Effect of Pore Water Chemistry on Hydro-Mechanical Behavior of Compacted Expansive Clay

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Submitted to the
Institute of Graduate Studies and Research
in the partial fulfillment of the requirements for the Degree of

Master of Science
in
Civil Engineering

Eastern Mediterranean University
August 2014
Gazimağusa, North Cyprus

Approval of the Institute of Graduate Studies and Research

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ABSTRACT

Pore water chemistry affects properties such as volume changes, including swell-shrinkage and compressibility, water retention capacity, hydraulic conductivity. The soil swell-shrinkage behavior causes deformations both in horizontal and vertical directions, leading to cracking of soils which result in increase in hydraulic conductivity due to formation of preferential flow paths in the location of cracks. The impact of these deformations on volume change behavior of soils still are to be fully understood, hence there is still room for research in this subject. This thesis presents the results of an experimental program studying the effect of pore solution concentration on hydro-mechanical behavior of unsaturated soils. In addition compressive strength and tensile strength properties are also studied. The experimental results are interpreted in relation to the diffuse double layer theory. NaCl and $CaCl_2$ are used in concentrations of 0.5M and 1M to change the pore fluid chemistry, and the behavior is compared to the behavior of soil with distilled water as the pore fluid. The experimental results conform to the results observed in some previous research works, hence contributing to the literature on the effect of pore fluid chemistry on the behavior of unsaturated soils exposed to saline environments. The findings of this study indicated that salts in the pore fluid reduced the swell-shrinkage potential and compressibility, while increasing the strength properties and hydraulic conductivity.

Keywords: Desiccation, pore-water chemistry, shrinkage, unsaturated soils.

ÖZ

Boşluk suyu kimyasının zemin-su sistemlerinin davranışına etkisi çoklu ilişkiler sonucu oluşur ve zemin-su ilişkisinde farklı süreçleri etkiler. Bu süreçler, şişme-büzülme ve kompressibiliteyi içeren hacimsel değişimler ile su tutma kapasitesi ve hidrolik iletkenlikten oluşur. Zeminin büzülme-şişme davranışı yatay ve düşey deformasyonlara ve tercihli akım kanallarına neden olan çatlakların oluşmasına, dolayısıyla hidrolik iletkenlik artışına neden olur. Halen zemin deformasyonlarının hacimsel değişimler üzerindeki etkisi tam manasıyla anlaşılamadığından, bu konuda yapılabilecek çok araştırma vardır. Bu tezde boşluk suyu konsantrasyonunun doygun olmayan zeminler üzerindeki rolü irdelenmiştir. Ayrıca serbest basınç ve gerilme mukavemetleri de farklı boşluk suyu kimyasında çalışılmıştır. Deneyler damıtık su, ve 0.5M ve 1M NaCl ile 0.5M ve 1 M $CaCl_2$ kullanılarak hazırlanmış nümuneler üzerinde yapılıp, deneysel sonuçlar ‘diffuse double layer’ (DDL) hipotezi yaklaşımı ile açıklanmaya çalışılmıştır. Sonuçlar temel beklentiler doğrultusunda çıkarken, bu yaklaşımların tuzlu ortamlarda suya doygun olmayan zeminlerin davranışlarının daha iyi anlaşılmasına yararlı olacak bilgiler edinilmesini de sağlamıştır. Deneysel çalışma sonucunda boşluk suyunda tuz konsantrasyonu arttıkca şişme-büzülme ve kompressibilite potansiyelinde azalma izlenirken, mukavemet ve hidrolik iletkenlik faktörlerinde de artış gözlemlenmiştir.

Anahtar kelimeler: Kuruma, boşluk suyu kimyası, büzülme, suya doygun olmayan zeminler.

ACKNOWLEDGMENT

I want to appreciate help of my best teacher in my entire life associate professor Huriye Bilsel who is like my mother to and teach me many things in my life.

I want to thank eastern mediteranean university, Civil Engineering department, they support me every time.

I am so grateful to my colleagues and friends Mohsen Ramezan Shirazi, Shirin Moallem, Reihane Rafi Manzelat, Mohammad Reza Golhashem, Orod Zarrinkafsh, Hasan Moniri, Sandra Ghavam, Shadab Salmani, Hamed Khadivar and specially the laboratory engineer Ogun Kilic who helped me throughout my laboratory experiments.

DEDICATION

To my family

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Chapter 1

INTRODUCTION

1.1 General

One of the factors which can modify the chemical composition of clays is by exchangeable cations in the pore fluid chemistry. In most cases, the exchangeable cations govern the engineering properties of clays. The problems related to expansive clays, such as swell-shrinkage or settlements, usually arise from changes in physicochemical characteristics, resulting in changes in the density of the diffusive double layer. Pore fluid chemistry influences the adsorbed water layer thickness surrounding the clay platelets. Intense concentration of cations with high cation valence and in addition an acidic environment notably reduces the double layer thickness.

The effect of the pore fluid chemistry, which may change in time, may cause detrimental damage to soil structures such as impermeable clay liners. Bowders and Daniel (1986), Sridharan (1991), Di Maio (1996), Kaya and Fang (2000), and Ören and Kaya (2003) showed that inorganic salt solutions have strong effect on the engineering behavior of swelling clays, whereas the effects of saline waters are not very well known. The importance of studying the effect of saline water on the engineering behavior of soils is especially significant for fine grained soils near coastal areas.

The increase of salinity is because of lowering of the groundwater level in coastal areas, resulting in seawater penetrating inland and moving to the water table. Don et al. (2006) showed that some important problems occurred because of extensive ground subsidence due to salinity intrusion from pumping of groundwater in the Shiroishi lowland plain, southwestern Kyushu Island of Japan. Similarly, the groundwater level is dropping continuously in various parts of the coastal areas of Turkey as well as North Cyprus. The main cause of this is the excessive consumption of water for industrial and agricultural purposes.

1.2 Aim of the Study

The aim of this study is to determine the changes in some of the geotechnical engineering properties of expansive clay when exposed to salts with respect to distilled water. The soil is gathered from behind the Faculty of Communication in Eastern Meditereanian University, North Cyprus. This soil is subjected to consistency limits, sedimentation, volume change, and compressibility and strength tests. It is believed that this study fills an important gap in the literature, and provides new data and approach for the researchers and engineers to evaluate effect of salt solutions on clayey soils.

1.3 Scope of the Study

In the scope of this thesis, a literature review on expansive clays, effect of chemistry and effect of salinity on clays are given in Chapter 2. In Chapters 4 and 5 experimental work, discussions of the test results and conclusions are presented respectively.

Chapter 2

LITERATURE REVIEW

2.1 Introduction

Decrease in moisture content of some soils causes changes in specific volume, strength and hydraulic conductivity when become unsaturated. These types of soils exhibit considerable alteration in volume, shear strength and hydraulic properties due to a slight change in degree of saturation. Most of soil mechanics principles are related to soils at saturated states. Extension of these principles to unsaturated soils needs precise attention to these fundamental topics: (1) volume change behavior associated with suction or change in moisture content, (2) shear strength behavior related to suction or saturation changes, (3) hydraulic behavior conjunction with suction or change in degree of saturation. Variation in degree of saturation or suction can cause soils experience significant volume change. Some soils swell upon wetting, some collapse and some do both. The rate of swelling and collapse depend on the stress level. The large volume changes associated with moisture change can lead to intense damage to foundations and structures. Due to characteristic of unsaturated soils, they behave non-linearly, usually an unsaturated soil is considered to be a three-phase system. However, the independent properties and ongoing bounding surfaces of the air-water boundary need its prudence as a fourth phase (Davies and Rideal, 1963). An element of unsaturated soil can thus be considered as a mixture with two phases that become in equilibrium with others under applied stresses (i. e. soil particles and the contractile skin) and two phases that flow under applied pressure (i. e. the air and water). The air phase is assumed to be continuous.

2.2 Expansive Soils

When expansive soils are exposed to moisture changes, mineralogical composition can cause large volume changes or volumetric traction of expansive soils. They shrink on drying and swell on wetting respectively (Bolt, 1955; Jennings and Knight, 1957). Unsaturated soils include special categories such as expansive clays that experience large swelling strains on wetting. Unloading by erosion or excavation can lead to swelling strains in fully saturated expansive soils. Large swelling strains can cause significant damage in structures and foundations constructed on expansive soils. To explore swell and shrinkage behavior of expansive soils, some important researches have been performed during the last century (e.g., Katz, 1918; Terzaghi, 1925, 1926, 1931, 1943; Hamilton, 1963; Chen 1975; Fredlund and Morgenstern, 1976; Gens and Alonso, 1992; Lytton, 1994; Briaud et al., 2003; Ng et al., 2003; Fityus et al., 2004; Vu and Fredlund, 2006, and Nelson et al., 2012). Since environmental factors cannot be logically predicted or estimated, expansive soils are considered a challenge for geotechnical engineers.

2.3 Origin of Expansive Soils

Formation of expansive soils are complex and depends on interaction of a number of controlling variables such as, parent rock type, weathering and erosion, dominant climate, and local topography. Expansive soils have a worldwide distribution, especially prevalent in arid to semi-arid climates, in which evapotranspiration exceeds precipitation for considerable part of the year. This is partly described by the theory that lack leaching in semi-arid regions causing formation of monmorillonite mineral (Mitchel, 1993). In the arid and semi-arid climates expansive soils generally exists in an unsaturated state. However, in the humid climates, the soil is not partially saturated and the difficulty appears when the soil begins to desiccate. The term “unsaturated” is herein used to describe both partially saturated and dry state of the soil.

2.3.1 Distinction between Clay and Clay Mineral

The term clay minerals refers to phyllosilicates minerals and minerals which give plasticity to clay when wet and cause rigidity upon drying or firing as defined by AIPEA (Association International pour I’Etude des Argiles) and CMS (the clay mineral society) nomenclature committees (Guggenheim and Martin, 1995). In contrast, the term clay refers to a naturally occurring material composed primarily of fine-grained minerals, which is usually plastic at adequate moisture contents and will become stiff when loses water or dries (Guggenheim and Martin, 1995). Based on the two terms given here, a clay mineral is a particular mineral which is a naturally occuring homogeneous solid with a definite (but usually not fixed) chemical composition and a regular atomic order (Herlbut and Klein, 1977), in which atoms of these elements are organized into crystalline forms. Clay is mainly a size term which relates to non-minerals and minerals with particular grain size range.

2.3.2 Clay Mineral Structure

According to Mitchell (1993) clays have complex clay mineral composition. The clay particles consist of sheets such as aluminum phyllosilicates or layer silicates made up of two molecular modules in the form of a sheet of corner-linked to tetrahedral and a sheet of edge linked to octahedral (Bailey, 1980; Brown, 1984; Moore and Reynolds, 1997). When some of the tetrahedral and octahedral spaces are filled by cations isomorphous replacement happens. After that a constant negative charge will be created to build the need of cations among the layers so that it can hold the electrical neutrality. These cations could be founds either in stable (e.g., K+ in mica) or exchangeable situations (e.g., Na+, Ca2+ in smectite). Moore and Reynolds (1997), Amonette et al.(2002), Mitchell and Soga (2005) show that these inconvenient interactions are generally satisfied by H+ and OH- which is depend on pH value of environment. Different cations accompanied with the variation of permanent surface charge and pH relevant edge charge simultaneously, achieve the complicated physiochemical attributes of clay minerals.

2.3.3 Attraction of Water to Clay

Water (H2O) is the polar molecule composed of a V-shaped order of dipoles (two hydrogen nuclear poles of positive charge) attracted to one negatively charged oxygen atom resulting into ion hydration. Since opposite charges imbibe, the positive molecules attract the negative molecules forming a series of water molecules (Chong, 2002 and Chaplin, 2007).Since water has polar molecules, the slightly partially negative dipole ends are attracted to the ions of clay minerals with unsatisfied forces. The attraction takes place through a variety of chemical mechanisms such as hydrogen combination, cation and anion swapping, attraction by osmosis and charged surface – dipole attraction (Mitchell, 1993).

The mechanism leading to hydrogen combination takes place by either, the negative corner of oxygen (O-) of the silica attracting the positive corner (H+) of water molecules, or the positive hdroxyls attracting the negative edges of water molecules. When dry clay is exposed to water vapor, water is adsorbed onto the clay surface. The tendency of exchangeable cations to hydrate has a strong impact on the inward pull of the negative corners of water molecules. The quantity of water adsorbed relies on the type of interchangeable cations. The ordinary exchangeable cations in smectite are Ca2+, Mg2+, Na+ and H+, though Li+ and K may be found in a number of smectite stratums. Air-dried smectite clays including Ca2+ and Mg2+ as adsorbed ions can adsorb water in surplus of liquid limit. In clays where Na+ is the superior exchangeable ion, the swell exceeds that of clay containing Ca2+. This is attributed to the particle size, wide surface area, high diffusion and electrical forces between the particles of Na+ (Odom, 1984).

More often than not, clay mineral platelets have negative electrical charges on their plane surfaces and positive charges on their borders. The equal clay surfaces with negative electrical charges draw water molecules to keep electrical neutrality. The pull produces higher condensation of the water molecules at the clay levels than at the mid-flat. Although the influence reduces as the expanse from the clay surfaces increments in the direction of the Centre, it makes structural lack of order at the mid-plane in which like poles direct adjacent to each other. Because the like poles repel one another, the created gap between the poles is definitely filled with cations in an attempt to make neutrile or equilibrium the repulsive forces. Expansive clay minerals such as montmorillonite are noted for high cation exchange capacity while non-expansive clay minerals like kaolinite have a low cation exchange capacity. In smectite, the interlayer cations are exchangeable and both interlayer surfaces and cations are subject to hydration, thus increasing the tendency to sorb water. The size of space among the layers (mid-plane) is ruled by the type of cations in the interlayer, thus monovalent cations like Na+ motive more split (expansion) than do divalent cations like Ca. The separation (expansion) is increased by condensation of ions in the mid-layer as well as the quantity of water present in the soil.

2.4 Conceptual Models for the Microstructure Level

A number of theories have been established to illustrate clay compressibility at the particle-water-cation plane. The Gouy-Chapman double layer theory has been successfully applied by Mitchell (1976), Callaghan and Ottewill (1974), Sridharan and Jayadera (1982), and Jayadeva and Sridharan (1982). However, Low and Margheim (1979), Low (1980, 1991) claim that the double layer theory does not satisfactorily explain the experimental swelling results of montmorillonite. They have recommended an exponential empirical connection that relates swelling compression to inter-layer space. Baveye, Verbug and Beilders (1991), Tessier (1991) reported that direct mechanical impact of suction might meaningfully contribute to the volume change behavior. This is in addition to the osmotic and hydration results that are addressed by the present concepts. Apart from this, the question of whether or not microstructural deformations are reversible has not yet been fully addressed. Various conceptual models of grain organizations and treatment have been proposed. The conceptual models change depending on the formulations. Volume changes in expansive soils are known to be results of physical-chemical interaction phenomena at the clay-platelet level. Alonso Gens and Hight (1987) referred to micro-fabric as that which governs the status of the water in soils, particularly potential or suction. They proposed that the mineralogical composition influences the adsorption component of matric suction, while the inner geometry controls the capillary ingredient. However, their theory was based on the results relating to non-expansive to moderate expansive soils. Gens and Alonso (1992) critically surveyed some of the principle aspects of soil microstructure in conjunction with the behavior of expansive soils. They emphasized the significant role played by the different phenomena taking place at particle level in response to outer activities (load, chemical phenomenon, and suction changes). The review indicated a lack of general agreement in depicting the behavior of the microstructural level, specifically for expansive soils. The complexity of modeling the interactions at particle level was obvious.

2.5 Stress State Variables

The effective stress principle proposed by Terzaghi, (1936) has been notably useful in characterizing the stress-strain behavior of fully saturated soils. This led to the belief that the effective stress principle controlled the behavior of soils over all ranges of degree of saturation, with full saturation being one limit condition. Severe attempts to describe unsaturated soils in terms of effective stress are listed in the literature. The works of Aitchison and Donald (1956), Bishop (1957), Bishop and Donald (1961), Jennings (1961), Coleman (1962), Bishop and Blight (1963), Burland (1965), Matyas and Radhakrishna (1968) can be cited in this regard. Bishop and Blight (1963), Burland (1965) suggested that the mass change in unsaturated soils could be freely related to the net total stress and suction stress variables. Fredlund and Morgenstern (1977) formally proposed that any pair of the following stress domains forms appropriate framework to describe the stress-strain-strength behavior of unsaturated soils: (i) the net total stress, σ (ii) the effective stress, σ' and (iii) soil suction (u_a-u_w). In these manifestations, u_a is pore air pressure and u_w is pore water pressure. Therefore, unsaturated soils and expansive soils have since been described in terms of suction. Vanapalli, Fredlund and Pufahl (1999) referred to the soil-water characteristic curve as a conceptual and interpretative tool, by which unsaturated soil-behavior can be understood. The idea of soil water potential has been adopted to characterize the effect of the forces acting on an infinitesimal body of water in the attainable force field. The term “soil water potential” explains the unique potential energy of soil water to that of water under standard reference state. The concept has been very functional in visualizing soil water phenomena. For example, it has helped in incorporating the phenomena of water retention and movement.

