PREDICTION OF SWELLING BEHAVIOR OF EXPANSIVE SOILS USING MODIFIED FREE SWELL INDEX, METHYLENE BLUE AND SWELL OEDOMETER TESTS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

AMIR JALEH FOROUZAN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

FEBRUARY 2016

Approval of the thesis:

PREDICTION OF SWELLING BEHAVIOR OF EXPANSIVE SOILS USING MODIFIED FREE SWELL INDEX, METHYLENE BLUE AND SWELL OEDOMETER TESTS

Submitted by **AMIR JALEH FOROUZAN** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Gülbin Dural Ünver
Dean, Graduate School of **Natural and Applied Sciences** ____________

Prof. Dr. İsmail Özgür Yaman
Head of Department, **Civil Engineering** ____________

Prof. Dr. Erdal Çokça
Supervisor, **Civil Engineering Dept., METU** ____________

Examining Committee Members

Asst. Prof. Dr. Nabi Kartal Toker ____________________
Civil Engineering Dept., METU

Prof. Dr. Erdal Çokça ____________________
Civil Engineering Dept., METU

Asst. Prof. Dr. Nejan Huvaj Sarıhan ____________________
Civil Engineering Dept., METU

Asst. Prof. Dr. Onur Pekcan ____________________
Civil Engineering Dept., METU

Associate Prof. Dr. Cem Akgüner ____________________
Civil Engineering Dept., TED University

Date: 02 /02/2016

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name: AMIR JALEH FOROUZAN

Signature :

ABSTRACT

PREDICTION OF SWELLING BEHAVIOR OF EXPANSIVE SOILS USING MODIFIED FREE SWELL INDEX, METHYLENE BLUE AND SWELL OEDOMETER TESTS

Jaleh Forouzan, Amir
M.S. Department of Civil Engineering
Supervisor : Prof. Dr. Erdal Çokca

February 2016,135 pages

Expansive soils are recognized as problematic soils that impose several challenges for civil engineers. Such soils undergo significant volume change in case water penetrates into them, and they shrink as they lose moisture. Lightly-loaded engineering structures such as pavements, single story buildings, railways and walkways may experience severe damages when they are founded on such soils. Determination of expansive soils and quantifying their swelling potential and pressure caused by their expansion are essential in geotechnical engineering. Therefore it is necessary to develop models to predict swelling pressure and swelling potential of expansive soils.

This research presents an experimental investigation of swelling behavior (swelling pressure and swelling potential) of expansive soils. The expansive soil specimens were prepared in the laboratory by mixing kaolinite and bentonite at different percentages. Atterberg limits, Grain size distribution, G_s, Optimum water content, Maximum water content , Swelling pressure, Methylene blue value (MBV), Modified Free swell index (MFSI) and swell potential of the mixtures were assessed. The correlations between the swelling behavior of test samples and fundamental properties of test samples were studied. Additionally, the correlations between swelling behavior and MBV, MFSI and some of the index properties of test samples were investigated. These tests were repeated on the natural expansive soil samples and the results were evaluated.

As final conclusion of this research, the values of swell pressure and swell potential of the test samples from the experimental investigation are compared with the predictive values of the same based on currently proposed and other suggested models.

Key Words: Expansive Soil, Swelling Potential, Methylene Blue Test, Modified Free Swell Index Test, Swelling Pressure Test, Swell Percent Test

ÖZ

MODİFİYE SERBEST ŞİŞME ENDEKSİ, METİLEN MAVİSİ VE ÖDOMETRE ŞİŞME TESTLERİNİ KULLANARAK ŞİŞEN ZEMİNİN ŞİŞME DAVRANIŞININ TAHMINI

Jaleh Forouzan, Amir
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. Erdal Çokca

Şubat 2016,135 sayfa

Şişen zeminler problemli zeminler olarak kabul edilip inşaat mühendisleri için çeşitli sorunlar yaratmaktadır. Bu tür zeminler suya maruz bırakıldıklarında, önemli hacim değişikliğe uğrayıp, kurutulduklarında büzüşürler. Geçmişteki tecrübelere göre kaldırımlar, tek katlı binalar, demiryolları ve yürüyüş yolları gibi hafif yüklü mühendislik yapıları böyle zeminler üzerinde kurulduğunda ciddi zararlar görmüşlerdir. Bu yüzden şişen zeminlerin belirlenmesi, onların şişme potansiyeli ve şişme basıncın hesaplanması, jeoteknik mühendisliğinde esastır. Şişen zeminlerin şişme basıncı ve şişme potansiyelini tahmin etmek amacıyla model geliştirmek gerekmektedir.

Bu araştırmada şişen zeminlerin şişme davranışı (şişme potansiyeli ve şişme basıncı) deneysel olarak incelenmiştir. Bu çalışma kapsamında şişen zeminler hazırlanması, laboratuvar ortamında, farklı oranlarda kaolin ve bentonit karıştırarak gerçekleştirilmiştir. Tane boyu dağılımı, Kıvam limitleri, G_s, maksimum kuru yoğunluk, optimum su içeriği, şişme basıncı, metilen mavisi değeri, modifiye serbest şişme değeri ve karışımların şişme potansiyeli belirlenmiştir. Test örneklerinin şişme

davranışı ve temel özelliklerinin arasındaki ilişki incelenmiştir. Ayrıca, test örneklerinin şişme davranışı, MBV, MFSI ve endeks özelliklerinden bazıları arasındaki ilişkiler incelenmiştir. Bu testler, bozulmamış doğal şişen zemin örnekleri üzerinde tekrarlanmış ve sonuçlar değerlendirilmiştir.

Bu çalışmanın sonucu olarak, test örneklerinin şişme potansiyeli ve şişme basıncı değerleri, aynı bazda olan, yeni önerilen ve daha önce önerilmiş olan diğer modellerin değerleri ile karşılaştırılmıştır.

Anahtar Kelimeler: Şişen zemin, Şişme potansiyeli, Metilen mavisi deneyi, Modifiye serbest şişme indisi deneyi, şişme basıncı deneyi, şişme yüzdesi deneyi

ACKNOWLEDGMENTS

I would like to express my grateful thanks to my academic supervisor, Prof. Dr. Erdal Çokça. His concern and educational support were invaluable throughout my studies and in the completion of this thesis at the Middle East Technical University. I also wish to thank Prof. Dr M. Yener Özkan for first giving me the opportunity to study at this university.

Thanks are also to all soil mechanics laboratory staff and all people for all the help they gave during my experiments. In particular, I would like to express my sincer thanks to Ali Bal, Mr. Ulaş Nacar, Mr. Kamber Bilgen, for their valuable support and friendship. Also, I am thankful to Mr. İlyas Özkan for his suggestions and friendly approach throughout the laboratory works.

I thank to Civil Engineering Department of Middle East Technical University for supporting me with teaching and technical facility. I would also like to give heartful respect to Dr. Noasrat Taheri for his endless help.

I am also thankful to Farzin Golzar and Farhad Hanifepoursardroud for their friendship, advice and continuous support and encouragement, which created such a motivating environment for research and enjoyable life.

In addition I would like to express my gratitude to Ataollah Khanlari, Farnoud Khakzad, Nasrin Mehmandoost and Yosef Badali for their help, toleration and encouragements.

Also, I would like to give credit to my colleagues Behzad sedghi Saray and Ali Zarifi at URUM SPOTA Company for their friendship and motivation. I am grateful for their help and being my friends for almost decade.

Last but not least, I express my sincere thanks to my father Rahim Jaleh Forouzan, to my mother Hamideh Parnia and my brother Saeid Jaleh Forouzan for their boundless inspiration, encouragement, sacrifice and blessings. This thesis is dedicated to them without their support, love, patience and belief in me I would never have accomplished this.

TABLE OF CONTENTS

APPENDICES

LIST OF TABLES

TABLES

LIST OF FIGURES

FIGURES

LIST OF ABBREVIATIONS

ASTM: American Society for Testing and Materials

ANFOR: Association French Normalization Organization Regulation

A_c: Activity

C_c: Clay Content

CST: Constant Swell Test

CEC: Cation exchange capacity

CVT: Constant Volume Test

DDL: Diffuse double layer

DOT: Double Oedometer Test

DGR: Deep Geological Repository

FS: Free Swell

FSI: Free Swell Index

FSM: Free Swell Method

FST: Free Swell Test

G_s: Specific Gravity

Hi = Initial height of the sample

Hf = Final height of the sample

LBT: Load-Back Test

LI: Liquidity Index

LL: Liquid limit

M_0: Dry Soil Mass

MBT: Methylene Blue Test

MBV: Methylene Blue Value

METU: Middle East Technical University

MFSI: Modified Free Swell Index

N: Number of Blows

P: Swelling Pressure

PI: Plasticity index

PL: Plastic limit

R^2: R square

RST: Restrained Swell Method

S: Standard Error

S_p: Swell Potential

SEM: Scanning Electron Microscope

SI: Shrinkage Index

SL: Shrinkage Limit

SSA: Specific Surface Area

TOT: Tetrahedron-Octahedron-Tetrahedron

TS: Turkish Standard

USC: Unsaturated Swelling Clay

V: Sediment volume of 10 gr of oven dried soil passing sieve NO.40 placed a 100 ml graduated measuring jar containing distilled water

V_0: Volume of dry soil

V_k: Sediment volume of 10 gr of oven dried soil passing sieve NO.40 placed a 100 ml graduated measuring jar containing kerosene.

USCS: Unified Soil Classification System

W_1 = Empty Mass of Pycnometer

W_2 = Mass of Pycnometer + Oven Dry Soil

W_3 = Mass of Pycnometer + Oven Dry Soil + Water

W_4 = Mass of Pycnometer + Water full

w_n: Moisture Content of Soil

w_i: Initial Water Content:

ZST: Zero Swell Test

$\Delta H = (H_f - H_i)$

γ_d: Maximum Dry Density

ρ_w: Density of Water

CHAPTER 1

INTRODUCTION

1.1 Background

Expansive soils, known as swelling soils or reactive soils, composed predominantly of high percentage of fine-grained clay particles. Also high plastic clays are defined as fine-grained clays with a plasticity index greater than 35% (Holtz and Kovacs, 1981). This soil type is prone to remarkable volumetric changes due to changes in moisture content. The main causes of volume change behavior are the increase and decrease in soil moisture content which results in swelling and shrinkage phenomena respectively. Other factors affecting soil volume change behavior include soil structure, particle interactions (mineralogy), stress history and specific surface (Scott, 1963; Chen, 1975; Nelson and Miller, 1992; Pusch and Yong, 2006; Murray, 2007).

The swelling soil deposit can be found in many areas especially in semi-arid regions located in the tropical and temperate climate zones worldwide (Chen, 1988).

Figure 1.1 indicates regions of swelling clays in the USA. The region shown with red color demonstrates the areas with high amount of expansive clays with high expansion potential and the region shown with blue color displays the areas of less than 50% expansive clays but have high swelling potential. The areas with deposits of less than 50% swelling clays and with temperate swelling potential are specified with orange color (There are different criteria such as MBV, FSI and swell potential to classify swelling soils).

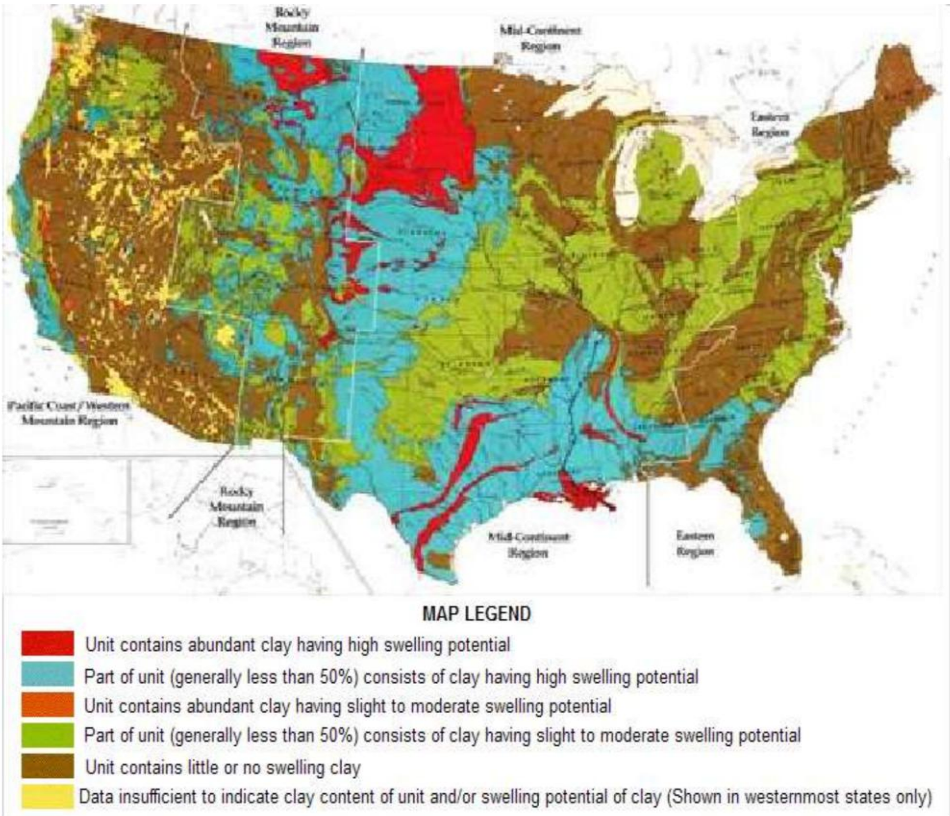

Figure 1.1 Extent of clay mineral deposits in United States (Olive, 1989)

Due to swelling and shrinking behavior of highly plastic clays (expansive soils) considerable damages to physical infrastructures are reported (Jones and Holtz, 1973). Unlike natural clays with low plasticity, when swelling occurs in high plastic clays it exerts tremendous amount of pressure often causing distress to substructures, such as light weight structures (Fig 1.2), shallow foundations, pavements (Fig 1.3), embankments, and dams.

The mitigation of the effects of expansive soil on engineering structures becomes quite a challenge to the designer of substructures upon this type of soil. Therefore the swelling and shrinking characteristics of the soil on which engineering structures are designed must be considered. The annual damage caused by expansive soils costs about $1 billion in the USA, ₤150 million in the United Kingdom and billions of pounds all over the world (Das, 2009). Also it was shown by Das (2009) that much of the damage caused by swelling clays is not because of the lack of proper engineering solutions but to the failure to recognize the presence of swelling soils and quantifying their potential expansivity in geotechnical site investigation.

“In Turkey, presence of swelling soils in many regions such as West Anatolia region, some parts of the Central Anatolia region, Southeast Anatolia region and Eastern Anatolia are reported” (Çokça, 1991). Ankara, the capital of Turkey and located in semiarid region, is famous for its swelling clay. This clay is categorized between soils with medium to highly plastic swelling characteristics because of its swelling mineralogy. Annually several cases of damage to buried utilities such as water pipes, garden walls and small buildings are reported due to the expansion caused by Ankara clay. Characterization of the swelling behavior of Ankara clay is a study of considerable importance in the southwestern regions of Ankara (Erguler and Ulusay, 2003).

Despite disadvantages of swelling soil, there is also a positive side of expansive soil. The high sensitivity of expansive clay due to moisture content changes provides a self-sealing ability (through swelling) and a low hydraulic conductivity which is beneficial for clay-based sealing materials such as geosynthetic clay liners (Koerner, 1998). Also the usefulness of swelling soil is attributed as a protective barrier material to surround nuclear fuel waste containers in the deep geological repository (DGR) concept (AECL, 1994; Graham et al, 1997).

Figure 1.2 Damage on a masonry wall due to the shrinkage in Soil (www.basementsystems.com, 2016)

Figure 1.3 A view of road undergoing swelling

As it was stated before, determination of swelling soils and evaluation of their swelling potential is necessary in geotechnical land use and project planning. Hence, geotechnical engineers encounter the challenge of characterizing unsaturated swelling clays (USC) behavior. There are field and laboratory methods to determine swelling soil and classify their behavior such as strength, permeability, swelling pressure, shrinkage and swelling potential. Various methods and interpretations are utilized by geotechnical engineers to determine and classify expansive soils. Expansive soils can be classified according to swelling degree of non-expansive to highly-expansive soils according to their physical properties, chemical composition and mineral content. For instance, Chen (1975) evaluated swelling behavior of soil in terms of the probability of volume changes for expansive soil (Chen, 1975) and Pusch and Yong (2006) mentioned soils swelling phenomena in terms of the thickness of interlayer hydrates (Pusch and Yong, 2006). Recently two general approaches are developed to consider studies on the swell potential of soil: the Macro-scale and Micro-scale approaches. Traditionally, macro-scale tests include direct and indirect swelling potential measurements. A number of common direct approaches are used in different forms. The most common techniques used to assess soils swelling parameters are Free Swell (FS) test, the Load-Back (LB) test and the Constant Volume (CV) test. In general, Swelling properties of soils are measured by use of an odometer type device. Also modified triaxial experiments are performed to assess expansive soil properties. Micro-scale tests, such as Methylene blue test, include methods used to determine mineralogy of soil samples.

In spite of the variety of swell pressure measurement tests, one dimensional consolidometer technique is the most applicable and practical technique for geotechnical designers to study swelling pressure and swelling potential of highly-plastic clays (Attom and Barakat, 2000). The Free Swell Index test, referred as Free Swell or Differential Free Swell, is another method of estimating swelling pressure (Holtz and Gibbs, 1956). Methylene Blue Test is another approach which is used to detect soil properties consisting swell index, cation exchange capacity (CEC), specific surface area (SSA) and swell potential. For example, Methylene blue test was used by Taylor and Çokça to study the soil cation exchange capacity (Taylor, 1985; Çokça and Birand, 1993b). Also swell potential of soil by use of Methylene blue was studied by Çokça (Çokça, 1991, 2001; Çokça and Birand, 1993a). Preliminary site investigation can be evaluated by development of correlation between index properties and Methylene Blue Value (MBV). On the other hand, the indirect approaches make use of index properties and other variables of the soil in correlations which submits expansion behavior of such soils. Generally macro-scale methods are more common than micro-scale techniques.

Nowadays, it is clear that there is a need to explore a relation between most common experimental methods used to investigate the soil swelling potential and soil fundamental properties to present a correlation between them.

1.2 Research Hypotheses

The main goal of this study is to develop a correlation between the swelling behaviors of test samples, swell potential and swelling pressure, and fundamental properties of test samples. Also, the relations between swelling behavior and MBV, MFSI and some of the index properties of test samples are studied.

1.3 Research Scope

The overall focus of the present thesis is to develop a correlation between common tests performed to assess the swelling behavior of expansive soil, MBT, Modified Free Swell Index Test, Free Swelling Test, Swelling Pressure Test and some of the index properties of swelling soils. Also the focus of this study is towards the development of correlation between the index properties of test samples and their swelling behavior, swell potential and swelling pressure.

Steps taken to achieve this overall objective include:

1) To investigate the index properties of samples
2) To investigate samples swelling potential and swelling pressure experimentally by common methods such as Free Swell Test, Modified Free Swell Index Test, Methylene Blue Test and Swelling Pressure Test
3) To investigate the correlation between index properties of test samples and techniques used to assess swelling behavior, swell potential and swelling pressure, of test samples
4) To investigate the correlation between methods utilized to recognize the swelling behavior of test samples

1.4 Outline of Thesis

This dissertation is organized into 6 chapters: Introduction; Literature Review; Experimental Methods and Materials; Test Results and Discussion; Conclusion and Recommendations for future works and References.

Chapter 1: The requirement for the research study, the objectives, the scope of the study and the outline of the thesis are presented in this chapter.

Chapter 2: Chapter 2 presents a literature study of the current knowledge in expansive clays. The literature review provides a fundamental basis for the concepts and work presented in the thesis. It reviews existing works related to this research and is concluded by outlining the uniqueness of this research to justify the significance of this research.

Chapter 3: Proposed experimental methods and materials are provided in details in this chapter. This chapter covers preparation of materials used in this research. Also experimental procedures and modified techniques are explained in this chapter. In addition, equipment used through this study and their physical properties are presented in this part.

Chapter 4: This part includes the experimental results obtained through this study and provides a shed of light on the correlation between index properties and swelling behavior of test samples, swell potential and swelling pressure. Also the correlations between results of the swell determination tests are discussed.

Chapter 5: This part of the thesis presents the summary of the research, conclusions and contribution of the study. The unique contributions provided by this thesis are summarized in this chapter. Additionally this chapter suggests some recommendations for future study.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Clays are raw materials that exist all around the world abundantly. Clays are well-known for their variety of uses and special properties that belong to their minerals and compositions (Murray, 2007). Within clay, mineral structures are arranged in such a way that they react with water and result in soil volumetric swell or shrinkage with adsorption and desorption of water respectively. Expansive clays, also known as swelling clays exist especially in regions with arid and semi-arid climate. They cause problems for light substructures due to their swelling potential and shrinkage behavior happen in wet season and dry season of year respectively (Mishra et al, 2008).

Many countries encounter challenges related to swelling soil in the world. Quarter of the USA is covered with this type of soil and loss of more than nine billion dollars is reported annually (Lin and Cerato, 2012). Although this problem is mentioned by engineers in many countries and several swell determinative tests are performed to explore subsoil swelling behavior, in underdeveloped countries much of expansive soil problems are not recorded. By determination of this problem in these countries number of countries prone to this problem will increase (Chen, 1988).

As it was acknowledged in the introductory part of this research, determination and quantifying of soil swelling potential is required in geotechnical project planning.

Generally, various laboratory techniques are used to survey expansive soil behavior. Also, interpretations used to classify and determine swelling soil behavior based on experimental results are not unique.

Hence, it is required to detect a relation between the most common experimental methods used to investigate the soil swelling behavior and soil fundamental properties to present a correlation between them. The main aim of this thesis is to provide the essential experimental survey and theoretical basis to gain this goal.

This chapter summarizes a brief review of the methodologies, techniques, and observations of some of the previous studies to provide useful knowledge on clay mineralogy, soil structure and swell investigation tests in order to provide comprehensive review of the existing research results.

2.2 Clay Particle and Clay Mineralogy

As Scott (1963) states, the soil mineral types affect the engineering properties and behavior of the soil. Increase in mineral effects on soil behavior is obvious by decrease in soil particle size which results in excess interparticle forces. Influence of smaller soil particles, types of minerals and interparticle forces, it is likely that clay triggers changes in soil behavior and properties. Whereas soil science researchers and agricultural engineers concern about types of clay minerals and clay structures, civil engineers are more focused on water seepage through soil and its effect on soil mechanical behavior. Macroscale and microscale researches simplify the comprehension of clay swelling phenomenon because they clarify fundamental properties of expansive soils. Since it is evident that mineralogy is the basic parameter dominating the size, shape and properties of soil particles, possible ranges of chemical and physical properties of any given soil can be determined by soil mineralogical structure. Also, type of soil mineral has a key role in determination of soil expansion degree. That is why there is a need for mineralogical classification of clays. Commonly defined particle size ranges are shown in Figure 2.1. There is an arbitrary division between soil groups based on their size. In general clay is defined by considering particle size. Constituents of soil particles smaller than specific size, 0.002 mm (2 µm), are known as clay particles in engineering assortment. As mineral term (Mitchell, 2005), clays express special minerals that can be recognized by:

(a) Small particle size

(b) A net negative electrical charge

(c) Plasticity when mixed with water

(d) High weathering resistance

Considering that clay minerals are primarily hydrous aluminum silicates and it is possible to find nonclay soil particles smaller than 2 µm and clay particles coarser than 2 µm, to avoid confusion it is useful to use the terms Clay Size Content and Clay Mineral Content. Nevertheless, the amount of material finer than 2 µm is a common reference to determine the portion of clay mineral in a soil profile. Also, Particle shape is referred as an important parameter to differentiate clay from nonclay minerals. Mostly the clay particles minerals possess platy shape, and in a few cases they take tubular or needle shape; whereas, the nonclays are composed primarily of bulky particles.

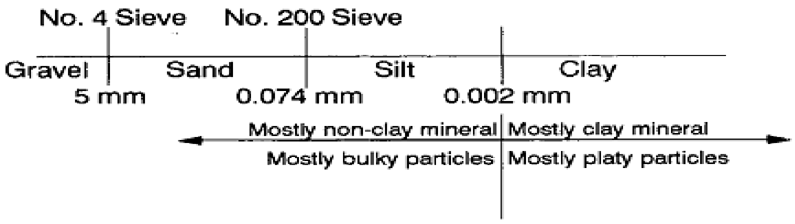

Figure 2.1 Particle size range in soil

The main structural unit of clay minerals are two fundamental crystal sheets, the silica and alumina sheets. Variety of combinations and arrengments of these blocks form various clay minerals. The silica sheet, are combination of tetrahedral units that consist of a single silicon atom and four oxygen atoms enclosing it. On the other hand, a combination of octahedral units possesses six oxygen or hydroxyls surrounding aluminum, magnesium, iron, or other atom forms alumina sheet. Gibbsite material forms when all the anions of octahedral sheet are hydroxyls and aluminum fills two-thirds of the cation positions. Holtz (2011) states and emphasizes that “the mineral called Brucite can be formed when magnesium was substituted for the aluminum in the sheet and it filled all the cation positions” (Holtz et al, 2011).

Figure 2.2 and Figure 2.3 demonstrate a silica tetrahedron, a silica sheet, an octahedron and an octahedron sheet. Also, characteristics of some clay minerals are given in Table 2.1.

Clay minerals are classified into three groups, as follows:

1. Kaolinite Group
2. Illite Group
3. Smectite Group

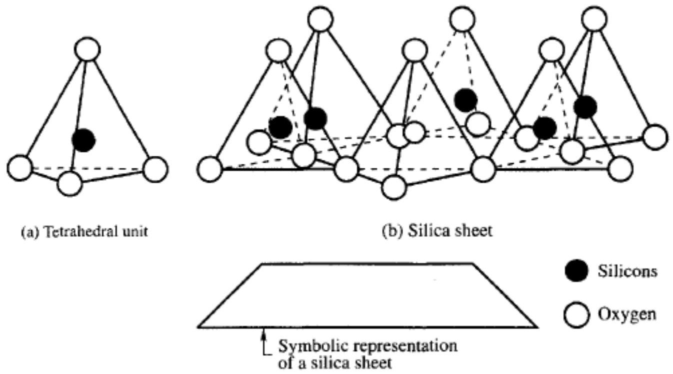

Figure 2.2 Basic structural units in the Silica sheet (Murthy, 2002)

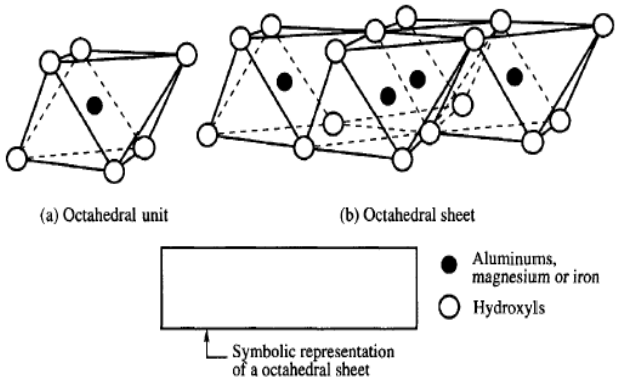

Figure 2.3 Basic structural units in the Octahedral sheet (Murthy, 2002)

Table 2.1 Characteristics of some clay minerals (Nelson and Milner, 1992)

Mineral Group	Basal Spacing (Å)	Particle Features	Interlayer Bonding	Specific Surface (m^2/g)	Atterberg Limits[a] LL (%)	PL (%)	SI (%)	Activity[b] (PI/% Clay)
Kaolinites	14.4	Thick, stiff 6-sided flakes 0.1 to 4 × 0.05 to 2 μm	Strong hydrogen bonds	10–20	30–100	25–40	25–29	0.38
Illites	10	Thin, stacked plates 0.003 to 0.1 × 1.0 to 10 μm	Strong potassium bonds	65–100	60–120	35–60	15–17	0.9
Montmorillonites	9.6	Thin, filmy, flakes >10 Å × 1.0 to 10 μm	Very weak van der Waals bonds	700–840	100–900	50–100	8.5–15	7.2

[a]LL, PL, SL, liquid, plastic, and shrinkage limits, respectively.
[b]From Skempton (1953).
Summarized from Mitchell (1976).

All three of clay mineral groups have layered crystal form. Physical arrangement of different layers and types of the bond between individual structural units are the main source of differences between mineralogy of clay mineral groups. Layers are connected through basic bonds known as hydrogen bonds, potassium bonds and van der Waals bonds.

“The total area of the surface of the grain expressed in square centimeters per gram or per cubic centimeter of the dispersed phase is defined as Specific Surface Area (SSA)”. This parameter increases from kaolinite mineral to montmorillonite mineral. Reactivity with water directly depends on SSA. In geotechnical engineering

Atterberg limits are referred to characterize soil reactivity with water. Also, Liquid limit, plastic limit and shrinkage limit, known as Atterberbeg limits, are utilized by geotechnical engineers to classify clay minerals. According to previous studies highest values of the liquid limit and plastic limit belong to montmorillonite group, on the other hand, this group possesses lowest shrinkage limit in the clay minerals (White, 1949).

The other parameter which is commonly used to classify clay minerals is activity. Plasticity index and percentage of clay particle in soils are referred to determine the activity value of clay minerals. The swelling potential for soil is related to its activity. As activity increases, the swell potential increases.

2.2.1 Kaolinite Group

Kaolinite, soft, earthy and usually white mineral, with the chemical composition $2SiO_2Al_2O_32H_2O$ generated through the chemical weathering from aluminum silicate minerals like feldspar. Kaolin or china clay is a type of rock that is rich in kaolinite (Pohl, 2011).

Some clay minerals consist of repeating layers of two-layer sheets; kaolin is the most important clay of this type. Deer (1992) stated that a" layered silicate mineral, with one tetrahedral sheet which is linked through oxygen atoms to one octahedral sheet of alumina octahedral is known as Kaolin". The repeating layers are hold together through hydrogen bonding and secondary valence forces (Das, 2008) (Fig.2.4, 2.5). There is no or little swelling in the presence of water because of sufficient bonding between layers which results in no interlayer swelling (Mitchell and Soga, 2005). "When kaolinite sheets are stacked on each other, the hydroxyl of octahedron sheets are drawn to the oxygen of the silica tetrahedron sheet by means of oxygen bonds." Cleavage occurs because such ionic and covalent bonds are not strong enough in comparison of the primary bonds. Development of structural sheets in two directions results in crystals of 70 to 100 layers thick (Oweis, 1998). Hydraulic conductivity of 10-6 cm/s or higher is result of low expansion possibility (Oweis & Khera, 1998).

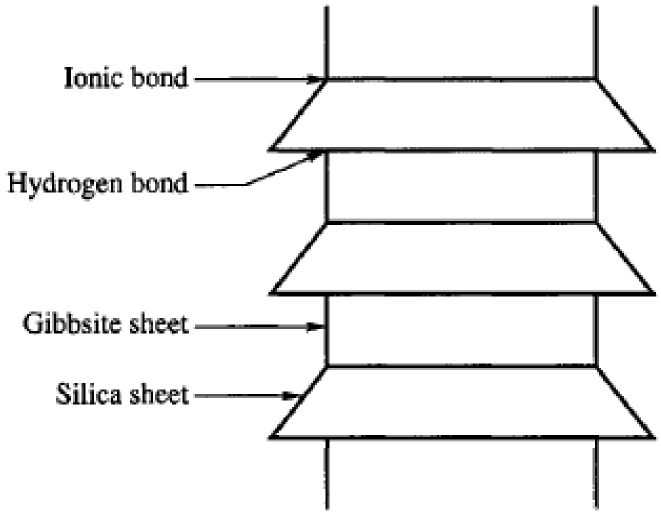

Figure 2.4 Structure of Kaolinite layer (Murthy, 2002)

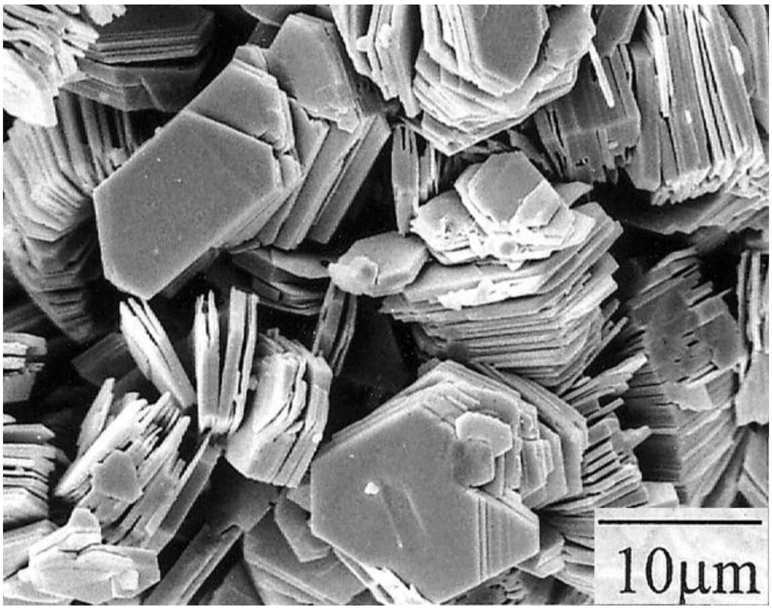

Figure 2.5 SEM of Kaolinite (Source: www.claymin.geoscienceworld.org)

2.2.2 Illite Group

When micas, with the major parent of muscovite, begins to weather it often leads to Illite, which has chemical formula (K, H_3O) (Al, Mg, Fe)$_2$ (Si, Al)$_4$$O_{10}$ [$(OH)_2$, (H_2O)]. Although, its main structural unit is similar to that of montmorillonite, it has less swelling potential than montmorillonite mineral. Main structure of illite is layered alumino-silicate, also known as phyllosilicate. The repetition of tetrahedron – octahedron – tetrahedron (TOT) layers constitutes its structural basis (Fig 2.6.a, 2.6.b). When some of the silica atoms are replaced by aluminum atoms charge deficiency balance occurs by potassium ions, which exist between layers of the unit. The reason for the lower swelling potential of illite is the bonds with the nonexchangeable K^+ ions. In comparison with hydrogen bonds, these bonds are weaker (Murthy 2002). The high stability of illite is responsible for its abundance and persistence in soils and sediments.

