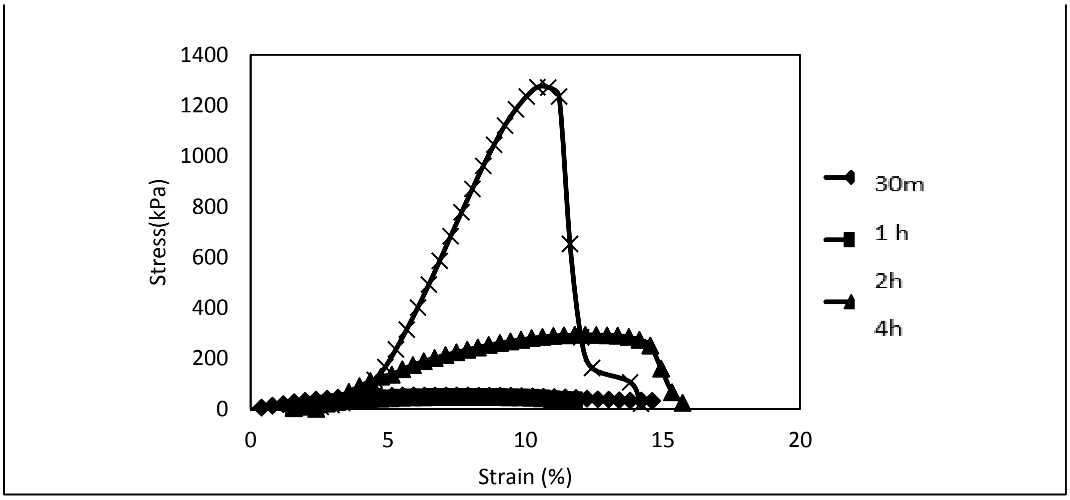

Figure 4.15: Stress-strain relationship of specimens drying at 40° C in different time intervals

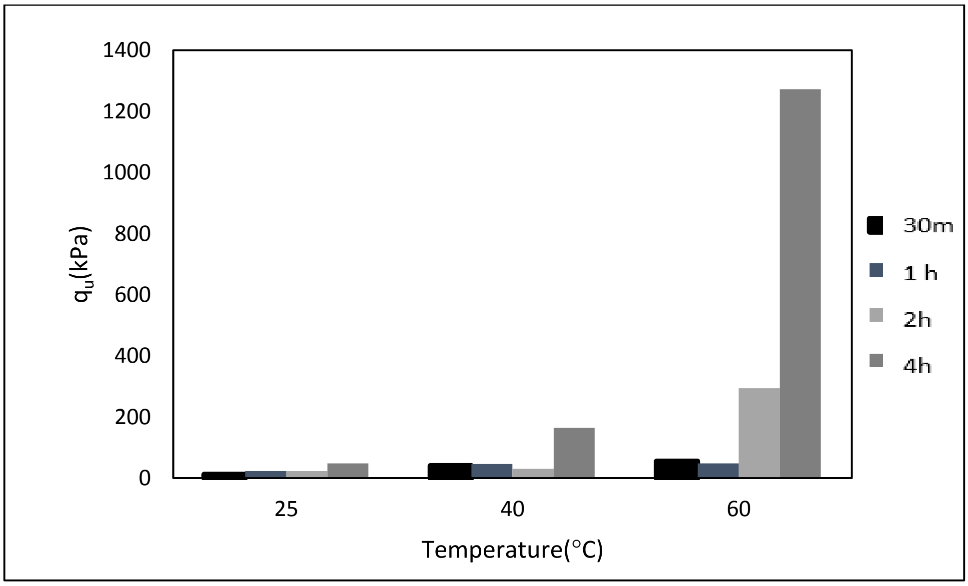

Figure 4.16: Unconfined compressive strength under drying at different durations and temperatures