2.6 Swelling Potential

There are two definitions for swell potential: Holtz (1959) indicated as the volume change of air-dried undisturbed sample, whereas Seed et al. (1962) defined it as change in volume of a disturbed sample. McCormack and Wilding (1975) explained that clay content can be a measure of swelling potential like the Atterberg limits in soils dominated by illite. Also, some believed that cation exchange capacity (CEC), saturation moisture and plasticity index (PI) are the most important characteristics in the estimation of swelling potential. Snethen et al. (1977) considered liquid limit (LL) and plasticity index (PI) as the best indicators of potential swell.

2.6.1 Shrinkage and Shrinkage Limit

Content and type of the clay are among so many effects which attributes to the soils. Characteristics of the soil shrinkage are widely used for characterizing soil structure. Tuller and Or (2003) showed that, as a result of the evolvement of new pore-space approaches, measurement and modeling of soil shrinkage has become a popular research subject. Haines (1923) and Stirk (1954) in their study showed that the soil shrinkage is specified as the specific volume change of soil relative to its moisture content which is principally because of clay swelling properties.

Sposito and Giraldez (1976), and Chertkov (2000) proved that shrinkage of clay paste occurs in two phases which are; load line phase and air entry point (AE). Clay paste remains saturated and each cubic centimeter of water lost corresponds to a 1cm3 volume reduction in the phase 2. When air inters the paste, shrinkage reduces and a lowest degree of clay-paste volume is attained. It is called the shrinkage limit (SL). McGarry and Malafant (1987), and Tariq and Durnford (1993) studied the shrinkage of structured soil samples in the beginning stage of soil science (e.g., Tempany, 1917).

It seems that, linear shrinkage test has been first introduced by the Texas Highway Department in 1932 (Heidema, 1957) and is currently described as a standard test procedure in British Standard BS 1377:1990. It was said that the linear shrinkage is the most reliable soil constant in road construction (Netterberg, 1978) and most important indicator of plasticity/cohesion for gravel wearing course material (Paige and Green, 1989). Haupt (1980) and Emery (1985) performed a research to determine subgrade moisture prediction models. They referred to this point that the inclusion of the bar linear shrinkage produced as good, if not better, prediction models than the inclusion of any of the other Atterberg limit results. Paige et al.(1999) estimated different bar linear shrinkage values and concluded that the bar linear shrinkage test is a more effective test to show material performance than the more traditional Atterberg limits.

Shrinkage measurement process is a complex and time-consuming process. Therefore, a number of methods were developed and tested for measuring the shrinkage. As Braudeau et al. (1999), Chertkov (2005), Chertkov et al. (2004) Garnier et al. (1997), and Oleszczuk et al (2003) explained, most of these methods were based on assessing the shrinkage-related height changes of a soil sample which assuming isotropy or the vertical and horizontal deformations were considered individually.

2.6.2 Cyclic Swelling and Shrinking

Because of alternative precipitation and evapotranspiration, most surficial soils are subjected to wetting–drying cycles. These wetting–drying cycles can affect the hydro-mechanical behavior of unsaturated soils. An important number of studies, such as by Cui and Delage (1996), Lloret et al. (2003) and Zhan and Ng (2006) have been conducted on the impact of water content on the hydro-mechanical behavior of unsaturated soils. Recently, some researchers like Gens et al. (2006) paied more attention to the role of wetting–drying cycles in the hydromechanical behavior of unsaturated soils.

Swell potential of expansive soils is generally determined by one-cycle of wetting. However due to environmental effects, soils in nature are ceaselessly subjected to wetting-drying cycles. It has been showed that the number of wetting-drying cycles play an important role in the behavior of expansive soils. Many researchers such as, Popesco (1980), Chen and Ma (1987), SubbaRao and Satyadas (1987), Dif and Blumel (1991), Day (1994) Al Homoud et al. (1995), Bilsel (2002), and Tripathy (2002) have studied the effect of cyclic swell-shrink on the swelling behavior of compacted expansive soils. They concluded that when soils swell fully and shrink to their initial water content, they demonstrate less expansion. And it's because of the fatigue of clay after each cycle. Popesco (1980), Day (1994), and Guney (2007) deduced that swelling potential rise with the number of cycles. Al Homoud et al. (1995) also stated that wetting-drying cycle resulted in particle aggregation. He supported his findings by the decrease in clay content and the plasticity index values of the soils after the increasing number of cycles. This subject caused reduction in the swelling characteristic. Therefore the effect of cycle number on the swelling and shrinking behavior of expansive soils should be considered in the characterization of swell-shrink behavior.

Expansive soils present serious irreversible bulk volume increment upon wetting. In other words, as Chu and Mou (1973), Likos and Lu (2006) proved volume changes are increasing along the further drying paths. This reaction may be related to irreversible changes in clay texture resulting from interlayer swelling upon wetting.

Many researchers like Al-Homoud et al (1995), and Osipov et al (1987) by using oedometers without suction control have studied the volumetric behavior of compacted expansive soils with wetting–drying cycles. However only a few experimental programs have been conducted, such as by Alonso et al (2005), Chu and Mou (1973), Cuisinier and Masrouri (2005), Lloret et al (2003), and Nowamooz and Masrouri (2008) by using suction controlled tools. It was found that considerable irreversible accumulation of swelling or shrinkage strain in an expansive soil occurs upon wetting–drying cycles. Alonso et al. (1995), Gehling et al. (1995), Nowamooz and Masrouri (2009), and Sharma (1998) showed that swelling or shrinkage is irreversible and is a function of compaction conditions and the following change of net stress and suction, rather than being determined only by soil type. Cui et al (2002) proved that only reversible volume change response to wetting–drying cycle occurs in dense compacted expansive soil. It seems that the irreversibility of volume change during wetting–drying cycles is hard to understand the effect of wetting–drying cycles on the mechanical behavior of unsaturated expansive soils. Likos and Lu (2003), and Miao et al. (2002) showed that important hydraulic hysteresis was observed in compacted expansive soils, and the respective water-retention behavior.

2.7 Desiccation Cracks

Desiccation cracking is a common event which occurs in clay materials and significantly increases soil hydraulic conductivity. This topic is one of the major concerns in the design and construction of landfill covers, particularly, in dry regions. Fractures are inclining to develop within cycles of long withered spells by raising the plasticity of the clay. Water pulls in clay until full the cracks and fissures during periods of precipitation that follow the dry spells. Furthermore, when hydrostatic forces increased, the water cannot be absorbed quickly by clay. Some clays which have high plasticity index have more fracture and shrink/swell treatment. In addition the seasonal shrinking and swelling manner of the cracked clay zone caused less volume shear strength of the clay to the point where it may approach its residual strength. So, engineers should pay more attention to such modifications in engineering practices during cyclic drying and wetting and designing the landfill covers and liners and barriers in arid regions. Recently, the impact of cyclic drying and wetting on permeability of clayey of soils has been inspected and discussed widely. For instance, Eigenbrod (1996), and Rayhani et al. (2007) observed that the fine grained soils experienced variations in hydraulic conductivity during cyclic drying and wetting. Chertkov (2002) surveyed the features of fracture dimension in saturated soils and indicated the relationship between the minimum dimension of a shrinkage-induced, quasi-brittle fracture and the dimensions of a crack network in a swelling clay soil. This kind of relation proposes the requirement of better findings on cracking and hydraulic properties of clay soils.

Desiccation cracks cause negative effects on the whole stability and bearing capacity of cracked soil lumps and permeability and the integrity of structures. This problem leads to clay buffers for nuclear waste isolation. Furthermore, desiccation in clay soils can result in shrinkage of the soil and subsidence of the ground; this may lead to damage to buildings. As the soil re-hydrates, it can swell, resulting in ground heave; this may also cause damage to buildings. Strength reduction, with the appearance of desiccation cracks. Desiccation cause crack formation which can extremely influence the stability of slopes. Drying cracks in the core of an earth dam walls can lead to overall degradation of the structure, preferential paths for water permeation, and it can cause the increase of pore water pressures, leakage and eventually failure of the structure. Estimation for such harm is hard and many researchers such as Albrecht and Benson (2001), Kalka (2009), Konrad and Ayad (1997), Miller et al. (1998), Morris et al. (1992), Nahlawi and Kodikara (2006), Peng et al. (2006), Péron et al. (2009), Rodríguez et al. (2007), Tang et al. (2008), and Yesiller et al. (2000) have studied this topic for decades. Desiccation fractures are also significant in agricultural applications. Prendergast (1995) has showed that the movement of pesticides and fertilizers to the root zone, as well as the efficiency of irrigation operations are affected by the presence of desiccation cracks and other forms of macro-pores.

According to equilibrium thermo dynamics law, when a soil starts to dry, the phase change happens between pore liquid and vapor, so that the unique vapor and liquid potentials stay equal (Coussy, 1998; Mainguy, 2001). Platen (2001) has showed that evaporation needs drying. Liquid particles evaporate faster from warmer zone than they do from cooler zone. Therefore, as Kowalski (2003) showed in his research, the capillary rise of the liquid bridge occurs towards the warmer zone. This is because of difference of surface tension between menisci. Peron (2008) proved that isothermal drying can be adopted if the phase change rate is restricted by the mass transfer, and not regulated by the heat exchange in most of the soils.

Gray (1989) has showed that cracking occurs due to either a volumetric change in the soil body or a result of pressure applied on the soil body. Categorized fractures happen in two main types, in line with the system of their organization:

1. Mechanical cracks: These are cracks that form because of deposition or due to unsuitable construction. Cracks with no good link between lifts and poor compaction are good pattern of this.

2. Physicochemical cracks: We can divide it into three subdivisions: syneresis fractures, cracks caused by freeze-thaw cycles and cracks started by complete drying of the material.

The resulting effects of the shrinkage and cracking activities of soils are due to clay mineralogy, clay content, compaction conditions, drying process, wetting and drying cycles, soil particle orientation, unit weight, pore fluid, and exchangeable ions. Holtz (1981) has showed that formation of fissures is increased by the availability of clay. Shrinkage and swelling increase, when the plasticity index is high. While the extent of reduction in size drops to a minimum. However, Kleppe (1985) proved that by stabilizing clay soil with coarse-grained materials, the risk of shrinkage and fracturing become minimized. For the fabrication of a liner in dried sites, Daniel and Wu (1993) proposed the use of clayey sand with a low hydraulic conductivity and low shrinkage values.

Fredlund et al. (2010) have showed that matric suction of soil increases with increasing crack volume. It occurs because of evaporation and infiltration conditions. The matric suction is basically uniform along the ground surface, when there is a considerable volume of fractures in the soil.

Exact assessing of soil shrinkage fractures geo-material factors is not an easy work which is done by direct measurement. Irregular shape and complex crack patterns can cause measurement errors. Pagliai and Petruzzelli (1978) have used image analysis techniques extensively to characterize the crack network with enhanced preciseness.

2.8 Geometry of Cracks

Determination of the crack geometry is required to describe different phenomena related to desiccated soils. Bosscher and Douglas (1988) defined the need for exact depiction of the spatial quality of joint systems involving desiccation cracks. This feature was needed for groundwater models to conclude flow in fractured soils and for geotechnical models which determine the strength elements of cracked soils. Domenico and Schwartz (1990) have proved that understanding of the spatial characteristic of coupled systems is needed in environmental applications to model dispersion of contaminants correctly. Benson and Daniel (1994) also emphasized on the importance of knowledge regarding the geometry of macro-pores for porous media flow models.

Morris et al (1994) suggested an analytical explanation to predict the depth of fractures for a steady state suction distribution from ground surface to water table. It suggested that suction was a maximum value at ground surface and zero at the water table. Fredlund and Rahardjo (1993) also mentioned another analytical solution for depth of fracturing as a function of depth to groundwater table, Poisson's ratio, soil density, matric suction, and soil elastic moduli. They assumed a linear matric suction extending from the groundwater table to the ground surface.

Morris et al. (1992) reported that macro-cracks produced by the growth of micro-cracks under tensile loading at crack tips. He also stated that the suction at the crack tip is contrarily proportional to the radius of capillaries and hence to particle size.

Morris et al. (1992) illustrated that matric suctions in uncracked soils create compressive stresses between the particles. Therefore, conditions of crack development are most favorable at the ground surface where self-weight stresses are zero and matric suctions are at maximum level. They proved that the depth of cracks is probably restricted by the increasing stresses due to soil weight, and their surficial length limited by intersection with other fractures.

Characterization of desiccation cracks is highly dependent on the cycles of desiccation being observed. The initial crack pattern is primarily linked to many small branches. Crack generated during this cycle did not penetrate the entire depth of the soil layer. Following moisture addition, desiccation is allowed to continue. The cracks which formed after moisture addition developed a polygonal pattern of crack networks and some penetrated the entire liner thickness.

2.9 Effect of Pore Water Chemistry

There has been incrementing acknowledgment over the past 40 years or so of the unique difficulty related to geotechnical engineering in expansive clays. Mostly, these difficulties are associated with the modifications of volume caused by humidity regime or pore water chemistry. These volume changes can be enormous enough to create undesirable distortion of foundations resulting in super-structure harm. Changes in moisture regime and pore water chemistry also produce large alterations in other soil properties mainly shear strength, soil stiffness and permeability.

Expansive clays can experience large volume changes in reaction to changes in their moisture conditions or pore-water chemistry. It is usual for moisture caused strains of up to 10% to be faced (Aitchison et al, 1973).

Both moisture dependent and salt dependent volume changes are due to interactions between reactive clay minerals and the pore water ingredient, meaning the dissolved salts and water molecules. Volumetric expansion is caused by a decline in cation charge density. This is induced by a reduction in the pore water salt gathering or by a reduction in the salt cation valency, such as substitution of Ca2+ with Na+ ions (e.g, Richards et al., 1984). Making moist of a soil with a solution with salt concentration less than as salts do not enter into the evaporation water, and also induced shrinkage.

Increase in pore water salt concentrations or a growth in exchangeable cation valency will create shrinkage. Desiccation of the soil is the elimination of moisture induced by evaporation. As salt can’t be evaporating like water, then the concentration of salts will increment as moisture is eliminated and shrinkage will happen.

In practice engineers are interested in moisture and salt flow process to make possible an analysis of the anticipated importance and rate of change in the moisture and salt regimes. These analyses consider the change of permeability, and with changing moisture state is significant when soil is in unsaturated state (Hilel, 1980). Aitchison et al. (1977) pointed out the significance of time influence. It has been presented for the first time that many properties of expansive clays are dependent to water content and salts in the pore water, namely volume, stiffness, shear strength and permeability. So to quantify the amount and rate of changes in salt and moisture, we are also concerned with flow process in expansive soils.

It is widely identified that clay particles are plate-like in shape and have negative charges upon their faces and positive charges along their edges (Lambe and Whitman, 1969). The pore fluid within a clay soil consists of two primary components: water molecules and dissolved salt ions. The dissolved salt ions commonly present are H+, Ca2+, Mg2+, Na+, K+, NH4+, OH-, CL-, HCO3-, NO3-, SO42-, Co32- (Hillel, 1980).there are commonly some salt exist as they are essential for the chemical of the clay minerals, despite the fact that they might have been removed through prior leaching process.

Water is held to the clay surface by surface adsorption forces (hydrogen bonding) and also by cation hydration of exchangeable cations. The presence of a cation concentration gradient between the adsorb layer and the free water effect in an osmotic exertion of force gradient, with moisture at the clay surface being at a lower pressure than the water in the free water. The osmotic pressure gradient is efficiently a surface adsorption impose on the water as it transfer water close to the clay surface. This mechanism is mentioned as osmotic inhibition (Richard, 1980).