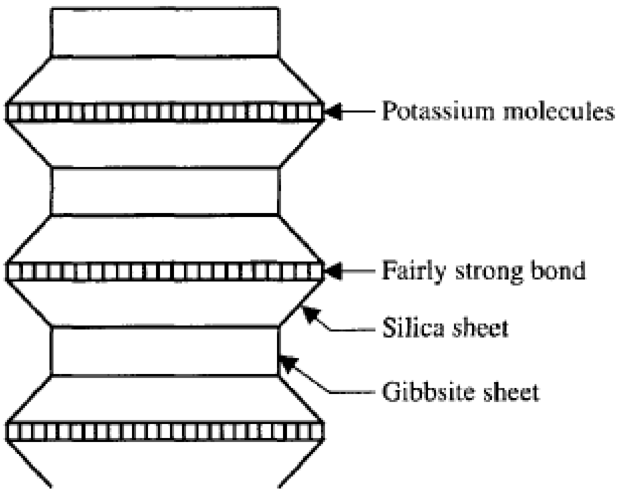

Figure 2.6.a Structure of Illite layer (Murthy, 2002)

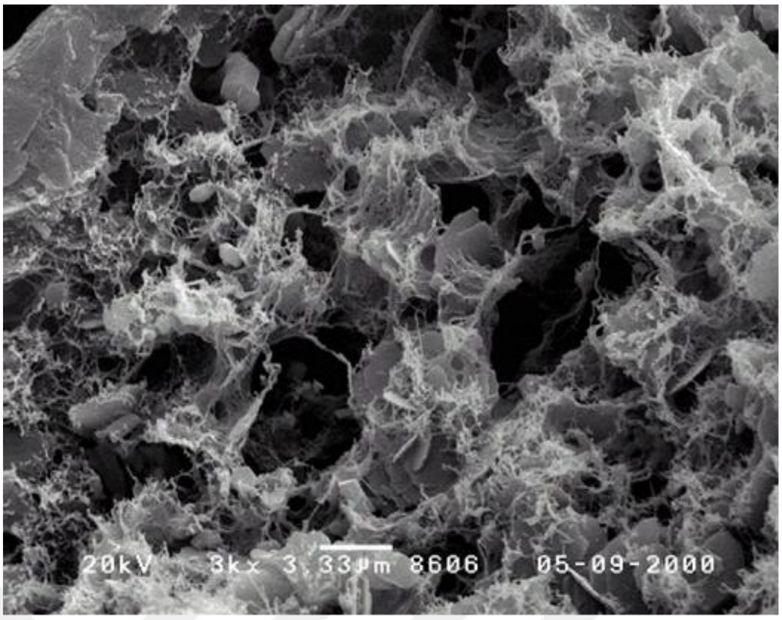

Figure 2.6.b SEM of Illite (www.ssokinc.com)

2.2.3 Montmorillonite Group

When magnesium-rich rocks weather under humid, moderately drained conditions montmorillonite forms which is very soft phyllosilicate group of minerals. As stated above, montmorillonite and illite have same constitutive structure. The main constituent of Bentonite, derived by weathering of volcanic ash, is montmorillonite. Montmorillonite is a 2:1clay, which means 2 tetrahedral sheets sandwiching a central octahedral sheet. Montmorillonite, the most useful member of the smectite group, has plate-shaped particles with an average diameter around 1 μm.

In the central octahedral sheet, magnesium substitutes aluminum partially. “The water molecules and exchangeable cations other than potassium occupy the space between the combined sheets”. There is a weak bond between the connected sheets because of the existent ions (Craig, 2004). Montmorillonite can expand when it comes into contact with water because of the weak bonds which are prone to break when any polar cationic fluids such as water penetrates between structural sheets. The water penetration is easily found out through the layers swelling considerably and bearing much smaller particles with a very large SSA (Oweis and Khera, 1998). The soils with high amount of montmorillonite minerals consists high swelling potential and it exhibits shrinkage characteristic when it is dried out. This member of the smectite group is distinctive for its highest swelling potential, activity and liquid limit in clay soils. In comparison with sodium montmorillonite, montmorillonite including calcium has lower swelling potential and cation exchange capacity. Bentonite, which is a type of montmorillonite, includes both sodium bentonite and calcium bentonite. It is reported by Oweis (1998) that, the amount of sodium bentonite is higher than the amount of calcium bentonite. Bentonite is mainly used for drilling mud, binder, and as a groundwater barrier (Hosterman, J.W. and S.H. Patterson, 1992). Structure of montmorillonite is given in Figure 2.7.

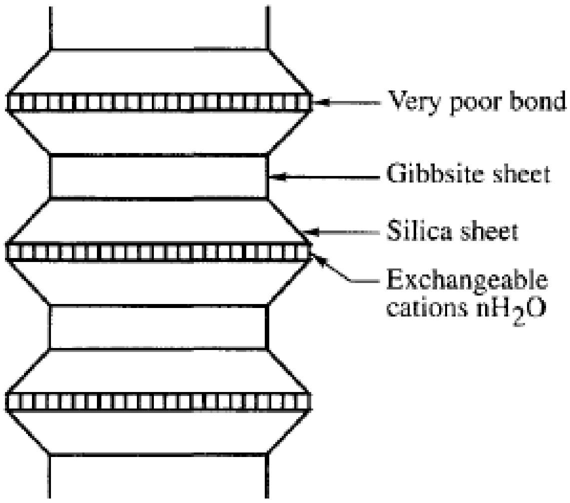

Figure 2.7 Structure of Montmorillonite layer (Murthy, 2002)

2.3 Clay Structure

Soil particles interactions can be influenced by the spacing between the particles and the orientation of the soil particles. Clay elementary structure is divided into two basic structures Dispersed and Flocculated structures (Figure 2.8). Dispersed structure forms when the net particle force is repulsive, on the other hand, when the net particle force is attractive the floccuated structure forms. Flocculated clays are prone to swell more than dispersed clays because of the spacing between the particles which are larger in the flocculated structure than in dispersed structure.

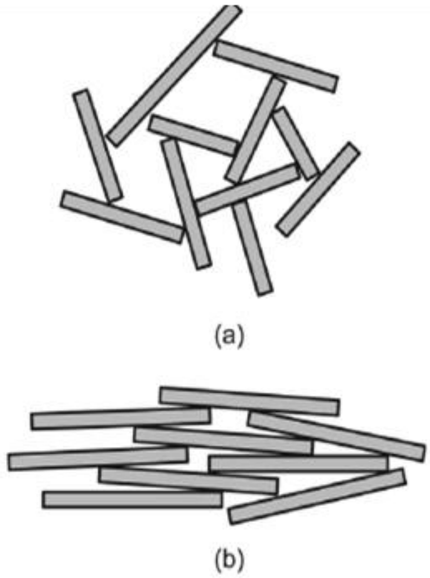

Figure 2.8 (a) Flocculated structure (b) Dispersed structure (Lambe and Withman, 1969)

2.4 Diffuse Double Layer

Swelling occurs in soils with clay minerals, which are prone to influence of their chemical structure by moisture (Carter and Bentley, 1991).

Surfaces of clay particles, which are negatively charged, attract the existent cations in the pore water electrostatically. Simultaneously, cations tend to diffuse back to the pore fluid where there is smaller concentration (Van Olphen, 1963) (Figs 2.9). The water being held by this high concentration of cations, as it is the water not cations that add volume during swelling. The spatial ionic distribution in the liquid surrounding the charged surface caused by two opposite trends is called Diffuse Double Layer (DDL).

The main factors which affect the thickness of diffuse double layer are valence and concentration of cations. Smaller thickness of the double layer can be caused by cations with higher ionic valence. On the other hand, bigger thickness of DDL can be caused by the cations that has lower valence. For instance, in comparison to smectite with Ca^{2+}, smectite with Na^{+} has higher expansion potential.

Mitchell (2005) stated that, increase in DDL and swelling can be caused due to lower concentration of cations. Cations which are highly concentrated near the surface of clay particle form the repulsive force between DDL systems. Temperature is the other parameter that influences the thickness of DDL. Increase in temperature increases the thickness of DDL.

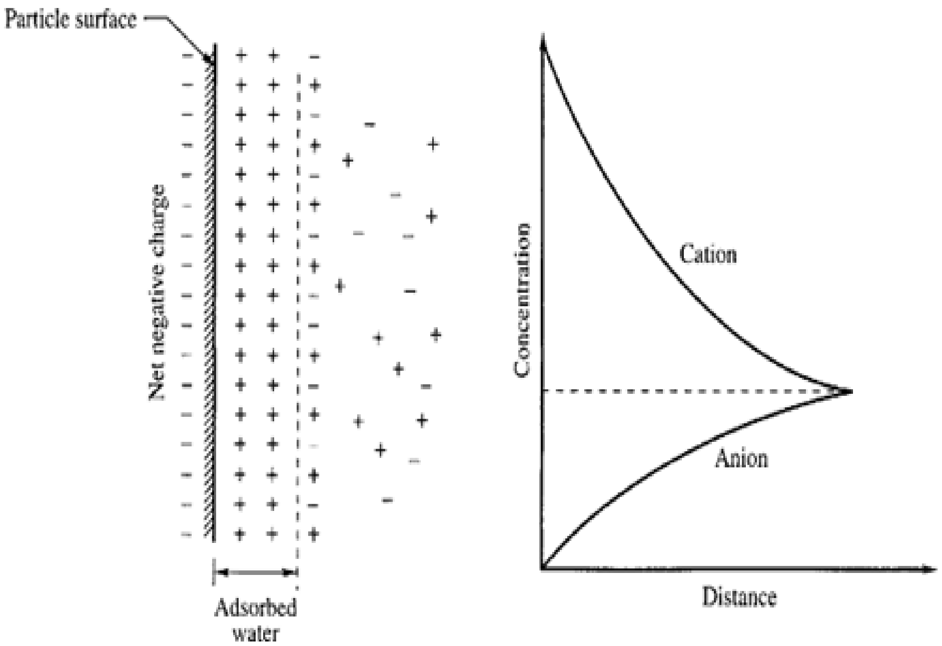

Figure 2.9 Distribution of cations and anions adjacent to a clay particle surface according to the diffuse double layer theory (Keijzer, 2000).

2.5 Cation Exchange Capacity

“Cations held on the clay and organic matter particles in soils can be replaced by other cations, thus, they are exchangeable”. For instance, potassium can be replaced by cations such as calcium or hydrogen, and vice versa. “The total number of cations a soil can hold, or its total negative charge, is known as the soil's Cation Exchange Capacity”. In other words, the term CEC is referred to as the quantity of exchangeable cations required to balance the charge deficiency on the surface of the clay particles. Higher CEC, means higher surface activity and consequently higher water absorption potential. Clays with larger specific surface area experience higher water adsorption. Also, Oweis (1998) defined CEC of soil as “the number of cations in miliequivalents that neutralize one hundred grams of dry clay (meq/100 g)”. One miliequivalents is defined as one miligram of hydrogen or any ion that will combine with one milligram of hydrogen or displace it (Oweis, 1998). Table 2.2 illustrates the different value of the CEC with respect to types of the clay minerals.

CEC is referred to as a significant parameter to determine clay mineral properties. Two fundamental properties of clays, surface area and charges on this surface area, can be measured by CEC. As presented in Figure 2.10 clay surface includes two parts, external surface and internal surface.

Number of bonding sites of cations on the external surfaces shows the external exchange capacity. The external CEC is a direct function of the crystal size, for a specific volume or mass.

The bigger the external surfaces, the smaller the crystal size. Therefore it is possible to get information on mean crystal sizes according to the measurement of the external CEC. The overall charge imbalance on the layer structure and clay absorption capacity can be determined by the internal exchange capacity.

Table 2.2 Cation exchange capacity with respect to clay minerals (Lambe and Whitman, 1968)

Mineral	CEC (meq/100 g)
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

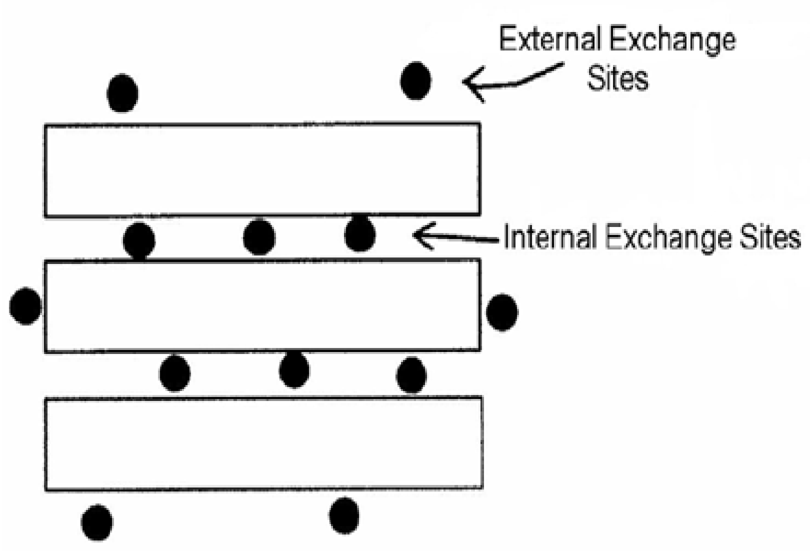

Figure 2.10 Different types of exchange sites on clay particles, Surface and absorbed ion interlayer sites

2.6 Mechanism of Swelling

Fundamentally, clay's swelling consists of two main mechanisms. The first one is the expansion that happens between soil particles. Through this mechanism, seen in all clay minerals, the capillary gap between clay crystals in clay accumulations holds these clay crystals together by its water vacuum force. The clay unit swells when it is subject to moisture, resulting in the release of this tensile force. The second swelling mechanism is generally seen in montmorillonite group clays. When the clay is exposed to water, it percolates through clay crystals as well as weak-bonded singular surfaces that form crystals. Consequently, due to water adsorption volumetric increase, known as clay swell, occurs (Popescu, 1986) (Fig 2.11).

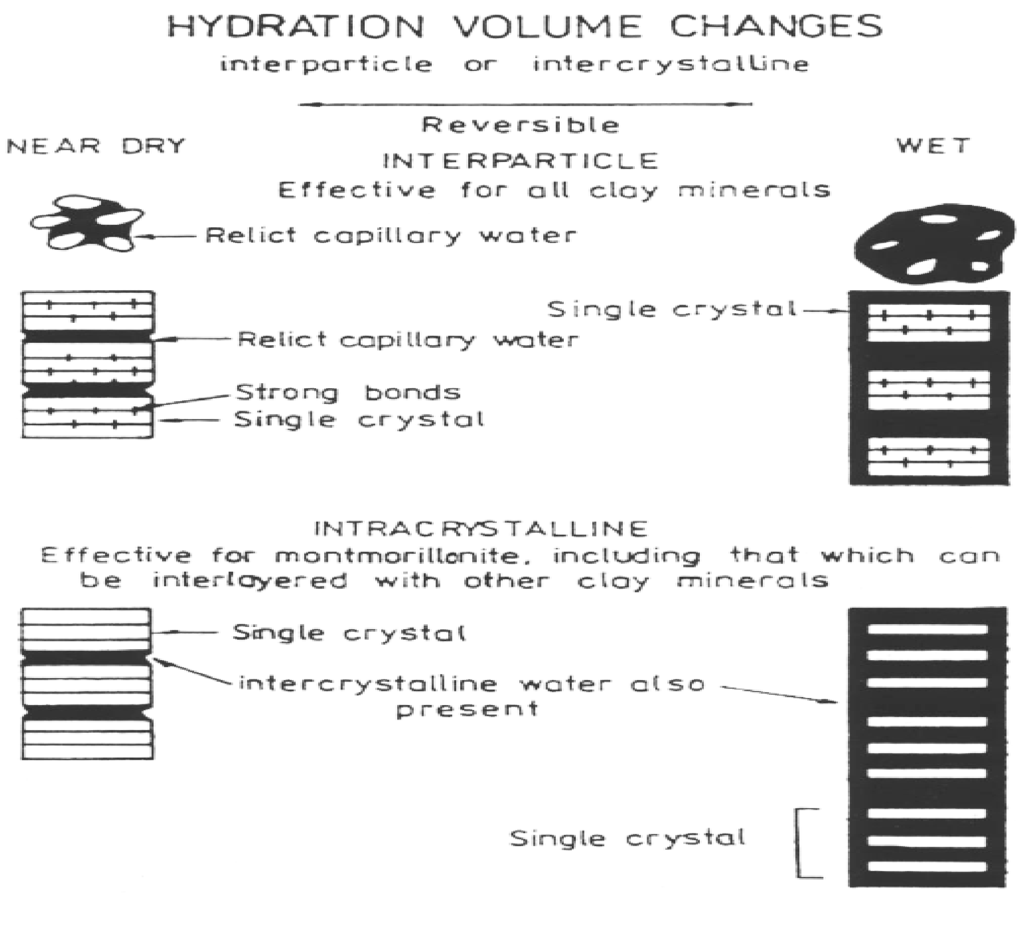

Figure 2.11 Mechanism of swelling (Popescu, 1986)

2.7 Factors Affecting Swelling Behavior of Soil

Factors influencing the swelling behavior of soils are classified into three groups (Nelson & Miller, 1992):

- Soil properties influencing swell potential
- Environmental factors affecting swell potential
- Stress conditions affecting swell potential

Affecting factors are summarized below in section 2.7.1, 2.7.2 and 2.7.3.

2.7.1 Soil Properties Influencing Swell Potential

2.7.1.1 Clay Mineralogy

Kaolinite, illite and montmorillonite are three groups of clay. Montmorillonite mineral possesses highest swelling potential. Also, montmorillonite mixture with other soils at low percentage causes expansion. Although, kaolinites and illites minerals are usually known as nonexpansive soils, they can cause volume change if their particle sizes are extremely fine.

2.7.1.2 Soil Water Chemistry

Increase in cation concentration and cation valence yield decrease in clay swelling. For instance, Mg^{2+} cations (which have thinner DDL and flocculated structure) in the soil water would causes less swelling than Na^{+} (which have thicker DDL and dispersed structure)

2.7.1.3 Plasticity

Actually, soil swelling potential can be demonstrated through its plasticity. In general, soils with greater potential of swelling and shrinkage have higher plasticity index and higher liquid limit.

2.7.1.4 Soil Structure and Fabric

Expansion occurs in flocculated clays more than dispersed clays. Cemented particles are able to reduce swelling. Fabric and structure of clay change because of compaction at high water content or remolding. Kneading compaction has been illustrated to cause dispersed structures with lower swelling potential than soils which are compacted statically with lower water contents.

2.7.1.5 Dry Density

Higher densities affiliated with closer particle spacing which means greater repulsive forces between particles, which bring about higher tendency for expansion.

2.7.2 Environmental Factors Affecting Swell Potential

2.7.2.1 Initial Moisture Content

Naturally expansive soils with low or no moisture content have higher tendency for water than the soil profile with more water content. Conversely, water lose occurs swiftly in a soil profile at higher water content on exposure to drying effects and shrink more than a relatively desiccated profile.

2.7.2.2 Moisture Variations

When the soil profile experiences changes in its water content in the active zone near the upper part of the profile, swelling occurs. The largest variation in moisture content and volume changes of expansive soils occurs in those layers.

2.7.2.2.1 Active Zone Depth

A fundamental criterion of evaluating the swelling surface challenge is the active zone depth (Fig 2.12). The depth in a soil to which periodic changes of moisture occurs (Coduto, 2005). Since moisture content below the active zone depth can be accepted as constant, heaving would not occur in layers beneath active zone depth. Depending on the location of the site, the depth of the active zone varies. Some typical active zone depths for American cities are suggested in Table 2.3. Shrinkage cracks can extend deep into the active zone. Figure 2.13 shows interconnected shrinkage cracks extending from the ground surface into the active zone in expansive clay.

To determine the active zone depth of a field it is necessary to plot the liquidity index against the depth of the soil profile over several seasons (Das, 1999; Güngör, 2002).

$LI = \left(W_n - PL\right)/PI$ …………………….Equation 2.1

Where

LI: Liquidity index of the soil

W_n : Moisture content of the soil

PL: Plastic limit of the soil

PI: Plasticity index of the soil

After the calculation of LI from specified formulation above (Eqn 2.1), active zone depth can be estimated from Fig 2.14 (There is no moisture change in regions with constant LI).

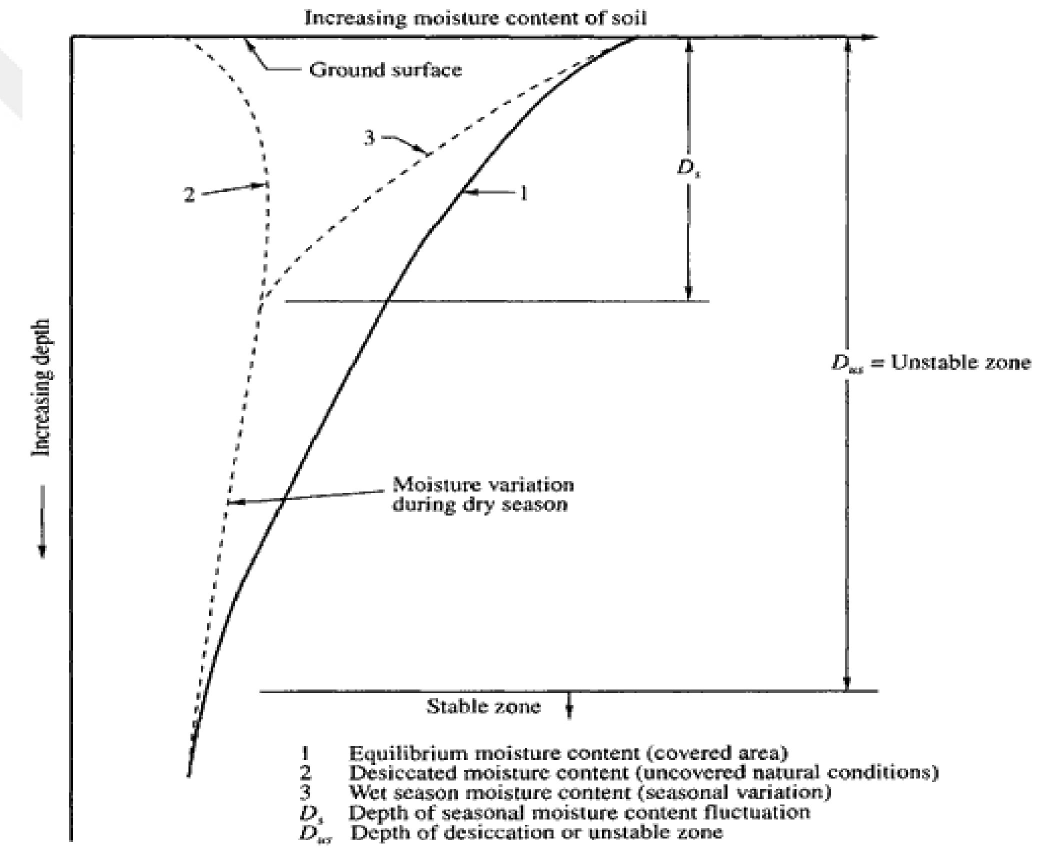

Figure 2.12 Definition of active zone (Kraynski, 1967)

Table 2.3 Typical active –zones depth in some U.S. cities (O'Neil and Poormoayed, 1980)

City	Depth of active zone (m)
Houston	1.5 t0 3
Dallas	2.1 to 4.6
San Antonio	3 to 9
Denver	3 to 4.6

Figure 2.13 Interconnected shrinkage cracks extend from the ground surface into the active zone (Petry, 2000)

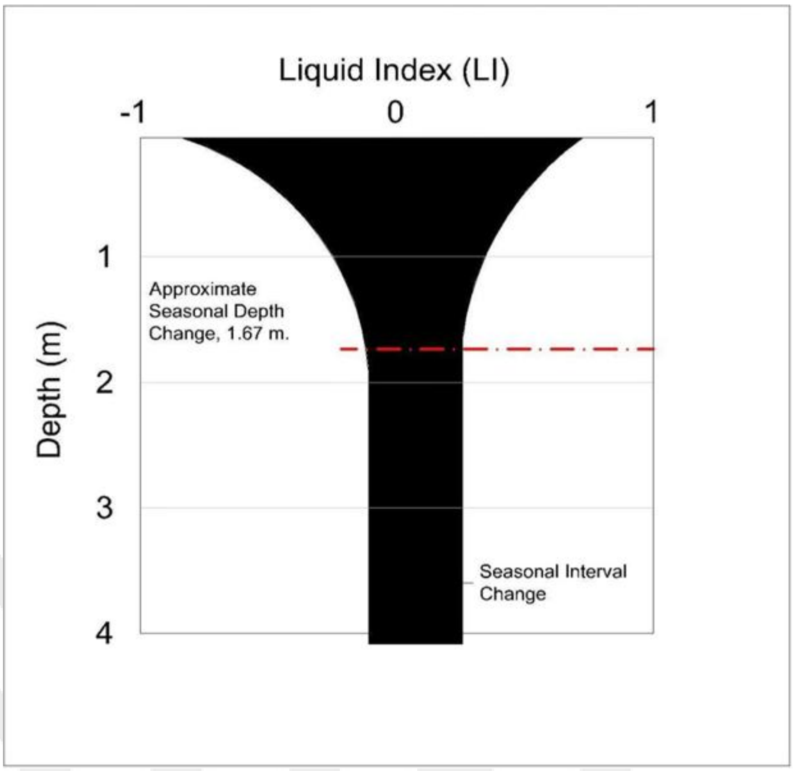

Figure 2.14 Approximate determination of active zone depth (Das, 1999; Güngör, 2002)

2.7.2.3 Climate

The soil moisture availability and depth of seasonal moisture variation are considerably affected by the amount and variation of rainfall and evapotranspiration. The most seasonal heave is seen in semiarid and arid climates which have short wet periods.

2.7.2.4 Groundwater

Occasionally, swelling occurs due to shallow water tables and fluctuating water tables which contribute to moisture.

2.7.2.5 Drainage

Poor surface drainage around a house foundation, poor roof drainage and garden next to shallow foundation result in creation of small body of water, which provides soil access to water in greater depth beneath foundation. Increase in water content causes expansion of swelling soils.

2.7.2.6 Vegetation

The moisture evaporates because of transpiration through trees, grasses and shrubs which cause differential wetting of soil.

2.7.2.7 Permeability

Higher permeability of soil mass, due to cracks and fissures in the field, leads to higher migration of water and accelerate the rates of swell.

2.7.2.8 Temperature

As the temperature increases moisture diffuses towards cooler areas, especially under buildings and pavements.

2.7.3 Stress Conditions Affecting Swell Potential

2.7.3.1 Stress History

An over-consolidated soil is prone to expansion more than the same soil which is consolidated normally at the same void ratio. The pressure caused through soil swelling increases in aging of compacted clays, but swelling degree is not affected under light loading by aging. Swelling reduces through repeated wetting and drying process in laboratory specimens, but after certain number of wetting-drying cycles, no changes in swelling is detected.

2.7.3.1.1 Cyclic Swelling Shrinkage Behavior

Unexpected displacements and cracks in the structure can be caused via up and down movement of foundations constructed on soil with high expansion potential due to swelling-shrinkage cycles. The studies on wetting–drying cycles show greater influence of this process on swelling potential of swelled surfaces (Tripathy and Subba Rao, 2009). On the other hand, different outcomes are reported by researchers (Türköz, 2009). Researchers exploring this issue evaluate the cyclic swelling-shrinkage behavior in different manner. Some researchers states that when the clay sample repeatedly experiences swelling and shrinkage, the sample will exhibit fatigue phenomenon and consequently less swelling occurs. However other scholars express that in the case of that sample is exposed to water content which is below the limit of sample's shrinkage, swelling potential increases by the amount of wetting and drying cycles. Studying on that issue shows that after the certain cycle of swelling-shrinkage swelling reaches to balance. Türköz (2009) reported that through increase in number of cycle's amount, swelling capability of surface with high expansion potential decreases regarding increasing particle size when cycling effect on swelling potential of surfaces is assessed.

2.7.3.2 In-situ Conditions

To assess the probable consequence of loading the soil mass and/or changing the moisture environment therein, it is required to estimate the initial stress state in a soil. In order to determine the initial effective stress over consolidation ratio geotechnical engineers can take sample from the field and perform tests on it in a laboratory. Also making in situ measurements expresses acceptable data base about soil behavior.

2.7.3.3 Loading

The magnitude of surcharge load specifies the quantity of volume change that will occur for special moisture content and density. Exerted external load acts to reduce expansion and balance interparticle repulsive forces.

2.7.3.4 Soil Profile

Potential volume changes of expansive layers are considerably affected by the thickness and location of potentially swelling layers in profile. Under circumstances where high potentially expansive clays extending from the profile surface to depths below the active zone the greatest movement occurs. When expansive soil layer is overlain by nonexpansive material or overlies bedrock at shallow depth, less movements will be detected.

2.7.3.5 Soil Suction

Soil suction is an influent parameter which is an independent effective stress variable. In unsaturated soils, soil suction is represented by the negative pore pressure. Gravity, surface tension, pore size and shape, saturation, electrical and chemical characteristics of the soil particles and moisture affect the soil suction.

2.8 Common Soil Swelling Determinative Tests

As it was stated before considerable studies have been done in an attempt to evaluate the swelling behavior of plastic clays. Researchers have given greater attention to empirical investigations of the swelling behavior of compacted and natural soils (Holtz and Gibbs, 1956). In general numerous experimental techniques have been suggested to determine and classify swelling characteristics of expansive soils. Interpretations used to qualify expansive clays are not only dissimilar but also based either on soil index properties or results given directly from swelling determination tests.

Many criteria have been proposed to identify and characterize expansive soil, such as liquid limit (Table 2.4, 2.5), plasticity index (Table2.6), shrinkages limit (Table 2.7), free swell index (Table 2.7), percent free swell (Table 2.7, 2.8) and modified free swell index.

Chen (1975) reported that there was no observation to confirm the correlation between shrinkage limit and swelling potential. Sridharan and Prakash (1970) observed that the mechanism governing the clay swelling and shrinkage are different, so it is not useful to use shrinkage limit to predict the swell potential. Holtz and Gibbs (1956) suggested the percent free swell test to assess soils swell potential. It was discussed by Sridharan and Prakash (2000) that it is not possible to make a satisfactory prediction on soil expansivity upon index properties such as liquid limit, plasticity index and related parameters, as they do not consider the effect of clay mineralogy. On the other hand, the free swell ratio predicts soil swelling properties more realistically and satisfactorily. Additionally, this test presents additional information about the nature of the clay mineralogy of soils (Table 2.8).

Table 2.4 Proposed expansive soil classification based on plasticity index properties (Neil and Poormaayed, 1980)

Liquid limit	Plasticity index	Potential Swell (%)	Potential swell classification
<50	<25	<0.5	Low
50-60	25-35	0.5-1.5	Marginal
>60	>35	>1.5	High
Potential swell = vertical swell under a pressure equal to overburden pressure			

Table 2.5 Proposed expansive soil classification based on liquid limit (Chen, 1975)

Degree of expansion	W_L: %	
	Chen	IS 1498
Low	<30	20-35
Medium	30-40	35-50
High	40-60	50-70
Very high	>60	70-90

Table 2.6 Proposed expansive soil classification based on plasticity index (Chen, 1975; Holtz and Gibbs, 1956 ; IS 1498)

Degree of expansion	I_p: %		
	Holtz and Gibbs	Chen	IS 1498
Low	<20	0-15	<12
Medium	12-34	10-35	12-23
High	23-45	20-55	23-32
Very high	>32	>35	>32

Table 2.7 Proposed expansive soil classification based on other measures (Holtz and Gibbs, 1956; Seed H. B. and Woodward R.J, 1962)

Degree of expansion	Colloid content: % minus 0-.001mm	Shrinkage limit: %	Shrinkage index: %	Free swell index: %	Percent expansion in oedometer* as per holtz and Gibbs	Percent expansion in oedometer* as per Seed et al
Low	<17	>13	<15	<50	<10	0-1.5
Medium	12-27	8-18	15-30	50-100	10-20	1.5-5.0
High	18-37	6-12	30-60	100-200	20-30	5-25
Very high	>27	<10	>60	>200	>30	>25

*From dry to saturated condition under a surcharge of 7 kPa.

Note: Shrinkage index = (plastic limit-shrinkage limit).

2.8.1 Free Swell Test

The most common and supported methods of identifying the swelling potential and swelling pressure of plastic clay are direct measurement methods. One of the direct measurements used to assess the swelling soil behavior is use of conventional one-dimensional consolidometer, which is referred as Free Swell Test or One Dimensional Swelling Test (Chen 1975).

Methods for one- dimensional swell test and settlement potential of cohesive soil are explained in standard test ASTM D 4546-08. Three alternative experimental techniques are covered to measure the magnitude of one-dimensional wetting-induced expansion or collapse of unsaturated soils. Additionally, one method to measure the load-induced compression subsequent to wetting induced deformation is presented. Achieved results play a key role in design of floor slabs on grade and assessment of their performance. Since lateral swell and lateral confining pressure are not simulated through this experiment, swelling parameters determined from these experimental methods in order to estimate in situ heave of foundation and compacted soil may not be representative of field conditions. The combination of climatic conditions and the swelling characteristics of the soil play a key role in the quantity of free swell percentage (Nelson and Miller, 1992). For instance, free swell values between1200% to 2000% was reported by testing commercial bentonite. Dawson (1953) stated that in Texas clay free swell value of 50% was observed, which was mentioned as considerable expansion value. Table 2.8 was suggested by Sridharan and K.Prakash (2000) to classify expansive soils due to percent expansion in the oedometer. ASTM D 4829-11 proposed other criteria to determine and classify expansive soil due to expansion index (Table 2.9).

Table 2.8 Proposed expansive soil classification based on oedometer percent expansion (A.Sridharan and K.Prakash, 2000)

Oedometer per cent expansion*	**Free swell ratio**	**Clay type**	**Soil expansivity**
<1	<1.0	Non-swelling	Negligible
1-5	1.0-1.5	Mixture of swelling and non-swelling	Low
5-15	1.5-2.0	Swelling	Moderate
15-25	2.0-4.0	Swelling	High
>25	>4.0	Swelling	Very high

* From air dry to saturated condition under a surcharge of 7 kPa

Table 2.9 Typical values of the expansion index and potential parameter (ASTM D, 4829-11)

Expansion Index	**EI Potential Expansion**
0-20	**Very Low**
21-50	**Low**
51-90	**Medium**
91-130	**High**
>130	**Very High**

2.8.2 Free Swell Index Tests

The free swell or differential free swell, also termed as free swell index, is one of the commonly used simple experiments performed by geotechnical engineers for getting estimates of soils expansion potential (Holtz and Gibbs,1956).

Free swell index test is nothing just increases in volume of soil without any external constraint when subjected to submergence in water. The procedure of this method consists of pouring 10 cm^3 of oven dried soil (passing sieve no.40) into 100 cm^3 measuring jar filled with distilled water and let the sample to rest. Then, the free swell is defined as the increase in the volume of the soil expressed as the percentage of initial volume (Eqn 2.2). The disadvantage of this method is that the measure of 10 cm^3 is not easy and the personal judgment which can be accompanied with error is one of the effecting parameters. It is acceptable to quantify 10 cm^3 as the volume engrossed by 10 gr of soil and it doesn't account for changes of density.

IS 1498 states a criterion to predict the swell potential of soil. This approach is based on the free swell ratio, defined as ratio of the sediment volume of soil in distilled water to that in kerosene or carbon tetrachloride (Eqn 2.3). In some cases, for kaolinite-rich soil, these method results negative free swell indices, subsequently this technique may underestimate the swell potential of monmorillonitic soil if the soils include high amount of kaolinite clay material.

To work out this problem modified free swell index (MFSI) was proposed by Sridharan (1985). This method is based on the ratio of the equilibrium soil volume to the dry weight of the soil. To ready the sediment 10gr soil sample must be oven dried and mixed thoroughly with the distilled water in a 100 ml measuring jar then allow settling (Eqn 2.4).

It was observed that, the alluvium volume occupied with specific weight of the dry soil sample together with in kerosene provides acceptable information about the soil expansivity and constitution of soil type- expansive/non-expansive/ combination of both (Table 2.10).

$FS = \left(V - V_0 / V_0\right) \times 100$ …………………………….. Equation 2.2

$FSI = \left(V - V_k / V_k\right) \times 100$ …………………………… Equation 2.3

$MFSI = V/10$ ……………………………………... Equation 2.4

V: Sediment volume of 10 gr of oven dried soil passing sieve NO.40 placed a 100 ml graduated measuring jar containing distilled water

V_k : Sediment volume of 10 gr of oven dried soil passing sieve NO.40 placed a 100 ml graduated measuring jar containing kerosene.