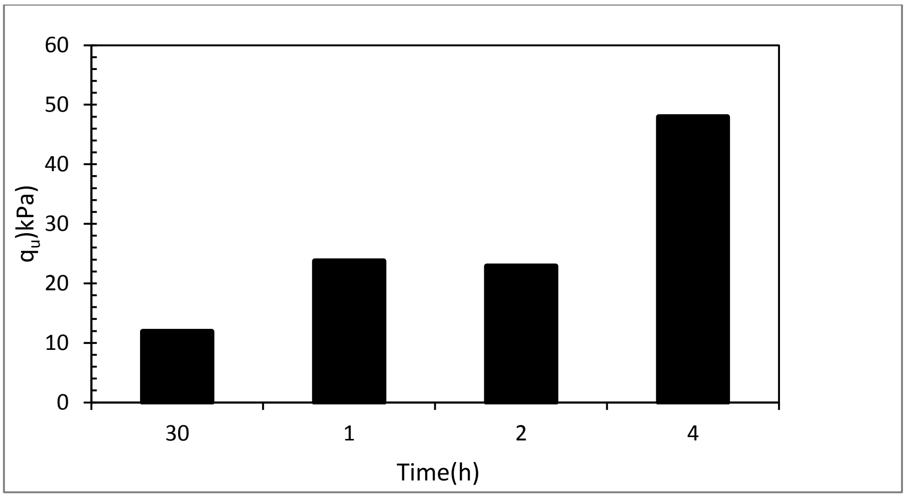

Figure 4.17: Unconfined compression versus time at 25° C

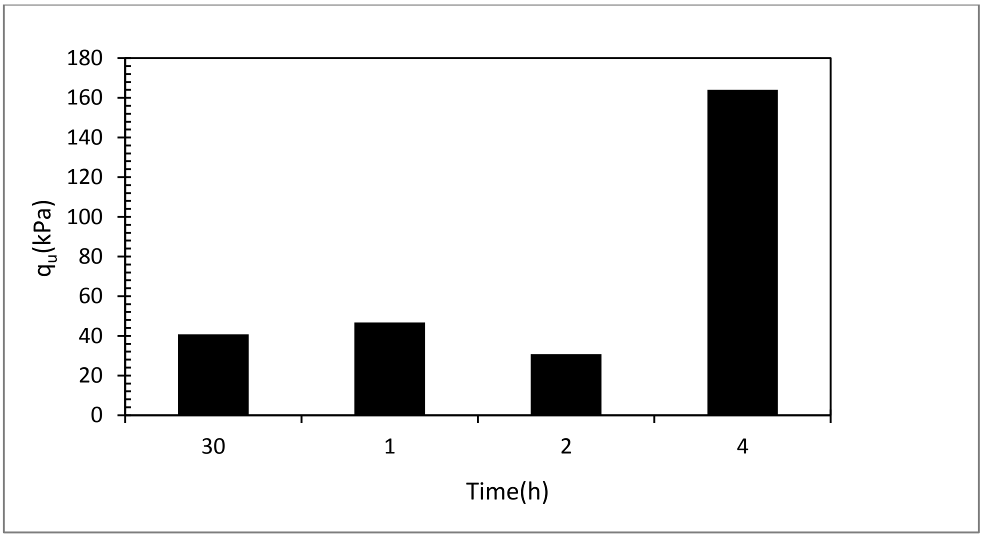

Figure 4.18: Unconfined compression versus Time at 40° C

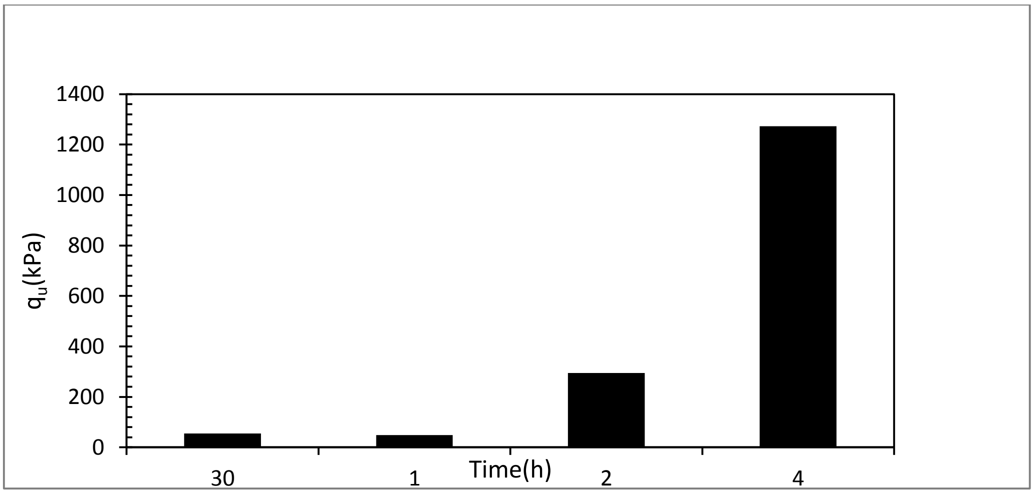

Figure 4.19: Unconfined compression versus time at 60° C

4.7 Soil-water Characteristic Curve

These curves display the water content- total suction relationship along the desorption path. The experimental data obtained from vapor equilibrium technique are fitted by two models using SoilVision. Figures 4.20- 4.22 show van Genuchten's model, and Figures 4.23- 4.25 gives Mualem's model respectively fitted to the experimental data. The model fitting parameters are given in Table 4.5, where the most important parameter is the air entry value (AEV), and the total suction at which air starts penetrating the largest pores. It can be observed that this value gets higher with increasing temperature. This observation substantiates the earlier findings regarding the increasing stiffer and desiccation bonds, hence density with temperature. Therefore samples get stiffer and more difficult for the air to enter.