Free water at air-water interface is in equilibrium with the pore air humidity in conformity with kelvins equation, which depicts equilibrium between capillary surface tension impact and the pore air humidity, in addition to the equation of parington, which depicts the equilibrium conditions between a solution of salts and the pore air humidity (Wray, 1984; Schrieiner, 1987).

Sridharan and Jajadeve (1982) discovered the connection between void ratio and pressure is non-linear for salt assembly above 10^-4 where salts have an impact on p versus e relationships.

Less information is available on shrinkage characteristic of organic soils such as peats or clay peats (Gebhardts, 2011). According to Peng and Horn (2007), and Szatylowicz (1996) they differ from those of soils. Brandyk (2002) mentioned that the shrinkage intensity depends on type of organic matter.

Our goal is mitigate the impact of desiccation cracks and investigate effect of pore water chemistry on this aim.

2.10 Salinity

In spite of extensive research on the topic, it is difficult to understand (1) the role of soil structure, texture and pore-fluid characteristics, (2) shrinkage-swelling hysteresis, and (3) the effects of the stress history on volumetric deformations. For example, Musielak and

Mierzwa (2009), and Péron et al. (2007) had showed that soils of varying textures demonstrate different volume change behavior when the pore-fluid chemistry changed. Dane and Klute (1977), Jozefaciuk et al (2006), Lima and Grismer (1992), Musielak and Mierzwa (2009), Péron et al. (2007), Waller and Wallender (1993) illustrated that soil texture and pore-fluid properties influence the depth, width, spacing, and form of shrinkage cracks Yukselen-Aksoy et al. (2008), Mansour et al. (2008), Arasan and Yetimoglu (2008), and Shariatmadari et al. (2011) have demonstrated increase in salinity of pore fluid causes decrease in Atterberg limits. Sivapullaiah and Manju (2005) have investigated the influence of NaOH solution on some geotechnical features of the soil with low plasticity and concluded that the liquid limit of the soil increases with increase of NaOH solution concentration. The effect of NH4 Cl, KCl, CuSO4 and FeSO on consistency limits of CL clay and CH clay was examined by Arasan and Yetimoglu, (2008) who showed that salt concentration increases the liquid limit of low plasticity clay while decreases the liquid limit of high plasticity clay.

Another important test is the one-dimensional consolidation. This test indicates soil compressibility quantity that is one of the consequential features of the soil. The result of Yukselen-Aksoy et al. (2008) showed that seawater has restricted influence on the compressibility of clayey soils with liquid limit less than 110%, however its influence is significant when liquid limit is more than 110% (Na-bentonites). Rao and Mathew (1995) have reported the impact of monovalent, divalent and three-valent cations on the test results of consolidation. They have showed that substitution of monovalent cations by cations of higher valency changes the dispersion to flocculation, as it reduces double layer thickness. Siddiqua et al. (2011) have inspected the effect of NaCl and $CaCl_2$ solutions on light and dense backfill materials and observed that compressibility and swelling indices decrease considerably as pore fluid salinity increases. Shariatmadari et al. (2011) also have shown that as the salt concentration increases, the compression index (C_c) reduces. Again it is concluded that this occurred because of reduction in diffuse double layer thickness. Chattopadhyay (1972), Di Maio and Fenelli (1994), Alamdar (1999), Tiwari et al. (2005), Ayininuola et al. (2009), Naeini and Jahanfar (2011), and Siddiqua et al. (2011) had been estimated the influence of water chemistry on shear strength of soil. Shear strength of soil has risen due to concentration of water is augmented.

To understand the relationship between pore water salinity and soil properties information on the constituents of the dissolved salts and sodium concentration are required. Increasing electrolyte concentration enhances flocculation, whereas sodium saturation may cause dispersion. Adsorbed sodium causes dispersion of soil particles mainly due to the large size of its particles which causes sufficient separation of soil particles to cause repulsive forces between like molecules exceeding bonding forces. This leads to dispersion. Therefore, as Miller and Donahue (1999), Ayers and Westcot (1976), Bauder (2001), Bauder and Brock (2001), Buckman and Brady (1967), Chen and Banin (1975), Falstad (2000), Frenkel et al (1978), Hanson et al. (1999), Hardy et al. (1983), and van de Graaff and Paterson (2001) studied before, soil dispersion initially starts with high sodium concentrations, which later due to sodium saturation causes platelet and aggregate swelling. The other ions such as calcium and magnesium are smaller, therefore they do not show the same effect, and instead non-hydrated divalent cations cluster closer to the clay particles. This is due to +2 charge has a stronger attraction to clay surfaces than sodium, which has a +1 charge. Basically, attractive forces which bind clay particles together are disrupted when too many sodium ions get between the clay particles. Hanson et al. (1999), Buckman and Brady (1967), Chen and Banin (1975), Falstad (2000), and Frenkel et al. (1978), show that when such separation occurs repulsive forces begin to dominate, and the soil disperses.

Recently, many researches, such as Lee et al. (2005) have studied the interaction between clay soils and different types of fluids. Hydraulic conductivity is one of the important basic soil characteristics. The hydraulic conductivity should be less than or equal to 10^{-7}cm/s for barrier soils. It shows that fluids have various effects on geotechnical properties of clays. Mesri and Olson (19711) reported that calcium cations in the fluids passing through montmorillonite minerals, increases the hydraulic conductivity up to 28 times. Yong et al. (1992) concluded that increasing the concentration of salts, especially NaCl, by decreasing the inter-particle repulsion among negative charged plates increase the permeability in a natural soil.

As a result, we can say that, the increase in hydraulic conductivity, when the concentration of the salt solutions was increased, is due to the decrease in the thickness of DDL, which resulted in flocculation of the clay particles. Quigley (1993) pointed out that clay minerals might experience large interlayer shrinkage in contact with specific chemicals. This phenomenon is joined by enormous loss in Diffuse Double Layer (DDL) thickness, potential cracking, and increase in hydraulic conductivity values.

Mitchell (1993) and Fukue et al (1999) pointed out that the thickness of the DDLs is a significant factor which can control the structural development, hydraulic conductivity, and other physicochemical and mechanical properties of soils. Moreover, Van Olphen (1963) showed that the thickness of DDLs around clay particles is controlled by the concentration of salt and type of cation(s) in the soil moisture. Referring to Gouy-Chapman theory, when the ion concentration increases which is the outcome of flocculation of the clay particles forming larger pore channels through which flow can take place, the thickness of the DDL reduces (Mitchell, 1993; Gleason et al., 1997; Kaya and Durukan, 2004). Additionally, Bowders and Daniel (1987) supported that many chemical factors tended to reduce the thickness of the DDL, and cause the soil skeleton to shrink causing a decrease in repulsive forces, and it can promote flocculation of clay particles and dehydrate interlayer regions of expandable clays, which subsequently become granular. Kaya and Fang (2000) also implied that as repulsive forces reduce the soil particles flocculate and form aggregates because of attractive forces between particles, it leads to expansion of the effective flow area and resulting in enlarged hydraulic conductivity of the soil-pore fluid.

Soil structure affected by the thickness of the diffused double layer, and it result in permeability changes. The thickness of this diffused double layer can be influenced by various parameters such as the dielectric constant of the medium, cation valence, electrolyte concentration. Sharma and Levis (1994) implied that when fluids containing various chemicals or leachate permeate underlying clay liners, they change different factors that can impact the thickness of the diffused double layer and the permeability of the permeated clays. Mitchell (1993) stated that based on double layer theories, a decline in the fluid dielectric constant decreases the double-layer thickness and permits clay particles to approach closer to each other. This leads to shrinkage of soil clusters. In the same way, many other investigators like Gleason et al. (1997), Ruhl and Daniel (1997), Kaya and Fang (2000) indicated that low dielectric constant liquids may cause the shrinkage of the clays, which leads to cracking and hence increase in hydraulic conductivity.

Changes in the chemical composition of the pore fluid have effects on clays in different ways. These changes can cause exchanges of cations between the mineralogical units, variations in the electrochemical forces acting between different platelets and variations in the osmotic pressure. From the microscopic scale, the distance between different unit layers depends on the valence, dimension and hydration of the interlayer cations. From the mesoscopic scale, the ionic strength of the solution controls the repulsion forces between different particles as well as the osmotic pressure in the micro pores. Musso et al (2003) pointed out that at the macroscopic scale; the distance between peds is such as the repulsion effect is lost, while the effect due to changes in the bulk osmotic pressure remains. Yong and Warkentin (1975) determined a decrease in swell potential by increasing salt concentrations in clays containing univalent exchangeable cations. Basma and AlSharif (1994) then reported that the effect of the salt concentration on the pores decrease by exceeding ascertain degree of concentration.

Reaction of soil-salt has been studied by a number of researchers. Moore (1973) found that sodium chloride can release silica and alumina. Van Lier et al. (1960) concluded that the solubility of silica was greatly increased in the presence of sodium chloride. Singh and Das (1999) found that sodium chloride is a stabilizing agent for highway construction, and they reported that California bearing ratio test value, unconfined compression strength

(UCS), resilient modulus, and indirect tensile strength are greatly improved with the inclusion of sodium chloride as a stabilizing agent. According to Moore (1973), salt treatment produces higher maximum dry density at lower moisture content; increases the drying shrinkage in most soils and decreases the permeability. Moh (1962) found that increasing salt can increased the compressive strength of the treated soils by mixing with chemical admixtures such as sodium chloride, calcium chloride and sodium hydroxide in the clay.

Modmoltin and Voottipruex (2009) also concluded that the Na^+ in the natural clay increases the dissolving soil silica and alumina, which induces higher strength than that of Ca^{2+}. Modmoltin et al. (2004) showed that the salt in soil can decrease the effect of organic matter on the strength of soil.

2.11 Soil Suction

According to Fredlund and Rahardjo (1993), Bulut et al. (2001), Ridley et al. (2003), Rao and Shivananda (2005), and Sreedeep and Singh (2006) soil suction is a microscopic state that shows the severity or free energy degree (force per unit area) of water that the soil attracts. Soil suction includes two ingredient; osmotic and matric (capillary) suctions. Matric suction is a fine and exact phenomenon because of capillary nature, texture and adsorptive forces of unsaturated soils, and it changes with changes in soils water content. The osmotic suction is the result of the attendance of dissolved salts in the pore fluid. The whole amount of the matric suction and osmotic suction are related to total suction. Chen (1998) defined the relationship between the total suction and the osmotic and matric suctions, based on isothermal needs is as displayed in Equation 2.1.

$$h_t = h_0 + h_m \qquad (2.1)$$

Where h_t is the total suction, h_0 is the osmotic suction and $h_m$is the matric suction

Most of engineering-related difficulties are due to partially saturated soils where the void spaces between tiny particles are partly filled with air and partially with water. This leads to negative pore water pressures (or suctions), which affect the stress system. Therefore, the exact measurement and interpretation of soil suction is needed to understand the behavior of unsaturated soils. However, according to Murray and Sivakumar (2010), magnitudes of suction can fluctuate immensely (between 0 and 1 GPa) and the tools and measurement methods are practical only over unique suction ranges.

Soil suction and water content are among significant parameters that govern many geotechnical properties of unsaturated soils including hydraulic conductivity, volume change, compressibility and shear strength. Barbour (1998) stated that soil water characteristic curve (SWCC) illustrates the relationship between moisture content and suction in unsaturated soils. Because of its importance, different test procedures have been developed to develop SWCC in. Several mathematical models have been developed to fit the experimental data of SWCC. The most famous and widely used model is van Genuchten (1980) and its several modifications and extensions.

For suction measurements, filter paper method was used in Europe in the 1920s and Gardner brought it to the United States in 1937. After that, the filter paper method has been used and studied by many researchers such as Fawcett and Collis-George (1967), McQueen and Miller (1968), Al-Khafaf and Hanks (1974), McKeen (1980), Hamblin

(1981), Chandler and Guierrez (1986), Houston et al (1994), Swarbrick (1995), who implemented the method differently.

Calibration technique of the filter paper method has been studied by Houston et al. (1994), Bulut et al. (2001), and Leong et al. (2002) to obtain accurate calibration of water content of filter papers in terms of suction. Different calibration curves were established for different filter paper brands. The most popular filter papers are Whatman No. 42 and Schleicher &Schuell No. 589-WH.

2.12 Tensile Strength

Geotechnical engineers are familiar with classical shear strength concepts, but not with fracture mechanics, Mode I (tensile), II (shearing), and III (tearing) fracture behaviors. Tensile strength of soil is the resistance of the soil to forces inducing cracks. According to Amarasiri and Kodikara (2011), soil tensile strength controls initiation and propagation of cracks. Tang and Graham (2000), Kim and Hwang (2003), Nahlawi et al. (2004), Ibarra et al. (2005), and Kim et al. (2007) have carried out extensive studies including difficult sampling techniques and complex experimental procedures for determination of effect of clay content (CL), clay mineralogy, cation exchange capacity (CEC) of the clay, soil organic matter and wetting/ drying cycles (Guerif 1990; Barzegar et al. 1994, 1995), hydrogen ion concentration (Kim et al. 2007), soil suction, soil electrical conductivity, sodium adsorption ratio, and the amount of organic matter on tensile strength.

Leonards and Narain (1963) used a simple technique to measure tensile strength by flexural test using a clay-beam. Conlon (1966) conducted free tension experiments on clayey silt similar to tri-axial apparatus. Spencer (1968) and Suklje (1969) assessed the impact of tensile strength in cohesive slopes, where creep and critical stress states with tensile stresses occurred in the upper parts of the slopes. Bofinger (1970) usd a prismatic specimen, the ends of which were bonded to steel plates with quick setting polyester resin, and the tensile force was applied through a cap. George (1970) applied the brittle fracture theory to evaluate crack growth and its effects on stabilized soil-cement mixtures.

Chapter 3

MATERIALS AND METHODS

The material used in this study is swelling soil gathered from South Campus of the Eastern Mediterranean University, Famagusta, North Cyprus. The experimental methods described in this chapter are conducted on swelling soil samples prepared with distilled water, and with four different concentrations of salt in pore water. NaCl and $CaCl_2$ prepared at molarities of 0.5 M and 1 M for both to simulate seawater (0.6 M) and salt concentration higher than sea water.

3.1 Physical properties

3.1.1 Liquid Limit

The moisture content is a percentage of the weight of the oven-dried soil at the boundary between the liquid and plastic states of consistency, which is the liquid limit of a soil. The moisture content at this boundary is arbitrarily specified as the water content at which two halves of a soil cake will flow together, for a distance of 12.7 mm along the bottom of a groove of standard dimensions separating the two halves, when the cup of a standard liquid limit apparatus is dropped 25 times from a height of 10 mm at the rate of two drops/second. Using ASTM D 4318 the liquid limit in this study is obtained to be 61%.

3.1.2 Plastic Limit

The plastic limit of a soil is the moisture content, expressed as a percentage of the weight of the oven-dry soil, at the boundary between the plastic and semi-solid states of consistency. It is the moisture content at which a soil will just begin to crumble when rolled into a thread 3mm in diameter using a ground glass plate or other acceptable surface. The expansive soil tested is found to possess a plastic limit of 30%.

3.1.2 Linear Shrinkage

This test covers the determination of the linear shrinkage of a disturbed soil sample. It is a boring and expensive test that is done only on soils (other than sands) when the dispersion percentage is >50 or volume expansion tests fail to saturate or shrink. This test is performed on dispersive soils only.

Linear shrinkage (BS-1377:90) means the decrease in length of a soil sample when oven-dried, starting with a moisture content of the sample at the liquid limit.

$$L_S\,(\%) = (1 - (L_{avg}/L_0) \times 100 \qquad (3.1)$$

Where:

L_{avg} = average length (mm)

L = length of mould (mm)

L_s = longitudinal shrinkage of specimen (mm)

L = length of the specimen at beginning of test

No specific information on the exactness of the test is available. However, experienced operators pointed out that only one determination is necessary for each soil sample. It is generally preferable, for soil characterization, to conduct the test for a number of samples rather than duplicate determinations of the one sample.

Figure 3.1: Linear shrinkage test molds and dried samples of varying pore-water chemistry.

3.1.3 Grain Size Distribution (Sedimentation test)

Percentage of sand, silt and clay in inorganic fraction of soil is measured in this procedure. The method is based on Stoke's law governing the rate of sedimentation of particles suspended in water. The sample is treated with sodium hexametaphosphate to disperse Ca^{++}, Al^{3+}, Fe^{3+}, and other cations that bind clay and silt particles into aggregates. Organic matter is suspended in this solution. The density of the soil suspension is determined with a hydrometer calibrated to read in grams of solids per liter after the sand settles and again after the silt settles. Corrections are made for the density and temperature of the dispersing solution. ASTM D422-63 is employed to do this test.