V_0 : Volume of dry soil

Table 2.10 Expansive soil classification based on MFSI (Sridharan et al, 1986)

MFSI: cm^3/g	Sediment volume in carbon tetrachloride: cm^3/g	Clay type	Soil expansivity
<1.5	**1.10-3.00**	**Non-swelling**	**Negligible**
1.5-2.0	**>1.1 and <MFSI**	**Mixture of swelling and non-swelling**	**Low**
1.5-2	**<1.1**	**Swelling**	**Moderate**
2.0-4.0	**<1.1**	**Swelling**	**High**
>4.0	**<1.1**	**Swelling**	**Very High**

2.8.3 MBV Test

Initially the application of Methylene Blue test was developed in France to determine the suitability of granular material in manufacturing concrete while detecting clay content of granular material. Methylene blue powder, $C_{16}H_{18}N_3SCl$, behave like a cationic dye when mixed with water. In the case of mixing with soil suspension its chloride ions change place with cation in clay minerals to be adsorbed on the surface of clay minerals. According to the clay type and the amount of clay minerals, specific surface area of clay per unit mass and cation exchange capacity the amount of blue methylene solution adsorbed by a given mass of clay changes. Since

Methylene blue molecules have high propensity to be adsorbed onto the negatively charged surface which might otherwise attract cations, evaluation with methylene blue can also be mentioned to give a relative measure of the cation exchange capacity of a clay soil (Çokca and Birand, 1993). Methylene blue test has become a popular method because it does not require specialized expensive test setup and it is easily applicable. This method is a reliable and simple measurement on the existence and characteristics of clay minerals in soil sample, especially in the first stage of exploration (Verhoef, 1992). In general, there are two test methods that have been used in practice, A) Turbidimetric method and B) Spot method. The spot method is more common and kind of a simplified titration technique. To calculate MBV (methylene blue value) a definite amount of methylene blue solution is added in certain volumes to a suspension of fine grained soil, then clay particles of the suspension adsorb methylene blue and the total amount of adsorbed methylene blue is used to obtain MBV (Nevins and Weintritt, 1967; Taylor, 1985; Hills and Pettifer, 1985; Verhoef, 1992). It is useful to provide a correlation between soil index properties (liquid limit, plasticity index, etc.) and MBV to make preliminary evaluations of soil profile. Methylene blue test enables engineers to assess specific surface area (Chiappone et al., 2004; Yukselen and Kaya, 2008), cation exchange capacity (Taylor, 1985; Çokça and Birand, 1993b), swell potential (Çokça, 1991, 2002; Çokça and Birand, 1993a) and fine fraction determination in loose material (Pantet et al., 2007).

Determination of ion adsorption capacity of the soil is possible through methylene blue stain test. This goal can be obtained by verifying the amount of methylene blue needed to cover the entire surface area of clay particles in the soil.

The basis of this method is on titration caused by chemical reaction between free cations of methylene blue acquired by dissolving methylene blue in water and interchangeable clay cations. The biggest capacity for cation exchange belongs to clay particles with the largest specific surface area and the highest negative electrical charge. Increase in specific surface area and electrical charge of the clay particle results in increased adsorption capacity. A number of studies have been carried out by Chiappone (Chiappone et al, 2004), to compare practicality and evaluation of methylene blue test used in laboratories to identify clay minerals as stated in standards. According to ANFOR NF P 94-068 analysis, it is suggested to take soil test sample with 30 to 60 gr in clayey or excessively clayey soils and 120 gr in less clayey soil. On the other hand, ASTM standard which follows same test procedure as ANFOR standard suggests to use 2 gr of soil test sample and acidic milieu (pH ranges between 2.5 and 3.8) is recommended as solvent. Chiappone (2004) stated that for homogenous fine-grained soil test samples it is suggested to use ASTM standard (solely verifying the clay content), whilst ANFOR standard defined test method submits reliable results which presents entire soil test samples, thus is recommended for heterogeneous samples.

2.8.4 Swelling Pressure Test

During the swelling process, the expansion tendency may be fully or partly restrained depending upon the engineering structure in contact of the soil. The pressure exerted from soil under confined condition can uplift the above layers. Several studies have been done to assess the swelling pressure both qualitatively and quantitatively. Also numerous investigation have attempted to identify the various factors affecting the expansion and the pressure caused by it.

The conventional oedometer (consolidation apparatus) was adopted to assess the swelling pressure of expansive soil by Holtz and Gibbs (1954) and Jenning and Knight (1957) for first time. The pressure which must be applied to the soil such that it prevents the expansive soil specimen from any further swelling through wetting process is called swelling pressure. This experiment is also termed as Zero Swell Test (ZST) (Basma et al, 1995; Fattom and Barakat, 2000). On the other hand, Consolidation Swell Test (CST) consists of opposite procedures. Through the CST the soil specimen is allowed to completely heave under a specific applied load by the setting process, then application of gradual load recompresses the soil specimen to its original volume. Therefore, the value of the final applied pressure that prevents the swelling process is termed as swelling pressure.

The Double Oedometer Test (DOT) was proposed by Knight (1957). The settlement rate or total heave can be predicted through this technique .The philosophy of the double oedometer test is based on the void ratio vs applied effective pressure of two normally consolidated and similar samples.

Shanker (1982) reported a comparison between the swelling pressures caused by undisturbed soil and remolded specimen of the same soil (Shanker et al, 1982). Constant Volume Method (CVM) (same as ZST), and Free Swell Method (FSM) were used to evaluate the swelling pressure at the same initial circumstance. Undisturbed samples have higher swelling potential than the remolded samples. Also, FSM method yields greater swelling pressure than the CVM method for both kinds of soils. It was demonstrated that for the given soil the pressure caused through swelling is proportional to dry unit weight and clay content percentage directly; on the other hand, initial water content and initial applied pressure affect the swelling pressure inversely (Yevnin and Zaslavsky, 1970; El-Sohby and El-Sayed, 1981; Basma et al, 1995). The ZST, CST, DOT and RST techniques were used to obtain the previous observation. Restrained Swelling Test (RST) was performed by Basma et al (1995). Through this method a sample is compacted under incremental pressure until it attained equilibrium deformation, then the specimen is subjected to submergence in water until it approaches the full swelling. The swelling potential of the soil is defined as the ratio of the maximum expansion to initial height. The pressure which prevents expansion is expressed as swelling pressure.

The swelling pressure aspect of the soil expansive behavior was explored by Shuani (1996) through standard and modified oedometer tests. To simulate the experimental results a theoretical model was suggested. Since measuring lateral swelling pressure by oedometer setup had difficulties, the study was limited to one-dimensional framework. However, it was discussed that, it is necessary to mention the coefficient of permeability as one of the important parameters required for theoretical simulation.

A reliable inexpensive and cost-effective computer system was designed by Thompson (2006) in order to exert more precise control over the increments of applied pressure during constant volume swelling pressure tests on samples obtained from several field sites. The comparison of the results obtained from these tests with Load-Back Swelling Test (LBST is similar to CST) results clarified that the swelling pressures observed from the LBSTs overestimated the uplift skin friction. Moreover, the swelling pressures obtained from tests reported in literature were comparable with those resulted from constant volume swelling pressure tests.

ASTM D 4546 provides three alternative test methods for evaluating the swell pressure. All the three following techniques require that a soil specimen be confined laterally while loaded axially in oedometer apparatus with access to free water.

Method A) The sample is submerged in water and allowed to undergo vertical volume change at the seating pressure, 1kPa, exerted by the load of top porous stone and load plate. There is no loading until the primary swell is complete. Then additional load is applied until its initial void ratio/height is obtained.

Method B) A vertical pressure, usually equivalent to the in situ vertical overburden pressure or structural loading, is applied to the specimen or both before the specimen is given access to water. Later, the sample is allowed to be submerged with water. The amount of expansion or settlement can be measured at the applied load after movement is negligible. The final applied load which is added to keep the specimen at the initial height is referred as swelling pressure.

Method C) This procedure includes keeping the specimen at constant height by adjustment in vertical load after the specimen is given the access to free water. The pressure that keeps the volume constant is interpreted as swelling pressure.

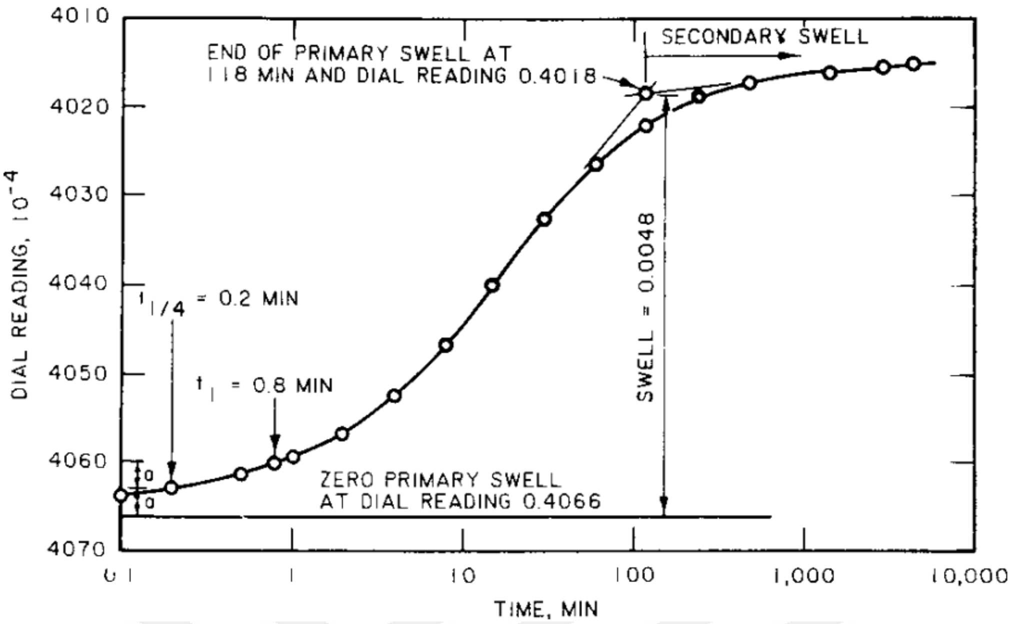

Figure2.15 Time-swell curve (ASTM D (2013). Standard test method for expansion index of soils. Annual Book of ASTM Standards, PA 4546)

2.9 Treatment of Expansive Soils

Recently, due to the increase in population and subsequent increase in urbanization, construction on soils with high expansion potential is unavoidable. Therefore, it is required to utilize some techniques in order to mitigate the damages caused by swelling soils. The appropriate treatment options before and after construction of engineering structures depends on the environmental conditions and soil, and the degree of risk the owner is willing to assume (Nelson and Miller, 1992).

Since the objective of this study is not improving or stabilization of expansive soils, the treatment options are listed below with brief discussion.

Chemical additive

This technique involves application of chemical admixtures to improve the behavior of expansive soil. Chemical stabilization used to stabilize the soil can be found in the form of lime and cement materials and a combination of them. Additionally, the waste materials such as phosphogypsum, ground granulated blast furnace slag and fly ash are utilized to stabilize expansive soil. Chemical additives are used to reduce the permeability of the soil, improve the shear strength, increase bearing capacity, and reduce the settlement and expediting the construction.

Prewetting

The aim of this method is to allow desiccated foundation soils to heave before the construction. One of the most common wetting techniques are ponding or submerging of an area in water. However, it takes long time even years for wetting the foundation subsoil by ponding to increase the water content to the required depths due to the clay chemical structures. It is possible to decrease the time required to arrange the soil moisture content at point where maximum heave will occur to few months by prewetting with the grid of vertical wells (Stavridakis, 2006). Whereas the bearing capacity of the soil reduces because of saturation this method is the most economic method.

Compaction

Actually, the expansive soil with a low dry density may have less expansive potential with respect to soil with a higher dry density, so by reducing the dry density of this soil the swelling potential of expansive soil can be decreased. In other words, Holtz (1959) stated that "compaction of expansive soil, which is compacted at lower density and at water content above the optimum moisture content produces less swell potential than compaction of this soil, which is compacted at high density and low moisture content". This method is not useful for all types of expansive soils because some kind of swelling soils have such a high potential for volume change that compaction control cannot reduce swell potential significantly (Nelson and Miller, 1992).

Soil removal replacement

The process of soil removal and replacement with non-expansive soils is one of the common methods to stabilize the expansive soil. The main reason is that, non-expansive soil compacted at higher density exhibits high bearing capacity than expansive clay. In this method expansive soil should be removed and replaced by non-expansive soil fill to a depth necessary to prevent excessive swell. Chen (1988) proposed that non-expansive soil fill should be at least 1 to 1.3 m. This method is preferable by engineers, since removal and replacement require less delay to construction than some other techniques such as prewetting (Nelson and Miller, 1992).

Surcharge loading

In the case of low swell pressure, such as in a secondary highway system, surcharge loading can be effective. Before applying the surcharge load it is required to determine the depth of the active zone and the maximum swell pressures with soil testing program. In addition, drainage control can be done during surcharge process (Nelson and Miller, 1992).

Thermal methods

Little studies have been done in the United States to apply thermal treatment method to expansive soil. By heating clays to approximately 200°C significant reduce in the potential of volume change occurs. However, economical and applicable methods have not yet been developed (Nelson and Miller, 1992).

2.10 Empirical Correlation

Many empirical correlations have been developed by researchers to predict the swelling properties, swell potential and swell pressure, of natural and compacted soils on the basis of physical and index properties such as consistency limits, clay content, initial moisture content and density. Proposed correlations include some limitation. Since the models are developed based purely on the results of the experimental investigation conducted on particular number of test samples, the results proposed by thee models for other soils are not satisfactory. Additionally, there is no theoretical basis to support the validity of the predictive models.

Various correlations between fundamental properties of soils and their swelling characteristics have been proposed in the past for a variety of expansive soils, some of which are presented in Table 2.11.

Table 2.11 Empirical correlations for predicting the swelling behavior of expansive soils by various researchers.

$Sp(\%) = Be^{A(PI)}$	Chen (1975)
$Sp(\%) = 7.518 + 0.323(C)$	Muntohar (2000)
$Sp(\%) = 60K(PI)^{2.44}$	Holtz et al (1956)
$Sp(\%) = 1.92A + 0.68w_i - 7.55$	J. Israr et al (2014)
$Sp(\%) = k(A^{2.44})(C^{3.44})$	Seed et al (1962)
$Sp(\%) = (k)(M)(PI)^{2.44}$	Seed et al (1962)
$Sp(\%) = m(SI)^{2.67}$	Ranganatham (1965)
$Log(P) = 2.132 + 0.0208(LL) + 0.000665(\gamma_d) - 0.0269(w_i)$	Komornic and David (1969)
$P = (3.5817 \times 10^{-2})(PI)^{1.12} C^2 / {w_i}^2 + 3.7912$	N. V. Nayak (1971)
$Log(P) = -4.812 + 0.01405PI + 2.394\gamma_d - 0.0163w_i$	Yusuf Erzin et al (2004)
$Log(P) = -5.197 + 0.01457PI + 2.408\gamma_d - 0.0163I_L$	Yusuf Erzin et al (2004)

A: Activity

PI: Plasticity Index

C: Clay Fraction

w_I: Initial water Content

SL: Shrinkage Index (Liquid Limit – Shrinkage Limit)

LL: Liquid Limit

γ_d : Dry Density

B, m, k and M: Empirical Constant

CHAPTER 3

EXPERIMENTAL STUDY

3.1 Purpose

As revealed before macro-scale methods are more common to verify swelling behavior of highly plastic clays. The necessity of studying the relation between these macro-scale methods is obvious. The focus of the present experimental study is to investigate the swelling behavior of expansive soils via common swelling determination techniques such as Methylene Blue Test, Free Swell Index Test, Swell Percent Test and Swelling Pressure Test. In addition, the relation between the stated experiments is studied to develop a correlation between them. Also, the correlations between the index properties of the soil samples and Methylene Blue Test, Free Swell Index Test, Swell Percent Tests and Swelling Pressure Test are evaluated. The expansive soil samples are prepared artificially in the laboratory by mixing kaolinite and bentonite at different percentages. Moreover, these tests are repeated on the natural expansive soil samples to assess the results. In order to accomplish these objectives, correlation matrices are obtained, which includes fundamental properties of test samples and results of swell determination tests. These matrices are presented in table 3.4. Furthermore, the values of swell potential and swelling pressure based on currently proposed and previous models are submitted in table 3.3.

In this chapter, the selection of materials, the determination of soil index properties and soil preparation for further experiments are explained. Afterward, the testing procedures used through each experiment to assess soils expansive behavior are addressed in detail in subsections. Also, each subsection describes apparatus used in this research; modified procedure and equipment are discussed, if there is any.

3.2 Material Selection

Considering the main purpose of this research, the appropriate materials proportions (bentonite and kaolinite) are selected based on their moderate to high expansive potential to create test samples with different expansivity with a view to simulate the field conditions. The swell potential of the investigated test samples are required to be low to high in order to ensure that the test samples include necessary clay minerals to qualify for the research.

In that consideration, Bentonite and Kaolinite have been selected to prepare artificial expansive test samples in the laboratory by mixing them at different percentages. Also, two natural expansive soil samples undergo the same experiments.

Kaolinite: Gravel sized Kaolinite were taken from Kaolin Industrial Minerals Company. They were crushed and passed through # 40 sieve after being oven dried at 105 °C for 24 hours (Figure 3.1.a).

Bentonite: Bentonite is production of Karakaya Bentonit Sanayi ve Ticaret A.Ş which is located in Esenboğa/ Ankara. Bentonite was oven dried at 105 °C for 24 hour. Then it was sieved through # 40 sieve before usage (Figure 3.1.b).

Six sets of soil mixtures were prepared by mixing kaolinite and bentonite in different percentages. Soil mixture samples were sieved through # 40 sieve for two times after blending with glass stirrers in order to obtain more homogenous samples. Ceramic containers were used to keep mixtures. Since, clay losses its electromagnetic qualities through contact with metal, it should never be stored in a metal container or stirred with a metal spoon. The only materials that should be used in preparation or storage are wooden spoons or glass stirrers, and either glass or ceramic containers. The first sample, named A, is 100% Kaolinite. Sample B is mixture of 90% kaolinite and 10% bentonite. By adding more 10% bentonite and reducing same amount of kaolinite from the previous test sample on each step, the next test samples are prepared. The most expansive test sample, and the last soil mixture proportion consists of 50% Kaolinite and 50% Bentonite (Table 3.1).

Two natural expansive soil samples were taken from Ankara (from Konya Yolu and Bilkent regions), known as Ankara clay. Similar to the artificial test samples, they are crushed and passed through # 40 sieve after being oven dried at 105 °C for 24 hour. (Figure 3.1.c, Figure 3.1.d)

Figure 3.1.a Views of bentonite and kaolinite used in this study

Figure 3.1.b Views of natural samples used in this study

Table 3.1 Test soil samples and their symbols

Tests Soil Samples	Character
100% Kaolonite	A
90% Kaolonite+10% Bentonite	B
80% Kaolonite+20% Bentonite	C
70% Kaolonite+30% Bentonite	D
60% Kaolonite+40% Bentonite	E
50% Kaolonite+50% Bentonite	F
Ankara Clay Type 1	G
Ankara Clay Type 2	H

3.3 Properties of Soil

In order to classify the test samples and assess the existing correlations between their swelling properties and their material characteristics, a series of tests are conducted. The tests that evaluate the soil properties on which their classification and identification are based listed below;

- Soils Index Properties Tests (Grain Size Distribution including Sieve Analysis and Hydrometer test, Consistency Limits, Plasticity Index)
- Specific Gravity Tests
- Harvard Miniature Compaction

3.3.1 Soil Index Properties Test

The soil properties on which their identification and classification are based are referred as index properties. The index properties which are used are: A) Grain Size Distribution B) Consistency Limits C) Plasticity Index.

3.3.1.1 Grain Size Distribution

Grain size analyses, known as soil gradation test, are performed on essentially all geotechnical particulate materials ranging from clay to boulders. This fundamental experiment refers to discerning the percentage of particles (by dry mass) within a specified particle size range across all the sizes represented for the soil samples. Soil gradation is determined by analyzing the results of a sieve analysis, or a hydrometer analysis for soils containing appreciable quantities of fine fraction (less than 75µm). The properties of the soil are dominantly influenced by the amount of clay and other fractions. Sedimentation methods, hydrometer analysis of fine grained soils, describes the process of particles falling through a fluid, and is used to separate the particles by size in space and time.

Since all test samples are fine grade soils, hydrometer tests were performed to determine grain size distribution of test samples according to ASTM D422 (2007). (Figure 3.2a, Figure 3.2b)

Figure 3.2a Determination of grain size distribution via Hydrometer test

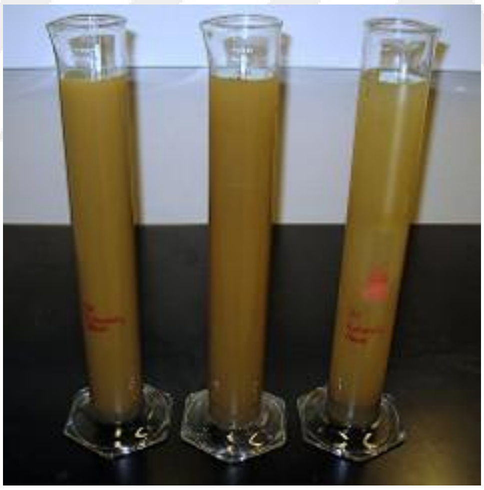

Figure 3.2b Determination of size distribution of test samples

In addition to hydrometer test, there are numerous methods to determine particle size distribution. Laser light scatter, LD is commonly used to determine size distribution of soil particles. In this method, the particle size distribution is determined by intensity of light scattered by a particle and angle of the diffracted laser beam. The soil particle size is directly proportional to the intensity of the scattered light, on the other hand there is an inverse relation between angle of diffracted laser beam and soil particle size.

3.3.1.2 Consistency Limits

The term Consistency of Soil, specifically used for fine grained soils, is referred as the physical state of soil with respect to moisture content present at that time. Also it can be defined as" the resistance to deformation caused by mechanical stress or firmness of fine-grained soils at various moisture contents"

Atterberg observed that the consistency of fine-grained soils are greatly affected by the amount of moisture content present in these soils, therefore, the moisture content at which the soil changes from one state to another state is defined Consistency Limits or Atterberg Limits (Murthy, 2002). Depends on water content, fine-grained soil can exist in any of four states. A) Solid State B) Semi Solid State C) Plastic State D) Liquid State (Figure 3.3)

When a dry soil is subject to water a film of adsorbed water covers each particle. When more water is added the thickness of the water film on a particle increases. Increasing the thickness of the water films enables the particles to slide past one another more easily. According to this fact, the behavior of the soil is related to the amount of water in the system. The boundaries of stated four states are called as "limits" as follows:

Liquid limit: The boundary between the liquid and plastic states;

Plastic limit: The boundary between the plastic and semi-solid states;

Shrinkage limit: The boundary between the semi-solid and solid states.

The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays.

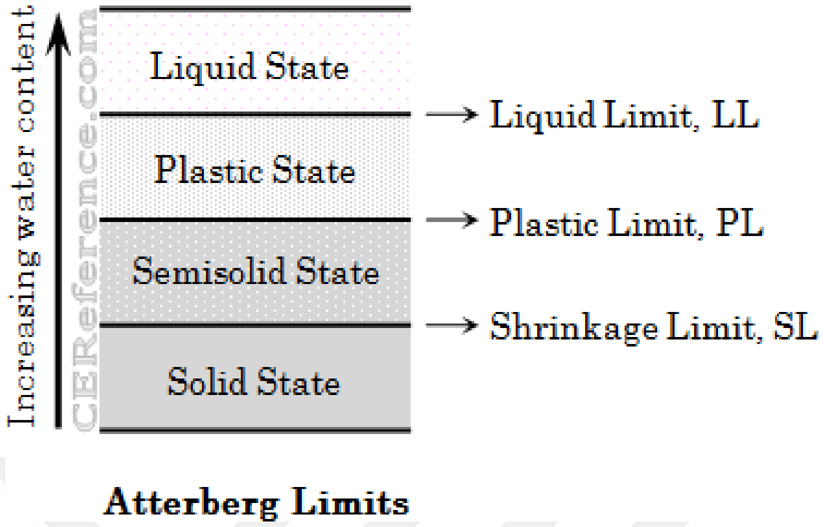

Figure 3.3 Fine-grained Soils States Boundaries According to water content and Atterberg Limits

The moisture content above which the soil-water mixture passes to liquid state is defined as Liquid Limit. At this state the soil-water mixture has such a small shear strength that it behaves like a viscous fluid under its own weight. Any change in water content either side of the liquid limit results in volume change of the soil. There are two common methods to define the liquid limit in laboratory:

- Casagrande Liquid Limit Test; According to this method LL is defined as " the moisture content at which two sides of a groove come closer together for a distance of 12.7 mm under the impact of 25 number of blows" (Figure 3.4, Figure 3.5). Since it is time consuming and difficult to obtain a test with exactly 25 numbers of blows, the procedure is performed multiple times with a range of water contents and the results are interpolated
- Fall cone test method; this method defines Liquid limit as "the moisture content at which a standard cone, starting at soil surface, penetrates with in the soil for 20 mm when it sinks freely for 5 seconds. Since it is time

consuming and difficult to obtain a test with exactly 20 mm penetration, the procedure is performed multiple times with a range of water contents and the results are interpolated. The Fall Cone has the advantage over the Casagrande apparatus. The operation of the apparatus is not influenced by the operator, so the results are comparable independent of the operator. While utilizing the Fall Cone Apparatus, the operator should be aware of the state of the cone, since a worn cone can affect the fall depth, and thereby the results of the Liquid Limit. The soil should be compacted carefully because air pockets trapped in the soil around the point of impact can also affect the measured fall depth. Fall cone test method was conducted according to BS 1377 (2010) (Figure 3.6.a, Figure 3.6.b).

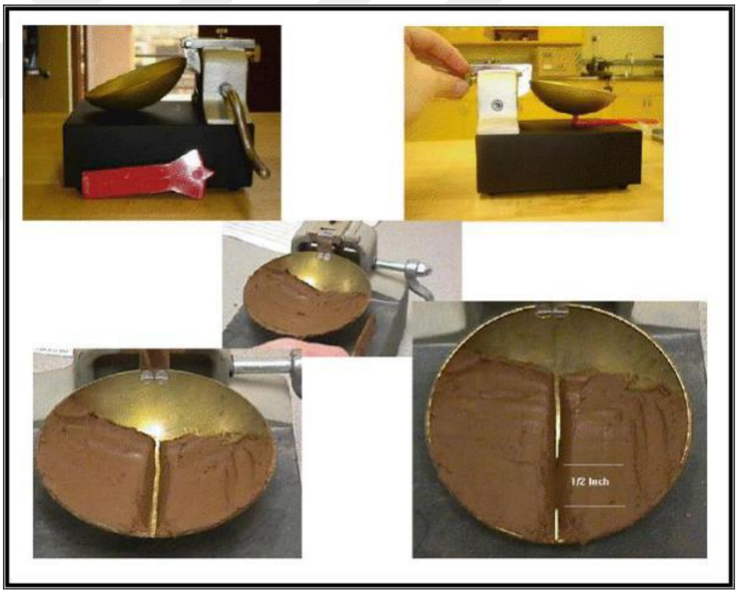

Figure 3.4 Casagrande liquid limit test

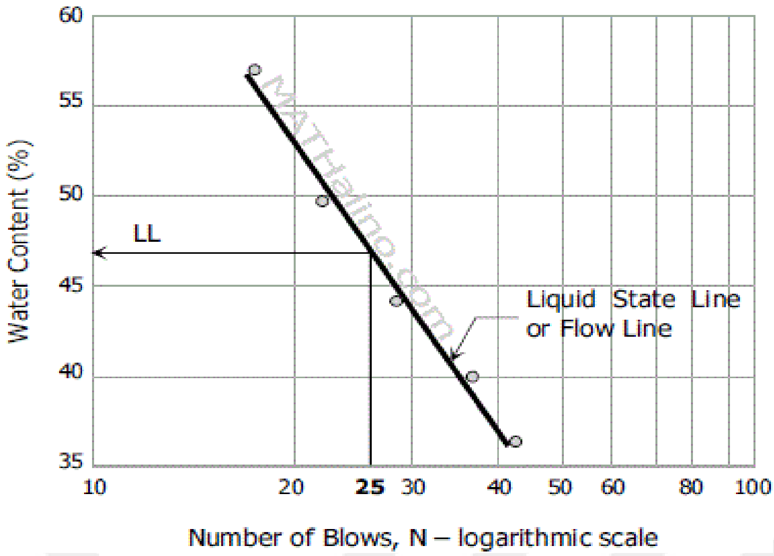

Figure 3.5 Casagrande liquid limit test results

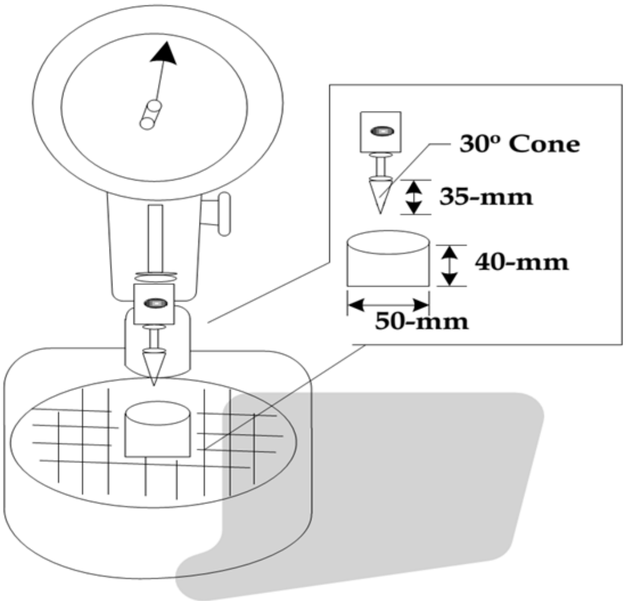

Figure 3.6.a A view of fall cone apparatus

Figure 3.6.b A view of fall cone test

Plastic limit, is defined as moisture content above which the soil-moisture mixture is in plastic state. At this state the mixture undergoes deformations to any shape under any small pressure. By reducing the water content the mixture passes to semi-solid state. Any change in water content at either side of PL causes volume changes of the soil. A small increase in moisture above the plastic limit destroys cohesion of the soil. Two common methods are used to determine plastic limit, the first one is based on ASTM D-4318. In this method, PL is defined as the moisture content (%) at which the soil begins to crumble when rolled up into a thread of 3.2mm (1/8 in) in diameter (Figure 3.7).The alternative method is Fall cone method (as used to determine the liquid limit) which is more accurate which is independent of the operator.

While performing Fall cone method it must be considered that, a cone with a mass of 80 gr and an apex angle of 30° is used to determine the LL, While a cone of similar geometry but with a mass of 240 gr is used to determine the plastic limit (Das 2008) (Figure 3.8).

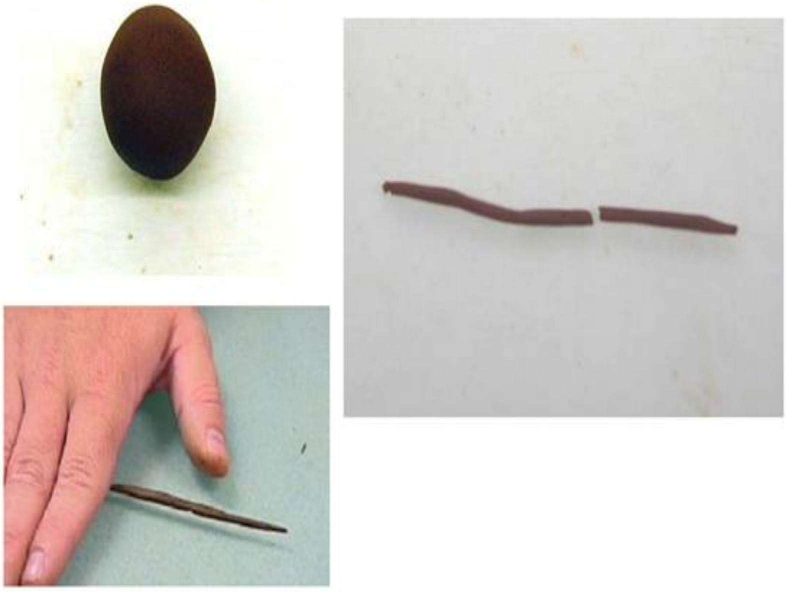

Figure 3.7 Soil crumbles through plastic limit

Figure 3.8 Fall cone masses and cones used to determine LL and PL

The maximum moisture content at which the decrease in water content cause no reduction in total volume of soil but the increase in moisture content results in an increase in moisture content is called Shrinkage Limit (SL). Above this moisture content the soil-water mixture passes to semi-solid state. On the other hand, below this water content the mixer has solid state. Any increase in water content is associated with volume change but no change happens in soil volume as the water content decreases. SL is also expressed as "the lowest water content at which the soil-water mixture is still completely saturated. Determination of volumetric shrinkage limit is carried out according to ASTM D-427. The procedure involves the measurement of initial wet soil mass, dry soil mass and water content of the soil as a percentage of dry mass (Figure 3.9), Finally shrinkage limit is calculated as water content of the soil as a percentage of the dry mass as (Eqn 3.1):

$$SL = W - \left\{ \left[\left(V - V_0 \right) \times \varphi_0 / M_0 \right] \times 100 \right\} \text{ ……………………….Equation 3.1}$$

Where;

SL: Shrinkage limit

W: Initial water content of the soil

V: Volume of the mercury held in the shrinkage dish through test procedure, according to ASTM D-427

V_0: Volume of the mercury displaced into the evaporating dish through test procedure, according to ASTM D-427

φ_0: Density of water equal to 1.0 gr / cm^3 at 20 c° temperature (62.4 lb / ft^3)

M_0: Dry soil mas

Plasticity Index, denoted by PI = LL – PL (Eqn 3.2), is expressed as the range of water content over which a soil behaves plastically. It is referred as the range of consistency with in which the soil exhibit plastic properties.

Figure 3.9 Set up used to determine SL

3.3.2 Specific Gravity

Specific gravity is referred as the ratio of the density of a substance to the density of a reference substance such as water. ASTM D854 suggests a method to determine fine grained- soil specific gravity. Samples are oven-dried at 105 for a period of 16 to 24 hours. To perform the test, it is necessary to have empty weight of pycnometer and weight of pycnometer with oven dry soil. Then add water to cover the soil in the pycnometer and screw on the cap. To remove entrapped air it is necessary to shake the pycnometer well and connect it to the vacuum pump for about 10 to 20 minutes, finally fill the pycnometer with water (Figure 3.10).

The Specific gravity of soil solids (G_s) is calculated using the following equation (Eqn 3.3):

$$G_s = (W_2 - W_1) / \left[(W_2 - W_1) - (W_3 - W_4) \right] \text{ ………….Equation 3.3}$$

Where;

W_1 = Empty weight of pycnometer

W_2 = Weight of pycnometer + oven dry soil

W_3 = Weight of pycnometer + oven dry soil + filled water

W_4 = Weight of pycnometer + filled with water only

Figure 3.10 A view from soil specific gravity test

3.3.3 Harvard Miniature Compaction

To improve loose soils in construction of highway, earth dams and many other engineering structures, it is necessary to compact them in order to strengthen them by increasing their unit weight. Compaction is defined as densification, rearrangement of soil particles, of soil by removing air voids using mechanical equipment such as highway compaction machines. Soil dry unit weight is reference parameter to measure the degree of compaction. Increasing the bearing capacity of foundation, decreasing the undesirable settlement of engineering structures, control undesirable volume changes, reduction in hydraulic conductivity and increasing the stability of slopes are the main objectives of compaction. It must be considered that through compaction densification there is no fluid flow; on the other hand consolidation (other kind of densification) involves pore water flow under load.