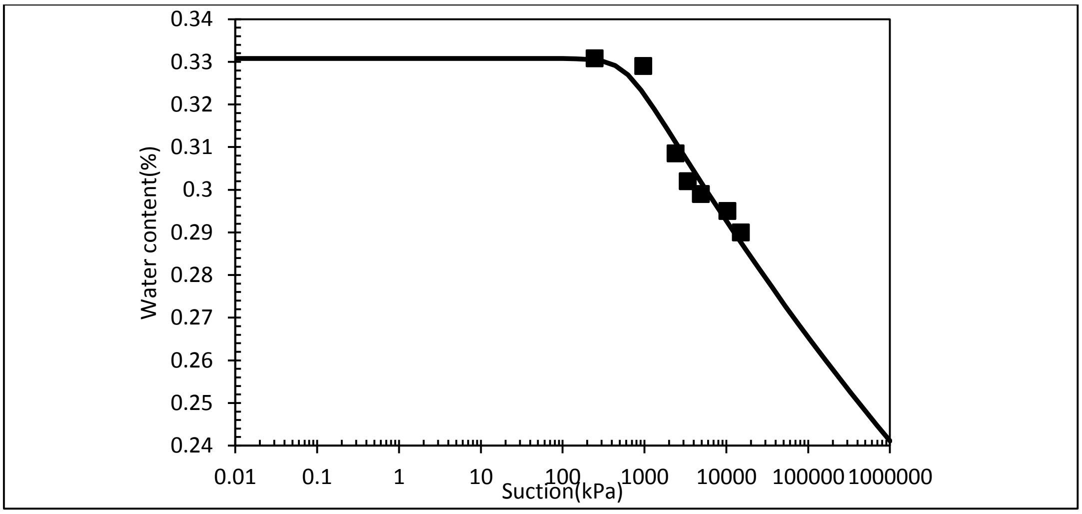

Figure 4.20: SWCC at 25° C fitted by van Genuchten's model

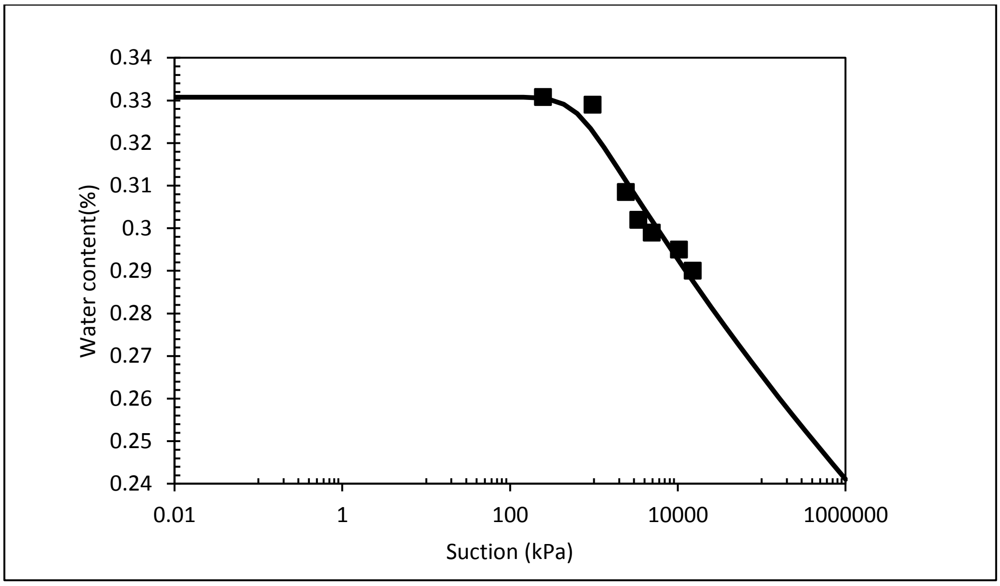

Figure 4.21: SWCC at 40° C fitted by van Genuchten's model

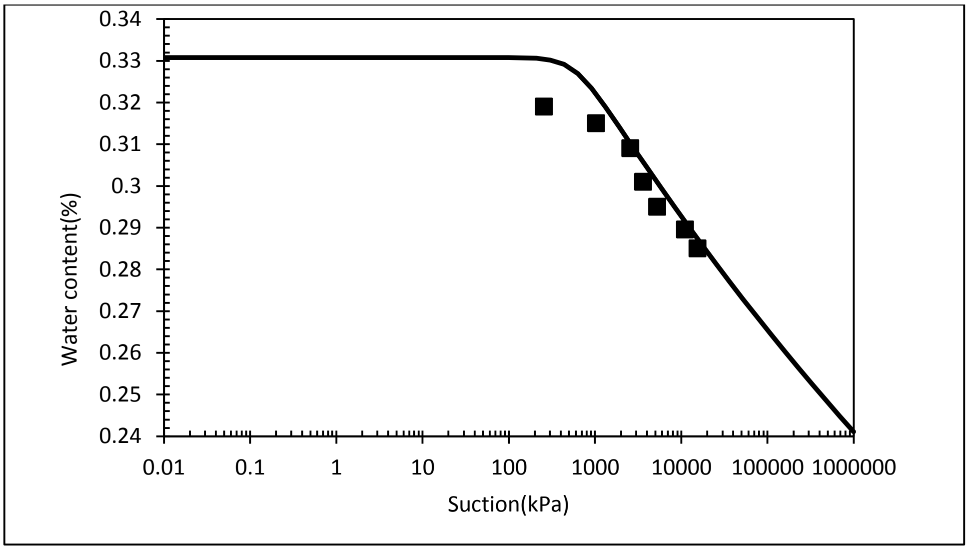

Figure 4.22: SWCC at 60° C fitted by van Genuchten's model

Table 4.5: SWCC fitting parameters of Van Genuchten's model

Temperature	n	m	R^2	w_r	AEV (kPa)
25°C	3.00	0.018	0.887	10	358.88
40°C	2.99	0.016	0.944	10	540.53
60°C	2.91	0.015	0.983	10	899.61

Residual water content, w_r, is the maximum gravimetric water content, at which the water capacity (the rate of change of gravimetric water content with respect to suction) approaches zero, and the unsaturated hydraulic conductivity becomes zero. The residual water content given in figure 4.20 (van Genuchten, 1980) remains the same for all samples, which is not realistic.