3.1.4 Specific Gravity

Specific gravity is the ratio of the mass of unit volume of soil at a stated temperature to the mass of the same volume of gas-free distilled water at a stated temperature. To determine the specific gravity of soil a pycnometer is used and the test is conducted according to ASTM D 854-00. The specific gravity of the clay is found to be 2.62 in this study.

3.2 Compaction Test

Compaction test is the densification of soil materials by using mechanical energy. Soil is compacted by removing air and water from its pores. There is a certain moisture content at which soil can attain a maximum unit weight. There are two measurement techniques presented by the American Association of Highway Officials and American Standard Testing Materials: the Standard Proctor test (ASTM D-698, AASHTO T-99) and the Modified Proctor test (ASTM D-1557 and AASHTO T-180). Standard Proctor compaction test method was used in this study. The optimum water content and maximum dry density were found to be 25% and 1.451 respectively.

3.3 One- dimensional Swell Test

One-dimensional swell test was conducted in accordance with ASTM D4546-96. Soil specimens to be tested were mixed with distilled water or with NaCl and $CaCl_2$ solutions of 0.5 and 1.0 molarity added to optimum water content. After the mellowing time, they were compacted to maximum dry density using Proctor energy.

Specimens were inundated by filling the consolidation cells with water or salt solutions of different molarities and subsequently swell measurements were recorded at increasing time intervals.

3.4 Volumetric Shrinkage

After seven days of swelling, samples were taken out and left to desiccate at 40°C. At every one hour interval, the samples were weighed, and the height and diameter of each were recorded. After 48 hours specimens were dried at 110°C to find the water content. Figure 3.2 shows the specimen mass and dimensions measurement of volumetric shrinkage testing procedure.

For cyclic swelling-shrinking test the same procedure for one-dimensional swell and volumetric shrinkage was repeated four times.

Figure 3.2: Volumetric shrinkage test: Measurements of (a) mass, (b) dimensions of drying specimens.

3.5 Consolidation Test

This test was performed to determine the magnitude and rate of compressibility that a laterally confined soil specimen undergoes when subjected to different vertical pressures in the axial direction, samples prepared and inundated with salt solutions. From the measured data, consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining compression index, rebound index preconsolidation pressure (or maximum past pressure) as well as swell pressure of the soil. In addition, the data obtained can also be used to determine coefficient of consolidation which yields saturated hydraulic conductivity indirectly. The test is performed according to ASTM D 2435 - Standard Test Method. Figure 3.3 shows the consolidation tests performed using transducers for deformation measurements connected to a computerized data acquisition system.

Figure 3.3: One-dimensional swell-consolidation tests run with transducers.

3.6 Unconfined Compression Test

The undrained shear strength (s_u) of a cohesive soil is equal to half the unconfined compressive strength (q_u) when the soil is under the $\varphi_u = 0$ condition (φ_u = undrained angle of internal friction). The most critical condition for the soil usually occurs immediately after construction, which represents undrained condition, when the undrained shear strength is basically equal to the cohesion (c). This is expressed as $c = q_u/2$.The undrained shear strength (s_u) of clays is usually determined using an unconfined compression test. ASTM D 2166 - standard method is implemented in this study on compacted specimens of 38 mm diameter and 76 mm height. Figure 3.4 displays a failed specimen at the end of compression test, and the computer screen showing the test in progress. Data were collected by data acquisition system and evaluated by the appropriate software plotting the stress-strain relationship spontaneously during the test.

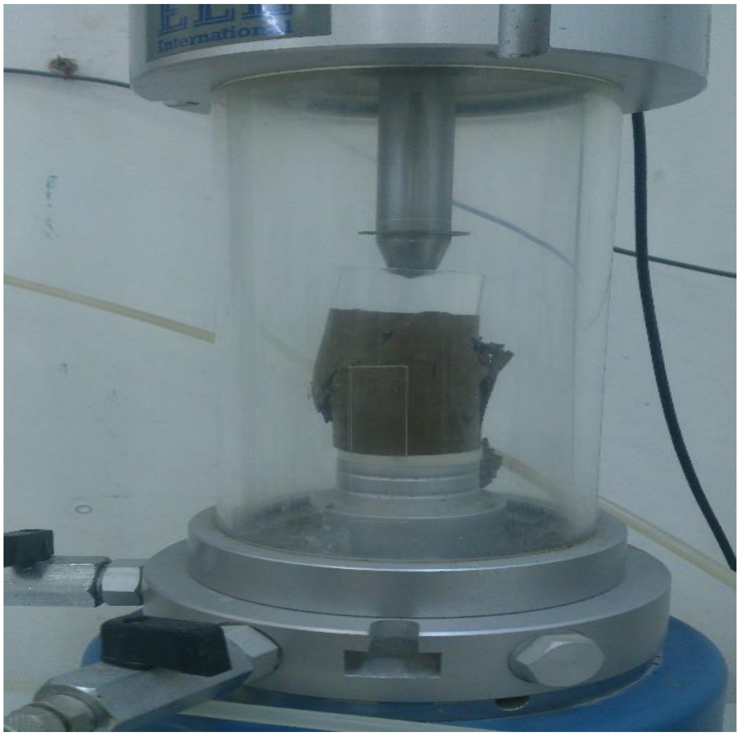


Figure 3.4: Unconfined compression test (a) sheared specimen, (b) test progress monitored by computer.

3.7 Unconfined Penetration Test (Tensile Strength Test)

In this test, soil sample was compacted in the compaction mold. Its weight was measured to calculate the unit weight, after each specimen was prepared. Then the specimen was placed on the base plate and was lined up with the disks. This alignment between two disks and the soil specimen is very important in the application of load. After that, by moving the axis of the loading arm until the bottom surface of the specimen was in contact with the bottom disk, the bottom disk was raised. A load was steadily applied at the selected rate (2%= min, 1%= min, 0:5%= min, or 0:1%= min). The force was applied until failure occurred in the specimen. During testing, the force and displacement were recorded through the data logger. The peak load was used for calculating the tensile strength of the mixed soil specimen by Equation 3.2. After the test, the fractured specimen was collected quickly and weighed both before and after oven drying to determine its water content.

$$\sigma t = Pmax/(\pi(MbH - a^2) \quad (3.2)$$

Where:

M= $\tan(2 \propto + \emptyset)$

a = Radius of the disk

ϕ = Undrained friction angle of soil

b and H = Specimen radius and height

Figure 3.5 shows the aligned specimen before test commenced, as well as a failed one after the test was completed.

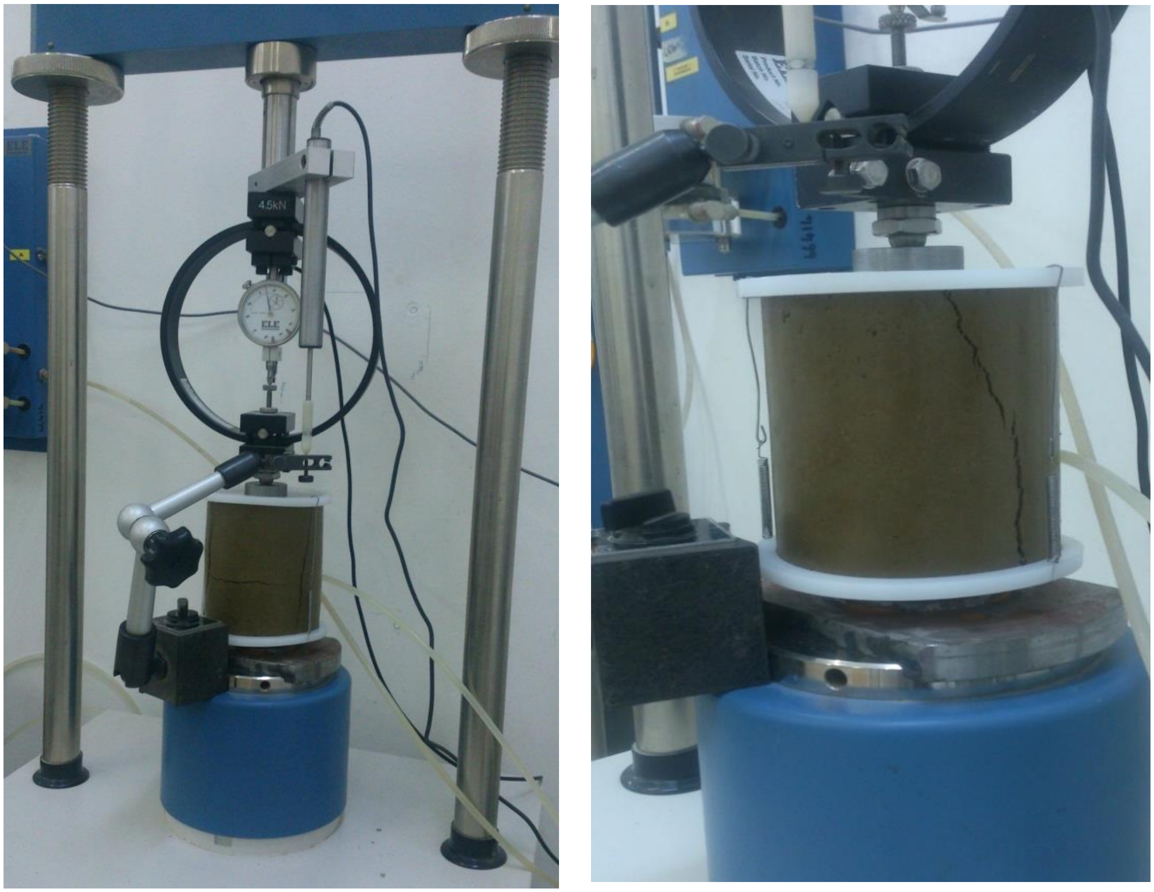


Figure 3.5: (a) Unconfined penetration test set up, (b) Failed sample.

3.8 Filter Paper Method for Suction Measurement

For the implementation of the engineering behavior of unsaturated soils theories, assessment of soil suction is essential. Because of its easy testing procedure and data analysis, the filter paper method is widely used to measure soil suction. The filter paper method uses a filter paper to reach vapor equilibrium with the soil. When filter paper is in intimate contact with the soil, moisture from the soil samples migrates to the filter paper by capillary flow. The water content of the filter paper can be correlated to matric suction. When the filter paper is kept above the sample, water is passed to the filter paper above the sample by vapor transfer, and thus gives a measure of total suction. The most generally used filter papers are the Whatman No. 42, and the Schleicher and Schuell No. 589 papers, both of which have ASTM calibration curves (ASTM D 5298, 2003).

The mellowed soil at optimum water content was manually compacted in moisture containers to maximum dry density. When all samples were compacted in containers they were sprayed with water and kept sealed, and more water was added until full saturation was assured. Figure 3.6 shows the containers which are left closed for the water content of soil samples to equilibrate. Then they were placed in the oven at 40°C and at different time intervals matric suction measurement by filter paper technique was applied. Figure 3.7 shows the samples in containers desiccating at 40° C in the oven. Figure 3.8 depicts the filter paper placement on a sample which is at a certain level of drying. The test procedure involves placing a piece of initially oven dried Whatman No. 42 filter paper against the compacted soil specimen and sealing the container to prevent evaporation. The paper was ensured to have an intimate contact with the soil, so that matric suction could be obtained. A sacrificial paper was placed first and two more papers were placed on top of it for two measurements of water content in order to get an average value. Each sealed container is then placed in a styrofoam box and kept in a cabinet. The filter paper absorbs moisture up to water content in equilibrium with the sample and careful measurement of the water content of the filter paper enabled the soil matric suction to be obtained from a previously established calibration curve.

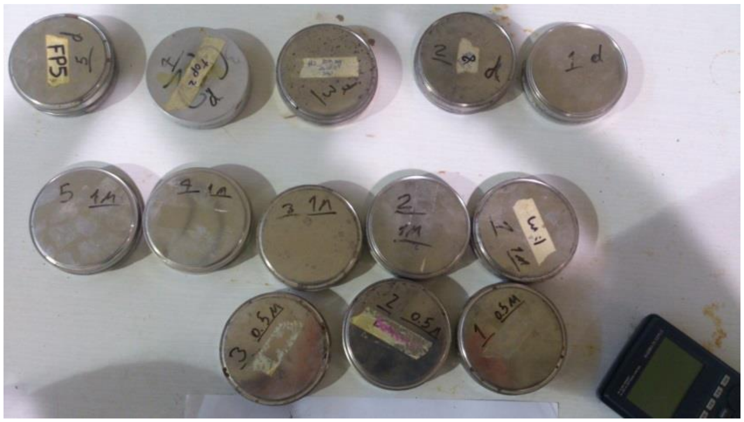


Figure 3.6: Compacted and saturated samples sealed in moisture containers.

Figure 3.7: Desiccation of saturated samples in the oven at 40° C.

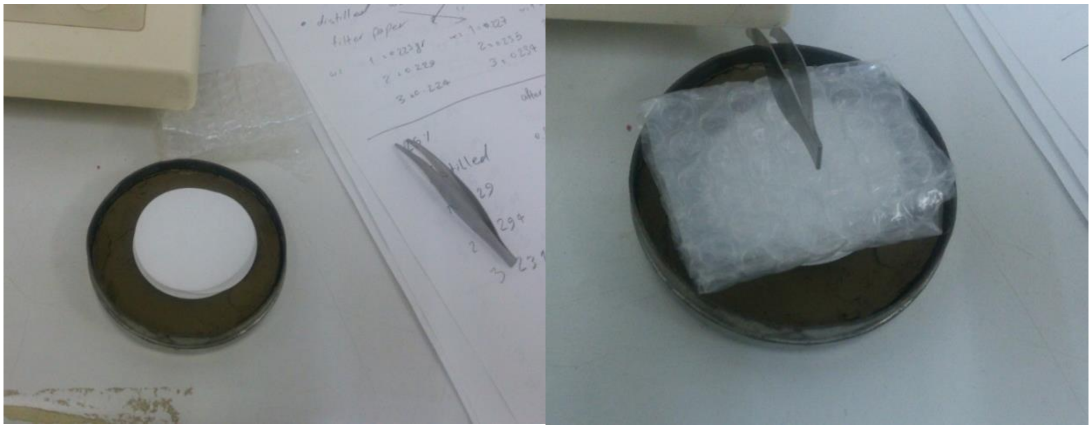


Figure 3.8: (a) Placement of filter papers on soil, (b) Use of bubble wrap to ensure the filter papers will be in intimate contact with the soil when the container is sealed.

3.9 Desiccation of Large Scale Samples in Tanks

In order to simulate natural condition in the laboratory, soil was placed in a tank of diameter 40 cm and height 30 cm to a predetermined level and filled with water. The water was let to drain from the holes at the lower side of the tank. More water was added until the soil was saturated completely and then left to dry. It has taken approximately 1 month to be saturated and about one month to have dried. Photographs were taken before and after desiccation to determine crack intensity factor (CIF) with Photoshop software. Figure 3.9 shows the tank filled with the expansive soil in saturated and desiccated conditions. This part of the study was conducted on soil saturated with distilled water and with 0.5 M NaCl solution only. The aim was to observe the effect of salt on the crack intensity factor.

Figure 3.9: (a) saturated soil, (b) Desiccating soil with surface cracks.

3.10 Crack Intensity Factor (CIF) Determination Method

The crack intensity factor (CIF) was introduced as a descriptor of the extent of surficial cracking. CIF is defined as the ratio of area of cracks (A_c) to the total surface area (A_t) of a drying soil mass. A computer aided image analysis program was used to determine the CIF values. The areas were determined using photographs of desiccating soils. Cracks appear darker than the remaining uncracked soil surface in photographs of a drying soil. The contrast between the color of the cracks and the uncracked soil is used to calculate the CIF.

After taking photography a calibration of image-pixel size was performed as follows: (1) Using the magnetic lasso tool in Adobe Photoshop, the inside perimeter of the outer confining ring of oedometer apparatus with 50 mm diameter was traced and saved as a new layer, (2) for consistency, the selection's color threshold level was set to 128 (in 256 color-mode) so that the selection file can only have pixel values of 0 or 1 and (3) by knowing the diameter of the ring (R), the image pixel area (A PIXEL) was obtained by dividing the total internal area of the specimen by the total number of white pixels. The CIF is then determined as Surface area of cracks/Total surface area of soil. The desiccation cracks are also studied in smaller samples (50 mm diameter) which started cracking at the end of third wetting-drying period.