Degree of compaction is influenced by four control factors:

A) Compaction effort

B) Soil type and gradation

C) Moisture content

D) Dry unit weight (dry density)

Moisture content of the test soil sample affect compaction under constant compaction effort. During compaction added water acts as softening agent on the soil particle and the dry unit weight increases as the moisture content increases to a point. Beyond a certain moisture content sample dry unit weight reduces by adding water. The moisture content at which maximum dry unit weight is attained under constant mechanical effort is referred as Optimum Water Content. (Figure 3.11)

Generally through soil compaction process mechanical effort is constant and the critical point is to determine the optimum water content to obtain maximum dry density. ASTM D698 proposed a standard to simulate field compaction in lab. Two types of tests are suggested to evaluate the optimum water content, A) Standard proctor test and B) Modified standard proctor test. The differences between these methods are based on the compaction effort and layers of compaction (reteg). (Figure 3.12)

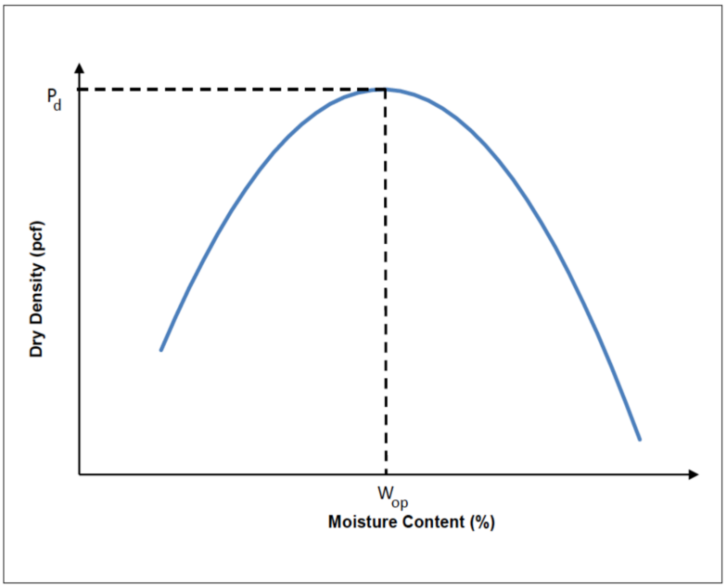

Figure 3.11 Maximum dry unit weight and optimum water content determination through proctor test

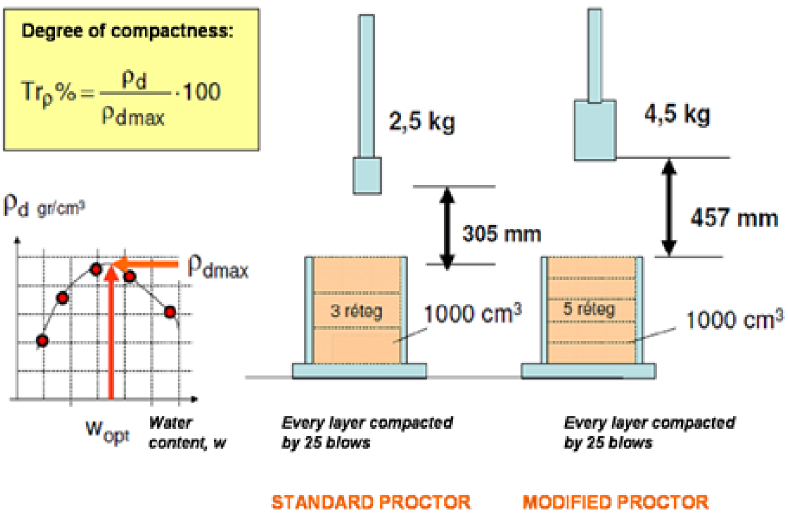

Fig 3.12 Characteristics of standard and modified proctor tests apparatuses

In this study, due to insufficient test sample Harvard Miniature Compaction apparatus is used to determine the optimum water content and maximum dry density.

Harvard miniature compaction apparatus duplicates the kneading action of e sheepsfoot roller type of compaction. The apparatus is furnished with a specimen ejector, collar remover with spacer plate, mold holder, mold and collar, compaction tamper with 20 lbs. (9.07kg) or 40 lbs. (18.2kg) spring (Figure 3.13). Test samples were compacted in three layers after curing for 24 hours using 25 well disturbed pushes with the compaction tamper. Finally, similar to standard compaction procedures proposed in ASTM698, optimum water content and maximum dry density is calculated.

Figure 3.13 Harvard miniature compaction apparatus

3.4 Free Swell Test, Experimental Procedure and Modified Experimental Equipment

Generally, the laboratory test methods to measure the magnitude of One-Dimensional wetting-induced free swell of unsaturated compacted soils are conducted by simple Oedometer test apparatus according to ASTM D4546 (Fig 3.14).

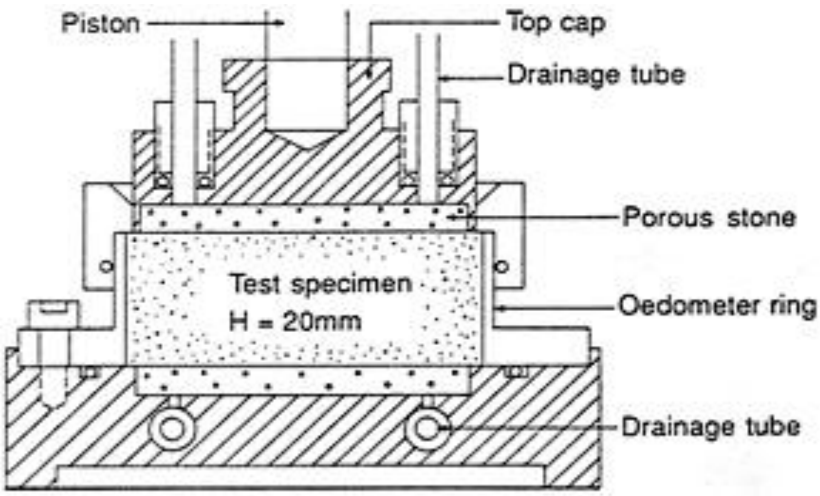

Figure 3.14 Simple oedometer setup

3.4.1 Modified Experimental Equipment

Since the test samples used through this study have high swell potential, it is required to utilize modified setup with similar performance and properties. The utilized setup is comprised of following three parts:

- The base with a ring-shape porous stone
- Rigid circular mold
- Axial load applying device with attached porous stone

The Base With a Ring-Shaped Porous Stone-The setup base, on which the specimen is placed, is stiff enough to prevent lateral and vertical deformation due to swelling. The material of the setup base is noncorrosive in relation to the soil or pore fluid. The air dried porous stone is infixed in the base so that the compacted soil sample stays on it. Considering that the soil sample should be subjected to water from both upper and lower porous boundaries, some grooves and holes are improvised at the bottom of the base. Inner surface of the base around wall has threads that match the outer tracks of the mold (Fig 3.15).

Figure 3.15 The base with ring-shaped porous disk

Rigid Circular Mold -The compaction mold is made of a material with sufficient rigidity to tolerate lateral soil expansion without experiencing changes in its inner diameter more than 0.04% of the diameter under load. Similar to base material, the mold is noncorrosive in contact with soil or pore fluid. The mold has 80 mm height and 50 mm inner diameter (Figure 3.16).

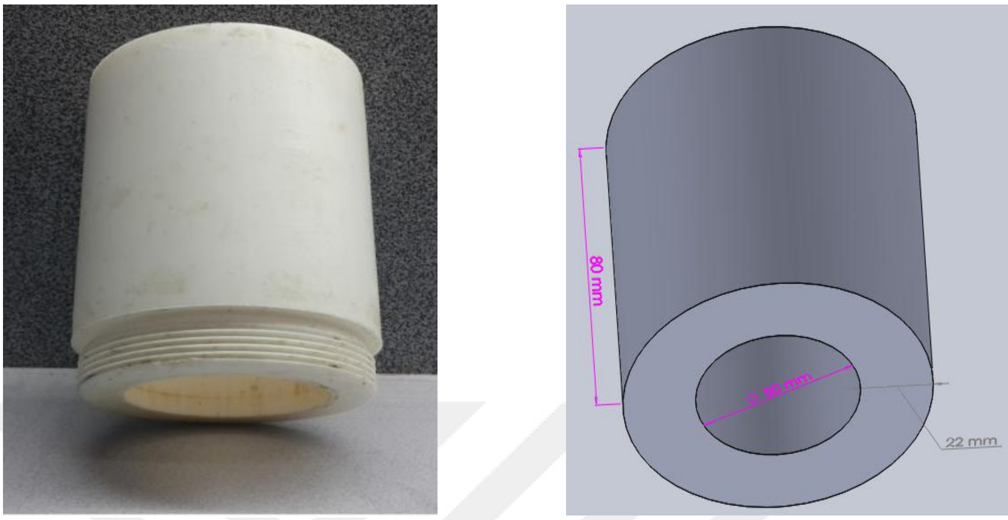

Figure 3.16 The rigid mold used through the study

Axial Load applying device with attached porous stone-According to ASTM D4546 it is required to apply 1 kPa load before wetting the soil sample. This load is applied via metal bar and attached pore stone weight. Some rings are improvised at the top of the bar to hold additional weights if required while applying 1 kPa load. The bar should be polished against corrosion when subjected to water contact (Figure 3.17).

Figure 3.17 A view of axial load applying device with attached porous stone and extra weights

Porous Disks- The porous discs should be of material that resist against corrosion. Sufficient hydraulic conductivity and being fine enough are necessary for porous discs to allow flow of water through specimen and prevent soil penetration into it respectively. Ring-shaped porous disks are boiled in water for about 10 minutes and then exposed to air. They should be completely air-dried before use because even a small amount of water can cause initial soil to swell. To avoid extrusion of the soil specimen while swelling, it is essential to fit porous disks close to soil surface (Figure 3.18).

Figure 3.18 Porous disks used through the study

3.4.2 Experimental Procedure

The procedures used to perform free swell experiments are based on the procedures proposed in ASTM D4546. Prior to soil specimen compaction, the inner surface of the mold was coated with a low-friction material .All test specimens are compacted with a maximum dry density at optimum water content, after 24 hours of curing. The compaction process was done by the help of a hydraulic jack (Figure 3.19) and three spacers (compressor bars) in three stages (Figure 3.20). Final height of the compacted specimen is 20 mm and its diameter was equal to that of mold, 50 mm. After compaction, two porous discs with filter paper were used at the bottom and top of the specimen. According to the procedure expressed in ASTM D4546, it is required to apply 1 kPa load on compacted soil specimen. In order to exert 1kPa pressure axial load applying device is utilized with additional weight. Eventually, dial gage is adjusted to measure the vertical displacements. Subsequently the cylinder containing the mold is filled with water to submerge the specimen. Test water used to submerge soil specimen is Ankara potable tap water (Ankara potable tap water chemical characteristics are presented in appendix A). The improvised tracks at the bottom of the base enable the soil specimen to achieve water as if it gets moisture from the upper level through top porous disc (Figure 3.21).

The soil specimen starts swelling and vertical displacements are recorded until the expansion is completed and no vertical movement is observed (Fig 3.22). Eventually, vertical displacements are plotted against time.

The free swell ratio, also known as swell percent is defined as follows (Eqn 3.4):

Swell percent = $\left(\Delta H / H_i\right) \times 100$ ………………..Equation 3.4

Where ΔH is height difference which is between initial height and final height

$\Delta H = \left(H_f - H_i\right)$

H_i = Initial height of the sample

H_f = Final height of the sample

Figure 3.19 Hydraulic jack used to compact samples for free swell test and prepared test sample

Figure 3.20 Spacers used to compact samples

Figure 3.21 A view of free swelling test

Figure 3.22 A view of test samples after swell completion

3.4.3 Friction of Molds

To determine the friction stress of the inner section of the molds, prior to soil specimen compaction, the inner surface of the mold is coated with a low-friction material then test specimens are compacted in three steps with a maximum dry density at optimum water content, after 24 hours of curing. The specimen is loaded gradually by use of triaxial shear set up, the load at which the sample begins to move is recorded as friction force (F_f). Friction stress is determined as the ratio of friction stress (σ_f) to side area (A_s) (Eqn 3.5). For all test samples, the ratio of friction stress to swell pressure is about than 12%.

$\sigma_f = F_f / A_s$ ………………..Equation 3.5

$A_s = 2\pi r \times t$ ………………..Equation 3.6

Where;

σ_f : Friction Stress

A_s : Side area

F_f : Friction Force

3.5 Free Swell Index Test, Experimental Procedures and Equipment

In 1959 a more convenient and quick method was suggested by Mohan and Goel (1959) to assess the free swell behavior of expansive soil. According to Indian standard (IS: 2720 ,Part XL, – 1977) this method consists of pouring 10 gr oven-dry soil passing No.40 sieve separately into two glass graduate cylinders of 100 ml capacity, one cylinder is filled with distilled water and the others containing kerosene up to 100 ml. To remove entrapped air of suspension it is essential to shake or stir the suspension gently with a glass rod. The final volume of the soil in each cylinder is read out after they attain the state of equilibrium (for not less than 24hours)

Free swell or differential free swell, also termed as free swell index is expressed as percentage as follows (Eqn 3.7):

Free swell index percent = $\left[(V_d - V_k)/V_k\right] \times 100$ ……………….Equation 3.7

Where;

V_d = The volume of soil specimen read from the graduated cylinder containing distilled water.

V_k = The volume of soil specimen read from the graduated cylinder containing kerosene

Some precautions should be mentioned in case of highly expansive soils such as Sodium bentonites. The sample can be 5 gr instead of 10 gr or alternatively 250 ml capacity cylinders can be used for 10 gr of samples.

In this study some challenges are encountered through performance of experiments according to highly expansive test samples. To work out these challenges some changes were applied in the experimental procedures.

Inasmuch as, highly expansive soils exhibit quick reactions with water, a layer of mud is formed while pouring water. This layer prevents water penetration to lower part of the specimen, Water penetration blockage causes disturbance in the completion of sample free swell (Fig 3.23).

To work out this problem a glass rod was utilized to stir the suspension to enable lower levels of the specimen to access water in order to swell, because of the particle adhesion to cylinder body and glass rod, and soil particle pallets this alternative was not useful. The process used to provide water accessibility for all soil particles is using 250 ml capacity cylinder and pouring 10 gr specimens in multiple stages. After each soil pouring stage the cylinder was capped by small rubber stopper and the solution was agitated by turning the cylinder upside down in back for few times. Finally, allow the suspension to attain the state of equilibrium, which may takes about few days.

In this study, due to lack of Kerosene, Gasoline was used. The molecular formulas of Kerosene can range from $C_{12}H_{26}$ to $C_{15}H_{32}$, on the other hand Gasoline possesses molecular formula of C_8H_{18}. The only difference between Kerosene and Gasoline is the molecular weight of these chemical compounds. Since there is no reaction between Gasoline and soil particle molecules, due to lack of Kerosene, Gasoline was used instead of Kerosene to quantify the volume of dry soil during this study (Fig 3.24).

In the case of samples with significant amount of kaolinite the presented method yields negative value for free swell index. To eliminate this problem a Modified free Swell Index method (MFSI) was proposed by Sirdharan (1985). MFSI is defined as the ratio of V_d to dry weight of test sample (10 gr). In this study, due to presence of kaolinite, MFSI of samples are assessed instead of FSI.

Figure 3.23 Formation of mud which prevent water penetration into lower layers

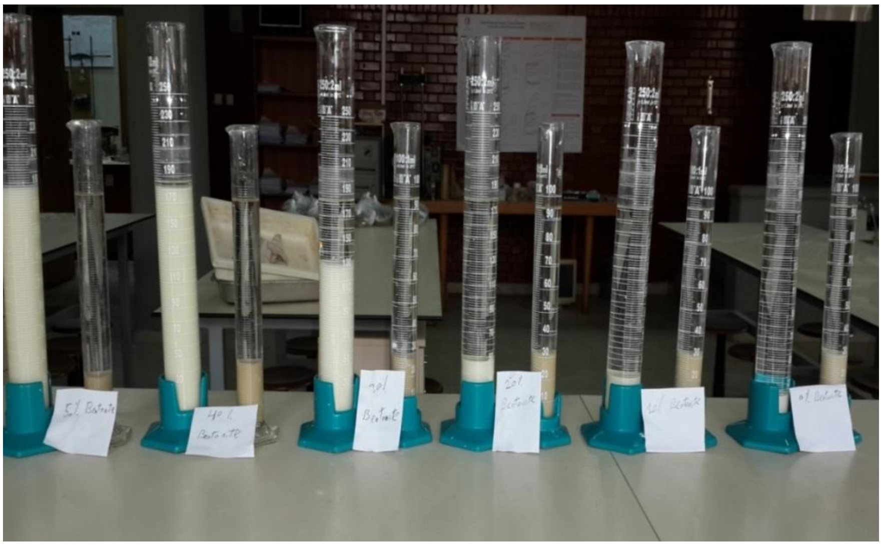

Figure 3.24 A view of free swell index test and use of Gasoline instead of Kerosene

3.6 Methylene Blue (MB) Test, Experimental Procedures and Equipment

Methylene Blue test is conducted according to French standard NF P 94-068 (AFNOR, 1993). The setup used to perform MB test is given in Figure 3.25.

The Methylene blue used in this study is identified with the chemical formula: $C_{16}H_{18}N_3SCl$. Methylene blue adsorption (MBA) test is a simple and reliable measure of the clay particle surface area, which is influenced by clay type and helps to obtain information on the presence and properties of clay minerals in specimen.

Preparation of Methylene blue dye includes mixing 10±0.1 g of methylene blue in 1L of distilled water in a beaker for 30 minutes. A total 30 gr of oven dried soil, sieved through sieve No.40, in 200 ml distilled water is placed in a beaker and the suspension in prepared by mixing soil-water mixture for 5 minutes at 700 rpm (Çokça, 1991). After adjusting the mixer speed to 400 rpm, the next step is to add 5ml of methylene blue solution to the soil suspension and mix it for 1 minute at 400 rpm, a drop of the solution is taken using a pipette and dropped onto the filter paper and subsequently occurrence of blue ring is checked. The filter paper should be supported in such a way that the wetted surface does not touch any liquid or solid. Systematically 5 ml methylene blue solution is added to the soil suspension with 1 minute interval until a halo of light blue dye surrounds the dark blue spot on the filter paper. By detection of the light blue ring that surrounds the spot the solution should be mixed for 5 minutes without adding any methylene blue to solution. At the end of each minute a drop is taken from the solution to determine the stability of the light blue halo. In the case of blue ring disappearance, 2 ml of methylene blue solution is added to the soil-water-methylene blue mixture with 1 minute intervals to follow the same procedure (Figure 3.26).

The equation 3.8 is used to calculate the methylene blue value for 100 gr of soil.

MBV (g /100g) = V/f …………….Equation 3.8

Where,

V = volume of methylene blue solution injected to the soil solution (ml)

f = dry weight of the sample used (g)

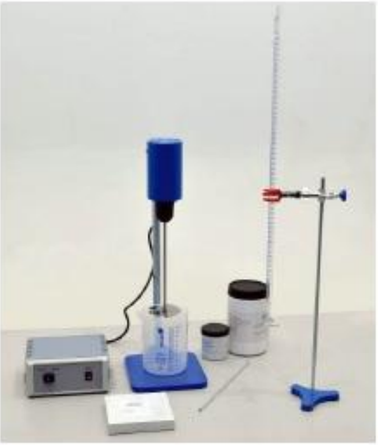

Figure 3.25 Methylene blue test setup

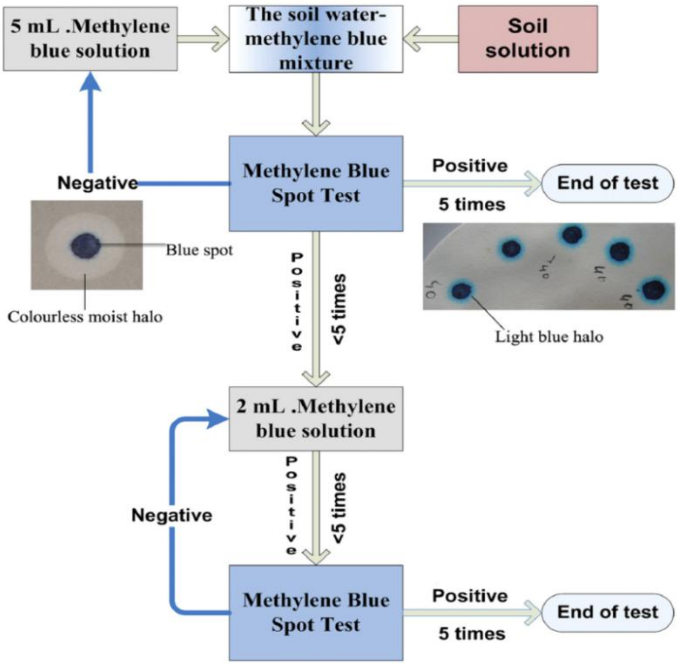

Figure 3.26 Methylene blue stain test flow diagram (Türköz and Tosun, 2011)

3.7 Swelling Pressure Test, Experimental Procedure and Equipment

The swelling pressure is defined as the maximum external load which should be exerted to the soil to prevent expansive soil from any more deformation while wetting.

In general, the conventional consolidometer setup is used by geotechnical engineers in laboratories to evaluate and measure the magnitude of swelling pressure caused by One-Dimensional wetting-induced expansion (Figure 3.27).

After 24 hour of curing specimens at optimum water content to have homogenous samples, specimens were compacted with maximum dry density in holding ring via hydraulic jack and metal spacers (Fig 3.28). The compacted specimen has 20 mm height and 50 mm diameter. Porous stones, completely air-dried after being boiled for 10 minutes, are embedded at the bottom and top of the specimen with ring-shaped filter papers. The ring containing the compacted specimen is placed in circular fiber glass-made cylinder. Before the submergence of the specimen in water, load applicator bar is adjusted and dial gage is reset to zero in order to measure the vertical displacement of compacted specimen by penetration of water. Finally, Ankara potable tap water is used to soak the specimen. By the start of the vertical deformation, pressure is added in small increments to prevent swelling. This process continues until the specimen ceases to heave. Whenever no deformation (or less than 0.05 mm) is observed for few hours, the experiment is completed and the total pressure applied to prevent sample expansion is referred as swelling pressure. As it was acknowledged before, the test used to determine swelling pressure is termed as Zero Swell Test (ZST) (Basma et al, 1995; and Fattom and Barakat, 2000).

Figure 3.27 A view of conventional consolidometer setup

Figure 3.28 A View of Static Compaction with the Hydraulic Jack

3.8 Test Results

In order to obtain exact results all experiments were performed at least three times. Since all results were close, the outcomes of final experiments are submitted as final results. This section includes figures presenting the correlation between sample types and their index properties (Fig 3.29 to fig 3.40). In addition, the correlations between test samples and Free swell percent, Modified free swell index, Methylene blue value and swelling pressure are evaluated. The figures show the results of experiments for artificial test samples.