Mualem's model parameters are presented in Table 4.5 which show a similar trend of increasing AEV. With this model it is also striking to observe that the residual water content is increasing with temperature. This might arise from the increasing density, swell potential and hence water capacity. A measure of water capacity is the slope of the SWCC. This is not very well observed in van Genuchten's model, but Mualem's model gives an increasing n value, which is the fitting parameter directly proportional to the water capacity. Since the water capacity increases, the residual water content can be predicted to be increasing when soil is saturated and dried at elevated temperatures.

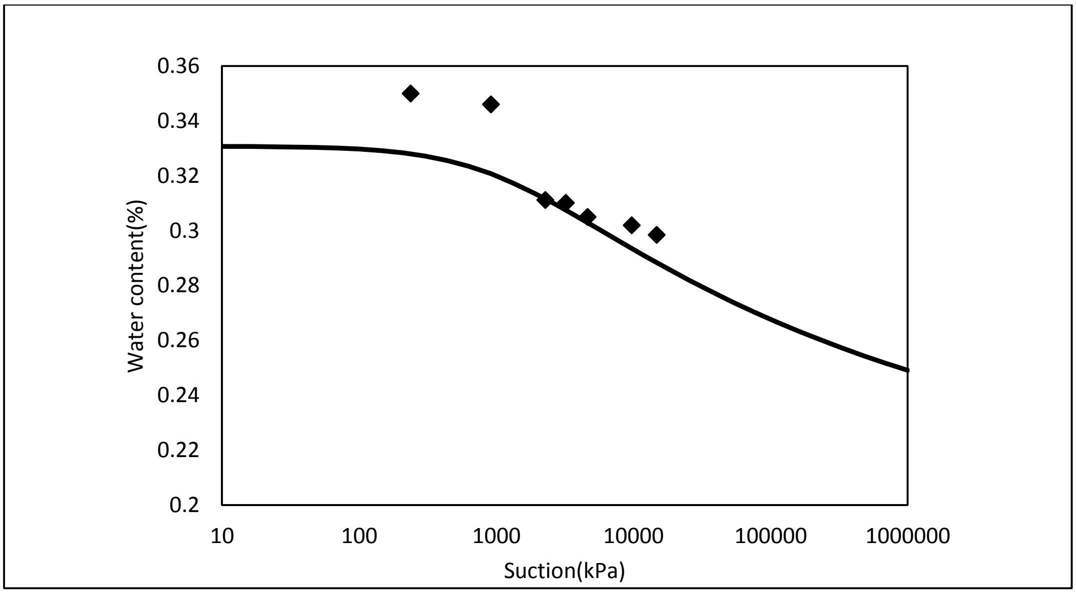

Figure 4.23: SWCC at 25° C fitted by Mualem's model

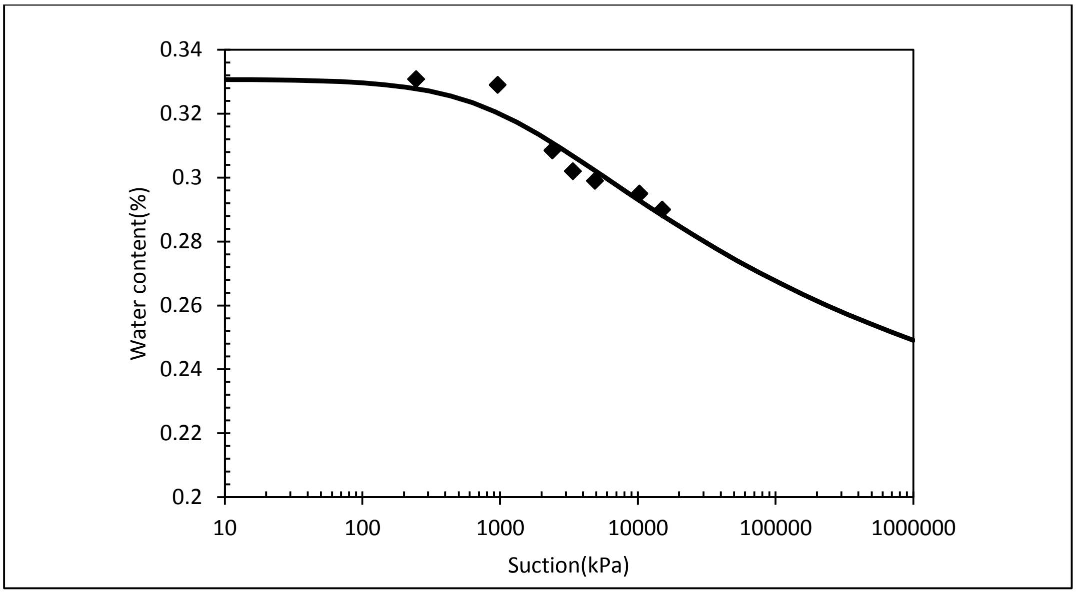

Figure 4.24: SWCC at 40° C fitted by Mualem's model

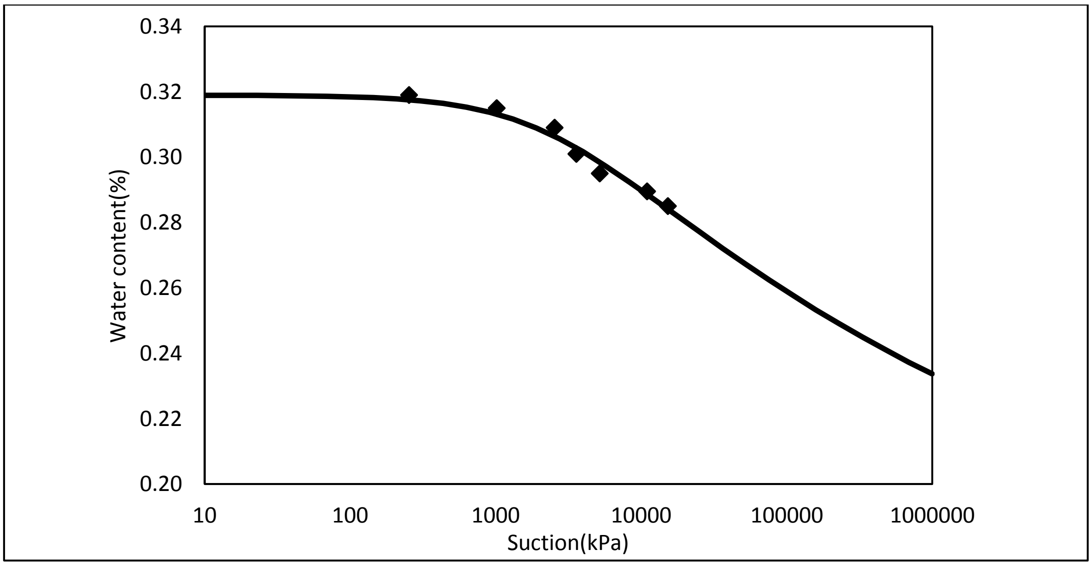

Figure 4.25: SWCC at 60° C fitted by Mualem's model

Table 4.6: SWCC fitting parameters of Mualem's model

Temperature	n	R^2	Wr	AEV (kPa)
25°C	1.07	0.85	8.0%	361.7
40°C	1.14	0.91	10.0%	509.21
60°C	1.15	0.97	15.4%	1237.37

4.8 Saturated Hydraulic Conductivity

Hydraulic conductivity under full saturation is obtained from consolidation test results, through estimating coefficient of consolidation, and coefficient of volume compressibility, mv. These parameters are directly proportional to the saturated hydraulic conductivity. Figure 4.26 shows the variation of C_V and mv with temperature. It is concluded that temperature causes increase in hydraulic conductivity as well. Since amount of settlement in a specimen subjected to surcharge load has a direct relationship with the ratio of drainage of water through that sample, therefore the saturated hydraulic conductivity (k_s) could be calculated indirectly from the one-dimensional compression test results applying the Equation.