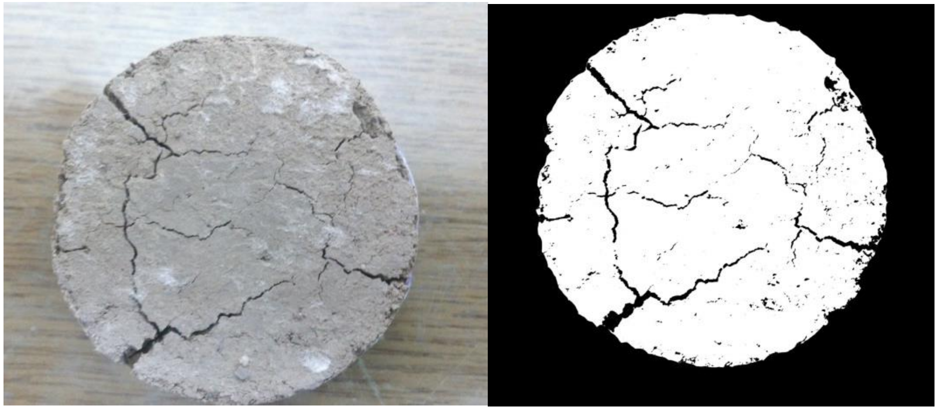


Figure 3.10: (a) Desiccated soil after third wetting-drying cycle, (b) Photoshop image of the cracked surface

3.11 Cyclic Swelling and Shrinking

Vertical displacements of the specimens during swelling and shrinking were recorded using a dial gauge with an accuracy of ±0.002 mm and a travel of 25 mm. The combination of one swelling and shrinkage cycle is designated as one swell–shrink cycle, shrinkage part is the same as volumetric shrinkage I mentioned earlier. After one swell-shrink cycle the specimen was inundated again with water for the subsequent swelling and then shrinkage process was conducted to complete the second cycle. Therefore, several cycles of swelling and shrinkage were performed until the specimen reached equilibrium conditions. Equilibrium was defined as the condition where swelling and shrinkage were of constant magnitude for each cycle. In the case of full swelling followed by full shrinkage tests, specimens in each cycle were allowed to fully swell and then shrink to a stage where there was no further change in height. The time allowed for each swelling process was about 7 days, and the full shrinkage process required was about 4 days to complete at a temperature of 40°C. The swell/shrink strain is calculated using Equation 3.3.

$$S\,(\%) = \Delta H / H0 \qquad (3.3)$$

In this equation, (ΔH) is the vertical deformation of the specimens represented as the change in height of the specimen (during either swelling or shrinkage) and is defined as a percentage of the initial height of the specimen (H_0) at each cycle. By plotting the vertical deformations of a specimen for several swell–shrink cycles, the percent change in height of the specimen during any of the swelling or shrinkage cycles can be observed.

Chapter 4

RESULTS AND DISCUSSIONS

4.1 Physical Properties

In this section discussions on experimental results are presented which describe the effect of salinity on physical properties, volume change, strength and hydraulic properties, and intensity of crack deformations as measured using the (RRM) and digital image correlation method. By using existing concepts of drying, the results were analyzed and qualitatively interpreted. A variety of parameters including soil texture, chemistry, mineralogy, clay content and spatial scale which influenced the volumetric response (horizontal and vertical) of the soil system.

Soil water salinity can affect soil physical properties by causing fine particles to film bind together into aggregates. This process is named flocculation and it is good in terms of soil aeration. At high levels salinity it can have negative and potentially detrimental effects for both engineering and also agriculture usage, although increasing soil salinity has a positive effect on soil aggregation and stabilization. Figure 4.1 illustrates comparison of liquid limit, plastic limit and plasticity indices of soils, with different concentrations of NaCl and $CaCl_2$. With increasing molarity of salts liquid limit decreases gradually while plastic limit increases, therefore higher the salinity lower is the plasticity index. The decrement in plasticity index is more defined when pore fluid consists of 1 M NaCl solution.

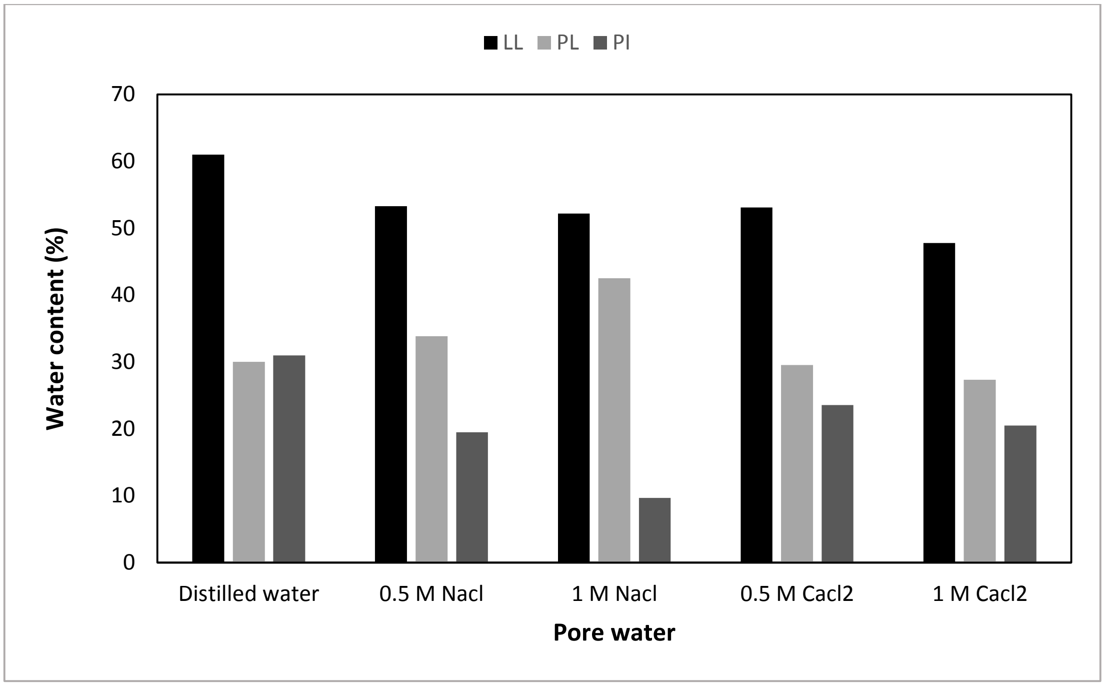


Figure 4.1: Variation of plasticity properties with pore water type.

In Figure 4.2 particle size distribution of the soil sample is given, from which can be observed that it consists of 55% clay, 35% silt and 10% sand.

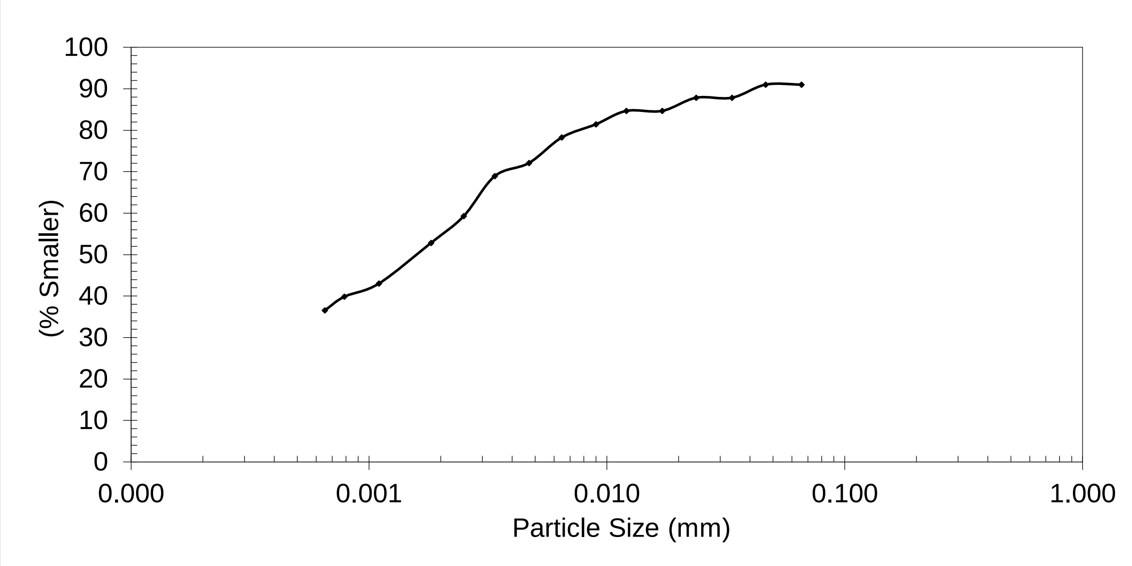


Figure 4.2: Particle size distribution of the soil used

Specific gravity (G_s) of the soil is 2.62 found as an average of several tests repeated. Another physical property test carried out is linear shrinkage in accordance to British Standard (BS-1377:90) the results of which are as depicted in Figure 4.3 which shows that increase in salinity reduces linear shrinkage.

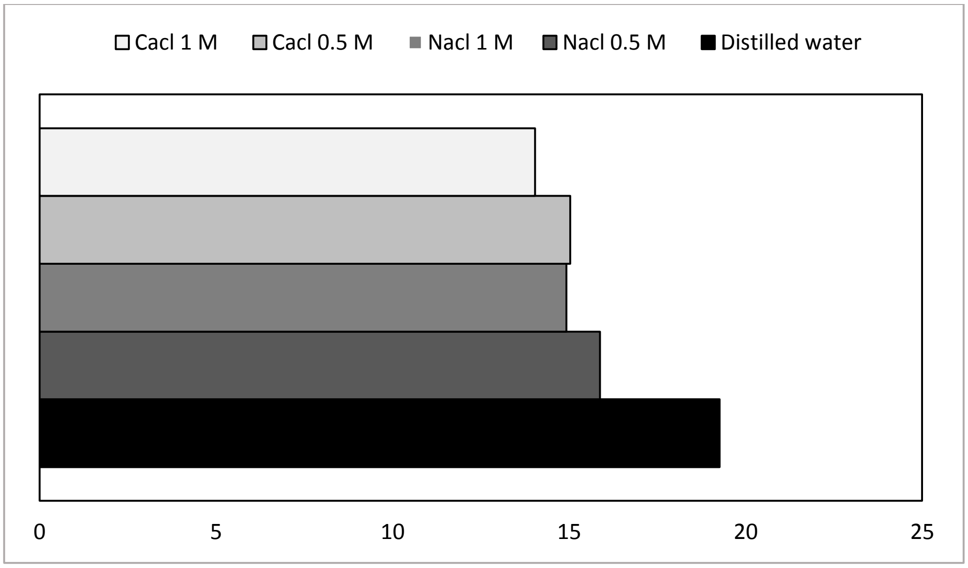


Figure 4.3: Variation of linear shrinkage with pore water chemistry

The samples for volume change and strength tests were prepared by compacting at Standard Proctor optimum water content and maximum dry density which are 25% and 1451 kg/m^3 respectively.

4.2 One-Dimensional Swell Test

Studying the swell curves in Figure 4.4, it can be observed that swell potential of the expansive soil used drops appreciably when the salt concentration in the pores as well as in the environment increases. The swell potential is slightly influenced by NaCl, whereas $CaCl_2$ reduces it to more than half of the swell percentage of samples in distilled water. Therefore, salt is observed to reduce swelling; in this treatment $CaCl_2$ is more effective than NaCl.. This behavior can be explained by diffuse double layer theory (DDL). Thickness of the diffuse double layer is dependent on the concentration of electrolyte and the valence of the exchanged cations. Van Olphen. (1997) indicated that increasing NaCl and $CaCl_2$ concentration is interpreted as being due to a decrease in double-layer swelling between quasi-crystals. As the electrical double layer adjacent to the quasi-crystal surface is compressed with increasing NaCl and $CaCl_2$ concentrations, the aggregate swell may decrease.

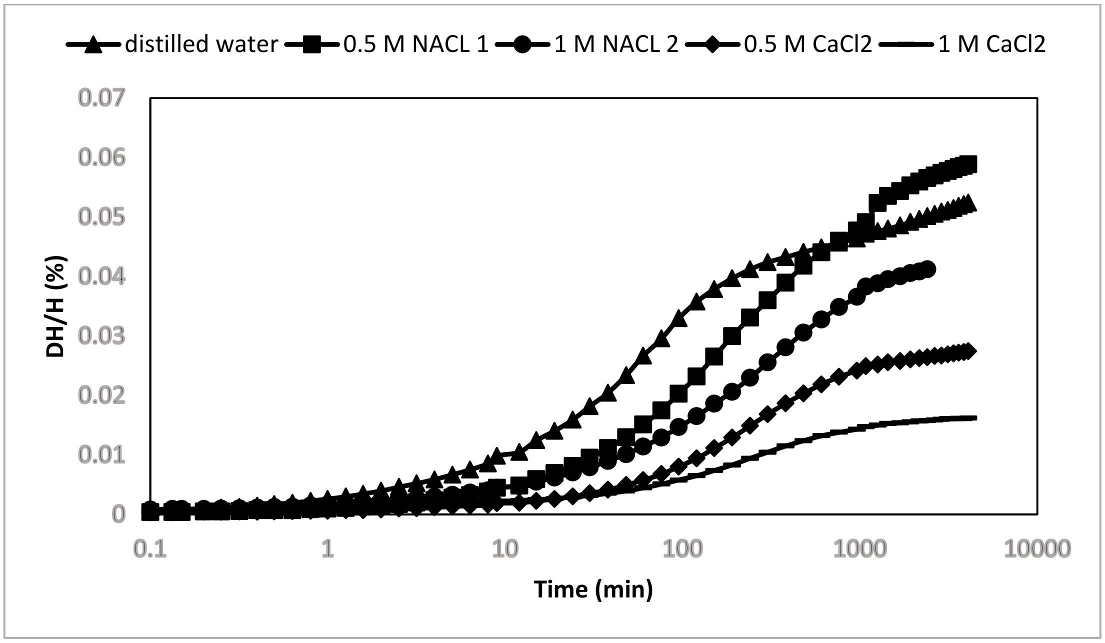


Figure 4.4: One-dimensional swell versus time curves

This behavior can further be explained using Derjaguin Landau, Verwey and Overbeekn (DLVO) theory (Szilvasszy, 1984). Figure 4.5 shows the interaction forces according to the Derjaguin Landau, Verwey and Overbeekn (DLVO) theory. This theory suggests that the stability of a particle in solution depends upon balance of attractive and repulsive interactions. The figure shows the difference between the resultant forces in low and high concentration of electrolytes in solution.

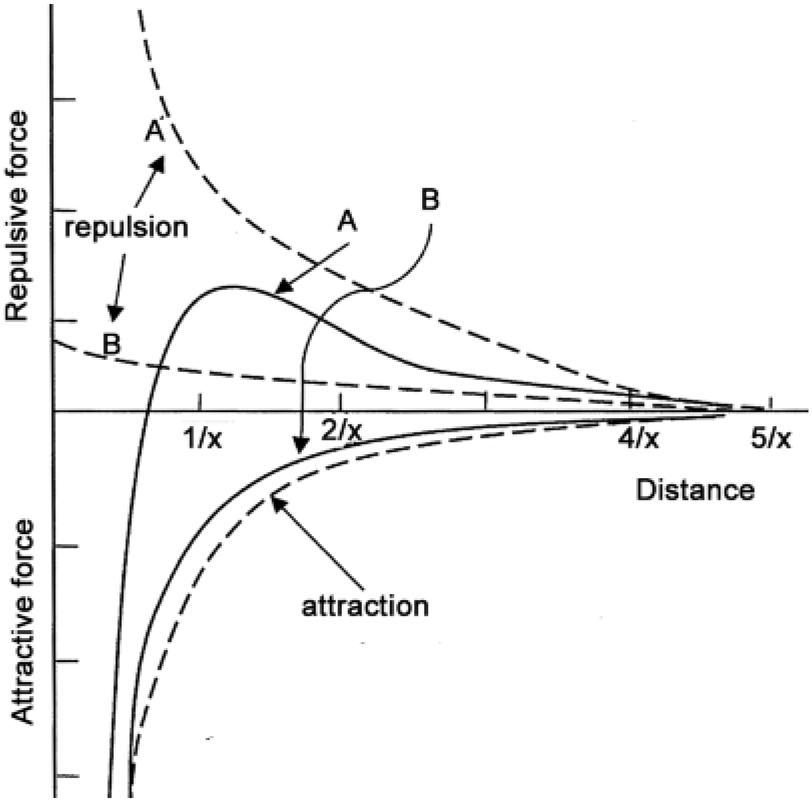


Figure 4.5: Interaction forces according to DLVO theory.