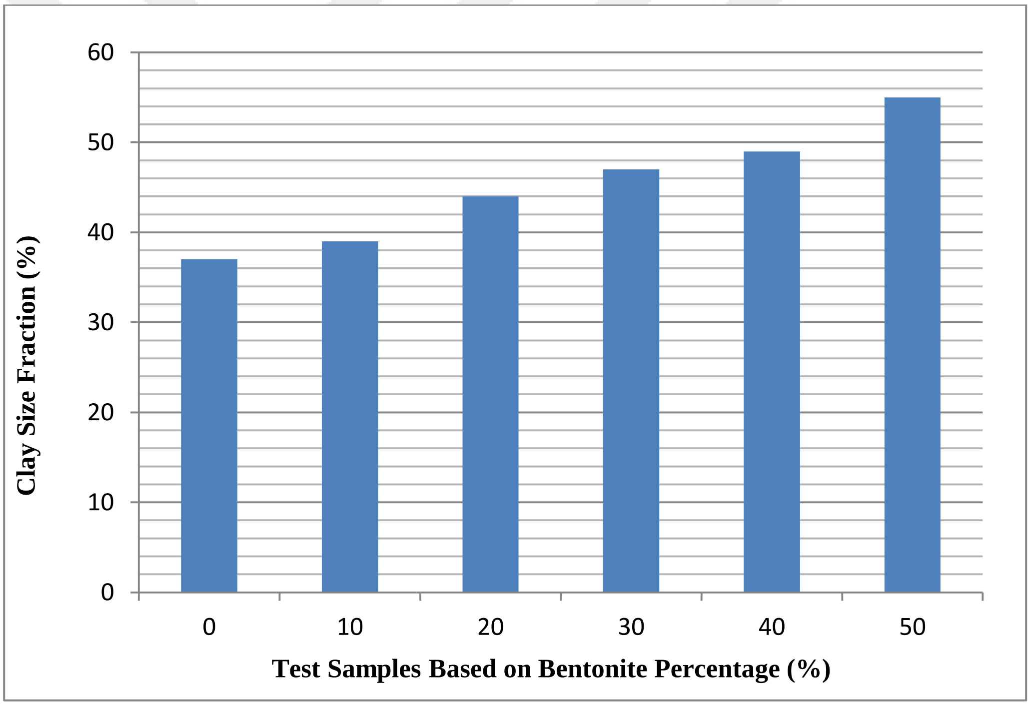

Figure 3.29 Clay Content vs Bentonite Content

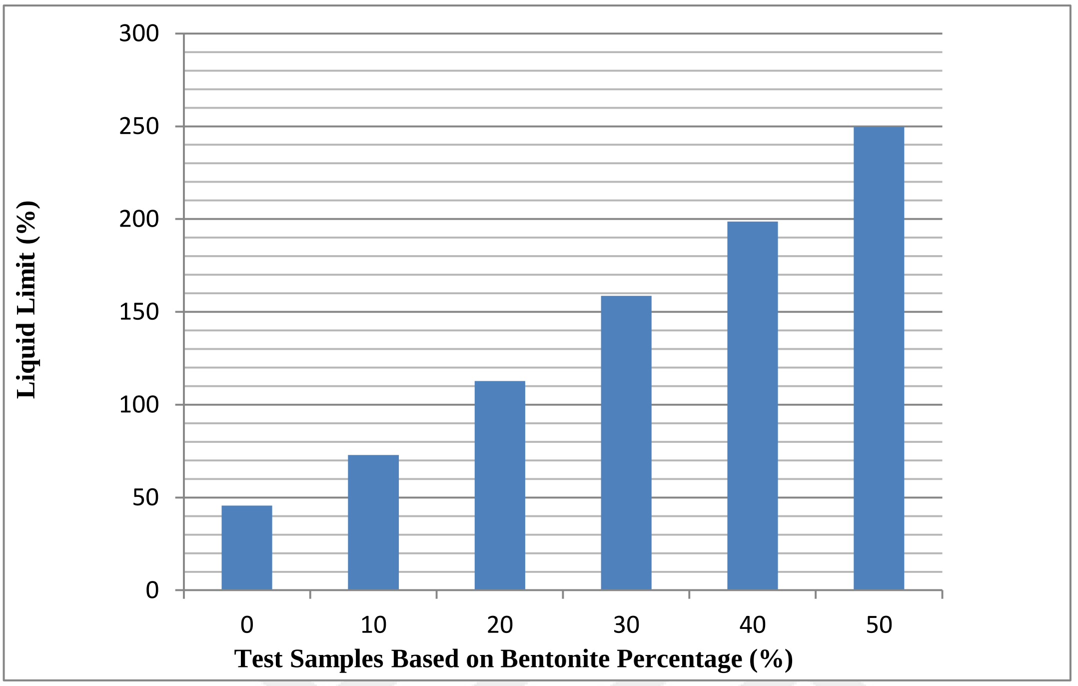

Figure 3.30 Liquid limit vs Sample types

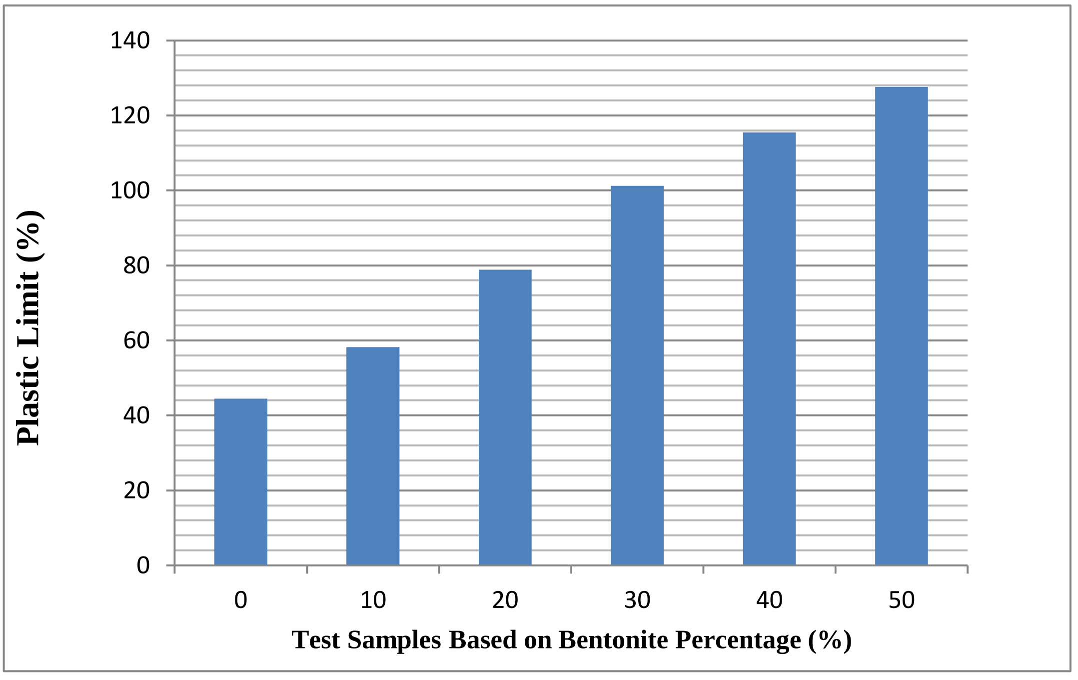

Figure 3.31 Plastic limit vs Sample types

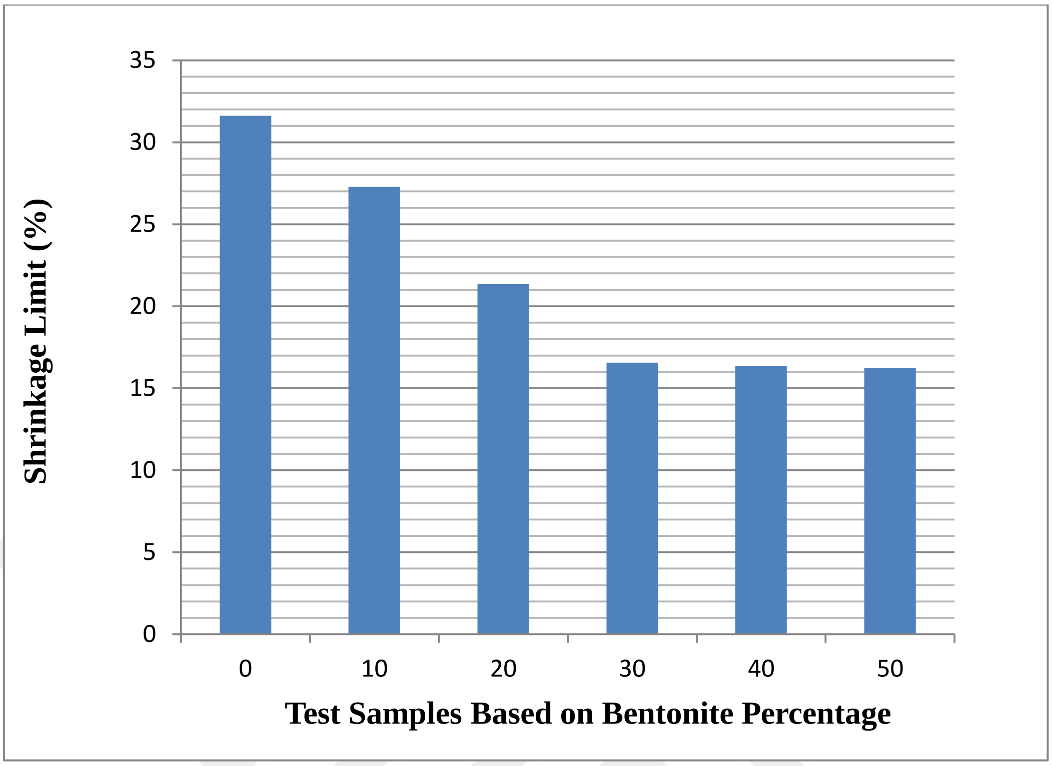

Figure 3.32 Shrinkage limit vs Sample types

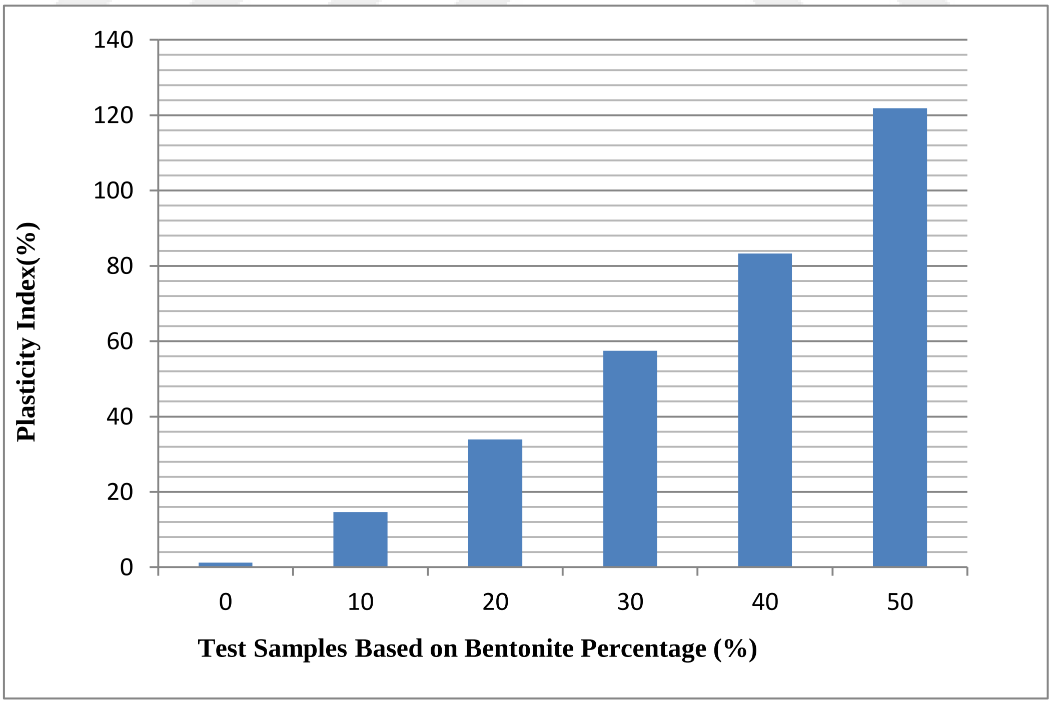

Figure 3.33 Plasticity index vs Sample types

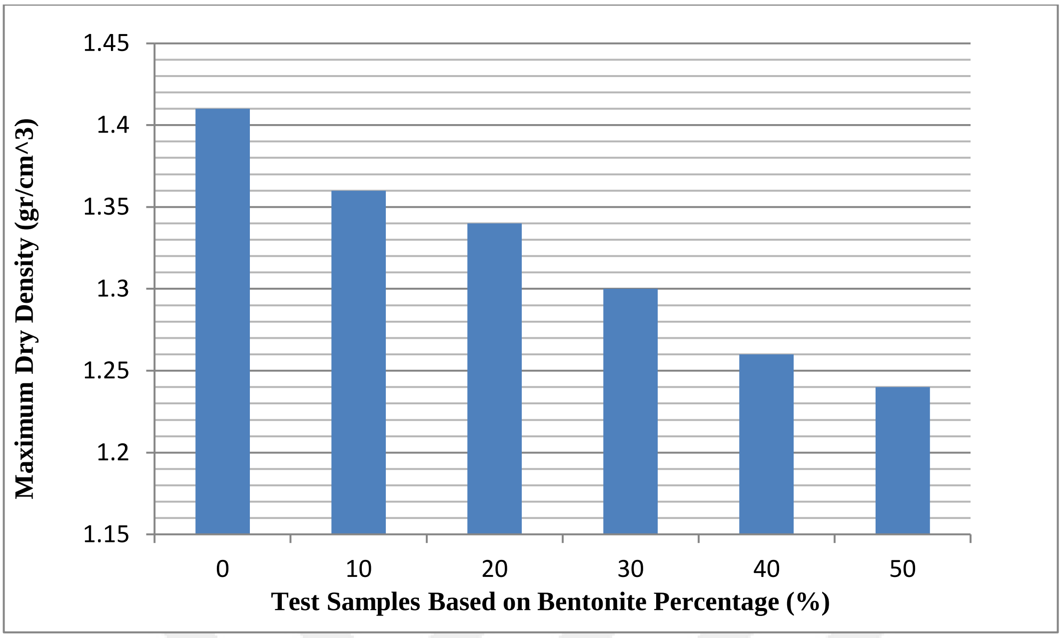

Figure 3.34 Maximum dry density vs Sample types

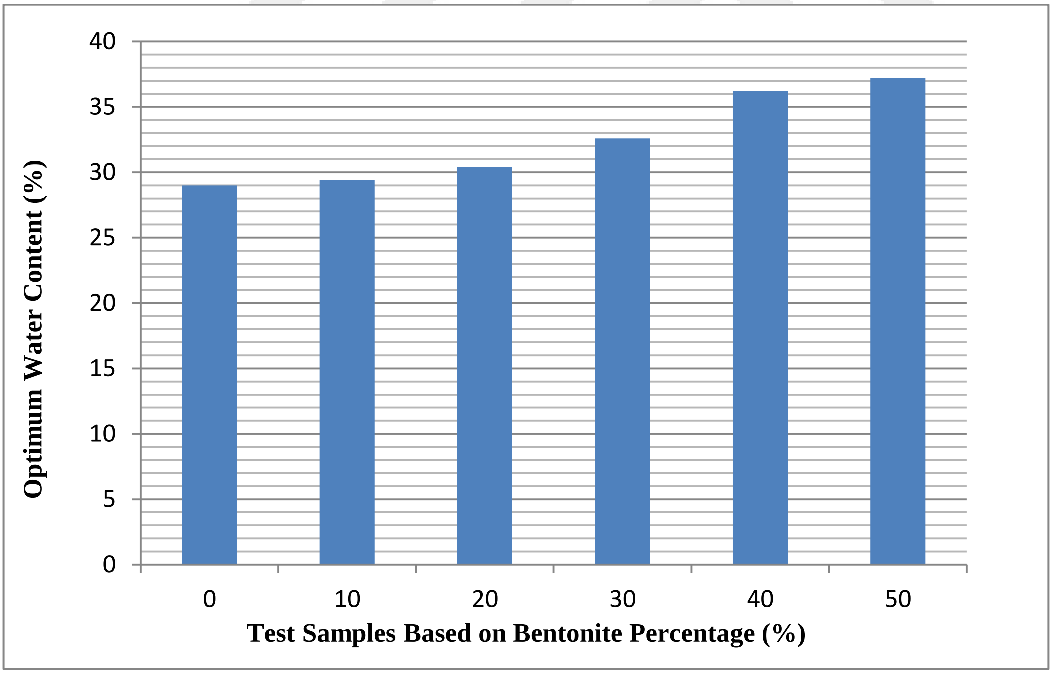

Figure 3.35 Optimum water content vs Sample types

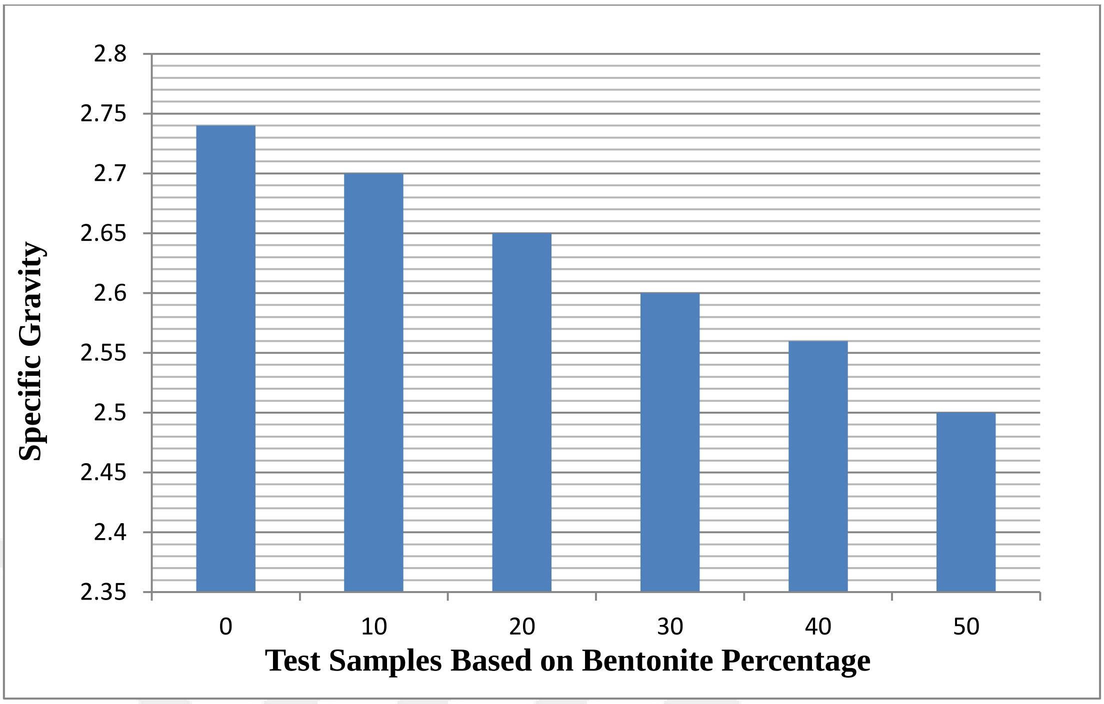

Figure 3.36 Specific gravity vs Sample types

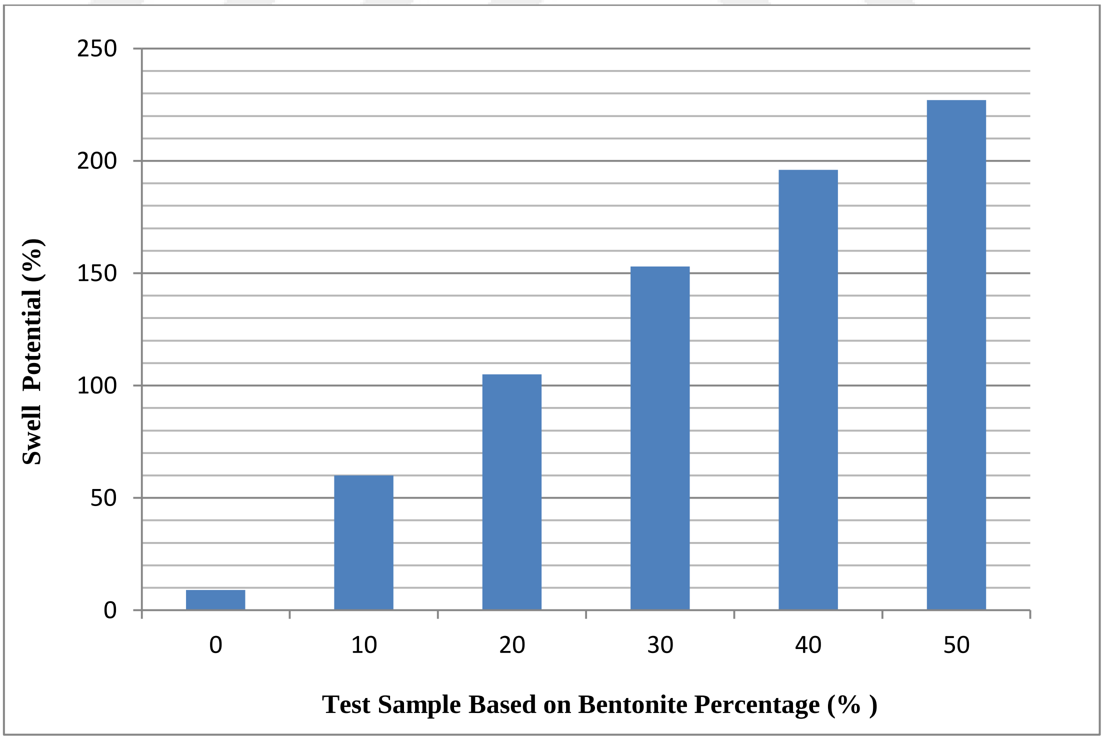

Figure 3.37 Swell Potential vs Sample types

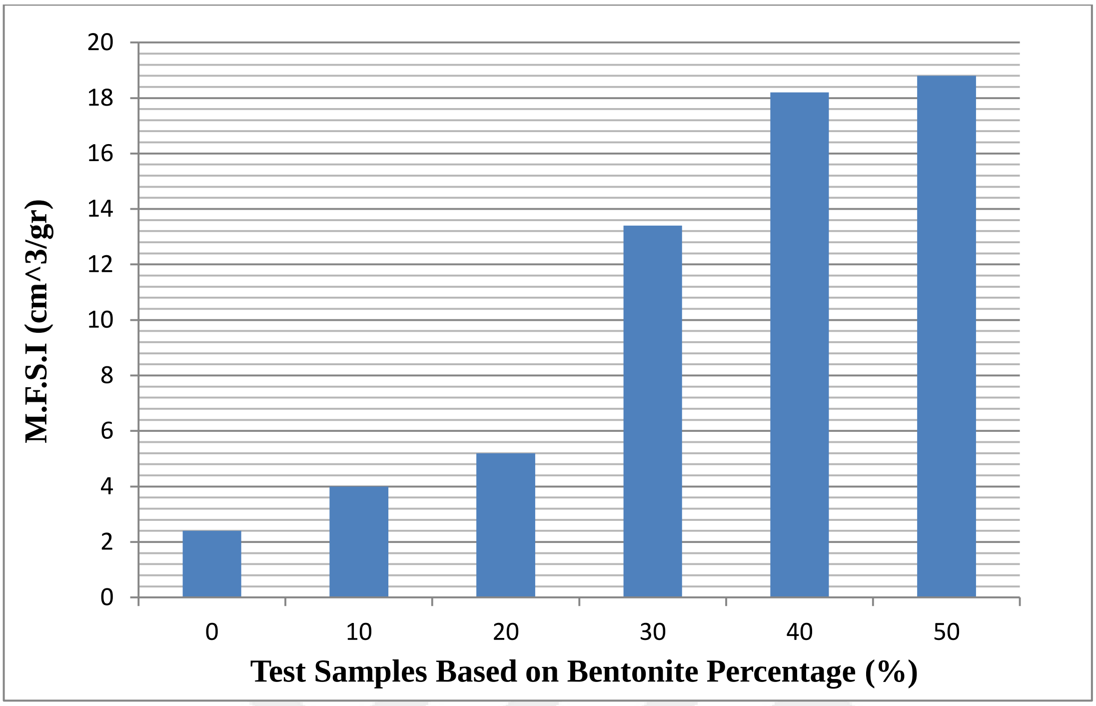

Figure 3.38 MFSI vs Sample types

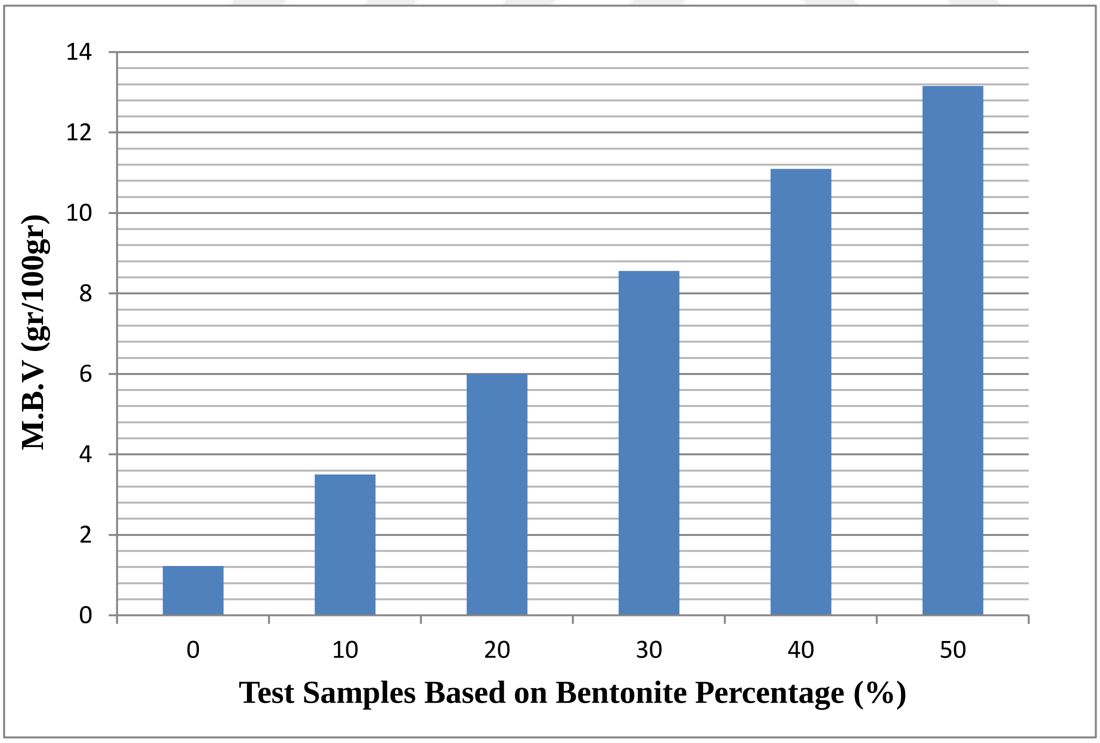

Figure 3.39 M.B.V Sample types

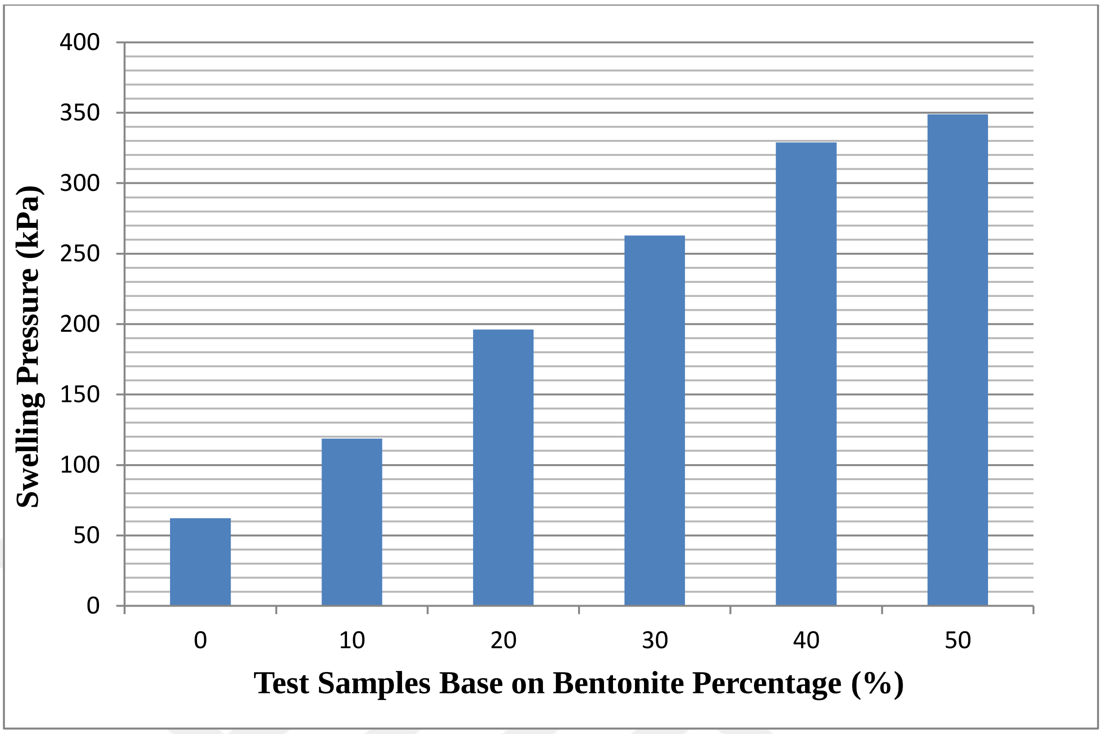

Figure 3.40 Swelling pressure vs Sample types

Table 3.2 Test samples expansivity classification based on swell potential according to criteria proposed by Holtz and Gibbs

Tests Soil Samples	**Soil Expansivity**
100% Kaolinite	Low
90% Kaolinite+10% Bentonite	very high
80% Kaolinite+20% Bentonite	very high
70% Kaolinite+30% Bentonite	very high
60% Kaolinite+40% Bentonite	very high
50% Kaolinite+50% Bentonite	very high
Ankara Clay Type 1	Low
Ankara Clay Type 2	High

Table 3.3 Correlation Matrix A

Sample Type	Swell Potential According to Free Swell Percent (Sp) (%)	Modified Free swell Index (M.F.S.I) (cm^3/gr)	Methylene Blue Value (M.B.V) (g/100g)	Cation Exchange Capacity (C.E.C) (meq/100 g)	Specific Surface Area (S.S.A) (m^2/gr)	Swelling Pressure (P) (kPa)
0%B+100%K	9	2,4	1,23	2,75	25,81	62,11
10%B+90%K	60	4	3,5	7,8	73,25	118,68
20%B+80%K	105	5,2	6	13,38	125,58	196,11
30%B+70%K	153	13,4	8,56	19,1	179,3	262,91
40%B+60%K	196	18,2	11,1	24,75	232,32	328,91
50%B+50%K	227	18,8	13,16	29,36	275,58	348,85
Natural Soil Type 1	3	1,7	8	17,84	167,44	28,38
Natural Soil Type 2	21	2,3	8,5	18,95	177,9	83,34

Table 3.4 Correlation Matrix B

Sample Type	Shrinkage Limit (SL) (%)	Liquid Limit (LL) (%)	Plastic Limit (PL) (%)	Plasticity Index (PI) (%)	Clay Size Fraction (CC) (%)	Activity	Specific Gravity (Gs)	Maximum Dry Density (cm^3/gr)	Optimum Water Content (%)
0%B+100%K	31,61	45,64	44,45	1,19	37	0,03	2,74	1,41	29
10%B+90%K	27,28	72,86	58,22	14,64	39	0,38	2,70	1,36	29,4
20%B+80%K	21,34	112,76	78,82	33,94	44	0,77	2,65	1,34	30,4
30%B+70%K	16,56	158,64	101,22	57,42	47	1,22	2,60	1,3	32,6
40%B+60%K	16,34	198,76	115,46	83,30	49	1,70	2,56	1,26	36,2
50%B+50%K	16,25	249,51	127,62	121,89	55	2,22	2,50	1,24	37,2
Natural Soil Type 1	9,60	64,43	56,18	8,25	57	0,14	2,68	1,38	33,8
Natural Soil Type 2	10,93	86,51	70,32	16,19	53	0,31	2,74	1,4	32,5

CHAPTER 4

ANALYSIS OF RESULTS AND DISCUSSION

4.1 Introduction

The aim of this chapter is to analyze and discuss the experimental results. The correlations between soil fundamental properties, soil swell potential and soil swell pressure are discussed in order to obtain a comprehensive appreciation of swelling behavior of expansive soils. Additionally, the relations between results of common soil swelling tests are evaluated to present a general understanding of soil swelling behavior, swell potential and swelling pressure.

This chapter is organized into the following sub-sections:

- Assessment and discussion of the fundamental properties of test samples.
- Evaluation and discussion of the swelling behavior of test samples, containing analysis of the relations between the test samples, mixtures of bentonite and kaolinite in different percentages, and the results of the soils expansion tests such as, Free swelling test, Modified free swell index test, Methylene blue test and Swelling pressure test.
- Develop the correlation between the swelling behavior of test samples (swell potential and swelling pressure) and fundamental properties of test samples. Additionally, the relations between swelling behavior and MBV, MFSI and some of the index properties of test samples are investigated.

4.2 Analysis and Discussion of the Fundamental Properties of the Test samples

As it was acknowledged before, soil index properties, such as liquid limit and plasticity index, are affected by the type and amount of clay minerals and the percentage of clay fraction. Also, swelling behavior of soils, swell potential and swelling pressure, are influenced by index properties of soils. One of the common methods to predict swelling properties of expansive soils is indirect method, involving use of formulated correlations based on basic soil properties. In this study, several laboratory experiments are conducted on the test samples to investigate their index properties and the existent correlation between obtained results and swelling characteristics of test samples are explored.

As expected, the clay content increases in samples with higher bentonite percentage (Fig 4.1). Also the liquid limit and the plasticity index increase significantly with the bentonite content (Fig 4.2; Fig 4.3). Subsequently the increase in the activity of test samples with higher bentonite fraction is rational (Fig 4.4). Since bentonite has high tendency to water retention, obtained results confirm the suitability of using bentonite as an artificial tool to increase the Atterberg limit of test samples, except maximum dry density.

The optimum water content increases as bentonite content of the test samples increases (Fig 4.5). Test samples with higher bentonite fraction possess lower maximum dry density (Fig 4.6) and optimum water content decreases as the maximum dry density increases (Fig 4.7).

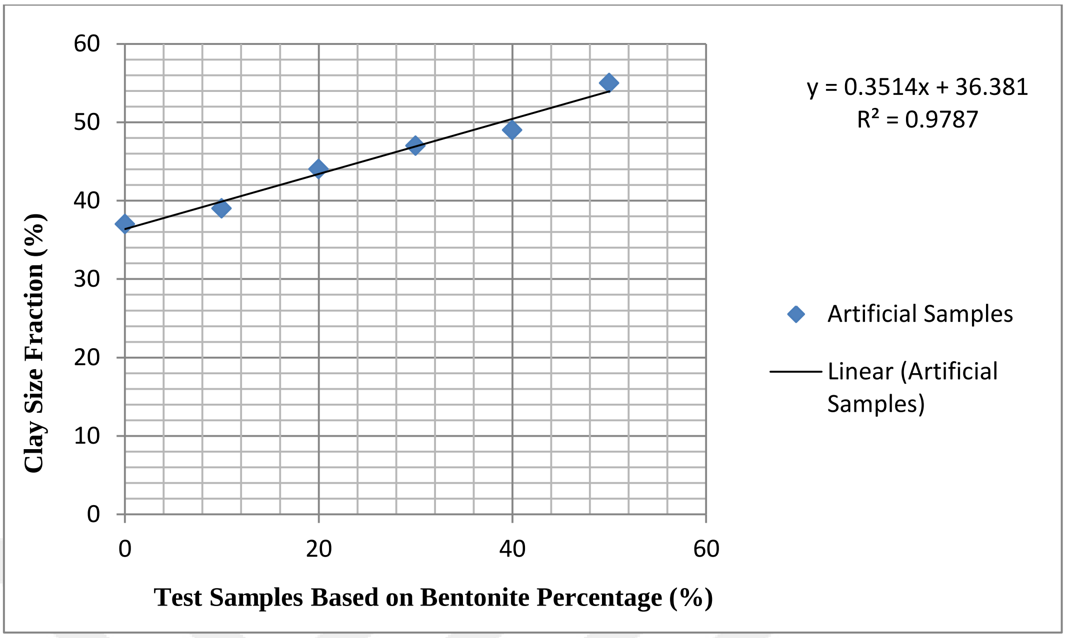

Figure 4.1 Clay Content vs Bentonite Content

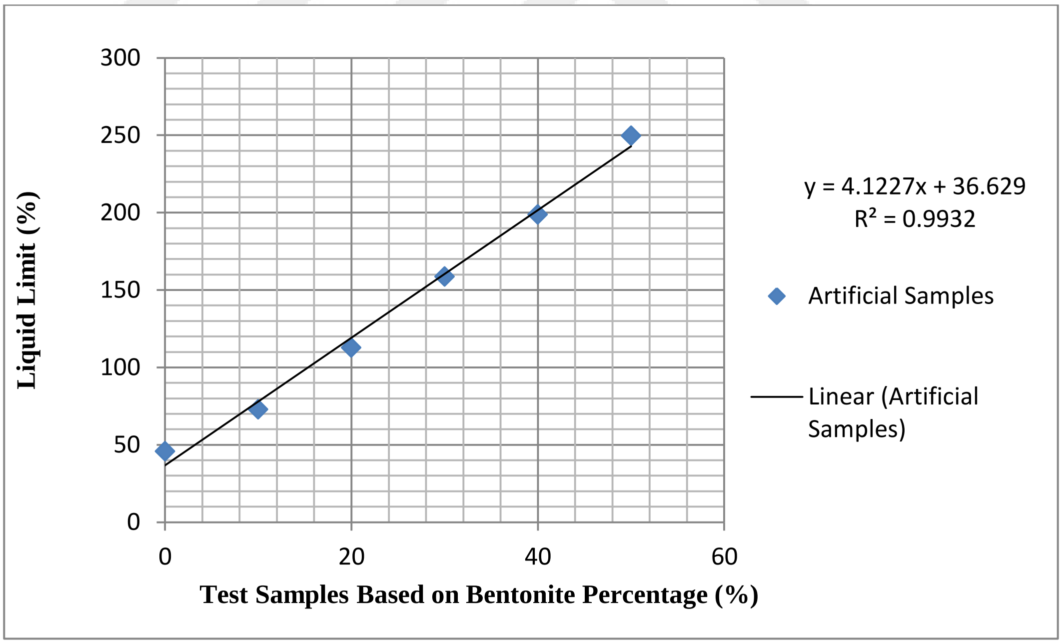

Figure 4.2 Liquid Limit vs Test Samples

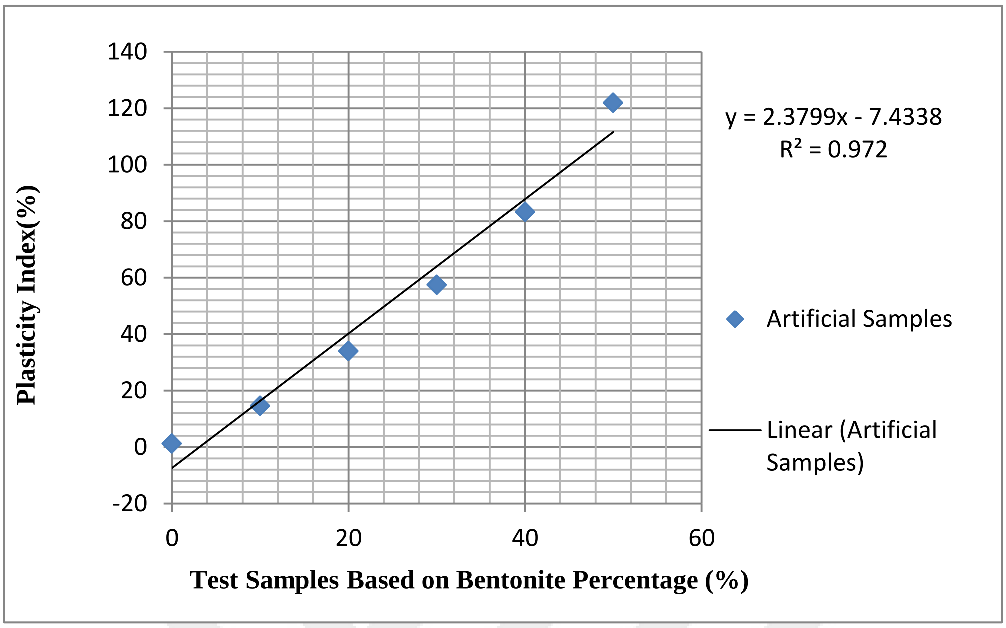

Figure 4.3 Plasticity Index vs Test Samples

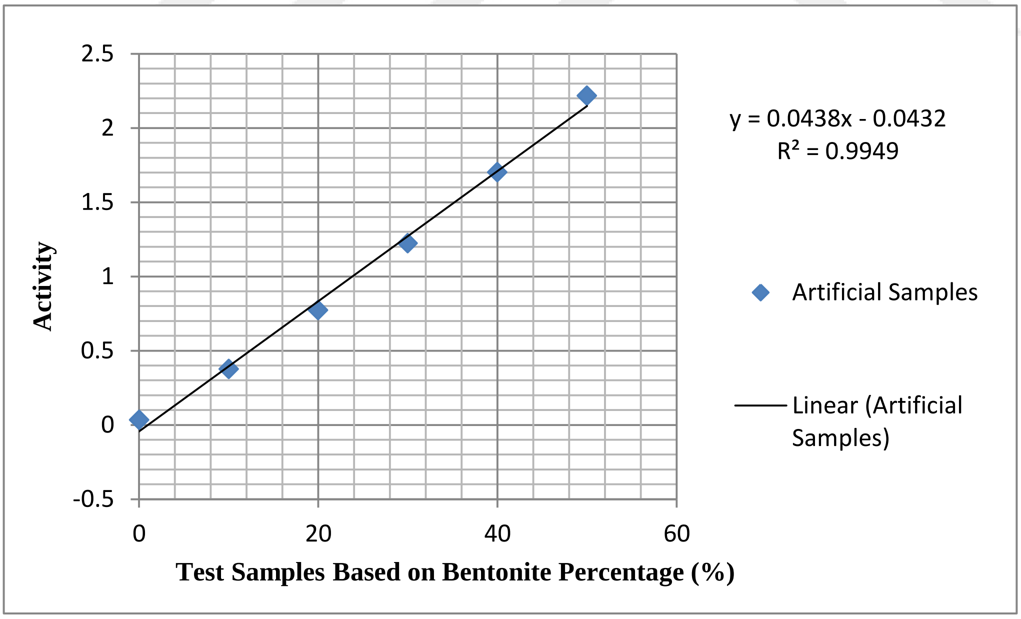

Figure 4.4 Activity vs Test Samples

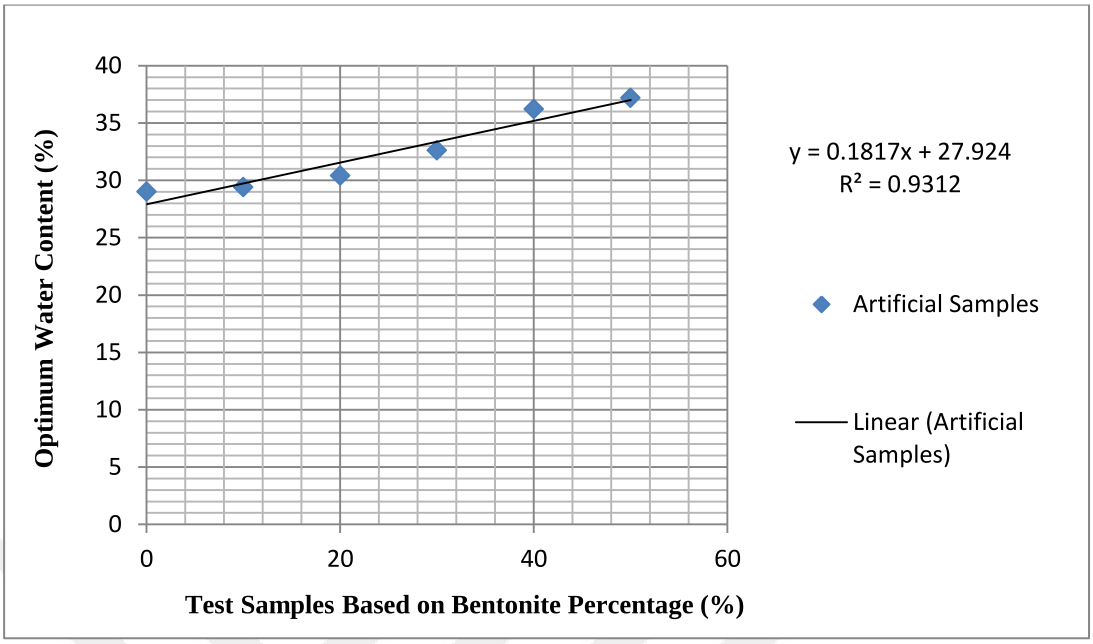

Figure 4.5 Optimum Water Content vs Test Samples

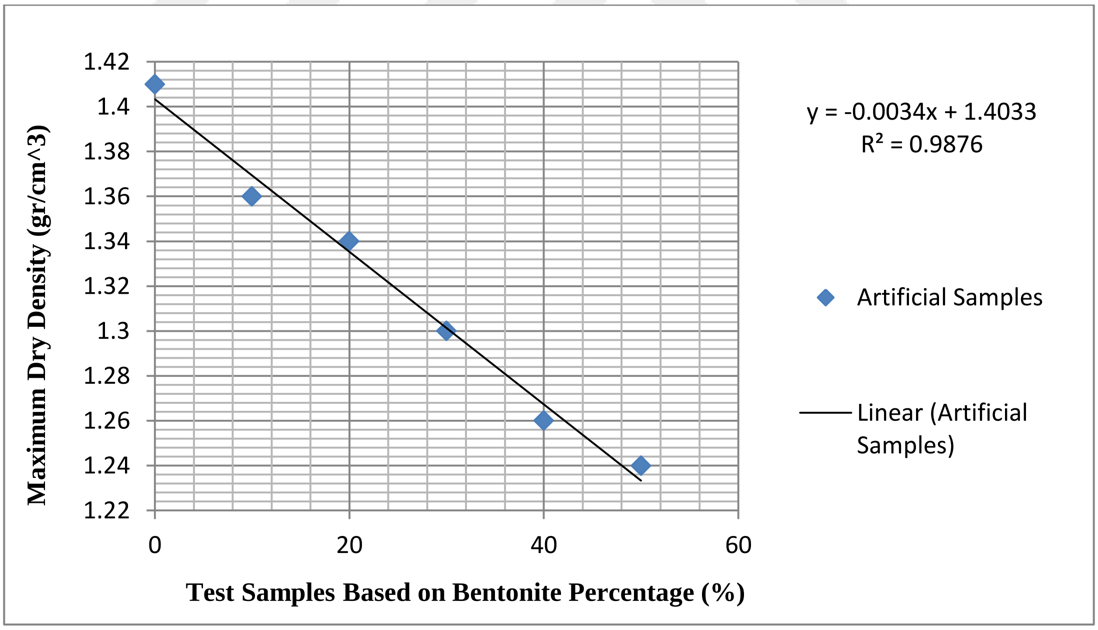

Figure 4.6 Maximum Dry Density vs Test Samples

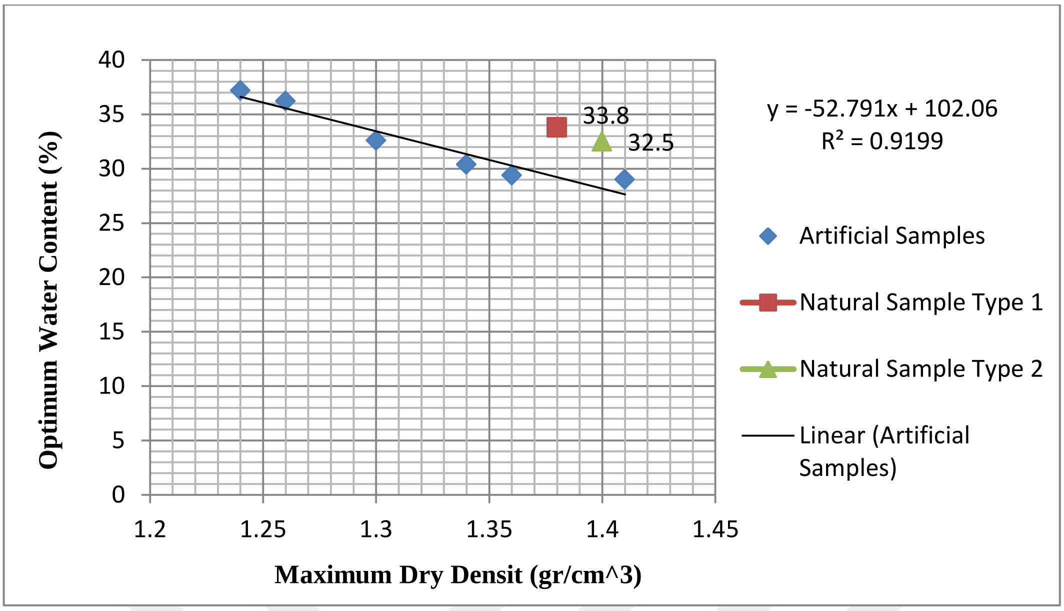

Figure 4.7 Optimum Water Content vs Maximum Dry Density

4.3 Analysis and Discussion of the Free Swelling Test Results

Determination and quantifying of swelling potential of expansive soils possess high degree of importance in the geotechnical engineering. As it was stated, the assessment of swelling behavior of such soils includes both direct and indirect measurements. The direct methods which submit more exact information about swelling parameters of expansive soils involve the physical measurements of swell potential and the pressure exerted by soil expansion. However, the formulated correlations based on fundamental properties of expansive soils are proposed as indirect methods to predict the swelling parameters of expansive soils including swell potential and swell pressure.