$$k_s = \gamma_w . m_v . c_v$$

Where:

- K_s is the saturated hydraulic conductivity
- γ_w is the unit weight of water,
- m_v is the coefficient of compressibility and
- C_v is the coefficient of consolidation

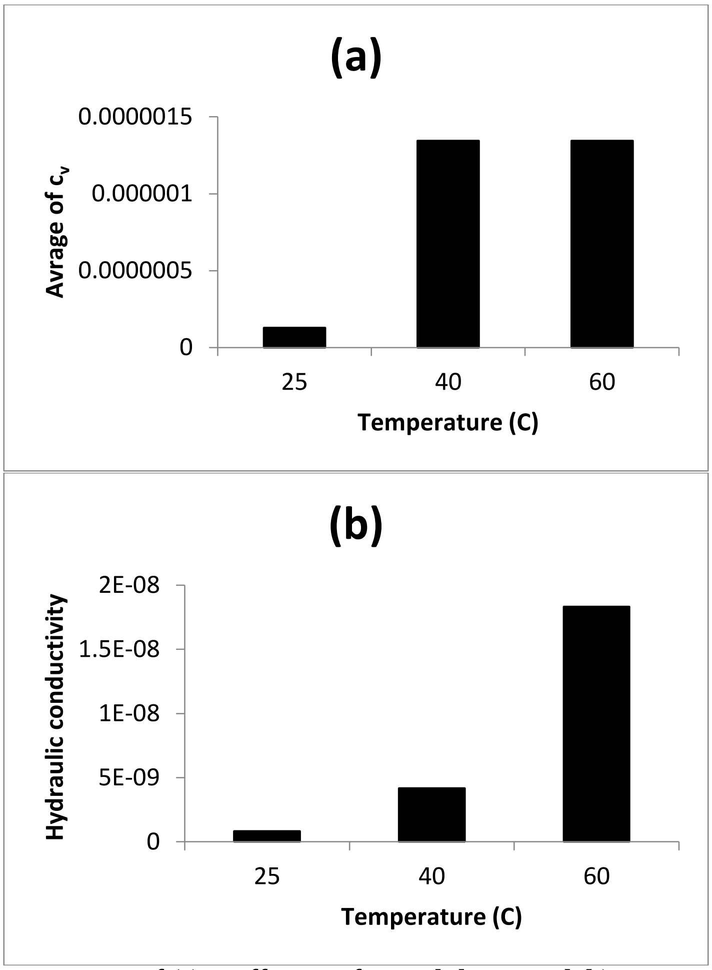

Figure 4.26: Variation of (a) coefficient of consolidation and (b) saturated hydraulic conductivity with temperature

Chapter 5

CONCLUSION

This study displayed the findings of an experimental work. Investigating the effect of temperature on hydro-mechanical behavior of expansive clay. The results are presented as variation of physical properties and engineering properties due to effect of temperature.

The following are the major conclusions extracted from this work:

1. Temperature directly influences swell percentage and swelling pressure. The results revealed that both increase with increase in temperature.
2. Temperature increases the shrinkage strains, since higher the swell capacity, higher will be the shrinkage.
3. Increase of temperature leads to higher compressibility at pressures higher than the preconsolidation pressure. Once the cementation and desiccation bonds break, the soil which has been under higher temperatures will compress more.
4. Another influence of temperature on hydro- mechanical behavior of expansive soils is the variation of hydraulic conductivity (k). Temperature is most important factor concerning this parameter, as it increases with increase temperature.
5. Temperature effect is an important parameter in the variation of soil suction. Soil suction is directly related to water content and temperature. This work

investigated the effect of temperature on soil suction as well and it was observed that soil suction increases with reduction of water content and water content decreases with rise of temperature.

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