The attractive forces at the surfaces of clay particles stay constant but repulsive forces are dependent on the concentration of electrolytes. Lower concentration induce greater repulsive forces, whereas, the higher concentration induce lower repulsive forces and finally result in attractive force at the particle surfaces. Therefore, due to low concentration of salt, the particles have sufficiently high repulsion, the dispersion will resist flocculation and the colloidal system will be stable. However, repulsion mechanism does not exist in solutions of high concentrations of electrolyte and then flocculation or coagulation eventually takes place. The results of the present study are in agreement with the DLVO theory, concluding that higher the electrolyte concentration lower is the repulsion forces, hence lower is the swelling.

4.3 Volumetric Shrinkage

In volumetric shrinkage, water moves out of the clay interlayer region due to the gradient in free energy induced by the elevated concentration in the bulk pore water, when the concentration of cations in the bulk solution increases. Concentration has the largest effect on shrinkage for the monovalent (NaCl, KCl, and LiCl) solutions. Shrinkage decreases more rapidly in the solutions with divalent cations or trivalent cations ($CaCl_2$, $MgCl_2$, $ZnCl_2$, and CuCl2). If the concentration increases from 0.005 to 0.1 M, and thereafter (0.1–1 M) free swell decreases only a small amount, however a rapid decrease in shrink occurs.

In this study volumetric shrinkage has been conducted with different concentrations and defined that salt can reduce shrinkage. The results are given in Figure 4.6, as plots of volumetric strain ($\Delta V/V_0$) versus time. As can be observed, there is no marked difference in shrinkage strain between 0.5 M and 1 M concentrations of both salt types, however $CaCl_2$ has proved to be more effective in reducing the volumetric shrinkage.

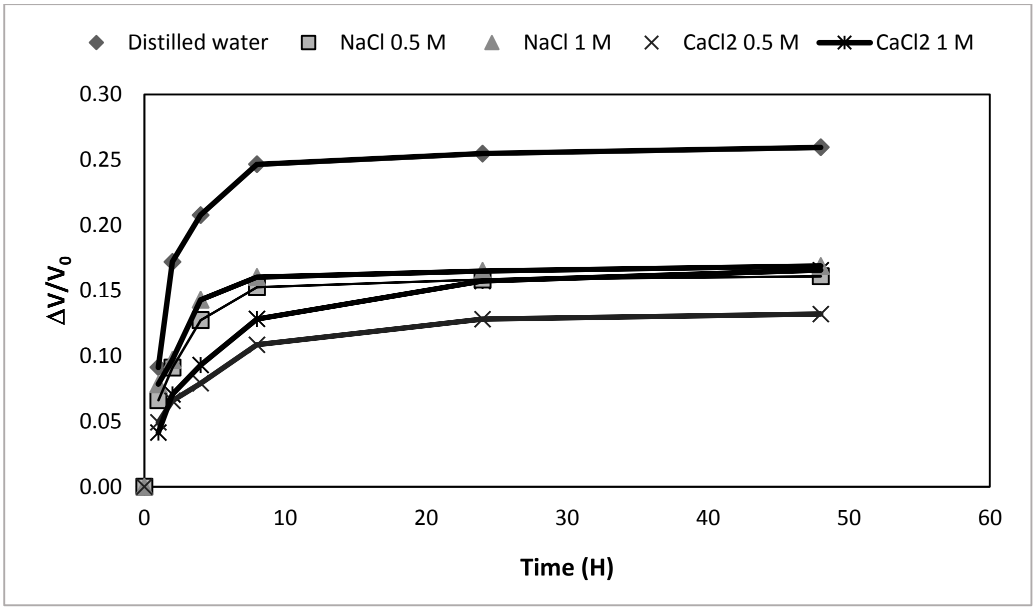


Figure 4.6: Volumetric strain versus time curves.

It is clearly seen that salinity has enormous effect on the swelling and shrinking rate of compacted clay. Suzuki et al. (2005) indicated that it might be due to a decrease in double layer swelling between quasicrystals by the existing salts in pore fluid. As the electronic double layer adjacent to the quasicrystal surface is compressed with increasing salt concentration, the aggregate swelling may decrease. Thus, aggregate swelling for salt solution is possibly controlled by both crystalline swelling and double layer swelling between quasicrystals. This behavior also influences the volumetric shrinkage.

The effect of the salinity of the pore water in clays has been researched and reported by many authors, such as Petrov and Rowe (1997), Alawaji (1999), Shackelford et al. (2000), Jo et al. (2001, 2004, 2005), Kolstad et al. (2004) Lee and Shackelford (2005), Lee et al. (2005) Mishra et al. (2005). A common finding was that salt solutions caused a collapse in the clay structure, a decrease in the thickness of the diffused double layer (DDL), an increase in hydraulic conductivity and a decrease in swell potential. All these effects are noted in this research study as well.

4.4 Cyclic Swelling and Shrinking

Cyclic swelling and shrinking behavior was studied only with the distilled water in the pores due to time limitation. Therefore the results presented in this section do not include any comparison with salt concentrations in the pores. At the end of the first shrinkage cycle, the void ratios and water contents attained by the specimens were at a minimum. However, at the end of other cycles higher void ratios and water contents were observed. Similarly at all swelling cycles, the void ratio and water content attained by the specimen at the end of the first swelling cycle were lower than the corresponding values. The water content and void ratio at the end of the second swelling cycle were higher than at any other cycle. The water content and void ratio decreased slightly and then stabilized, beyond the second cycle. Figure 4.7 depicts the swell percentage versus time curves during each swelling cycle.

For the first three cycles, hysteresis in swelling and shrinkage paths were observed. The void ratio at the end of the cycle changed, when each specimen was allowed to shrink. In the same way, when the specimen was rewetted, the void ratio and water content of the specimen at the end of the swelling cycle changed. This changing occurred until about the third cycle. In the fourth swell–shrink cycle, the path traced by the specimen during both swelling and shrinkage was found to be the same, signifying reversibility in the swelling and shrinkage paths and the elimination of hysteresis. Shrinkage part is not related to swelling part and first cycle has the lowest shrinkage and each shrinkage step increased gradually. Figure 4.98 shows the diametric and volumetric strains versus time at each drying cycle respectively, and Figure 4.9 depicts the maximum strains versus cycles of swelling and shrinking.

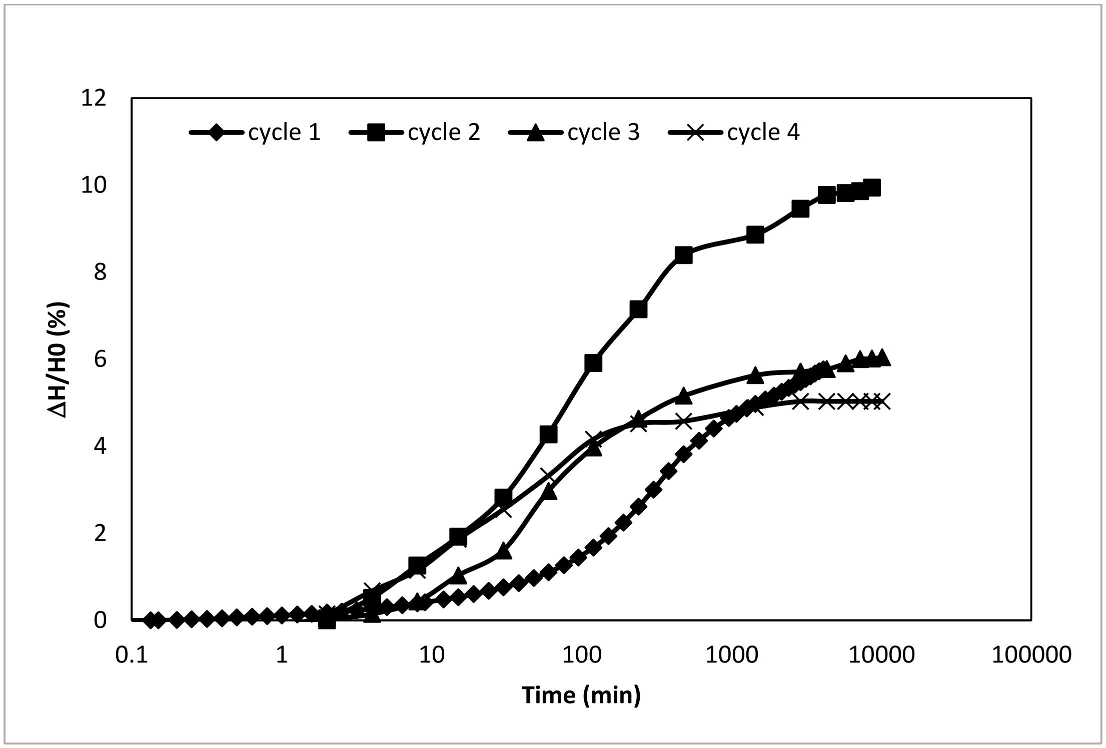


Figure 4.7: Percent swell versus time at each wetting cycle.

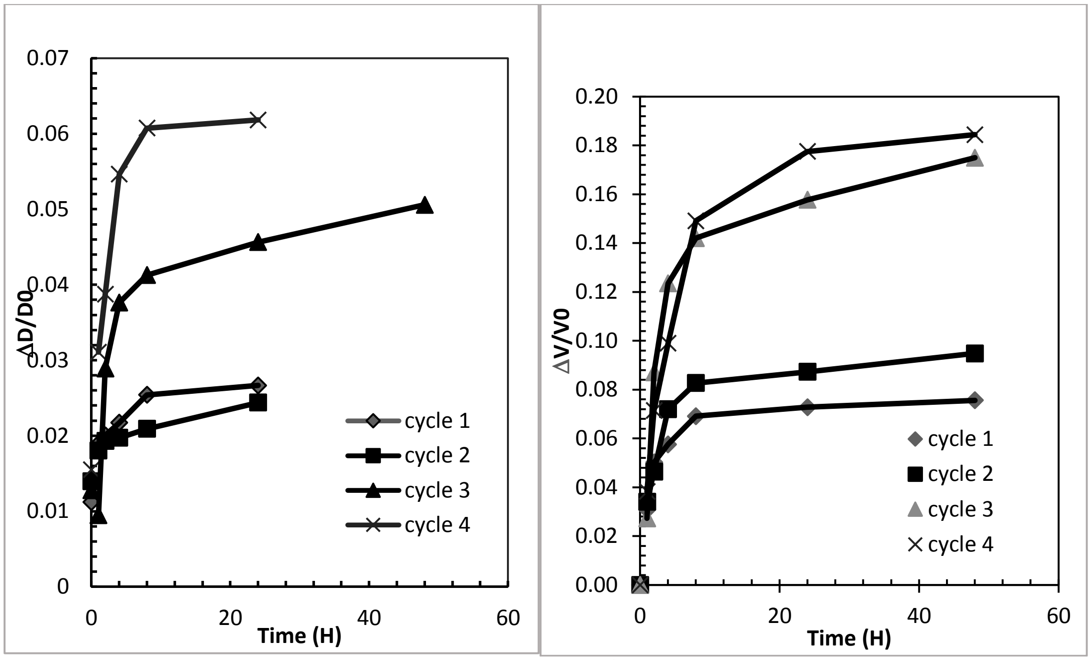


Figure 4.8: Shrinkage- versus time curves in terms of (a) diametric and (b) volumetric strains.

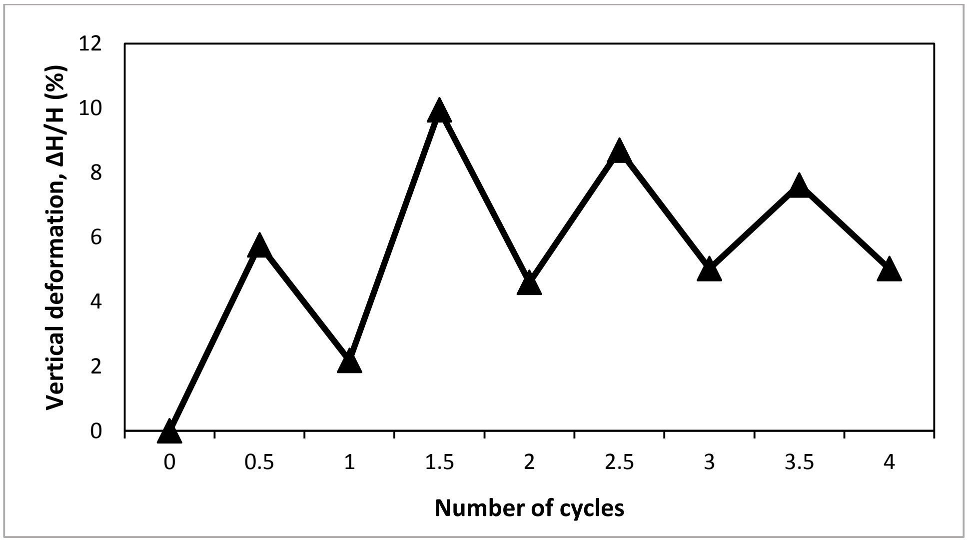


Figure 4.9: Maximum swell and shrinkage deformations versus number of cycles in cyclic wetting-drying process

4.5 Consolidation Test

Consolidation test results are presented as void ratio versus effective consolidation pressure in Figure 4.10. The consolidation test parameters of compression (C_c) and rebound (C_r) indices are presented in Figure 4.11. As can be observed both parameters reduce with increased salt concentrations. These findings are in good agreement with the results of swell tests, indicating similar behavior in rebound (or swell) index, and reduced compressibility due to cementation effect of the salt.1 M $CaCl_2$ solution has been most effective in reducing these indices.

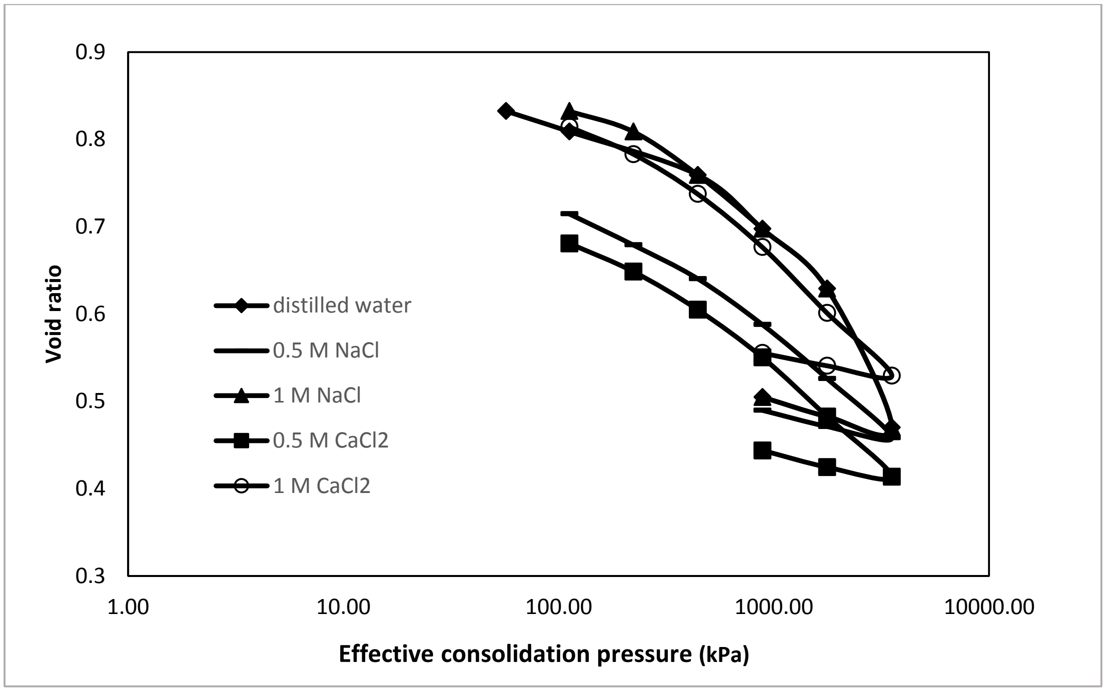


Figure 4.10: Void ratio versus effective consolidation pressure curves.