A number of empirical models suggested to predict swell potential are presented in table 4.1. These correlations are used to for the comparison of the equations developed by this study.

Table 4.1 Empirical correlations for predicting the swelling potential by various researchers.

Predictive Model	Author
$S_p = 7.518 + 0.323$ (Cc)	Muntohar (2000)
S_p= (2.29*10^{-2}) ($PI^{1.45}$) (Cc/W_i) + 6.38	Nayak and Christensen (1971)
S_p= (3.6*10^{-5}) (${A_c}^{2.44}$) ($Cc^{3.44}$)	Seed et al (1962)

Note:

S_p: Swell Potential (%)

Cc: Clay Content (%)

W_i: Initial Water Content (%)

A_c: Activity

There is special consideration in previous correlations to clay content and activity of expansive soil, which have a key influence on soils demonstrating swelling behavior. Multiple regressions were performed to obtain a predictive correlation between the parameters considered through this study. Various mathematical functions were employed to analyze the correlations. Statistically, the linear functions proved to be the most precise among all and regression equations were established between index properties and swelling potential and swelling pressure, also the regression between swell potential and common swell determination techniques are assessed.

In this study, the presented prediction model (model 1) incorporates the maximum dry density, optimum moisture content (as initial moisture content) and soil activity as the key parameters which affect swelling potential of test samples (Eqn 4.1). Since soil activity represents liquid limit, plastic limit and clay content of test samples, the correlation model which incorporates this factor with optimum moisture content and maximum dry density present a comprehensive prediction of swelling parameters (swelling potential and swelling pressure) based on soil index properties.

Moreover, a relatively new approach of common swell determination tests has been adopted and a correlated model (model 2) is proposed based on the results obtained from the Modified free swell index test (MFSI), Methylene blue test (MBV), Maximum dry density and optimum water content, as initial water content(Eqn 4.2).

The proposed correlations are established using Excel 2010 software at 95% confidence level.

Both correlations were proved to be statistically acceptable.

$S_p = a_1 + a_2 A_c + a_3 \gamma_d + a_4 W_i$ ....Equation 4.1 (Model 1)

$S_p = b_1 + b_2 MFSI + b_3 MBV + b_4 \gamma_d + b_5 W_i$ ....Equation 4.2 (Model 2)

Where;

S_p : Swell Potential (The ratio of the amount of swell to the original height of the test sample expressed as a percentage) (%)

A_c : Activity

γ_d : Maximum Dry Density (gr/cm^3)

W_i : Initial Water Content (Optimum water content) (%)

MFSI: Modified Free Swell Index (cm^3/gr)

MBV: Methylene Blue Value (gr/100gr)

Table 4.2 Intercepts, coefficients and regression statistics of correlation equations

Equations	Intercept	Coefficients				Regression Statistics	
Equation 4.1	a_1=1404.476	a_2=47.203	a_3=-907.305	a_4=-4.098	_	R^2=0.996	S=7.95
Equation 4.2	b_1=598.009	b_2=0.825	b_3=15.731	b_4=-335.768	b_5=-4.714	R^2=0.999	S=3.42

Note:

R^2 : R Square

S: Standard Error

According to bentonite tendency for moisture retention, with increasing percentage of bentonite a drastic increase was observed in the swell potential value of test samples due to water adsorption and subsequent volume increase (Fig 4.8). The correlations between swell potential and samples index properties such as activity, maximum dry density and optimum water content, as initial moisture content, are discussed below.

As the particle size decreases the specific surface area, the parameter controlling how much wetting is required to transfer a soil from one phase to another such as across the liquid limit or the plastic limit, increases which results in increase of attracted water to the soil surface. On the basis of this reasoning the Activity, the ratio of plasticity index to percent of clay-sized particle, is proposed as a parameter to determine the volume changes of soils when wetted and dried.

In this study the test samples with higher percentage of bentonite have higher fraction of the sizes smaller than 0.002 mm, the subsequent increased specific surface area causes more moisture adsorption and resultant higher swell potential. Hence high swell potential is expected as the activity of test samples increases (Fig 4.9). As it was stated before, the mixtures with higher bentonite percentages possess lower maximum dry density, and higher swell potential (Fig. 4.10). Basma (1995) and Rashid (2013) reported that decreasing water content of the test samples to an optimized level enhances the swelling properties (swell potential and swelling pressure) of all swelling soils. The results obtained through this study confirm the presented consequence (Fig 4.11).

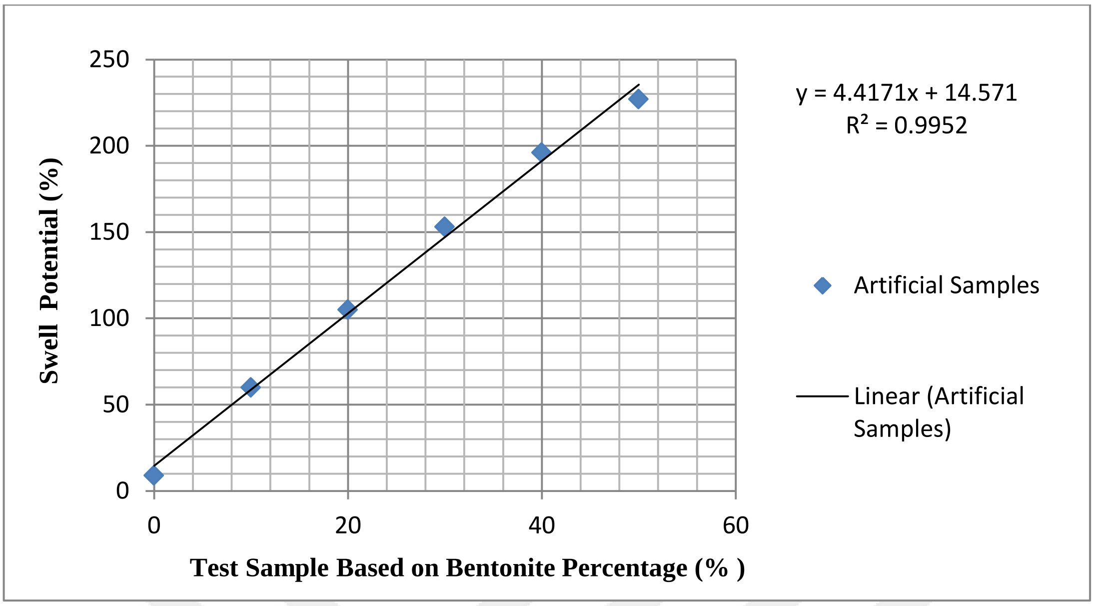

Figure 4.8 Swell Potential vs Bentonite Content

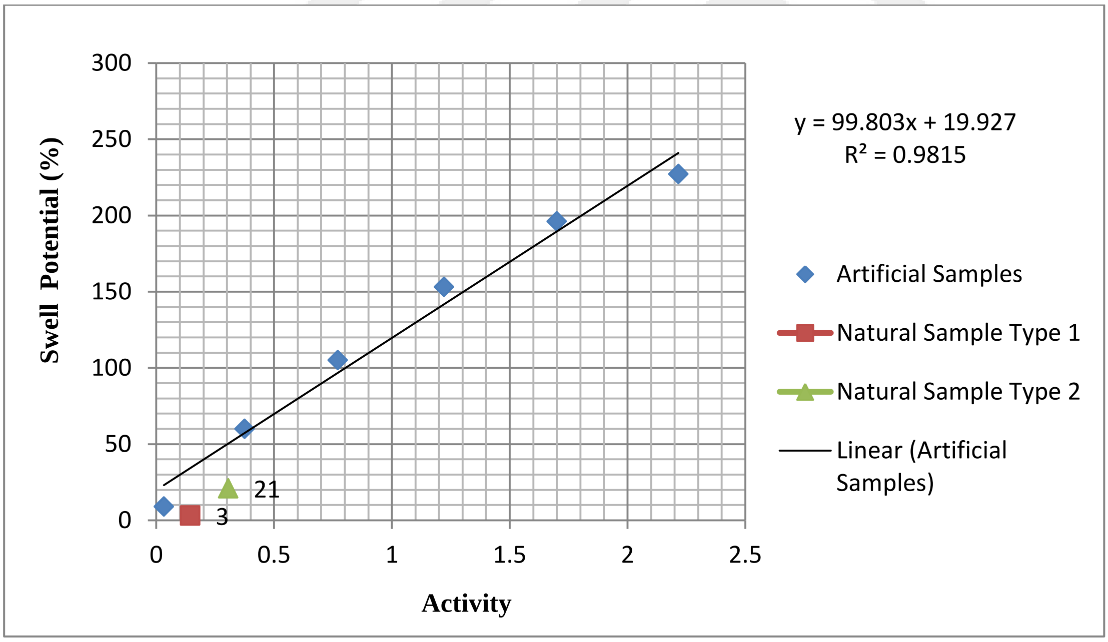

Figure 4.9 Swell Potential vs Activity

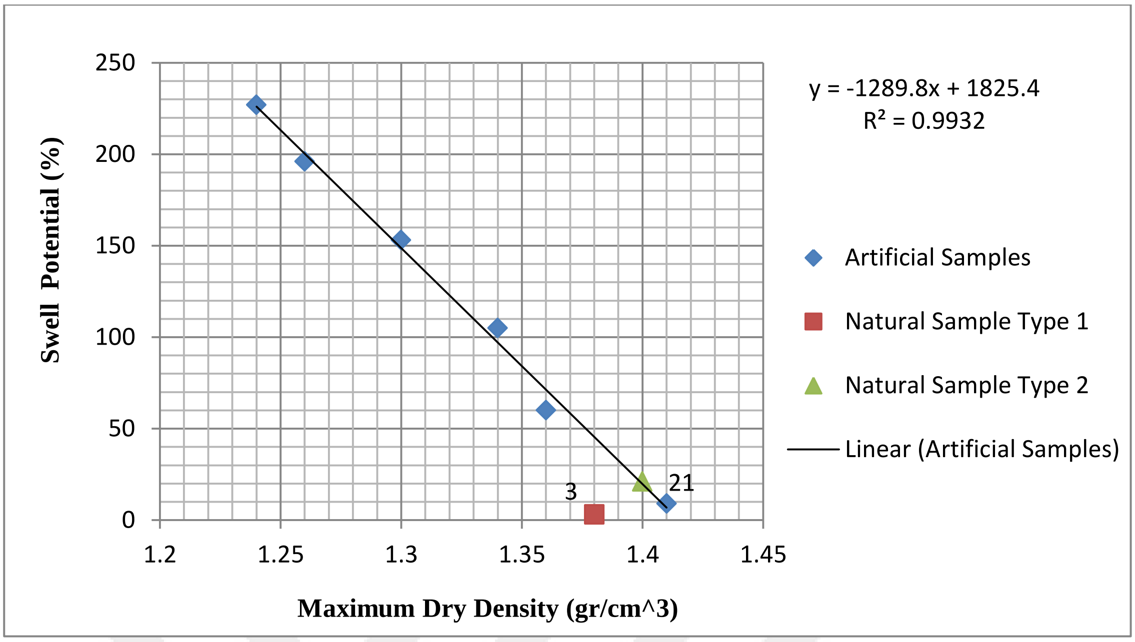

Figure 4.10 Swell Potential vs Maximum Dry Density

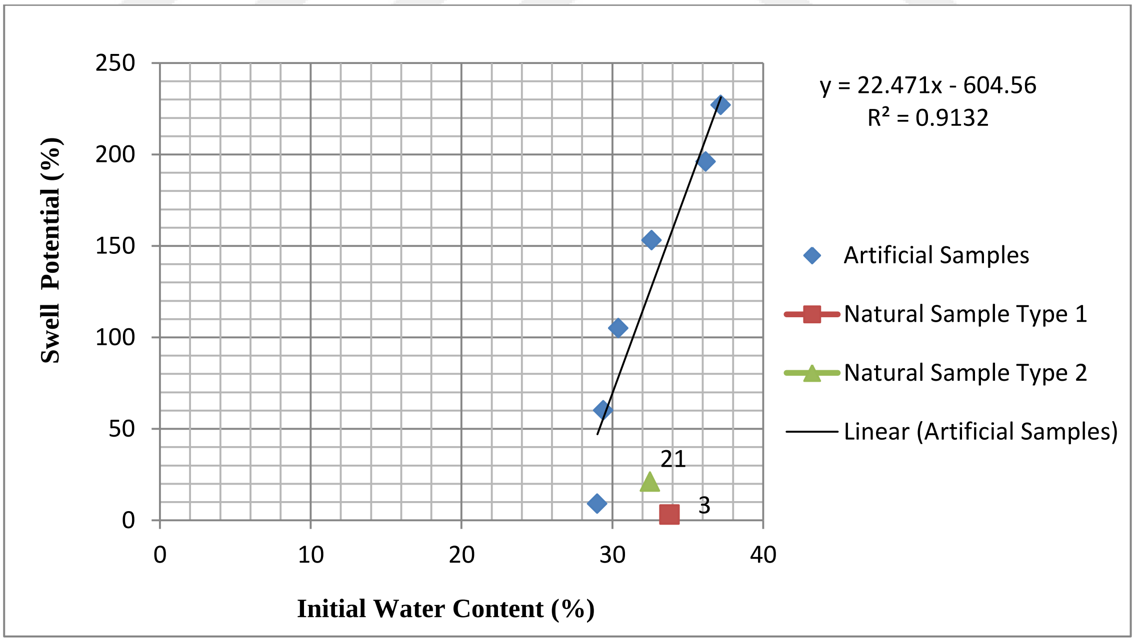

Figure 4.11 Swell Potential vs Initial Water Content

In this part of the research, the correlations between swell percent and other common swell determination tests are evaluated. The results of the performed experiments exhibit that there are direct relationships between swell potential and MFSI, MBV and swelling pressure.

As it was acknowledged in Chapter 2 Modified free swell index test is a method to assess the swell potential of soils based on the reactions between water molecules and soil particles without any external load. The increasing bentonite percentage in test samples results in more volume change due to bentonite tendency to adsorb water molecules. Thus the test samples with high percentage of bentonite possess higher MFSI (Fig 4.12).

MBV of mixtures with higher percentage of bentonite is expected to increase according to high cation exchange capacity of a bentonite. Hence, if MBV is plotted versus swell potential an ascending slope is expected with increase in swell potential due to presence of increasing bentonite percentage of test samples (Fig 4.13).

P, swelling pressure, is defined as the load per unit area needed to prevent increase in height of the test sample upon water addition. So it can be inferred that the source of exerted pressure due to volume change in test samples is water adsorption occurs via soil particle. Bentonite tendency to react with water molecules is the main reason of high swelling pressure caused by this clay. As expected Fig 4.14 demonstrates increase in swell potential as swell pressure increases.

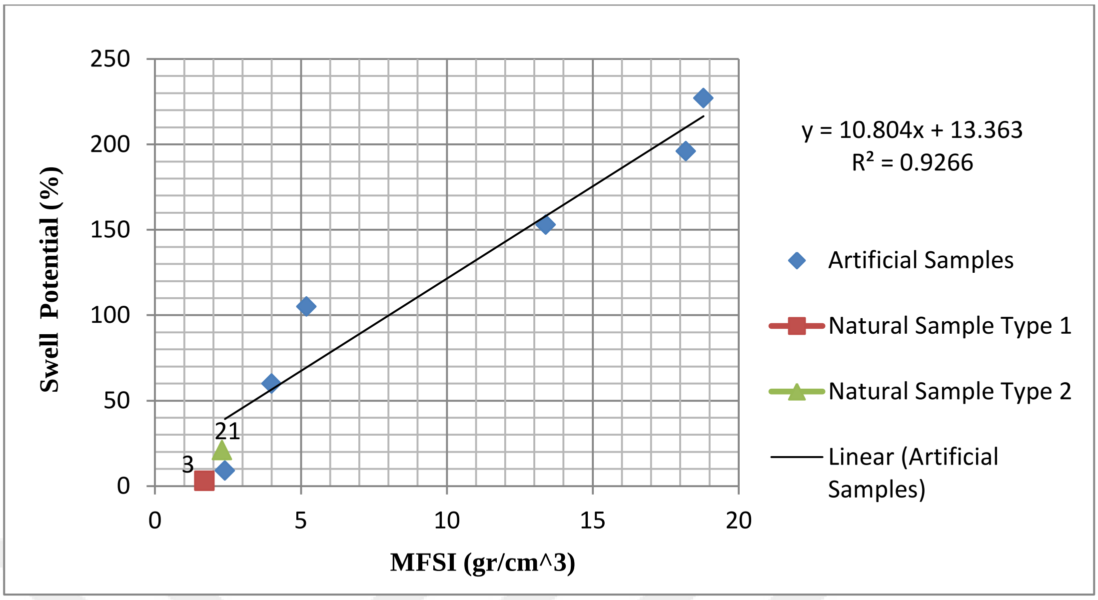

Figure 4.12 Swell Potential vs MFSI

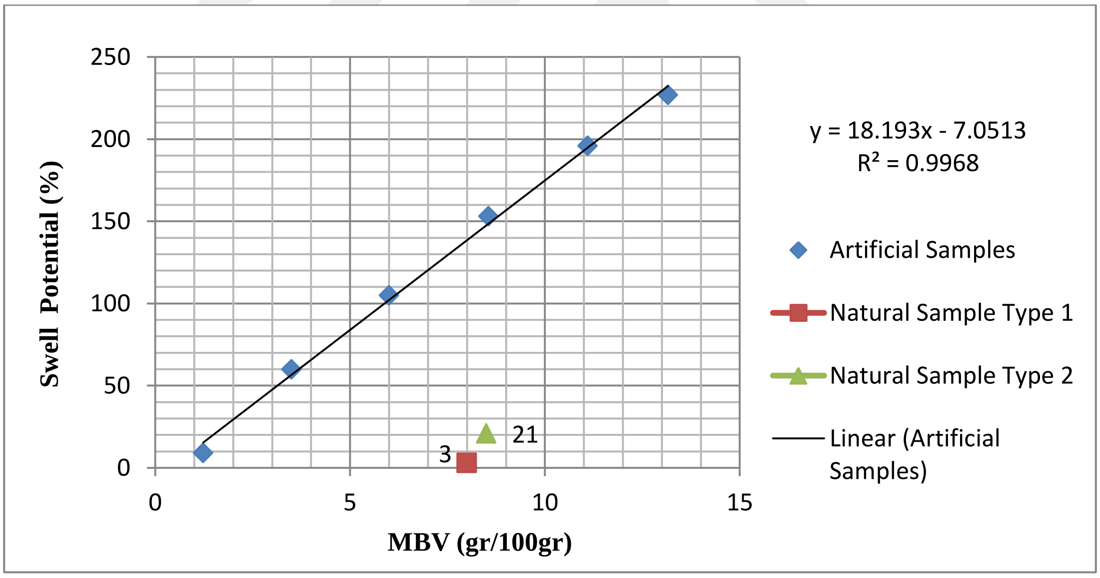

Figure 4.13 Swell Potential vs MBV

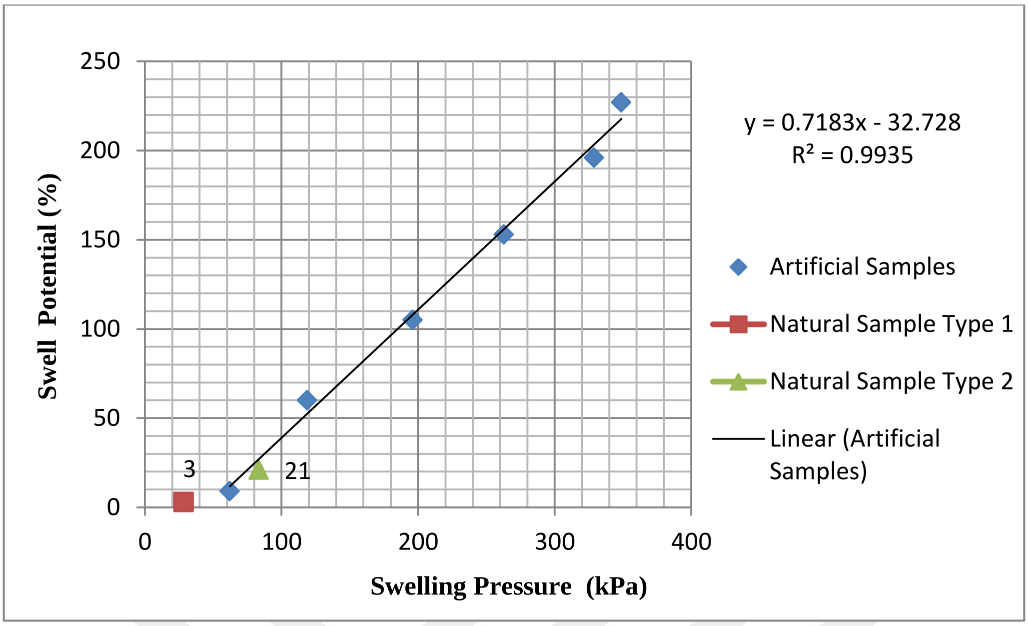

Figure 4.14 Swell Potential vs Swelling Pressure

4.3.1 Validation of the Swell Potential (S_p) Models

In this part of the study the validity of the presented correlations are considered. Experimental results of swell potential of artificial test samples are plotted against the predicted values of the same based on activity, maximum dry density and initial water content (Fig 4.15). Also, the graphical comparison between experimental and predicted results of swell potential base on MFSI, MBV, Maximum dry density and Initial water content is made (Fig 4.16).

Graphical comparisons show that the scatter of result points largely follows the trend of 1:1 line. For both models, a conservative prediction is revealed when the predictive values are compared with the experimental values of the same. Conclusively a good correlation between predictive and experimental results acknowledges the suitability of linear correlation in the regression analysis for the current test results. According to the predictive models the correlation is made based upon one constant and three variables, whereas, the swelling behavior of test samples are controlled by many variables in actual, accordingly, deviation of some of data points from 1:1 line is justified.

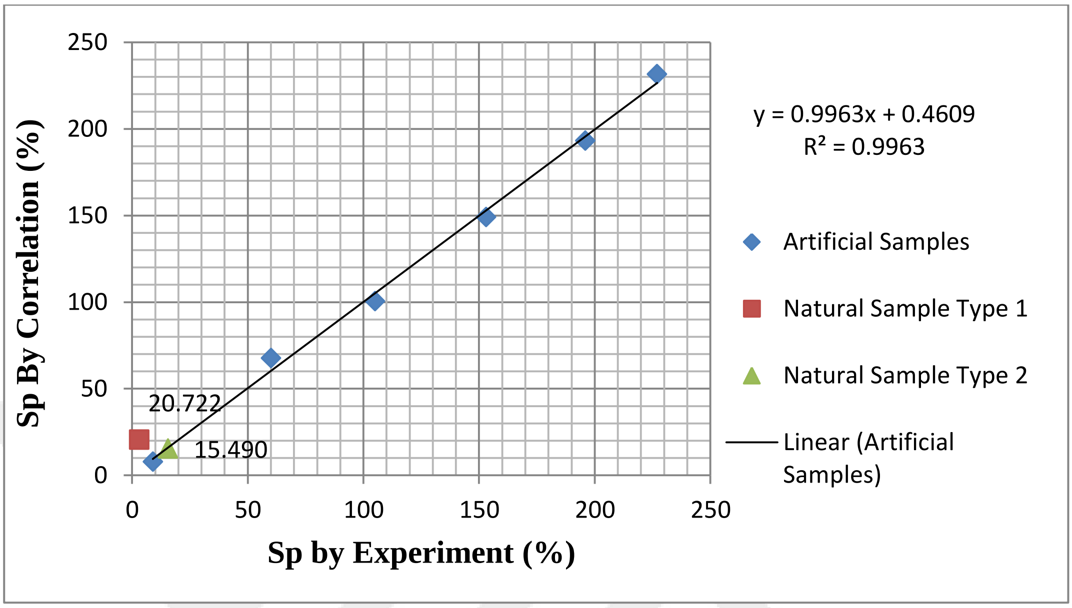

Figure 4.15 Comparison between experimental and predicted values of swell potential from model 1

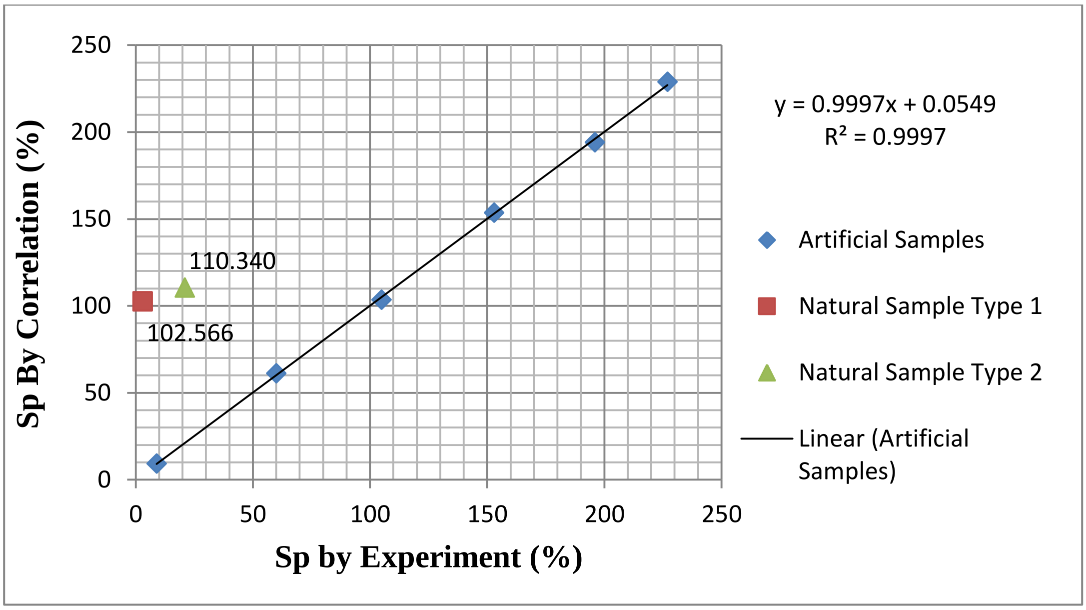

Figure 4.16 Comparison between experimental and predicted values of swell potential from model 2

Figure 4.17 shows the comparison between the results of swell potential obtained from currently proposed correlation and the results of swell potential from the predictive models proposed by Seed et all (1962), Muntohar (2000) and Nayak and Christesen (1970). Graphical comparisons demonstrate a better correlation between experimental values and the predicted values of the model developed in this study than those from the correlations proposed by other researchers. It can be determined that the incorporation of maximum dry density, beside activity and initial moisture content, as variables in currently proposed correlated model has increased the precision of prediction. The scatter of the data points plotted based on other proposed correlations proves that the previous models largely underestimate the swell potential.

Additionally, the results of swell potential from the experimental investigation are compared with the results of the both suggested correlations simultaneously in Figure 4.18.The scatter of the data points plotted by the both of currently proposed models, not only shows a good correlation with the experimental values, but also, represents so small discrepancies between themselves.

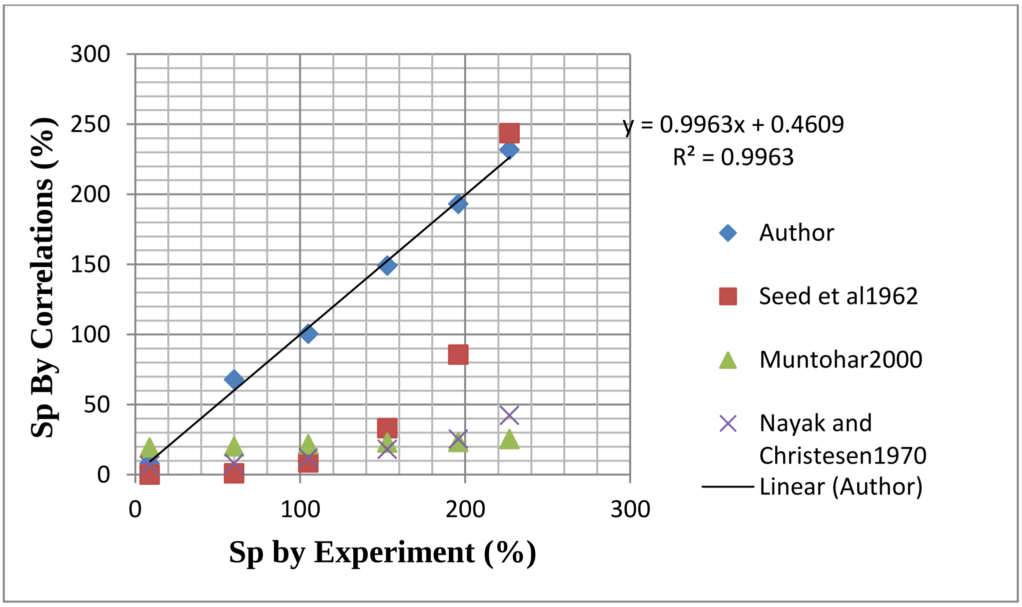

Figure 4.17 Comparisons of experimental and predicted values of swell potential from various models

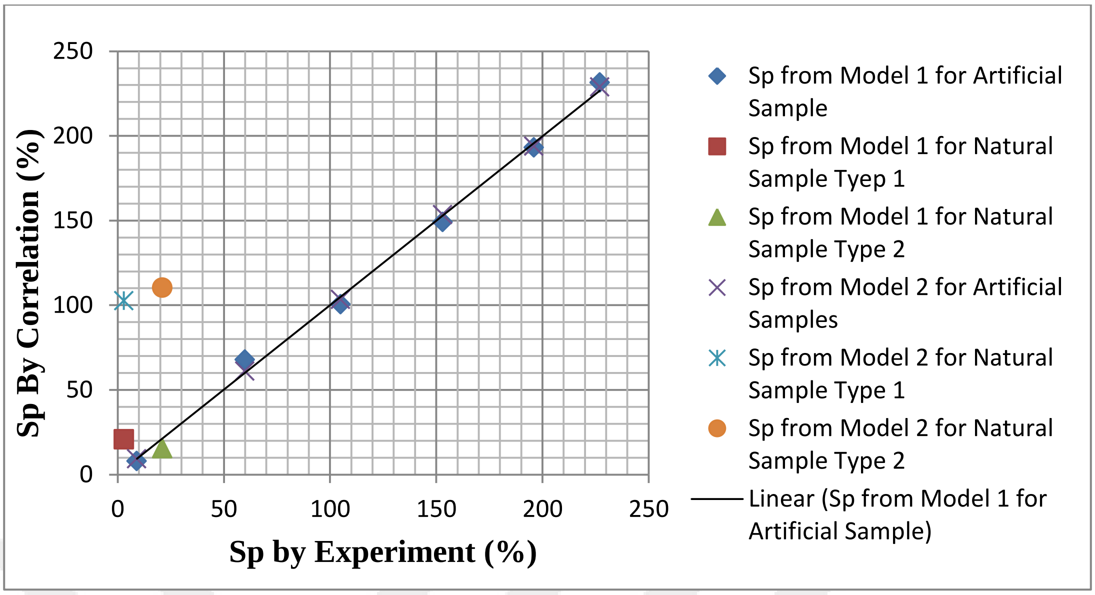

Figure 4.18 Comparisons of experimental and predicted values of swell potential from both currently proposed models

4.4 Analysis and Discussion of the Swelling Pressure (P) Test Results

The tremendous amount of pressure exerted by high plastic clays while swelling is source of damages to light weight engineering structure such as shallow foundations and pavements. In swelling soil studies determination and quantifying of swelling pressure are a challenge to geotechnical engineers who design substructure upon this type of soil. Similar to evaluation of swell potential, there are direct and indirect techniques to assess the pressure exerted by expansive soil in the case of access to water.

In spite of few direct methods to measure swelling pressure, one dimensional consolidometer method is the most common method. However, geotechnical designers utilize a number of empirical models, as indirect method, to predict the swelling pressure. Extensive experimental investigation by Komornik and David (1969) prepared on a number of undisturbed natural soil samples to develop a predictive model for swelling pressure based on statistical analysis (Eqn 4.3).

$$\text{Log (P)} = 2.132 + 0.0208\ (LL) + 0.000665\ (\gamma_d) - 0.0269\ (W_i) \ldots\ldots \text{Equation 4.3}$$

Note:

LL: Liquid Limit (%)

γ_d : Dry Density of soil Samples (kg/m^3)

W_i : Water Content (%)

As was stated earlier a number of parameters influence swelling behavior of expansive soils. In this study several tests were conducted to investigate the contribution of initial compaction degree, initial moisture content and clay activity to the swelling pressure. In addition, the relation between swelling pressure and other swell determination techniques are explored. Additionally, an empirical predictive model (model 3) is proposed to predict the swelling pressure based on index properties of soils, moreover, other predictive model is developed based on MFSI, MBV, γ_d and W_i (model 4) (Eqn 4.4; Eqn 4.5).

$$\text{Log(P)} = c_1 + c_2 A_c + c_3 \gamma_d + c_4 W_i \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{Equation 4.4 (Model 3)}$$

$$\text{Log(P)} = d_1 + d_2 MFSI + d_3 MBV + d_4 \gamma_d + d_5 W_i \ldots\ldots\ldots \text{Equation 4.5 (Model 4)}$$

Where;

P: Swelling Pressure (The load per unit area needed to prevent increase in height of the test sample upon water addition) (%)

A_c : Activity

γ_d : Maximum Dry Density (g/cm^3)

W_i : Initial Water Content (Optimum water content) (%)

MFSI: Modified Free Swell Index (cm3/gr)

MBV: Methylene Blue Value (gr/100gr)

Table 4.3 Intercepts, coefficients and regression statistics of correlation equations

Equations	Intercept	Coefficients				Regression Statistics	
Equation 4.4	c_1=14.155	c_2=0.021	c_3=-7.469	c_{4}=-0.063	_	R^2=0.975	S=0.073
Equation 4.5	d_1=9.425	d_2=0.007	d_3=0.062	d_4=-3.718	d_5=-0.085	R^2=0.984	S=0.083

Note:

R^2: R Square

S: Standard Error

The analysis of experimental results are presented in series of plots providing a comprehensive understanding of the swelling pressure of the selected expansive test samples in this research. The swelling pressure and its relationship to bentonite percentage and index properties of test samples can be observed from Figure 4.19 to Figure 4.22.