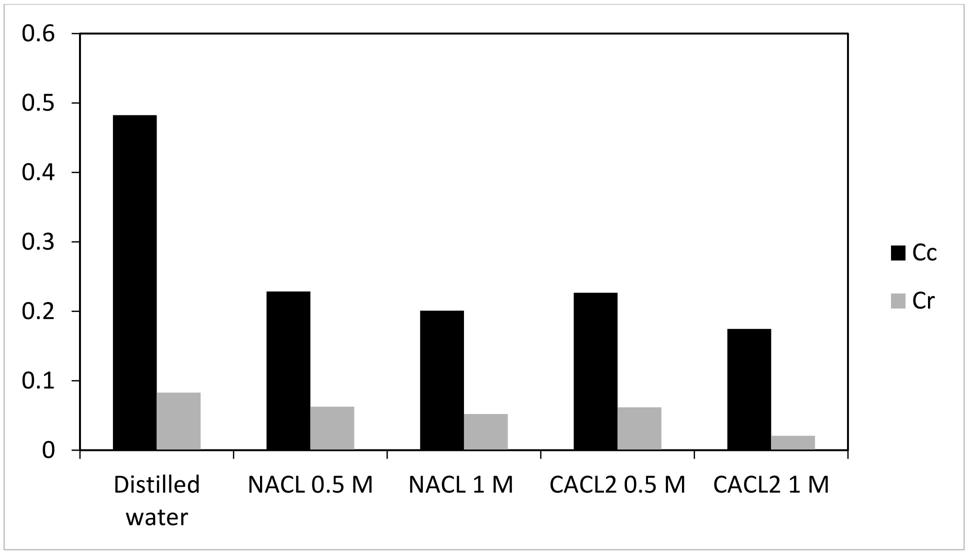


Figure 4.11: Compression and rebound indices versus pore water type.

Consolidation test results also yielded swell pressure and preconsolidation pressure of each specimen. The results in Figure 4.12 indicate reduction in swell pressures with increasing salt concentrations. The lowest swell pressure is obtained for 1 M $CaCl_2$ treated specimen. Preconsolidation pressure however increases with salt concentrations. Both salts and almost all concentrations reveal almost the same preconsolidation pressure, which actually does not have anything to do with the stress history but just represents a measure of cementation of the compacted soils.

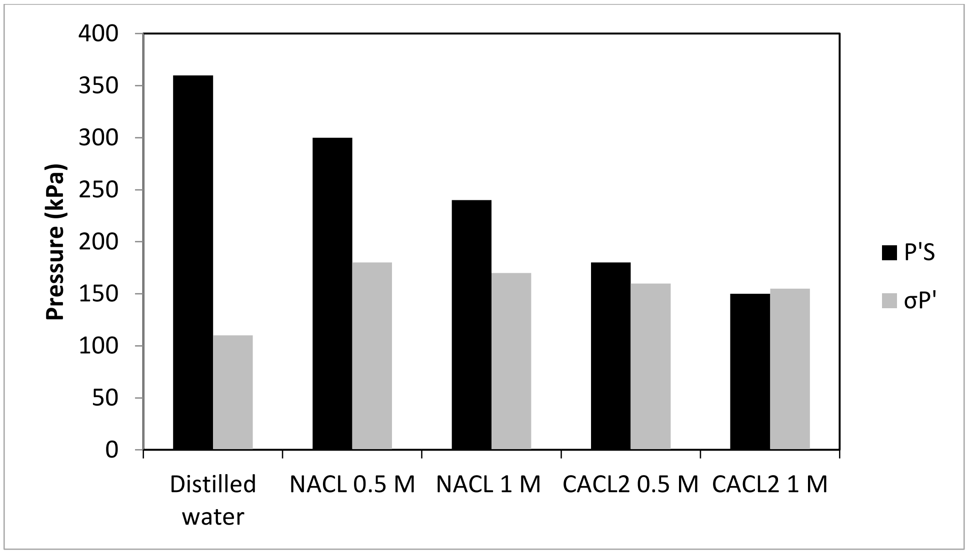


Figure 4.12: Swell pressure and preconsolidation pressure versus pore water type.

4.6 Saturated Hydraulic Conductivity

Another parameters obtained from consolidation test results is hydraulic conductivity, Deformation versus square root of time curves were obtained at each pressure increment, and the coefficient of consolidation (c_v) was obtained using Taylor's square root of time ($\sqrt{t}$) method (Figure 4.14).

The coefficient of volume change (mv) was also calculated using Equation 4.2 from void ratio versus effective consolidation pressure curves. (Figure 4.14)

$$m_v = a_v/(1+e) \tag{4.2}$$

Where, a_v = the coefficient of compressibility and equals to the ratio between the change in the void ratio (δe) to the change in the pressure (δp) within a selected stress range. The hydraulic conductivity (k) was calculated using Equation 4.3 for various pressure increments using the coefficient of consolidation (c_v) and coefficient of volume change (m_v)

$$k = c_v m_v \gamma_w \tag{4.3}$$

Where, γ_w = unit weight of the pore fluid.

Figure 4.15 shows that hydraulic conductivity has a tendency to increase with salt concentrations, as soil particles become more granular with cementation effect of the salts. This effect is not very significant under higher confining pressures.

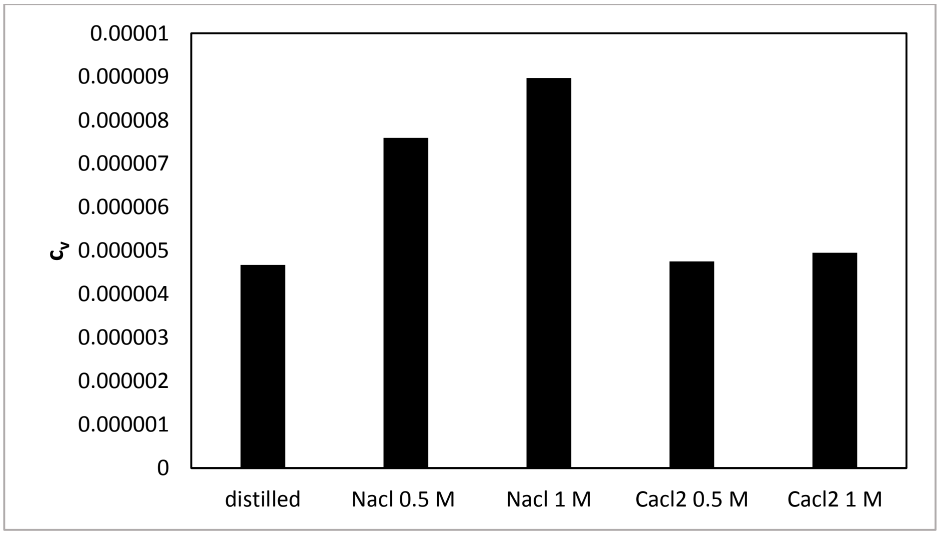


Figure 4.13 Variation of coefficient of volume change with different pore fluids

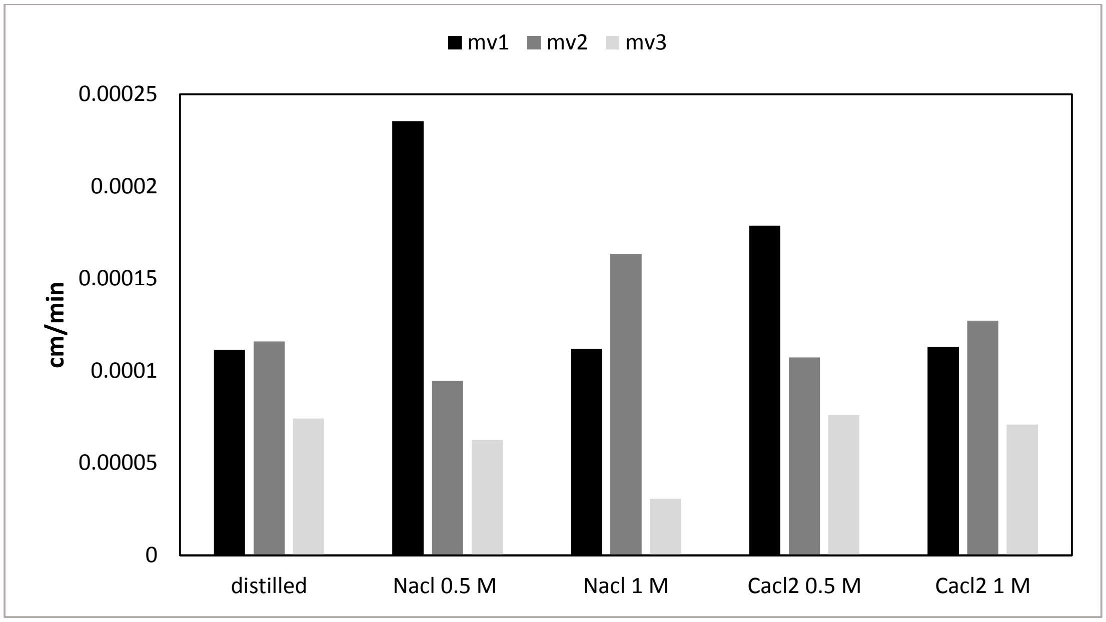


Figure 4.14: Variation of coefficient of consolidation with different pore fluids

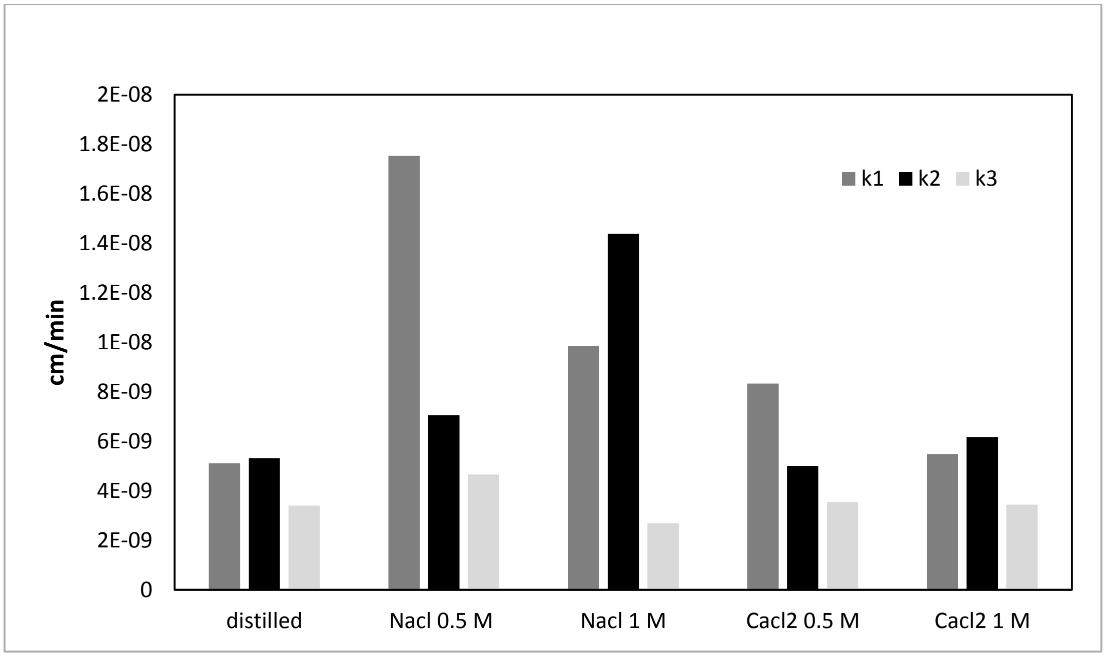


Figure 4.15: hydraulic conductivity of soil

4.7 Soil –water Characteristic Curve

SoilVision program was used to fit models to the laboratory data of water content versus suction results The SWCC models used in this study are van Genuchten (1980) and Fredlund and Xing (1993).

van Genuchten model is given in Equation 4.5.

$$\Theta n = 1/\ [1 + (a_{vg}\Psi)\ n_{vg}]\ m_{vg} \qquad (4.5)$$

Where:

a_{vg} = fitting parameter primarily related to inverse of air-entry value (units equal to 1/kPa)

n_{vg} = fitting parameter primarily related to rate of water extraction from soil once air-entry value has been exceeded

m_{vg} = fitting parameter that are primarily related to residual water content conditions

Fredlund and Xing model is given in Equation 4.6

$$W\ (\Psi) = c\ (\Psi)\ (ws/\ \{\ln\ [e + (\Psi/a_f)\ n_f]\}\ m_f \qquad (4.6)$$

Where:

a_f = fitting parameter which is primarily a function of air- entry value of soil

n_f = fitting parameter which is primarily a function of rate of water extraction from soils once air- entry value has been exceeded

m_f = fitting parameter which is primarily a function of residual water content

$c\,(\Psi)$= correction factor which is primarily a function of suction corresponding to residual water content.

The experimental data and the fitted models are shown in Figures 4.16 and 4.17. The model parameters are as given in Tables 4.1 and 4.2. The Fredlund and Xing air entry values (AEV) are more acceptable as expected to be higher when soil is molded with distilled water and gets lower with the addition of salts. Overall the data obtained is quite erratic mainly due to the difficulty of suction measuring procedures, which are highly influenced by the testing environment and human factor.

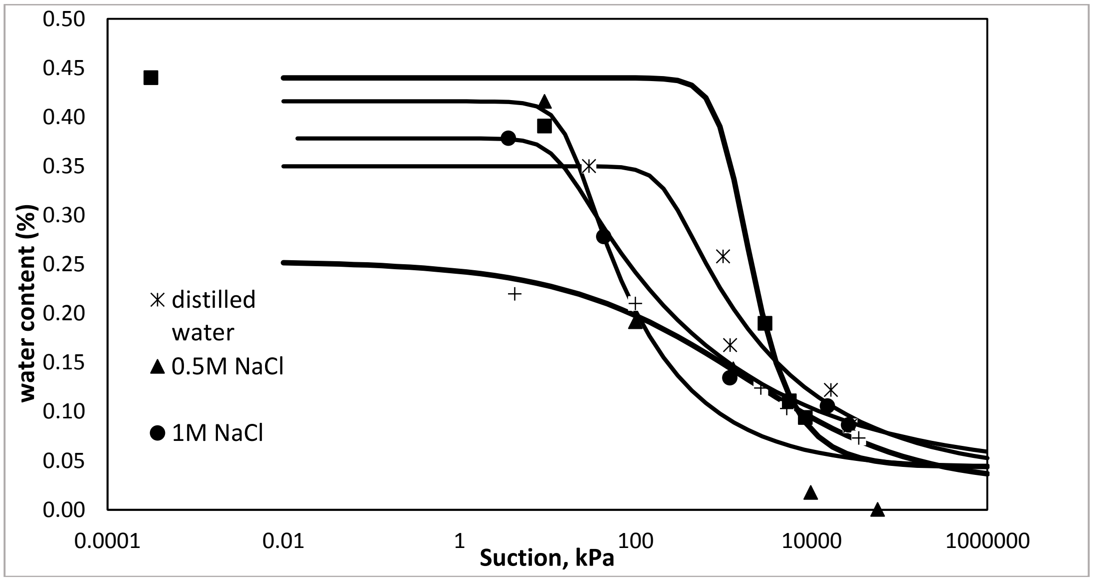


Figure 4.16: SWCC of specimens with different pore water chemistry fitted by van Genuchten

Table 4.1: SWCC fitting parameters of van Genuchten model (1980)

Pore water	a_{vg}	n_{vg}	m_{vg}	Residual WC (%)	R^2	Genuchten AEV
Distilled water	0.096	3	0.081	10	1.00	8.35
0.5 NaCl	0.058	3	0.158	10	0.93	12.51
1M NaCl	0.004	3	0.113	10	0.92	16.99
0.5 M CaCl2	0.072	3	0.132	10	0.98	15.11
1 M CaCl2	0.007	3	0.1	10	0.98	20

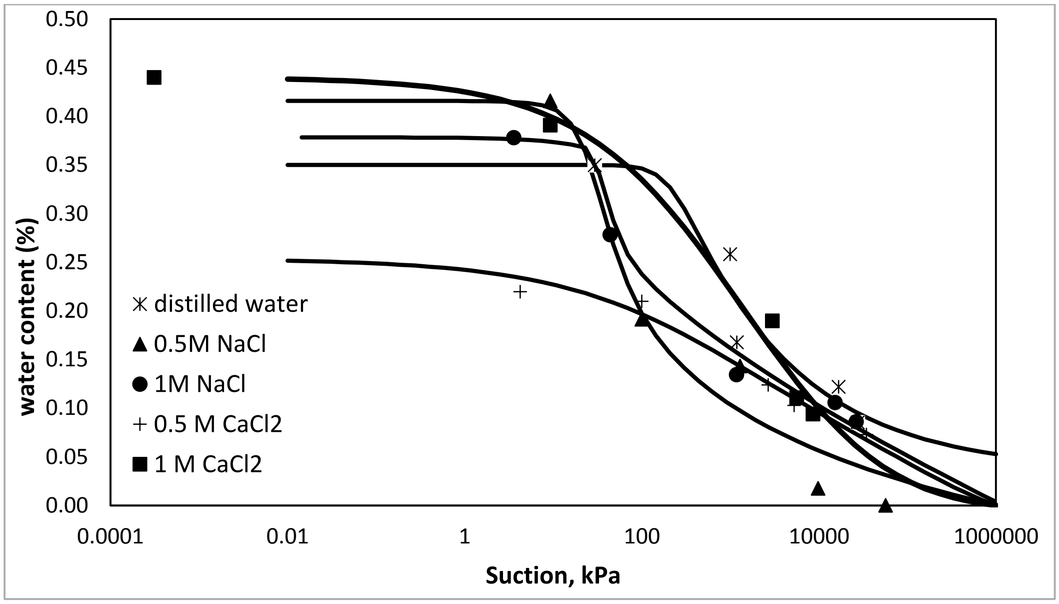


Figure 4.17: SWCC of specimens with different pore water chemistry, fitted by Fredlund and Xing (1994)

Table 4.2 SWCC fitting parameters of Fredlund and Xing model (1994)

sample	af	nf	M_f	R^2	Residual VWC	Fredlund AEV
Distilled water	777.37	20	0.192	0.94	17.2%	568.5
0.5 M NaCl	23.6	2.98	0.467	0.96	12.9%	593
1 M NaCl	27.7	13.9	0.138	0.99	20.6%	20.8
0.5 M CaCl2	551.74	0.405	1.61634	0.97	3.21%	13.16
1 M CaCl2	1257.6	0.469	2.69554	0.99	3.34%	21.38

4.8 Unconfined Compressive Strength

Unconfined compression test is another test which is performed in this study. It is conducted by increasing the concentration of salts which increase the unconfined compressive strength and failure strain is larger than the samples prepared with distilled water. The addition of s salt causes an increase in the ion concentration of the pore water with concomitant reduction in the double layer thickness and, in turn, this causes a reduction in the antiparticles repulsion and an increase in the attraction. Perloff, (1976) pointed out that it results in increasing cohesion. The compaction effort also affects the strength of the cohesive soil. By the increase in the compaction effort, the unconfined compressive strength increases. The results, presented Figure 4.18 indicate that the maximum shear strength found was of the soil treated with calcium chloride. The addition of $CaCl_2$ to the soil cause hardening and more strength as compared to the soil specimens containing other salts additives. It occurs because of very small pozzalonic reaction. The pozzolonic reaction usually occurs in lime stabilization. Salty water contains very little amount of Ca2+ which is enough for causing the pozzolonic reaction. In $CaCl_2$ treated specimens it is observed that stiffness increases and the failure strain reduces making them more brittle.

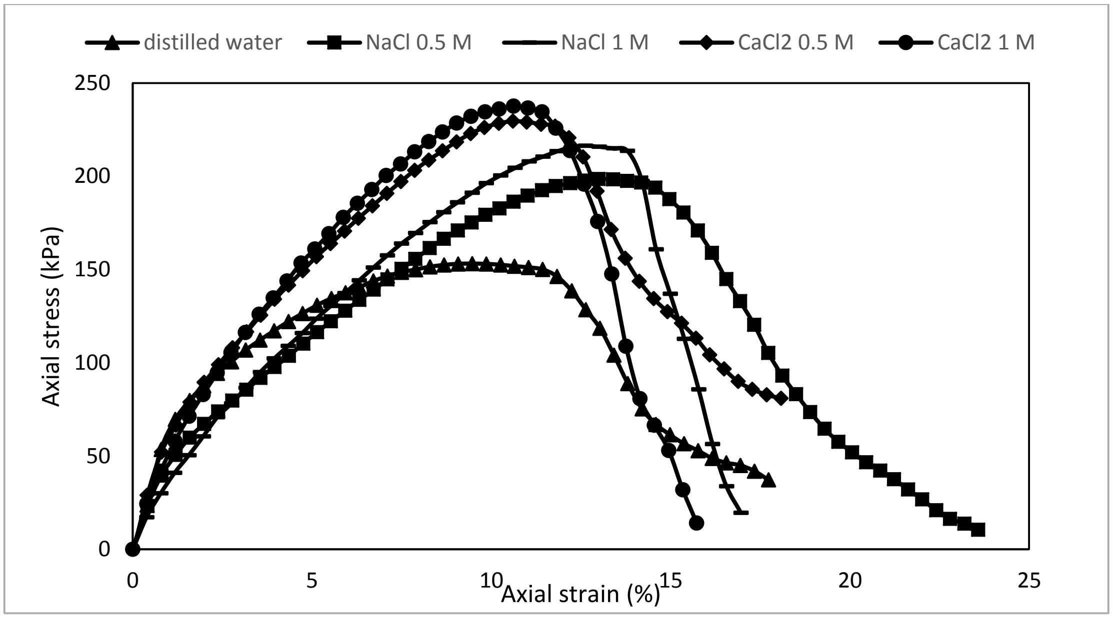


Figure 4.18: Axial stress versus axial strain

4.9 Tensile Strength

Tensile strength is another test conducted in this research work on the soil with different of salt concentrations, and results show that salt solutions can increase strength of soil, particularly NaCl being more effect.

Barzegar et al. (1995) explained that soil strength is influenced by several factors. These factors associated with the soil clay fraction such as exchangeable cations, clay content, clay type, and the amount of dispersible clay.

Clay particles are engaged in binding or cementing soil particles but clay particles may not be extensively involved in binding or cementing other particles are flocculated and aggregated by calcium ions and organic matter, when they are flocculated and aggregated by calcium ions and organic matter. Therefore, total clay may not reflect the soil strength.

In addition to its influence on dispersion, as Barzegar et al. (1995) claimed, clay mineralogy may also affect the nature of individual contact points and the strength of bonding between soil particles.

During the various stages of decomposition, the amount and type of organic matter were considered the most important factor in the stability of soil structure. As Emerson (1984) pointed out, the complex soil system makes it very difficult to explain the mechanism involved in the soil structure stability. The dual role of organic matter can be explained in two processes which include:

1. Increasing the repulsive forces between particles and increase in negative charges of pure clay which intensify the colloidal condition of soil particles, and increase the clay dispersion.

2. The formation of bonds with adsorbed cations, increases the physical resistance of soil aggregates against dispersion, and therefore increases the aggregate stability.

Important factors about the physical and mechanical properties of soils are the amount of soil moisture and wetting/drying cycles. However, its quantitative effect depends on experimental conditions. Note that, experimental conditions are not assessed here.

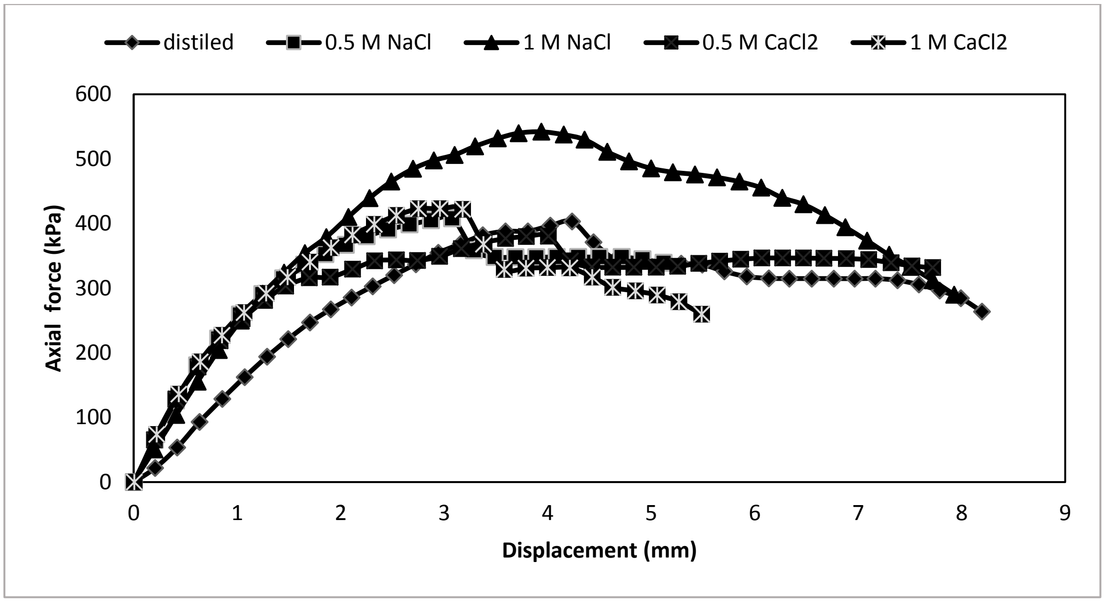


Figure 4.19: Axial force versus displacement

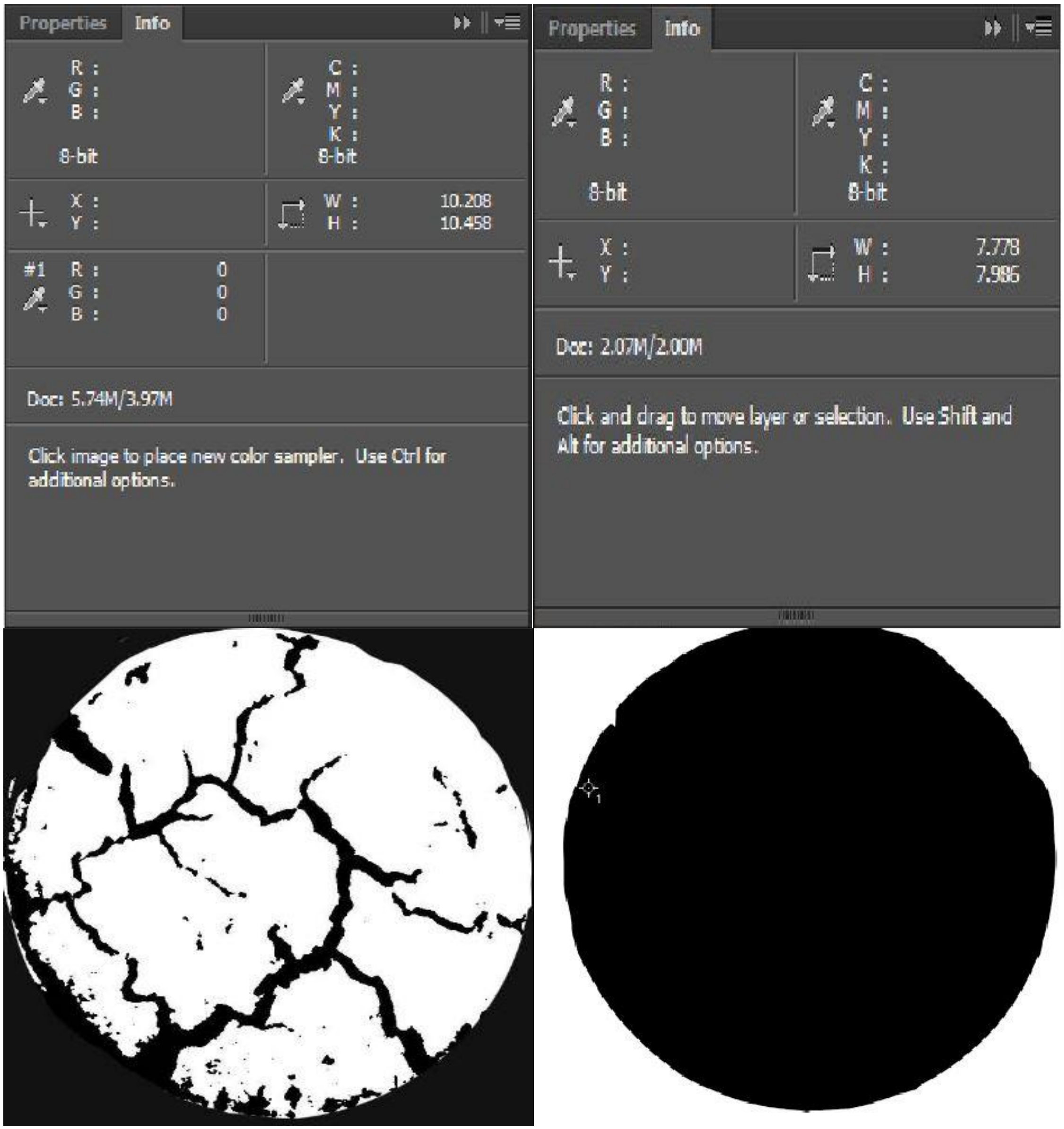
Maximum tensile strength values are as shown in Table 4.3 indicating that 1 M NaCl has the highest value. These values are not very compatible with the unconfined compressive strength test results, as it was earlier mentioned that $CaCl_2$ became more brittle, which does not reveal similar results in Table 4.3

Table 4.3: Maximum tensile strength

Pore water	**Tensile strength (kPa)**
Distilled water	2.33
0.5 M NaCl	2.94
1 M NaCl	3.13
0.5 M CaCl2	2.204
1 M CaCl2	2.44

4.10 Crack Intensity Factor

In Chapter 3 CIF determination was explained. Figure 4.20 shows the Photoshop software data on pixels of the dark and white areas. The dark areas on the cracked surface represent the cracks. By counting the pixels in the dark area and dividing it to the total pixel number of the whole intact specimen surface, CIF is calculated to be 35.7% when distilled water was used and 28.7% when 0.5 M NaCl solution was used. Therefore, presence of salts in the pore water acts like cementing agents, reducing the shrinkage and crack formation potential of expansive soils.



W : 10.208
H : 10.458

#1 R : 0
G : 0
B : 0

Doc: 5.74M/3.97M

Click image to place new color sampler. Use Ctrl for additional options.

W : 7.778
H : 7.986

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Click and drag to move layer or selection. Use Shift and Alt for additional options.

Figure 4.20: Pixels of the area a) white pixels of cracked surface b) entire pixels of sample

Chapter 5

CONCLUSIONS

This thesis investigated the effect of different pore solutions on the hydro-mechanical behavior of soil systems at the laboratory scale. The results revealed that the solution concentration (salinity) directly influences the engineering parameters including physical properties, swelling-shrinking, unconfined compressive strength, tensile strength, hydraulic properties. The volume change parameters have reduced appreciably while the strength properties and the hydraulic conductivity have increased by incrementing the NaCl and $CaCl_2$ concentrations of the pore fluid. The conclusions derived from this study can be listed as follows:

1. Addition of salt solutions to the expansive soil reduced the plasticity index significantly, thus reducing both swelling and shrinking potentials of expansive soil. This is mainly due to the effect of salts on mineralogy of expansive clay and suppression of diffuse double layer as explained in literature review and in the discussions of the experimental findings.

2. Based on consolidation test results, addition of salts indicated reduction in compression and rebound indices. Moreover, the swell pressures are also reduced which has substantiated the findings of swell-shrinkage testing. The observed increment in preconsolidation pressure, however, is a good indication of cementation between the particles.

3. Hydraulic conductivity showed a tendency to increase under low confinement, and remained almost the same under higher effective stress levels. This is also due to cementation and aggregation of soil particles, becoming more granular, hence porous.

4. Increase in unconfined compressive and tensile strengths is also in good agreement with the reduced compressibility and increased preconsolidation pressures. The failure strain however increases in NaCl added specimens, thus becoming more ductile, while $CaCl_2$ is observed to make specimens more brittle.

5. Salts are observed to cause reduction in crack formation on the surface of specimens during desiccation, thus the crack intensity factor is higher when distilled water is used.

5.1 Recommendation for Further Research

While this work indicated that soils including a high quantity of salts may need longer desiccation periods to crack and may even be capable of staying un-cracked (as compared to non-saline soils), when drying periods are short, the complexity of the soil behavior at the field-scale, environmental and structural variations and other processes will affect the overall shrinkage and cracking behavior. Therefore, further research is recommended to do a more expanded and detailed research on the impact of salinity on soil structure and behavior. This study can be extended to field-scale, especially in connection to cracking which is still a topic of high interest in unsaturated soils, mainly in the area of nuclear/ sanitary waste containment design.

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