The importance of the bentonite content of test samples on the swelling pressure exerted by test samples determined by zero swell test is shown in Figure 4.19. All specimens are compacted with an initial water content equal to optimum water content at maximum dry density. As it was expected, the swelling pressure increases as the bentonite percentage of test samples increase due to water adsorption of the bentonite particles which results in samples volume changes.

The effect of activity on swelling pressure is presented in figure 4.20. As it was explained before, Activity is referred as a parameter to specify the volume changes of soils in the case of moisture accessibility. The observation approves that the swelling pressure increases with the increase in activity. Figure 4.21 depicts the swelling pressure results for various maximum dry densities. In all cases, the associated decrease in swelling pressure is observed with increase in maximum dry densities of the different specimens with increasing bentonite content. Similar to swelling potential, the swelling pressure shows a tendency to increase with enhancement in initial water content of the different specimens with increasing bentonite content, which is equal to maximum dry density (Fig 4.22).

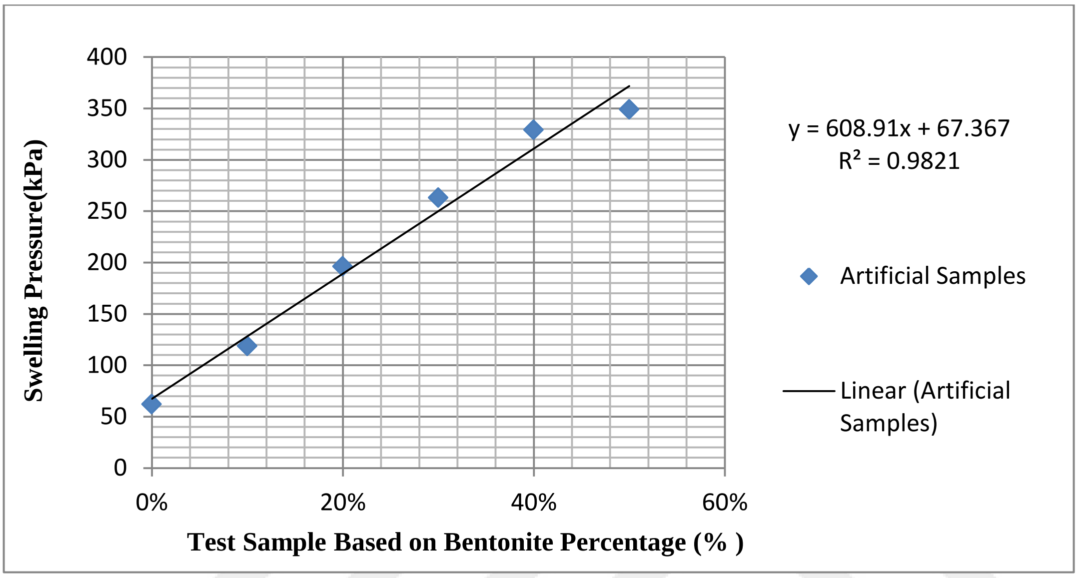

Figure 4.19 Swelling Pressure vs Bentonite Content

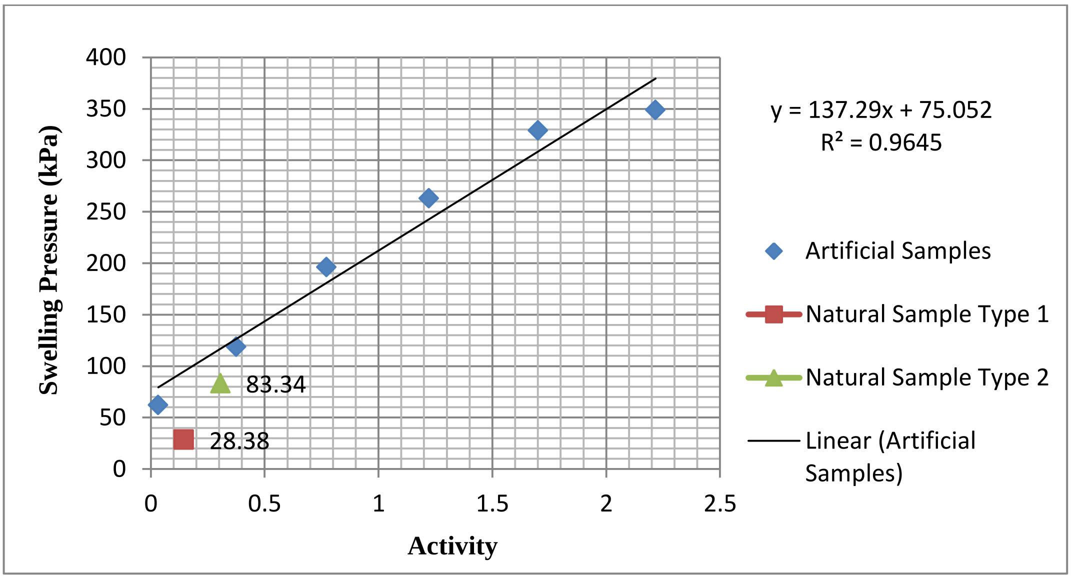

Figure 4.20 Swell Pressure vs Activity

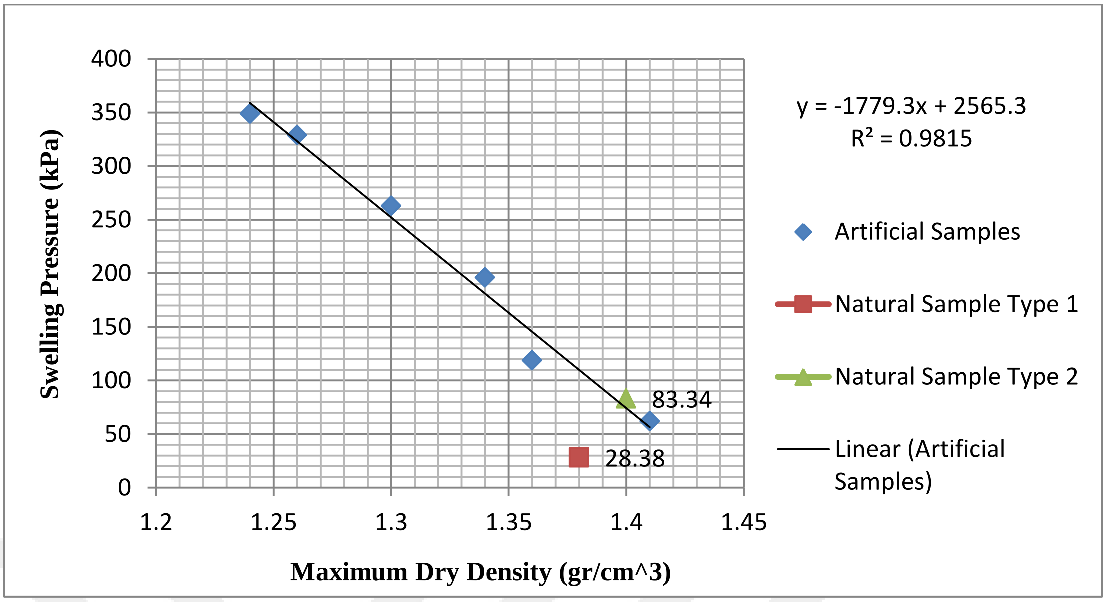

Figure 4.21 Swelling Pressure vs Maximum Dry Density

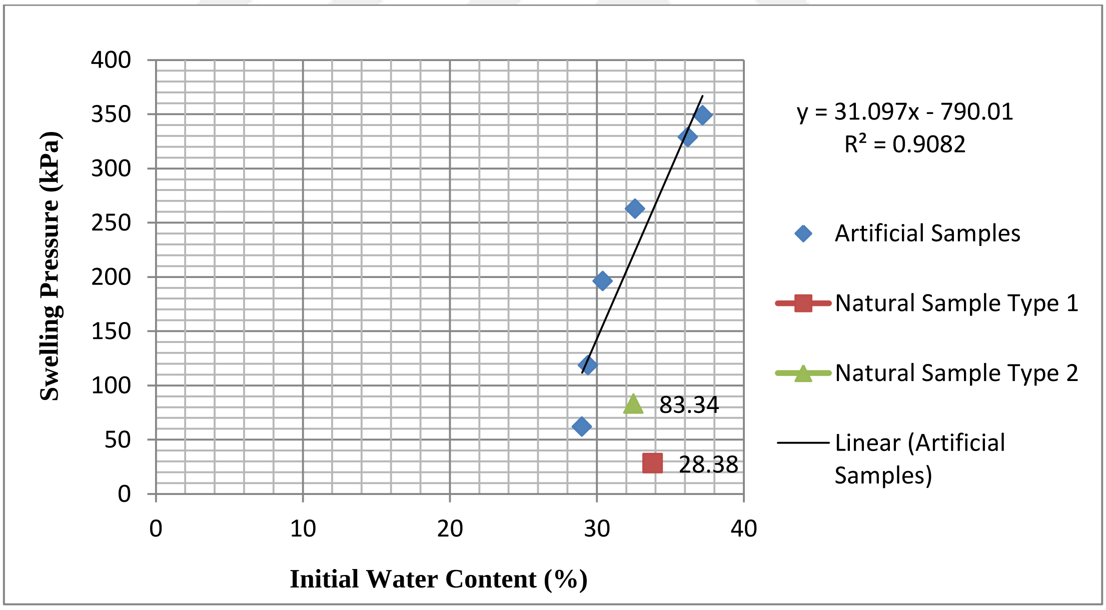

Figure 4.22 Swelling Pressure vs Initial Water Content

Variations of swelling pressure according to MFSI, MBV and Sp are assessed in Figures 4.23; 4.24 and 4.25 respectively. As can be noted in test samples with higher bentonite percentage, there are direct correlations between swelling pressure and

MFSI, MBV and Sp. Based on the earlier interpretation higher cation exchange capacity and consequently more volume change can be observed in test samples with increase in bentonite content. Reasonably, incremental slope is expected in the graphical assessments of the correlations between swelling pressure and MFSI, MBV and Sp.

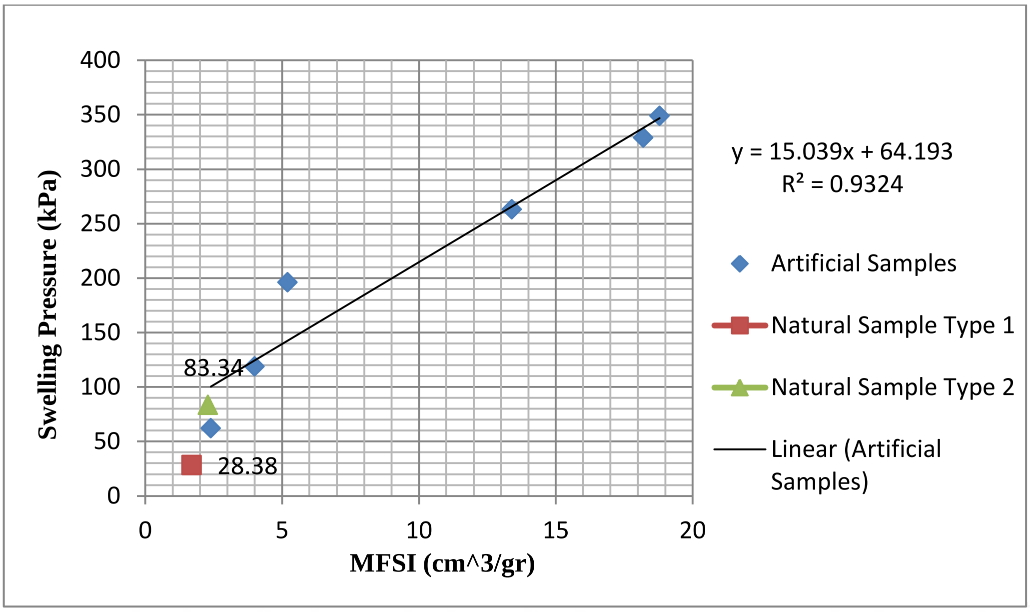

Figure 4.23 Swelling Pressure vs MFSI

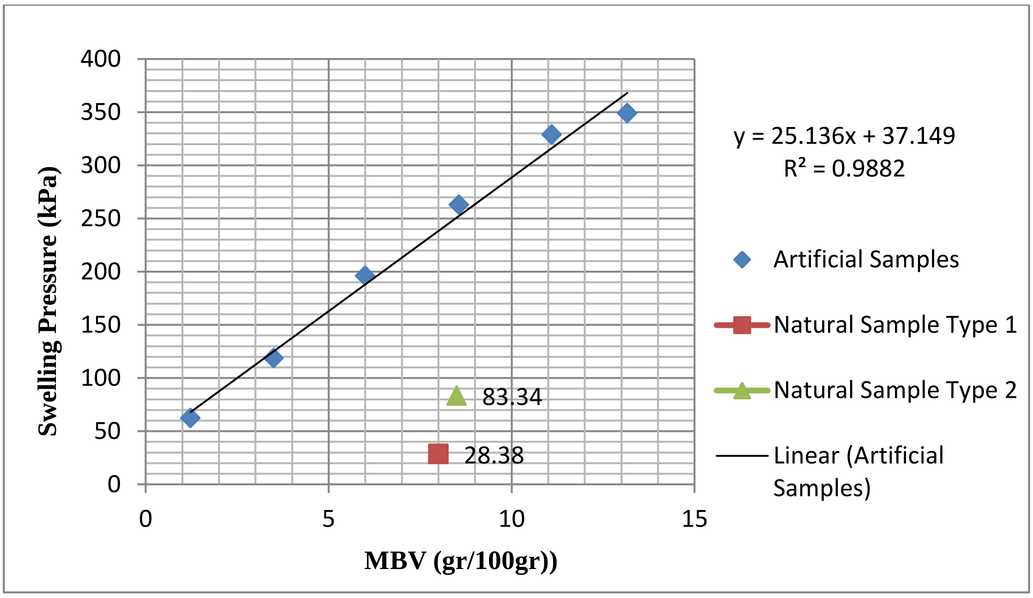

Figure 4.24 Swelling Pressure VS MBV

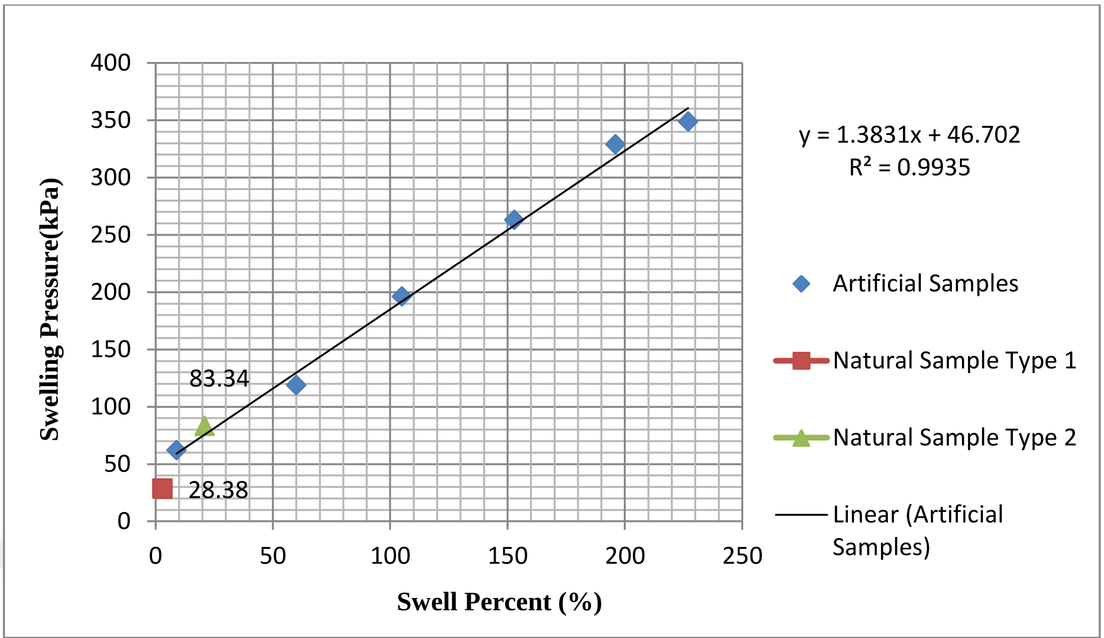

Figure 4.25 Swelling Pressure vs Swell Potential

4.4.1 Validation of the Swelling Pressure Models

In the face of the complicated behavior of expansive soil and the multiple parameters that influence it, the ultimate aim would seem to evaluate the validation of the models proposed to predict the swelling pressure of test samples. The validity of the proposed correlations is considered by comparing the experimental values of swelling pressure tests and the results obtained from predictive models (Fig 4.26; 4.27).

As can be noted from Figures 4.26 and 4.27, plotted data points which are so close to 1:1 line show that the predictive models proposed in this study are capable of estimating swelling pressure with acceptable accuracy.

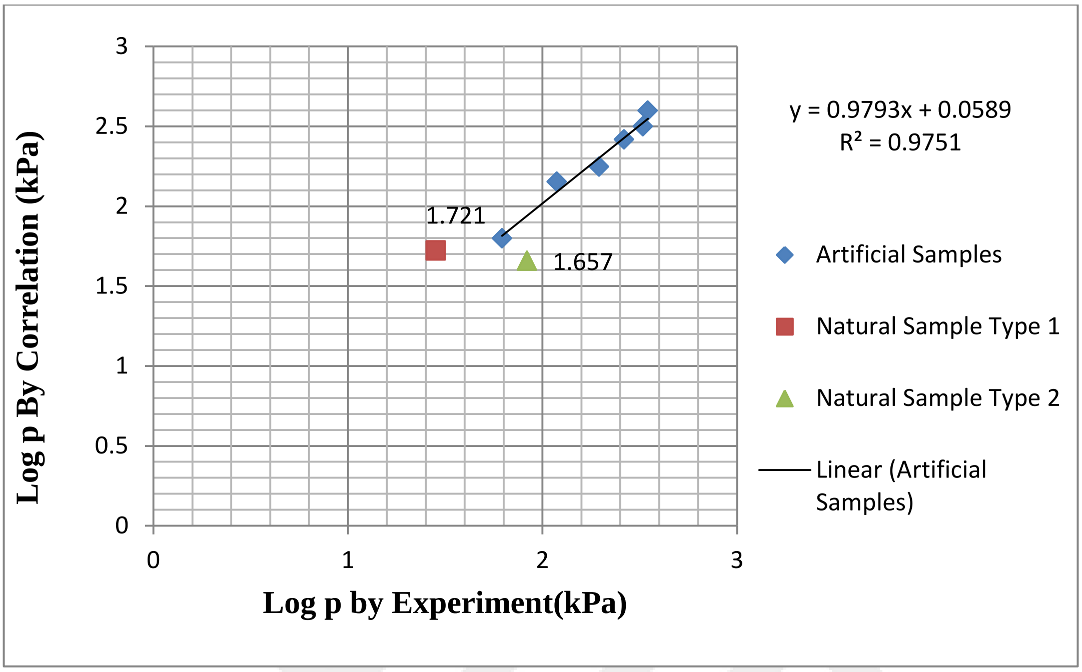

Figure 4.26 Comparison between experimental and predicted values of swelling pressure from model 3

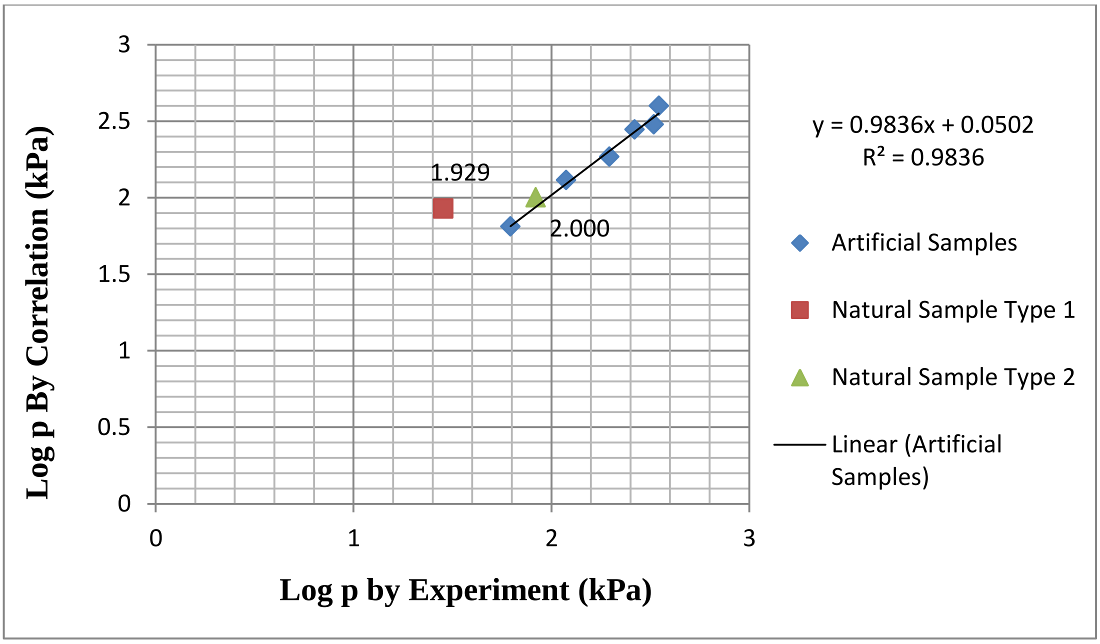

Figure 4.27 Comparison between experimental and predicted values of swelling pressure from model 4

Graphical comparisons between the predicted values of swell potential from the correlation proposed in this study and the predictive model proposed by Komornik and David (1969) depict that, there is a better correlation between experimental swelling pressure results and those obtained from currently proposed predictive model, unlike previously proposed model by Komornik and David (4.28). In addition, the values obtained from both of the swelling pressure predictive models are compared with each other. The scatter of the data points exhibits the results based on currently proposed correlations are in a good coordination with small discrepancies (4.29).

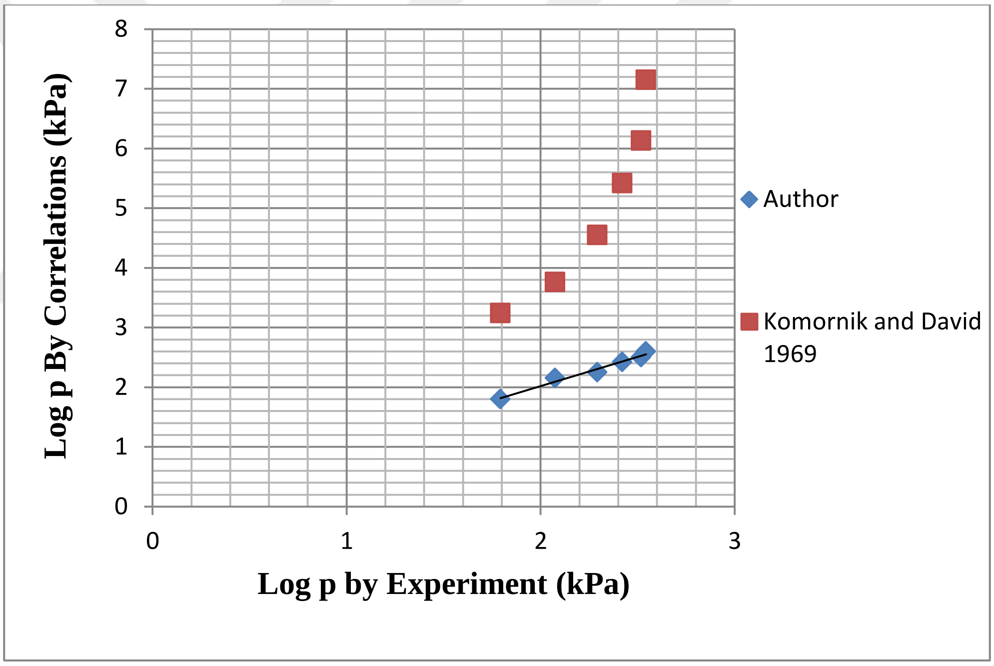

Figure 4.28 Comparisons of experimental and predicted values of swelling pressure from various models

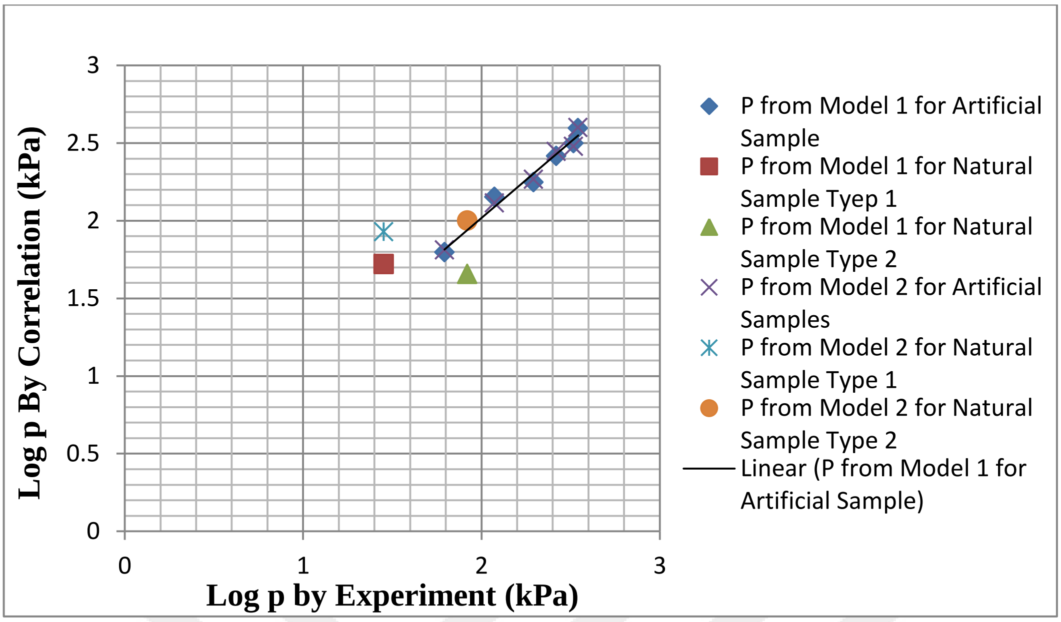

Figure 4.29 Comparisons of experimental and predicted values of swelling pressure from both proposed models

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary of Research and Contribution

Aim of this research was to quantitatively investigate the swelling mechanism of expansive soil. In particular, the swelling behavior of test samples was studied by means of two measurements; swelling potential and swelling pressure. Two factors had been controlled to explore the swelling pressure and swelling potential during the current study, maximum dry density and optimum water content, as initial water content, of test samples.

The experimental investigation in current research had two parts one part to measure the fundamental characteristics of test samples and another part consisting a series of techniques such as Methylene blue test, Modified free swell index test, One-dimensional free swell test and swelling pressure test to assess swelling behavior, swell potential and swelling pressure, of test samples respectively.

Moreover, the experimental results are analyzed and correlations are developed and the obtained values based on experimental investigation and currently proposed predictive models are compared with the same based on previously suggested predictive models.

5.2 Conclusions

- Experimental results in current study indicate that in artificial test samples the clay content, liquid limit and plastic limit increase in samples with higher bentonite percentage. Subsequently, there is a significant increase in plasticity index and activity of test samples with higher bentonite percentage.
- Unlike optimum water content, maximum dry density of artificial test samples decrease while adding bentonite to kaolinite. Experimentally obtained results indicate decline in shrinkage values of test samples with increase in bentonite percentage. The measured values of specific gravity indicate that with increase in bentonite percentage of artificial test samples, specific gravity values decrease.
- The obtained experimental results indicate that as the bentonite percentages of test samples increase, swell potential and swelling pressure of test samples increase. As expected, it is observed that with the addition of bentonite in the kaolinite-bentonite mixtures swelling potential (S_p) and swelling pressure (P) of mixtures increase significantly. Additionally, it is noted that MBV and MFSI increase with bentonite percentage in artificial test samples.
- As the main aim of the current study, formulated correlations are developed based on artificial test samples as indirect methods to predict swelling parameters of test samples including swell potential and swelling pressure. The accuracy of the presented predictive models are considered by graphical comparison between values obtained from experiments and those based on currently proposed models and good correlations are observed between them.
- It can be inferred that the incorporation of maximum dry density beside activity and optimum water content as initial moisture content in developed models has enhanced the precision of prediction.
- Current study prove the key influence of particles mineralogy on their fundamental properties such as Atterberg limits, maximum dry density, optimum water content, clay content and activity. Additionally, mineralogy

of tests samples, especially in natural test samples, has key role in determination of their swelling behavior, swell potential and swelling pressure. The currently developed predictive models based on artificial test samples, with different mineralogy form natural test samples, does not predict swelling behavior of natural test samples with high accuracy, which proves influence of the samples mineralogy on swelling behavior of them.

5.3 Recommendations for Future Study

Some of subjects and suggestions that could be pursued in the future research:

- It should be considered that the current study focused mainly on mixtures of bentonite and kaolinite in different percentages. The equations developed through this study cannot be used for all specimens. Since the values according to which predictive models are developed pertain to artificial test samples, it is suggested to use more natural test samples with different degree of swell potential to develop other correlations which can predict swelling behavior of natural soils, swell potential and swelling pressure, more exactly.
- In the current study, two predictive models are developed based on some of the fundamental properties of test samples and the other two predictive models are based on swell determination tests (MBT, MFSI and Free swell percent) maximum dry density and initial water content. Observations depict that to develop more precise correlations, especially for natural samples, it is recommended to determine their mineralogy and develop predictive models that incorporate mineralogical properties of test samples.
- The experimental data obtained in this research can be used to obtain parameters for future analytical or numerical modeling.
- As final conclusion, four predictive models are developed (Models 1,2,3,4)

$S_p = a_1 + a_2A_c + a_3\gamma_d + a_4W_i$ …………………… (Model 1)

$S_p = b_1 + b_2MFSI + b_3MBV + b_4\gamma_d + b_5W_i$ .... (Model 2)

$Log(P) = c_1 + c_2A_c + c_3\gamma_d + c_4W_i$ …………………… (Model 3)

$Log(P) = d_1 + d_2MFSI + d_3MBV + d_4\gamma_d + d_5W_i$ .... (Model 4)

REFERENCES

Aecl. *Environmental impact statement on the concept for disposal of Canada's nuclear fuel waste.* Atomic Energy of Canada Limited Report, AECL-10711, COG-93-1. 1994.

AFNOR. *Mesure de la quantité et de l'activité de la fraction argileuse (Norme Française NF pp. 68-94).* Association Française de Normalization (ANFOR), La Défense, Paris, France. 1993.

AFNOR (L'Association Francaise De Normalisation), *Essai au bleu de methylene.* 18–592, AFNOR 80181, Paris La Defence. 1980.

Altmeyerw. T. *Discussion on engineering properties of expansive soils.* Transactions of ASCE, 121. 1956.

ASTM, *Standard Test Method for Expansion Index of Soils.* Annual Book of ASTM Standards, PA 4829: 4-8. 1988.

ASTM, *Standard Test Method for Expansion Index of Soils.* Annual Book of ASTM Standards, PA 4546: 2013.

ASTM, *Standard Test Method for Laboratory Compaction Characteristics of Soil Using Standard Effort,* Annual Book of ASTM Standards, D698 - 12, 2012.

ASTM, *Standard Test Method for Linear Shrinkage of Preformed High-Temperature Thermal Insulation Subjected to Soaking Heat,* Annual Book of ASTM Standards, C356-10, 2010.

ASTM, *Standard Test Method for Liquid Limit, Plastic Limit and Plasticity Index of Soils,* Annual Book of ASTM Standards, D4318, 2010.

ASTM, *Standard Test Method for Particle-Size Analysis of Soils,* Annual Book of ASTM Standards, D422, 2007.

ASTM, *Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils,* Annual Book of ASTM Standards, D4546 – 08, Vol. 04 -08, pp. 733-738, 2008.

Attom, M. F., & Barakat, S. (2000, August).*Investigation of three methods for evaluating swelling pressure of soils.* Environmental & Engineering Geoscience, pp. 293-299.

Basma, A. A., Al-Homoud, A. S., and Malkawi, A. H. *"Laboratory assessment of swelling pressure of expansive soils,"* Applied Clay Science, 9, 355-368. 1995.

Basma, A. A. *Cyclic Swelling behavior of clays,* Geotechnical Engineering Journal, 121-7, 562-565. 1995.

BS, *Methods of Test for Civil Engineering Purposes Classification Tests,* British Standard Institution, BS1377, 2010.

Bureau of Iındıan standards; *Indian Standard Classi cation and Identi cation of Soils for General Engineering Purposes.* BIS, NewDelhi, (Rearmed 1987), IS 1498. 1970

Carter M. and Bentley S.P., *Correlations of Soil Properties,* (London: Pentech Press, UK). 1991.

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

Chen, F. H., *Development in Geotechnical Engineering 12: Foundations on Expansive Soils*. The Netherlands: Elsevier Scientific. 1975.

Chen, F. H., *Foundation on swelling soils,* Development on Geotechnical Engineering 12. Elsevier Scientific Co, Amsterdam, 150-158. 1975.

Chiappone A, Marello S, Scavia C, Setti M., *Clay mineral characterization through the methylene blue test: comparison with other experimental techniques and applications of the method.* Can. Geotech. J., 41: 1168-1178. 2004.

Craig, R. F., *Craig's Soil Mechanics*, 7th Edition, Spon Press, London, 2004.

Çokca E., Birand AA. *Determination of cation exchange capacity of clayey soils by the methylene blue test.* Geotech. Testing J., 16 (4): 518-524. (1993b).

Çokca E., Birand AA. *Prediction of swelling potential of Ankara soils by methylene blue test.* Doğa-Turkish J. Eng. Environ. Sci., 17: 57-63. (1993a).

Çokça E., *Swelling potential of expansive soils with a critical appraisal of the identification of swelling of Ankara soils by methylene blue tests.* PhD. Thesis, Department of Civil Engineering, Middle East Technical University, Ankara, Turkey, p. 246. 1991.

Çokça, E., *Relationship between methylene blue value, initial soil suction and swell percent of expansive soils.* Turkish J. Eng. Environ. Sci., 26: 521-530. 2002.

Çokça, E., and Birand, A., *“Determination of Cation Exchange Capacity of Clayey Soils by the Methylene Blue Test”,* Geotechnical Testing Journal, GTJODJ, 16, 4, 518-524, 1993.

Çokça, E., *Use of Class C Fly Ashes for the Stabilization of an Expansive Soil,* Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, No. 7, pp. 568-573, 2001.

Das, B. M., *Principles of Geotechnical Engineering. Stamford,* CT. Cengage Learning. 2009.

Das, B. M., *Advanced Soil Mechanics,* 3rd Edition, CRC Press, New York, 2008.

Das, B. M., *Principles of Foundation Engineering,* 4th Edition, PWS Publishing, 1999.

Dawson, R. F., *“Movement of small houses erected on an expansive clay soil,”* Proc. 3 rd Int. Conf. Soil Mech. Found. Eng., 1:346-350. 1953.

Deer, W.A., Howie, R.A., Zussman, J., *An introduction to the rock-forming minerals (2 ed.).* Harlow: Longman. 1992.

El-Sohby, M. A., and El-Sayed, A. R., *“Some Factors Affecting Swelling Of Clayey Soils,”* Geotechnical Engineering, Vol. 12, 19-39. 1981.

Ergüler, Z.A. and Ulusay, R., *A simple test and predictive models for assessing swelling potential of Ankara (Turkey) Clay*. Engineering Geology, 76, 331–352. 2003.

Fattom, M., and Barakat, S., *“Investigation of Three methods for Evaluating Swelling pressure of Soils,”* Environmental & Engineering Geoscience, Vol. VI, No. 3, pp. 293-299. 2000.

Graham, J., Chandler, N. A., Dixon, D. A., Roach, P. J., To, T., & Wan, A. W. L., *The Buffer/Container Experiment: Results, synthesis, issue*. Report AECL-11746, COG-97-46-1.Atomic Energy Canada Limited, Ontario. 1997.

Güngör, A. G., *Bursa Yöresi Şişebilen Kil Zeminlerin Yol Dolgularında Kullanılması Amacı ile Uçucu Kül ile Islahı,* M.S Thesis, Gazi University, FBE, 2002.

Hills JF, Pettifer GS., *The clay mineral content of various rock types compared with the methylene blue value.* J. Chem. Tech. Biotechnol., 35A: 168-180. 1985.

Holtz, R. D., & Kovacs, W. D., & Sheahan, T. C., *An Introduction to Geotechnical Engineering,* 2nd Edition, Pearson Education, Michigan, 2011.

Holtz, W. G. and Gıbbs H. J. *Engineering properties of expansive clays.* Transactions of ASCE, 121, 641-663. 1956.

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

Holtz, W.G., & Kovacs, W. G., *An Introduction to Geotechnical Engineering.* Englewood Cliffs, NJ: Prentice-Hall. 1981.

Hosterman, J.W. and S.H. Patterson., *Bentonite and Fuller's earth resources of the United States.* U.S. Geological Survey Professional Paper 1522. United States Government Printing Office, Washington D.C., USA. 1992.

http://geosurvey.state.co.us/hazards/Swelling%20Soils/Pages/Definition.aspx [January, 2016]

http://www.basementsystems.ca/foundation-repair/wall-cracks-repair/settlement [January, 2016]

Israr, J., *Development of correlations between index properties and swelling characteristics of swelling soils,* M.Sc. Thesis, Department of Civil Engineering, University of Engineering and Technology Lahore, Pakistan. 2012.

Jennings, J. E. B., and Knight, K., "*The Predıctıon Of Total Heave From The Double Oedomoter Test,"* Proc. Symp. Expansive Clays (South Africa Inst. Of Civil Engineers, Johannesburg), Vol. 7, No. 9, pp. 13-19. 1957.

Jones, D. E., & Holtz, W. G., *Expansive Soils - The Hidden Disaster,* National Emergency Training Center, 1973.

Keijzer, T. J., *Chemical Osmosis in Natural Clayey Materials*, Faculty of Earth Sciences, Utrecht, 2000.

Koerner, R. M., *Designing with Geosynthetics.* NJ: Prentice Hall. 1998.

Komine, H., and Ogata, N., *"Experimental study on swelling characteristics of compacted bentonite,"* Can. Geotech. J, 31, 478-490. 1994.

Komornik, A. and David, D., *Prediction of swelling pressure of clays:* J. ASCE, *Soil Mechanics and Foundation Division,* SM No. 1, pp. 209-225. 1969.

Lambe, T.W. & Withman R. V., *Soil Mechanics,* John Wiley, New York. 1969.

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

Mishra. A. K., Dhawan S., Rao. S. M., *Analysis of Swelling and Shrinkage Behavior of Compacted Clays*, Geotechnical and Geological Engineering. Vol.26. No. 3. pp. 289-298. 2008.

Mitchell, J. K., & Soga, K., *Fundamentals of Soil Behavior,* 3rd Edition, John Wiley & Sons, New York, 2005.

Muntohar, A. S., *2Prediction and classification of swelling clay soil. (eds),* Swelling soils recent advances in characterization and treatment, London, 25-36. 2000.

Murat Türköz and Hasan Tosun., *The use of methylene blue test for predicting swell parameters of natural clay soils.* Scientific Research and Essays Vol. 6 (8), pp. 1780-1792. 2011.

Murray, H. H., *Applied Clay Mineralogy.* Netherlands: Elsevier. 2007.

Murray, H. H., "*Applied Clay Mineralogy: Occurrences, Processing And Application Of Kaolins, Bentonites, Palygorskite-Sepiolite, And Common Clays,"* Elsevier. 2007.

Murthy, V. N. S., Geotech*nical Engineering: Principles and Practices of Soil Mechanics and Foundation Engineering,* CRC Press, New York, 2002.

N. V. Nayak and R. W. Chrıstensen., *Swellıng Characterıstics Of Compacted Expansive Soils. Clays and Clay Minerals*, 1971, Vol. 19, pp. 251-261. Pergamon Press. Printed in Great Britain. 1970.

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

Nevins MJ, Weintritt DJ., *Determination of cation exchange capacity by methylene blue adsorption. Ceramic Bull.,* 46 (6): 587-592. 1967.

O'neıll, M. W., and Poormoayed, N., *"Methodology for Foundations on Expansive Clays,"* Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 106, No. GT12, pp. 1345 –1367. 1980.

Oweis, I. S., & Khera, R. P., *Geotechnology of Waste Management,* 2nd Edition, PWS Publishing, Boston, 1998.

Pantet, A., Parriaux, A., Thelin, P., *New method for in situ characterization of loose material for landslide mapping purpose.* Engineering Geology 94, 166–179. 2007.

Pohl, Walter L., *Economic geology: principles and practice: metals, minerals, coal and hydrocarbons–introduction to formation and sustainable exploitation of mineral deposits.* Chichester, West Sussex: Wiley-Blackwell. 2011.

Popescu, M. E., *A Comparison between the Behavior of Swelling and of Collapsing Soils,* Engineering Geology, Vol. 23, pp. 145-163, 1986.

Pusch, R., and Yong, R. N. *"Microstructure of Smectite Clays and Engineering Performance,"* Taylor & Francis. (2006).

Scott, R. *Principles of Soil Mechanics,* Addison-Wesley Publishing Company, Inc. (1963).

Seed, H, B., Woodward, R., J., & Lungren, R., *Prediction of Swelling Potential for Compacted Clays,* Journal of the Soil Mechanics and Foundations Division, Proc. Of the ASCE, pp. 53-87, 1962.

Shanker, N. B., Rao, A. S., and Swamy, A. S. R. *"Swelling Behaviour of Undisturbed and Remoulded Samples of Black Cotton Clay,"* Indian Geotechnical Journal, vol. 12, 152-159. (1982).

Shuai, F, *Simulation of Swelling Pressure Measurements on Expansive Soils,* Doctoral Thesis, University of Saskatchewan. 1996.

Sridharana. Prakashk. *Mechanism controlling the shrinkage limit of soils.* Geotechnical Testing Journal, ASTM, 21, No. 3, 240-250. 1998.

Sridharana., Raos. M., Murthyn. S. *A rapid method to identify clay type in soils by the free-swell technique.* Geotechnical Testing Journal, ASTM, 9, No. 4, 193-203. 1986

Stavridakis I.E., *Stabilization of problematic soils using cement and lime, Expansive Soils Recent Advances in Characterization and Treatment.* Taylor & Francis Group. London. UK. pp. 384-397. 2006.

Taylor, R. K., Cation *exchange in clays and mudrocks by methylene blue.* J. Chem. Technol. Biotechnol., 35A:195-207. 1985.

Thomas P.J, Baker J.C, Zelazny L.W, Hatch D.R. (2000). *Relationship of map unit variability to shrink-swell indicators.* Soil Sci. Soc. Am. J., 64: 262-268. 1985.

Taylor, R, K., *Cation exchange in clays and mudrocks by methylene blue*. J. Chem. Technol. *Biotechnol.,* 35A:195-207.

Tripathy, S., & Subba, Rao., *Cyclic Swell - Shrink Behavior of a Compacted Expansive Soil*, Geotech Geol Eng 27, pp. 89-103, 2009.

Türköz, M., *Sıkıştırılmış Şişen Killerin Mikroyapısal Değişiminde Şişme-Büzülme Çevrimin Etkisi*, Eskişehir Osmangazi Üniversitesi Mühendislik Mimarlık Fakültesi Dergisi Cilt: XXII, Sayı 1, 2009.

Van Olphen. H., *An Introduction to Clay Colloidal Chemistry*, 2nd. Edition, Wiley Inter Science. 1977.

Van der Merwe, D., H., *The prediction of heave from plasticity index and the percentage clay fraction of the soils.* The Civil Engineer, South African Institution of *Civil Engineers*, 6, 103-107. 1984.

Verhoef, P.N.W., *The methylene blue adsorption test applied to geomaterials, Memoirs of the centre of Engineering Geology in the Netherlands.* Deltf Uviversity of Technology, Geomat. 101: 2- 70. 1992.

Villar, M. V., and Lloret, A., *"Influence of dry density and water content on the swelling of a compacted bentonite," Applied Clay Science*, 39, 38-49. 2008.

White, W. A., *Atterberg plastic limits of clay minerals:* Am. *Mineral.,* vol. 34, pp. 508-512. 1949.

Yevnin, A., and Zaslavsky, D., "*Some factors affecting compacted clay swelling,"* Canadian Geotechnical Journal, Vol. 7, Issue 1, 79-89. 1970.

Yükselen Y, Kaya A., *Suitability of the methylene blue test for surface area, cation exchange capacity and swell potential determination of clayey soils.* Eng. Geol., 102: 38-45. 2008.

APPENDIX A

ANKARA POTABLE WATER AND ANKARA CLAY PROPERTIES

Table A.1 Ankara potable tap water chemical characteristics

Parameter	**Ankara potable tap water quality**
PH	7.4
Aluminum (μg/l)	26.2
Ammonium (mg/l)	<0.06
Magnesium (mg/l)	9.6
Copper (mg/l)	<0.003
Calcium (mg/l)	30.8
Potassium (mg/l)	3.2
Sodium (mg/l)	18.05
Iron (μg/l)	<5

Table A.2 Summary of semi-quantitative whole-soil mineralogy of the samples from Ankara clay with carbonate concretions based on XRD

Clay and Nonclay Minerals (%)				**Clay Minerals (%)**		
Calcite	Quartz	Feldspar	Clay	Smectite	Illite	Kaolinite
4.0-33.0	8.3-18.0	1.7-6.7	52.6-84.0	38.3-60.2	4.7-27.8	7.7-16.7
(14.6)	(11.8)	(4.3)	(69.9)	(49.3)	(10.1)	(11.7)

Proposed empirical predictive models to predict the swelling pressure based and swell potential based MFSI, γ_d and W_i are presented as follow (Eqn A.1, Eqn A.2)

$$\text{Log(P)} = a_1 + a_2\text{MFSI} + a_3\gamma_d + a_4 W_i \quad \text{……..Equation A.1 (Model A.1)}$$

$$S_p = b_1 + b_2\text{MFSI} + b_3\gamma_d + b_4 W_i \quad \text{…………..Equation A.2 (Model A.2)}$$

Table A.3 Intercepts, coefficients and regression statistics of correlation equation

Equations	Intercept	Coefficients			Regression Statistics	
Equation A.1	a_1=14.173	a_2=0.004	a_3= -7.462	a_4=0.068	R^2=0.97	S=0.07
Equation A.2	b_1=1830.101	b_2=0.162	b_3=-1287.35	b_4=-0.296	R^2=0.99	S=10.71

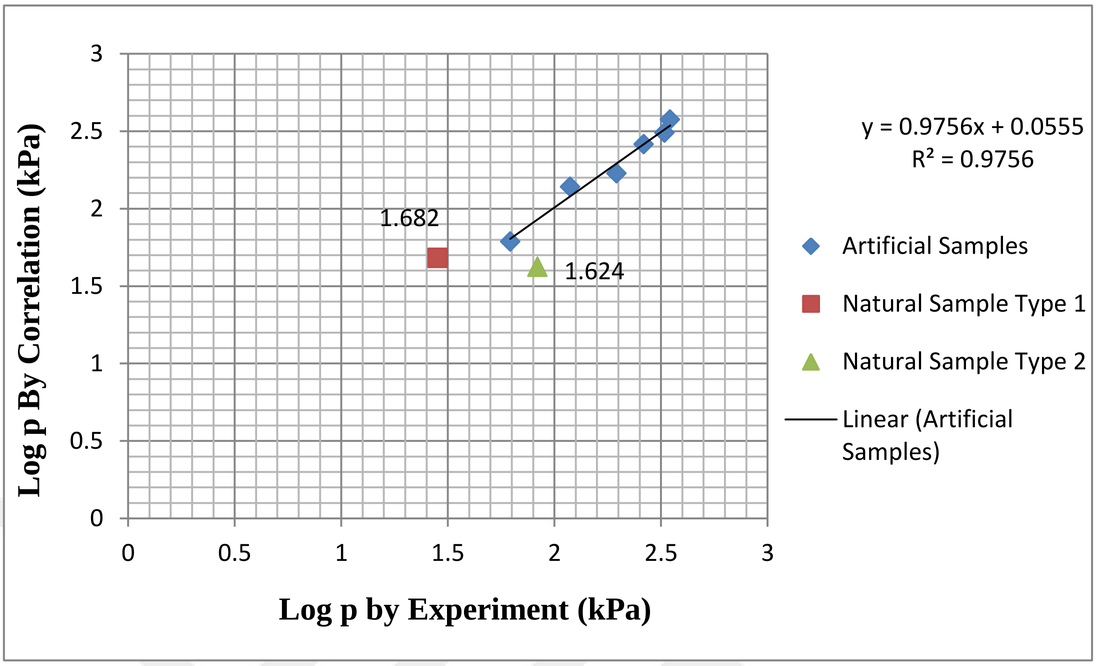

Figure A.1 Comparison between experimental and predicted values of swelling pressure from model A.1

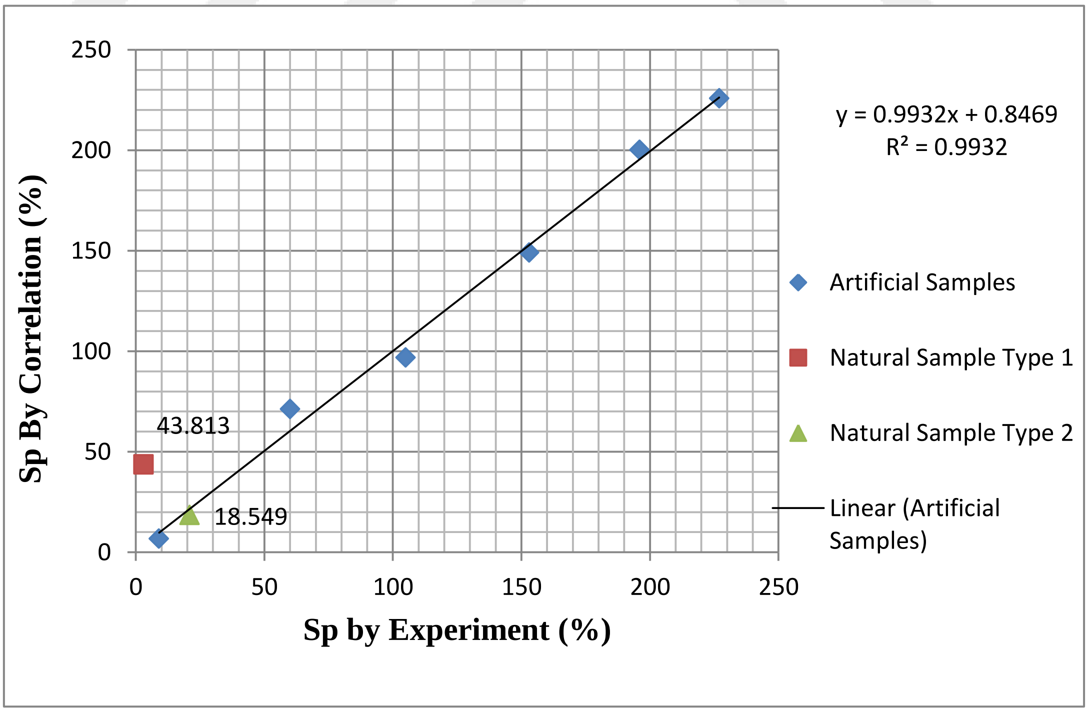

Figure A.2 Comparison between experimental and predicted values of swell potential from model A.2

APPENDIX B

EXPERIMENTAL TEST RESULTS

This section consists of the correlation between the values obtained from experimental investigation and analytic evaluation.

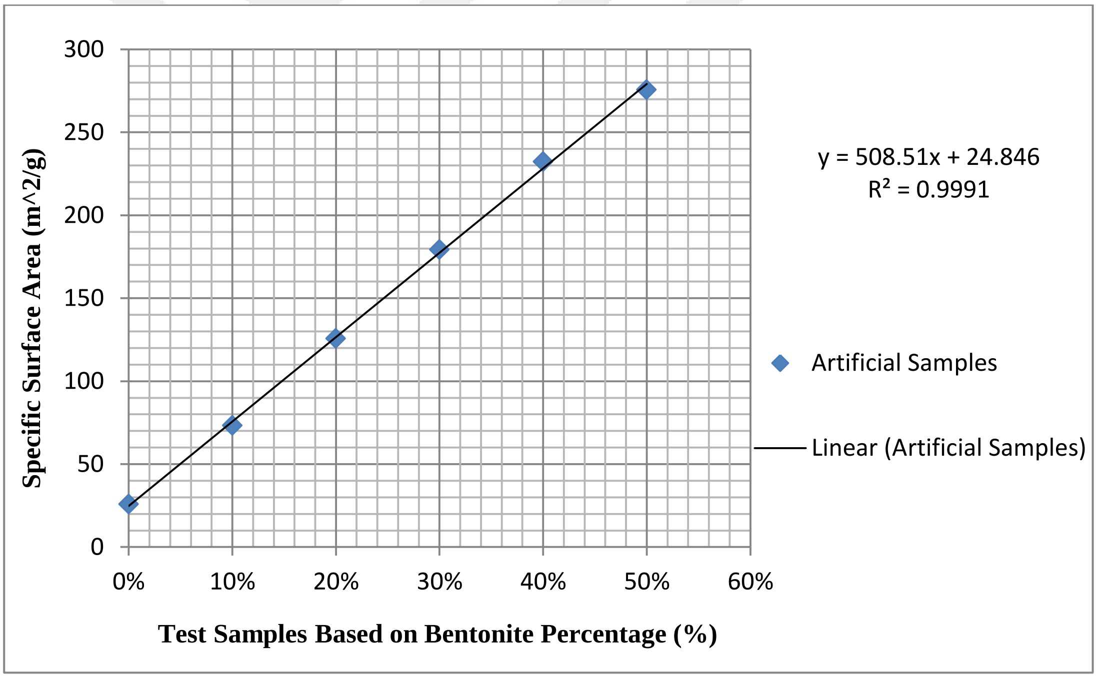

Figure B.1 S.S.A vs Test Samples

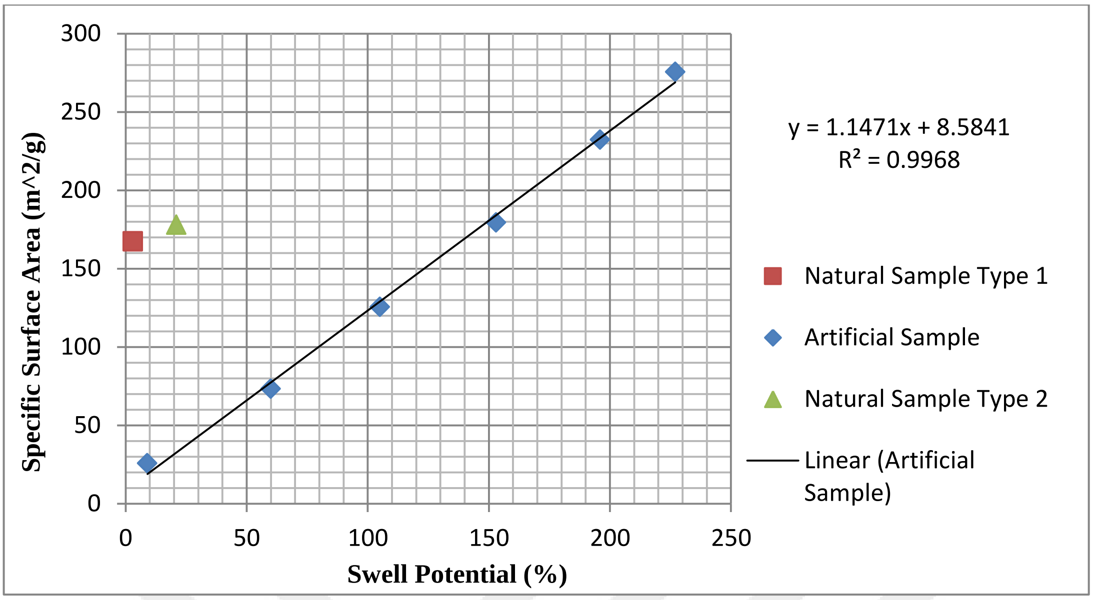

Figure B.2 S.S.A vs Swell Potential

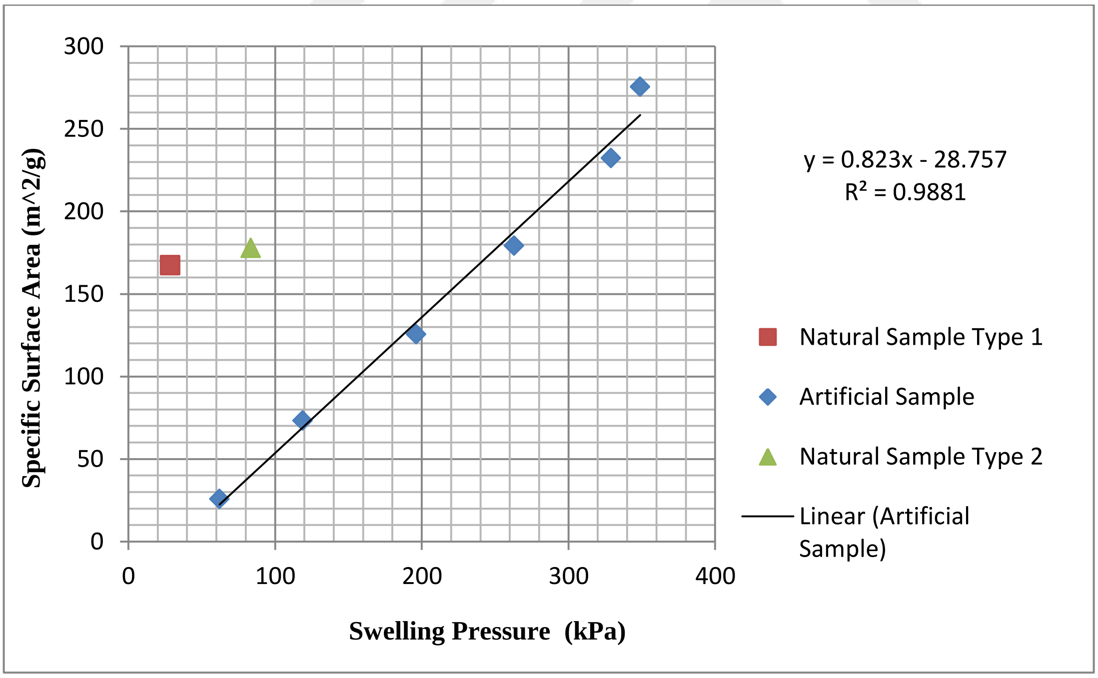

Figure B.3 S.S.A vs Swelling Pressure

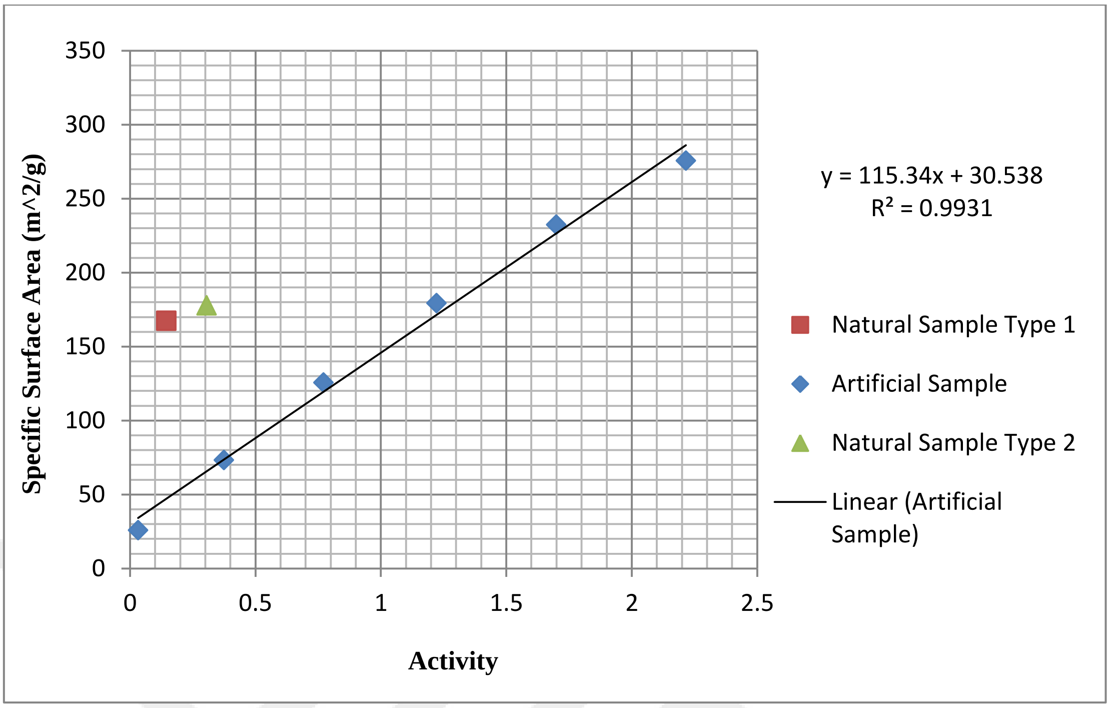

Figure B.4 S.S.A vs Activity

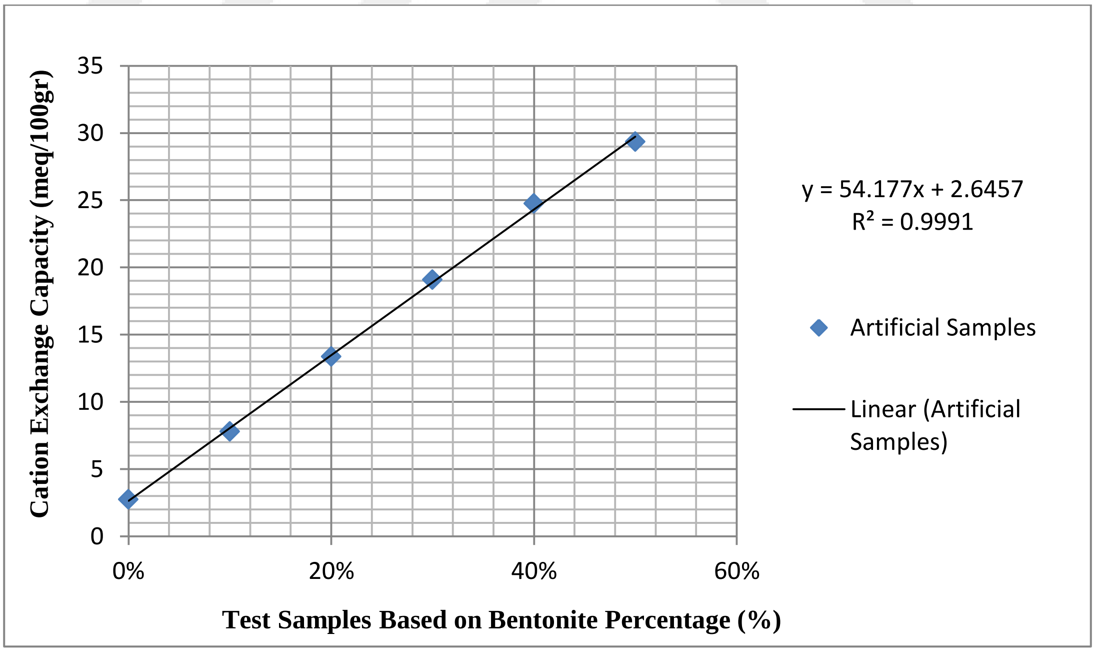

Figure B.5 C.E.C vs Test Samples

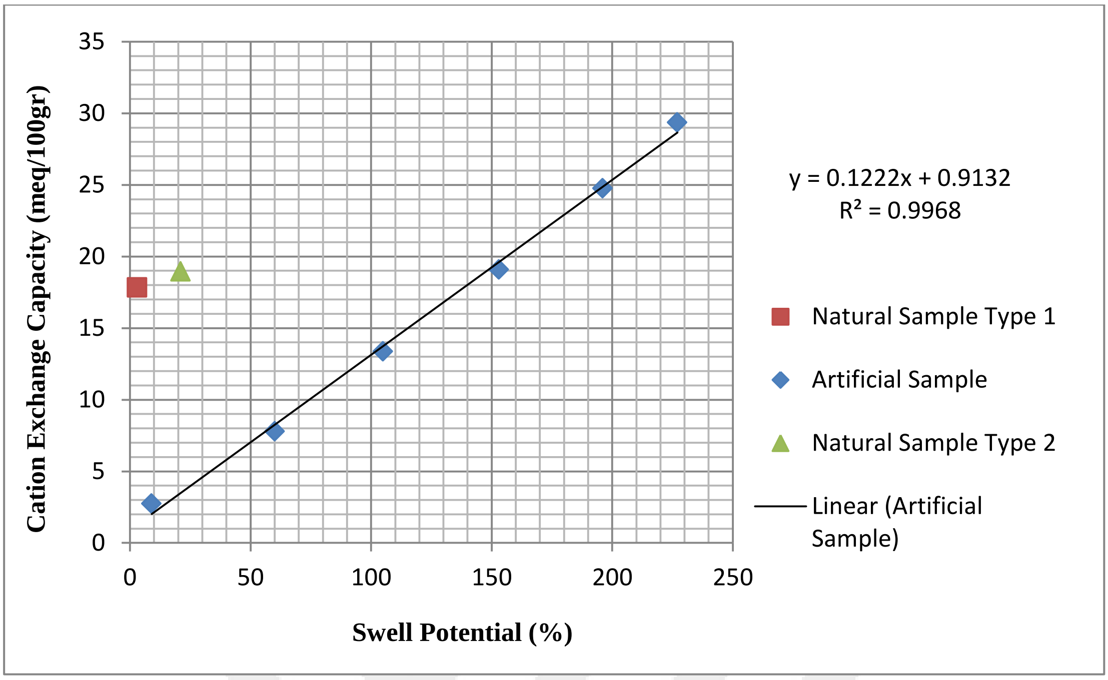

Figure B.6 C.E.C vs Swell Potential

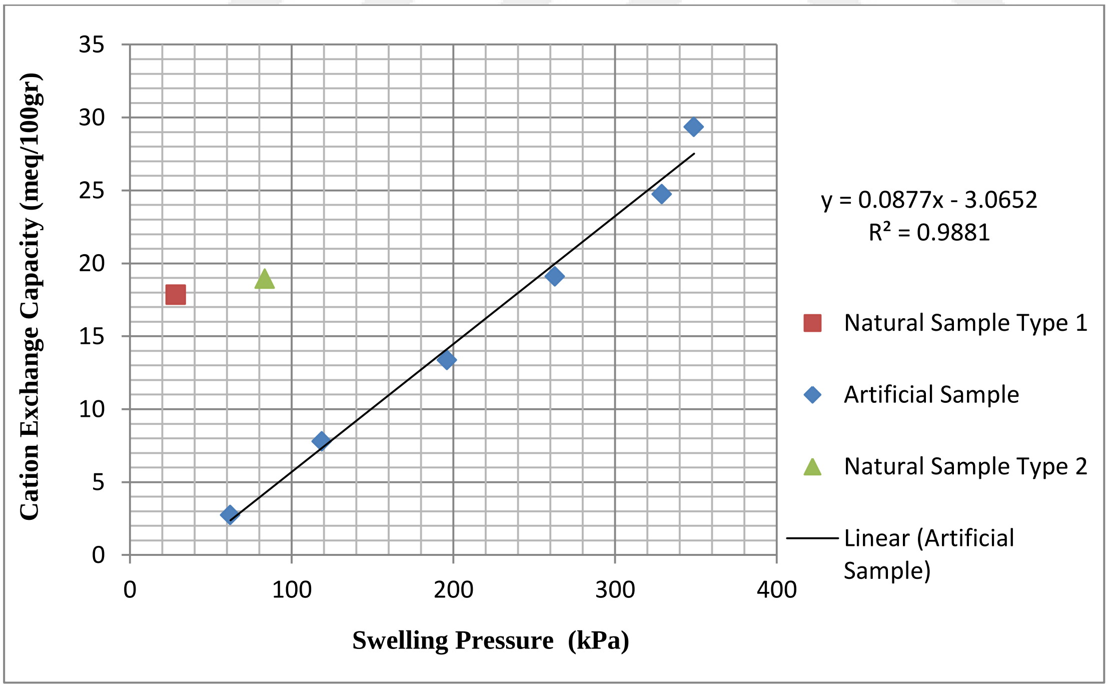

Figure B.7 C.E.C vs Swelling Pressure

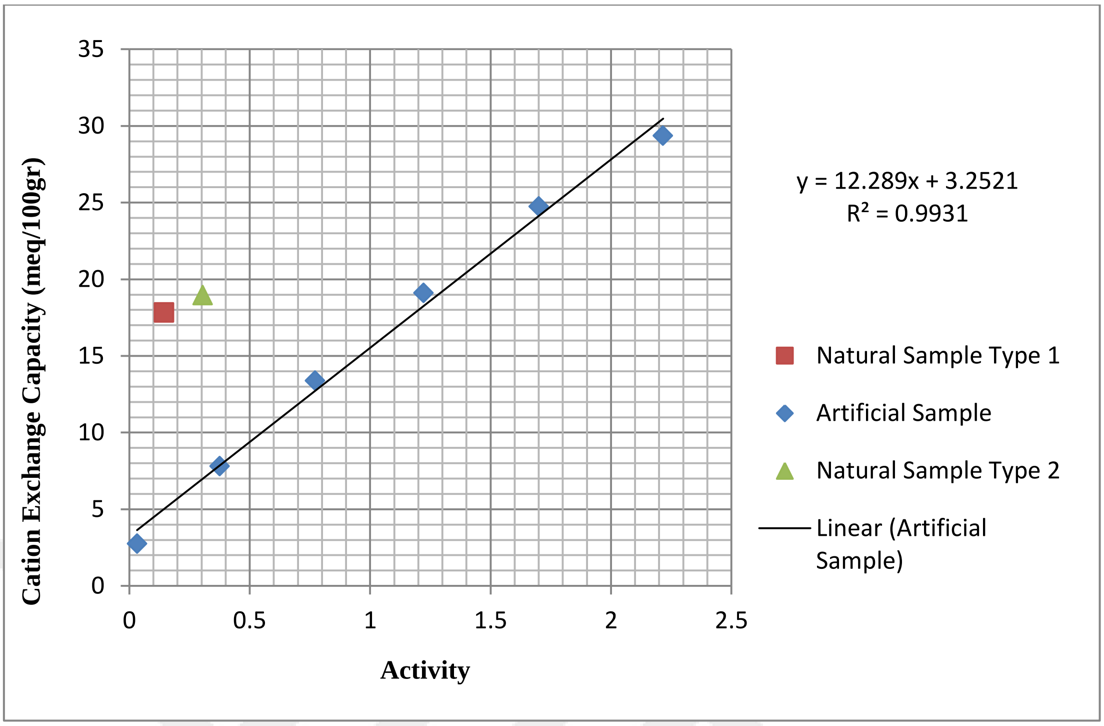

Figure B.8 C.E.C vs Activity